Course CT3410

Design of Open-Channels and Hydraulic Structures

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N.B.
Deze studiehandleiding dekt de tentamen-stof niet geheel. Tevens kunnen sommige onderwerpen als 'leesstof' worden beschouwd.

Aangeraden wordt om het college te volgen, waar de relevante onderwerpen worden behandeld. Ook het 'operationeel maken' van bestaande kennis staat op het college centraal.
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1. WATER CONTROL SYSTEMS

1.1. Introduction

1.1.1. Importance of Lowlands

Lowlands. Some 1200 million ha of the 1500 million ha of croplands on earth are flat, and are 'lowlands'. A major part of these lowlands consists of deltas and low-lying coastal areas and belongs to the world's most densely populated areas. About one third of the world's population lives less than 100 km from the coastline. Many deltas have average population densities from 200 to more than 500 persons per km². For instance, the Netherlands has about 15 million inhabitants on only 3 million ha (500 per km²), with most of the population in the low-lying western part (900 inhabitants per km²).

Deltas possess a high potential for agricultural and industrial development, as these areas have natural advantages with respect to soil, topography, water and location. Extensive civil works have to be implemented to make full use of these potentials.

It is estimated that the food production of some 900 million hectares of these flat areas are hampered by frequent floodings, high groundwater levels, and regular water shortages. Here, the crop production can be improved considerably by flood control, drainage and irrigation.

Multi-discipline aspects. The development of lowlands is often more difficult and costly than the development of 'upland floodplains'.

In lowlands, not only embankments ("dijken") are needed to protect the areas against periodic floods by the sea and/or the rivers, but also the quality of the water creates problems as brackish ("brak") water may affect the crops. The soils in lowland areas may suffer from acidity ("verzuring") after reclamation or may consist of thick layers of peat ("veen") which offer a poor environment for agriculture. These soils require normally a strict water management regime.

The Netherlands probably has the most developed coastal lowland area in the world. It has complete water management ("waterbeheer") and flood control ("hoogwaterbescherming").
1.1.2. Lowland Development

Lowland. 'Lowland' ("laagland") is defined as: "land affected by fluctuating water levels and where human activities are already existing or are being proposed" (Miura 1994).

It means that lowland consists of tidal lands, deltas, inland marshes and flood plains. Moreover, shallow seas and lakes may belong to lowlands when impoldering is considered.

Lowland development. 'Lowland development' is a term which covers a wide field of technical activities. It is used for the development and improvement of land for agricultural, residential, industrial and other purposes. Lowland development requires an integrated approach between hydraulic engineering, hydrology, soil science, agricultural engineering, economy, sociology, etc.

Classification. Lowland development can be classified into three methods, see figure 1.1:
- tidal land reclamation ("landaanwinning"), i.e. reclamation of tidal forelands and coastal marshes;
- impoldering ("inpoldering"), i.e. reclamation of shallow seas and lakes;
- flood control ("hoogwaterbeheer"), i.e. the protection of lowlands against river floodings.

Often, the lowlands have become a set of polders where all elements of the hydrological cycles are manipulated, except of course the rainfall. In these polders, the water levels are controlled artificially by 'water control' ("waterbeheer"): 
- drainage ("ontwatering"), i.e. to evacuate the excess water from the soil;
- irrigation ("bevloeiing"), i.e. to supply water to the agriculture.

But water control is also widely applied in the upland floodplains.

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**Figure 1.1. Lowland Development and Water Control Systems.**
1.1.3. Terms in Lowland Development

Reclamation. The term 'reclamation' ("droogleging, ontginning") is also used in the concept of lowland development. Reclamation is defined by the International Commission on Irrigation and Drainage (ICID 1996) as: "the act or process of reclaiming swampy, marshy, deteriorated, desert and virgin lands, and making them suitable for cultivation or habitation; also conversion of foreshore ("buitendijks land") into properly drained land for any purpose, either by enclosure ("bedijking") and drainage, or by deposition of material ("ophogen") thereon".

Tidal land reclamation. A silting-up of the tidal forelands ("buitendijkse aanslibbing") is taking place along many sea coasts in deltaic areas because of deposition of sediments, see figure 1.2.

Tidal land reclamation ("landaanwinning", "indijken") can be carried out when the foreshore ("buitendijks land: kwelders, schorren, gorzen") has reached a certain elevation. Typical engineering problems are:
- suitable elevation of tidal foreland for reclamation;
- silting-up of the outfall channels;
- the defence of the land against storms;
- the layout of these progressive impoldered areas.

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**Figure 1.2.** Tidal lands with creeks.
Impoldering. Reclamation of shallow seas and lakes through **impoldering** is a very special and expensive type of lowland development, and has been implemented so far in a few countries with serious land problems (e.g. the Netherlands, Japan). Typical hydraulic engineering problems are:
- the defence of the low-lying land against inundations;
- seepage ("kwel") of brackish ("brak") water;
- pump-lift drainage ("bemaling").

**Flood control.** Lowlands are generally located in deltaic areas which have been formed morphologically by river deposits ("rivierafsettingen") during flooding of the delta. As deltas belong to the most productive and the most densely populated parts of the earth, flood control is needed in many cases.

**Flood control** ("hoogwaterbescherming") is needed to protect the areas against **external** floods, i.e. water from outside the area. Construction of embankments (dikes, levees) is often a part of the required measures. Typical hydraulic engineering problems related to flood control, are:
- construction of embankments ("dijken") makes the flood levels to become higher,
- the internal drainage to the river is obstructed,
- the process of gradual land raising ("colmatage") by periodic sedimentation is halted.

Flood control methods include the construction of embankments (dikes, levees), straightening of the river channel, widening of the river channel, flood diversion ("afleiding") into another channel, flood retention ("berging") in a reservoir.

**Polders.** Both tidal land reclamation and impoldering create new land that can be called a 'polder'. But also the embanking of areas as part of flood control works can create a 'river polder'. A polder is defined as (Segeren 1982): "a polder is a level area which has originally been subject, permanently or seasonally, to a high water level (groundwater or surface water) and is separated from the surrounding hydrological regime, to be able to control the water levels in the polder (groundwater and surface water)".

Thus, a polder shows the following characteristics:
- a polder does not receive any foreign water from a water course, but there is only water inflow by rain and seepage, or by irrigation intake;
- a polder has an outlet structure ( sluice or pump) that controls the outflow;
- the groundwater levels and the surface water levels in the polder are independent from the water levels in the adjacent land. These water levels are artificially maintained in order to optimize the objectives of the polders, on basis of land elevation, soil properties, climate, type of land use, etc.

**Drainage.** Swamps and other water-logged areas may have a poor **internal** drainage system to evacuate rainfall. They require a good land drainage system of sub-surface and/or surface drainage systems. Typical hydraulic engineering problems related to land drainage, are:
- subsidence ("klink") of the drained land,
- irrigation of the drained areas might become necessary during part of the year.

**Drainage systems.** Drainage systems consist of two components, see figure 1.3:
- **field drainage systems** ("ontwateringssysteem"), also called 'land drainage systems'. Field or land drainage is the collection of the excess water from the land by means of a network of field drains. Two principal types of field drainage may be distinguished: (i) a surface drainage systems of ditches ("greppels"), (ii) a sub-surface drainage system of pipe drains ("drainage buizen").
main drainage systems ("afwateringssysteem"). The function of the main drainage system is to collect the water from the field drainage system, to transport the water to the outlet, and to evacuate the water from the area.

![Diagram of drainage system](image)

**Figure 1.3.** Main drainage system and field drainage system.

Irrigation. Many lowlands, but also many upland plains, have a rainfall pattern that does not supply the agriculture with timely and sufficient water. Irrigation of the agricultural land is good solution when rivers are still abundant with water.

The term 'irrigation' is often used in two meanings:

- the actual watering of the crop, by means of basin irrigation ("terras bevloeiing"), furrow irrigation ("voren bevloeiing"), sprinkler irrigation ("beregening"), etc.
- the combination of the artificial supply of water for the use of agriculture and the systematic distribution over the area, with the superfluous water to natural drains, after that as much as possible, benefit out of the water is extracted.

The second meaning of irrigation is considered here, with emphasis on the hydraulic aspects of the water control system, see figure 1.4. Typical hydraulic engineering problems related to irrigation are:

- discharge diversion ("aanzoerverdeling") throughout the whole irrigation scheme;
- water level control ("peilbeheer") at the diverting structures ("verdeelwerken")
- matching the total water need of the scheme ("watervraag") with the available supply in the river ("waterbeschikbaarheid").

![Irrigation scheme](image)

**Figure 1.4.** An irrigation scheme with a water control system.
1.1.4. Literature

Publications on tidal land reclamation. There are no handbooks at professional level on the general aspects of tidal land reclamation. However, two publications discuss the technical and historical aspects of deltaic areas:

Publications on Flood Control. There are also no handbooks at professional level on flood control. However, a wide range of books on hydrology, hydraulics, river engineering, irrigation and dams exist.

Publications on Polders. There is only a limited number of handbooks, textbooks and manuals available on impoldering:

Publications on Drainage. There are several publications on drainage, most of them concentrating on the field drainage aspects. The most comprehensive publication is:

Publications on Irrigation. There is only a limited amount of handbooks on the civil engineering aspects of irrigation:
1.2. Flood Control Systems

1.2.1. Floods in Deltas

Prevention of inundations. Lowlands of deltas have been formed by river deposits ("rivier-afzettingen") during 'floodings' or 'inundations' ("overstromingen"). However, these inundations have to be prevented when the deltas are becoming densely populated.

Inundations. The source of 'inundations' ("overstromingen") can be either:
- **poor internal drainage** ("slechte interne drainage"). The inundations occurs because the rainfall cannot be discharged properly;
- **external flooding** ("overstroming door buitenwater"). Inundations occur because water enters from outside the area. External flooding can enter from two sources:
  - *sea floods* from the sea,
  - *river floods* from the rivers.

Sea floods. The 'sea floods', or 'storm surges' ("stormvloeden"), are normally generated by landward winds. The maximum flood levels are dependent on the amplitude of the tide, but also on other factors like the direction and duration of the wind.

For instance, the sea tide in the North Sea differs in amplitude along the coast, with the maximum average tide in the Southern part of 2.00 m+MSL and 2.00 m-MSL. The maximum sea levels during storms are much higher and the sea dikes are presently designed for sea levels of 5.00 m+MSL, i.e. a return period ("herhalingstijd") of 1:10,000 years. Moreover, a *freeboard* ("waking") of 10.00 m is added for wave run-up. Flood protection against sea floods is not discussed here.

River floods. The 'river floods' depend on the physical characteristics of the river basin (size, slopes, soils, etc.) and the storm rainfall ("zware regenval"). Whether these river floods will inundate the area, depends on the capacity of the river channel. Furthermore, inundations may also occur during a coincidence of sea floods and river floods.

![Graphs](https://via.placeholder.com/150)

**Figure 1.5.** Flashy floods and gentle floods.
Type of river floods. The river flood in a delta can be either (i) flashy flood, or (ii) a gentle flood, see figure 1.5.

**Flashy floods** (Indonesian: "bandjirs") rise quickly, have sharp peaks and have a rapid recession ("uitputting"). The water may rise as much as a few meters within 24 hours, or even within a few hours. Flashy floods occur in small river basins with surface runoff ("oppervlakte afstroming"), especially in basins with steep slopes, impervious soils and heavy (tropical) rains of short durations.

**Gentle floods** show slow rises (0.05 to 0.10 m/day) and flat flood peaks. Their duration may extend over several months. Gentle floods occur in large river basins and with prolonged rainfalls. Examples are the large rivers during the wet monsoon in South Asia (Ganges) and in South-East Asia (Irrawaddy, Chao Phya, Mekong).

**Flood damage.** It is often difficult to assess the benefits ("baten") of flood control projects. In fact, the benefits are the 'non-occurrence of damage' ("niet-optreden van schade").

Damage by floods in general comprises in the first place, **direct damage** which can be readily evaluated, i.e. loss of crop, damage to infrastructure and houses, etc. These costs are often quite low. For instance, damage to the paddy means that it needs a second planting, but land preparation is not required again.

Next comes **indirect damage**, like loss of income, interruption of communications, caring of refugees, etc. Indirect damages are not so easy to estimate in costs.

Thirdly, there is **'intangible' damage** ("negatieve effecten"), like loss of human lives, public morale, and above all the limitations to land potentials.

**Return period.** Hence, floods are often responsible for the unfavourable economic conditions in an area. Complete prevention of flooding is physically impossible, and flood protection can only be attained to the degree that is technically and economically feasible.

A commonly used criterion is that of the minimum total of annual costs and damage. In practice, it means that flood control is obtained against floods of a certain average 'return period' ("gemiddelde herhalingsstijd"). For instance, typical agricultural projects are protected against the 1 : 20 years' flood, urban areas in developing countries ("ontwikkelingslanden") are sometimes protected against 1 : 50 years' floods, urban areas in the Netherlands are protected against river floods of return periods of 1 : 1800 years, etc., etc.

**Flood control.** **Flood control** is "the provision of a specific amount of protection from floods" (ICID 1967).

More practical, flood control is the reduction of floods to a certain extent by measures in the upper portion of a river basin and by measures on location (Volker 1991).

Flood control ("hoogwaterbescherming") is needed to protect the areas against external floodings,

**Methods of flood control.** The following methods of flood control can be distinguished:

- **catchment management**, by land management and by construction of a flood control reservoirs ("dalafsluiting") in the catchment area;
- **channel improvements** ("bed verbetering"), by enlarging the discharge capacity of the river;
- **flood embankments** ("dijken") along the river, also called 'levees' or 'dikes';
- **flood diversion** ("afleiding") through another channel, i.e. a 'flood diversion channel'.

1.2.2. Catchment Management

Land management. Floods can be reduced by agricultural measures in the catchment area, such as afforestation ("her-bebossing"), terracing ("terras aanleg"), contour ploughing ("contour ploegen").

In general, these measures have some beneficial effect by (i) reducing the flood peaks, and (ii) retaining a part of the sediments ("erosiecontrole"). But often, the effect is quite small for major floods because only a part of the catchment area can be treated.

Flood control reservoirs. The construction of a reservoir ("dalafsluiting") for flood control is normally not the appropriate solution for flood control.

Suitable dam sites are rare, and often there is still a considerable inflow into the river below the dam.

Furthermore, reservoirs with flood control as a single purpose are seldom economically feasible. Multi-purpose reservoirs may have conflicting operation requirements: irrigation and hydropower require a full reservoir, but flood control requires an empty reservoir.

1.2.3. Channel Improvements

Channel improvements. Channel improvements for flood control consist of, see figure 1.6:
- enlarging the cross-sectional area ("dwarsoorsnede") by deepening and/or widening the channel bed;
- shortening of the alignment ("tracé"), by making cut-offs and by straightening of the channel.

![Figure 1.6](image)

ALTERNATIVE I
Enlarging Cross-section

ALTERNATIVE II
Enlarging Gradient of Channel

Enlarging the cross-sectional area. The cross-section of the channel can be increased by deepening of the channel, and/or widening of the channel bed. These measures are usually limited in duration.

It should be recalled that the rivers created these lowlands through sedimentation. Some equilibrium is reached between the natural cross-section and the frequency of floodings. An enlarged cross sectional area will change the sediment-transporting capacity, and often new sedimentation will start. It means that the life-time of the enlarged cross-sectional area is often short.
Shortening the alignment. Shortening of the alignment by making cut-offs through meanders will steepen of the gradient ("verhang") of the channel. A steeper gradient means that the flow-capacity of the channel is increased. Moreover, the sediment-transporting capacity is increased as well. A check should be made, whether the channel bed remains stable against erosion.

The shortening of the alignment appears to be quite effective in comparison with other flood control measures. However, the possibilities for straightening depends on the present alignment and the rate of meandering. Generally, only a small decrease of river length can be obtained, and thus the gradient $s$ is increased by a small ratio. Furthermore, the flow-capacity of the channel is increased by the root ("vierkantswortel") of the gradient, so by $\sqrt{s}$.

1.2.4. Flood Embankments

Flood embankments. The application of embankments ("dijken") go far back in history, and they are the most simple way of flood control. Embankments are also called 'levees' or 'dikes'. The design of embankments concerns two basic questions, see figure 1.7:

- what is the height ("hoogte") of the embankment?
- what is the mutual distance ("onderlinge afstand") between the embankment?

Height of embankments. The height of the embankment is determined by: (i) the return period of the flood, (ii) the design flood level, (iii) and the freeboard above the flood level.

Discharge through the flood plains. The discharge through the flood plains ("uiterswaarden") will often be small in comparison with the discharge through the channel. The flood plains have often a high roughness, i.e. a low Strickler coefficient $k$, because of vegetation. It means that the flood level is often hardly be reduced by making a wide flood plain.

The construction of embankment at some distance from the channel will increase storage. This effect is also limited to flashy floods. The effect is negligible for gentle floods.

![Alternative I: Constructions of Embankments](image1)

![Alternative II: Construction of Flood Plains](image2)

**Figure 1.7.** Flood control by construction of embankments.

Scouring of embankments. Wide flood plains provide a protection against channel erosion to the embankments. Meandering rivers do not have stable banks and scouring at the outer bends can be expected. The constructing of embankments close to the channel will sooner or later require expensive river training works to protect these embankments.

Geologically seen, meandering is a very fast process and the whole alignment is changed e.g. every 300 years. The speed might be much lower from a civil engineering point of view, when the river bend advances locally e.g. some 20 m every 10 years.
Effects. The construction of flood embankments has often extensive effects on the river, and its population living on the natural levees. All these effects should be considered when designing flood embankments:

- **Higher flood water levels.** The construction of embankments along a river will lead to higher flood water levels. Firstly, the cross sectional area is reduced, so that the water level will increase for a given discharge. Secondly, the function of the embankment is that no spillage will occur any more, so the discharge will increase and at the same time the flood water level.

- **No further delta formation.** The natural process of delta formation will be halted by the construction of flood embankments. Experiments in Burma, Cambodia and Rumania with the operation of sluices for the controlled admission of silt-laden waters have not been very successful.

- **Sediment deposition on flood plains.** When flood plains are designed, all the sediment will be deposited at the flood plains. When no flood plains are constructed, the river will transport the sediment to the sea. So, the flood control engineer has to make an important decision for the future generations: a sediment deposition of e.g. 0.01 m per year, will raise the flood plains above the surrounding land with 1.00 m per century.

- **Impact on population.** The population likes to live on the natural levees along the river, as the soils and drainage conditions are better here. Construction of embankments, and certainly the application of flood plains will create problems for the population.

- **Drainage of surrounding land.** Construction of embankments will block the drainage outfalls from the backswamps to the river. Sluices have to be constructed, but they will often not solve the problems as they have to be closed (river in flood) when the drainage is required (after rainfall). It is often better to construct a separate 'internal' drainage that debouches ("stroom uit, loost") directly into the sea.

### 1.2.5. Flood Diversion

**Flood Diversion.** 'Flood diversion' ("afleiding van hoogwaters") is a widely used method in a delta. The floods are diverted from one channel into another with a higher capacity. A prediction must be made on the morphological effect of this diversion. A channel with a lower discharge has an exponential reduction of the sediment transporting capacity. So, the splitting of a river into two channels means that the cumulative sediment transport capacity is reduced. Flood diversion is also a matter of hydraulic engineering, concentrating on the design of the flood diversion channel ("afleidingskanaal") and the design of the flood diversion structure ("verdeelwerk"), see figure 1.8.

![Figure 1.8. Flood diversion structure for flood control.](image)
1.2.6. Hydrology of Floods

**Background.** Design of drainage systems and flood control works rely on an accurate estimate of the flood discharges. Sometimes, the assessment of the peak discharge is only sufficient, e.g. for the determination of the capacity of the channel. Often, the whole hydrograph with its volume is required, in order to perform studies on flood routing, damage of spillage, etc.

Many methods exit to estimate the size of the design flood. Examples are the synthetic unit hydrograph method, the curve number method. Other methods are regionally developed and are based on experience. These methods are not discussed here, as they belong to hydrology.

**Introduction.** Hydrological data can be processed in several different ways, see figure 1.9:  
- **complete series,** when all data are used, e.g. all peak floods during the years,  
- **annual series,** when only the annual extremes are used, e.g. the annual maximum peak flood,  
- **partial series,** when all data above a certain base are used, e.g. all peak floods above 250 m$^3$/s.

The selection is based on the purpose of the analysis, the availability of data, and on personal preference.

![Figure 1.9. Complete, partial (•) and annual (•) series.](image)

**Complete series.** Hydrological data can involve an enormous amount of data, see also figure 1.9. For example, discharges that have observed hourly during the last 10 years, means some 90,000 data. These data are not independent, since the flow is only gradually changing.

Processing all these data, i.e. the 'complete series', is normally not practical or necessary when studying the extreme values. Monthly and annually rainfall data require normally the complete series of data. Often 'normal probability paper' ("normaal waarschijnlijkheidspapier") is used in the statistical analysis.
Annual series. 'Annual series' ("jaar serie") are normally used for the determination of the extremes, i.e. the maximum and/or minimum flow with a certain return period. Now, only the maximum or minimum event for each year is extracted from the complete series.

Annual series are used e.g. for determining peak floods, daily and hourly peak rainfall. Often 'Gumbel probability paper' ("Gumbel waarschijnlijkheidspapier") is applied in the statistical analysis.

Partial series. A year may contain two or more high peak floods which are higher than other yearly maxima, and which are independent events, see also figure 1.9. So, an analysis based on annual series ignores the second- and lower-order events of each year which may be even greater than the annual floods of other years.

'Partial series' ("partiële serie") will overcome this limitation, by evaluating all extremes above some arbitrary base value, e.g. 250 m³/s. The partial series are plotted on single or double logarithmic paper as no statistical paper is available for these series.

In principle, the use of partial series is more correct than the use of annual series. However, the difference is very small when the extreme values are calculated from a data series of more than ±10 years. Partial series are needed:
- when only a few years of data are available, e.g. < 5 years.
- for the determination of events of a small return period, e.g. for return periods \( T < 2 \) years. Now, it is practical to use not all data of e.g. 20 years, but to select at random a shorter period, e.g. 3 - 5 years.

Short return periods from annual series. Events with short return periods, e.g. the twice-per-year flood: \( T = 0.5 \) year, cannot be read from 'Gumbel probability paper' or from 'normal probability paper'. Gumbel probability paper shows return periods of \( T = 1.01 \) years and higher, normal probability paper shows return periods of \( T = 2 \) years and higher.

However, it is also possible to convert the return period of the annual series to the 'true' return period as follows from the partial series. Therefore, the following formula can be used (Chow 1964):

\[
T_{\text{annual}} = \frac{1}{\ln(1 - e^{-1/T_{\text{partial}}})}
\]

where:
- \( T_{\text{annual}} \) is the return period, according to the annual analysis,
- \( T_{\text{partial}} \) is the 'true' return period, according to the partial analysis.

Example. The once-per-year flood of \( T = 0.5 \) years, i.e. \( T_{\text{partial}} = 0.5 \) year of the partial series, equals the flood of \( T_{\text{annual}} = 1 / (1 - e^{-2}) = 1.16 \) year.

The determination of the once-per-year flood from the annual series through the above formula is less accurate. This is because most of the actual flood data in that range are not used in the annual analysis.

Statistical analysis. The statistical analysis, to determine the magnitude of the event (e.g. the peak discharge) with a certain return period (e.g. \( T = 20 \) years) is as follows:
- take \( N \) years of data;
- select the type of data, i.e. annual, partial, complete;
- arrange the data in the descending order, so from large to small;
- assign the order number \( m \) to the data, i.e. \( m = 1 \) to the largest, etc.;
- estimate the frequency of all data, i.e. assign the 'estimated' return period \( T \) or the 'estimated' probability of non-exceedance \( P \) to all data;
• select the type of distribution, e.g. log-Gumbel, log-normal, linear-normal, by plotting the
data on different probability paper, and by testing on a straight distribution;

• plot the data on the selected probability paper;
• draw a straight line on 'best fit' through these points;
• extend this straight line to the range of the desired return periods;
• read the magnitude of the event \( Q_r \) for the required return period \( T \).

**Estimation of the plotting position.** The data are first arranged in descending order of
magnitude and assigned to an order number \( m \), where \( m = 1 \) for the largest value. These data
have to be plotted on paper. The question is: how to plot?

The magnitude of the event, e.g. discharge is \( m^3/s \), is available and is related to the
**vertical axis.** The parameter for the **horizontal axis** is either:

• the 'return period' ("herhalingsfijn"), \( T \) in years,
• the 'probability of exceedance' ("kans van overschrijding"), \( P \) in \%,
• the 'probability of non-exceedance' ("kans van onderschrijding"), \( P' \) in \%.

**The return period.** There are many formulas available to estimate the return period of an
event. The term 'estimated' is important, because the hydrologist will determine 'true' return
period only after the plotting process. The 'estimated' return period is commonly estimated
by the formula (Chow 1964):

\[
T = \frac{N + 1}{m}
\]

where:

\( T \) is the 'estimated' return period of the event, in years
\( N \) is the number of years of record,
\( m \) is the order number of event, where \( m = 1 \) for the largest value.

For **annual series**, the highest value of the rank \( m \) is determined by the number \( N \) of
years of record, and equal \( m = N \). Thus, the **lowest** plotting position \( T \) depends on
the number of years of record only. For instance, an annual series of \( N = 10 \) years leads to \( m = 10 \) for the lowest event, so \( T = (N+1)/m = (10+1)/10 = 1.10 \). Thus, the 'estimated'
return period \( T \) will always be \( > 1 \).

For **partial series**, the highest rank of \( m \) is determined by the choice of the "base
value". For instance, the rank of event \( m \) could become e.g. \( m = 35 \), while the number of
years is only \( N = 4 \), thus \( T = (4+1)/35 = 0.14 \) years. Thus, \( T \) will also become \( < 1 \).

**Probability of exceedance.** The probability of exceedance ("kans van overschrijding") follows
from:

\[
P = \frac{1}{T} \times 100\% \quad \text{for } T > 1 \text{ year}
\]

where:

\( P \) is the probability of exceedance in \%,
\( T \) is the 'estimated' return period of the event, in years.

For instance, a return period of \( T = 20 \) years equals the probability of exceedance \( P = 0.05
= 5\% \). And also, the return period of \( T = 2 \) years equals the probability of exceedance \( P = 0.50 = 50\% \), which means that "on average, the flood happens every second year".

The probability of exceedance ("kans van overschrijding") for extreme values has only
a meaning for \( P > 50\% \).
Figure 1.10. Four types of probability paper.
Probability of non-exceedance. The probability of non-exceedance ("kans van onderschrijding") follows from:

\[ P' = 100\% - P \]

and thus:

\[ P' = \left(1 - \frac{1}{T}\right) \times 100\% \quad \text{for } T > 1 \text{ year} \]

where:
- \( P' \) is the probability of non-exceedance in \%,
- \( P \) is the probability of exceedance in \%,
- \( T \) is the 'estimated' return period of the event, in years.

For instance, a return period of \( T = 20 \) years equals the probability of non-exceedance \( P' = 0.95 = 95\% \).

It is often more practical in analysis of extremes to use just the 'return period', or alternatively the 'probability of non-exceedance' \( P' \). The return period has also the advantage that it can be used for frequent events. The twice-per-year flood has a return period of \( T = 0.5 \) years, while the probability of non-exceedance would read a meaningless \( P' = -100\% \).

Type of distribution. It is necessary that the type of distribution of the event is known. This can be assessed by plotting the data on different types of probability paper, and to choose that distribution for which the points can be represented by a straight line on the corresponding paper.

Some of the possible combinations are, see table 1.1 and figure 1.10:
- for a horizontal axis:
  - Normal (Hazen) probability distribution ................. for "complete series"
  - Gumbel probability distribution ....................... for "annual series"
  - logarithmic scale ................................... for "partial series"
- on the vertical axis:
  - linear scale ....................................... "x"
  - logarithmic scale ................................... "ln x" or "log x"

Drawing of the straight line. Statistics 'by plotting data' is essentially based on drawing a straight line. It is only a straight line that can be extended to the range of the required return periods.

The advantage of the statistics 'by plotting data' is that an optical insight is obtained in the 'best fit' and thus in the accuracy of the analysis. Moreover, the hydrologist can make his personal decision to underweight certain points. For instance, the points of the lower return periods in the annual analysis tend downwards because of many data in this range have been filtered out. Also, a very high flood in a series of \( N = 15 \) years, may not have the 'estimated' return period of e.g. \( T = 16 \) years, but a real return period of \( T = \pm 100 \) years.

Table 1.1. The use of plotting paper.

<table>
<thead>
<tr>
<th>Type of Series:</th>
<th>COMPLETE SERIES</th>
<th>PARTIAL SERIES</th>
<th>ANNUAL SERIES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Data:</td>
<td>annual rainfall</td>
<td>extremes ( T &lt; 2 ) years</td>
<td>maximum floods</td>
</tr>
<tr>
<td></td>
<td>monthly rainfall</td>
<td>short extreme series</td>
<td>minimum discharge</td>
</tr>
<tr>
<td>Type of Paper:</td>
<td>linear-Normal paper</td>
<td>(linear-log paper)</td>
<td>linear-Gumbel paper</td>
</tr>
<tr>
<td></td>
<td>log-Normal paper</td>
<td>log-log paper</td>
<td>log-Gumbel paper</td>
</tr>
</tbody>
</table>
1.3. Drainage Systems in Polders

1.3.1. Polder Development

Different professional disciplines. Many professional disciplines are involved in design of polders. The first activities on polder development are covered by the hydraulic engineers. They design and construct a ring dike ("ringdijk") and one or more pumping stations. The main channels to the pumping stations are constructed by underwater dredging. Finally, the water is removed by the pumping stations.

The second group of activities are taken over by drainage engineers and by agricultural engineers. They make the new land fit for agricultural use.

Finally, a third group of activities is covered by rural planning engineers and other specialists. The polder is developed with the necessary infrastructure (villages, roads, etc.) and landscaping. Also the non-technical matters are covered, such the selection of "immigrants" and the setting up of a Public Administration.

An example of the empoldering of a shallow lake, the Hachiro Gata polder in Japan, is presented in box 1.1, see figure 1.11.

**Box 1.1. Hachiro Gata polder.**

The Hachiro Gata polder is located in shallow lagoon in Japan and was completed in 1972. The design of the Hachiro Gata polder comprises:

- the size of the polder is 16,000 ha, and the depth is some 5.00 m below sea level;
- a part of the lagoon remains open-water, i.e. the 'belt canal' ("randmeer, boezem"), and has a function on (i) water storage for irrigation, (ii) prevention of groundwater drawdown in the old-land, and (iii) for collection of surface drainage water from the old-land;
- there is a separated ("gescheiden") irrigation and drainage system in the polder to allow the cultivation of paddy ("rijk");
- the drainage modulus of the paddy fields is based on a design storm ("ontwerpbu"") of 225 mm in three consecutive days ("oepenvolgende dagen"), when a water level rise on the paddy fields of 0.10 m is allowed. This leads to a drainage modulus of 5 l/s.ha or 43 mm/day;
- a sub-drainage system under the paddy fields at a depth of 0.50 - 0.70 m and at a distance of 10 m has been designed, to leach ("uitspoelen") the soil as to prevent problems with acid-sulphate clay ("katteklei").

Components. The water management system in a polder exists of different components, see figure 1.12:

- **field drainage system** ("ontwateringssysteem"), to maintain the groundwater table under the root zone;
- **main drainage system** ("afwateringssysteem"), to divert the drainage water from the field drainage system to the outlet;
- **sluices and/or pumping stations** ("lozingssysteem"), to evacuate the water from the main drainage system to the bordering canals;
- **belt system** ("boezem") surrounding the polder.
Figure 1.11. Hachiro Gata polder, Japan.
1.3.2. Main Drainage System

Function. The function of the main drainage system ("afwateringssysteem") is to collect the water from the field drainage system, to transport the water to the outlet, and to evacuate the water from the area.

Types. The main drainage system of a typical new polder in the Netherlands consists of: (i) canals, and (ii) main ditches and plot ditches.

The canals are often navigable and have (stagnant) water depths of some 1.50 - 2.50 m. The gradient of the canal bed is normally horizontal. The main ditches and plot ditches run along the plots, e.g. the rectangular fields of about 500 m $\times$ 1200 m.

Polder water level. The main drainage system flows mainly because of rainfall. So most of the time it contains stagnant water. The water level of this stagnant water must be carefully controlled. A lower or a higher water level might have negative effects on the crops and on land subsidence. This target water level of the open-water drainage system of a polder can be called the polder water level ("polderpeil"), see figure 1.13.

Thus, water management in a polder is mainly determined by water level control ("peilbeheer") in the drainage system, and to a lesser extent by discharge control ("debietbeheer") like in irrigation schemes.
Figure 1.13. The 'polder water level' is the target the open-water level.

Capacity. The required capacity of the drainage system depends not only on the rainfall but also on the seepage, e.g. 1 mm/day (0.1 l/s.ha) or higher, and on lockage water from polder shiplocks, e.g. 1 mm/day (0.1 l/s.ha).

Furthermore, the required capacity depends also on the percentage of open-water, which is kept low in the modern polders (1 - 2%) with electric pumping stations, but might be much larger (5 - 15%) in older peat polders with pumping by windmills.

Table 1.2 presents some typical values of canal capacities in the main drainage system for moderate climates (Luijendijk 1982). These discharges are of the order of 13 mm/day (1.5 l/s.ha) for the smaller ditches and of 11 mm/day (1.3 l/s.ha) for the larger canals. For urban polder areas, values for the main system of 20 mm/day (2.3 l/s.ha) and more are often applied.

<table>
<thead>
<tr>
<th>Polder type</th>
<th>Soils</th>
<th>Land use</th>
<th>Open water %</th>
<th>Polder water level m-terrain</th>
<th>Canal capacity mm/day</th>
</tr>
</thead>
<tbody>
<tr>
<td>old polders</td>
<td>peat</td>
<td>grass</td>
<td>5 - 10</td>
<td>0.20 - 0.50</td>
<td>8 - 12</td>
</tr>
<tr>
<td>old polders</td>
<td>clay</td>
<td>grass</td>
<td>3 - 10</td>
<td>0.40 - 0.70</td>
<td>8 - 12</td>
</tr>
<tr>
<td>old polders</td>
<td>clay</td>
<td>crops</td>
<td>5 - 10</td>
<td>0.80 - 1.00</td>
<td>8 - 12</td>
</tr>
<tr>
<td>new polders</td>
<td>clay</td>
<td>crops</td>
<td>1 - 2</td>
<td>1.40 - 1.50</td>
<td>11 - 14</td>
</tr>
<tr>
<td>urban polders</td>
<td></td>
<td>crops</td>
<td>3 - 8</td>
<td>1.50 - 1.80</td>
<td>15 - 30</td>
</tr>
</tbody>
</table>
Operational objectives. The operation of gravity ("zwaartekracht") drainage systems is normally not required during the periods of rainfall. The field drains just confluence into the smaller drains, and these smaller drains discharge into the larger drains. There is no need to maintain a certain minimum water level in the drains, so that they may fall dry during periods of no rainfall.

However, water level control ("peilbeheer") in the drainage system might be needed during the dry periods, for instance to store groundwater in the soil. This groundwater can be used by the crop by "capillary rise" ("capillaire opstijging"), or will avoid subsidence ("klink").

Layout. The main drainage system of a polder can be designed in a grid pattern, or in a branching pattern, see figure 1.14.

Moreover, the question may arise in polders with irrigation water needs, whether separate irrigation and drainage systems ("gescheiden stelsel") are constructed, or that the mixed irrigation and drainage system ("gemengd stelsel"), where the irrigation supplies are taken from the drainage system.

A grid pattern can be applicable for flat polders. It has the advantage that the canals can be used as boundaries between land holdings. Examples of grid patterns can be found in the polders of the Netherlands.

A branching pattern ("vertakt") of the main drainage system might be applied for sloping polders. A branching drainage system is also applied in polders with sulphate-acid problems to flush the acid water. Examples can be found in Indonesia.

Irrigation. Irrigation supply to the crops with a grid drainage system is possible by means of capillary rise of the groundwater ("capillaire opstijging"). The appropriate groundwater level is maintained by bank-infiltration ("oever infiltratie") from the open-water system.

The other possibility for irrigation in a grid drainage system is by means of pumping by the individual farmers on an 'on-demand' supply basis, where the open-water level in the polder is maintained by subsequent intake into the polder.

A branching drainage system must be applied in polders with a separate irrigation and drainage systems, such as in Japan, Korea and Taiwan. Here, gravity ("zwaartekracht") is used to irrigate from the higher irrigation system to the fields, while the water levels in the drainage system are kept sufficiently below terrain level.

![Diagram of drainage systems](image)

Figure 1.14. Pattern of main drainage system in polders.
1.3.3. Polder Water Level

Determination. The polder water level, i.e. the stagnant water level, is often established on agricultural considerations to obtain the highest economic return of the crops (Kley 1969). The relevant parameters are:

- **type of the soil**, like peat that requires shallow groundwater,
- **proposed land use**, and the yield curve ("opbrengstcurve") for different open-water levels, see figure 1.15 for example,
- **elevation of the terrain**.

But the polder water level has also to meet interests of navigation, recreation and nature preservation ("natuurbeheer"). Moreover, it should prevent or reduce the irreversible subsidence ("klink") of the soil.

An example of the determination of the polder water level based on the highest crop yields of figure 1.15 is given in box 1.2 and in table 1.3.

![Graph showing crop yields for different open-water levels](image)

**Figure 1.15.** Example of crop yields for different open-water levels.

**Box 1.2.** Determination of the polder water level.

A certain polder has a size of 1200 ha. A lower area at 0.00 m+MSL of 900 ha of peat soils will be used for grass, a higher area at 1.00 m+MSL of 300 ha of clayey soils will be used for crop cultivation.

The yield curves for different (stagnant) open-water levels ("polderpeilen") in the channels below terrain level, are presented in figure 1.15. The economic value of arable land ("bouwland") is twice as high as the value of grassland. Thus, the annual yield of arable land is $2X$, against an annual yield of grassland of $X$. The calculation is done by means of table 1.3.

Each assumed open-water level gives a certain relative yield of the grassland and of the arable land. It shows that the total annual yield of $1470X$ is maximum for **one polder water level of 0.40 m-MSL**.

As an alternative, two different polder water levels can be used: 0.40 m-MSL for the grassland and 0.60 m-MSL for the arable land. The total annual yield would be $900X + 600X = 1500X$. The extra construction costs would include a gated drop structure between the two polder sections.
Table 1.3. Example: Calculation of the polder water level.

<table>
<thead>
<tr>
<th>Polder water level</th>
<th>Grass land (900 ha)</th>
<th>Arable land (300 ha)</th>
<th>Total polder (1200 ha)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m-M SL</td>
<td>Water level m-terrain</td>
<td>Yield in $</td>
</tr>
<tr>
<td>0.60</td>
<td>0.60</td>
<td>0.75 × 900X</td>
<td>1.60</td>
</tr>
<tr>
<td>0.50</td>
<td>0.50</td>
<td>0.90 × 900X</td>
<td>1.50</td>
</tr>
<tr>
<td>0.40</td>
<td>0.40</td>
<td>1.00 × 900X</td>
<td>1.40</td>
</tr>
<tr>
<td>0.30</td>
<td>0.30</td>
<td>0.93 × 900X</td>
<td>1.30</td>
</tr>
</tbody>
</table>

Soils. Generally, peat soils ("veen") require high polder water levels of some 0.30 - 0.60 m below terrain level to prevent the subsidence of the soil. Thus, grass and other crops only with shallow roots can be cultivated.

Clayey soils permit deep polder water levels of some 1.20 - 1.80 m below terrain. It means that dryland crops with a thick root zone can be cultivated.

Even greater depths of 1.50 - 2.00 m below terrain have to be selected in polders with brackish seepage and with low rainfall. Capillary rise of the brackish water can be avoided by over-irrigating the crops and allowing a percolation flow to the drains.

By-law. There are conflicting interests in the determination of the polder water level: the highest crop yield for the whole polder may not give the highest yield to each individual farmer, and may not meet the requirement for subsidence, or may not allow navigation in the channels, etc.

Thus, the ultimate polder water level will be a compromise between the different parties. This selected polder water level is laid down in a by-law ("peilbesluit") of the Water Board ("waterschap").

Very often a summer polder level ("zomerpeil") and a deeper winter polder level ("winterpeil") is applied, as to provide for more drainage in the wetter winter season.

Box 1.3. Polder sections.

- In alternative I, the drainage water is discharged from the higher to the lower sections by means of gated drop structures. Only one pumping station, with a capacity of \( Q_1 + Q_2 + Q_3 \) is required, but at the highest operation costs, which are related to \( (Q_1 + Q_2 + Q_3) \times H_1 \), where \( H \) and \( Q \) are the required pumping head and discharge of the different polder sections, respectively.
- In alternative II, the drainage water is gradually pumped to the higher polder sections.
  The total operation costs will be lower than for the above alternative and will be related to \( H_1 Q_1 + H_2 Q_2 + H_3 Q_3 \).
- In alternative III, the minimum operation costs and the minimum pump capacity costs can be obtained by draining the three polder sections directly to the external water system. The lower and the higher sections are located along the external water system, and can discharge directly. The middle polder section in figure 1.16 should be connected with the external water system directly, or by means of a "belt canal" ("ringkanaal"). Three pumping stations are required here, with capacities related to: \( Q_1, Q_2 \) and \( Q_3 \), respectively. The total operation costs are at a minimum and are related to: \( H_1 Q_1 + H_2 Q_2 + H_3 Q_3 \).
Polder Sections. Polders may be divided into compartments or sections having different polder levels ("polderafdelingen"), depending on the topography as to limit the variation in groundwater depth to e.g. 0.50 m. Different polder levels are also applied when soil conditions differ, e.g. peat soils with high polder levels ± 0.50 m below terrain, clay soils with deep polder levels at ± 1.50 m below terrain. So is the Wieringermeer Polder (the Netherlands) divided into four sections with different levels, and the Flevo Polder (the Netherlands) into two sections. An example of a polder with three polder sections is presented in figure 1.16 and in box 1.3 (Kley 1969).

Alternative I: One pumping station (maximum energy requirements).

Alternative II: Three pumping stations (minimum energy requirements).

Alternative III: Three pumping stations & high "Belt Canal"
1.3.4. Water balance

Units of the water balance. The terms of a water balance are often expressed in a water depth ("waterschijf") per time unit: in 'mm/day', like the evaporation and rainfall data. However, the unit 'l/s.ha' for water depth is alternatively used in the design of drainage systems. The conversion between these two units is:

- 1 mm/day = 0.116 l/s.ha,
- 1 l/s.ha = 8.64 mm/day.

Water balance equation. The water balance equation of a polder reads:

\[ P + S + I = E_o + ET_c + Q + \frac{\Delta V}{\Delta T} \]

The inflow consists of precipitation \( P \), seepage from the sub-soil \( S \), and intake water \( I \) for irrigation supplies and lockage water from ship-locks.

The outflow consists of open-water evaporation \( E_o \), evapo-transpiration of the vegetation \( ET_c \), and the pumping and sluicing discharges \( Q \).

The difference between the in- and outflow over the considered period \( \Delta T \) determines the changes in storage ("berging") \( \Delta V \) in the soil and in the open-water. This time interval \( \Delta T \) can be taken as e.g. one year for determination of the annual pumping costs, or as one day or even shorter for the determination of storm-drainage requirements of the canals and the pumping stations.

Seepage. The assessment of the seepage ("kwel") to new polders is quite difficult as the seepage depends on the permeability of the sub-soil, the thickness of the aquifers and on several other geo-hydrological parameters.

The seepage can be divided into (i) a seepage through the embankment, and (ii) a seepage from the deeper soil layers.

The total seepage of existing polders can be calculated by means of the water balance equation. The typical seepage value of polders in the Netherlands is 1 mm/day (0.1 l/s.ha), but values as high as 13 - 16 mm/day (1.5 - 1.9 l/s.ha) are also encountered (Schultz 1992).

Irrigation needs. Many polders require a regular intake of fresh water during a part of the growing season. For instance, grass-polders in the Netherlands may require an irrigation supply upto 0.6 l/s.ha during dry years. Some 1.5 l/s.ha is required in the paddy-polders in Indonesia.

The quality of deep seepage water is often poor, as it can be brackish or even salty. Thus, the salt water balance of the polder should be checked. Polders in (semi) arid climates may require a considerable irrigation supply together with a good drainage system, to control the saline conditions of the root zone of the crops.

Water storage. Water storage in open-water channels ("open-water berging") depends on the percentage of open-water in the polder and the allowable rise in water level.

For instance, a modern polder may have 1% of open-water, with an allowable rise of water level of 0.20 m. The rate of open-water can also be expressed in a water depth ("waterschijf") over the whole polder. So, the polder in the above example has an open-water storage of \( 0.01 \times 0.20 = 0.002 \) m = 2 mm.

Water storage in the soil depends on factors like the "drainable pore space" ("bergings-
coefficient") of the soil and the allowable rise of the groundwater table (Luijendijk 1982). So, polders with deeper open-water level during the dry season will store more water in the soil during heavy rainfall, see table 1.4.

Required pump capacity. The water balance with a time interval of ΔT = 1 day can be used for a rough estimate of the required pump capacity (Kley 1969). An example is presented in box 1.4 and in table 1.5.

Table 1.4. Typical values of water storage in the soil.

<table>
<thead>
<tr>
<th>Open-Water Level below terrain level</th>
<th>Water storage in soil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>peat</td>
</tr>
<tr>
<td>0.20 m</td>
<td>20 mm</td>
</tr>
<tr>
<td>0.40 m</td>
<td>45 mm</td>
</tr>
<tr>
<td>0.60 m</td>
<td>75 mm</td>
</tr>
<tr>
<td>0.80 m</td>
<td>105 mm</td>
</tr>
<tr>
<td>1.00 m</td>
<td>60 mm</td>
</tr>
<tr>
<td>1.20 m</td>
<td>85 mm</td>
</tr>
<tr>
<td>1.40 m</td>
<td>115 mm</td>
</tr>
</tbody>
</table>

Table 1.5. Water balance of a polder for the design rainfall.

<table>
<thead>
<tr>
<th></th>
<th>Day no.1</th>
<th>Day no.2</th>
<th>Day no.3</th>
<th>Day no.4</th>
<th>Day no.5</th>
<th>Day no.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall mm/day</td>
<td>0</td>
<td>18</td>
<td>33</td>
<td>12</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Percolation (Rain-day1) mm/day</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percolation (Rain-day2) mm/day</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percolation (Rain-day3) mm/day</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total percolation mm/day</td>
<td>0</td>
<td>6</td>
<td>17</td>
<td>21</td>
<td>15</td>
<td>4</td>
</tr>
<tr>
<td>Percolation to open-water mm/day</td>
<td>5</td>
<td>15</td>
<td>19</td>
<td>14</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Rainfall on open-water mm/day</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seepage mm/day</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Total flow to open-water mm/day</td>
<td>1</td>
<td>8</td>
<td>19</td>
<td>21</td>
<td>15</td>
<td>5</td>
</tr>
<tr>
<td>Pumping from open-water mm/day</td>
<td>-1</td>
<td>-8</td>
<td>-14</td>
<td>-14</td>
<td>-14</td>
<td>-14</td>
</tr>
<tr>
<td>Open-water storage mm</td>
<td>0</td>
<td>0</td>
<td>5</td>
<td>12</td>
<td>13</td>
<td>4</td>
</tr>
<tr>
<td>Open-water elevation m+MSL</td>
<td>0.00</td>
<td>0.00</td>
<td>0.05</td>
<td>0.12</td>
<td>0.13</td>
<td>0.04</td>
</tr>
<tr>
<td>Water storage in soil mm</td>
<td>0</td>
<td>12</td>
<td>28</td>
<td>19</td>
<td>4</td>
<td>0</td>
</tr>
<tr>
<td>Groundwater elevation m+MSL</td>
<td>0.10</td>
<td>0.29</td>
<td>0.54</td>
<td>0.40</td>
<td>0.16</td>
<td>0.10</td>
</tr>
</tbody>
</table>


Box 1.4. Water balance of a polder.

A polder may have an open-water ratio of 10% and the groundwater level is initially 0.10 m above the stagnant open-water level, i.e. at 0.10 m + MSL, see also table 1.5.

The water storage in the soil is 7 mm per 0.10 m groundwater rise, i.e. 'drainable pore space' ("bergingscoefficient") of 0.07. A seepage flow to the polder amounts to 1 mm/day (0.1 l/s.ha). The design rainfall may consist of a three-day storm of 18 mm/day, 33 mm/day and 12 mm/day, respectively.

It is assumed that the rainfall of one day percolates during 3 days, i.e. during the day of rainfall and the two following days. This percolation and the seepage is discharged to the open-water, where the pump with a capacity of 14 mm/day, is removing the water.

An open-water storage of 4 mm in day no.3 makes a rise of the open-water level of 0.004/10% = 0.04 m. The groundwater storage of 28 mm in day no.3 was created by the rainfall storage (18+33) minus the percolation (6+17). This 28 mm give a groundwater rise over the land area 90% of 0.028/(90% × 0.07) = 0.44 m.

Thus, it follows from the calculations in table 1.5 that the average groundwater level will rise by 0.54 - 0.10 = 0.44 m, and the open-water level by 0.14 m for an assumed pump capacity of 14 mm/day (1.6 l/s.ha).

1.4. Irrigation Systems

1.4.1. Irrigation Development

Dual-management. Many large-scale irrigation schemes are under 'dual-management'. The Government, or another entity, is charged with the operation, management and maintenance of the main infrastructure of the scheme. This part of the scheme belongs to the general interest of the population, and requires technical, financial and organizational capabilities that are beyond the capacity of the individual farmer.

However, such an entity cannot be charged with the operational aspects at the lowest level, such as the operation and maintenance at farm level. This remains the task of the individual farmer, or may become the task of a group-of-farmers.

Irrigation systems. Thus, many irrigation schemes are divided into two parts, with each their own management responsibilities, see figure 1.17:

- the 'main irrigation system', ("hoofdsysteem", "dienstleidingen"), i.e. the part that is under the control of the Government or another entity, through e.g. an 'O&M Agency' ("beheersdienst", "irrigatiedienst");
- the tertiary unit, ("tertiaire vak", "bevolkingsvak"), i.e. the part that is under the control of a number of farmers, to be grouped in e.g. 'Water Users Associations' ("watergebruikersorganisatie").
Terminology. The terminology of irrigation system as used here, is shown in figure 1.18. The 
feeder canal ("onverdeelde hoofdkanaal") diverts the irrigation water from the headworks
("watervang") to the irrigation scheme. The primary canal ("hoofdkanaal") transports the
water to the different sub-areas of the irrigation scheme. Often, the gradient of such a
primary canal is flat, as to maintain the highest head.

Secondary canals ("secundaire kanalen") take off through secondary offtakes
("verdeelkunstwerken", "secundaire aftappingen") from the primary canal, and may divert
into other (sub)secondary canals. These (sub)secondary canals distribute the water through
the irrigation scheme to the tertiary offtake ("tertiaire aftapping").

The main irrigation system ("hoofdsysteem", "dienstleidingen") consists of the
headworks, the feeder canal, the primary canal, the (sub)secondary canals, and all the
structures on these canals. Thus, the tertiary offtakess belong to the main system.

The tertiary unit ("tertiaire vak", "bevolkingsvak") is the irrigation area supplied by
one tertiary offtake. The tertiary unit consists of tertiary canals ("tertiaire kanalen", 
"bevolkingsleidingen") and quaternary canals ("quartearne kanalen") with their structures.
The tertiary unit is under responsibility of the water users association. The individual farms
receive their water from the quaternary canals.

Tertiary offtake. The structure that diverts the water from the main system into the tertiary
unit is called 'tertiary offtake' ("tertiaire aftapping"). Other names are 'turnout', 'outlet',
'inlet' and 'mogha'. The tertiary offtake is the official point at which water passes from the
control of the O&M agency to the water users association. But it is also the site of struggle
between the water users and the staff of the O&M agency.
1.4.2. Main Irrigation System

Function. As elaborated above, the main system is managed by the O&M Agency and not by the farmers. The function of such a main irrigation system is to supply irrigation water to the tertiary units according to the 'operational objective'.

Main irrigation systems can only be well designed when the ultimate operational objectives have been specified accurately. Otherwise, controversy will remain during the operation of the project, whether the system is performing 'to the requirements'.

Moreover, the technicians (civil engineers, agriculturists, economists, etc.) involved in the design of a new irrigation system, and the ultimate users of this system (local administration, O&M Agency, farmers, etc.) have to agree jointly on these operational objectives.

Irrigation requirement. The 'irrigation requirement' ("irrigatiebehoeften") of the crop depends on many factors, such as the water requirement of the crop, the effective rainfall, and the application efficiency. The crop water requirement is normally calculated by a formula, such as the Penman-Monteith formula (Ankum 1996). The irrigation requirement is normally expressed in the unit of a 'water depth per time', such as mm/day or l/s.ha., whereas 1 l/s.ha = 8.64 mm/day.

Canal capacity. The capacity of an irrigation canal is the maximum discharge, in m³/s, that should be accommodated during the peak period. It is also often expressed in l/s.ha, to make it independent of the command area ("verzorgingsgebied").
There are several methods to determine the canal capacity of an irrigation system. Traditionally, the capacity of irrigation systems is based on experience and not on calculations. For instance, typical canal capacities in Indonesia were taken at 1.4 l/s.ha for the irrigation need of paddy ("rijst") during the wet-season.

At present, more emphasis is given to the calculation of the canal capacity. The required canal capacity depends on the peak irrigation requirement of the crop and the water losses in the system.

In India, the canal capacities were intentionally taken below the peak irrigation requirement of the crop. It is a dilemma that a supply of the full crop water requirement to a (small) area means that the remainder area will not receive any irrigation water at all. Thus, it was decided to supply many farmers with less than the required irrigation supplies. Such an irrigation method is called 'protective irrigation', or also 'extensive irrigation'. Typically, the canal capacities are only 0.3 - 0.6 l/s.ha to irrigate the dryland crops ("gewassen") in a large command area. Thus, the cropping pattern ("gewaspatroon") is less than 100%, which means that a part of the area might be fallow ("braak") during the irrigation season.

Importance of the off-peak period. It should be acknowledged that the operation of the main system during the peak discharges differs from the operation during off-peak discharges with less flow through the main system.

Often, the operation during the peak period is simple, as the canals are just flowing at their maximum discharge. The operation might become cumbersome during the off-peak period, see figure 1.19. These off-peak conditions determine normally the operational objective of the main system.

\[ \text{Figure 1.19. Example of flow diversion during the 'off-peak period'.} \]
Diversion structures. Water control systems in irrigation have to divert the larger discharges into smaller portions during the peak period. Moreover, the diversion of the required flow is more cumbersome during the off-period period when the canals are flowing at a lower discharge than their capacities.

Therefore, diversion structures ("verdeelkunstwerken") are needed at all bifurcations ("splitingen"), see figure 1.20. These structures may have one or more of the following components, see also the next chapters (Ankum 1995):

- discharge regulator ("debiet regelaar"), i.e. to regulate the discharge,
- water measurement structure, ("meetkunstwerk"), i.e. to measure the discharge,
- water level regulator ("peilregelaar"), i.e. to regulate the water level.

![Diagram of irrigation system with diversion structures at bifurcations](image)

**Figure 1.20.** Diversion structures at all bifurcations in a main irrigation system.

### 1.4.3. Flow Control Methods

**Introduction.** Many irrigation schemes were initially constructed as simple flooding systems, by which an unregulated supply of water from the river was spread over the fields. A basic improvement was made by splitting the flow in a proportional way, as to relate the irrigation flow to the size of the command area.

A next step in the development of flow control methods was the transition of these main irrigation systems into upstream controlled systems by placing gates in the proportional control structures, see figure 1.21. The simple use of these gates on an 'on/off' basis
improved the efficient use of water in many cases. The introduction of water measurement structures with the above gates allowed the systematic distribution of water by an O&M Agency.

Since the introduction of the downstream control concept in the 1930s, 'self-management' of main irrigation systems are possible. These systems react on changes in water demand without system management, see also figure 1.21.

Since the 1960s, the technology of telemetry on the individual canal reaches (volume control, ELFLO control) and since the 1980s also of computerized control on the main system as-a-whole, allows for a further optimization of the distribution of irrigation water. These methods are not discussed here, and can be found in the literature (e.g. Ankum 1995).

**Figure 1.21.** Control methods in irrigation.

**Proportional control.** Irrigation systems under 'proportional control' ("proporzionele verdeling") divide and distribute the water according to a fixed ratio ("vaste verhouding"), see figure 1.22. Proportional control is a pure type of discharge control: the primary purpose of a structure is to maintain a certain ratio-in-discharges to a downstream destination. An irrigation system under proportional control is very simple in construction and in operation.

Proportional control can well be applied in modern irrigation schemes. For instance, when an irrigation scheme has one cropping pattern and a uniform growing stage for all tertiary units, there is no need to divert more water to certain tertiary units. Thus, irrigation water can be released from the headworks and can be proportionally distributed to all tertiary units on basis of the command area. A condition is of course, that the lower discharges do not give managerial problems within the tertiary units, e.g. a too low water level for gravity irrigation, or a too low discharge to be managed by the farmers.

Proportional control is the most simple control method in irrigation. However, it cannot be used efficiently for different crops with different water needs, since the flow cannot be regulated.
Structures under proportional control. Although there is no active discharge regulation is needed, structures are still required at every offtaking canal. Otherwise, the downstream water levels will influence the proportional distribution. This happens because of the different depth-discharge relations of the canals \( Q = k A R^{2/3} s^{1/2} \).

The most simple structure in proportional control is the Fayoum weir (Kraatz 1975, Bos 1989, Ankum 1995), see figure 1.23. The Fayoum weir as a divisor for proportional control, consists of two similar type of overflow structures to the continuing canal and to the offtaking canal. A fixed division of the discharges is obtained when both crest-height \( p \) above the canal bed are at the same. The widths \( b_c \) and \( b_t \) of weirs determine the ratio of the discharge into each canal.

Upstream control. The above proportional control may not meet the expectations of a main irrigation system, and a need for a more active regulation of flow may arise. A logic decision is to equip the above proportional distribution structures with gates, so that regulation of the discharge becomes possible. Thus, a system under 'upstream control' ("bovenstrooms peilbeheer") is created, see figure 1.24.

However, new drawbacks on the flow control method are created by solving the imperfections of the proportional control:
• a central system management is required, where the O&M Agency decides on the water releases to the tertiary irrigation units, and on the cumulated discharges through the whole main system;
• it appears that efficient operation of an upstream controlled system is very cumbersome, because of the response time due to the filling and the emptying of the ‘dynamic canal storage’.

Canals under upstream control. Structures under upstream control maintain the upstream water level at the target water level, which means that the upstream water level is constant for each discharge, see also figure 1.24.

However, such a water level control will not lead to a workable water distribution through the irrigation scheme: it will effect a proper diversion of discharges.

Figure 1.24. The principle of ‘upstream’ control.

Upstream control is a ‘control-of-discharges’. A main irrigation system under upstream (water level) control is essentially a discharge controlled system. Water is released from the headworks and is distributed over the smaller canals and ultimately towards the tertiary units, see figure 1.25. Discharge regulation and discharge measurement is required at all offtaking canals, as to control the discharge that leaves the canal. The continuing canal is transporting the remainder part of the discharge.

Figure 1.25. Diversion of discharges through an ‘upstream’ controlled system.
Structures under upstream control. Irrigation structures under upstream control are quite complex. These structures may have one or more of the following functions, see figure 1.26: (i) discharge regulation, (ii) discharge measurement, and (iii) water level regulation.

Moreover, there are many alternatives to achieve these functions, such as 'fixed', 'manual', or 'hydro-mechanical' regulation, 'underflow' or 'overflow'. These matters will be discussed in the following chapters.

OFFTAKE TO A SECONDARY CANAL

OFFTAKE TO A TERTIARY UNIT

Figure 1.26. Irrigation structures under upstream control.

Downstream control. Downstream control is the control of the water level regulator, based upon changes in the water level immediately downstream of it, see figure 1.27.

The regulators in the main system maintain a constant water level at the downstream side of the structure, without regarding discharges. Such a regulation of structures means that more supply is given to a canal reach when the water level drops. The effect is that the discharge at each regulator is automatically adjusted to the accumulated downstream demand for irrigation water.

Figure 1.27. Principle of 'downstream' control.
Downstream control as 'self-management'. The term main system management refers to the management of the system-as-a-whole, and deals with e.g. determination of the target water levels and/or target discharges. The main system management can either be (Ankum 1992b), see figure 1.28:

- **no management** of the main system, which occurs with proportional control because any adjustment of structures is impossible;
- **central management** of the main system, which occurs with upstream control because a 'water operation centre' should manage the inflow and the total outflow of the whole system;
- **self management** of the main system, which occurs with downstream control as the water control system adjust the inflow to a changing outflow.

![Diagram showing proportional, upstream, and downstream control](image)

**Figure 1.28.** Overall 'system management' of main irrigation systems.

**Structures under downstream control.** The water level regulators at the secondary offtakes are located in the incoming canal reach, see figure 1.29. These regulators are not located in the continuing canal reach, like in upstream controlled systems. A discharge regulator at a (sub)secondary offtake is absent, and it is not matching with downstream control.

However, the tertiary offtakes have to be equipped with discharge regulators, as here water changes 'management', i.e. from the 'O&M Agency' to the 'Water Users Association', see also figure 1.29.

![Diagram showing irrigation structures under downstream control](image)

**Figure 1.29.** Irrigation structures under downstream control.
2. OPEN CHANNEL FLOW

2.1. Flow of Water

2.1.1. Uniform flow

**Background.** Uniform flow ("eenparige stroming") in an irrigation canal, a drainage channel or a river can be described by one of the three formulae, see figure 2.1:

- the **Strickler formula**: \( Q = k A R^{2/3} s^{1/2} \),
- the **Manning formula**: \( Q = 1/n A R^{2/3} s^{1/2} \),
- the **Chézy formula**: \( Q = C A R^{1/2} s^{1/2} \),

in which:
- the velocity of the water \( v = Q / A \)
- the water cross-sectional area \( A = (b+my)l \)
- the hydraulic radius \( R = A / (b + 2y \sqrt{(1+m^2)}) \)

where: \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( v \) is the velocity in \( \text{m/s} \), \( A \) is the wet cross-sectional area in \( \text{m}^2 \), \( R \) is the hydraulic radius in \( \text{m} \), \( s \) is the energy (channel) gradient, \( b \) is the bed width in \( \text{m} \), \( y \) is the water depth in \( \text{m} \), \( m \) is the side slope (\( 1_{\text{Vert}} : m_{\text{Hor}} \)), \( k \) is the Strickler coefficient in \( \text{m}^{1/3}/\text{s} \); \( n \) is the Manning roughness coefficient in \( \text{s}/\text{m}^{1/3} \), \( C \) is the Chézy roughness coefficient in \( \text{m}^{1/2}/\text{s} \).

![CROSS SECTION](image)

![LONGITUDINAL PROFILE](image)

**Figure 2.1. Parameters in the Strickler formula.**

**Relation between Chézy, Manning and Strickler formulae.** Basically, the Chézy, the Manning and the Strickler formula are the same equations. The difference lies in the determination of the 'roughness coefficient' ("rughheitscoefficient"):

- **Chézy** presented his formula in 1769. His roughness coefficient, \( C \) in \( \text{m}^{1/2}/\text{s} \), is not a constant for a specific channel but depends also on the water depth.

  The roughness coefficient should be calculated through another formula. Examples are the Kutter formula, the Basin formula, the Powell formula, the Nikuradse formula, etc.
Manning presented his formula \( v = \frac{1}{n} R^{2/3} s^{1/2} \) in 1889. He concluded that the roughness coefficient, \( n \) in \( \text{s/m}^{1/3} \), is independent of the water depth in the channel.

The coefficient, however, does depend on the general characteristics of the channel, such as soil, vegetation and (meander) form.

Strickler presented his formula \( v = k R^{2/3} s^{1/2} \) independently in 1923. The Strickler coefficient, \( k \) in \( \text{m}^{1/3}/\text{s} \), is related to Manning through \( n = 1/k \), and to Chézy through \( C = k R^{1/6} \). The 'roughness' ('"rawheid"') coefficient \( k \) as used in the Strickler formula is in fact a 'smoothness' ('"gladheid"') factor. An increasing roughness will decrease the value of \( k \). Therefore, it is better not to use the term 'roughness' coefficient but to use the term 'Strickler' coefficient instead.

The Strickler coefficient is independent of the water depth, but depends strongly on the maintenance ('"onderhoud"'). A well-maintained channel may have a Strickler coefficient of \( k = 30 \text{ - } 40 \text{ m}^{1/3}/\text{s} \).

Discussion on of the Chézy, Manning and Strickler formulae. The use of the Chézy, Manning or Strickler formula is mainly based on tradition:

- the Chézy formula is often applied by river engineers and in hydro-dynamic models;
- the Manning formula is widely used in the English-speaking countries by irrigation and drainage engineers;
- the Strickler formula is widely applied by irrigation and drainage engineers from the European continent.

The Manning and the Strickler formula have the practical advantage over the Chézy formula that they use a coefficient which is not dependent of the water depth. The Strickler formula is recommended here.

Strickler coefficient. The coefficient \( k \) in the Strickler formula has a constant value, that does not depend on the water depth \( y \). However, this Strickler coefficient can change considerably between cross sections and over the time. The factors that have the greatest influence upon the Strickler coefficient \( k \) are described below:

- surface roughness, depending on the size and shape of the grains of the bed material;
- vegetation, the Strickler coefficient may vary during the year which is due to seasonal growth of aquatic plants and grass in the channel or on the banks. Thus, maintenance has a major effect on the roughness;
- silting and sedimentation, the flow through a cross section will decrease after siltation and before it is removed during maintenance. This effect can only be expressed in the Strickler formula by adjusting the Strickler coefficient to a lower value;
- channel irregularity, comprising abrupt variations in cross sections along the channel length, by e.g. sand bars, obstructions, holes in the channel bed;
- channel alignment, meandering streams may decrease the Strickler coefficient \( k \) with up to 30%. However, the effect of the curvation is influenced by the flow velocity.

Design Criteria. It is not simple to determine the 'correct' Strickler coefficient \( k \). Different engineers will obtain different results.

Therefore it is essential that the value of the Strickler coefficient after it has been determined, is well communicated with others and that other engineers may present their comment at an early stage, before extension design work is done. Often the best procedure will be to present and discuss a special report 'design criteria' ('"ontwerpdoorschriften"'), where the procedure for design is well explained.

Values. The Strickler coefficient \( k \) of drainage channels is for a major part dependent on the vegetation in the channel, which is determined by the maintenance ('"onderhoud"'), the
season, the soil and the water depth. Often a Strickler coefficient \( k = 40 \text{ m}^{1/3}/\text{s} \) is selected for channels with frequent and good maintenance. But normally a Strickler coefficient \( k = 30 \text{ m}^{1/3}/\text{s} \) is selected for drainage channels as maintenance will be less frequent. A channel full with weeds ("planten") may have Strickler coefficients as low as \( k = 5 - 10 \text{ m}^{1/3}/\text{s} \). Some values are presented in table 2.1.

Table 2.1. Strickler coefficients \( k \) in \( \text{m}^{1/3}/\text{s} \).

<table>
<thead>
<tr>
<th>Description of channel</th>
<th>STRAIGHT ALIGNMENT</th>
<th>MEANDERING CHANNELS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( k ) ( \text{m}^{1/3}/\text{s} )</td>
<td>( k ) ( \text{m}^{1/3}/\text{s} )</td>
</tr>
<tr>
<td>LINED CHANNELS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>100</td>
<td>75 - 110</td>
</tr>
<tr>
<td>Brickwork</td>
<td>65</td>
<td>55 - 80</td>
</tr>
<tr>
<td>Concrete</td>
<td>75</td>
<td>65 - 90</td>
</tr>
<tr>
<td>EXCAVATED CHANNEL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>clean, short grass</td>
<td>40</td>
<td>25 - 45</td>
</tr>
<tr>
<td>some weeds</td>
<td>30</td>
<td>20 - 35</td>
</tr>
<tr>
<td>brushwoods</td>
<td>20</td>
<td>10 - 20</td>
</tr>
<tr>
<td>standing timber</td>
<td>15</td>
<td>5 - 15</td>
</tr>
<tr>
<td>NATURAL CHANNELS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>clean, short grass</td>
<td>35</td>
<td>25 - 45</td>
</tr>
<tr>
<td>some weeds</td>
<td>30</td>
<td>20 - 35</td>
</tr>
<tr>
<td>brushwoods</td>
<td>15</td>
<td>10 - 20</td>
</tr>
<tr>
<td>standing timber</td>
<td>10</td>
<td>5 - 15</td>
</tr>
<tr>
<td>FLOOD PLAINS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>short grass</td>
<td>30</td>
<td>20 - 35</td>
</tr>
<tr>
<td>crops</td>
<td>20</td>
<td>15 - 25</td>
</tr>
<tr>
<td>brushwoods</td>
<td>15</td>
<td>10 - 20</td>
</tr>
<tr>
<td>heavy timber</td>
<td>10</td>
<td>5 - 15</td>
</tr>
</tbody>
</table>

Programme 'Profile'. The computation with the Strickler formula is somewhat cumbersome: the discharge \( Q \) can be calculated directly when the other parameters are known. But, the water depth \( y \) or the bed width \( b \) can only be calculated by iteration. The formula is easily programmable. The PC-programme 'Profile' can be downloaded from the internet site of the Section Water Management (http://www.landandwater.tudelft.nl). The programme is 'public domain', and can be copied freely. The output of the programme 'Profile' should be printed as a 'File'. This file has to be entered as an "ASCII-text file" into e.g. Word. A sample of the output is shown in box 2.1.

Box 2.1. Results from the computer programme 'Profile'.

<table>
<thead>
<tr>
<th></th>
<th>DATE : 07-13-2002</th>
</tr>
</thead>
<tbody>
<tr>
<td>FILENAME :</td>
<td></td>
</tr>
<tr>
<td>( Q \text{ m}^3/\text{s} )</td>
<td>( h \text{ m} )</td>
</tr>
<tr>
<td>25.00</td>
<td>1.15</td>
</tr>
<tr>
<td>25.00</td>
<td>0.98</td>
</tr>
<tr>
<td>25.00</td>
<td>0.86</td>
</tr>
<tr>
<td>150.00</td>
<td>2.76</td>
</tr>
<tr>
<td>150.00</td>
<td>2.46</td>
</tr>
<tr>
<td>150.00</td>
<td>2.46</td>
</tr>
</tbody>
</table>
Composite sections. Above, trapezoid-shaped channels have been discussed. Flow through 'composite sections' ("samengestelde doorsneden") is also possible. These channels with composite sections can be divided into, see figure 2.2:

- **channels with 'berms'** ("bermen") are characterized that the berms run parallel to the main channel. Thus in the hydraulic calculations, the berms have the same gradient $S_1$ as the gradient $S_1$ of the main channel;
- **channels with 'floodplains'** ("uitterwaarden") are characterized that the main channel is meandering between the embankments. Thus in the hydraulic calculations, the floodplains have a steeper gradient $S_2$ as the gradient $S_1$ of the main channel.

![Composite cross-sections with berms and with forelands.](image)

**Figure 2.2.** Composite cross-sections with berms and with forelands.

Flow over berms and floodplains. The flow through the main channel is normally higher than the flow over 'forelands' ("buitendijksland"), i.e. the berms and the floodplains, because:
- the main channel is hydraulically smoother as there is less vegetation, so the Strickler coefficient $k_{\text{channel}} > k_{\text{forelands}}$;
- the main channel has a deeper water depth, so the water depth $y_{\text{channel}} > y_{\text{forelands}}$;
- the hydraulic radius of the main channel is reduced by a smaller wetted perimeter ("natte omtrek").

On the other hand, the bed width $b_{\text{channel}}$ of the main channel is often smaller than the width $b_{\text{forelands}}$ of the forelands, so that $b_{\text{channel}} < b_{\text{forelands}}$.

**Strickler formula twice.** Flow through channels with forelands ("buitendijks land") can be approximated by applying the Strickler formula twice:

$$Q = k_{\text{channel}} \left( \frac{\{b_{\text{channel}} + m y_o\} + (y - y_o) (b_{\text{channel}} + 2 m y_o)}{[b_{\text{channel}} + 2 y_o \sqrt{1 + m^2}]^{2/3}} \right)^{5/3} \{S_{\text{channel}} \}^{1/2} +$$

$$k_{\text{forelands}} \left( \frac{(y - y_o) \times [b_{\text{forelands}} + m (y - y_o)]} {b_{\text{forelands}} + 2 (y - y_o) \sqrt{1 + m^2}} \right)^{5/3} \{S_{\text{forelands}} \}^{1/2}$$

where: $Q$ is the total discharge through the channel and over the forelands in $m^3/s$, $b_{\text{channel}}$ is the bed width of the channel in $m$, $b_{\text{forelands}}$ is the width of the forelands, $y_o$ is the depth of the channel bed below the forelands in $m$, $y$ is the water depth above the channel bed in $m$, $m$ is the side slope ($1_{\text{Vert}} : m_{\text{Hor}}$) of the channel banks and of the embankments, $k_{\text{channel}}$ is the Strickler coefficient of the channel in $m^{1/3}/s$; $k_{\text{forelands}}$ is the Strickler coefficient of the forelands in $m^{1/3}/s$.

The gradient $s_{\text{channel}}$ of the channel equals the gradient $s_{\text{forelands}}$ in case of berms which run parallel with the channel.

The gradient $s_{\text{channel}}$ of the channel is less than the gradient $s_{\text{forelands}}$ in case of the channel is meandering through the floodplains. It is obvious that the calculation of the latter case is subject to many approximations which makes the calculation less accurate.
Hydraulic exponent $N$. The Strickler formula can also be re-written into (e.g. Chow 1959):

$$Q = c \cdot y^N$$

where: $Q$ is the discharge in m$^3$/s, $c = k \cdot s^{1/2}$ is a coefficient in m$^{1/3}$/s, $y$ is the water depth in m, and $N$ is the hydraulic exponent.

The hydraulic exponent $N$ depends on the water depth $y$, so that the above equation can only be applied for specific calculations. Examples are the 'sensitivity' at a proportional distribution, with small values of $\Delta y$.

The value of the hydraulic exponent $N$ depends on the side slope $1_{\text{Vert}}: m_{\text{Hor}}$ and on the width-to-depth ratio $n = b/y$ between the bed width $b$ and the water depth $y$, and follows from, see box 2.2:

$$N = \frac{5}{3} \left( \frac{n + 2m}{n + m} - \frac{4}{3} \frac{\sqrt{1 + m^2}}{n + 2\sqrt{1 + m^2}} \right)$$

Typical, the hydraulic exponents $N$ range from:

- $N = 2.01$ for small channels with side slopes $m = 1$ and the width-to-depth ratio $n = 1$,
- $N = 1.95$ for channels with side slopes $m = 1.5$ and the width-to-depth ratio $n = 2$,
- $N = 1.83$ for larger channels with side slopes $m = 2$ and the width-to-depth ratio $n = 5$.

**Box 2.2. Derivation of the hydraulic exponent $N$ (see also Chow 1959).**

The Strickler formula $Q = k \cdot A \cdot R^{2/3} \cdot s^{1/2}$ can be re-written as a function of the water depth $y$ (Chow 1959): $Q = c \cdot y^N$, where $N$ is the 'hydraulic exponent'.

- **Processing of the equation with the hydraulic exponent:**
  Taking the logarithm on both sides of the $Q = c \cdot y^N$ gives: $\ln Q = \ln c + N \ln y$,
  and differentiating with respect to $y$, leads to:
  $$\frac{d}{dy} (\ln Q) = \frac{5}{3} \frac{d}{dy} (\ln y)$$
  and so:
  $$\frac{d}{dy} (\ln Q) = \frac{N}{y}.$$  

- **Processing of the Strickler equation:**
  The hydraulic radius $R$ in the Strickler formula can be expressed in $R = A / P$,
  where $P$ is the perimeter of the wet cross-section, so $Q = k \cdot A^{5/3} \cdot P^{-2/3} \cdot s^{1/2}$.
  Taking the logarithm on both sides of the Strickler formula gives:
  $$\ln Q = \ln (k \cdot s^{1/2}) + 5/3 \ln A - 2/3 \ln P$$
  and differentiating with respect to $y$, leads to:
  $$\frac{d}{dy} (\ln Q) = \frac{5}{3} \frac{d}{dy} (\ln A) - \frac{2}{3} \frac{d}{dy} (\ln P)$$
  Furthermore:
  $$A = b \cdot y + m \cdot y^2,$$  
  so that:
  $$\frac{dA}{dy} = \frac{d(b \cdot y + m \cdot y^2)}{dy} = b + 2my$$
  $$P = b + 2y\sqrt{1 + m^2},$$  
  so that:
  $$\frac{dP}{dy} = \frac{d(b + 2y\sqrt{1 + m^2})}{dy} = 2\sqrt{1 + m^2}$$
  and finally:
  $$\frac{d}{dy} (\ln Q) = \frac{5}{3} \frac{b + 2my}{by + my^2} - \frac{2}{3} \frac{2\sqrt{1 + m^2}}{b + 2y\sqrt{1 + m^2}}.$$  

- **Solving $N$ from the two equations:**
  $$N = \frac{5}{3} \left( \frac{b + 2m}{y} - \frac{4}{3} \frac{\sqrt{1 + m^2}}{y + 2\sqrt{1 + m^2}} \right)$$
  $$= \frac{5}{3} \left( \frac{n + 2m}{n + m} - \frac{4}{3} \frac{\sqrt{1 + m^2}}{n + 2\sqrt{1 + m^2}} \right)$$
2.1.2. Bankfull Capacity

Natural levees. Figure 2.3 shows the cross section of a river reach in a delta. The capacity of natural rivers in deltaic areas is limited, and the higher discharges will overtop the river banks. During these overtoppings, the heavier sediments, such as sand and silt, are deposited first on the riverbanks, i.e. on the 'natural levees' ("oeverwallen"), and the finer particles, clay, are transported further to the 'backswamps' ("komgronden").

The relative height of natural levees in the upstream parts of a delta may be as much as 2.00 - 3.00 m, and the width as much as 2 - 3 km. Natural levees offer the best possibilities for human settlement, as they are better drained than the backswamps. Also, the depth of flooding is smaller than in the backswamps. Even in developed and protected deltas, most of the population is still living on these ridges.

Backswamps consist of heavy soils with a low permeability. They are often waterlogged, even during dry periods. In some deltas, the formation of peat takes place in the backswamps.

![Figure 2.3. Natural river with 'natural levees' during overtopping.](image)

Bankfull capacity. The discharge which just fills the channel is often called the 'bankfull discharge' ("kapaciteit van het rivierbed").

The 'bankfull capacity' or the 'bankfull discharge' of a channel is defined as the discharge corresponding to the state at which the river berms are just about to be submerged (ICID 1967).

Field data show that the crest of the natural levee is raised to the level of a flood with a return period ("herhalingstijd") between 0.5 to 1.5 years, depending on the nature and the duration of the floods. Floods with return periods of 10 to 100 years are too rare to build up the levee to that elevation. It means that the 'bankfull discharge' of a natural river reach may equal to the 0.5 to 1.5 years flood (Henderson 1966).

Calculation of the bankfull capacity. The calculation of the bankfull capacity of a cross-section can be done with the Strickler formula for a known Strickler coefficient $k$ in m$^{1/3}$/s. The following procedure may be followed, see figure 2.4:

- determine the gradient of the channel from the longitudinal section. Therefore, draw a straight line through the lowest points of the bed, i.e. the 'thalweg' ("talweg"). Read the gradient $s$ from this line;
- schematize the bottom of the channel-bed by drawing a horizontal line in the cross-section. Read the bed width $b$ from this bottom-line;
- draw the maximum water level in the cross-section at which 'bank-overtopping' begins.
Read the maximum water depth $y$ between the 'bottom-line' and this **water-line**;
- make a schematization of the side slopes, and draw **two side-lines** of the channel, starting from the bottom-line with width $b$. Read the width $w$ of the water-line between these side-lines. Calculate the side slope $m$ ($m_{vert}$ : $m_{hor}$) from the equation:

\[ w = b + 2my, \]  
\[ m = \frac{w - b}{2y}. \]

**Figure 2.4.** Determination of the 'bankfull capacity'.

### 2.1.3. Non-uniform flow

**Types of backwater curves.** The water levels in drainage channels and in rivers often form backwater curves (**"stuwkrommen"**). The water levels may form either, see figure 2.5:
- 'positive' backwater curves, when there is a backing up (**"opstuwning"**) of the water level,
- 'negative' backwater curves, when there is a drawing down (**"afzuiging"**) of the water level.

**Figure 2.5.** Positive and negative backwater curves.
Energy depth. Backwater curves are determined by the energy depth at the 'tail end' or 'downstream end' of a channel reach ("pand"). This 'downstream end' can be formed by:

- the **control section** ("stuwgedeelte") of a structure, such as a 'fixed weir' ("vaste stuw"), a 'control notch' ("vernaaiwing"), a screen ("vast scherm"), or a 'gate' ("schuif", "deur"), see figure 2.6;
- the **downstream canal reach**. In the following, the conditions at the downstream canal reach are also called: 'control'.

![Diagram of various control sections](image)

**Figure 2.6.** The 'control section' of a structure determines the energy head $H_{\text{control}}$.

**Backing up / drawing down?** The water line in a reach forms a curved line for non-uniform flow:

- there is a **backing up** ("opstuwwing") when this energy depth $H_{\text{control}}$ at the control above the bed level is more than the uniform energy depth $H_{\text{uniform}}$, thus when $\Delta H = H_{\text{control}} - H_{\text{uniform}}$ is positive.
- there is a **drawing down** ("afzuiging") when $\Delta H = H_{\text{control}} - H_{\text{uniform}}$ is negative. Obviously, there is **uniform flow** for $\Delta H = H_{\text{control}} - H_{\text{uniform}} = 0$.

**Use of programmable pocket calculator.** The calculation of a backwater curve by hand is a laborious matter, specifically when the cross section is trapezoid.

It is more practical to prepare a computer programme. Such a programme can be based on the 'standard step' method, and will discussed below.

**Parameters.** The water depth $y_L$ at a location $x = L$ from a 'control' is determined by the following parameters, see figure 2.7:

- the energy depth $H_c$ in m at the control,
- the discharge $Q$ in m$^3$/s,
- the characteristics of the channel reach, i.e. the bed gradient $S$, the bed width $b$ in m, the side slope $m$ ($1_{\text{vert}} : m_{\text{hor}}$), and the Strickler coefficient $k$ in m$^{1/3}$/s.
Figure 2.7. Parameters for the calculation of backwater curves.

**Vertical 1.** The calculation is done step-by-step from vertical 1 to vertical 2, see figure 2.8. Subsequently, vertical 2 becomes the new vertical 1, and the next vertical 2 is calculated, etc.

The first vertical 1 is taken just upstream of the 'control'. There is no friction loss between the 'control' and vertical 1, so \( H_1 = H_C \).

The water depth \( y_1 \), the velocity \( v_1 \) and the gradient \( s \) of the energy line have to be calculated at vertical 1:

- **known are:** the discharge \( Q \) in \( \text{m}^3/\text{s} \), the bed width \( b \) in \( \text{m} \), and the side slope \( m \) (\( 1_{\text{Vert}} : m_{\text{Hor}} \)). Thus, water depth \( y_1 \) and the velocity \( v_1 \) at vertical 1 can be calculated from the two equations through iteration:

\[
H_1 = y_1 + \frac{v_1^2}{2g}
\]

\[
v_1 = \frac{Q}{A}, \quad \text{where:} \quad A = (b + my_1) y_1
\]

The iteration is done as follows:
- assume \( v_1 = 0 \), calculate \( y_1 = H_1 - \frac{v_1^2}{2g} \) and \( v_1 = \frac{Q}{[(b + my_1) y_1]} \).
- check the 'new' \( v_1 \) with the 'assumed' \( v_1 \). If there is still a difference, repeat the iteration with the new \( v_1 \). Otherwise, the value of the velocity \( v_1 \) and the value of the water depth \( y_1 \) have been found;

- **known are:** the velocity \( v_1 \) in \( \text{m/s} \), the water depth \( y_1 \) in \( \text{m} \), the bed width \( b \) in \( \text{m} \), the side slope \( m \) (\( 1_{\text{Vert}} : m_{\text{Hor}} \)) and the Strickler coefficient \( k \) in \( \text{m}^{1/3}/\text{s} \). Thus, gradient \( s_1 \) of the water line at vertical 1 can be calculated from the Strickler equation:

\[
s_1 = \frac{v_1^2}{k^2 R^{4/3}}, \quad \text{where:} \quad R = \frac{(b + my_1) y_1}{b + 2y_1 \sqrt{1 + m^2}}
\]

**Difference in energy head.** Vertical 2 is defined on basis of the change \( \Delta H \) in the energy depth, and on the shape of the backwater curve, see also figure 2.8:
- for a **backing up** ("opstuwving") when the energy depth \( H_{\text{control}} \) at the control is larger than the uniform energy depth \( H_{\text{uniform}} \): \( H_2 = H_1 + \Delta H \), with \( \Delta H < 0 \).
- for a **drawing down** ("afzuiging") when the energy depth \( H_{\text{control}} \) at the control is smaller than the uniform energy depth \( H_{\text{uniform}} \): \( H_2 = H_1 + \Delta H \), with \( \Delta H > 0 \).

The value of difference \( \Delta H \) is often taken at \( \Delta H = -0.01 \text{ m} \) for backing up, and \( \Delta H = +0.01 \text{ m} \) for drawing down.
**Vertical 2.** As the new $H_2$ at vertical 2 is known now, the water depth $y_2$, the velocity $v_2$ and the gradient $s_2$ of the energy line can be calculated accordingly to the above calculations.

**Mutual distance.** The mutual distance $\Delta x$ between vertical 1 and vertical 2 follows from the standard-step equation (e.g. Chow 1959):

\[
\Delta x = \frac{\Delta H}{\frac{s_1 + s_2}{2} - S}
\]

where: $\Delta x$ mutual distance in m between vertical 1 and vertical 2, $\Delta H = H_2 - H_1$ in m, $s_1$ is the energy gradient at vertical 1, $s_2$ is the energy gradient at vertical 2, and $S$ is the gradient of the channel bed.

**Distance from the control.** The total distance from the 'control' at $x = 0$ can be calculated by, see figure 2.9:

\[
x_2 = x_1 + \Delta x, \quad \text{or in general:} \quad x_2 = \Sigma x + \Delta x
\]

Check the calculated distance $x_2$ with the required distance $L$ to the control:

- when $L > x_2$, calculate next vertical, by considering the vertical 2 becomes the new vertical 1;
- when $L < x_2$, the calculated water depth $y_2$ is the required water depth $y_L = y_2$ at the distance $x = L$ from the control.

An example of a backwater calculation is shown in box 2.3.
**Figure 2.9.** Calculation of total distance from the 'control'.

**Box 2.3. Example of a backwater curve.**

A lined urban drainage channel is constructed with the following characteristics: bed width $b = 6.00$ m, side slope $m = 1 \ (1_{\text{Hor}} : 1_{\text{Vert}})$, bed gradient $S = 0.24 \times 10^{-3}$, and a Strickler roughness coefficient $k = 55 \ \text{m}^{1/3}/\text{s}$.

The channel is designed for a normal discharge of $Q = 11.43 \ \text{m}^3/\text{s}$. A corresponding water depth of $y = 1.60$ m and velocity $v = 0.93 \ \text{m/s}$ can be calculated with the Strickler formula.

The channel has been equipped with an automatic water level regulator (gate) so the water level of $y = 1.60$ m is always maintained at that location. The bed level, at this regulator, is at 19.84 m+Datum.

A computer programme may calculate a backwater curve for a discharge of $Q = 5 \ \text{m}^3/\text{s}$, at the following distances from the regulator:

<table>
<thead>
<tr>
<th>&quot;x&quot; in m</th>
<th>waterdepth in m</th>
<th>velocity in m/s</th>
<th>tractive force in N/m²</th>
<th>bedlevel m+Datum</th>
<th>waterlevel m+Datum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>0.75</td>
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<td>21.43</td>
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<td>0.55</td>
<td>1.35</td>
<td>20.32</td>
<td>21.57</td>
</tr>
</tbody>
</table>
2.2. Flow of Sediment

2.2.1. Sediment

Sediment transport. It is not only that the 'flow of water' through a channel should be guaranteed, but also the 'flow of sediment' must be sufficient.

The main rule for the design of stable channels is that: "all sediment which enters the channel reach should be transported through without sedimentation".

Thus, the 'relative transport capacity' is of importance. When the relative transport capacity reduces, it will lead to sedimentation.

Classification of Sediment. A classification of sediment transport can be made on basis of the transport mechanism of the transported material, see figure 2.10 (e.g. Jansen 1979):

- 'Bedload' ("bodem transport") is the movement of particles in contact with the bed by rolling, sliding and jumping;
- 'Suspended-load' ("zwevend transport") is the movement of particles that has no contact with the bed, and the weight of the particles is continuously compensated by the turbulent action of the water.

Also a classification of sediment transport can be made on basis of the origin of the transported material:

- 'Bed-material load' ("bodem-materiaal transport") has its origin in the bed, which means that the transport is determined by the bed and flow conditions;
- 'Wash load' ("spoeltransport") has its origin outside the bed and is supplied by erosion in the catchment area. This material has no direct relation with the bed-material and is generally fine material ($D < 0.060\ mm$).

Importance of washload. Typical figures from deltaic areas in Indonesia, Nigeria and the Netherlands show that only a small portion ($\pm 10\%$) of the annual sediment transport is transported as 'bedload', and the remainder as 'washload'.

![Figure 2.10. Sediment transport classification.](image)

Concentration. The concentration of suspended-load in a sediment-water mixture is usually expressed in 'concentration'. However, the concentration can be expressed in various ways, of which the most commonly used are:

- concentration by weight, that is the dry weight of solids per unit weight of mixture, expressed in kg/m$^3$ or mg/l and also often in ppm;
- concentration by volume, the absolute volume of solids per unit volume of mixture, expressed in ppm.
Ppm. It is customary to express the sediment concentration in a concentration by weight in mg/l (=ton/m³) or in 'ppm' (parts per million). One per cent (1%) equals 10,000 ppm.

The conversion of 'mg/l' to 'ppm' assumes that 1 ppm equals 1 milligram sediment per 1 liter of water, i.e. 1 mg sediment per 1000 g water, so 1 mg/l = 1 ppm.

Fall velocity. The fall velocity of a sediment particle is an important parameter in studies on suspension of sediments. The fall velocity \( w \) in m/s can be calculated with the Rubey formula for a sediment diameter \( D_m < 0.2 \) mm (e.g. Jansen 1979):

\[
\begin{align*}
    w &= \frac{1}{18} \frac{g \rho_s - \rho_w}{\rho_w} \frac{D_m^2}{v} \\
    \text{with: } v &= \frac{40 \times 10^{-6}}{20 + t}
\end{align*}
\]

where \( w \) is the fall velocity of sediment particles in m/s, \( D_m \) is the diameter of the particles in m, \( \rho_w = 1000 \text{ kg/m}^3 \) is the density of clear water, \( \rho_s = 2600 \text{ kg/m}^3 \) is the density of sediment, \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity, \( v \) is the kinematic viscosity, and \( t \) is the temperature in °C.

The Rubey formula becomes for a water temperature \( t = 20 \) °C:

\[
    w = 0.87 \times 10^6 D_m^2
\]

A warning should be given on the use of this formula, as many factors such as the shape of the particles, make that this formula is quite inaccurate!

Dry density. Sometimes, it is necessary to convert the weight of the sediment into a volume. This may happen to estimate the life of a reservoir. Thus, the 'dry density' must be known.

The dry density \( \rho \) is the mass of dry sediment per unit volume, in kg/m³. It depends on the grain size, the condition of settling and on the time. An empirical relation for estimating the dry density of deposits in a reservoir follows from the literature:

\[
    \rho_T = \rho_o + B \log T
\]

where: \( \rho_T \) is the dry density after \( T \) years in kg/m³, \( \rho_o \) is the initial dry density (for \( T = 1 \) year) in kg/m³, \( B \) is the consolidation coefficient, see table 2.2, and \( T \) is the consolidation time in years.

Table 2.2. Dry density of sediment deposits.

<table>
<thead>
<tr>
<th>Submerging of the Sediment Deposits</th>
<th>Dry density at ( T = 1 ) year</th>
<th>Consolidation coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>sand/kg/m³</td>
<td>silt/kg/m³</td>
</tr>
<tr>
<td>always submerged</td>
<td>1500</td>
<td>1050</td>
</tr>
<tr>
<td>often submerged</td>
<td>1500</td>
<td>1185</td>
</tr>
<tr>
<td>normally submerged</td>
<td>1500</td>
<td>1275</td>
</tr>
</tbody>
</table>

2.2.2. Bedload transport capacity

Einstein-Brown formula per unit width. The Einstein-Brown formula has been derived empirically for bedload transport, although it might also be valid for suspended-load. The formula reads (e.g. Henderson 1966, Raudkivi 1993):

\[
    \frac{q_s}{G \sqrt{g D^3 \left( \frac{\rho_s}{\rho} - 1 \right)}} = 40 \left[ \frac{R s}{\left( \frac{\rho_s}{\rho} - 1 \right) D} \right]^3
\]

or:
\[ q_s = \frac{40}{\rho_s} \left( \frac{\rho}{\rho_s} - 1 \right)^{\frac{5}{2}} \left( \frac{g}{D} \right)^{\frac{1}{2}} \left( \frac{R^3 s^3}{D^2} \right) = 96 \frac{g}{D} \left( \frac{R^3 s^3}{D^2} \right) \]

where: \( q_s \) is the sediment transport per unit width in \( \text{m}^2/\text{s} \), \( R \) is the hydraulic radius of the cross-section in \( \text{m} \), \( s \) is the gradient of the channel, \( G = 2/3 \) is the particle fall function, \( D \) is the grain size of sediment in \( \text{m} \), \( \rho_s = 2600 \text{ kg/m}^3 \) is the density of the sediment in \( \text{kg/m}^3 \), \( \rho = 1000 \text{ kg/m}^3 \) is the density of water, \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity.

Thus, the ‘relation’ between the sediment transport \( q_s \) per unit width, the hydraulic radius \( R \) and the gradient of the channel is:

\[ q_s \sim R^3 s^3. \]

**Einstein-Brown formula for the whole cross-section.** The Einstein-Brown formula can be transformed into the bedload transport \( Q_s \) for the whole cross-section by multiplying with the width \( b \). Thus, the total bedload transport \( Q_s \) in \( \text{m}^3/\text{s} \) becomes according to the Einstein-Brown equation:

\[ Q_s \sim b R^3 s^3. \]

**Box 2.4. Further processing of the Einstein-Brown formula.**

There are two formulae important for the ‘relative transport capacity’ \( Q_s/Q \) of the bedload through a cross-section:

- the **Einstein-Brown formula** for the total bedload transport \( Q_s \) in \( \text{m}^3/\text{s} \) reads:
  \[ Q_s \sim b R^3 s^3, \]

- the **Strickler formula** for the flow of water reads:
  \[ Q \sim A R^{2/3} s^{1/2}. \]

Thus, the ‘relative transport capacity’ \( S/Q \) of the bedload becomes:

\[ \frac{Q_s}{Q} \sim \frac{b R^3 s^3}{A R^{2/3} s^{1/2}}, \]

and so:

\[ \frac{Q_s}{Q} \sim \frac{b}{A} R^{7/3} s^{5/2}, \]

where \( Q_s \) is the bedload transport per cross-section in \( \text{m}^3/\text{s} \), \( Q \) is the discharge of water in \( \text{m}^3/\text{s} \), \( b \) is the bed width in \( \text{m} \), \( A \) is the wet cross-sectional area in \( \text{m}^2 \), \( R \) is the hydraulic radius in \( \text{m} \), and \( s \) is the gradient of the channel.

The hydraulic radius \( R \) for **wide channels** becomes equal to the water depth \( y \), which means that the hydraulic radius becomes: \( R \approx A/b \). For **narrow channels** of a bed width \( b = 30 \text{ m} \), side slopes \( m = 2 \) and a water depth \( y = 2 \text{ m} \) to \( 5 \text{ m} \), the hydraulic radius \( R \) can be calculated at \( R \approx (A/b)^{9/12} \). However, the hydraulic radius \( R \) is assumed here at \( R = (A/b)^{9/11} \) in order to obtain a ‘practical’ relation with the velocity and the gradient. So:

\[ \frac{Q_s}{Q} \sim \frac{b}{A} R^{7/2} s^{5/2} = R^{10/2} s^{5/2} \]

The velocity \( v \) follows from the Strickler formula \( v \sim R^{2/3} s^{1/2} \), so that the hydraulic radius can be substituted in the above formula through \( R \sim v^{2/3} s^{-3/4} \):

\[ \frac{Q_s}{Q} \sim \frac{10}{9} \frac{5}{2} \frac{5}{2} = \left( \frac{3}{v^2 s^4} \right)^{10/3} \frac{5}{2} = \left( \frac{10}{v^6 s^6} \right)^{5/6} = \frac{10}{v^6} s^6. \]

It means that the ‘relative transport capacity’ \( Q_s/Q \) of the bedload through a cross-section is related to the velocity \( v \) in \( \text{m/s} \) and the gradient \( s \) by means of:

\[ \frac{Q_s}{Q} \sim v^{10/6} s^{10/6}. \]
Relative transport capacity. Channels should be designed in such a way that all bedload which enters the channel reach should be transported toward the end without sedimentation. Thus, the 'relative transport capacity' \( Q_s/Q \), which is the total bedload transport \( Q_s \) per total discharge \( Q \), is of importance. When the relative transport capacity \( Q_s/Q \) reduces, it will lead to sedimentation.

Further processing of the Einstein-Brown formula, see box 2.4, leads to the relation between the 'relative transport capacity' \( Q_s/Q \) of the bedload through a cross-section, the velocity \( v \) in m/s and the gradient \( s \) by means of:

\[
\frac{Q_s}{Q} \sim v^{10/6} s^{10/6}
\]

It means that the factor \( v \times s \) should not decrease in downstream direction to prevent sediment deposits. It will be shown below that the criterium for bedload transport is equal to the criterium for the transport of washload.

### 2.2.3. Suspended-load transport capacity

Suspension as an energy problem. Studying the suspended-load at different depths of rivers and canals, it was found that particles smaller than 0.06 mm generally are uniformly distributed over the water depth (Vlugter 1962). It would appear that these particles are not 'sinking' but are 'floating'.

Several researchers (deVos 1925, Vlugter 1941 and 1962, Bagnold in 1956, Wang in 1959, Schoemaker 1983) considered suspended-load transport as a problem of energy. They considered that "the fluid must lift the solids at the rate that they are falling under gravity".

The most systematic derivation of the energy equation has been done by Vlugter (Vlugter 1941), and is followed here. He compared the flow of water without sediment, 'clear water', with the flow of a water - sediment mixture, see figure 2.11. Through an energy balance, the criteria for suspension are finally reached.

![Diagram of energy balance](image)

**Figure 2.11.** Loss of potential energy by water 'without' and 'with' sediment.
Loss of potential energy in clear water. When 'clear water' is flowing with a velocity \( v_0 \), it will lose potential energy. The loss of potential energy, in Watt or in Nm/s, by flowing clear water with a volume \( V_w \) in m\(^3\) depends on the drop in vertical distance \( "v_0 \sin \alpha" \), and reads:
\[
\rho_w \ g \ V_w \ v_0 \sin \alpha
\]
where: \( \rho_w = 1000 \text{ kg/m}^3 \) is the density of clear water, \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity, \( V_w \) is the volume of water in the channel in m\(^3\), \( \alpha \) is the gradient of the channel in radians, \( v_0 \) is the velocity of the clear water in m/s.

This loss of potential energy equals the losses by friction at the bed and the side slopes. It means that the friction losses of clear water, in Watt or in Nm/s, can be calculated from:
\[
\rho_w \ g \ V_w \ v_0 \sin \alpha
\]

Water - sediment mixture. When 'sediment-loaded water' is flowing with a (higher) velocity \( v_m \), there are three energy components, in Watt or in Nm/s:
- the potential energy that comes available by the drop in vertical distance \( "v_m \sin \alpha" \):
  \[
  \rho_w \ g \ V_w \ v_m \sin \alpha + \rho_s \ g \ V_s \ v_m \sin \alpha
  \]
- the energy needed for suspension equals the energy that comes available for 'sinking' sediment, thus:
  \[
  (\rho_s - \rho_w) \ g \ V_s \ w
  \]
- the friction loss of the water - sediment mixture is higher than the friction loss of clear water: \( \rho_w \ g \ V_w \ v_0 \sin \alpha \), because of the higher water velocity \( v_m > v_0 \).

The friction losses are related to 'square' of the velocity (conform the Strickler formula \( v \approx s^{1/2} \)), so the higher water velocity \( v_m \) of the water - sediment mixture makes that the friction losses increases by a factor " \( (v_m / v_0)^2 \) " , so:
\[
\rho_w \ g \ V_w \ v_0 \sin \alpha \times \frac{(v_m / v_0)^2}{(\rho_s - \rho_w) \ g \ V_s \ w + \rho_w \ g \ V_w \ v_0 \sin \alpha \ (v_m / v_0)^2}
\]
where: \( \rho_w = 1000 \text{ kg/m}^3 \) is the density of clear water, \( \rho_s = 2600 \text{ kg/m}^3 \) is the density of sediment, \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity, \( V_w \) is the volume of water in the channel in m\(^3\), \( V_s \) is the volume of sediment in the water in m\(^3\), \( w \) is the fall velocity of the sediment in m/s, \( v_0 \) is the velocity of the clear water in m/s, \( v_m \) is the velocity of the water - sediment mixture in m/s, \( \alpha \) is the gradient of the channel in radians.

Energy balance for water - sediment mixture. The energy balance for the water - sediment mixture reads:
- Potential energy = energy needed for suspension + friction loss
  \[
  \rho_w \ g \ V_w \ v_m \sin \alpha + \rho_s \ g \ V_s \ v_m \sin \alpha = \rho_s - \rho_w \ g \ V_s \ w + \rho_w \ g \ V_w \ v_0 \sin \alpha \ (v_m / v_0)^2
  \]
A further processing of this equation is shown in box 2.5, and leads to the formula of Vlugter for the concentration \( C \) by weight:
\[
C = \frac{\frac{v_m}{v_o} - 1}{1 - \frac{\Delta_s \ w}{v_m \sin \alpha}}
\]
where: \( w \) is the fall velocity of the sediment in m/s, \( v_o \) is the velocity of the clear water in m/s, \( v_m \) is the velocity of the water - sediment mixture in m/s, \( \alpha \) is the gradient of the channel in radians, and \( \Delta_s = (\rho_s - \rho_w) / \rho_s = 0.6 \).
Box 2.5. Further processing of the energy balance.

The energy balance for the water - sediment mixture reads:

\[ \rho_w g V_w v_m \sin \alpha + \rho_s g V_s v_m \sin \alpha = \frac{(\rho_s - \rho_w) g V_s w}{1 + \frac{\rho_s V_s}{\rho_w V_w} - \frac{\rho_s - \rho_w}{\rho_w} \frac{V_s}{V_w} \frac{w}{v_m} \frac{\sin \alpha}{v_o} \frac{v_m}{v_o} (v_m / v_o)^2} \]

The balance is divided by \( \rho_w g V_w v_m \sin \alpha \):

\[ 1 + \frac{\rho_s V_s}{\rho_w V_w} = \frac{\rho_s - \rho_w}{\rho_w} \frac{V_s}{V_w} \frac{w}{v_m} \frac{\sin \alpha}{v_o} \frac{v_m}{v_o} \]

and the term \( \frac{\rho_s V_s}{\rho_w V_w} \) is brought to the left:

\[ \frac{\rho_s V_s}{\rho_w V_w} \left( 1 - \frac{\rho_s - \rho_w}{\rho_w} \frac{w}{v_m} \frac{\sin \alpha}{v_o} \frac{v_m}{v_o} \right) = \frac{v_m}{v_o} - 1 \]

As \( \Delta_s = (\rho_s - \rho_w) / \rho_s \) and as the term \( \frac{\rho_s V_s}{\rho_w V_w} \) equals the concentration \( C \) of the sediment by weight, finally the formula of Vlugter is reached:

\[ C = \frac{\frac{v_m}{v_o} - 1}{1 - \frac{\Delta_s w}{v_m \sin \alpha}} \]

Discussion on the formula of Vlugter. The meaning of the formula of Vlugter lies in the term:

\[ \frac{\Delta_s w}{v_m \sin \alpha} \]

There are three options for this term:

- when the term \( (\Delta_s w) / (v_m \sin \alpha) = 1 \), the concentration \( C \) becomes \( \infty \). Thus, the sediment with a fall velocity \( w \) remains in suspension, at any concentration \( C \). It means that the washload can be transported through the channel reach;
- when the term \( (\Delta_s w) / (v_m \sin \alpha) > 1 \), the concentration cannot exist and deposition of the sediment will happen;
- when the term \( (\Delta_s w) / (v_m \sin \alpha) < 1 \), the sediment with a fall velocity \( w \) remains in suspension, while even the value \( v_m \sin \alpha \) can be decreased. This leads to the 'de Vos & Vlugter criterium' for suspended sediment transport. This is in line with the results of Bagnold and Wang (e.g. Raudkivi 1976, Chien 1999).

Criterion of 'De Vos & Vlugter'. The above rule on suspended sediment transport was first found by De Vos in 1925 and later re-presented by Vlugter in 1941:

- sediment with a fall velocity \( w \) remains in suspension for: \( v \times \sin \alpha > \Delta_s \times w \)
- as the energy gradient \( s = \sin \alpha \) and \( \Delta_s = 0.6 \), the De Vos - Vlugter criterium becomes: "washload with fall velocity \( w \) remains in suspension for \( v s > 0.6 w \)" where \( v \) is the velocity of the water in m/s, \( s \) in the gradient, and \( w \) is the fall velocity of the sediment with a diameter \( D_m \) in m.

Criterion Schoemaker. Schoemaker presented an energy model in line with the above thoughts of de Vos and Vlugter, and derived the relation (Schoemaker 1983b):

\[ E = \rho g v s \]

where: \( E \) is the energy dissipation of the mixture in Watt/m³, \( \rho = 1000 \text{ kg/m}^3 \) is the density
of water, \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity, \( v \) is the velocity of the water in m/s, and \( s \) is the gradient of the channel.

Thus, the 'criterium of Schoemaker' reads: "the transport capacity for suspended sediment is guaranteed when the value of \( E = \rho g v s \) does not decrease in downstream direction".

### 2.2.4. Discussion on Sediment Transporting Capacity

**Relative transport capacity.** It has been discussed above, that the main rule for the design of stable channels is that: "all sediment which enters the channel reach should be transported through without sedimentation". Thus, the 'relative transport capacity' is of importance.

The 'relative transport capacity' \( Q_s/Q \) is the total bedload transport \( Q_s \) per total discharge \( Q \). When the relative transport capacity \( Q_s/Q \) reduces, it will lead to sedimentation.

**Criterium for bedload.** The relative transport capacity \( Q_s/Q \) of the bedload through a cross-section depends on the velocity \( v \) in m/s and the gradient \( s \) by means of (as derived above):

\[
\frac{Q_s}{Q} \sim v^{10} s^{10}
\]

It means that the factor "\( v \times s \)" should not decrease in downstream direction to prevent sediment deposits.

**Criterium for washload.** Also the relative transport capacity \( Q_s/Q \) of the washload through a cross-section depends on the velocity \( v \) in m/s and the gradient \( s \) by means of (as derived by Vlugter and Schoemaker):

\[
\frac{Q_s}{Q} \sim v s
\]

Also here, the factor "\( v \times s \)" should not decrease in downstream direction to prevent sediment deposits.

**Criterium of Schoemaker.** Apparently the 'criterium of Schoemaker' can be applied for the relative sediment transport capacity for both bedload as washload.

Thus, "the transport capacity of all sediment is guaranteed when the value of: \( E = \rho g v s \) does not decrease in downstream direction", where \( E \) is the energy dissipation of the mixture in Watt/m\(^3\), \( \rho = 1000 \text{ kg/m}^3 \) is the density of water, \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity, \( v \) is the water velocity in m/s, and \( s \) is the gradient of the channel.

**Workable form.** Sometimes, a more workable form of the criterium of Schoemaker is obtained by replacing the velocity \( v \) by the ratio "\( Q/A \)", so that:

\[
E = \rho g \frac{Q}{A} s
\]

where \( E \) is the energy dissipation of the mixture in Watt/m\(^3\), \( \rho = 1000 \text{ kg/m}^3 \) is the density of water, \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity, \( Q \) is the discharge in m\(^3\)/s, \( A \) is the wet cross-sectional area in m\(^2\), and \( s \) is the gradient of the channel.
Meaning for the cross-sectional area. The above relation can be made more operational for the design by considering the relation emerging from the Strickler formula:

\[ v \sim R^{2/3} s^{1/2} \]

so that:

\[ E \sim \rho g \times R^{2/3} s^{1/2} \times s \]

and finally the design condition for maintaining the 'relative transport capacity' \( E \) of the sediment through the alignment:

\[ R \sim s^{-9/4} \]

with the hydraulic radius \( R \) in m and the channel gradient \( s \). The meaning of this condition is that the hydraulic radius \( R \) of a sediment-loaded channel should increase for a decreasing gradient \( s \), see figure 2.12.

![Cross-Section Diagrams](image)

**Figure 2.12.** Changing cross sections for stable sediment transport at changing gradients.

---

### 2.3. Scouring and Sedimentation in Cross Sections

#### 2.3.1. Tractive Force

**Total tractive force on reach.** The 'total tractive force' ("totale schuifkracht") is the pull of flowing water on the wetted area of a reach of a channel ("kanaalpand"). In uniform flow, the tractive force is equal to the component of the gravity force parallel to the channel gradient. Thus, the tractive force on a channel reach follows from, see figure 2.13:

\[ F = \rho g A L s \]

where: \( F \) is the total tractive force on bed and slopes of the reach in N, \( \rho = 1000 \text{ kg/m}^3 \) is the density of water, \( g = 9.8 \text{ m/s}^2 \) is the acceleration of gravity, \( A \) is the wet cross sectional area in \( \text{m}^2 \), \( L \) is the length of the channel reach in m, and \( s \) is the (energy) gradient.
Figure 2.13. Distribution of the shear stress or 'tractive force'.

**Unit tractive force.** The 'unit tractive force' is the average value of the tractive force per unit wetted area. It is also called the 'average shear stress' ("gemiddelde schuifspanning").

The unit tractive force can be calculated from:

\[ T_o = \rho g A L s / (P L) = \rho g R s \]

where: \( T_o \) is the unit tractive force per unit wetted area in N/m², \( P \) is the wetted perimeter in m, \( R \) is the hydraulic radius in m.

The tractive force is not uniformly distributed along the wetted perimeter. Thus, this average value of the unit tractive force cannot be used in the design of stable channels, as the actual tractive force on the bed is higher than the average value.

**Distribution of unit tractive force.** USBR calculated in 1952 the distribution of the (unit) tractive force in trapezoidal cross sections through membrane analogy and analytical methods. The results are extensively copied in the literature (e.g. Chow 1959, Henderson 1966). A typical distribution over the bed and the slopes is shown in also figure 2.13.

**Peak value in the cross-section.** It has been found by USBR that the peak value of the unit tractive force is dependent on the ratio \( b/y \) and on the side slopes \( m \). The peak value of the tractive force at the bed \( T_{bed} \) and of the side \( T_{side} \) follows from the general formulae:

\[ T_{bed} = \alpha_{bed} \rho g y s \]
\[ T_{side} = \alpha_{side} \rho g y s \]

where \( \alpha_{bed} \) and \( \alpha_{side} \) are coefficients, see table 2.3. It can be seen that the coefficient \( \alpha_{bed} \) for the tractive force at the bed can be approximated by \( \alpha_{bed} \approx 1 \).

**Table 2.3. Coefficients for the peak tractive force on trapezoidal cross sections.**

<table>
<thead>
<tr>
<th>b/y RATIO</th>
<th>VERTICAL SIDE SLOPE</th>
<th>SIDE SLOPE 1V : 1.5H</th>
<th>SIDE SLOPE 1V : 2H</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>on bed ( \alpha_{bed} )</td>
<td>on slope ( \alpha_{side} )</td>
<td>on bed ( \alpha_{bed} )</td>
</tr>
<tr>
<td>0.5</td>
<td>0.20</td>
<td>0.30</td>
<td>0.72</td>
</tr>
<tr>
<td>1</td>
<td>0.40</td>
<td>0.46</td>
<td>0.80</td>
</tr>
<tr>
<td>2</td>
<td>0.68</td>
<td>0.68</td>
<td>0.89</td>
</tr>
<tr>
<td>3</td>
<td>0.87</td>
<td>0.73</td>
<td>0.94</td>
</tr>
<tr>
<td>4</td>
<td>0.94</td>
<td>0.74</td>
<td>0.97</td>
</tr>
<tr>
<td>5</td>
<td>-</td>
<td>-</td>
<td>0.98</td>
</tr>
<tr>
<td>7</td>
<td>-</td>
<td>-</td>
<td>0.99</td>
</tr>
<tr>
<td>10</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
</tr>
</tbody>
</table>
2.3.2. Critical Tractive Force on the Bed

Critical tractive force. The tractive force on the bed should remain below a certain critical value, i.e. the 'critical tractive force' $T_{cr}$, as to prevent scouring. Thus:

$$T = \rho ghys \quad \text{and} \quad T \leq T_{cr}$$

where: $T$ is the tractive force on the bed during the bed-forming discharge in N/m², $T_{cr}$ is the critical tractive force for that soil in N/m², $\rho = 1000$ kg/m³ is the density of water, $g = 9.8$ m/s² = 9.8 N/kg is the acceleration of gravity, $y$ is the water depth during the bed-forming discharge in m, and $s$ is the energy (water) gradient.

Values. The critical tractive force depends on factors like soil type, grain size, sediment content of the water, etc. Values for the critical tractive force are presented in table 2.4.

The difference between 'sediment-loaded water' and 'clear water' is based on the observation that sediment-loaded water makes a 'cementation' between the grains, so that they can resist a higher shear stress. Similarly, 'colloidal soils' (clay, loam) can resist a higher shear stress.

The critical tractive force of grains sizes of $D > 5$ mm may also follow from (e.g. Chow 1959):

$$T_{cr} = 0.9 D$$

where: $T_{cr}$ is the critical tractive force in N/m², and $D$ is the diameter of the grains in mm, see table 2.5.

Reduction of tractive force. When the tractive force of a channel is too high, a reduction may be obtained by:

- **reducing the water depth** $y$ by making the width $b$ of the channel bed wider, thus increasing the ratio $b/y$. Normally, the hydraulic radius will increase and more earth work is required;

- **reducing the (energy) gradient** $s$ of the channel. It means that expensive drop structures ("stortdammen") are required, to dissipate the excessive energy in the stilling basin ("woelbak").

Table 2.4. Critical tractive force $T_{cr}$.

<table>
<thead>
<tr>
<th>MATERIAL ON BED AND BANKS</th>
<th>SEDIMENT-LOADED WATER</th>
<th>CLEAR WATER</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T_{cr}$ N/m²</td>
<td>$u^*$ m/s</td>
</tr>
<tr>
<td>Fine sand</td>
<td>3.7</td>
<td>0.061</td>
</tr>
<tr>
<td>Sandy loam</td>
<td>3.7</td>
<td>0.061</td>
</tr>
<tr>
<td>Silt loam</td>
<td>5.4</td>
<td>0.073</td>
</tr>
<tr>
<td>Ordinary firm loam</td>
<td>7.3</td>
<td>0.085</td>
</tr>
<tr>
<td>Alluvial silts</td>
<td>7.3</td>
<td>0.085</td>
</tr>
<tr>
<td>Fine gravel</td>
<td>15.6</td>
<td>0.125</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>22.5</td>
<td>0.150</td>
</tr>
<tr>
<td>Alluvial silts</td>
<td>22.5</td>
<td>0.150</td>
</tr>
<tr>
<td>Mixture loam-sand-gravel</td>
<td>32.2</td>
<td>0.179</td>
</tr>
<tr>
<td>Shales and hardpans</td>
<td>32.7</td>
<td>0.181</td>
</tr>
<tr>
<td>Coarse gravel</td>
<td>32.7</td>
<td>0.181</td>
</tr>
<tr>
<td>Mixture loam-sand-gravel</td>
<td>39.1</td>
<td>0.196</td>
</tr>
<tr>
<td>Cobbles and shingles</td>
<td>53.7</td>
<td>0.232</td>
</tr>
</tbody>
</table>
Table 2.5. Size range of particles.

<table>
<thead>
<tr>
<th></th>
<th>Size range in m</th>
<th>Size range in mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>large boulders</td>
<td>1 - 5</td>
<td>2 - 10</td>
</tr>
<tr>
<td>small boulders</td>
<td>0.50 - 1</td>
<td>0.15 - 0.50</td>
</tr>
<tr>
<td>coarse cobbles</td>
<td>0.15 - 0.50</td>
<td>0.05 - 0.15</td>
</tr>
<tr>
<td>small cobbles</td>
<td>0.05 - 0.15</td>
<td></td>
</tr>
<tr>
<td>coarse gravel</td>
<td>0.01 - 0.05</td>
<td>0.002 - 0.01</td>
</tr>
<tr>
<td>fine gravel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>coarse sand</td>
<td></td>
<td>0.15 - 2.0</td>
</tr>
<tr>
<td>fine sand</td>
<td></td>
<td>0.060 - 0.150</td>
</tr>
<tr>
<td>coarse silt</td>
<td></td>
<td>0.020 - 0.060</td>
</tr>
<tr>
<td>fine silt</td>
<td></td>
<td>0.004 - 0.020</td>
</tr>
<tr>
<td>coarse clay</td>
<td></td>
<td>0.001 - 0.004</td>
</tr>
<tr>
<td>fine clay</td>
<td></td>
<td>0.0002 - 0.0010</td>
</tr>
</tbody>
</table>

2.3.3. Discussion on Scouring

'Shear stress' or 'tractive force'? It is obvious from above that the term 'tractive force' is actually wrong as the unit is 'N/m²'. The term 'shear stress' is more appropriate, and is also used in some literature.

However, the term 'tractive force method' is popular in design practices, and usually the expression 'tractive force' is also used instead of 'shear stress'.

Tractive force on side slopes. The literature gives much attention to the calculation of the stability of side slopes (e.g. Chow 1959, French 1994). This is related to the phenomenon that the peak value \( T_{\text{side}} \) of the tractive force on the side slope is lower than on the bed, while also the permissible tractive force on the side slopes is lower. However, it is also concluded in the literature that the rolling-down effect from the side slopes is negligible for channels in cohesive material. It means that the calculation on the side slope stability is generally not relevant, and that the stability of the bed is normally the determining factor in the design of stable channels.

Permissible velocity. In the past, it was believed that the permissible velocity \( v \) can be used as a design condition. It was thought that such a permissible velocity \( v \) would prevent the scouring of the bed, as well as the sedimentation of washload.

Since \( \pm 1930 \), the 'tractive force concept' ('\( T = \rho g y s \)') is widely accepted as a tool to describe the physical process of scouring, while it is more and more accepted that the 'energy concept' ('\( E = \rho g v s \)') describes the process of sedimentation.

As an example, different channels with a permissible velocity \( v = 0.6 \text{ m/s} \) are presented in table 2.6. Apparently, the design discharges \( Q = 100 \text{ m}^3/\text{s}, 10 \text{ m}^3/\text{s}, 1 \text{ m}^3/\text{s} \) and \( 0.1 \text{ m}^3/\text{s} \) have an effect on the tractive force \( T \) and on the sediment transport capacity \( E \).

It can be concluded that the 'permissible velocity concept' is not in line with the modern judgement on the tractive force \( T \) and the sediment transport capacity \( E \). Thus, the 'permissible velocity method' should not be considered anymore, as a better methods are available.
Table 2.6. Example of channels with a 'permissible velocity': $v = 0.6 \text{ m/s}$.

<table>
<thead>
<tr>
<th>$Q$ (m$^3$/s)</th>
<th>$h$ (m)</th>
<th>$k_s = (1/n)$</th>
<th>$b$ (m)</th>
<th>$n$</th>
<th>$v$ (m/s)</th>
<th>$T$ (N/m$^2$)</th>
<th>$E$ (W/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100.00</td>
<td>1.93</td>
<td>40.00</td>
<td>82.50</td>
<td>2.00</td>
<td>0.10</td>
<td>42.67</td>
<td>0.60</td>
</tr>
<tr>
<td>10.00</td>
<td>1.45</td>
<td>40.00</td>
<td>8.50</td>
<td>2.00</td>
<td>0.20</td>
<td>5.86</td>
<td>0.60</td>
</tr>
<tr>
<td>1.00</td>
<td>0.58</td>
<td>40.00</td>
<td>1.75</td>
<td>2.00</td>
<td>0.80</td>
<td>3.03</td>
<td>0.60</td>
</tr>
<tr>
<td>0.10</td>
<td>0.24</td>
<td>40.00</td>
<td>0.25</td>
<td>2.00</td>
<td>3.50</td>
<td>1.06</td>
<td>0.60</td>
</tr>
</tbody>
</table>

2.3.4. Discussion on Sedimentation

Side slope and siltation. Experience shows that in channels with considerable suspended-load, the side slopes become steeper than initially constructed, see figure 2.14.

It is observed that washload is not deposited at the bed of the channel, but at the side slopes. This phenomena is widely reported in the Indian literature (e.g. Garde 1977), but not in the USA literature. It can be understood by considering:

- the initial sedimentation ("afsetting") takes place at the side slopes. Here, the sediment-transporting capacity $\rho g v s$ is the lowest at the side slopes, because the velocity $v$ is here the lowest,

- the initial 'scouring' ("uitschuring") takes places at the bed. At the bed, the scouring forces (tractive force) $\rho g y s$ are higher than at the side slopes.

Instability. The result of the depositions on the side slopes makes that the steep slopes become steeper than constructed, e.g. up to $1_{\text{Vert}} : 0.5_{\text{Hor}} \left( m = 0.5 \right)$.

These steeper side slopes are not stable and will slide down after some time. The flow is obstructed and will scour the opposite bank, thus starting degeneration and meandering.

The energy approach can be used for the design of irrigation systems with washload. The criterium that 'the energy dissipation $E = \rho g v s$ should not be decreased below an initial value somewhere in the system' is simple to apply. It gives also an insight in the effect of the design variables on the washload transport. The energy approach also illustrates the unsolvable problem of sedimentation of washload on the side slopes, which ultimately leads to a capacity reduction in irrigation canals and to unstable banks in rivers.

![Figure 2.14. Deposits of sediment on the side slopes of a channel.](image)

Sand trap in an irrigation system. The 'criterium of Schoemaker' reads: "the transport capacity for washload is guaranteed when the value of $E = \rho g v s$ does not decrease in downstream direction". The criterium of Schoemaker can be applied in the design of irrigation systems with washload. Normally, such a system is equipped with a sandtrap. The
sandtrap should not only prevent the entrance of bedload into the irrigation system, but should also control washload.

An example is presented in figure 2.15. The earlier-designed primary and secondary canals will provide the value of \( E = \rho g v s \) for each reach, i.e. 0.48 W/m³, 0.72 W/m³, 0.36 W/m³, 0.52 W/m³, 0.47 W/m³, etc. The sandtrap has to be designed so that its energy dissipation is less than the lowest value in the whole system, i.e. 0.36 Watt/m³, to avoid sedimentation in one of the canal reaches.

**Size of the washload.** The size of the sediment that is allowed in the irrigation systems cannot easily be determined with the above approach. It seems true that the fall velocity \( w \) can be determined from the condition for an infinite concentration of washload according to Vlugter:

\[
\frac{\Delta_x w}{v \sin \alpha} = 1 \quad \text{with: } \Delta_x = \frac{\rho_s - \rho_w}{\rho_s} \quad \text{and: } \sin \alpha = s
\]

hence the fall velocity \( w \):

\[
w = \frac{v s}{0.6}
\]

while \( E = \rho g v s \), the equation for the fall velocity \( w \) can be re-written into:

\[
w = \frac{E}{0.6 \rho g}
\]

where: \( w \) is the fall velocity of the sediment in m/s, \( \rho_s = 2600 \text{ kg/m}^3 \) is the density of sediment, \( \rho_w = 1000 \text{ kg/m}^3 \) is the density of clear water, \( \rho = 1000 \text{ kg/m}^3 \) is the density of water-sediment mixture, \( v \) is the velocity of the water-sediment mixture in m/s, \( \alpha \) is the channel angle in radials, \( s \) is the gradient of the channel, \( E \) is the energy dissipation of the mixture in Watt/m³, and \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity.

Thus, the fall velocity \( w \) of the washload that can be transported through the canal system of the above example can be calculated at:

\[
w = E / [0.6 \rho g] = 0.36 / [0.6 \times 1000 \times 9.8] = 0.061 \times 10^{-3} \text{ m/s}.
\]

and a corresponding diameter \( D_m \) follows from the Rubey formula:

\[
D_m = (w / 0.87 \times 10^5)^{1/2} = 8.4 \times 10^{-6} \text{ m} = 0.008 \text{ mm}.
\]

However, a warning should be given on the validity of the value fall velocity as calculated with the Vlugter formula.

---

![Figure 2.15](image)

Figure 2.15. The 'criterium of Schoemaker' applied on an irrigation system.
To change the energy dissipation, Changing the canal parameters, such as the bed width \( b \), the channel gradient \( s \), the side slope \( m \) and the Strickler coefficient \( k \), have all there effect on the value of the energy dissipation \( E \). However, some of the parameters have more effect on the energy dissipation than others.

As an example, a canal reach with a capacity of \( Q = 10 \text{ m}^3/\text{s} \) is designed for a Strickler coefficient \( k = 40 \text{ m}^{1/3}/\text{s} \) at a bed width \( b = 5 \text{ m} \), gradient \( s = 0.2 \times 10^{-3} \), and side slopes \( 1_{\text{Vert}} : 2_{\text{Hor}} \). The corresponding water depth \( y = 1.82 \text{ m} \), and the value \( E \) of the energy dissipation amounts to \( E = 1.25 \text{ Watt/m}^3 \).

The change of these design variables on the transport capacity of washload is shown in figure 2.16. The energy dissipation, and thus washload transport capacity, increases for smaller a bed width \( b \), a steeper gradient \( s \), steeper side slopes \( m \), and a higher Strickler coefficient \( k \).

\[
\begin{align*}
\text{OPTION 1: the bed width } b &= 5 \text{ m is changed.} \\
\text{OPTION 2: the canal gradient } s &= 0.2 \times 10^{-3} \text{ is changed.} \\
\text{OPTION 3: the side slope } 1_{\text{Vert}} : 2_{\text{Hor}} &= \text{ changed.} \\
\text{OPTION 4: the Strickler coefficient } k &= 40 \text{ m}^{1/3}/\text{s} \text{ is changed.}
\end{align*}
\]

Figure 2.16. The effect of parameters \( b, s, m \) and \( k \) on the energy dissipation \( E \).
3. DESIGN OF OPEN-CHANNELS

3.3. Design of Earthwork

3.3.1. Design Procedure

Need to design. Deltas are created by sedimentation during the frequent floodings. But, the human occupation requires these floodings are controlled and that the land is adequately drained. So, the hydraulic engineer has to play an active role in the water management of deltaic areas, and many flood control, irrigation and drainage works have to be implemented.

Improvements of the major rivers in these coastal areas will mainly consist of the construction of embankments, removal of sediment and vegetation, straightening of bends, slope protection, etc.

However, situations may arise where it is necessary to reshape ("normaliseren") an unstable river to new dimensions, or to 'rehabilitate' ("herstellen") the unstable river to its old dimensions. When flood diversion is applied, the hydraulic engineer has to design the dimensions of the new flood channel. Furthermore, many new open-water drainage channels and irrigation canals have to be designed, as to manage the water in the deltaic area.

Present design. The present design procedure of flood channels, drainage channels and irrigation canals is still rather empirical. The design can be divided into:
- the determination of the 'alignment' ("trace") and the location of structures. This is not discussed here;
- the preparation of the 'design criteria' ("ontwerovoorschriften"), such as the side slopes, the freeboard, the dimensions of the embankments;
- the 'hydraulic design', such as the 'morphological method' and the 'regime method'.

3.1.2. Side Slopes

Soil mechanics. The side slope ("talud helling") $1_{\text{Vert}} : m_{\text{Hor}}$ depends on the principles of soil mechanics, but it is often selected on basis of regional experiences.

Typically, the side slope is taken as a function of the design discharge. These assumptions are usually laid down in 'design criteria' ("ontwerovoorschriften"). For example, $m = 1$ for $Q < 0.5 \text{ m}^3/\text{s}$, $m = 1.5$ for $0.5 < Q < 4 \text{ m}^3/\text{s}$, and $m = 2$ for $Q > 4 \text{ m}^3/\text{s}$. 
3.1.3. Freeboard

Need for freeboard. The 'freeboard' ("waking") is the vertical distance between the design water level and the top of the banks or the embankments. The required freeboard is often established on basis of experience or on 'good engineering practice'. The magnitude of the freeboard should be specified in 'design criteria' ("ontwerpvoorschriften").

Freeboard is needed for several reasons, but mainly to protect the embankments against breaching:
• the water levels may become higher than the design water levels, because of inaccuracies in the hydraulic design, waves, operational mistakes, etc.;
• the top of the embankment may become lower, because of inaccuracies in the construction, settlement ("setting") of the earthfill, settlement of the sub-soil, etc.;
• a higher freeboard with the same top width of the embankment creates a longer length for seepage ("dijkswel").

Freeboard in irrigation. The freeboard of irrigation canals is often based on the design water depth \( y \), or on the design discharge \( Q \). Examples are:
• the USA standards for irrigation canals refer often to the recommendations of the USBR (e.g. Chow 1959): \( F = c \sqrt{y} \) in m, with values of \( 0.68 < c < 0.87 \) for \( 0.6 \text{ m}^3/\text{s} < Q < 85 \text{ m}^3/\text{s} \), respectively.

Thus, an irrigation canal of \( Q = 0.68 \text{ m}^3/\text{s} \) with a water depth \( y = 0.70 \text{ m} \) requires a freeboard of \( F = 0.57 \text{ m} \). And a canal of \( Q = 85 \text{ m}^3/\text{s} \) with a water depth \( y = 2.80 \text{ m} \) requires a freeboard of \( F = 1.46 \text{ m} \).
• Indonesia applies freeboards of 0.40 m to 1.00 m on irrigation canals for design discharges of \( Q = 0.5 \text{ m}^3/\text{s} \) to 15 \text{ m}^3/\text{s} \), respectively (DHV 1986).

Freeboard in drainage. The freeboard of drainage channels is more difficult to assess than the freeboard of irrigation canals. Irrigation canals have design discharges that are also the 'maximum' discharges. Drainage channels may have a design discharge, but this discharge is related to a 'return period', e.g. the 20-years flood, and higher discharges may happen as well.

Strictly-speaking in drainage, discharges higher than the design discharge are allowed to breach the embankment. However, a somewhat higher freeboard is often applied in drainage engineering to create an additional protection.

For instance, a drainage channel with a freeboard of \( F = 2.00 \text{ m} \) above the water depth \( y_{5\text{-years}} = 3.00 \text{ m} \) during the design flood of \( Q_{5\text{-years}} \) may actually accommodate the 20-years' flood.

Example of the freeboard in drainage nad in flood control are:
• Indonesia applies freeboards of 0.40 m to 1.00 m on the main drainage systems for design discharges of \( Q_{5\text{-years}} = 0.1 \text{ m}^3/\text{s} \) to 40 \text{ m}^3/\text{s} \), respectively (DHV 1986).
• the Netherlands applies river embankments with a freeboard of 0.50 m plus the 'wave run-up' ("golfoploop") above the design flood \( Q_{1250\text{-years}} \),
3.1.4. Embankments

Spoil banks. When excavation ("ontgraving") exceeds filling ("ophoging"), extra earth has to be disposed. Normally it is very costly to transport this soil by mechanical means to other locations. An option is to provide more width on embankments when applied. Otherwise, 'spoil banks' ("stortplaatsen") must be designed, see figure 3.1.

The spoil banks are heaps of soil, and they should be discontinued at suitable intervals as to allow cross drainage of surface water.

Borrow pits. When filling exceeds earthwork in excavation, the earth has to be brought from somewhere. The pits, which are dug for bringing earth, are known as 'borrow pits' ("grondhaling").

Those pits can be located outside the channel (external pits), but they can also be located within the channel or in the foreland or flood plain (internal pits). Again, it is very costly to bring soil from a distance.

External pits are not preferred, as land has to be expropriated ("onteigend") and is reshaped into ponds that collect rainfall and may cause mosquito nuisance. Internal pits can be applied in the larger channels or in the flood plains. These internal borrow pits may not endanger the stability of banks. Moreover, they may not be constructed over the total length but only in compartments as to avoid flowing water that may cause scouring.

![Diagram of spoil banks and borrow pits](image)

**Figure 3.1.** Spoil banks and borrow pits.
3.1.5. Construction Drawings

Role of Drawings. Many people are involved in realization of open-channels. Drawings play an essential role in all stages of the works:
- the topographer prepares topographical maps, including contour lines ("hoogtelijnen");
- the geologist prepares geological and or soil-technical maps;
- the design engineer designs the works, and prepares draft drawings for the longitudinal profile ("lengteprofiel") and for typical cross-sections ("dwarsdoorsneden");
- the draughtsmen prepares the construction drawings for the tender documents ("bestekken");
- the contractor carries out the work on basis of these drawings.

Longitudinal Profile. The 'longitudinal profile' ("lengteprofiel") of an open-channel, together with a few typical cross sections, show basically all design information, see also figure 3.2. These (draft) drawings are made by the design staff. They show all technical data, such as:
- the design parameters (discharge, roughness, side slope, gradient, water depth, bed width);
- elevation of the groundlevel at the axis;
- design levels of the embankments, waterline and the bed;
- any structures.

Cross sections. The longitudinal profiles should be supplemented by a large number of 'cross sections' ("dwarsdoorsneden"), e.g. at intervals of 50 or 100 m.

Draughtsmen have to prepare these drawings on basis of the information of longitudinal profiles and the typical cross sections. The cross sectional drawings are used for the 'bill-of-quantity' ("hoeveelhedenstaat", "resultaatsbeschrijvingen") of the tender documents, and for the implementation in the field.

3.1.6. Tender Documents

Information. A contractor ("aannemer") will be invited to construct the designed open-channel with the related structures. Tender documents ("bestek") are required for the preparation of the Contract between the Government and the Contractor.

Tender documents. The tender documents ("bestek") will be prepared by the Programming and Implementation Unit of the Project, using standard bidding documents ("standaard bepalingen"). The Design Unit has to provide the following information:
- Description of the Works ("Omschrijving van het Werk"),
- Construction Drawings ("Bouwtekeningen"),
- Bill of Quantities ("Staat van Hoeveelheden", "Inschrijvingsstaat").

Cost estimate. Moreover, an engineering cost estimate ("directiebegroting ") has to be made for this project, i.e. the estimated costs of the construction contract, and any additional costs, such as for the detailed design and the construction supervision (toezicht op de uitvoering).

The engineering cost estimate will be based on 'unit prices' ("eenheidsprĳzen"), as collected from different projects in similar regions. An example of the unit rates is presented in table 3.1.
Figure 3.2: Longitudinal profile of a flood channel.

<table>
<thead>
<tr>
<th>Profile Number</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance, in m</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td>250</td>
</tr>
<tr>
<td>Total Distance, in m</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>300</td>
</tr>
<tr>
<td>Ground Level, in m+</td>
<td>9.0</td>
<td>9.0</td>
<td>8.8</td>
<td>8.8</td>
<td>8.6</td>
<td>8.6</td>
<td>8.0</td>
<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Embankment Level, in m+</td>
<td>10.1</td>
<td>10.1</td>
<td>10.1</td>
<td>10.1</td>
<td>10.1</td>
<td>10.1</td>
<td>10.1</td>
<td>10.1</td>
<td>10.1</td>
</tr>
<tr>
<td>Water Level, in m+</td>
<td>6.6</td>
<td>6.5</td>
<td>6.5</td>
<td>6.5</td>
<td>6.5</td>
<td>6.5</td>
<td>6.5</td>
<td>6.5</td>
<td>6.5</td>
</tr>
<tr>
<td>Bed Level, in m+</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
</tr>
<tr>
<td>Design Parameters:</td>
<td>Q = 200 m³/s</td>
<td>m = 2</td>
<td>k = 35 m¹/³/s</td>
<td>Q = 350 m³/s</td>
<td>m = 2</td>
<td>k = 35 m¹/³/s</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q = discharge</td>
<td>b = bed width</td>
<td>b = 35 m</td>
<td>s = 1.0 o/oo</td>
<td>b = 65 m</td>
<td>s = 0.5 o/oo</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>y = water depth</td>
<td>s = bed gradient</td>
<td>y = 2.61 m</td>
<td>v = 1.90 m/s</td>
<td>y = 2.78 m</td>
<td>v = 1.78 m/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 3.1. An example of 'unit prices'.

<table>
<thead>
<tr>
<th>Earthwork and Roads:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>• Land clearing</td>
<td>$0.50 /m²</td>
</tr>
<tr>
<td>• Large scale excavation and disposal, within 50 m</td>
<td>$2.00 /m³</td>
</tr>
<tr>
<td>• Dike construction, when soil directly available</td>
<td>$3.00 /m³</td>
</tr>
<tr>
<td>• Transport of material, per 1000 m</td>
<td>$5.00 /m³</td>
</tr>
<tr>
<td>• Grass planting on slopes</td>
<td>$1.00 /m²</td>
</tr>
<tr>
<td>• Village road, unpaved, 3.00 m wide</td>
<td>$5.00 /m¹</td>
</tr>
<tr>
<td>• Inspection road, with stones, 2.00 m wide</td>
<td>$50.00 /m¹</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Materials:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>• Masonry lining (&quot;gemetselde bekleding&quot;) of canals, 0.30 m thick</td>
<td>$100.00 /m²</td>
</tr>
<tr>
<td>• Masonry with brick blocks (&quot;metselwerk&quot;)</td>
<td>$200.00 /m³</td>
</tr>
<tr>
<td>• Stone pitching (&quot;gezette natuursteen&quot;) in concrete</td>
<td>$100.00 /m³</td>
</tr>
<tr>
<td>• Reinforced concrete (&quot;gewapend beton&quot;), 110 kg steel/m³</td>
<td>$200.00 /m³</td>
</tr>
<tr>
<td>• Mass concrete (&quot;stamp beton&quot;) and work floors, 280 kg cement/m³</td>
<td>$100.00 /m³</td>
</tr>
<tr>
<td>• Vertical gate with hoisting device (&quot;windwerk&quot;), upto ±1.00 m wide</td>
<td>$500.00 /m²</td>
</tr>
<tr>
<td>• Vertical gate with hoisting device, 1.50 m - 2.00 m wide</td>
<td>$900.00 /m²</td>
</tr>
</tbody>
</table>

Description of the Works. The 'Description of the Works' is particular to each Contract and must be specially written. It should include (i) a general description of the Project ('Het Project'), (ii) the work to be performed under the Contract ('Het Werk'), (iii) site location ('Locatie') and information as to its accessibility ('bereikbaarheid'), and (iv) other relevant information.

Construction Drawings. The 'Construction Drawings' will be made by the Design Unit on basis of the pre-design drawings.

Table 3.2. Bill of Quantities for 'General Items'.

<table>
<thead>
<tr>
<th>Description of the Work Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Rate in US$</th>
<th>Amount in US$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobilization (&quot;Mobilisatie&quot;, &quot;Voorbereiding&quot;), including but not limited to preparing, shipping and erecting construction plant and camp for the personnel of the Contractor</td>
<td>-</td>
<td>-</td>
<td>Lump Sum</td>
<td>(&quot;stel post&quot;)</td>
</tr>
<tr>
<td>Providing offices for the Engineer as specified, including furniture and operation &amp; maintenance ('Directiekosten')</td>
<td>-</td>
<td>-</td>
<td>Lump Sum</td>
<td></td>
</tr>
<tr>
<td>Demobilization (&quot;Demobilisatie&quot;), including but not limited to removing and shipping construction plant and camp, and cleaning the sites</td>
<td>-</td>
<td>-</td>
<td>Lump Sum</td>
<td></td>
</tr>
</tbody>
</table>

| Total of the Bill: |       |
Bill of Quantities. The 'Bill of Quantities' ("inschrijvingsstaat") itemizes the work in sufficient detail between different classes of works. The Bill of Quantities is divided here into the following work items:

- Bill of Quantities for 'General Items' ("Werken van Algemene Aard"), see table 3.2,
- Bill of Quantities for 'Earthwork' and 'Roads' ("Grondverzet en Wegen"), see table 3.3,
- Bill of Quantities for 'Structures' ("Kunstwerken"), see table 3.4 and table 3.5.

### Table 3.3. Bill of Quantities for Earthwork and Roads.

<table>
<thead>
<tr>
<th>Description of the Work Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Rate in US$</th>
<th>Amount in US$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clearing and grubbing (&quot;verwijdering struikgewas&quot;) the site of the canals</td>
<td>ha</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavate topsoil (&quot;ontgraving bodemlaag&quot;) to maximum depth of 0.25 m and stockpile (&quot;opslaan&quot;) for reuse within 50 m</td>
<td>m²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavate soil (&quot;grond ontgraving&quot;), depth of 0.25 m to 1.00 m, and dispose (&quot;storten&quot;, &quot;verwerken&quot;) within 50 m</td>
<td>m³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavate soil, depth of 1.00 m to 3.00 m, and dispose within 50 m</td>
<td>m³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavate fill material from approved borrow pits (&quot;grondput&quot;, &quot;cunet&quot;), haul (&quot;grondvervoer&quot;) up to 50 m, deposit (&quot;aanbrengen&quot;), shape (&quot;verwerken&quot;) and compact (&quot;verdichten&quot;) to fill (&quot;ophoging&quot;)</td>
<td>m³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Place, shape and compact of fill material (&quot;grond in ophoging&quot;) from excavated soil</td>
<td>m³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overhaul of material (&quot;grondvervoer&quot;) per 100 m distance</td>
<td>m³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Replace topsoil (&quot;aanbrengen van bodemlaag&quot;) to 0.20 m thickness, and plant grass (&quot;gras inzaaien&quot;) on slopes (&quot;taluds&quot;) and crests (&quot;kruin&quot;) of embankments</td>
<td>m²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Supply, place and compact stone-fill material (&quot;steemengsel&quot;) as road pavement (&quot;wegverharding&quot;)</td>
<td>m²</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Total of the Bill:
### Table 3.4. Bill of Quantities for ... cross regulators.

<table>
<thead>
<tr>
<th>Description of the Work Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Rate in US$</th>
<th>Amount in US$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavating soil (&quot;grondontgravende&quot;) for the structure</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Blinding layer concrete (&quot;beton voor fundering&quot;), Class B-1½</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Reinforced Concrete Class A-1½, including shuttering (&quot;bekisting&quot;) and 110 kg steel/m³</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Masonry (&quot;metselwerk&quot;) of solid concrete blocks</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Supply and fix vertical gates (&quot;vlakke schuiven&quot;) with hoisting device (&quot;windwerk&quot;), upto ±1.00 m wide</td>
<td>No.</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Supply and fix vertical gates with hoisting device, 1.50 m - 2.00 m wide</td>
<td>No.</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Place and compact backfill (&quot;grond aanvulling en verdichting&quot;) for structure</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
</tbody>
</table>

**Total of the Bill:**

### Table 3.5. Bill of Quantities for ... inflow structures.

<table>
<thead>
<tr>
<th>Description of the Work Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Rate in US$</th>
<th>Amount in US$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavating soil (&quot;grondontgravende&quot;) for the structure</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Blinding layer concrete (&quot;beton voor fundering&quot;), Class B-1½</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Reinforced Concrete Class A-1½, including shuttering (&quot;bekisting&quot;) and 110 kg steel/m³</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Masonry (&quot;metselwerk&quot;) of solid concrete blocks</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
<tr>
<td>Place and compact backfill (&quot;grond aanvulling en verdichting&quot;) for structure</td>
<td>m³</td>
<td>...×</td>
<td>............</td>
<td></td>
</tr>
</tbody>
</table>

**Total of the Bill:**
3.2. Hydraulic Design

3.2.1. Three Degrees of Freedom

Present literature. The present literature on design of open-channels does not provide a proper methodology. Generally it overlooks the statement (Ankum 1996): "channels have three degrees of freedom (bed width, water depth and channel gradient), which require three equations for its solution".

Here, several design methods will be discussed, i.e. the 'Strickler method', the 'permissible velocity method' and the 'tractive force method', and it will be shown that they provide less than the three required equations. Furthermore, the 'regime method' is discussed which provides even more than the three required equations. The 'morphological method' (Ankum 1996, Ankum 2000) provides just the three equations to solve the three degrees of freedom.

Furthermore, it should be acknowledged here that the maximum discharge is not necessary also the 'bed-forming discharge' which determines the morphological stability of the channel.

Three unknown parameters. The hydraulic design of open-channels depends on several assumptions laid down in 'design criteria', such as the Strickler coefficient $k$ in $m^{1/3}/s$, the side slope $l_{\text{vert}} : m_{\text{hor}}$, etc., etc.

For a given design discharge, the hydraulic design of the open-channel should provide three unknown parameters:
- the bed width $b$,
- the water depth $y$,
- the gradient $s$ of the bed.

It should be emphasized that the channel gradient $s$ is not equal to the gradient of the surrounding land, although some relation may exist. It means that the channel gradient $s$ has to be designed, and that it is not a 'given' parameter as is suggested by several authors (Jensen 1983, James 1988, Chaudhry 1993).

3.2.2. Discussion on the Design Discharge

Difficulties. Design of flood channels and drainage channels is more difficult than the design of irrigation canals.

Irrigation canals have a well-defined design discharge, which is the maximum canal discharge. Normally, the 'headworks' ("watervang") of the system prevents inflow of bedload into the canals. Also the transport of washload through the canals can be controlled by constructing a 'sandtrap' ("zandvang").

Flood and drainage channels have fluctuating discharges, and the 'bed-forming discharge' is normally lower, e.g. $Q_{1\text{-year}}$, than the (maximum) design discharge, e.g. $Q_{20\text{-years}}$. The inflow of bedload and washload cannot be controlled. Moreover, the sediment transport becomes quite difficult for decreasing channel gradients.
Design discharge. Thus, the discharge in a flood and drainage channels is varying and is not constant like in irrigation canals. The capacity of the flood and drainage channels should be set at a 'maximum discharge' $Q_{\text{max}}$ of a certain return period. However, the hydraulic design should also focus on the 'dominant discharge' $Q_{\text{dom}}$, which is the bed-forming discharge. The term 'design discharge' is confusing, and should be used only for the 'capacity' or for the 'maximum discharge' of the open-channel.

Dominant discharge. Thus, the design methods of open-channels make often use of the concept of a 'dominant discharge' ("bedvormende afvoer").

The concept of 'dominant discharge' originates from the regime design method and was put forward by Inglis in 1946 (e.g. Garde 1977): "the dominant discharge is that hypothetical steady discharge which would produce the same result in terms of average channel dimensions as the actual varying discharge". The dominant discharge $Q_{\text{dom}}$ may be taken at half to two-third of the maximum discharge according to Inglis, or may be set at a certain return period of e.g. 1.4 years (e.g. Garde 1977).

What is the dominant discharge? The following typical values for the dominant discharge are normally used:

- for irrigation canals, the dominant discharge is often equal to the 'full supply discharge', i.e. the maximum discharge or the design discharge;
- for rivers and drainage channels, the dominant discharge may be taken as the discharge with return period of e.g. $T = 0.5$ to $1.5$ years. It means that scouring may occur during the higher discharges, and that sedimentation occurs during the lower discharges.
- for rivers in the lower delta with frequent overtoppings of the riverbanks, the dominant discharge may equal the 'bankfull discharge'. The frequency of overtopping may have return periods of $T = 0.5$ to $1.5$ years, for instance (Henderson 1966). It is obvious that the dominant discharge of a river may change considerably when the hydrology of the river is changed. For instance, the reduction of peak floods by a reservoir or by a flood diversion will have an impact on the dominant discharge.

Design criterion. The concept of 'dominant discharge' means that scouring is allowed for the larger discharges and that sedimentation may occur during the lower discharges. The designer may translate the concept of dominant discharge into the design criterion: "no scouring nor sedimentation may occur during the dominant discharge $Q_{\text{dom}}$".

3.2.3. Discussion on the Design Method

Strickler method. It is sometimes suggested that the design of open-channels can be done by the 'Strickler formula' only (e.g. Chow 1959).

It is true that the Strickler formula is a valuable equation for uniform flow. But the formula describes only the relation between the parameters. In general, the design discharge is known, while also an assumption can be made on the Strickler coefficient and the side slope of the channel. As stated above, the (re-)construction of the channel by the contractor requires three parameters, i.e. the bed width $b$, the design water depth $y$ and the gradient of the bed $s$, while only one equation is available.

The Strickler method is recommended in literature for designing 'non-erodable channels'. In this method, the designer simply computes the dimensions of the channel by a uniform-flow formula and then decides the final dimensions on basis of 'hydraulic efficiency', or empirical rule of best section, practability, and economy (e.g. Chow 1959).
Thus, the Strickler method provides only one equation and fails to solve the three degrees of freedom.

**Permissible velocity method.** The survey of Fortier and Scobey in 1926 on the permissible velocity \( v \) became the basis for the 'permissible velocity method' (e.g. Chow 1959). It was thought that such a permissible velocity would prevent scouring of the bed, as well as deposition of sediment. The permissible velocity method uses only two equations: (i) the permissible velocity \( v \) for that soil type, and (ii) the Strickler formula. The channel gradient \( s \) has to be assumed.

The limitations of the permissible velocity method is shown by the following example, for which a permissible velocity \( v = 1 \) m/s is applied. A flood channel with a capacity of 100 m\(^3\)/s, side slopes \( 1_{vert} : 2_{hor} \), a Strickler coefficient \( k = 40 \) m\(^{1/3}\)/s and an assumed gradient \( s = 0.3 \times 10^{-3} \), leads to a bed width \( b = 50 \) m and a water depth \( y = 1.87 \) m. Another channel with capacity of 10 m\(^3\)/s would have a bed width \( b = 5 \) m and a water depth \( y = 1.31 \) m for an assumed gradient \( s = 0.7 \times 10^{-3} \). Evaluation of these designs by the physics of scouring, shows that the larger channel has a tractive force \( T = 5.5 \) N/m\(^2\) at the bed and the smaller channel \( T = 9.0 \) N/m\(^2\) while e.g. a permissible tractive force \( T_{critical} = 6 \) N/m\(^2\). It means the larger channel is stable at this velocity, but that the smaller channel will scour at the permissible velocity \( v = 1 \) m/s.

Thus, the permissible velocity method ignores the physical process of scouring which is well described by bed tractive force. Furthermore, the method provides only two equations instead of three. Therefore, the permissible velocity method should not be used anymore, although it is still recommended in recent literature (Jensen 1983, James 1988, Chaudhry 1993, ASCE 1995\(^a\)).

**Tractive force method.** The 'tractive force method' was developed by Lane in the 1950s, and focuses on the threshold of bed scouring (e.g. Chow 1959, French 1994). The tractive force method uses two equations: (i) the tractive force formula, and (ii) the Strickler formula. Also here, the channel gradient \( s \) has to be assumed.

For instance, a flood channel with a capacity of 100 m\(^3\)/s will be constructed with side slopes \( 1_{vert} : 2_{hor} \) and a Strickler coefficient \( k = 40 \) m\(^{1/3}\)/s. The soil has a permissible tractive force of \( T_{critical} = 6 \) N/m\(^2\). When a channel gradient \( s = 0.2 \times 10^{-3} \) is assumed, a bed width \( b = 25 \) m and a water depth \( y = 3.00 \) m can be calculated. The assumption of another gradient \( s = 0.4 \times 10^{-3} \) would lead to other dimensions: a bed width \( b = 60 \) m and a water depth \( y = 1.50 \) m.

The tractive force method is not practical for the design as it uses two equations to solve the three degrees of freedom. Moreover, it ignores the process of sedimentation.

**Two methods.** The present methods for the design of stable channels can be divided into two approaches:

- the 'morphological method' uses hydraulic theories to define a stable channel, such as the uniform flow formula for the flow of water, the tractive force formula to prevent scouring, the sediment transportation formulae for the flow of sediment. The 'morphological method' is recommended for further use.

- the 'regime method' is based on the belief that design rules can be derived from observations on stable channels.

The regime method is mainly a product of the Anglo-Indian school of hydraulic engineering. It was developed on irrigation and drainage projects throughout the Middle East, India and Egypt with canals in fine-grained soils, of less than 1 mm particle size and for capacities up to 400 m\(^3\)/s. The 'regime method' is discussed here because of its widely use, but is not recommended here.
3.3. Morphological Design Method

3.3.1. Methodology

Three unknown. The morphological design method acknowledges that three parameters has to be determined: (i) the bed width \(b\), (ii) the water depth \(y\), and (iii) the gradient \(s\) of the channel. Thus, three equations are required.

Two discharges. Furthermore, the morphological design method acknowledges that there are two discharges relevant in the design (Ankum 1996, Ankum 2000):
- the 'dominant discharge' ("dominante afvoer"), also called the 'bed-forming discharge', ("bedvormende afvoer") for the stability of the channel in order to avoid scouring and sedimentation on an annual basis;
- the 'design discharge' ("ontwerpafvoer"), also called the 'maximum discharge' or the 'capacity', for the water transport capacity of the channel, in order to avoid overtopping of the banks.

Equation 1. The morphological design method uses the Strickler formula as its first equation to describe the flow of water:

\[ Q = k A R^{2/3} s^{1/2}, \text{ and } Q = v A \]

with the wet cross-sectional area \(A\):

\[ A = (b + ym)y \]

and the hydraulic radius \(R\):

\[ R = \frac{A}{b + 2y \sqrt{1 + m^2} \sin \theta} \]

where: \(Q\) is the discharge in \(m^3/s\), \(v\) is the velocity in \(m/s\), \(A\) is the wet cross-sectional area in \(m^2\), \(R\) is the hydraulic radius in \(m\), \(s\) is the water level (energy) gradient, \(b\) is the bed width in \(m\), \(y\) is the water depth, in \(m\), \(m\) is the side slope (1:1.5 : 1.5), and \(k\) is the Strickler coefficient in \(m^{1/3}/s\).

The values of the Strickler coefficient \(k\) and the side slope ('talud helling') \(m\) of the channel should be considered as assumptions in the 'design criteria' ('ontwerpvoorschriften').

Equation 2. The second equation is related to the flow of sediment. It is assumed that there are two different situations, see figure 3.3:
- or, the channel is subject to scouring during the dominant discharge. It means that the channel has to be checked on the criterion of the critical 'tractive force' ('wand-schuifspanning') \(T_{cr}\) to prevent scouring. Scouring can be prevented e.g. by reducing the gradient \(s\);
- or, the channel is subject to sedimentation during the dominant discharge. It means that the channel should be checked on the sediment transporting capacity \(Q_s/Q\). Sedimentation can be prevented e.g. by increasing the gradient \(s\).

These two different situations cannot occur at the same time, as a channel cannot 'scour' the bed and deposits its sediments at the same time. This would be reflected in the values of the allowable 'tractive force' \(T_{cr}\) and of the 'sediment transporting capacity' \(E_{min}\). Therefore,
only one equation can be used in the design. Furthermore, it is acknowledged here that there are several gradients $s$ without scouring or sedimentation, because the process of scouring ($T = \rho g y s$) is described by other parameters than the process of sedimentation ($E = \rho g v s$).

The above assumption for equation 2 is in accordance with findings by Hjulström (1935), see also figure 3.3. However, he related the sediment transport to the velocity $v$ of the water, which is considered now as out-of-date by recent research.

Alternative for equation 2. The operational objective of main drainage systems in polders is to maintain the target water level ("streepeil"), i.e. the polder water level ("polderpeil"). However, it is only the water level at the outlet that can be maintained at the polder water level during all discharges, see figure 3.4. At all other locations, the water level during the design discharge will differ from the target level as a water level gradient $s$ is needed. It means that the actual canal levels at some distance $L$ from the outlet will deviate from the polder water level with a difference $\Delta y = L \times s$, where $s$ is the gradient of the water line.

The design of open-channels in polders may not use the flow of sediment as the condition in equation 2, but use another condition that is related to water level variations $\Delta y$ during the design discharge. Thus, a low gradient $s$ is selected often as the equation 2 in polder areas.

For instance, the velocity $v$ during the design discharge is set in the Netherlands at a low value $v = 0.25$ m/s. This makes the gradient $s$ of the water level during the design discharge quite, in the order of 0.05 - 0.10 m per kilometer ($s = 0.05 \times 10^{-3}$ to $0.10 \times 10^{-3}$). It means that the water level deviations from the (stagnant) polder water level $\Delta y$ are only small for changing discharges.
Figure 3.4. Water levels in the open-water system of a polder.

Equation 3. The third equation is related to the geometry of the cross-section. This is expressed by the relation $b/y$, between the bed width $b$ and the water depth $y$.

This width-to-depth ratio $n = b/y$ is a logic condition, as a design water depth e.g. $y = 0.25$ m should not be designed in a channel with a bed width of $b = 25$ m. However, satisfying values for the width-to-depth ratio $n = b/y$ cannot be found in the literature.

In general, smaller design capacities require also smaller width-to-depth ratio $n = b/y$. For instance, $Q \approx 1$ m$^3$/s requires $b/y \approx 1$, and $Q \approx 10$ m$^3$/s requires $b/y \approx 5$, etc.

3.3.2. Width-to-depth Ratio

Considerations. The following considerations can be used in selecting the proper width-to-depth ratio $n = b/y$, between the bed width $b$ and the water depth $y$:

- **minimum wet cross-sectional area** will lead to lower earthwork. It will be calculated below that the minimum wet cross-sectional area will lead to a small values of $n$, thus to narrow channels;

- **minimum maintenance costs** will be obtained, for instance, when the channel can be cleaned by a machine in a single run. This is better possible for a small values of $n$, thus for narrow channels;

- **minimum construction costs** will not be obtained by the minimum earthwork only, but also by the type of earthwork: earthwork on deep channels will involve higher 'unit rates' ('eenheidsprisen') because of the deeper layers are harder and the deeper layers will require more vertical lift.

  Thus, the lower 'unit rates' will be obtained by a large value of $n$, thus by wider channels;

- **minimum water level variations** in the channel are attractive for several reasons: (i) the stability of the embankments is better, (ii) the maximum water level, and so the embankments, are lower, (iii) navigation is possible during many discharges, etc.

  Minimum water level variations are obtained by a large value of $n$, thus by wider channels;
3. DESIGN OF OPEN-CHANNELS

• **maximum channel storage** is important when peak floods have to be reduced by 'flood routing', or when storage have to be created when a (tidal) sluice is closed during a certain time.

  The maximum channel storage is obtained by a large value of $n$, thus by wider channels;

• **minimum seepage into the drain** ("kwel") is important in polder areas with a 'Dutch profile'. A drain with a small width-to-depth ratio $n = b/y$ will cut deep through horizontal impervious layers as is observed in the polders of the Netherlands. Thus, the seepage will be limited by taking large values of $n$, thus by wide channels.

Minimum wet cross-sectional area. Sometimes, designers try to design at the 'minimum wet cross-sectional area $A$'. It can be proven, see also box 3.1, that the minimum cross-sectional area for any side slope $m$ is guaranteed when: "the trapezoidal shape is formed by tangents ("raaklijnen") to the semi-circle, having its centre in the water surface", see figure 3.5.

  The corresponding width-to-depth ratio $n = b/y$, between the bed width $b$ and the water depth $y$ for a minimum wet cross-sectional area reads, see also box 3.1:

  $\frac{b}{y} = 2\sqrt{m^2 + 1} - 2m$

with the bed width $b$, the water depth $y$ and the side slope $1_{\text{Vert}} : m_{\text{Hor}}$.

**No practical value.** Thus, the minimum wet cross-sectional area is obtained for the following width-to-depth ratio $n = b/y$, between the bed width $b$ and the water depth $y$:

  o for a side slope $m = 1$ : the width-to-depth ratio $n = b/y = 0.83$,
  o for a side slope $m = 1.5$ : the width-to-depth ratio $n = b/y = 0.61$,
  o for a side slope $m = 2$ : the width-to-depth ratio $n = b/y = 0.47$.

These values have no practical value for the design of unlined open-channels, as the value of the width-to-depth ratio $n = b/y$ is too low, and thus the cross section is too narrow, to meet the other considerations.

![BASIC GONIOMETRY](image)

$$\tan \alpha = \frac{1}{m}$$

$$\tan \frac{1}{2} \alpha = \frac{(1 - \cos \alpha)}{\sin \alpha}$$

**Figure 3.5. 'Best' hydraulic cross-section.**

**Cross-section for lined channels.** The approach of the 'minimum wet cross-sectional area', as discussed above, is applicable for the design of lined channels ("beklede kanalen"). So, it is even possible to calculate the minimum cross-section for a variable side slope $m$, see also box 3.1.

  The optimum cross-section for a trapezoid is 'one half of a hexagon' ("de helft van een gelijkbenige zeshoek"), see also figure 3.5: the three sides of the cross section are equal long, the bed width $b$ is twice the water depth $y$, and the side slope amounts to $1_{\text{Vert}} : 0.58_{\text{Hor}} (m = 0.58$ or $\alpha = 60^\circ)$, and the width-to-depth ratio $n = b/y = 1.2$. 
Minimum cross-section for a known side slope $m$?
The determination of the width-to-depth ratio $n = b/y$ for the minimum cross-section is usually transformed into the question: "what width-to-depth ratio $n = b/y$ gives the maximum discharge $Q$ for a fixed cross sectional area $A$?" This is solved in the following way (Chow 1959):
- the Strickler formula reads: $Q = kA R^{2/3} s^{1/2} = k A^{5/3} p^{2/3} s^{1/2}$,
- the hydraulic radius is: $R = A/P$,
- so that the discharge is: $Q = k A^{5/3} p^{2/3} s^{1/2}$,
- the bed width is: $b = A/y - m y$, because of the equation: $A = (b + ny)^2 y$,
- the wet perimeter $P = b + 2y \sqrt{m^2 + 1}$, so $P = A/y - m y + 2y \sqrt{m^2 + 1}$.

Considering that the parameters $Q$ and $A$ are fixed, the criterium becomes one of minimising the term $P$. Thus: $dP/dy = -A y^2 - m + 2 \sqrt{m^2 + 1} = 0$.

As $A = (b + ny)^2 y$, this equation becomes $-b/y - m - m + 2 \sqrt{m^2 + 1} = 0$, and finally $b/y = 2 \sqrt{m^2 + 1} - 2m$.

Are the bed and the slopes the 'tangents' to a semi-circle?
The side slope $1_{\text{ver}} : m_{\text{Hor}}$ has an angle $\alpha$ with the horizontal. Thus:
$$\tan \alpha = 1/m, \text{ or } \alpha = \arctan m, \sin \alpha = 1 / \sqrt{m^2 + 1}, \text{ and } \sin \alpha = m / \sqrt{m^2 + 1}$$
The width-to-depth ratio $n = b/y$ for the minimum cross-section reads:
$$b/y = 2(\sqrt{m^2 + 1} - m) = 2\left(\frac{1}{\sin \alpha} - \frac{\cos \alpha}{\sin \alpha}\right) = 2\left(\frac{1 - \cos \alpha}{\sin \alpha}\right)$$

Basic goniometry learns that $\tan 1/2 \alpha = (1 - \cos \alpha) / \sin \alpha$, so that: $\tan 1/2 \alpha = 1/2 b/y$.

This condition means that the point $M$ in the middle of the water line is also the centre of a circle, which tangents are the bed and the side slopes of the cross-section, see figure 3.5.

What side slope $m$ gives the most optimum cross-section?
The above width-to-depth ratio $b/y = 2 \sqrt{m^2 + 1} - 2m$ leads to an expression for the width $b$:
$$b = 2y \sqrt{m^2 + 1} - 2my$$
The cross-sectional area: $A = by + my^2 = 2y^2 \sqrt{m^2 + 1} - my^2$, so that:
$$y^2 = A / [2\sqrt{m^2 + 1} - m]$$
The wet perimeter $P = b + 2\sqrt{m^2 + 1} = 2y\{2\sqrt{m^2 + 1} - m\}$, so that:
$$P^2 = 4 \frac{A}{2\sqrt{m^2 + 1} - m} \left(2\sqrt{m^2 + 1} - 1 - m\right) = 4A \left(2\sqrt{m^2 + 1} - 1 - m\right)$$

and finally: $P = 2\sqrt{A} \left(2\sqrt{m^2 + 1} - m\right)$

The minimum value is found for $dP/dm = 0$, so:
$$\frac{dP}{dm} = \frac{A^{1/2}}{2\sqrt{m^2 + 1} - m^{1/2}} \left(\frac{2m}{(m^2 + 1)^{3/2}} - 1\right) = 0 \text{ for } m = \frac{1}{\sqrt{3}}$$

and so for $\alpha = 60^\circ$.  

Maximum value for \( E = \rho g v s \). The geometry of a cross-section (bed width \( b \), water depth \( y \) and side slope \( 1_{\text{vert}} : m_{\text{Hor}} \)) will also influence the 'deVos value' of the energy dissipation \( E = \rho g v s \).

The determination of the width-to-depth ratio \( n = b/y \) for the maximum value of \( E = \rho g v s \) can also be transformed into the question: "what width-to-depth ratio \( n = b/y \) gives the maximum velocity \( v \) for a fixed cross sectional area \( A \)?", and follows the calculation of box 3.1.

Thus, the maximum value of \( E = \rho g v s \) for a given discharge \( Q \) and a given gradient \( s \), follows also from the above formula of the minimum cross-sectional area:

\[
b/y = 2\sqrt{m^2 + 1} - 2m
\]

Again, this formula has no practical value for the design of unlined open-channels, as the value of \( n = b/y \) is too low, and thus the cross section is too narrow, to meet the other considerations.

Width-to-depth ratio \( b/y \). The width-to-depth ratio \( n = b/y \), between the bed width \( b \) and the design water depth \( y \), is often assessed on basis of practical considerations. Considerations may include wider channels have less water level variation, deep channels may cut through impervious horizontal layers, deep channels require less expropriation, as well on economic considerations.

Different relations have been developed for irrigation and drainage channels in different countries:
- In USA, the USBR-formula is used: \( b/y = 1.65 Q^{0.28} \),
- The Indonesian design standards are based on the Kennedy equation, but applied together with the tractive force concept.

Examples of often-used width-to-depth ratio \( n = b/y \) in irrigation are presented in table 3.6.

Width-to-depth ratio \( b/y \) in the design. It is obvious that the width-to-depth ratio \( n = b/y \) cannot be defined on strict objective grounds. Therefore, it is advisable to set a range of the width-to-depth ratio \( n = b/y \) in the design criteria, instead of just one value. Some guidance can be obtained from the USBR-formula: \( b/y = 1.65 Q^{0.28} \). For instance: \( Q_{\text{dom}} = 30 \text{ m}^3/\text{s} \) needs a range in the width-to-depth ratio \( n = b/y \) of \( 3 < n < 5 \).

In this way, the design remains flexible and can start the design with an assumption on the gradient \( s \), and can finally check this assumption of the width-to-depth ratio \( n = b/y \).

### Table 3.6. Values for the width-to-depth ratio \( n = b/y \) in irrigation canals.

<table>
<thead>
<tr>
<th>Design Discharge m³/s</th>
<th>&quot;best-&quot; Hydraulic b/y</th>
<th>USA standard b/y</th>
<th>Indonesia standard b/y</th>
<th>Regime canals sand/cohes. b/y</th>
<th>Regime canals sand b/y</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>(1.2)</td>
<td>1.4</td>
<td>1.2</td>
<td>5.5</td>
<td>5.9</td>
</tr>
<tr>
<td>1</td>
<td>(1.2)</td>
<td>1.7</td>
<td>1.5</td>
<td>4.2</td>
<td>6.1</td>
</tr>
<tr>
<td>2</td>
<td>(1.2)</td>
<td>2.0</td>
<td>2.0</td>
<td>6.4</td>
<td>6.9</td>
</tr>
<tr>
<td>5</td>
<td>(1.2)</td>
<td>2.6</td>
<td>2.9</td>
<td>7.5</td>
<td>7.9</td>
</tr>
<tr>
<td>10</td>
<td>(1.2)</td>
<td>3.1</td>
<td>3.9</td>
<td>7.9</td>
<td>8.4</td>
</tr>
<tr>
<td>20</td>
<td>(1.2)</td>
<td>3.8</td>
<td>5.8</td>
<td>8.9</td>
<td>9.5</td>
</tr>
<tr>
<td>50</td>
<td>(1.2)</td>
<td>4.9</td>
<td>10.0</td>
<td>10.4</td>
<td>11.7</td>
</tr>
<tr>
<td>100</td>
<td>(1.2)</td>
<td>6.0</td>
<td>-</td>
<td>11.8</td>
<td>12.8</td>
</tr>
</tbody>
</table>

* Simons & Albertson equations, with \( m = 1 \) for \( Q = 0.5 \text{ m}^3/\text{s} \), with \( m = 1.5 \) for \( 1 \text{ m}^3/\text{s} \leq Q \leq 5 \text{ m}^3/\text{s} \), with \( m = 2 \) for \( Q \geq 10 \text{ m}^3/\text{s} \).
3.3.3. Design on Basis of the Morphological Method

Objectives of the channel. The objectives of the channel should be agreed between the 'client' (= "opdrachtgever") and the designer before the hydraulic design starts. These objectives include:

- the 'dominant discharge' ("bedvormende afvoer") \( Q_{\text{dom}} \) for which the channel will not scour or deposits its sediment. This is e.g. the twice-per-year flood \( Q_{0.5,\text{-year}} \) or the once-per-year flood \( Q_{1,\text{-year}} \);
- the maximum discharge \( Q \) for which the embankments may not overtop. This is e.g. the once-per-20-years flood \( Q_{20,\text{-years}} \) or the once-per-50-years flood \( Q_{50,\text{-years}} \);
- the maximum value of the tractive force during the dominant discharge, i.e. the 'critical tractive force' \( T_{\text{max}} \) in N/m², as to prevent scouring;
- the minimum value of 'sediment transport capacity' \( E_{\text{min}} \) in Watt/m³ during the dominant discharge, as to prevent sedimentation;
- the water elevation at the 'tail-end' ("benedenstrooms") of the new channel reach during the dominant and the maximum discharges.

This elevation has to be tied to the 'reference level' ("referentie peil"), such as 'NAP' in the Netherlands, the 'mean sea level' ("gemiddeld zeeniveau"), the 'project datum' ("project peil"), etc.

Furthermore, information may be required on the water elevation at the 'head-end' ("bovenstrooms"), the terrain elevations, allowable variations of the water line, other functions of the channel, etc., etc.

Unknown. There are basically, three unknown parameters in the hydraulic design of open-channels:

- the bed width \( b \),
- the water depth \( y_{\text{dom}} \) during the 'dominant discharge' ("bedvormende afvoer"), which is the basis for the design of a stable channel, and
  - the water depth \( y_{\text{max}} \) during the maximum discharge, which is the basis for the design of the earthwork and the embankments ("dijken").
- the gradient \( s \) of the bed, which equals the gradient ("verhang") of the energy line and of the water line.

Furthermore, the elevation of the new channel has to be tied to the reference level ("referentie peil"), so that the contractor knows at what elevations are required.

Three equations. There are three equations available to solve the above unknown parameters:

- equation 1 describes the flow of water by means of the Strickler formula:
  \[
  Q = k A R^{2/3} s^{1/2}, \quad \text{and} \quad Q = v A
  \]
  with the wet cross-sectional area \( A \):
  \[
  A = (b + y m)y
  \]
  and the hydraulic radius \( R \):
  \[
  R = \frac{A}{b + 2y \sqrt{1 + m^2}}
  \]
- equation 2 is related to the flow of sediment. It is assumed that there are two different situations, and that only one equation can be used in the design:
  - or, the channel is subject to scouring during the dominant discharge. It means that the channel has to be checked on the criterion of the critical 'tractive force' ("wand- schuifspanning") \( T_{\text{max}} \) to prevent scouring
\[ T = \rho g y s \quad \text{and} \quad T \leq T_{\text{max}} \]

- or, the channel is subject to sedimentation during the dominant discharge. It means that the channel should be checked on the sediment transporting capacity \( E_{\text{min}} \) to prevent sedimentation:

\[ E = \rho g v s, \quad \text{or} \quad E = \rho g \frac{Q}{A} s, \quad \text{and} \quad E > E_{\text{min}} \]

- the alternative equation 2 is applied when flow of sediment is not relevant. For instance, water level changes \( \Delta y \) should be limited, which means that the gradient \( s \) should be kept at a low value;

- equation 3 is related to the geometry of the cross-section during the dominant discharge, or the maximum discharge. This is expressed by the width-to-depth ratio \( n = b/y \), between the bed width \( b \) and the water depth \( y \);

where: \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( T \) is the tractive force in \( \text{N}/\text{m}^2 \), \( E \) is the energy dissipation in \( \text{Watt}/\text{m}^3 \), \( v \) is the velocity in \( \text{m}/\text{s} \), \( A \) is the wet cross-sectional area in \( \text{m}^2 \), \( R \) is the hydraulic radius in \( \text{m} \), \( s \) is the water level (energy) gradient, \( b \) is the bed width in \( \text{m} \), \( y \) is the water depth in \( \text{m} \), \( m \) is the side slope (\( 1_{\text{Vert}} : n_{\text{Hor}} \)), \( k \) is the Strickler coefficient in \( \text{m}^{1/3}/\text{s} \), \( \rho = 1000 \text{ kg}/\text{m}^3 \) is the density of water, and \( g = 9.8 \text{ m}/\text{s}^2 = 9.8 \text{ N/kg} \) is the acceleration of gravity.

Assumptions in 'design criteria'. Several design parameters have to be confirmed with the 'client' (\( \approx \) "opdrachtgever") in design criteria ("ontwerpdoorsnitten"):

- the Strickler coefficient \( k \) in \( \text{m}^{1/3}/\text{s} \),
- the side slope \( m \),
- the width-to-depth ratio between bed width and water depth \( n = b/y \),

and other parameters like: freeboard, crest width of embankments, width of berms, side slopes of the embankments at land-side, minimum radius of a bend, location of borrow pits, location for spoil, etc., etc.

Actual design. The hydraulic design may provide different cross-sections that are valid. It is even not justified to aim at the most 'optimum' design, as so many assumptions have been made. The design can be done in the following steps, see figure 3.6:

- **step 1**: determine the elevation above the reference level at the tail end of the channel;
- **step 2**: draw from this reference level the straight water line during \( Q_{\text{dom}} \) into upstream direction. The uniform flow during the dominant discharge avoids backwater effects that may influence locally sedimentation and scouring.

The gradient \( s \) has to be estimated sufficient flat, as to prevent erosion and to match with the head-end water elevations.

The gradient \( s \) has to be estimated sufficient steep, as to prevent sedimentation and to avoid drop structures at the head-end;

- **step 3**: design the water depth \( y_{\text{dom}} \) and the bed width \( b \) of the channel by incorporating the width-to-depth ratio \( n = b/y \). Check the sediment transporting capacity \( E_{\text{min}} \) and the critical tractive force \( T_{\text{max}} \), and return to step 2 if necessary;
- **step 4**: calculate the water level \( y_{\text{max}} \) during \( Q_{\text{max}} \) with the Strickler formula. Also here, a 'straight' line may be taken by ignoring the backwater effects in the tail-reach of the channel. Check at the head-end, whether:
  - the calculated water elevation is *not higher* than the available water elevation (water cannot flow into the new channel),
  - the calculated water elevation is *lower* than the available water elevation, because a drop structure is required.
3.3.4. Example of the Morphological Method

Flood diversion channel. A new flood diversion channel has to be designed between a 'flood diversion structure' and the sea. The channel will have a length of 15 km.

The conditions at the tail-end are determined by a sea-level at 0.00 m+, while the tide in the sea can be ignored.

The conditions at the head-end are determined by the flood diversion structure. This structure should always flow by 'free flow', so that the water elevation during $Q_{\text{max}}$ may not become higher than at 14.00 m+. Lower water levels are allowed.

The dominant discharge is $Q_{\text{dom}} = 100$ m$^3$/s, and the maximum discharge is $Q_{\text{max}} = 250$ m$^3$/s.

The maximum value of the tractive force during the dominant discharge, i.e. the 'critical tractive force' $T_{\text{max}}$ = 30 N/m$^2$, as to prevent scouring. The minimum value of 'sediment transport capacity' $E_{\text{min}}$ = 10 Watt/m$^3$ during the dominant discharge, as to prevent sedimentation.

Assumptions in 'design criteria'. It has been agreed with the 'client' (≈ "opdrachtgever") that the following parameters are valid in design criteria:

- the Strickler coefficient $k$ = 35 m$^{1/3}$/s,
- the side slope $m = 2$,
- the width-to-depth ratio $n = b/y$ between bed width and water depth during the dominant discharge $Q_{\text{dom}} = 100$ m$^3$/s is set at the range $6 < n < 9$.

Some related parameters of the Strickler values have been calculated with the 'Profile' computer program, are presented table 3.7. They can be used in the design of the flood diversion channel.

Table 3.7. Some Strickler values for the flood diversion channel.

<table>
<thead>
<tr>
<th>$Q$</th>
<th>$\gamma$</th>
<th>$kS = (1/n)$</th>
<th>$b$</th>
<th>$m$</th>
<th>$S$</th>
<th>$n$</th>
<th>$v$</th>
<th>$T$</th>
<th>$E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>m$^3$/s</td>
<td>m</td>
<td>m$^{-1}$/3/s</td>
<td>m</td>
<td>(v:hm)</td>
<td>10^{-3}</td>
<td>(b/y)</td>
<td>m/s</td>
<td>N/m$^2$</td>
<td>W/m$^3$</td>
</tr>
<tr>
<td>100.00</td>
<td>2.88</td>
<td>35.00</td>
<td>20.00</td>
<td>2.00</td>
<td>0.50</td>
<td>6.94</td>
<td>1.35</td>
<td>14.14</td>
<td>6.61</td>
</tr>
<tr>
<td>100.00</td>
<td>2.57</td>
<td>35.00</td>
<td>20.00</td>
<td>2.00</td>
<td>0.75</td>
<td>7.78</td>
<td>1.55</td>
<td>18.91</td>
<td>11.39</td>
</tr>
<tr>
<td>100.00</td>
<td>2.37</td>
<td>35.00</td>
<td>20.00</td>
<td>2.00</td>
<td>1.00</td>
<td>8.44</td>
<td>1.71</td>
<td>23.23</td>
<td>16.74</td>
</tr>
<tr>
<td>250.00</td>
<td>4.77</td>
<td>35.00</td>
<td>20.00</td>
<td>2.00</td>
<td>0.50</td>
<td>4.19</td>
<td>1.77</td>
<td>23.41</td>
<td>8.70</td>
</tr>
<tr>
<td>250.00</td>
<td>4.28</td>
<td>35.00</td>
<td>20.00</td>
<td>2.00</td>
<td>0.75</td>
<td>4.68</td>
<td>2.05</td>
<td>31.47</td>
<td>15.06</td>
</tr>
<tr>
<td>250.00</td>
<td>3.95</td>
<td>35.00</td>
<td>20.00</td>
<td>2.00</td>
<td>1.00</td>
<td>5.06</td>
<td>2.27</td>
<td>38.79</td>
<td>22.22</td>
</tr>
</tbody>
</table>

Design process. The water elevation at the tail-end of the channel is known, as it equals the sea elevation of 0.00 m+.

A gradient $s$ is assumed for $Q_{\text{dom}}$. The water depth $y_{\text{dom}}$ and the bed width $b$ can be calculated by the Strickler equation (use e.g. program 'profile'), by considering the width-to-depth ratio $n = b/y_{\text{dom}}$, while the critical tractive force $T_{\text{cr}}$ and the minimum sediment transporting capacity $E_{\text{min}}$ are checked. Finally, a check is made on water elevation at the head-end during $Q_{\text{max}}$, see also figure 3.6.

Thus, the design of the channel follows a trial-and-error approach by assuming different gradients of the water line during $Q_{\text{dom}}$ and $Q_{\text{max}}$.
3. DESIGN OF OPEN-CHANNELS

- Choose a gradient of $s = 1.00 \, \% \, of = 1.00 \times 10^{-3}$:
  - Calculate the corresponding water depth $y_{dpm} = 2.37 \, m$ and bed width $b = 20 \, m$ for $n = 8.4, T = 23 \, N/m^2$ and $E = 17 \, W/m^3$.
  - Calculate the water $y_{max} = 3.95 \, m$.
  - The water elevation at channel tail-end for $Q_{dpm}$ is $0.00 \, m^+$, so the bed elevation at the channel tail-end is $0.00 - 2.37 = -2.37 \, m^+$.
    Thus, the bed elevation at the channel head-end is $-2.37 + 15 \times 1.00 = 12.63 \, m^+$, and the water elevation at channel head-end for $Q_{max}$: $12.63 + 3.95 = 16.58 \, m^+$.
    Conclusion: not successful, as the water elevation at $Q_{max}$ is higher than the objective of $14.00 \, m^+$.

- Choose a gradient of $s = 0.75 \, \% \, of = 0.75 \times 10^{-3}$:
  - Calculate the corresponding water depth $y_{dpm} = 2.57 \, m$ and bed width $b = 20 \, m$ for $n = 7.8, T = 19 \, N/m^2$ and $E = 11 \, W/m^3$.
  - Calculate the water $y_{max} = 4.28 \, m$.
  - The water elevation at channel tail-end for $Q_{dpm}$ is $0.00 \, m^+$, so the bed elevation at the channel tail-end is $0.00 - 2.57 = -2.57 \, m^+$.
    Thus, the bed elevation at the channel head-end is $-2.57 + 15 \times 0.75 = 8.68 \, m^+$, and the water elevation at channel head-end for $Q_{max}$: $8.68 + 4.28 = 12.96 \, m^+$.
    Conclusion: correct design, as the energy elevation allows for free flow!

![Figure 3.6. Longitudinal profile of the flood diversion channel.](image)

3.3.5. Discussion on the Morphological Method

Range for the gradient. The above described morphological method for the design of open-channels acknowledges the three design parameters and provides three design conditions. Furthermore, it is recognized that ‘sedimentation’ and ‘scouring’ of the cross-section are two different phenomena that cannot occur at the same time. Thus, gradient $s$ of the channel should be larger than the minimum gradient $s_{min}$ as to avoid sedimentation, and should be smaller than the maximum gradient $s_{max}$ as to avoid scouring.
Formula for the minimum gradient. The formula for the minimum channel gradient $s_{\text{min}}$ can be derived by using the energy dissipation formula $\frac{E_{\text{min}}}{\rho g v s} \geq E_{\text{min}}$, see box 3.2:

$$s_{\text{min}} = \left( \frac{E_{\text{min}}}{\rho g R} \right)^{\frac{8}{11}} \left( \frac{1}{k^3 Q} \right)^{\frac{2}{11}} \frac{(n + 2\sqrt{m^2 + 1})^{\frac{4}{11}}}{(n + m)^{\frac{2}{11}}}$$

with the minimum channel gradient $s_{\text{min}}$, the minimum value for the energy dissipation $E_{\text{min}}$ in Watt/m³ to avoid sedimentation, the discharge $Q$ in m³/s, the Strickler coefficient $k$ in m¹/³/s, the width-to-depth ratio $n = b/y$, the bed width $b$ in m, the water depth $y$ in m, the side slope $m$ (1 vert : $m_{\text{Hor}}$), the density of water $\rho = 1000$ kg/m³ and the gravity acceleration $g = 9.8$ m/s² = 9.8 N/kg.

Box 3.2. Formula for the minimum channel gradient $s_{\text{min}}$ to avoid sedimentation.

The formula for the minimum channel gradient $s_{\text{min}}$ to avoid sedimentation can be derived for a bed-to-depth ratio $n = b/y$ from the Strickler formula:

$$Q = k A R^{\frac{2}{3}} s^{\frac{1}{2}}$$

with the wet cross-sectional area $A$:

$$A = (b + my)y = (n + m)y^2$$

and with the hydraulic radius $R$:

$$R = \frac{A}{b + 2y \sqrt{1 + m^2}} = \frac{(n + m)}{(n + 2\sqrt{1 + m^2})} y$$

Hence:

$$Q = k \times [(n + m)y^2] \times \left[ \frac{(n + m)}{(n + 2\sqrt{1 + m^2})} y \right]^{\frac{2}{3}} \times s^{\frac{1}{2}}$$

and:

$$Q = k \times y^{\frac{8}{3}} \times \frac{(n + m)^{\frac{5}{3}}}{(n + 2\sqrt{1 + m^2})^{\frac{2}{3}}} \times s^{\frac{1}{2}}$$

so the equation for the water depth $y$ reads:

$$y = \left( \frac{Q}{k} \right)^{\frac{3}{8}} \times \frac{(n + 2\sqrt{1 + m^2})^{\frac{1}{4}}}{(n + m)^{\frac{5}{8}}} \times s^{-\frac{3}{16}}$$

and the wet cross-sectional area:

$$A = (n + m)y^2 = \left( \frac{Q}{k} \right)^{\frac{3}{4}} \times \frac{(n + 2\sqrt{1 + m^2})^{\frac{1}{2}}}{(n + m)^{\frac{1}{4}}} \times s^{-\frac{3}{8}}$$

Sedimentation will be avoided when $\rho g v s \geq E_{\text{min}}$ with the velocity $v = Q/A$, so:

$$s \geq \frac{E_{\text{min}}}{\rho g} \times \frac{A}{Q}$$

Hence:

$$s^{\frac{11}{8}} \geq \frac{E_{\text{min}}}{\rho g} \times \frac{1}{k^{\frac{3}{4}} Q^{\frac{1}{4}}} \times \frac{(n + 2\sqrt{1 + m^2})^{\frac{1}{2}}}{(n + m)^{\frac{1}{4}}}$$

and finally the formula for the minimum channel gradient $s_{\text{min}}$ to avoid sedimentation reads:

$$s_{\text{min}} = \left( \frac{E_{\text{min}}}{\rho g} \right)^{\frac{8}{11}} \left( \frac{1}{k^3 Q} \right)^{\frac{2}{11}} \frac{(n + 2\sqrt{m^2 + 1})^{\frac{4}{11}}}{(n + m)^{\frac{2}{11}}}$$
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Formula for maximum gradient. The formula for the maximum gradient \( s_{\text{max}} \) to avoid scouring can be derived by using the tractive force formula \( \rho g y s \leq T_{\text{max}} \), see box 3.3:

\[
s_{\text{max}} = \left( \frac{T_{\text{max}}}{\rho g} \right)^{16/13} \left( \frac{k}{Q} \right)^{6/13} \left( \frac{n + m}{n + 2 \sqrt{1 + m^2}} \right)^{10/13}
\]

with the maximum gradient \( s_{\text{max}} \), the maximum value for the tractive force \( T_{\text{max}} \) in N/m² to avoid scouring, the discharge \( Q \) in m³/s, the Strickler coefficient \( k \) in m¹/³/s, the width-to-depth ratio \( n = b/y \), the bed width \( b \) in m, the water depth \( y \) in m, the side slope \( m (1 \leq \text{vert} : m_{\text{Hor}}) \), the density of water \( \rho = 1000 \text{ kg/m}^3 \) and the gravity acceleration \( g = 9.8 \text{ m/s}^2 \).

Example. The meaning of these limits for the gradient \( s \) can be shown in the following example. A flood channel has a dominant discharge \( Q_{\text{dom}} = 100 \text{ m}^3/\text{s} \), a Strickler coefficient \( k = 40 \text{ m}^{1/3}/\text{s} \), side slopes \( 1_{\text{vert}} : 2_{\text{hor}} \), a maximum tractive force \( T_{\text{max}} = 6 \text{ N/m}^2 \), and a minimum energy dissipation \( E_{\text{min}} = 1.25 \text{ Watt/m}^3 \). Thus, the minimum and the maximum channel gradient \( s \) can be calculated for a given width-to-depth ratio \( n = b/y \), see figure 3.7.

Box 3.3. Formula for the maximum channel gradient \( s_{\text{max}} \) to avoid scouring.

The formula for the maximum channel gradient \( s_{\text{max}} \) to avoid scouring can be derived for a bed-to-depth ratio \( n = b/y \) by using the tractive force formula \( \rho g y s \leq T_{\text{max}} \) and by substituting the above value of \( y \), so:

\[
\rho g \times \left( \frac{Q}{k} \right)^{3/8} \times \frac{(n + 2 \sqrt{1 + m^2})^{1/4}}{(n + m)^{5/8}} \times s^{3/16} \leq s_{\text{max}} \leq \frac{T_{\text{max}}}{\rho g} \times \left( \frac{k}{Q} \right)^{3/8} \times \frac{(n + m)^{5/8}}{(n + 2 \sqrt{1 + m^2})^{1/4}}
\]

and finally the formula for the maximum channel gradient \( s_{\text{max}} \) to avoid scouring:

\[
s_{\text{max}} = \left( \frac{T_{\text{max}}}{\rho g} \right)^{16/13} \left( \frac{k}{Q} \right)^{6/13} \left( \frac{n + m}{n + 2 \sqrt{1 + m^2}} \right)^{10/13}
\]

Figure 3.7. Design example of a stable channel bed.
3.4. Regime Design Method

3.4.1. Introduction

In India, the 'regime method' ("regiem methode") for the design of water courses was first developed in India for irrigation canals. The regime method is based on the belief that design rules can be derived from observations on stable channels. The term 'regime' suggests an equilibrium between scouring and sedimentation on a yearly basis.

Many authors have developed their own regime formulae. Well-known is Lacey, who provided in 1937 a set of three equations (e.g. Raudkivi 1976). Simons & Albertson presented in 1963 a set of six equations, which were reduced by Graf (1971) to three equations.

The original regime method was related to the more or less constant discharges as happens in irrigation canals. The concept of the 'dominant discharge' has made it possible to apply the regime method also on the design of drainage channels with varying discharges.

3.4.2. Kennedy equations

Kennedy Equation. The regime method started with the observations of Kennedy on the Upper Bari Doab irrigation canal, India, in 1895. It was considered that this canal was 'stable', i.e. no maintenance was needed.

For regime conditions, Kennedy concluded that the velocity \( v \) in m/s, and the water depth \( y \) in m, were related by \( v = 0.55 y^{0.64} \).

Investigations in India and elsewhere have shown that the constants in the Kennedy equation vary for various canal systems, see table 3.8.

<table>
<thead>
<tr>
<th>Country</th>
<th>( C )</th>
<th>( n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>India (Upper Bari Doab, Kennedy)</td>
<td>0.55</td>
<td>0.64</td>
</tr>
<tr>
<td>Indonesia: flat (Haringhuizen)</td>
<td>0.49</td>
<td>0.64</td>
</tr>
<tr>
<td>Indonesia: steep (Haringhuizen)</td>
<td>0.55</td>
<td>0.64</td>
</tr>
<tr>
<td>India (Ponnar river)</td>
<td>0.60</td>
<td>0.64</td>
</tr>
<tr>
<td>India (Chenab)</td>
<td>0.57</td>
<td>0.57</td>
</tr>
<tr>
<td>India (Krishna)</td>
<td>0.53</td>
<td>0.52</td>
</tr>
<tr>
<td>Egypt</td>
<td>0.25 to 0.35</td>
<td>0.64 to 0.73</td>
</tr>
<tr>
<td>Thailand</td>
<td>0.35</td>
<td>0.66</td>
</tr>
<tr>
<td>Argentina (Rio Negro)</td>
<td>0.52</td>
<td>0.44</td>
</tr>
<tr>
<td>Burma (Shwebo)</td>
<td>0.55</td>
<td>0.57</td>
</tr>
<tr>
<td>USA (Imperial valley)</td>
<td>0.62 to 1.19</td>
<td>0.61 to 0.64</td>
</tr>
</tbody>
</table>

Haringhuizen equations. Haringhuizen developed regime equations for irrigation canals on Java, Indonesia, in the 1920's. Haringhuizen distinguished between 'flat' and 'mountainous' areas. For 'mountainous' areas, he has re-written the Kennedy equation into: \( y = 2.54 v^{1.56} \) and added a second equation between the velocity \( v \) and the discharge \( Q \).
The Strickler equation is the **third equation** needed for the determination of the three design parameters $y$, $b$, and $s$.

The terms 'flat' and 'mountainous' can be better translated in 'flat alignment' and 'steep alignment', as to indicate the lower and upper range of equilibrium. Thus, the Haringhuizen equations read:

- **flat alignment**  
  \[ v = 0.42 \, Q^{0.182}, \text{ and } y = 3.00 \, v^{1.56} \]

- **steep alignment**  
  \[ v = 0.46 \, Q^{0.182}, \text{ and } y = 2.54 \, v^{1.56}, \]

while  
\[ A = Q / v = (b + m \, y) \, y. \]

where: $Q$ is the discharge in m$^3$/s, $v$ is the velocity in m/s, $A$ is the wet cross-sectional area in m$^2$, $s$ is the energy (channel) gradient, $b$ is the bed width in m, $y$ is the water depth in m, $m$ is the side slope ($1_{\text{Vert}} : m_{\text{Hor}}$).

### 3.4.3. Lacey Equations

**Lacey equations.** In the 1930s, Lacey performed a systematic analysis of the available stable channel data in an attempt to improve the Kennedy equation. He established **three equations** for regime channels which are presented in literature in different forms. For design purposes it is advantageous to write them as (Raudkivi 1976):

- **wetted perimeter**  
  \[ P = 4.83 \, Q^{1/2} \]

- **wet cross sectional area**  
  \[ A = 2.28 \, f^{1/3} \, Q^{5/6} \]

- **channel gradient**  
  \[ s = 0.315 \times f^{5/3} \, Q^{1/6} \times 10^{-3} \]

where $Q$ is the dominant discharge in m$^3$/s, and $f$ is the Lacey silt factor. Note that the numerical coefficients are not dimensionless.

**Simplified Lacey equations.** The above Lacey equations are not very practical for the design engineer. They can be simplified, considering the wet cross sectional area $A = (b + my) y$, and the wetted perimeter $P = b + 2 \, y \sqrt{1 + m^2}$, see box 3.4.

After the above processing, the **simplified Lacey equations** can be written for different side slopes $1_{\text{Vert}} : m_{\text{Hor}}$ as:

- **equation 1:**
  \[ \text{for } m = 1 : \quad y = 1.32 \, Q^{1/2} - \sqrt{[1.74 - 1.25 f^{1/3} \, Q^{5/6}]} \]
  \[ \text{for } m = 1.5 : \quad y = 1.15 \, Q^{1/2} - \sqrt{[1.32 Q - 1.08 f^{1/3} \, Q^{5/6}]} \]
  \[ \text{for } m = 2 : \quad y = 0.98 \, Q^{1/2} - \sqrt{[0.96 Q - 0.92 f^{1/3} \, Q^{5/6}]} \]

- **equation 2:**
  \[ b = 4.83 \, Q^{1/2} - 2y \sqrt{[1 + m^2]} \]

- **equation 3:**
  \[ s = 0.315 \times f^{5/3} \, Q^{1/6} \times 10^{-3} \]

where $Q$ is the design discharge in m$^3$/s, $f$ is the Lacey silt factor in mm$^{1/2}$, $s$ is the (energy) gradient of the channel, $b$ is the bed width in m, $y$ is the water depth in m, $m$ is the side slope ($1_{\text{Vert}} : m_{\text{Hor}}$).

### Box 3.4. Further processing of the Lacey equations.

The Lacey equations can be simplified, considering the wet cross sectional area

\[ A = (b + my) y, \text{ and the wetted perimeter } P = b + 2 \, y \sqrt{[1 + m^2]} \].

Thus, $m \, y^2 + b \, y - A = 0$, inwhich $b = P - 2 \, y \sqrt{[1 + m^2]}$, and after substitution of $b$:

\[ (2 \sqrt{[1 + m^2]} - m) \, y^2 - P \, y + A = 0 \].

The value of $y$ can be solved:

\[ y = \left\{ P + \sqrt{[P^2 - 4 \, A \, (2 \, \sqrt{[1 + m^2]} - m)]} \right\} / \left\{ 4 \sqrt{[1 + m^2]} - 2m \right\} \].

After substitution of $P$ and $A$ of Lacey’s equations:

\[ y = \left\{ 4.83 \, Q^{1/2} + \sqrt{[4.83^2 Q - 4 \times 2.28 \, f^{1/3} \, Q^{5/6} \, (2 \, \sqrt{[1 + m^2]} - m)]} \right\} / \left\{ 4 \sqrt{[1 + m^2]} - 2m \right\} \].
Silt factor. The 'silt factor' \( f \) is based on the assumption that stable canal sections may exist. Lacey suggested that \( f = 1.59 \sqrt{D_m} \), where \( D_m \) is the sediment size in mm, see also table 3.9.

In practice, it is difficult to assess the value of \( D_m \). Measurements in existing canals may lead to a wide range of silt factors, as sediment deposits through the bed may vary considerably in sizes. Moreover, the sediment size of new channels is difficult to assess.

Therefore, the silt factor is often directly calculated from the channel characteristics through \( f = 2.5 \frac{v^2}{R} \), where \( v \) is the velocity in m/s and \( R \) is the hydraulic radius in m.

Validity. Criticism of the Lacey's method centres on the limited range of validity:
- the Lacey equations have been derived from regional data and for velocities between 0.3 m/s and 1.2 m/s;
- the Lacey equations are based on sediment observations in Punjab, India, for canals with suspended-load and where the concentration ranges between 1000 - 2000 ppm;
- the size of sediment has an effect on the wetted perimeter ("natte omtrek"), i.e. finer sediments give more narrow channels with steeper side slopes;
- basically, the Lacey equations determine the gradient \( s \) of the channel for a known discharge \( Q \) and a known silt factor \( f \). In river work, all three parameters \( Q, f \) and \( s \) are known, and they may not satisfy the calculated channel gradient.
- the Lacey equations provide for three equations to calculate three unknown parameters: the gradient \( s \), the bed width \( b \) and the water depth \( y \) for an assumed side slope \( m \). The Strickler formula is not yet used, so that also the Strickler coefficient \( k \) can be calculated. This is not logic, as the Strickler coefficient \( k \) is related e.g. to the maintenance of the channel.

<table>
<thead>
<tr>
<th>channel material</th>
<th>average diameter</th>
<th>silt factor ( f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>large stones</td>
<td>( D_m = 600 \text{ mm} )</td>
<td>40</td>
</tr>
<tr>
<td>large boulders</td>
<td>( D_m = 150 \text{ mm} )</td>
<td>20</td>
</tr>
<tr>
<td>small boulders</td>
<td>( D_m = 40 \text{ mm} )</td>
<td>10</td>
</tr>
<tr>
<td>gravel</td>
<td>( D_m = 10 \text{ mm} )</td>
<td>5</td>
</tr>
<tr>
<td>medium sand</td>
<td>( D_m = 1.5 \text{ mm} )</td>
<td>2</td>
</tr>
<tr>
<td>standard Kennedy silt (India)</td>
<td>( D_m = 0.40 \text{ mm} )</td>
<td>1.0</td>
</tr>
<tr>
<td>lower Mississippi silt</td>
<td>( D_m = 0.06 \text{ mm} )</td>
<td>0.4</td>
</tr>
</tbody>
</table>

3.4.4. Simons & Albertson Equations

Types of channels. An important step toward generalization of the regime method is the study of Simons and Albertson (Henderson 1966). They made a collection of field data from Indian and North American sources and distinguished five types of channels:
- coarse non-cohesive bed and coarse non-cohesive banks;
- sand bed and sand banks;
- sand bed and cohesive banks, with low sediment load;
- sand bed and cohesive banks, with heavy sediment load (2000 - 8000 ppm);
- cohesive bed and cohesive banks.
Basic equations. A separate equation could be fitted to each of these types. So, Simons and Albertson prepared a set of equations:

\[ b' = 0.9 \, P, \text{ where } P = K_1 \sqrt{Q}, \]
\[ b' = 0.92 \, W - 0.61 \]
\[ R = K_2 \, Q^{0.36} \]
\[ y = 1.21 \, R, \text{ for } R \leq 2.13 \, m, \quad \text{or: } y = 0.93 \, R + 0.61, \text{ for } R \geq 2.13 \, m \]
\[ v = K_3 \left( 10.76 \, R^2 \, s \right)^n \]
\[ v^2 = K_4 \, g \, y \, s \left( v/b \right)^{0.37} \]

where: \( Q \) is the design discharge in \( m^3/s \), \( v \) is the velocity in \( m/s \), \( R \) is the hydraulic radius in \( m \), \( P \) is the wetted perimeter in \( m \), \( s \) is the (energy) gradient of the channel, \( b' \) is the 'average' bed width in \( m \), \( y \) is the water depth in \( m \), \( v \) is the kinematic viscosity in \( m^2/s \).

It is obvious that this set of formulae is not very practical. Moreover, there are 6 equations against 3 unknown variables, i.e. \( b', \, y, \, s \). Graf (1971) skipped three of the above equations and has re-written the others. Moreover, the equations can be processed into a more practical shape, see box 3.5.

**Box 3.5. Further processing of the Simons and Albertson equations.**

Henderson simplified the Simons & Albertson equations by skipping three of them, and re-writing the others into (Graf 1971):

- **equation 1:** \( y = 1.21 \, K_2 \, Q^{0.36} \) for \( R \leq 2.13 \, m \)
  
or \( y = 0.93 \, K_1 \, Q^{0.36} + 0.61 \) for \( R \geq 2.13 \, m \)
- **equation 2:** \( A/y = 0.9 \, K_1 \, \sqrt{Q} \)
- **equation 3:** \( v^2 = K_4 \, g \, y \, s \left( v \cdot 0.9 \, K_1 \, \sqrt{Q} / v \right)^{0.37} \)

Even this set of formulae is not very practical for the design engineer:

- **equation 2** can be further processed as \( A/y = 0.9 \, K_1 \, \sqrt{Q} \) can be re-written through \( A/y = (b + m \, y) \) into: \( b = 0.9 \, K_1 \, \sqrt{Q} - m \, y \);
- **equation 3** can be converted into: \( s = v^2 / (K_4 \, g \, y) \times (v \cdot 0.9 \, K_1 \, \sqrt{Q})^{0.37} \times v^{0.37} = \)
  \[ = Q^{0.37-0.19} \cdot A^{2+0.37} \cdot K_4^{-1} \cdot G^{-1} \cdot y^{-1} \times 0.9 \cdot 0.37 \cdot K_1 \cdot v^{1.37} \times v^{0.37} = \]
  \[ = Q^{1.44} \cdot (0.9 \, K_1 \, \sqrt{Q})^{1.63} \cdot K_4^{-1} \cdot G^{-1} \cdot y^{-1} \times 0.9 \cdot 0.37 \cdot K_1 \cdot v^{1.37} \times v^{0.37} = \]
  \[ = Q^{1.44-0.82} \cdot K_4^{-1} \cdot G^{-1} \cdot y^{-1} \times 0.9 \cdot 0.37 \cdot K_1 \cdot v^{1.37} \times v^{0.37} = \]
  \[ = Q^{0.62} \cdot K_4^{-1} \cdot G^{-1} \cdot y^{-2.63} \times 0.9 \cdot 2 \cdot K_1 \cdot v^{1.37} = \]
  \[ = \left( 0.9^2 \cdot K_1^{-2} \cdot K_4^{-1} \cdot G^{-1} \cdot v^{0.37} \right) Q^{0.62} \cdot y^{-2.63} = K_5 \, Q^{0.62} \cdot y^{-2.63}. \]

Simplified Simons & Albertson equations. Thus, the three simplified Simons & Albertson equations depend on the design discharge \( Q \) in \( m^3/s \), the temperature \( t \) in \( ^\circ C \), and three coefficients \( K_1, \, K_2 \, \text{and} \, K_5 \) see also table 3.10:

- **equation 1:** \( y = 1.21 \, K_2 \, Q^{0.36} \) for \( R \leq 2.13 \, m \)
  
or \( y = 0.93 \, K_2 \, Q^{0.36} + 0.61 \) for \( R \geq 2.13 \, m \)
- **equation 2:** \( b = 0.9 \, K_1 \, \sqrt{Q} - m \, y, \text{ and check } R = (b+m)\sqrt{y} / \{b + 2y \sqrt{(1+m^2)} \} \)
- **equation 3:** \( s = K_5 \, Q^{0.62} \cdot y^{-2.63} \)

where: \( Q \) is the design discharge in \( m^3/s \), \( R \) is the hydraulic radius in \( m \), \( s \) is the (energy) gradient of the channel, \( b \) is the bed width in \( m \), \( y \) is the water depth in \( m \), \( m \) is the side slope (1Vert : 1Hor), \( v \) is the kinematic viscosity in \( m^2/s \) for \( t \) \( ^\circ C \).
Kinematic viscosity. The kinematic viscosity \( \nu \) depends on the temperature:

\[ \nu = 40 \times 10^{-6} / (20 + t) \text{ m}^2/\text{s} \text{ for } t \text{ °C.} \]

The effect of the temperature on the kinematic viscosity is normally small. Thus, \( \nu = 1.15 \times 10^{-6} \text{ m}^2/\text{s} \) for \( t = 15 \text{ °C} \) is assumed in the following.

### Table 3.10. Coefficients and exponents for Simons & Albertson equations.

<table>
<thead>
<tr>
<th></th>
<th>GRAVEL BANKS &amp; GRAVEL BED</th>
<th>SAND BANKS &amp; SAND BED</th>
<th>COHESIVE BANKS &amp; SAND BED</th>
<th>COHESIVE BANK &amp; COHESIVE BED</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>+low sedim.</td>
<td>+heavy sedim.</td>
<td>+heavy sedim.</td>
<td></td>
</tr>
<tr>
<td>( n )</td>
<td>0.29</td>
<td>0.33</td>
<td>0.33</td>
<td>0.29</td>
</tr>
<tr>
<td>( K1 )</td>
<td>3.17</td>
<td>6.34</td>
<td>4.71</td>
<td>3.08</td>
</tr>
<tr>
<td>( K2 )</td>
<td>0.25</td>
<td>0.57</td>
<td>0.48</td>
<td>0.37</td>
</tr>
<tr>
<td>( K3 )</td>
<td>5.46</td>
<td>4.24</td>
<td>4.88</td>
<td>4.88</td>
</tr>
<tr>
<td>( K4 )</td>
<td>n.a.</td>
<td>0.33</td>
<td>0.54</td>
<td>n.a.</td>
</tr>
<tr>
<td>( K5 )</td>
<td>n.a.</td>
<td>( 60 \times 10^{-6} )</td>
<td>( 67 \times 10^{-6} )</td>
<td>n.a.</td>
</tr>
</tbody>
</table>

### 3.4.5. Discussion on the Regime Method

Many attempts. Many attempts have been made to develop the regime method to the channel design. Many authors have developed their own regime formulae, such as Inglis in 1946, Lane in 1953, Henderson in 1961, Simons & Albertson in 1963, Blench in 1966, etc., etc.

General validity. It is still questionable whether the regime equations are of a general validity ("geldigheid"). Obvious, there are the following disadvantages:

- there are large discrepancies between the results from various equations when applied under similar conditions, see box 3.6;
- the channels designed with the regime method have flatter gradients than the tractive force theory would allow. This will lead to the construction of several drop structures in the channel, that could have been avoided by using the tractive force method;
- the regime method provides normally for more equations than required for solving the three design parameters, i.e. the bed width \( b \), the water depth \( y \) and the gradient \( s \). It means that the Strickler coefficient \( k \) can be calculated from the Strickler formula. This is contradictory to common understanding as the Strickler coefficient depends on physical parameters, such as soil type and maintenance.

The limitations of the regime method can be shown in the examples in box 3.6 and box 3.7.

Conclusion. There seems to be no agreement in the various regime equations. It seems obvious that the regime method can only be used for very special conditions for which the equations and its coefficients have been developed.
Box 3.6. Example of regime method for the design of a drainage channel.

A drainage channel with a design discharge of 10 m$^3$/s and for side slopes of $m = 2$ ($V_{vert} = 2V_{hor}$) have to be designed with the regime method. There are different regime equations, each requiring additional information:

- for the Haringhuizen equations, a Strickler coefficient $k = 40$ m$^{1/3}$/s is added. Moreover, there are the options of (i) a flat alignment, and (ii) a steep alignment;
- for the Lacey equations, there are the options of (i) medium sand with an average diameter of $D_m = 1.5$ mm, (ii) silt of the Punjab, India, with diameter $D_m = 0.40$ mm, and (iii) fine silt with diameter $D_m = 0.06$ mm;
- for the Simons & Albertson equations, there are the options for (i) sand bed and sand banks, (ii) sand bed and cohesive banks, with low sediment load, and (iii) cohesive bed and cohesive banks.

The results are presented below:

<table>
<thead>
<tr>
<th>REGIME CONCEPT</th>
<th>acc. Strickler formula</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bed width $b$ in m</td>
</tr>
<tr>
<td>HARINGHUIZEN FORMULAE</td>
<td></td>
</tr>
<tr>
<td>flat alignment</td>
<td>7.50</td>
</tr>
<tr>
<td>steep alignment</td>
<td>6.90</td>
</tr>
<tr>
<td>LACEY FORMULAE</td>
<td></td>
</tr>
<tr>
<td>medium sand ($D_m = 1.5$ mm)</td>
<td>11.02</td>
</tr>
<tr>
<td>Indian silt ($D_m = 0.40$ mm)</td>
<td>9.58</td>
</tr>
<tr>
<td>fine silt ($D_m = 0.06$ mm)</td>
<td>6.08</td>
</tr>
<tr>
<td>SIMONS &amp; ALBERTSON FORMULAE</td>
<td></td>
</tr>
<tr>
<td>sand banks &amp; sand bed</td>
<td>14.88</td>
</tr>
<tr>
<td>cohesive banks &amp; sand bed</td>
<td>10.74</td>
</tr>
<tr>
<td>cohesive banks and bed</td>
<td>9.05</td>
</tr>
</tbody>
</table>

Box 3.7. Example of regime method for the design of a flood channel.

A flood channel with a capacity of 100 m$^3$/s, side slopes $V_{vert} = 2V_{hor}$, and a Strickler coefficient $k = 40$ m$^{1/3}$/s has to be designed. The cohesive bed material allows a maximum tractive force of $T_{critical} = 6$ N/m$^2$.

The design according to the Lacey equations leads to a bed width $b = 32$ m, a water depth $y = 3.63$ m and a gradient $s = 0.03 \times 10^{-3}$. The resulting bed tractive force $T = 1.1$ N/m$^2$ is much lower than the maximum tractive force. The Strickler coefficient $k = 59$ m$^{1/3}$/s can be calculated from the Strickler formula, and does not match with the required Strickler coefficient.

The design according to the Simons & Albertson equations leads to a bed width $b = 30$ m, a water depth $y = 2.61$ m and a gradient $s = 0.08 \times 10^{-3}$. Now, the bed tractive force $T = 2.1$ N/m$^2$ and the Strickler coefficient $k = 71$ m$^{1/3}$/s, which are both not satisfactory.
4. SUPER-CRITICAL FLOW IN OPEN-CHANNELS

4.1. Hydraulics of Super-Critical Uniform Flow

4.1.1. Critical gradient of Open-Channels

General. Normally, hydraulic engineers are designing unlined open-channels for sub-critical flow only and not for super-critical flow. Super-critical flow would lead to excessive scouring and would require protection of bed and slopes by lining. The hydraulic engineers apply flow formulae such as the Strickler formula, and design criteria such as maximum permissible tractive force $T$ in N/m$^2$ to avoid scouring.

Only in specific cases, such as in chutes, the engineer will allow a super-critical flow in open-channels. Chutes are open-channels with super-critical flow, see figure 4.1. Usually, they are designed in concrete with vertical side slopes.

![Diagram of a chute with super-critical flow](image)

Figure 4.1. A chute with super-critical flow.

Critical flow. Critical flow in a channel is reached when the channel gradient equals the 'critical gradient' $s_c$, see figure 4.2. Steeper gradients lead to super-critical flow, and flatter gradients to sub-critical flow. Open-channels at just the critical gradient should be avoided as such a flow is characterized by flow instabilities such as waves.
Channel Gradient \( s \) smaller than Critical Gradient \( s_c \): Sub-Critical Flow

Channel Gradient \( s \) is equal to Critical Gradient \( s_c \): Critical Flow

Channel Gradient \( s \) larger than Critical Gradient \( s_c \): Super-Critical Flow

Figure 4.2. Type of flow in an open-water channel, depending on channel gradient.

Although the hydraulic literature extensively discusses the critical flow in open channels (e.g. Henderson 1966, Chanson 1999), no practical feeling is given to design engineers about the critical gradient \( s_c \) in a typical channel.

Critical depth. The critical water depth \( y_c \) normal to the bed in open-channels with trapezoidal cross-sections can be derived from the specific head equation. The specific energy head \( H \) of channels under an angle \( \theta \) in radials is defined by the 'specific energy equation for sloping glacis:

\[
H = y \cos \theta + \frac{\alpha Q^2}{2g A^2}
\]

with the water depth \( y \) normal to the bed in m, the discharge \( Q \) in \( m^3/s \), the wet cross-sectional area \( A \) in \( m^2 \), the velocity distribution coefficient \( \alpha = 1 \), the channel angle \( \theta \) in radials, and the gravity acceleration \( g = 9.8 \) \( m/s^2 \).

The criterium that the maximum discharge \( Q \) for a certain energy head \( H \) occurs during critical flow can be converted into the inverted statement: the minimum value of the energy head \( H \) is reached during critical flow. This leads finally to the equation (e.g. Montes 1998, Ankum 2002):

\[
y_c = -\frac{b}{2m} + \sqrt{\left(\frac{b}{2m}\right)^2 + \left(\frac{Q^2}{m^3g} (b + 2m y_c)\right)^{1/3}}
\]
with the critical water depth \( y_c \) in m, the bed width \( b \) in m, the side slope \( m \) (\( 1_{\text{vert}} : m_{\text{hor}} \)), the discharge \( Q \) in m\(^3\)/s, the channel angle \( \theta \) in radials, and the gravity acceleration \( g = 9.8 \) m/s\(^2\).

The value of the critical water depth \( y_c \) normal to the bed can be calculated by iteration: enter \( y_c = 0 \) in the second part of the equation and calculated a new \( y_c \), which can be entered as the new input.

**Critical gradient.** The critical gradient \( s_c \) of an open-channels can be calculated for a given discharge \( Q \) when the dimensions of the cross-section are known. There are two equations available: (i) the above formula for the calculation of the critical depth \( y_c \) for a certain channel angle \( \theta \) in radials, and (ii) the Strickler formula. Thus, the equation for the critical gradient \( s_c \) can be found (Ankum 2002):

\[
  s_c = \frac{Q^2}{k^2} \left[ \frac{b + 2y_c \sqrt{1 + m^2}}{(b + m y_c)^{1/3}} \right]^{1/3}
\]

with the discharge \( Q \) in m\(^3\)/s, the Strickler coefficient \( k \) in m\(^{1/3}\)/s, the critical water depth \( y_c \) in m, the bed width \( b \) in m, the side slope \( m \) (\( 1_{\text{vert}} : m_{\text{hor}} \)), and the gravity acceleration \( g = 9.8 \) m/s\(^2\).

**Typical values.** The two above equations for the critical depth \( y_c \) normal to the bed and for the critical gradient \( s_c \) has been used for channels with side slopes \( 1_{\text{vert}} : 2_{\text{hor}} \) and a Strickler coefficient \( k = 40 \) m\(^{1/3}\)/s. The critical gradient \( s_c \) for these channels have been plotted in figure 4.3.

It can be seen that critical flow for design discharges \( 1 < Q < 10 \) m\(^3\)/s occurs at channel gradients \( s \approx 10 \times 10^{-3} \). Generally, the high tractive forces \( T = \rho g y s \) will prohibite such a design because of the excessive tractive forces.

![Figure 4.3](image)

Figure 4.3. The critical gradient \( s_c \) for channels with side slopes \( 1_{\text{vert}} : 2_{\text{hor}} \) and a Strickler coefficient \( k = 40 \) m\(^{1/3}\)/s (Ankum 2002).
4.1.2. Hydraulic Parameters of Aerated Flow

Aerated flow. Chutes are used to destroy the excessive energy of the water in canals on steep slopes, see figure 4.4. They are designed at chute gradients $s = \tan \theta$ that are steeper than the critical gradient $s_c$ in order to force super-critical flow.

The high speed flow in chutes is characterized by super-critical flow and by aeration which increases the wet cross-sectional area. The typical appearance of 'white water' is an air-water mixture by air entrainment after a certain transition length.

Figure 4.4. Development of the aerated flow in a chute.

Transition flow. After passing over the weir crest, the water is accelerated by gravity over the chute. First, the flow does not yet entrain air and the flow remains 'black water', see also figure 4.4. In this transition length, a turbulent boundary layer is generated by bottom friction and develops in the flow direction. When this boundary layer reaches the free water surface, the flow becomes aerated and the 'white water' flow begins.

Parameters of the air-water mixture. The water is mixed with air by which its volume is increased, see figure 4.5. The amount of air is usually defined in terms of the air concentration ratio $u$, which is the ratio between the air volume $V_{air}$ to the volume of the air-water mixture $V_{mixture}$, so (e.g. Chow 1959):

$$u = \frac{V_{air}}{V_{mixture}}$$

The depth $y$ of the air-water mixture normal to bed depends on the air concentration ratio $u$. The 'pure' water depth $y_{water}$ can be calculated from:

$$y_{water} = (1 - u) y$$

The air-water mixture has an apparent velocity $v$ which follows from:

$$v = \frac{Q}{y b}$$

The (apparent) velocity $v$ of the air-water mixture is smaller than the real water velocity $v_{water}$, as:

$$\frac{v_{water}}{v} = \frac{y}{y_{water}}$$

hence:

$$\frac{v_{water}}{v} = \frac{1}{1 - u}$$
with the real water velocity $v_{\text{water}}$ in m/s, the (apparent) velocity $v$ of the air-water mixture in m/s, the depth $y$ of the air-water mixture in m, the pure water depth $y_{\text{water}}$ in m, and the air concentration ratio $u$.

Hager (1991) developed the following empirical expression for the air concentration $u$ on the basis of the work done by Straub and Anderson in 1958:

$$ u = 0.75 \left( \sin \theta \right)^{0.75} $$

with the air concentration ratio $u$ and the angle $\theta$ of the chute.

Figure 4.5. Air-water mixture of the aerated flow in a chute.

**Roughness of chutes.** Initial research by Vreedenburgh in 1924 found that the roughness of chutes is smoother than of channels with the same bed and slope material (Vreedenburgh 1924). He studied the flow of 'pure' water in chutes, and he found that the Strickler coefficient $k$ was increased to 146% and 157% for channel angles $\theta = 14^\circ$ ($s = 250 \times 10^{-3}$) and $\theta = 37^\circ$ ($s = 750 \times 10^{-3}$), respectively. A similar effect of steeper angles $\theta$ is confirmed by recent research work (e.g. Wood in 1991).

However, the present literature does not give a clear insight on the ratio between the Strickler coefficient $k_{\text{water}}$ during super-critical flow and the Strickler coefficient $k$ of the sub-critical flow (Ankum 2002).

### 4.1.3. Uniform Super-Critical Flow in Chutes

**Vreedenburgh formula.** Vreedenburgh (1924) recognized that the air-water mixture in a chute has a greater water depth $y$ than the water depth $y_{\text{water}}$ of 'pure' water in the chute. He proposed that the uniform flow in a chute would follow the Strickler formula, but with a different (Strickler) coefficient than for sub-critical flow. Although the steeper gradient $s$ would increase the Strickler coefficient for 'pure' water, he introduced a smaller Strickler coefficient $k_T$ for the air-water mixture. This reduced coefficient $k_T$ would lead to the larger water depth in the Strickler formula because of the larger volume of the air-water mixture.

Vreedenburgh measured the water depth $y$ of the aerated flow in existing chutes for a known discharge $Q$ and a known chute gradient $s$. He calculated the (apparent) Strickler coefficient $k_T$ and compared it with the Strickler coefficient $k$ for similar lining in channels with low gradients. Vreedenburgh (1924) found the relation for $0 < s < 750 \times 10^{-3}$ ($0^\circ < \theta < 37^\circ$):

$$ k_T = k \left( 1 - \sin \theta \right) $$

with the angle $\theta = \text{atan} \ s$

Thus, the water depth $y$ of aerated flow can be calculated by the Vreedenburgh formula:

$$ Q = k \left( 1 - \sin \theta \right) A R^{2/3} \sin^{1/2} \theta $$
with: \( A = b \times y \), \( R = \frac{b \times y}{b + 2\ y} \) and: \( \theta = \text{atan } s \)

with the discharge \( Q \) in \( \text{m}^3/\text{s} \), the (sub-critical) Strickler coefficient \( k \) in \( \text{m}^{1/3}/\text{s} \), the wet cross-sectional area \( A \) of the air-water mixture in \( \text{m}^2 \), the hydraulic radius \( R \) of the air-water mixture in \( \text{m} \), the chute width \( b \) in \( \text{m} \), the aerated water depth \( y \) in \( \text{m} \), the chute angle \( \theta \), and the chute gradient \( s \).

Present literature. The present literature does not provide a simple formula for a super-critical uniform flow, like the well-known formulae for the sub-critical uniform flow. Instead, it recommends a somewhat cumbersome approach with a set of formulae.

These methods (e.g. Hager 1991) determine the water depth \( y \) of the air-water mixture from the pure water depth \( y_{\text{water}} \) by considering the air concentration ratio \( u \):

\[
y = \frac{y_{\text{water}}}{(1 - u)} \quad \text{with: } u = 0.75 (\sin \theta)^{0.75}
\]

The pure water depth \( y_{\text{water}} \) follows from the Strickler formula with the Strickler coefficient \( k_{\text{water}} \) for pure water during super-critical flow:

\[
Q = k_{\text{water}} A_{\text{water}} R_{\text{water}}^{2/3} \sin^{1/3} \theta
\]

with: \( A_{\text{water}} = b \times y_{\text{water}} \), \( R_{\text{water}} = \frac{b \times y_{\text{water}}}{b + 2\ y_{\text{water}}} \) and: \( \theta = \text{atan } s \)

with the discharge \( Q \) in \( \text{m}^3/\text{s} \), the Strickler coefficient \( k_{\text{water}} \) for pure water in \( \text{m}^{1/3}/\text{s} \), the wet cross-sectional area \( A_{\text{water}} \) of the pure water in \( \text{m}^2 \), the hydraulic radius \( R_{\text{water}} \) of pure water in \( \text{m} \), the chute width \( b \) in \( \text{m} \), the pure water depth \( y_{\text{water}} \) in \( \text{m} \), the chute angle \( \theta \), and the chute gradient \( s \). The Strickler coefficient \( k_{\text{water}} \) for pure water is not further specified in literature, but is not equal to the Strickler coefficient \( k \) for sub-critical flow in \( \text{m}^{1/3}/\text{s} \).

Discussion. The approach by Vreedenburgh and the approach in the present literature agree quite well for chute gradients \( s < 700 \times 10^{-3} \) (Ankum 2002).

It is obvious that the approach by Vreedenburgh is more direct than the approaches followed by the present literature. Thus, it is recommended here to use the Vreedenburgh formula (1924).

### 4.2. Design of Chutes

#### 4.2.1. Water Depth in Chutes

Calculation of the water depth. It is recommended here to use the Vreedenburgh formula (1924), which is basically the modified Strickler formula, for the aerated uniform flow in chutes:

\[
Q = k (1 - \sin \theta) A R^{2/3} \sin^{1/3} \theta
\]

with: \( A = b \times y \), \( R = \frac{b \times y}{b + 2\ y} \) and: \( \theta = \text{atan } s \)

with the discharge \( Q \) in \( \text{m}^3/\text{s} \), the (sub-critical) Strickler coefficient \( k \) in \( \text{m}^{1/3}/\text{s} \), the wet cross-sectional area \( A \) of the air-water mixture in \( \text{m}^2 \), the hydraulic radius \( R \) of the air-water mixture in \( \text{m} \), the chute width \( b \) in \( \text{m} \), the aerated water depth \( y \) in \( \text{m} \), the chute angle \( \theta \), and the chute gradient \( s \).
The calculation of the water depth $y$ of the air-water mixture in a chute follows from the above formulae:

$$Q = k \left(1 - \sin \theta \right) \frac{b^{5/3} y^{5/3}}{(b + 2y)^{2/3}} \sin^{1/2} \theta$$

which can be re-written into:

$$y^{5/3} = \frac{Q}{b^{5/3} k \left(1 - \sin \theta \right) \sin^{1/2} \theta} \frac{1}{(b + 2y)^{2/3}}$$

Hence, the water depth $y$ of the air-water mixture follows from:

$$y = \frac{Q^{0.6} (b + 2y)^{0.4}}{k^{0.6} b \left(1 - \sin \theta \right)^{0.6} \sin^{0.3} \theta}$$

This equation is solved by iteration: assume $y = 0$, calculate the new $y_{\text{new}}$ and enter this $y$ again the equation, until $y_{\text{new}} = y$. An example calculation of the water depth of the air-water mixture of a chute is presented in box 4.1.

**Box 4.1. Calculation of the water depth of the air-water mixture in a chute.**

A drainage channel with a design discharge of 5 m$^3$/s follows a steep alignment. A concrete chute is used to dissipate the energy. This chute is 300 m long and the energy head drops by $z = 15.00$ m over this distance. Thus, the gradient of the chute is $s = 50 \times 10^{-3}$, and $\theta = \tan 0.05 = 2.86^\circ (= 0.05$ radials).

The concrete has a (sub-critical) Strickler coefficient $k = 70$ m$^{1/3}$/s. The cross-section of the chute is designed at a width of $b = 2.00$ m and with vertical walls.

The water depth $y$ of the air-water mixture follows from the equation:

$$y = \frac{Q^{0.6} (b + 2y)^{0.4}}{k^{0.6} b \left(1 - \sin \theta \right)^{0.6} \sin^{0.3} \theta} = \frac{5^{0.6} (2 + 2 \times y)^{0.4}}{70^{0.6} 2 (1 - 0.05)^{0.6} 0.05^{0.3}} = 0.343 (1 + y)^{0.4}$$

The equation is solved by iteration: assume $y = 0$, calculate the new value $y_{\text{new}} = 0.343$ m, and enter this $y$ again into the equation and calculate the next $y_{\text{new}}$. Finally, a water depth of the air-water mixture of a chute is calculated at $y = 0.392$ m.

When the gradient of the chute is increased to $s = 500 \times 10^{-3}$ ($\theta = 26.6^\circ$), a water depth $y = 0.271$ m can be calculated.

### 4.2.2. Energy Head in Chutes

**Real velocity of the water.** The (real) velocity of the pure water $v_{\text{water}}$ is higher than the (apparent) velocity $v$ of the air-water mixture, see figure 4.6. The real water velocity $v_{\text{water}}$ is based on the the air concentration $u$, which has been experimental determined by Hager (1991):

$$v_{\text{water}} = \frac{v}{1 - u} \quad \text{with:} \quad u = 0.75 \sin^{0.75} \theta \quad \text{and:} \quad \theta = \tan s$$

and the (apparent) water velocity $v$ of the air-water mixture:

$$v = \frac{Q}{b \times y}$$

where $v_{\text{water}}$ is the (real) velocity of the pure water in m/s, $v$ is the velocity of the air-water mixture in m/s, the air concentration ratio $u$, $Q$ is the discharge in m$^3$/s, $b$ is the width of the rectangular chute in m, $y$ is the water depth of the air-water mixture in m, the chute angle $\theta$, and the chute gradient $s$. 

Figure 4.6. The energy head $H$ of the (pure) water flow in a chute.

Box 4.2. Calculation of the energy head $H$ in a chute.

The real water velocity $v_{\text{water}}$ of pure water in the chute of the above example with a gradient of $s = 50 \times 10^{-3}$ ($\theta = 2.86^\circ$) depends on the (apparent) water velocity $v$: 

$$v_{\text{water}} = \frac{Q}{b \cdot y} = \frac{5}{2 \times 0.392} = 6.38 \text{ m/s}$$

and the air contraction ratio $u$:

$$u = 0.75 \sin^{0.75} \theta = 0.75 \sin^{0.75} 2.86^\circ = 0.079 = 7.9\%$$

and can be calculated by:

$$v_{\text{water}} = \frac{v}{1 - u} = \frac{6.38}{1 - 0.079} = 6.93 \text{ m/s}$$

And the energy head $H$ can be calculated from the following equation:

$$H = y (1 - u) \cos \theta + \frac{(v_{\text{water}})^2}{2g} = 0.361 + 2.45 = 2.81 \text{ m.}$$

When the gradient of the chute is increased to $s = 500 \times 10^{-3}$ ($\theta = 26.6^\circ$), a real water velocity $v_{\text{water}} = 15.7 \text{ m/s}$ can be calculated, and an energy head $H = 12.7 \text{ m.}$
Energy head. The energy head $H$ in the chute is not related to the apparent velocity $v$ of the air-water mixture but on the real water velocity $v_{\text{water}}$ of pure water. Thus, the energy head $H$ above the bed of the chute is defined by the 'specific head equation', see also figure 4.6:

$$H = y_{\text{water}} \cos \theta + \frac{(v_{\text{water}})^2}{2g} \quad \text{with: } \theta = \arctan s$$

and the pure water depth $y_{\text{water}}$:

$$y_{\text{water}} = (1 - u) y \quad \text{with: } u = 0.75 \sin^{0.75} \theta \quad \text{and: } \theta = \arctan s$$

with the energy head $H$ above the bed of the chute in m, the pure water depth $y_{\text{water}}$ in m, the (real) velocity $v_{\text{water}}$ of the pure water in m/s, the air concentration ratio $u$, the water depth $y$ of the air-water mixture in m, the chute angle $\theta$, the chute gradient $s$, and the gravity acceleration $g = 9.8 \, \text{m/s}^2$. An example is presented in box 4.2.

### 4.2.3. Transition Length of Chutes

Performance as 'chute' or as 'drop structure'? Although an open-channel at a steep slope is constructed, it is not yet guaranteed that it functions as a 'chute' with sub-critical uniform flow. The condition for sub-critical uniform flow is that the sloping energy level is parallel to the bed level of the chute. Thus, there is dissipation by friction on the sloping glacis, see figure 4.7.

For instance, steep and short flumes, or narrow flumes, are not functioning as a 'chute' with energy dissipation on the sloping glacis. They are too short or too smooth, so that the friction loss is very small and should be ignored. Thus, they function as a 'drop structure', where the energy dissipation takes place only in the stilling basin by the hydraulic jump and eddies.

**Figure 4.7.** Energy dissipation by a 'chute' and by a 'drop structure'.
Analysis. The analysis, whether a fall structure performs as a 'chute' or as a 'drop structure', can be simplified by considering 'energy lines'.

A chute has an energy line parallel to the sloping glacis, at a vertical distance $H$. This vertical distance $H$ can be calculated with the above formulae.

However, when this calculated energy line of the glacis is located higher than the available energy line of the upstream canal reach, it means that an unrealistic situation is created, see figure 4.8. Thus, the following conclusions can be drawn (Ankum 2002):

- uniform flow in the chute is not yet reached,
- the gradient of the energy line in the chute is less than the gradient of the glacis,
- the energy loss is smaller than expected in a chute and is still in the 'transition'. It is better to ignore the friction losses, and to decide that the structure functions as a 'drop structure' only.

![Diagram of chute-flow](image)

Figure 4.8. 'Chute-flow' with friction losses is not yet reached.

Analysis step-by-step. The following steps can be taken to obtain a first impression, see figure 4.9 (Ankum 2002):

- draw the (horizontal) energy line of the upstream canal reach in the longitudinal section of the chute,
- calculate the energy head $H$ above the sloping bed of the chute,
- find the location on the chute where the energy head $H$ coincide with the horizontal energy line of the upstream reach. This intersection point between the two energy lines may be called point A,
- the preliminary elevation of energy line in the chute follows the horizontal line until point A, and continuous with the sloping energy line on a distance $H$ above the bed of the chute,
- friction losses occur only beyond point A,
- if point A is not existing within the range of the chute, the friction losses have to be ignored and the structure does not function as a 'chute'. The uniform chute-flow has apparently not yet been reached and the flow may still belong to the transition part.
Figure 4.9. Checking on the 'chute-flow' with friction losses.

Transition length. The first part of the chute is a transition between the 'critical flow' at the control and the 'uniform and super-critical flow' in the chute. The water depth should be calculated with a backwater curve technique, which is not discussed here.

The horizontal length $L_{\text{trans}}$ of the transition zone before a uniform flow is reached in a chute depends on, see also figure 4.9 (Ankum 2002):

$$\frac{H - H_c}{L_{\text{trans}}} = \frac{s}{1}$$

hence:

$$L_{\text{trans}} = \frac{H - H_c}{s}$$

with the (horizontal) transition length $L_{\text{trans}}$ in m, the energy head $H_c$ at the control above the crest, the energy head $H$ above the sloping bed of the chute, and the chute gradient $s$. An example of the calculation is presented in box 4.3.

Box 4.3. Calculation of the transition length $L_{\text{trans}}$ of a chute.

The transition length $L_{\text{trans}}$ of the chute of the above examples can be calculated for the following design parameters:

* the energy head at the control notch at the beginning of the chute follows from the formula: $Q = 2.0 \ b \ H_c^{3/2}$, and amounts to $H_c = 1.16$ m for $b = 2.00$ m and $Q = 5$ m$^3$/s. A sill of $p = 1.00$ m is applied to avoid backwater curves in the upstream channel reach;

* the energy head $H$ above the sloping bed of the chute amounts to $H = 1.78$ m;

* the gradient of the chute is $s = 20 \times 10^{-3}$.

Thus, the transition length $L_{\text{trans}} = \frac{(H - H_c)}{s} = \frac{(1.78 - 1.16)/(20 \times 10^{-3})}{31.00}$ m. The actual transition length will be somewhat longer, but it is obvious the chute with the length of 300 m will function as a chute with energy losses by friction.
5. HYDRAULIC STRUCTURES

5.1. Types of Hydraulic Structures

5.1.1. Non-Regulating structures

Classification. 'Non-regulating structures' ("niet-regelende kunstwerken") are structures that transport the approaching flow, and do not have the primary function of regulating the water level or the discharge.

Non-regulating structures in flood and drainage channels can be divided into two different groups depending on the headloss:

- **conveyance structures** ("doorvoerkunstwerken"). Conveyance structures are required at crossings of flood and drainage channels with roads and irrigation canals, see figure 5.1. These structures are designed at a low headloss for all discharges.
  
  Examples are bridges, culverts ("duikers"), aqueducts and (inverted) siphons ("sifons, grondduikers"). But also, 'gated regulators' ("regelkunstwerken met afsluitmiddelen") with low headlosses are designed as a 'conveyance structure' during the maximum discharge.

![Figure 5.1](image)

**Figure 5.1.** 'Conveyance structure', with low headloss for all discharges.
• **drop structures.** Drop structures ("stortedammen") are required to dissipate the excessive energy at steep alignments to avoid erosion in unlined open-channels, see figure 5.2. These structures are designed at a **high headloss** for all discharges. As an alternative, 'chutes' ("hellende goten") can also be applied, which are just lined open-water channels with super-critical flow. Drop structures are discussed in chapter 12. Chutes have been discussed in chapter 4 on 'Super-critical flow in open-channels'.

![Diagram of drop structure](image)

**Figure 5.2.** 'Drop structure', with high headloss for all discharges.

### 5.1.2. Regulating structures

**Function.** 'Regulating structures' ("regelkunstwerken") are structures that may regulate:
- the water level, or
- the discharge.

**Water level regulator.** The structure that regulate the water level is called the 'water level regulator' ("peilregelaar"), also called 'check structure' or 'cross regulator' ("stuw"), see figure 5.3. The function of the water level regulator is to maintain a certain target water level, i.e. the 'setpoint-in-water-level' ("streepeil"), at every discharge;

![Diagram of water level regulator](image)

**Figure 5.3.** 'Water level' regulator, with the water level as target.

**Regulation of the water level.** The regulation of the water level is theoretically easy. The discharge formula of the water level regulator is not relevant for the regulation as the gate will be regulated simply by opening or by closing until the water level reaches the 'setpoint' ("streefwaarde", "streepeil"), see figure 5.4.
Discharge regulator. The structure that regulates the discharge is called the 'discharge regulator' ("debiertregelaar"), also called 'offtake' ("aftapping") or 'intake' ("inlaat"), see figure 5.5. Discharge regulators are structures that regulate the discharge from one channel to another. The function of the discharge regulator is to maintain a certain discharge, i.e. the 'setpoint-in-discharge' ("streefdebiet"), at every water level.

Discharge regulators are widely applied in irrigation and may occur at different places in the main irrigation system: at the 'headworks' ("watervang"), at the head-end of secondary and sub-secondary irrigation canals ("secundaire inlaat"), and at the 'offtakes' ("aftapping") to the tertiary irrigation units.

Discharge regulators are also used in flood control, when a flood diversion structure has to divert a certain flood volume into a flood diversion channel.

Discharge regulators are used less in drainage, where 'water level' control is normally applied.

Figure 5.5. 'Discharge' regulator, with the discharge as target.
Regulation of the 'on/off' discharge. Regulation of the discharge is more difficult than the regulation on the water level. Regulation on discharge implies that also the passing discharge should be known. This is simple for an intermittent water supply, with only an on-off ("aan/uit") function, see figure 5.6. When the regulator is fully closed, the discharge is zero. When the regulator is fully open, the maximum discharge is passing as per design.

![INTERMITTENT FLOW](image)

![ADJUSTABLE FLOW](image)

**Figure 5.6.** Regulation of 'on/off' discharge and of 'different' discharges.

Regulation of the 'different' discharges. Regulation of the different discharges, e.g. 75%, 30%, 55% of the maximum flow, is more combersome as also discharge measurement should be done. Several regulators, such as the vertical gate, have a complicated relation between the gate opening and the discharge. The regulation and the measurement of the discharge can be done in two ways, by:

- **regulation gates with an accurate rating curve**, such as the Romijn weir and the Crump-deGruyter gate;
- **regulating gates with a separate water measurement structure 'in series' ('achter elkaar'**), see figure 5.7. The regulating process of the gate is translated into the control of the 'tailwater level': the target discharge is obtained by adjusting the discharge regulator on basis of the tailwater level.

![DISCHARGE REGULATOR](image)

![DISCHARGE MEASUREMENT](image)

**Figure 5.7.** Regulation of the discharge, through the water level.
5.1.3. Manner of Regulation

Types of regulators. The water level and the discharge regulators can be divided into different groups, depending on the 'manner' of regulation, see figure 5.8:

- **passive regulators** ("passieve regelaars"). Passive regulators are applied to maintain a certain water level or to divert the flow. They also called 'fixed regulators' ("vaste regelaars"). Passive regulators do not have gates, but their function is based on the 'overflow' ("overlaat") or on the 'underflow' ("onderspuier") principle.

  Examples are the 'broad-crested weir' ("brede overlaat"), the 'sharp-crested weir' ("scherpe overlaat"), the 'long-crested weir' ("lange overlaat") or 'duck-bill weir' ("eendebek stuw"), the 'control notch' ("sluis"), the 'side-channel spillway' ("zijdelingse overlaat") and the 'siphon spillway' ("hevel overlaat");

- **manual regulators** ("handbediende regelaars"). Manual regulators are applied to maintain a certain water level or to divert the flow. They have manual-operated gates ("handbediende schuiven"). Manual regulation is applicable as long as the frequency of gate setting is limited. Manual regulation becomes labour-intensive when the flow is regularly changing.

  Examples are the drop-leaf gate ("klepstuw"), the flap gate ("duiker deur?"), the vertical gate ("vlakke schuif"), the Crump-deGruyter gate, and the radial gate ("segment deur");

![Diagram of various types of regulation](image)

**Figure 5.8.** Options for (local) regulation of structures.

- **hydro-mechanical regulators** ("hydro-mechanische regelaars"). Hydro-mechanical regulators are applied to maintain a certain water level. A higher water level leads to higher water forces on the gate, bywhich the gate opens.
Examples are the Begemann gate, the Vlugter gate, the AMIL gate, the AVIS gate and the AVIO gate.

- **electro-mechanical regulators** (*"electro-mechanische regelaars"*). Electro-mechanical regulators are applied to maintain a certain water. A programmable logic controller (PLC) processes the water level measurements by a sensor and instructs a motor that adjusts the gate. Examples are vertical and radial gates with PD, PI and PID-controllers. They will not be discussed here.

**Types.** There are many types of regulators. They can be divided into two groups depending on the flow type, see figure 5.9:
- **Overflow gates** (*"overlaten"*), such as the 'drop-leaf gate', the 'vertical gate for overflow', and the 'Romijn weir'.
- **Underflow gates** (*"onderspuiers"*), such as the 'vertical gate for underflow', the 'radial gate' and the Crump-deGruyter gate.

These types are discussed in the next chapters.

*Figure 5.9. 'Overflow' and 'underflow' type of the vertical gate.*

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**5.2. Structural Design of Hydraulic Structures**

**5.2.1. Scouring Protection**

**General.** All structures in open-water channels need protection against scouring (*"ontgrondingen"*) at the downstream side and against effects by groundwater. These structures need to be protected against:

- **scouring of the downstream channel bed**, by construction of a stilling basin (*"woelbak"*) and of coffers (*"koffers"*);
- **piping of groundwater** (*"onderloopsheid"*), by construction of a sufficient long 'critical seepage path';
- **uplift by groundwater pressure**, by construction of sufficient thick floors.
Stilling basins. It is a good principle to provide any structure where super-critical ("schiëtet") flow may occur, with a 'stilling basin' ("woelbak"), see figure 5.10. This stilling basin will force the hydraulic jump within the structure, so that excessive scouring of the downstream channel bed can be avoided.

The most simple stilling basin is just a lowering of the floor at the downstream end of the structure. An over-depth of e.g. \(0.3 \times y\) the water depth \(y\) is often sufficient.

![Figure 5.10. 'Coffers' and stilling basins for protection of structures.](image)

Coffers. 'Coffers' ("koffers") or 'cut-off walls' are provided at the end of floors and of stilling basins, see also figure 5.10. Usually, coffers are also applied at the upstream side of a floor.

Coffers are basically a 'beam' under the floor, to a depth of e.g. \(4 \times \) the floor thickness. The construction cost of coffers is only minor, while the benefits are high. The function of coffers is multifold:
- coffers protect the downstream channel bed against concentrated groundwater flow, and thus a higher outgoing velocity, see figure 5.11;
- coffers protect the floor against any scouring-holes ("ontgrondingen") in the downstream bed. These scouring-holes makes that the foundation soil below the floor is removed, which makes that the floor collapse. Moreover, these scouring holes will be propagated into the upstream direction, thus shortening the 'critical seepage path';
- coffers increase the critical seepage path of the groundwater flow.

![Figure 5.11. Groundwater flow and the coffer at the end of a stilling basin.](image)
Discussion on rip-rap. It is still a topic between hydraulic engineers whether bed and slope protection has to be applied at the downstream end of a structure.

It is believed that 'rip-rap' ('losse steenstorting') or 'lining' ('bekleding') is essential to protect the downstream channel bed against scouring. The length of such a protection may be in the order of $4 \times$ the downstream water depth $y$.

Others believe that a well-designed stilling basin will reduce the turbulence of the water and will dampen any waves ('golven') of the water, so protection is not needed.

5.2.2. Piping below the Floor

Piping. Flood diversion structures should be checked on the danger of 'piping' ('onderloopsheid'). Seepage ('kwel') will damage a structure when the foundation soil is removed by the flow and holes are formed. With 'piping', a direct connection is created between the upstream and the downstream part of the structure, resulting in more groundwater flow and dangerous losses of foundation soil. Ultimately the structure will collapse. The material transport and the resulting piping can be prevented by, see figure 5.12:

- **lengthening the distance** of seepage flow, so that the gradient of the groundwater flow becomes flatter. This can be effected by e.g.:
  - longer floors. However, the lengthening of the downstream floor will increase the uplift pressure,
  - cut-off walls and sheet piles ('schermen en damwanden')
- **the use of filters**. Good filters will prevent the removal of foundation soil. They can be applied at the end of the structure as part of 'rip-rap'. However, the turbulence in the water may damage filters here. Filters have to be applied in 'weep holes' ('≈ drainage openingen') of lined slopes ('talud bekledingen').

The danger of piping can be checked by the preparation of a flownet, and by several empirical methods such as the Lane's method.

![Diagram of piping in flood diversion structures](image)

Figure 5.12. Protection of structures against 'piping' (DHV 1986).
Lane’s method. The Lane’s method compares the ‘creep ratio’ below the structure with the allowable creep ratio for that soil. The creep ratio is the critical seepage path under the structure, divided by the drop in water level \( z \) over the structure. The ‘critical seepage path’ is the sum of, see figure 5.13:

- the vertical path distance \( \Sigma L_{\text{vert}} \) along the structure,
- one-third of the horizontal path distance \( \Sigma L_{\text{hor}} \),
- two times the short-cut path distance \( \Sigma L_{\text{short-cut}} \) that shortcuts through the soil.

The background is that the vertical path has three times more resistance against flow than the horizontal flow. Slopes steeper than 45° are regarded as vertical, and those less than 45° as horizontal. Thus, the ‘creep ratio’ \( C_L \) follows from:

\[
C_L = \frac{\sum L_{\text{vert}} + \frac{1}{3} \sum L_{\text{hor}} + 2 \sum L_{\text{short-cut}}}{z}
\]

where: \( C_L \) is the creep ratio, \( \sum L_{\text{vert}} \) is the sum of the vertical lengths in m, \( \sum L_{\text{hor}} \) is the sum of the horizontal lengths in m, \( \sum L_{\text{short-cut}} \) is the sum of the shortcut lengths in m, and \( z \) is the headloss over the structure in m.

Typical values for the allowable creep ratio are: \( C_L = 2 \) for hard clay, \( C_L = 3 \) for soft clay, \( C_L = 4 \) for fine gravel, \( C_L = 5 \) for coarse sand, \( C_L = 7 \) for fine sand, and \( C_L = 8 \) for silt. Thus, clay allows for the shortest critical seepage path.

![Diagram](image.png)

**Figure 5.13.** Piping of groundwater below a stilling basin, according to Lane’s method.

**5.2.3. Uplift of the Floor**

Uplift. Floors subject to ‘uplift’ pressure ("opwaartse druk") because of groundwater below the floor, see figure 5.14. Normally, the uplift pressure can be reduced by:

- making a longer floor at the upstream side of the structure;
- applying ‘weep holes’ with filters.

The uplift pressure can be calculated by drawing a flownet or with the assumptions used by the Lane’s critical seepage path.
Lane's method. The Lane's method calculates the uplift pressure \( P_x \) in a point \( x \) by assuming that the drop in pressure over the structure is proportional to the critical seepage path, thus:

\[
P_x = 10000 \left( y_{\text{floor}} + d \right) + \frac{L_x}{\sum L_{\text{vert.}} + 1/3 \sum L_{\text{hor.}} + 2 \sum L_{\text{short-cut}}} \times z
\]

where: \( P_x \) is the uplift water pressure in point \( x \) in N/m², \( y_{\text{floor}} \) is the water depth above the floor in m, \( d \) is the thickness of floor at point \( x \) in m, \( L_x \) is the relative critical seepage path from downstream to point \( x \) in m, \( \sum L_{\text{vert.}} \) is the sum of the vertical lengths in m, \( \sum L_{\text{hor.}} \) is the sum of the horizontal lengths in m, \( \sum L_{\text{short-cut}} \) is the sum of the short-cut lengths in m, and \( z \) is the headloss over the structure in m.

Minimum thickness of floor. The floor of the structure has to resist the uplift pressure of the groundwater. The minimum floor thickness \( d \) follows a stability calculation: "the uplift pressure \( P_x \) should be less than the weight of the floor plus the weight of the water above the floor". A unit weight of 2000 kg/m² is often assumed for the floor. Thus:

\[
20000 \times d + 10000 \times y_{\text{floor}} > P_x
\]

where: \( d \) is the thickness of the floor in m, \( y_{\text{floor}} \) is the water depth above the floor in m, \( P_x \) is the uplift pressure in N/m².

The above formula has to be applied for the stability of masonry ("metselwerk") floors. However, some reduction in floor thickness can be allowed for reinforced concrete floors, where the structure as-a-whole resists the uplift force.

Quick check on uplift. A quick check of the minimum thickness of masonry floors assumes proportional decrease of the water pressure under the floor, without considering the vertical seepage paths. Moreover, an empty stilling basin, thus \( y_{\text{floor}} = 0 \), is assumed. It means that the weight of the floor is determined by:

\[
20000 \times d > 10000 \left( d + \frac{L_x}{\sum L_{\text{hor.}}} \times z \right), \quad \text{hence by:} \quad d > \frac{L_x}{L_{\text{hor.}}} \times z
\]

Graphically, it means that the thickness \( d \) of the floor should be larger than the distance between the floor level and the (straight) water pressure line, see also figure 5.14.

Figure 5.14. Quick check for the uplift water pressure under a stilling basin.
5.2.4. Construction Drawings

Types of drawings. There are different types of drawings during the design of structures. The hydraulic engineer will use 'principe sketches' of the structures, with its hydraulic dimensions given in letters, such as the width $b$, the water depth $y$, the energy head $H$, and the sill height $p$.

Sometimes, there are '3D-drawings' with the artist impression of the structure, see figure 5.15. Often, these 3D-drawings are not very usual for the engineers, as these drawings hide too many details.

The design engineer will produce 'engineering drawings' of the structure, to be used by the drawing office, see figure 5.16. This drawings have a plan ('bovenaanzicht'), one or more longitudinal sections ('lengtedoorsneden'), and one or more cross sections ('dwardsdoorsneden'). The plan and sections are closely linked with each other, and the design engineer draws them simultaneously.

The draughtsmen in the drawing office prepares the 'construction drawings' to be used for the tendering ('aanbesteding') and for the construction by the contractor, see figure 5.17. All relevant details are given in this drawing. Also the measures and the levels are shown.

An 'as-built drawing' is often prepared after the structure is completed by the contractor. All revisions during the construction are shown.

Figure 5.15. Example of a '3D-drawing' of a structure.
Preparation of the 'engineering drawing'. The 'engineering drawing' of a structure can be drawn gradually in a certain sequence, see also figure 5.16.

Firstly, the plan is drawn, on basis of the data from the channel design. The location of the structure is selected. The vertical walls of the structure are drawn, with its transition to the incoming channel.

Secondly, the longitudinal section of the above is drawn. This is done at the same scale as the plan and precisely below the plan. The floor is drawn, and the water level regulator. Also the longitudinal section of the stilling basin is drawn. The 'coffers' or cut-off walls at the up-and downstream end of the floor are drawn.

Thirdly, the plan is adjusted to the above elements in the longitudinal section. The vertical walls of the structures are extended into the downstream direction so that they include the stilling basin. Also the transition of the vertical walls to the continuing channel is drawn.

Fourthly, a check is made whether everything is drawn. A bridge is often needed. The hatching ("arcering") of cross sections is made for soil, concrete, etc. Also the hatching of the slopes is made. The water is not drawn in the plan. The maximum water level is drawn in the longitudinal and in the cross sections.

![Diagram](image)

**Figure 5.16.** Example of an 'engineering drawing' of a structure.
Figure 5.17. Example of a 'construction drawing' of a structure (Nedeco 1986).
Figure 5.18. Example of a 'construction drawing' of a structure (Nedeco 1986).
6. DESIGN OF CONVEYANCE STRUCTURES

6.1. Types of Conveyance Structures

6.1.1. Culverts

Culverts. A 'culvert' ("duiker") is a structure that conveys a drainage channel or an irrigation canal underneath a road, a railway or another water course, see figure 6.1, figure 6.2 and figure 6.3.

There are several possible flow conditions through a culvert, depending on the discharge, the cross section, the length of the culvert and the downstream flow conditions. The flow conditions in the 'barrel' ("koker") can be:
- an open canal flow, with a free water surface,
- a pipe flow, with the barrel completely filled.

Furthermore, the water in the culvert may flow as:
- free flow ("volkomen, schietend") or 'inlet control', for large headlosses, which is basically contradictory to the requirement of a conveyance structure (i.e. low headloss);
- submerged flow ("onvolkomen, verdronken") or 'tailwater control'.

Design. The design of culverts is essentially based on the headloss calculation as presented below, thus for 'tailwater control'. The design velocity may range from 0.5 to 2.0 m/s and determines the headloss $z$ over the structure.

However, 'inlet control' is also possible for a large headloss with supercritical flow in the barrel, see figure 6.1. The inlet control is not discussed here, and should be considered from a hydraulic point of view as a 'drop structure'.

The choice of a single or multi-barrel culvert depends on the bed width of the canal, the water depth of the canal invert and the available headloss for the culvert.

![Figure 6.1. A culvert as a cross-drainage structure, with 'inlet control'.](image-url)

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**Figure 6.2.** A culvert in a canal.

**Figure 6.3.** Types of culverts, and types of culvert-flow (NEDECO 1986).
6.1.2. Aquaducts

Aquaducts. An 'aqueduct' ("aqueduct") is another type of a conveyance structure, that is designed at a low headloss. Aqueducts are used to divert irrigation canal over a drain or even a river, see figure 6.4. The greatest part of the live load is the water in the aqueduct. Thus, the velocity $v$ should be kept high to minimize weight. On the other hand, the velocity $v$ should be kept low as to minimize the headloss $z$. The velocity $v$ should be at least twice the canal velocity to discourage deposition of sediment, but generally should not exceed $v = 3.0$ m/s. The box section of the aqueduct acts as the spanning beam, so an economic design depends on a high ratio $b/y$, where $b$ is the width of the structure and $y$ the water depth. Ratios of $b/y = 1$ to $3$ are often used.

Figure 6.4. An aqueduct (NEDECO 1986).
6.1.3. Siphons

Inverted Siphons. An 'inverted siphon' ("sifon, grondduiker") is also designed normally at a low headloss. Inverted siphons are mainly used in irrigation canals for river crossings where an aqueduct is unsuitable, see figure 6.5.

The decision whether to use an aqueduct or an inverted siphon in an irrigation canal depends on a number of factors, the main one being sediment transport. The siphon will trap all sediment at low flow, as the velocity in the siphon will become much lower than in the canal. Special actions may be required, such as wash-out facilities, or simply gates to close one of the barrels during low discharges.

To prevent siltation and blockage, the velocity $v$ should be kept high. However, a high velocity leads to a high headloss. Velocities should be at least twice the normal canal velocity and in any case not less than 1.5 m/s. Maximum velocity should not exceed 3 m/s.

The top of the siphon entrance is set below the normal water surface. This will minimize possible reduction in the siphon capacity caused by the entering of air into the siphon. The depth of submergence is known as the water "seal". The required height of the seal depends on the slope and the size of the barrel, and can be taken at 1.5 times of the entrance loss, with a minimum of 0.15 m.

'Trashracks' ("krooshkken") have to be installed at the entrance of siphons as to avoid the involuntary entrance by persons or animals and to avoid clogging by floating debris. However, the blocking of the trashrack will increase the entrance headloss.

The inverted siphon will be under internal pressure and hence joints between the pipe elements should not be used. Instead a reinforced concrete collar may be used, preferably cast in-situ.

![Siphon diagram](image)

Figure 6.5. Siphon, to transport water below another water course (NEDECO 1986).
6.2. Hydraulics of Conveyance Structures

6.2.1. Energy Losses

Friction Losses. The 'headloss' ("wrijving") \( z_f \) in a conveyance structure due to the friction can be calculated with the Strickler formula: \( v = k R^{2/3} s^{1/2} \), where \( s = z / L \) is the hydraulic gradient over the structure, thus by:

\[
z_f = \frac{2 g L}{k^2 R^{4/3}} \times \frac{v^2}{2g}
\]

where \( z_f \) is the headloss by friction over the structure in m, \( L \) is the length of structure in m, \( v \) is the velocity in the structure in m/s, \( R \) is the hydraulic radius in m, \( k \) is the Strickler coefficient in m\(^{1/3}\)/s, \( g = 9.8 \) m/s\(^2\) = 9.8 N/kg is the gravity acceleration.

The Strickler coefficient \( k \) may have values of \( k = 50 \) m\(^{1/3}\)/s for stone masonry, to \( k = 70 \) m\(^{1/3}\)/s for concrete.

Transition Losses. The headloss \( z_{in} \) over inlet ("intree") and the headloss \( z_{out} \) over the outlet ("uitree") transition for sub-critical ("stromend, niet-schietend") structures can be calculated by (Chow 1959):

\[
z_{in} = c_{in} \left( \frac{v - v_u}{2g} \right)^2, \quad \text{and} \quad z_{out} = c_{out} \left( \frac{v - v_d}{2g} \right)^2
\]

where: \( c_{in} \) and \( c_{out} \) are headloss coefficients over inlet and outlet, \( v \) is the velocity within the structure in m/s, \( v_u \) is the velocity in the upstream canal in m/s, \( v_d \) is the velocity in the downstream canal in m/s.

In practice, the above formulae are often simplified and are related to the velocity head ("snelheidshoogte") in the structure:

\[
z_{in} = c_{in} \times \frac{v^2}{2g}, \quad \text{and} \quad z_{out} = c_{out} \times \frac{v^2}{2g}
\]

The headloss coefficient depends on the hydraulic shape of the transition and on whether it is an inlet or an outlet, see table 6.1 and figure 6.6.

It is recommended to use vertical-walled transitions of either (i) rounded-flared wall transition (\( R \geq 0.5 \) y), (ii) with straight-line walls under 45° to the canal centerline, or (iii) rectangular walls under 90°. A gradual transition, e.g. 'warped walls' ("vallende muren") can be applied when the transit losses have to be at a minimum. However, the construction of warped walls is somewhat more labour-intensive.

Normally the headloss is so small for these types of transitions that it may be neglected. However, a minimum headloss of 0.05 m is taken at structures with transitions to cover possible additional losses, such as turbulence caused by gate slots ("spuningen"), friction through the structure.
Figure 6.6. Transition head losses of inlets and outlets.

Table 6.1. Headloss coefficients $c_{\text{in}}$ for inlets, and $c_{\text{out}}$ for outlets.

<table>
<thead>
<tr>
<th>Type of transition</th>
<th>Headloss coefficients over inlet</th>
<th>Headloss coefficients over outlet</th>
</tr>
</thead>
<tbody>
<tr>
<td>warped transition</td>
<td>$c_{\text{in}} = 0.1$</td>
<td>$c_{\text{out}} = 0.2$</td>
</tr>
<tr>
<td>rounded transition</td>
<td>$c_{\text{in}} = 0.2$</td>
<td>$c_{\text{out}} = 0.4$</td>
</tr>
<tr>
<td>straight-line transition</td>
<td>$c_{\text{in}} = 0.3$</td>
<td>$c_{\text{out}} = 0.5$</td>
</tr>
<tr>
<td>rectangular transition</td>
<td>$c_{\text{in}} = 0.5$</td>
<td>$c_{\text{out}} = 1.0$</td>
</tr>
</tbody>
</table>

Table 6.2. Headloss coefficients in bends ($c_{\text{bend}}$) and in elbows ($c_{\text{elbow}}$).

<table>
<thead>
<tr>
<th>Angle $\alpha$ of Bend and Elbow $\alpha$</th>
<th>$30^\circ$</th>
<th>$45^\circ$</th>
<th>$60^\circ$</th>
<th>$90^\circ$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BEND:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$R_{\text{bend}} = 3D^*$</td>
<td>$c_{\text{bend}} = 0.1$</td>
<td>$c_{\text{bend}} = 0.1$</td>
<td>$c_{\text{bend}} = 0.1$</td>
<td>$c_{\text{bend}} = 0.1$</td>
</tr>
<tr>
<td>$R_{\text{bend}} = 2D^*$</td>
<td>$c_{\text{bend}} = 0.1$</td>
<td>$c_{\text{bend}} = 0.1$</td>
<td>$c_{\text{bend}} = 0.1$</td>
<td>$c_{\text{bend}} = 0.2$</td>
</tr>
<tr>
<td>$R_{\text{bend}} = D^*$</td>
<td>$c_{\text{bend}} = 0.1$</td>
<td>$c_{\text{bend}} = 0.2$</td>
<td>$c_{\text{bend}} = 0.2$</td>
<td>$c_{\text{bend}} = 0.3$</td>
</tr>
<tr>
<td>ELBOW:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Circular</td>
<td>$c_{\text{elbow}} = 0.1$</td>
<td>$c_{\text{elbow}} = 0.2$</td>
<td>$c_{\text{elbow}} = 0.5$</td>
<td>$c_{\text{elbow}} = 1.1$</td>
</tr>
<tr>
<td>Rectangular</td>
<td>$c_{\text{elbow}} = 0.2$</td>
<td>$c_{\text{elbow}} = 0.3$</td>
<td>$c_{\text{elbow}} = 0.6$</td>
<td>$c_{\text{elbow}} = 1.4$</td>
</tr>
</tbody>
</table>

* $D$ is the diameter of the pipe.
Bend and Elbow Losses. 'Bends' ("bochten") and 'elbows' ("knikken") in siphons or pipes cause a change in the direction of flow and consequently a change in the general velocity distribution.

Also the headloss $z_{\text{bend}}$ in a bend ("bocht") and the headloss $z_{\text{elbow}}$ in an elbow ("knik") may be expressed as a function of the velocity head in the pipe:

$$ z_{\text{bend}} = c_{\text{bend}} \times \frac{v^2}{2g}, \quad \text{and} \quad z_{\text{elbow}} = c_{\text{elbow}} \times \frac{v^2}{2g} $$

Values of headloss coefficients $c_{\text{bend}}$ in bends and headloss coefficients $c_{\text{elbow}}$ in elbows are presented in table 6.2.

### 6.2.2. Hydraulic Design of Conveyance Structures

**Froude number.** Conveyance structures should have sub-critical flow ("normal" flowing water) as to the limit the headloss. It means that they have a Froude number $Fr < 1$. In design, however, the Froude number is often limited to $Fr < 0.5$ to avoid standing waves at the water surface and to avoid flow to become critical because of decreased canal roughness.

The Froude number $Fr$ can be calculated for structures with vertical wall and an open-water surface by the formula:

$$ Fr = \frac{v}{\sqrt{gy}}, \quad \text{with} \quad v = \frac{Q}{by} $$

where $Fr$ is the Froude number, $Q$ is the discharge in m$^3$/s, $v$ is the velocity within the structure in m/s, $y$ is the water depth within the structure in m, $b$ is the (open-water) width of the structure in m, $g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg}$ is the acceleration of gravity.

The open-water surface width $b$ for full-flowing siphon pipes equals to zero, thus the Froude number is not defined. Typical values of the maximum velocities are presented in box 6.1.

### Box 6.1. Maximum velocity in conveyance structures with free water surface.

<table>
<thead>
<tr>
<th>$y$ (m)</th>
<th>$v_{\text{max}}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>1.1</td>
</tr>
<tr>
<td>0.75</td>
<td>1.4</td>
</tr>
<tr>
<td>1.00</td>
<td>1.6</td>
</tr>
<tr>
<td>1.25</td>
<td>1.8</td>
</tr>
<tr>
<td>1.50</td>
<td>1.9</td>
</tr>
<tr>
<td>2.00</td>
<td>2.2</td>
</tr>
<tr>
<td>2.50</td>
<td>2.5</td>
</tr>
<tr>
<td>3.00</td>
<td>1.7</td>
</tr>
</tbody>
</table>

The maximum design velocity in conveyance structures with an open water surface is limited by the Froude number $Fr < 0.5$ to avoid standing waves. Hence, the design velocity $v$ within the structure is limited by $v < 0.5 (gy)^{0.5}$, where $y$ is the water depth in the structure in m.

Thus, the following maximum design velocities have to be maintained:
Total headloss. The total headloss $z_{\text{structure}}$ in a conveyance structure is the sum of the entrance ("intree"), friction, bend and/or elbow and the exit ("uittree") losses, and varies with the velocity $v$ in the structure:

$$z_{\text{structure}} = \left( c_{\text{in}} + \frac{2 g L}{k^2 R^{4/3}} + m \times c_{\text{bend}} + n \times c_{\text{elbow}} + c_{\text{out}} \right) \times \frac{v^2}{2g}$$

where: $L$ is the length of structure in m, $R$ is the hydraulic radius in m, $k$ is the Strickler coefficient in m$^{1/3}$/s, $m$ is the number of bends, $n$ is the number of elbows, $c_{\text{in}}$ and $c_{\text{out}}$ are headloss coefficients over inlet and outlet, $v$ is the velocity within the structure in m/s, $g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg}$ is the gravity acceleration.

Hydraulic design. Some examples of the entrance and the exit of conveyance structures are presented in figure 6.7. In fact, these transition types can be applied for all kind of structures, such as drop structures, water level and discharge regulators.

It is advised to construct a simple 'stilling basin' ("woelbak") at every outlet transition to prevent scouring when super-critical flow ("schietend water") may happen during e.g. mismanagement of the canal. Such a stilling basin can be achieved by lowering the bed by some $\frac{1}{2} \times$ the water depth $y$.

Furthermore, it is strongly advised to construct a 'coffer' ("koffer") at the end at any flood, as to prevent the concentration of groundwater flow at the bed level.
7. DESIGN OF PASSIVE REGULATORS

7.1. Weirs

7.1.1. Broad-Crested Weirs

General. The rectangular 'broad-crested weir' ("brede overlaat") is an overflow structure with a horizontal crest, while the streamlines are straight and parallel, see figure 7.1.

There are no movable parts on the structure. The broad-crested weir can be used for water level regulation, but also for 'proportional discharge regulation' ("proportionele verdeling"). Furthermore, the broad-crested weir is applied in irrigation canals as a structure for discharge measurement ("afvoermeting") under small headloss, see figure 7.2.

![Diagram of Broad-Crested and Sharp-Crested Weirs](image)

Discharge Formula:  
\[ Q = 1.7 \cdot b \cdot H^{0.2} \]
Conditions:  
\[ H < \frac{3}{4} L \text{ or } H < 3 \cdot z \]

**BROAD-CRESTED WEIR-FLOW**

Discharge Formula:  
\[ Q = 1.9 \cdot b \cdot H^{0.2} \]
Conditions:  
\[ H > \frac{3}{4} L \text{ or } H < z \]

**SHARP-CRESTED WEIR-FLOW**

Figure 7.1. Broad-crested and sharp-crested weir flow.
Hydraulics. The basic discharge formula of a broad-crested weir with a rectangular control section under free flow is, see also figure 7.2:

\[ Q = c \times b \times H^{1.5}, \quad \text{with: } c = c_d \times \left( \frac{2}{3} g \right) = c_d \times \frac{2}{3} g 
\]

where \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( c \) is the weir coefficient in \( \text{m}^{1/2}/\text{s} \), \( c_d \) is the discharge coefficient, \( b \) is the width of the crest in m, \( H \) is the energy head above the crest in m.

![Figure 7.2. Hydraulic parameters of the broad-crested weir for discharge measurement.](image)

**Discharge coefficient.** The discharge coefficient \( c_d \) depends on the curve of the streamlines above the crest, and amounts to \( c_d = 1 \) for the broad-crested weir. Thus, weir coefficient of the broad-crested weir is \( c = 1.7 \) \( \text{m}^{1/2}/\text{s} \). Thus, the hydraulic design equation of a broad-crested weir with a rectangular control section is often simplified into:

\[ Q = c \times b \times H^{1.5} \quad \text{with } c = 1.7 \text{ m}^{1/2}/\text{s}. \]

**Validity.** The tailwater level may not rise too high to allow a 'free flow' ("volkomen stroming"). The free flow (non-submerged, modular) condition is satisfied for a headloss \( z \) over the structure of \( z \geq \frac{1}{2} H \).

### 7.1.2. Sharp-crested weir

**Hydraulics.** The basic discharge formula of a 'sharp-crested weir' ("scherpe overlaat", "meetschot") with a rectangular control section under free flow is similar to the formula of the broad-crested weir:

\[ Q = c \times b \times H^{1.5}, \quad \text{with: } c = c_d \times \left( \frac{2}{3} g \right) = c_d \times \frac{2}{3} g 
\]

where \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( c \) is the weir coefficient in \( \text{m}^{1/2}/\text{s} \), \( c_d \) is the discharge coefficient, \( b \) is the width of the crest in m, \( H \) is the energy head above the crest in m.

**Broad- and sharp-crested weir flow.** The difference between broad-crested and sharp-crested is a matter of curved streamlines above the crest, see also figure 7.1.

A broad-crested weir-flow occurs when the energy depth \( H \) is less than \( \frac{1}{2} \) times the crest length \( L \). For instance, a concrete crest of \( L = 0.30 \) m functions as a broad-crested weir for \( H < 0.15 \) m with the (critical) water depth \( y_c = \frac{1}{2} H = 0.10 \) m.
7. DESIGN OF PASSIVE REGULATORS

Figure 7.3. Types of sharp-crested weirs.

Weir coefficients. Typical values for the weir coefficient \( c \) are, see figure 7.3:
- \( c = 1.9 \, \text{m}^{1/2}/\text{s} \) for unrounded crests,
- \( c = 2.1 \, \text{m}^{1/2}/\text{s} \) for cylindrical crests ("cylindrische kruin"),
- even as high as \( c = 2.3 \, \text{m}^{1/2}/\text{s} \) for the specially designed 'ogee-crests' (USBR 1973).

Headloss. Unrounded crests require a large headloss \( z \) for 'aeration of the nappe' ("beluchting van de straal"), thus \( z > > H \). Cylindrical crests and ogee-crests require a minimum headloss \( z_{\text{min}} \approx \frac{1}{2} H \).

The radius \( R \) of cylindrical crests may not be taken too small, because of the sub-atmospherical pressure under the nappe. Usually, cavitation is prevented with a radius \( 0.1 \, H < R < 0.7 \, H \) for concrete weirs, and \( 0.3 \, H < R < 0.7 \, H \) for masonry weirs.

Cipoletti weir. The 'Cipoletti weir' is a special type of sharp-crested weir and has been developed as measuring device ("meetinrichting") in irrigation, see figure 7.4.

The Cipoletti weir has a trapezoidal control section: the crest is horizontal and sides are sloping outwards by \( 1_{\text{Hor}} : 4_{\text{Vert}} \). The hydraulic design equation of a Cipoletti weir is (Bos 1989, Kraatz 1975), see also figure 7.4:

\[
Q = 1.9 \, b \, H^{1.5}
\]

where \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( H \) is the energy head upstream in m, \( b \) is the width of the crest in m.

Application. The sharp-crested weir is normally used in rivers at headworks for irrigation ("watervang") and at flood diversion structures. The advantage is a shorter width \( b \) of the weir, because of the higher weir coefficient \( c \). This makes the costs of the weir lower. However, a sharp-crested weir requires more headloss for free flow conditions.
Figure 7.4. Hydraulic parameters of the Cipoletti weir for discharge measurement.

7.1.3. Control Notch

General. A 'control notch' ("vernauring") is used to control upstream water levels in a channel, and to maintain the uniform water depth in the channel for any flow, see figure 7.5. It is usually applied at entrances of super-critical flow structures, such as drop structures and chutes, thereby preventing increased 'shear stress' or 'tractive force' ("wand schuifspanning") in the channel which otherwise could cause erosion.

The advantages of the trapezoidal control notch are: it is a sturdy structure that can be constructed at low costs, and which does not need much maintenance, it does not need regulation, and the structure does not have a sill or an orifice, and therefore passes sediment and debris.

Figure 7.5. Control notch for water regulation at the uniform canal depth.
Hydraulics. The depth of the notch equals the bed level of the channel, see figure 7.6. The side slopes of the notch are trapezoidal-shaped. The bottom width, top width and the side slope of the notch are determined. The relation between the discharge $Q$ and the energy head $H$ of the control notch under free flow conditions, follows from the formula:

$$Q = 1.8 \left( B + \frac{2}{3} m H \right) H^{3/2}$$

where $Q$ is the discharge in $m^3/s$, $H$ is the (uniform) energy head in the channel in $m$, $B$ is the bed width of the notch in $m$, $m$ is the side slope of the notch walls ($m_{\text{Vert}} : m_{\text{Hor}}$).

Design. The trapezoidal-shaped opening of the control notch is designed to avoid backing up ("opstuwing") or drawdown ("afzuiging") for discharges in the range of e.g. 20% and 100% of the design flow.

The design steps are:
- determine the range of discharges for which the control must work, e.g. 20% - 100% of design discharge;
- calculate the energy ($\approx$ water) depths in the upstream channel for these two discharges;
- determine the design values $B$ and $m$ from the two equations:

$$Q_{100\%} = 1.8 \left( B + \frac{2}{3} m H_{100\%} \right) H_{100\%}^{3/2}$$
$$Q_{20\%} = 1.8 \left( B + \frac{2}{3} m H_{20\%} \right) H_{20\%}^{3/2}.$$

A design example of a control notch at a drop structure in a drain is presented in box 7.1.

---

**Figure 7.6.** Hydraulic parameters of the 'Control Notch'.

**Box 7.1.** Design example of a control notch.

The "control" of a fall structure is designed, to prevent racing of the water to the fall. The drain has a water depth of $y = 1.00$ m ($H = 1.01$ m) for the design discharge of $Q_{100\%} = 2\ m^3/s$, and of $y = 0.41$ m ($H = 0.41$ m) during $Q_{20\%} = 0.4\ m^3/s$.

The control notch should prevent any drawdown or backing up within this range. This is obtained when the energy depth at the control notch equals the uniform energy depth in the drain.

The two design parameters $B$ and $m$ of the notch follow from the two conditions:

$$2.0 = 1.8 \left( B + \frac{2}{3} m 1.00 \right) 1.00^{3/2}, \quad \text{or} \quad B + 0.67 m = 1.11$$
$$0.4 = 1.8 \left( B + \frac{2}{3} m 0.41 \right) 0.41^{3/2}, \quad \text{or} \quad B + 0.27 m = 0.85$$

Thus, $m = 0.26 / 0.40 = 0.66$, and $B = 1.11 - 0.67 \times 0.66 = 0.67\ m$.

As a check, the energy depth at the control notch during 60% of the design discharge ($Q_{60\%} = 1.2\ m^3/s$) is compared with the energy depth ($H_{60\%} = 0.75\ m$) of the uniform canal flow. The energy depth $H$ at the notch follows from:

$$H = \left( Q / \left[ 1.8 \times 0.67 + 1.8 \times \frac{2}{3} \times 0.66 H \right] \right)^{2/3} = \left( Q / \left[ 1.21 + 0.79 H \right] \right)^{2/3},$$

and amounts to $H = 0.76\ m$ for $Q_{60\%} = 1.2\ m^3/s$. So, there are also no backwater effects for $Q_{60\%}$. 

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7.2. Long-Crested Weirs

7.2.1. Types of Long-Crested Weirs

General. 'Long-crested weirs' ("lange overlaten") are passive regulators, and are frequently applied for water level control at flood diversion structures and in irrigation. A long-crested weir maintains the water level within certain narrow limits.

Types of long-crest weirs. Long-crest weirs have been developed into different types, as to match the length of the weir with the width of the canal, see figure 7.7:

- the diagonal weir, which has an angle "\( \alpha \)" of more than 0° with the cross section of the canal;
- the duckbill weir ("eende-bek overlaat") or horse-shoe weir, which has a U-shape and has an angle "\( \alpha \)" of more than 45°;
- the Z weir, which is a combination of the normal weir and a weir with an angle of "\( \alpha \)" with the cross section of 90°.

Of these, the duckbill weir is the most commonly used because it is providing optimum discharge in relation to the amount of construction works.

The provision of a gate is advantageous on larger structures to enable evacuation of the upstream reach.

![Diagram of types of weirs](image)

Figure 7.7. Types of long-crested weirs.

**Hydraulics.** The calculation of the discharge over a weir is based on the general weir formula for free flow:

\[
Q = c \cdot b \cdot H^{3/2},
\]

where: \( Q \) is the discharge in \( m^3/s \), \( c \) is the weir coefficient for free flow in \( m^{1/2}/s \), \( b \) is the width of the weir in \( m \), \( H \) is the (upstream) energy level above the crest level in \( m \). The tailwater level \( y_3 \) should be lower than \( \frac{3}{8} H \) to satisfy the free flow condition.
7.2.2. Duckbill Weir

Duckbill weir. The duckbill weir is a water level regulator without moving parts and can be used for passive water level control (Kraatz 1975), see figure 7.8 and figure 7.9.

The duckbill weir is a weir with a certain length of the crest. Such a long-crested weir controls the water level at a certain target level within relatively narrow limits. Certain fluctuations of the upstream water level with varying canal discharges \( \Delta Q = Q_{\text{max}} - Q_{\text{min}} \). These fluctuations are unavoidable and can be restricted by developing a large crest width \( b \). This range of water level fluctuations is called the 'decrement' ("afwijkings", "speling") \( \Delta H \). The decrement is found with all passive and hydro-mechanical water level regulators.

The narrower the decrement \( \Delta H \), the greater must be the crest length \( b \). Usual the decrements are in the order of \( \Delta H = 0.05 \) to 0.10 m, but they depend on the level requirements of the offtaking discharge regulators. The provision of a vertical gate is advantageous on larger structures to restrict the length of the structure, for emptying of the upstream reach, or for flushing of sediment, see also figure 7.9.

The calculation of the weir length \( b \) depends on the decrement \( \Delta H \) for the varying discharge \( \Delta Q = Q_{\text{max}} - Q_{\text{min}} \) over a long-crested weir and is based on the general weir formula for free flow \( Q = c \ b \ H^{3/2} \). Thus:

\[
b = \frac{\left( Q_{\text{max}}^{2/3} - Q_{\text{min}}^{2/3} \right)^{3/2}}{c \ \Delta H^{3/2}}
\]

where: \( Q \) is the discharge in m\(^3\)/s, \( c \) is the weir coefficient for free flow in m\(^{1/2}\)/s, \( b \) is the width of the weir in m, \( \Delta H \) the decrement for the varying discharge \( Q_{\text{max}} - Q_{\text{min}} \) in m.

Figure 7.8. Duckbill weir for a 'passive' water level regulation.
Figure 7.9. Long-crested weir for 'passive' upstream control.

Weir coefficient. The weir coefficient $c$ depends on the size and shape of the crest, as well as on the angle $\alpha$ of the crest with the cross section of the canal:

- for a channel flow rectangular on the weir direction, so the angle $\alpha = 0^\circ$, the coefficient $c$ of the normal broad-crested weir amounts to $c = 1.7 \text{ m}^{1/2}/\text{s}$;
- for a channel flow not rectangular on the weir direction, so the angle $\alpha > 0^\circ$, there will be an energy loss because of the sudden deflection of flow ("verandering van stroomrichting"). This can be incorporated in the weir formula by reducing the weir coefficient $c$ while calculating with the original energy depth $H$ above the crest. For instance (see also Kraatz 1975):
  - the diagonal weir, with an angle $0^\circ < \alpha < 45^\circ$, can be designed on a coefficients $c = 1.7 \text{ m}^{1/2}/\text{s}$ to $c = 1.5 \text{ m}^{1/2}/\text{s}$;
  - the duckbill weir, with an effective angle $45^\circ < \alpha < 90^\circ$, can be designed on a coefficients $c = 1.5 \text{ m}^{1/2}/\text{s}$ to $c = 1.2 \text{ m}^{1/2}/\text{s}$;
  - the Z weir, with an angle $\alpha = 90^\circ$, can be designed on a coefficients $c = 1.2 \text{ m}^{1/2}/\text{s}$.

Application. The great advantage of long-crested weirs is their simplicity in construction and maintenance, and their reliability in operation. Mismanagement of the structure is impossible.

A disadvantage of long-crested weirs is its trap sediment ability which prohibits their use when water is permanently charged with silt. Another disadvantage is that the length of the long-crested weir will become very long when a small decrement $\Delta H$ is allowed. Long-crested weirs have a headloss $z$ during the design discharge $Q$ in the range of $z \geq \frac{1}{2} H_{\text{max}}$. 
7.3. Escapes

7.3.1. Types of Escapes

Function. 'Escapes' are canal protective structures, that are used to protect canals and structures against damages caused by excess amounts of water, see figure 7.10. The capacity of such an escape should be adequate to discharge any excess amounts of water resulting:
- from floods entering the canal,
- during operational mistakes.
In fact, they protect the water level in the canal against rising to such a level that the banks will be endangered or overtopped.

Types. Protection can be realized with different types of structures:
- manual regulator, i.e. a manual-operated gate, as discussed in the previous chapter.
- 'side-channel spillways' ("zijdelingse overlaat") or lateral spillways, are constructed in the bank of the canal, at low cost but often over a considerable length;
- 'siphon spillway' ("hevel"), to evacuate a large flow for a fairly constant water level;
- hydro-mechanical regulators, i.e. the gates under upstream control, as discussed in the next chapter.

Figure 7.10. 'Escapes' are sometimes needed in irrigation systems.

7.3.2. Side-channel spillway

General. 'Side-channel spillways' ("zijdelingse overlaten") are passive water level regulators, figure 7.11. They are constructed as a long, protected, lowered part of the canal embankment for the purpose of spilling excess flow above a certain water level. The crest of the side-channel spillway is slightly above the 'target water level' ("streeftpel"), and its length determines the rise of the water level above this target water level.
Figure 7.11. A side-channel spillway.
Decrement. The 'decrement' ("afwijking") of the side-channel spillway is the difference between the design water level, i.e. which is just under the crest of the spillway, and the water level in the ongoing channel during the flood, i.e. sufficient above the spillway crest. It should be noticed that the water depth along the spillway decreases (!!) in upstream direction due to the increase of the velocity head at a constant energy head, see figure 7.12.

![Figure 7.12. Side-channel spillway as a 'passive' regulator.](image)

Design. A design formula of the side-channel spillway does not exist and a numerical method has to be used to calculate its length $L$, see figure 7.13. Basically, the weir formula ($dQ = c \cdot db \cdot H^{3/2}$) is valid. The energy head $H$ refers to the flow over the crest, and is rectangular to the flow direction in the canal. Therefore, the energy head $H$ equals to the canal water level $y-p$ above the crest, as the total energy head $v^2/2g$ is lost by the sudden flow deflection ("plotselinge verandering van stroomrichting").

The calculation is done in steps of $db$ into an upwards direction, and the outflowing discharge $\Sigma dQ$ is calculated until it equals the required value of $Q_{flood} - Q_o$. The corresponding value $\Sigma db$ equals the spillway length $L$. An example of the calculation is presented in box 7.2.

![Figure 7.13. Parameters for the step-by-step calculation of the side-channel spillway.](image)
Box 7.2. Example calculation of a side-channel spillway.

**Formulae. Discharge over spillway length "ΔL":**

\[ ΔQ = \Delta L \left(y_1 - p\right)^{3/2} \]

**Discharge at vertical "2":**

\[ Q_2 = Q_1 + \sum ΔQ \]

**Water depth at vertical "2":**

\[ y_2 = H - \left(\frac{Q_2}{A_2}\right)^2 \times \frac{1}{2g} \]

\[
\text{and rewritten for the use of the programmable calculator or computer:}
\]

- **the energy head } H: \quad H = p + \left[ \frac{Q_o}{A_o} \right]^2 \times \frac{1}{2g}
- **the discharge } Q_2 \text{ at } \Sigma(\Delta L): \quad Q_2 = Q_1 + \Delta L \left(y_1 - p\right)^{3/2}
- **the water depth } y_2 \text{ at } \Sigma(\Delta L): \quad (b + m y_2)^2 \left(H - y_2\right) = Q_2^2 \times 2g

**Spillway.** The normal discharge of } Q = 7 \text{ m}^3/\text{s} \text{ flows through a channel with bed width of } b_o = 5 \text{ m}, \text{ side slopes } l_{\text{vert}}: 1.5_{\text{hor}}, \text{ Strickler coefficient } k = 40 \text{ m}^{1/3}/\text{s} \text{ and a gradient of } s = 0.22 \times 10^{-3}. \text{ A full supply level of } y = 1.54 \text{ m} \text{ can be calculated with the Strickler formula.}

Flood waters can enter the channel at drainage inlet structures and a total discharge during flood of } Q = 12 \text{ m}^3/\text{s} \text{ can be expected in the channel.}

The flood water should be evacuated over a side-channel spillway. Downstream of this spillway the water level may rise 0.10 m higher than the full supply level or } y_o = 1.64 \text{ m}. \text{ The corresponding channel discharge amounts } 7.83 \text{ m}^3/\text{s}, \text{ as can be calculated with the Strickler formula. The remainder discharge should be spilled over the spillway. The 'step method' calculates a total spillway length of } L = 100 \text{ m} \text{ (steps of } ΔL=5 \text{ m).}

**'Controlled' spillway.** The spillway length can be reduced by choosing the site of the spillway upstream of an obstruction in the channel.

For instance, a siphon of } 1.80 \times 1.80 \text{ m}^2 (A = 3.24 \text{ m}^2) \text{ with a head loss of } 1.45 v^2/2g \text{ is located in the channel. The normal headloss in the siphon for } Q = 7 \text{ m}^3/\text{s} \text{ amounts } 0.35 \text{ m}. \text{ The headloss increases during flood } (Q = 7.83 \text{ m}^3/\text{s}) \text{ to } 0.43 \text{ m}, \text{ thus an increase of } 0.08 \text{ m}. \text{ The water level downstream of the spillway increases in this case to } y_o = 1.72 \text{ m} \text{ for } Q = 7.83 \text{ m}. \text{ A spillway length of } L=36 \text{ m} \text{ can be calculated, similar to the example shown above (steps of } ΔL=2 \text{ m).}

**Flood screen.** The construction of a flood screen downstream of the spillway which obstructs specially the higher discharges, can reduce the spillway length once more.

Assume in the above example that the headlosses at the flood screen increases with } 0.10 \text{ m} \text{ during the discharge of } Q_o = 7.83 \text{ m}, \text{ it means that } y_o = 1.82 \text{ m}. \text{ A spillway length of } L=18 \text{ m} \text{ can be calculated.
Hydraulic performance. The hydraulic performance of a side-channel spillway can best be understood by considering the energy line, see figure 7.14. There is sub-critical flow in the canal, thus the tailwater (downstream) conditions in the canal determine the situation at the spillway. The tailwater and the energy line in the downstream canal reach follows from the uniform flow and can be calculated through the Strickler formula. The energy line in the downstream canal is sloping down because of the friction losses.

The energy line in the canal at the spillway location equals the energy line of the downstream canal reach. The spillway length $L$ is relative short, and the headloss due to friction can be ignored here, and can be taken as horizontal.

The discharge at the upstream canal reach is much larger than in the downstream reach, because of the outflow. The energy line is the same, so the velocity head $v^2/2g$ makes that the upstream water level is lower than the downstream water level.

The velocity $v$ in the upstream canal reach is also much higher than in the downstream reach, so the friction losses in the upstream canal reach are higher than in the downstream reach. Therefore, the upstream energy gradient $s$ is steeper than in the downstream reach.

Figure 7.14. Backwater curves by a side-channel spillway.

Flood screen. The length $L$ of the side-channel spillway can be reduced considerably by applying a 'flood-screen' ("bandjir-scherm"), see figure 7.15.

A flood-screen is a beam above the target water level in a canal, with the function to create an 'orifice flow' ("onderspuier") with headloss for discharges higher than the design discharge (Eijjsvoogel 1932). It can be constructed as a simple beam of concrete, or it can be constructed in combination with a bridge. The effect of a flood-screen on the design of the side-channel spillway is also presented in box 7.2.

Figure 7.15. Side-channel spillway with a "flood screen".
7.3.3. Siphon spillway

General. 'Siphon spillways' ("hevels") are also passive water level regulators as they prevent the water level to raise above the target water level ("streepeil"), see figure 7.16.

Small excesses of water levels will initially cause the siphon to act as a weir. If the water level continues to rise, the discharge through the siphon will attain a velocity sufficient to 'prime' ("aan de gang brengen") the siphon. Priming is initiated as the water flow carries an air-water mixture out of the barrel ("koker"). A full prime occurs when all the air is removed from the barrel and pressure over the crest becomes sub-atmospheric.

A flow deflector is usually constructed in the downstream leg of the siphon. The flow deflector will initiate the priming at a low discharge by deflecting the flow to the top-side of the barrel. The turbulence thus created and the impact of the flow will mix the air in the barrel with the outflowing water.

Once fully primed, siphonic action continues until the water level drops below the inlet elevation or below the inlet of the siphon-breaker, and allowing an inflow of air to restore atmospheric pressure at the crest, ending the flow abruptly.

Hydraulics. The capacity of the siphon follows from the equation, see also figure 7.16:

\[ Q = \mu A \sqrt{2g} z, \quad \text{with} \quad z < H_0 \]

where: \( Q \) is the discharge in m\(^3\)/s, \( \mu \) is a coefficient, \( A \) is the cross-sectional area of the barrel in m\(^2\), \( z \) is the headloss over the siphon in m, \( H_0 = 10 \) m is the atmospheric pressure, \( g \) is the acceleration of gravity \( g = 9.8 \) m/s\(^2\) = 9.8 N/kg. The coefficient \( \mu \) depends on the energy losses due to bends, friction, the entrance and the exit. For a first estimate, \( \mu = 0.7 \) can often be used, thus \( z = 2.0 \sqrt{v^2 / 2g} \).

![Figure 7.16. Types of flow in a siphon spillway.](image-url)

Application. Spillway siphons are very effective in rapidly removing a large volume of water from a canal, but may be more expensive to construct than other escapes. A distinct advantage is its ability to discharge a large volume of water with a small rise in water surface.

A disadvantage to the use of a siphon spillway in a small canal is the abrupt starting and stopping of the discharge, which may produce surges or bore waves. It may also cause erosion in the downstream canal.
7.4. Baffle Distributors

7.4.1. Working Principle

General. Baffle distributors, also called 'Neyrtec distributors' or 'orifice modules', are designed to supply constant discharges that are independent of the upstream water levels, see figure 7.17. These constant discharges are obtained by its hydraulic function, and not by moving parts. The baffle distributor can be used for an on/off operation by opening or closing the slide plate or 'shutter'.

Application. Baffle ("scherm") distributors were initially applied by French engineers only. At present, baffle distributors are world-wide applied and are an international standard for constant discharge control at offtakes in irrigation systems.

The advantages of the baffle distributor is its simple operation. All that is required for discharge regulation is to open or close the different slide gates or 'shutters' ("luiken") of the compartments. The discharge regulation is independent of upstream water level variations.

The main disadvantage of the baffle distributor is its cost: the structure is expensive, especially when it has to be imported from France with 'hard' currency.

Furthermore, the headloss through the baffle distributor is larger than the combined headloss of a vertical gate for discharge regulation with a broad-crested weir for measurement.

Working principle. The baffle distributor consists of a weir, and one or two fixed metallic "baffle", see figure 7.18.

At low upstream levels, the structure operates as a sharp-crested weir with overflow ("overlaat"). When the upstream water level rises, the water level above the weir reaches the
bottom edge of the baffle, and the structure operates as an orifice with underflow ("onderspuijer"). The contraction of the jet becomes more pronounced as the upstream water level increases, thus reducing the discharge coefficient $C_D$ and maintaining an almost constant discharge. It should be noted that the baffle is not located above the crest but above the sloping face.

![Figure 7.18. Working principle of the single-baffle distributor.](image)

**Double-baffle distributor.** Essentially two types of distributors are available, the **single-baffle** and the **double-baffle** distributors.

The double-baffle distributor has two baffles: a **low** and **high** baffle, see also figure 7.19. The function of the low baffle is similar to the baffle of the single-baffle distributor, and forces an orifice flow for the higher water levels.

As the upstream water level rises further, the low upstream baffle is overtopped and the second baffle comes into action to limit the flow. Thus, the upstream low baffle has the dual function of (i) contracting the jet at low heads and (ii) acting as a weir at high heads.

Water passing over the low baffle causes an additional contraction of the jet. As a result, the double-baffle distributor maintains an almost constant discharge over a considerable variation in upstream water levels.

![Figure 7.19. Working principle of the double-baffle distributor.](image)
7.4.2. Hydraulics of Baffle Distributors

Hydraulics. The baffle distributor acts as a short-crested weir with rectangular control section, when the water surface is not in contact with the baffle(s). The discharge equation for such a weir reads:

\[ Q = 1.85 \cdot b \cdot H^{1.5} \]

When the discharge approximates the design discharge, the water surface touches the baffle and orifice flow commences. The free flow discharge through the orifice equals:

\[ Q = C_D \cdot b \cdot w \cdot \sqrt{2g \cdot y_1}, \text{ with: } C_D = \frac{\mu}{\sqrt{1 + \mu \cdot \frac{w}{y_1}}} \]

where \( Q \) is the discharge in m³/s, \( b \) is the width of the gate in m, \( C_D \) is the discharge coefficient, \( w \) is the vertical distance between the baffle and the weir in m, \( y_1 \) is the upstream water depth above the sill in m, \( \mu \) is the contraction coefficient of the jet and depends on the relation \( y_1/w \), and \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the gravity acceleration.

The headloss \( z \) through the baffle distributor depends on the orifice flow:

\[ z > 0.4 \cdot y_{\min} + V \text{ with: } V = y_{\max} - y_{\min} \]

where \( V \) is the variation is upstream water level between the minimum \( y_{\min} \) and the maximum water level \( y_{\max} \).

The design discharge without any deviation is reached at the 'nominal' water level. Usually, a fluctuation of 5% from the design discharge is allowed.

The rating curve of the single- and the double-baffle distributors are presented in figure 7.20 and figure 7.21. The permissible water levels are presented in table 7.1. An example of the design is presented in box 7.3.

![Hydraulics of Baffle Distributors](image)

**Figure 7.20.** Depth-discharge curve of the single baffle distributor.
Figure 7.21. Depth-discharge curve of the double baffle distributor.
7.4.3. Design of Baffle Distributors

Series of distributors. Five different series of baffle distributors are available. They are identified in terms of design discharge per unit width, see also table 7.1:
- 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,
- series CC : 200 l/s per 0.10 m width.
The series can be given an index '1' or '2', denoting single or double-baffle type, respectively.

An offtake with baffle distributors is identified by the series symbols, followed by the design flow. So, an offtake with a distributor L1-2250 has a design discharge of 2250 l/s, and a single-baffle distributor series L is used of a total width \( b = 2250 / 500 = 4.50 \) m.

Table 7.1. Permissible Water Level of Baffle Distributors.

<table>
<thead>
<tr>
<th>Type</th>
<th>Flow per 1 m in m³/s</th>
<th>Baffle Distributor</th>
<th>Baffle Distributor in series with AVIS/AVIO gate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>range in upstream water level in m</td>
<td>minimum headloss in m</td>
</tr>
<tr>
<td>X1</td>
<td>0.100</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>XX1</td>
<td>0.200</td>
<td>0.08</td>
<td>0.08</td>
</tr>
<tr>
<td>L1</td>
<td>0.500</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>C1</td>
<td>1.000</td>
<td>0.23</td>
<td>0.24</td>
</tr>
<tr>
<td>CC1</td>
<td>2.000</td>
<td>0.37</td>
<td>0.38</td>
</tr>
<tr>
<td>X2</td>
<td>0.100</td>
<td>0.14</td>
<td>0.05</td>
</tr>
<tr>
<td>XX2</td>
<td>0.200</td>
<td>0.22</td>
<td>0.08</td>
</tr>
<tr>
<td>L2</td>
<td>0.500</td>
<td>0.42</td>
<td>0.15</td>
</tr>
<tr>
<td>C2</td>
<td>1.000</td>
<td>0.67</td>
<td>0.24</td>
</tr>
<tr>
<td>CC2</td>
<td>2.000</td>
<td>1.06</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Box 7.3. Example of a design of an offtake with a baffle distributor.

An offtake has to be designed for a constant discharge of 0.25 m³/s and at a point in a canal where the levels vary from 23.18 to 23.12 m⁺. The water level in the offtaking canal, downstream of the offtake, will be always lower than 23.02 m⁺.

An offtake with XX1 distributors of total width of 1.25 m wide is suitable for the discharge of 0.25 m³/s. The lower and upper limit of the upstream water level of the XX1 distributor can be read in table 7.1 and values 0.22 to 0.30 m above the crest, while the downstream water level should be lower than 0.14 m below the crest.

The sill crest of the distributor can be designed at 23.12 - 0.22 = 22.90 m⁺. Then, the maximum upstream water level would be 22.90 + 0.30 = 23.20 m⁺, while the maximum downstream water would be 22.90 + 0.14 = 23.04 m⁺.
Regulation of the discharge. The discharge through a baffle distributor is regulated by opening or closing the shutter. The shutter ("scherm") can be locked in place, either fully opened or fully closed, see figure 7.22.

The X and the XX-series distributors are fitted with flat-slide gates. They are suitable for individual offtakes.

The L, LL and the C-series are fitted with sector-type gates, so as to reduce the operating forces by the high water forces. The CC-series are controlled by independent vertical gates. This equipment is primarily intended for major offtakes.

Compartments of Distributors. Since the discharge of one baffle distributor can only be operated by the shutter on an on/off basis, the only way to vary the discharge is to combine several baffle distributors of different widths within one offtake by creating 'compartments', see also figure 7.22.

The discharge is set to any required value by opening or closing a combination of different sized shutters of the compartments. It is not advisable to combine compartments of different series (e.g. XX and C) in one offtake.

Figure 7.22. Discharge regulation with the (single) baffle distributor.
8. DESIGN OF MANUAL REGULATORS

8.1. Overflow Gates

8.1.1. Drop-Leaf Gates

**General.** The 'drop-leaf gate' ("klepstuw") pivots along an **underwater axis**, and is of the overflow type, see figure 8.1. The load on the operating mechanism comes mainly from the hydrodynamic thrust on the leaf and is quite complex. Drop-leaf gates are widely used in the Netherlands.

**Hydraulics.** The rating curve of the drop-leaf gate is not available for practical use, as it depends on too many parameters, such as the shape of the rounded crest, the angle of opening and the submerged ratio. In fact, the knowledge on the precise rating curve ("Q-h kromme") is not very important as the operator regulates the gate on the basis of water levels and not on discharges.

**Design.** The hydraulic design of the structure is also not based on the rating curve of the drop-leaf gate. The design discharge has to pass over a fully lowered gate, which means that the conveyance formula ("ver nauwingsformule"), \( z = \alpha \, \frac{v^2}{2g} \), is valid for submerged conditions, and the weir formula ("overlaatformule"), \( Q = 1.7 \, b \, H^{3/2} \), for free flow conditions.

![Diagram of drop-leaf gate](image)

**Figure 8.1.** Drop-leaf gate for manual regulation.
8.1.2. Vertical Gate with Overflow

**General.** Vertical gates can also be manufactured as overflow gates, see figure 8.2. However, structural provisions have to be made to allow the gate-leaf to be lowered, either by applying a second gate or by designing a drop in the canal alignment.

**Hydraulics.** The hydraulic design equation of a vertical-gate-with-overflow equals the rating curve of the sharp-crested weir. However, the structure is not standardized. A discharge coefficient \( c = 1.9 \, \text{m}^{1/2}/\text{s} \) is recommended for a first estimate. Thus:

\[
Q = 1.9 \cdot b \cdot H^{1.5}
\]

where \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( H \) is the energy head upstream in m, \( b \) is the width of the crest in m.

The tailwater level must be kept quite low to allow the aeration of the nappe. This is satisfied for a tail water level of at least 0.05 m below the crest level. It means that the headloss \( z \) over the structure has to meet \( z > H \).

**Application.** The vertical gate with overflow is simple in reading but requires considerable headloss. Moreover, the rating curve is not accurate. The structure is not recommended for discharge measurement. As such, the vertical gate with overflow is often not the best solution for an offtake structure.

![Diagram of vertical gate with overflow](image)

**Figure 8.2.** Vertical gate with "overflow" for discharge regulation.

8.1.3. Romijn Weir

**General.** The Romijn weir has been developed in Indonesia during the 1930s for use in relative flat regions and for adjustable supply to the tertiary irrigation units (Bos 1989, Kraatz 1975). The Romijn weir has also been applied in the Netherlands and many other countries where Dutch engineers have worked. The structure is also called the "Hobrad weir", being the abbreviation of horizontal broad-crested adjustable weir (STOWA 1994).

The Romijn weir is basically a broad-crested weir, see figure 8.3. The broad-crested weir is mounted in a steel frame, and can be moved up- and downwards to decrease or increase the discharge. A (bottom) gate is optionally added in the steel guide-frame for flushing sediment that has been deposited in front of the gate.
Hydraulics. The hydraulic design equation of a Romijn weir equals the equation of the broad-crested weir with a rectangular control section \((Q = 1.7 \, b \, H^{1.5})\). It means also that the headloss \(z\) over the structure has to satisfy \(z \geq \frac{1}{3} H\).

The maximum head \(H\) over the weir table depends on the length \(L\) of the table as straight flow-lines above the table has to be guaranteed for the broad-crested weir flow. In practice, it means that \(H \leq 0.30 \, m\) for the standard length of \(L = 0.30 \, m\).

Consequently, standard widths \(b\) of the Romijn weir of \(b = 0.50 \, m, 0.75 \, m, 1.00 \, m\), and \(1.25 \, m\) will provide for design discharges upto \(Q = 0.30 \, m^3/s, 0.45 \, m^3/s, 0.60 \, m^3/s\), and \(0.75 \, m^3/s\), respectively.

![Figure 8.3. Hydraulic parameters of Romijn weir.](image)

Design. The hydraulic design of the structure is based on a known design discharge \(Q\). The above standard widths \(b\) will determine the required numbers of bays and the total width of the structure.

Furthermore, the design should also incorporate any variations \(V\) in upstream water levels at which the design discharge \(Q\) has to be supplied, see also figure 8.3. The variation \(V\) may originate from the "decrement" \(\Delta h\) of the water level regulator, and/or from backwater effects during non-uniform flow. The variation \(V\) might be small at offtakes in combination with water level regulators, but might become considerable for "free" offtakes with water level regulators at some distance. Thus, the sill level of the offtake has to be designed at \(2 \times (H + V)\) below the maximum upstream water level. The resulting headloss \(z\) over the structure is \(z \geq \frac{1}{3} H + V\).

Discharge measurement. The discharge measurement and the weir-setting is simple, although it needs three gauges, see also figure 8.3:

- a water level gauge in the canal, the **counter-gauge**, which shows a reading in centimetres;
- another fixed gauge on the frame, the **centimetre-gauge**, which is identical to the above counter-gauge;
- a gauge that moves with the weir, the **litre-gauge**, with a logarithmic scale of the \(H\)-values according to the weir formula: \(H = (Q / 1.7 \, b)^{0.67}\).
Application. The Romijn weir has a good sediment and floating debris capacity, is easy in operation and reading. It has a double function of both for regulation and for discharge measurement. However, the structure with gate is quite expensive and is not widely understood in many countries.

8.2. Underflow Gates

8.2.1. Vertical Gate for Submerged Underflow

General. The 'vertical slide gate' ("vlakke schuiven") consists of a simple rectangular gate-leaf ("blad") that is moved up- and downwards in grooves ("sponningen"), see figure 8.4. Slide gates are used only for the smaller openings ($b < \pm 3$ m) in structures and to a maximum of 3 m height, as the hoisting device ("windwerk") becomes otherwise too heavy to overcome the friction in the grooves.

The advantages of the vertical gate with underflow as discharge or as water level regulator are: the structure is relatively cheap, the structure is simple and sturdy, it provides a reasonable constant discharge for varying upstream water levels in the parent canal. Moreover, sediment can be transported through the structure.

The disadvantages of regulators with vertical gates are: the gate does not pass floating debris, and the discharge cannot be measured accurately.

Design. The design of a structure with a vertical gate for water level regulation, is normally based on the criterium: the headloss during the design discharge $Q_{100\%}$ should be low. Thus, the vertical gate should be under submerged ("verdonken") conditions. A higher headloss would make that the design water level in the upstream channel reach is rising with the same amount.

The minimum headloss $z$ of the structure will occur not only at submerged flow, but also when the vertical gate is lifted out of the water, see also figure 8.4. So, the flow through the structure is not a 'gate flow' anymore, but becomes a 'conveyance flow' like at a bridge or at a culvert.

![Diagram of a vertical gate under submerged underflow](image-url)

Figure 8.4. Design of a vertical gate under 'submerged underflow'.

Conveyance flow. The gated discharge regulator under (submerged) conveyance flow ("vernauwingsstroming") follows the rating curve ("$Q-h$ kromme") of conveyance structures, such as bridges, aqueducts.
The headloss $z$ in these structures reads:

$$z = \alpha \frac{v^2}{2g} \quad \text{with} \quad v = \frac{Q}{y \ b} \quad \text{and} \quad \alpha = \alpha_{in} + \alpha_{out}$$

with the entrance coefficient $\alpha_{in}$ ($\approx \frac{1}{2}$), the exit coefficient $\alpha_{out}$ ($\approx \frac{3}{2}$), the velocity $v$ within the structure in m/s, the discharge $Q$ in m$^3$/s, the (average) water depth $y$ within the structure, and the width $b$ of the structure. Thus, the rating curve becomes:

$$Q = \sqrt{\frac{1}{\alpha} \ b \ y \ \sqrt{2g \ z}}$$

The conditions for the conveyance flow are, see also figure 8.4:

- the gate is lifted out of the water, so $w > y$;
- the tailwater level is quite high: the headloss $z < \frac{3}{8} \ H$.

### 8.2.2. Vertical Gate for Free Underflow

**Design.** A regulator with a vertical gate can also be applied at a location with a high headloss $z$ during the design discharge $Q_{100\%}$. This may happen in a channel that follows a steep alignment ("tracee") and where drop structures ("stordammen") are required to avoid scouring of the channel bed.

Often, the design of discharge regulators, and also water level regulators, should meet the criterion of low costs of gate and structure. This is achieved when the water depth $y$ in the structure and the width $b$ of the structure are at a minimum. Both can be met by designing a structure where the gate can be lifted out of the water during the design discharge. This means that the free underflow ("volkomen onderspui"), $Q \approx H^{1/2}$, of the gated discharge regulator is transformed into a free overflow ("volkomen overlaat"), $Q \approx H^{3/2}$, see figure 8.5.

![Figure 8.5. Design of a vertical gate under 'free underflow'.](image)

**Free overflow.** The regulator with a vertical gate is designed with the gate lifted out of the water. For a high headloss, the flow becomes a free overflow ("volkomen overlaat"). The corresponding the rating curve ("$Q$-h kromme") is either:

- of a broad-crested weir, when there is a sill ("drempel"), with the discharge formula:
\[ Q = 1.7 \times b \times H^{3/2}, \]

- of a control notch, when there is no sill, with the discharge formula:
  \[ Q = 1.8 \times b \times H^{3/2}, \]

where \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( b \) is the width of the structure in \( \text{m} \), and \( H \) is the (upstream) energy head in \( \text{m} \).

The conditions for free overflow are, see also figure 8.5:
- the gate opening \( w \) is sufficient high, i.e. \( w > \frac{1}{3} H \), in order that the water does not touch the gate;
- the 'tailwater depth' ('achterwater diepte') is sufficient low, i.e. the headloss \( z > \frac{1}{3} H \), in order to maintain a free flow.

### 8.2.3. Radial gate

**General.** 'Roller gates' ('roldeuren') are essentially equal to the vertical slide gates, but they have rollers or wheels in the groove to limit the friction, see figure 8.6.

'Radial gates' ('segmentdeuren'), also called 'Tainter gates' or 'Segment gates', are used as underflow gates as an alternative for the vertical gates, see also figure 8.6. The choice of a radial gate above a vertical gate may depend on a variety of factors and each type has its own advantage.

Radial gates are not often used for small discharges. However, they have the advantage over sliding gates that the lifting force is only required against the weight of the gates, as all thrust on the gate leaf due to water pressure is passing through the axis. Moreover, lifting force is almost constant for all gate openings. Counterweights in order to counter balance the self weight, may also be used to further reduce the lifting force.

Hence, radial gates can be used with smaller lifting force for all heads than with the sliding gates. Even hand-operated hoisting mechanism ('windwerken') may suffice for smaller works, whereas the vertical lift gates of the same size might need a power mechanism.

The radial gate is often more costly than vertical gates. The rating curve of a radial gate is even more complex than that of the vertical gate, which make both less suitable for discharge measurement.

![hoisting device](image)

**Figure 8.6.** Types of vertical and radial gates.
8.2.4. Crump-deGruyter Gate

General. The Crump-deGruyter gate is a 'streamlined' vertical gate, see figure 8.7 (Bos 1989, Kraatz 1975). The gate is equipped with a flow-streamliner which avoids the curvation of the streamlines and the contraction of flow under the gate. Thus, the contraction coefficient \( \mu = 1.0 \).

Flow-streamliner. The diameter \( d \) of the flow-streamliner must satisfy \( d > 2w \) for a contraction coefficient \( \mu \approx 1 \).

Otherwise, the formula for the contraction coefficient \( \mu \) depends on the gate opening \( w \) (STOWA 1994):

\[
\mu = 0.51 + 0.1 \sqrt{23 - \left( \frac{d}{w} - 4.7 \right)^2}
\]

![GATE AT A LOW DISCHARGE](image1)

![GATE AT THE DESIGN DISCHARGE](image2)

Figure 8.7. Crump-deGruyter gate, for manual regulation.

Hydraulics. The hydraulic design equation of a Crump-deGruyter gate under free flow conditions is basically the orifice formula of the vertical gate, see also figure 8.7:

\[
Q = \mu b w \sqrt{2g(H - w)} = 4.43 \mu b w \sqrt{H - w}
\]

So, the discharge formula of the Crump-deGruyter gate under free flow and for \( \mu = 1 \) becomes:

\[
Q = 4.43 b w \sqrt{H - w}
\]

where \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( H \) is the energy head above the sill in \( \text{m} \), \( b \) is the width of the throat in \( \text{m} \), \( w \) is the gate opening in \( \text{m} \).

The conditions for the Crump-deGruyter gate are:
- the contraction coefficient \( \mu \approx 1 \) for \( w < 0.5d \), where \( d \) is the diameter of the flow-streamliner;
- the free flow condition is satisfied for a headloss \( z > 0.3H \);
- the gate opening \( w < \frac{3}{2}H \) to allow for underflow.

The maximum discharge depends on maximum gate opening \( w < \frac{3}{2}H \). Hence:

\[
Q_{\text{max}} = 4.43 \frac{3}{2} \frac{3}{2} H (H - \frac{3}{2}H)^{0.5} = 1.71 b H^{1.5}
\]

which appears the overflow formula.
Application. The Crump-deGruyter gate is an improvement of the vertical gate, as the rating curve is more simple. The Crump-deGruyter gate is recommended when an underflow regulator with measurement provisions is needed. Its hydraulic functions are good and the structure is reasonable in costs.

The disadvantages of the Crump-deGruyter gate are: the headloss is relative large, the discharge measurements needs two readings: upstream water level and gate opening. The structure faces problems with floating debris.

8.3. Hydraulics of Underflow Gates

8.3.1. Types of Flow

Types of flow. The flow through a gated discharge regulator is quite complex, and the rating curve is not straightforward. Basically, there are five different flow types through a structure with a gated discharge regulator, see figure 8.8 and figure 8.9:

- **free overflow.** The water level or 'nappe' ("straal") does not touch the gate, and the tailwater level is sufficient low to allow for a free overflow, like the flow over a broad-crested weir.

- **conveyance flow.** The water level does not touch the gate, and the tailwater level is so high that the flow is submerged, like the flow below a bridge.

- **free underflow.** The flow under the gate is not influenced by the tailwater level. The outflowing jet is open to the atmosphere because the hydraulic jump is formed at some distance from the gate. The rating curve will be discussed below.

- **partially-submerged underflow.** The flow under the gate is super-critical ($Fr > 1$). However, the flow is influenced by the tailwater level as the jet is overlaid by the hydraulic jump. The rating curve will be discussed below.

- **fully-submerged underflow.** The flow under the gate is sub-critical ($Fr < 1$), and as such, is influenced by the tailwater level. Also this rating curve will be discussed below.

![Figure 8.8. Types of "weir flow" through a structure with a vertical gate.](image-url)
Figure 8.9. Types of "orifice flow" through a structure with a vertical gate.

8.3.2. Hydraulics of the vertical gate under free flow

Bernoulli equation for free flow. The Bernoulli equation for a vertical gate under free flow, can be written, see also figure 8.9:

\[ H = y_1 + \frac{q^2}{2g \, y_1^2} = y_2 + \frac{q^2}{2g \, y_2^2} \]

hence for \( y_2 = \mu \, w \) the equation of the rating curve reads:

\[ q = \sqrt{2g \, y_2^2 \, (H - y_2)} = \mu \, w \sqrt{2g \, (H - \mu \, w)} \]

where \( q \) is the discharge per unit width in \( \text{m}^3/\text{m.s} \), \( y_2 \) is the water depth of the jet above the sill in m, \( H \) is the energy head above the sill in m, \( w \) is the opening of the vertical gate in m, \( \mu = 0.6 \) is the contraction coefficient of the jet, and \( g = 9.8 \, \text{m/s}^2 = 9.8 \, \text{N/kg} \) is the gravity acceleration.

General discharge equation. The above discharge equation is often not used as it cannot be applied for the submerged flow. Therefore, the Bernoulli equation is processed in an alternative way, and a more general discharge equation is obtained (Henderson 1966):

\[ q^2 = \frac{y_1 - y_2}{1} - \frac{1}{2g \, y_2^2} = \frac{1}{2g \, y_1^2} \left( \frac{y_1^2}{y_1 + \mu \, w} \right) \]

\[ q = \mu \, w \sqrt{2g \, \frac{y_1^2}{y_1 + \mu \, w}} = \frac{\mu}{\sqrt{1 + \frac{w}{y_1}}} \, w \sqrt{2g \, y_1} \]
Thus, the general vertical gate formula reads:

$$Q = C_D b w \sqrt{\frac{2g}{y_1}} \text{, with: } C_D = \frac{\mu}{\sqrt{1 + \mu \frac{w}{y_1}}}$$

where $Q$ is the discharge in m$^3$/s, $b$ is the width of the gate in m, $C_D$ is the discharge coefficient, $w$ is the opening of the vertical gate in m, $y_1$ is the upstream water depth above the sill in m, $\mu = 0.6$ is the contraction coefficient of the jet, and $g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg}$ is the gravity acceleration.

Discharge coefficient. The discharge coefficient $C_D$ for free flow can be calculated with the above formula for the ratio $y_1/w$. The calculated values for the higher ranges correspond reasonably well with experimental data. However, the calculated values of $C_D$ for the practical range $y_1/w < 4$ do not correspond well with the experimental data, as the calculated values are some 5% too high.

It is recommended to use the experimental values of the discharge coefficient $C_D$ as presented in figure 8.10 (Chow 1959, Henderson 1966, Bos 1989).

**Figure 8.10.** $C_D$ coefficients for the vertical gate under submerged and free flow.
Discharge coefficient for overflow. Two types of flow may occur when the gate opening \( w = \frac{7}{8} y_1 \); (i) the underflow when the nappe touches the gate, and (ii) the overflow when the nappe does not touch the gate. Also the overflow can be calculated by the vertical gate formula, but only for \( w = \frac{7}{8} y_1 \):

The discharge coefficient \( C_D \) follows from the actual discharge according to the (broad-crested) weir formula and the gate formula for \( w = \frac{7}{8} y_1 \):

**Weir formula**: \( q = 1.7 H^{3/2} \approx 1.7 y_1^{3/2} \),  **Orifice formula**: \( q = C_D \frac{7}{8} y_1 \sqrt{2g y_1} \)

and together:

\[
1.7 y_1^{3/2} = C_D \frac{7}{8} y_1 \sqrt{2g y_1}, \quad \text{and so:} \quad C_D = \frac{1.7}{\frac{7}{8} \sqrt{2g}} = 0.58
\]

Validity for free flow. Free flow through a vertical gate is guaranteed when the hydraulic jump does not disturb the jet. So, the tailwater depth \( y_3 \) has to be lower than the "conjugate depth", i.e. the tailwater depth needed to form a hydraulic jump with the water depth \( y_2 \) of the jet. Thus:

\[
y_3 < \frac{7}{8} y_2 \left( \sqrt{1 + 8 \frac{Fr_2^2}{w} - 1} \right), \quad \text{and} \quad y_2 = \mu w
\]

The Froude number \( Fr_2 \) of the jet with depth \( y_2 \):

\[
Fr_2 = \sqrt{\frac{q^2}{g y_2^3}} = \sqrt{\frac{C_D^2 w^2 2 g y_1}{g \mu^3 w^3}} = C_D \sqrt{\frac{2 \frac{y_1}{w}}{\mu^3}}
\]

Hence, the hydraulic jump will not occur for a ratio between the tailwater depth \( y_3 \) and the gate opening \( w \):

\[
\frac{y_3}{w} < \frac{\mu}{2} \left( \sqrt{1 + 16 C_D^2 \frac{1}{\mu^3} \frac{y_1}{w}} - 1 \right)
\]

The above equation is confirmed by the experimental results of figure 8.10 for \( y_1/w < 10 \). The calculated data are presented in table 8.1.

**Table 8.1. Parameters of the vertical gate under free flow.**

<table>
<thead>
<tr>
<th>Upstream Water Depth ( y_1/w )</th>
<th>Discharge Coefficient ( C_D )</th>
<th>Maximum Tailwater ( y_2/w )</th>
<th>Minimum Headloss ( z/w )</th>
<th>Minimum Rel. Headloss ( z/y_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1.5 Overflow)</td>
<td>(0.58)</td>
<td>(1.00)</td>
<td>(0.50)</td>
<td>(±0.33)</td>
</tr>
<tr>
<td>1.5</td>
<td>0.50</td>
<td>1.31</td>
<td>0.19</td>
<td>0.13</td>
</tr>
<tr>
<td>2.0</td>
<td>0.51</td>
<td>1.59</td>
<td>0.41</td>
<td>0.21</td>
</tr>
<tr>
<td>3.0</td>
<td>0.52</td>
<td>2.04</td>
<td>0.96</td>
<td>0.32</td>
</tr>
<tr>
<td>4.0</td>
<td>0.53</td>
<td>2.45</td>
<td>1.55</td>
<td>0.39</td>
</tr>
<tr>
<td>5.0</td>
<td>0.55</td>
<td>2.89</td>
<td>2.11</td>
<td>0.42</td>
</tr>
<tr>
<td>6.0</td>
<td>0.56</td>
<td>3.25</td>
<td>2.75</td>
<td>0.46</td>
</tr>
<tr>
<td>7.0</td>
<td>0.56</td>
<td>3.54</td>
<td>3.46</td>
<td>0.49</td>
</tr>
<tr>
<td>8.0</td>
<td>0.57</td>
<td>3.87</td>
<td>4.13</td>
<td>0.52</td>
</tr>
<tr>
<td>9.0</td>
<td>0.57</td>
<td>4.13</td>
<td>4.87</td>
<td>0.54</td>
</tr>
<tr>
<td>10.0</td>
<td>0.58</td>
<td>4.45</td>
<td>5.55</td>
<td>0.56</td>
</tr>
</tbody>
</table>
The above equation can be simplified by assuming $\mu = 0.6$ and $C_D = 0.55$, so that:

$$\frac{y_3}{w} < 0.3 \sqrt{1 + 16 \frac{0.55^2}{0.6^3} \frac{y_1}{w} - 0.3} = 0.3 \sqrt{1 + 22.4 \frac{y_1}{w} - 0.3}$$

Hence, a vertical gate will flow under free flow for:

$$\frac{y_3}{w} < 1.42 \left( \frac{y_1}{w} \right)^{0.5} - 0.3$$

**Free flow.** The maximum tailwater depth $y_3/w$ from Table 8.1 has been plotted against the upstream water depth $y_1/w$, see Figure 8.11.

In practical terms, it means that free underflow at a vertical gate occurs when the tailwater level $y_3$ is below the critical value:

$$\frac{y_3}{w} < 0.37 \frac{y_1}{w} + 0.75 \text{ or } y_3 < 0.37 y_1 + 0.75 w$$

This can be rewritten as a criterion for the headloss $z$:

$$z = y_1 - y_3 \text{ so: } z > y_1 - 0.37 y_1 - 0.75 w = 0.63 y_1 - 0.75 w.$$  

As the gate opening $w$ is limited in practice by $\pm 0.15 y_1 < w < 0.5 y_1$, the minimum headloss $z_{\text{min}}$ of a vertical gate under free flow becomes:

$$\frac{1}{4} y_1 < z_{\text{min}} < \frac{1}{2} y_1.$$  

where the headloss $z_{\text{min}} > \frac{1}{4} y_1$ is required for the larger gate openings $w \approx 0.5 y_1$.

---

**Figure 8.11.** Conditions for the vertical gate under free flow.
Vertical gate for discharge regulation and measurement. The use of a vertical gate for discharge regulation and discharge measurement, under free flow conditions, is shown in figure 8.12. The design of the structure is based on an upstream water level $y_1$ and a design gate opening $w = 0.5 \ y_1$. A larger design gate opening would create the danger that the underflow would be transformed into an overflow at $w = 0.67 \ y_1$.

The range for discharge measurement would be set at $Q_{25\%}$ to $Q_{100\%}$. It is calculated in box 8.1 that the gate opening for $Q_{25\%}$ would be at $w = 0.11 \ y_1$.

The required headloss over the vertical gate for accurate measurement for the above range would be $z > 0.56 \ y_1$.

The maximum discharge for underflow would amount to $130\%$ when the gate is lifted to $w = 0.67 \ y_1$. The discharge jumps to $150\%$ when the nappe does not touch the gate and overflow starts. The rating curve is presented in figure 8.13.

![Figure 8.12. Discharge and minimum headloss of the vertical gate.](image)

**Box 8.1. Example calculation of the vertical gate with free underflow.**

A gate with opening $w = 0.5 \ y_1$ ($y_1/w = 2.0$) requires a minimum headloss of $z = 0.21 \ y_1$ for free flow. The corresponding discharge $q$ amounts to $Q = C_D \ b \ w (2g \ y_1)^{0.5} = 0.51 \times b \times y_1/2.0 (2g \ y_1)^{0.5} = 0.255 \times b \times y_1 (2g \ y_1)^{0.5}$, and can be set $Q = 100\%$.

Similarly, the discharge for $w = 0.25 \ y_1$ ($y_1/w = 4.0$) can be calculated at $Q = 0.132 \times b \times y_1 (2g \ y_1)^{0.5}$, which is $Q = 50\%$, at a minimum headloss of $z = 0.39 \ y_1$ for free flow.

Also, the discharge for $w = 0.11 \ y_1$ ($y_1/w = 9.0$) can be calculated at $Q = 0.063 \times b \times y_1 (2g \ y_1)^{0.5}$, which is $Q = 25\%$, at a minimum headloss of $z = 0.54 \ y_1$ for free flow.

The maximum flow under the gate occurs for $w = ½ \ y_1$ ($y_1/w = 1.5$), when the discharge reaches $Q = 0.333 \times b \times y_1 (2g \ y_1)^{0.5}$, which is $Q = 130\%$, at a minimum headloss of $z = 0.13 \ y_1$ for free flow.

However, the maximum flow through the structure will occur when the gate is opened at $w = ½ \ y_1$ ($y_1/w = 1.5$) and the gate flow is transformed into a weir flow: $Q = 0.387 \times b \times y_1 (2g \ y_1)^{0.5}$, which is $Q = 150\%$, at a minimum headloss of $z = 0.33 \ y_1$ for free flow.
8.3.3. Hydraulics of the vertical gate under submerged flow

Bernoulli equation for submerged flow. The Bernoulli equation for the submerged flow under a vertical gate depends on the total water depth $y$ instead of water depth of the jet $y_2$. So the Bernoulli equation can be written as:

$$y_1 + \frac{q^2}{2gy_1^2} = y + \frac{q^2}{2gy_2^2}$$

hence for $y_2 = \mu w$ follows:

$$q^2 = 2g \mu^2 w^2 y_1 \frac{y_1 - y}{y_1^2 - \mu^2 w^2}$$
Discharge coefficient formula. As the general discharge equation reads: \( q^2 = C_D^2 \mu^2 \frac{w^2}{2g} \), the value of \( C_D \) can be calculated out of the formula for a known value of \( y \):

\[
C_D = \sqrt{\frac{2g \mu^2 \frac{w^2}{2} \frac{y_1^2}{y_1^2 - \mu^2 \frac{w^2}{2}}}{\frac{y_1 - y}{y_1^2 - \mu^2 \frac{w^2}{2}}}} = \mu \sqrt{\frac{1 - \frac{y}{y_1}}{1 - \frac{\mu^2 \frac{w^2}{2}}{y_1^2}}}
\]

Momentum equation. The value of the water depth \( y \) can be calculated by considering the momentum equation:

\[
\frac{q}{g y_2} + \frac{y^2}{2} = \frac{q^2}{g y_3} + \frac{y_3^2}{2}
\]

There are two equations (Bernoulli equation and momentum equation), one condition \( (y_2 = \mu \frac{w}{2}) \), and three known parameters \( (y_1, y_3 \text{ and } w) \), and three unknown parameters \( (q, y_2 \text{ and } y) \). So the unknown value of \( y \) can be calculated by the substitution of the term "\( q^2/g \)".

Discharge coefficient. The analytic solution of the value \( y \) appears to be very cumbersome. Furthermore, the calculated value of the discharge coefficient \( C_D \) differs from the experimental value.

Therefore, it is recommended to read the value of the discharge coefficient \( C_D \), also for submerged conditions, from the graphs in figure 8.10 (or figure 8.14).

Froude number. The Froude number of the jet under submerged conditions equals the above formula for the Froude number under free flow conditions.

\[
Fr_2 = C_D \sqrt{\frac{2 \frac{y_1}{w}}{\mu^3}}
\]

thus:

\[
C_D = Fr_2 \sqrt{\frac{\mu^3}{2 \frac{y_1}{w}}} = 0.33 Fr_2 \sqrt{\frac{1}{\frac{y_1}{w}}}
\]

The Froude numbers depends on the ratio \( y_1/w \) and the value of \( C_D \), and has been plotted in figure 8.14. It can be seen that the submerged conditions occur for both Froude numbers \( Fr < 1 \), as well as for Froude numbers \( Fr > 1 \).

Accuracy for submerged flow. It is obvious that the reading of the discharge coefficient \( C_D \) for submerged flow is not very accurate. Especially, the determination of the \( C_D \)-value for submerged flow may result in considerable errors if the head differences \( z \) between upstream water level \( y_1 \) and the tailwater level \( y_3 \) become small. Hence, the calculation of the discharge of a vertical gate under submerged conditions will be of a low accuracy, see box 8.2.
**Figure 8.14.** Froude numbers of flow under vertical gate.

**Box 8.2.** Example calculation of the vertical gate under submerged underflow.

The vertical gate of $b = 1.00$ m wide of the above example can also flow under submerged conditions. Assume a gate opening of $w = 0.20$ m. The upstream water level $y_1$ is kept constant by a water level regulator at $y_1 = 0.80$ m.

The discharge under free flow depends on the ratio $y_1/w = 4.0$ and the discharge coefficient $C_D = 0.53$, and can be calculated at amounts to $Q_{\text{free flow}} = C_D \cdot b \cdot w \cdot (2g \cdot y_1)^{0.5} = 0.53 \times 1 \times 0.20 \times (2g \times 0.80)^{0.5} = 0.42$ m$^3$/s. This discharge is valid for tailwater water levels $y_3 < 2.45 \times w = 2.45 \times 0.20 = 0.49$ m.

Higher tailwater levels $y_3$ lead to submerged flow and a reduction of the discharge, following the equation: $Q_{\text{submerged flow}} = C_D \cdot b \cdot w \cdot (2g \cdot y_1)^{0.5} = C_D \times 1 \times 0.20 \times (2g \times 0.80)^{0.5} = 0.79 \cdot C_D$. The value $C_D$ can be read from the graphs in figure 8.10, and the discharges for increasing tailwater levels can be calculated:

- $y_3 = 0.50$ m: $y_3/w = 2.5$ and for $y_1/w = 4.0$ is $C_D = 0.50$, thus $Q = 0.40$ m$^3$/s,
- $y_3 = 0.55$ m: $y_3/w = 2.8$ and for $y_1/w = 4.0$ is $C_D = 0.40$, thus $Q = 0.32$ m$^3$/s,
- $y_3 = 0.60$ m: $y_3/w = 3.0$ and for $y_1/w = 4.0$ is $C_D = 0.37$, thus $Q = 0.29$ m$^3$/s,
- $y_3 = 0.65$ m: $y_3/w = 3.3$ and for $y_1/w = 4.0$ is $C_D = 0.32$, thus $Q = 0.25$ m$^3$/s,
- $y_3 = 0.70$ m: $y_3/w = 3.5$ and for $y_1/w = 4.0$ is $C_D = 0.25$, thus $Q = 0.20$ m$^3$/s,
- $y_3 = 0.75$ m: $y_3/w = 3.8$ and for $y_1/w = 4.0$ is $C_D = 0.15$, thus $Q = 0.12$ m$^3$/s,
- $y_3 = 0.80$ m: $y_3/w = 4.0$ and for $y_1/w = 4.0$ is $C_D = 0.00$, thus $Q = 0.00$ m$^3$/s.
8.3.4. Hydraulics of the radial gate

Radial gate and the vertical gate. The hydraulics of the radial gate are basically the same as for the vertical gate. Thus, the general gate formula for the radial gate reads, see figure 8.15:

\[ Q = C_D \cdot b \cdot w \cdot \sqrt{\frac{2g}{y_1}} \]

where \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( b \) is the width of the gate in m, \( C_D \) is the discharge coefficient, \( w \) is the opening of the radial gate in m, \( y_1 \) is the upstream water depth above the sill in m, \( \mu \approx 0.6 \) is the contraction coefficient of the jet, and \( g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg} \) is the gravity acceleration.

![Diagram of the radial gate](image)

**Figure 8.15.** Hydraulics of the radial gate.

Contraction coefficient. The contraction coefficient \( \mu \) of the radial gate is larger than the coefficient of the vertical gate, as it is influenced by the angle \( \theta \) (in °) of the gate-lip with the horizontal plane. Thus:

\[ \mu = 1 - 0.75 \left( \frac{\theta}{90°} \right) + 0.36 \left( \frac{\theta}{90°} \right)^2 \]

The angle \( \theta \) (in °) of the gate-lip with the horizontal plane can be calculated from the gate opening \( w \) by simple goniometry, see figure 8.16. Thus:

\[ \frac{w}{R} = \frac{a}{R} - \sin (90° - \alpha_0 - \alpha), \quad \text{or:} \quad \sin (90° - \theta) = \frac{a}{R} - \frac{w}{R} \]

Radial Gate under free flow. The discharge coefficient \( C_D \) for the radial gate under free flow is basically equal to the equation of the vertical gate, see also figure 8.16:

\[ C_D = \mu \left[ 1 + \mu \cdot \frac{w}{y_1} \right] \]

The values of the discharge coefficient \( C_D \) for the radial gate \((a/R = 0.9)\) under free flow are presented in table 8.2.
Validity of free flow. Free flow through a radial gate is guaranteed when the hydraulic jump does not disturb the jet. The actual tailwater depth can be calculated by means of the Froude number, as shown for the vertical gate. It may be expected that the tailwater depth of the radial gate is more or less equal to the tailwater depth at the vertical gate. In practical terms, it would mean that a radial gate under free underflow requires a minimum headloss \( z_{\text{min}} = 0.37 \, y_1 + 0.75 \, w \), but further analysis may be required.

<table>
<thead>
<tr>
<th>Upstream Level</th>
<th>Gate Opening ( w/R = 0.1 )</th>
<th>Gate Opening ( w/R = 0.3 )</th>
<th>Gate Opening ( w/R = 0.5 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( y_1/R )</td>
<td>Tailwater ( y_3/R = 0.5 )</td>
<td>Tailwater ( y_3/R = 0.7 )</td>
<td>Tailwater ( y_3/R = 0.5 )</td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.6</td>
<td>0.33</td>
<td>0</td>
<td>0.50</td>
</tr>
<tr>
<td>0.7</td>
<td>0.44</td>
<td>0</td>
<td>0.59*</td>
</tr>
<tr>
<td>0.8</td>
<td>0.51</td>
<td>0.28</td>
<td>0.60*</td>
</tr>
<tr>
<td>0.9</td>
<td>0.58</td>
<td>0.39</td>
<td>0.61*</td>
</tr>
<tr>
<td>1.0</td>
<td>0.64</td>
<td>0.44</td>
<td>0.62*</td>
</tr>
<tr>
<td>1.1</td>
<td>0.69</td>
<td>0.48</td>
<td>0.62*</td>
</tr>
<tr>
<td>1.2</td>
<td>0.73*</td>
<td>0.53</td>
<td>0.63*</td>
</tr>
<tr>
<td>1.3</td>
<td>0.74*</td>
<td>0.57</td>
<td>0.63*</td>
</tr>
<tr>
<td>1.4</td>
<td>0.74*</td>
<td>0.59</td>
<td>0.64*</td>
</tr>
<tr>
<td>1.5</td>
<td>0.75*</td>
<td>0.62</td>
<td>0.64*</td>
</tr>
</tbody>
</table>

* Free flow
Radial Gates under submerged flow. The submerged flow through the radial gate can be calculated by means of the Bernoulli and the momentum equations, and the parameter y as for the vertical gate.

Model tests are only available for radial gates of $a/R = 0.1$, $a/R = 0.5$ and $a/R = 0.9$ (Chow 1959, Henderson 1966). The gate $a/R = 0.1$ is not applicable in irrigation and drainage engineering because of the large radius $(R = 10 \times a)$. Also the experiments on the gate $a/R = 0.5$ are not applicable, as the tailwater depth is taken quite high, see figure 8.17. Only the radial gate of $a/R = 0.9$ is of practical use. These discharge coefficients $C_D$ are also presented in table 8.2.

![Diagram of radial gate with model tests](image)

Figure 8.17. Model tests on the radial gate to determine the $C_D$ coefficients.
9. DESIGN OF HYDRO-MECHANICAL REGULATORS

9.1. Hydro-Mechanical Regulators for Drainage

9.1.1. Flap Gate

General. 'Flap gates' ("duiker-klep", "uitwateringsduiker") are automatic gates, and consist of a gate-leaf that pivot about a horizontal hinge. In drainage, flap gates are opened by the water pressure, i.e. the higher tailwater levels will close the gate, see figure 9.1. It is also possible that an operator opens and closes the gate manually. Flap gates as discharge regulators are widely applied at drainage outfalls.

Hydraulics. The rating curve ("Q-h kromme") of the flap gate is not available for practical use, as it depends on too many parameters, such as the angle of flap gate, the weight of the flap gate and the submerged ratio.

Design. The hydraulic design of the structure with a flap gate will be quite a problem as the rating curve of the flap gate is not readily available. The design discharge has to pass through the structure at the maximum gate angle, when the nappe has the maximum contraction

![Fig 9.1: Flap gate, for automatic regulation in drainage.](image-url)

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9.1.2. Tidal Sluice

**General.** Drainage water can be evacuated from a polder by gravity through a tidal sluice ("uitwateringssluis", "suatiesluis") in the dike during low tide, when a polder is surrounded by open water in which tides are active. These structures should be designed at the smallest dimensions, to limit the costs, by aiming at:

- **overflow** ("overlaat"), instead of underflow (orifice) flow,
- **free flow** ("volkomen stroming"), instead of submerged flow.

**Operation rules.** Drainage outlets are operated a sort of 'mixed control' that operates with two 'setpoints' ("streepeil"): the inside setpoint at the inner side of the structure, and the outside setpoint at the outer side, see figure 9.2. The operation rules are:

- the gate is closed at an internal water level ("binnenwaterstand") below the inside setpoint;
- the gate is open at an internal water level above the inside setpoint, and for an external water level ("buitenwaterstand") below outside setpoint;
- the gate is closed at an internal water level above the inside setpoint, but the external water level is above the outside setpoint.

![Diagram of Tidal Sluice](image)

**Figure 9.2.** Tidal sluice under combined up- and downstream control.

**Discharge formula.** The discharge formula of a sluice and a culvert, both with a free water level, reads:

- **for free flow:**
  \[ Q = 1.7 \ b \ H^{1.5} \]  
  valid for \( z > \frac{1}{3} H \)

- **for submerged flow:**
  \[ Q = \sqrt{\frac{1}{\alpha}} \ b \ y \ \sqrt{2 \ g \ z} = 4.4 \ \sqrt{\frac{1}{\alpha}} \ b \ y \ \sqrt{z} \]  
  valid for \( z < \frac{1}{3} H \)

where \( Q \) is the sluice discharge in m³/s, \( b \) is the width of the sluice in m, \( H \) is the upstream energy head above the sill in m, \( y \) is the (outer) water level above the sill in m, \( z \) is the (energy) headloss over the structure in m, and \( \alpha \) is a coefficient (\( \alpha \approx 1.0 \)).

**Method of calculation.** The design calculations of a tidal sluice have to be done in time-steps. For each time step, the outfall volume should be calculated for that specific tidal water level. This outfall volume decreases the volume of water in the polder, so the open-water level in the polder decreases. This new water level is the input for the next time-step, see figure 9.3.

The whole calculation can be made by means of a table, see table 9.1. An example is presented in box 9.1.
Box 9.1. Design of tidal sluice.

As an example, the width of a tidal outlet structure is designed. This tidal sluice drains a polder area of 6000 ha. The open-water system has an area of 125 ha, and is supplied by a field drainage system at a rate of 10 mm/day.

The sill elevation is located at the bed level of the drainage canal at 1.20 m-MSL. The tide in the sea is a "single-daily" tide, i.e. a cycle of $12\frac{1}{2}$ hours high water and $12\frac{1}{2}$ hours low water, see figure 9.3.

The water level in the drainage system after flushing must be again at the target water level at mean sea level (MSL). The water level will rise during the period that the tidal gates are closed, e.g. during about 10 hours, thus $10/24 \times 6000 / 125 \times 0.010 = 0.20$ m. The total volume that have to be discharged during one sluicing-period amounts to $25/24 \times 0.010 \times 6000 \times 10^4 = 625,000$ m$^3$.

The flow will be mainly free flow. Submerged flow will occur only for $z < \frac{1}{3} H$, i.e. for the higher outer water levels:

- during the beginning of the sluicing period for an inner water level of 0.20 m+MSL, when $z < 1/3 \times 1.40 = 0.47$ m, thus for an outer water level of $y > 0.93$ m (> 0.27 m-MSL),
- at the end of the sluicing period for an inner water level of 0.00 m+MSL, when $z < 1/3 \times 1.20 = 0.40$ m, thus for an outer water level of $y > 0.80$ m ( > 0.40 m-MSL).

The calculation is made by means of a simple computer program or by hand, see table 9.1. The time-step may be taken at one hour. The total volume of outflowing discharge depends on the sluice width $b$, and amounts to $34.28 \times b \times 3600 = 123,408 \times b$ m$^3$.

It was already known that the total volume equals to $625,000$ m$^3$, so that $123,408 \times b = 625,000$. It means that the minimum required width $b$ of the sluice amounts to: $b = 5.06$ m.
Table 9.1. Design of a tidal sluice.

<table>
<thead>
<tr>
<th>Timestep in hrs</th>
<th>Water levels</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inside &quot;H&quot;</td>
<td>Outside &quot;y&quot;</td>
</tr>
<tr>
<td></td>
<td>m+MSL m</td>
<td>m+MSL m</td>
</tr>
<tr>
<td>11-12</td>
<td>+0.19 1.39  -0.02 1.18</td>
<td>0.21</td>
</tr>
<tr>
<td>12-13</td>
<td>+0.17 1.37  -0.10 1.10</td>
<td>0.27</td>
</tr>
<tr>
<td>13-14</td>
<td>+0.16 1.36  -0.38 0.82</td>
<td>0.54</td>
</tr>
<tr>
<td>14-15</td>
<td>+0.14 1.34  -0.56 0.64</td>
<td>0.70</td>
</tr>
<tr>
<td>15-16</td>
<td>+0.13 1.33  -0.80 0.40</td>
<td>0.93</td>
</tr>
<tr>
<td>16-17</td>
<td>+0.11 1.31  -0.88 0.32</td>
<td>0.99</td>
</tr>
<tr>
<td>17-18</td>
<td>+0.10 1.30  -0.96 0.24</td>
<td>1.06</td>
</tr>
<tr>
<td>18-19</td>
<td>+0.08 1.28  -1.00 0.20</td>
<td>1.08</td>
</tr>
<tr>
<td>19-20</td>
<td>+0.07 1.27  -0.96 0.24</td>
<td>1.03</td>
</tr>
<tr>
<td>20-21</td>
<td>+0.05 1.25  -0.92 0.28</td>
<td>0.97</td>
</tr>
<tr>
<td>21-22</td>
<td>+0.03 1.24  -0.82 0.38</td>
<td>0.86</td>
</tr>
<tr>
<td>22-23</td>
<td>+0.02 1.22  -0.70 0.50</td>
<td>0.72</td>
</tr>
<tr>
<td>23-24</td>
<td>+0.01 1.21  -0.56 0.64</td>
<td>0.57</td>
</tr>
<tr>
<td>24-25</td>
<td>+0.00 1.20  -0.33 0.87</td>
<td>0.33</td>
</tr>
</tbody>
</table>

Total Volume of outflowing discharge: 34.28×b×3600

9.2. Hydro-Mechanical Regulators for Upstream Control

9.2.1. Begemann gate

**General.** The Begemann gate is an automatic gate for a constant upstream water level and has been developed in Indonesia in the 1930s (Vlugter 1940, Brouwer 1987).

**Principle.** The Begemann gate consists of a flat gate-leaf which pivots around a horizontal axis, see figure 9.4. The upstream water pressure tends to open the gate, the torque from the weight of the gate and the counterweight tends to keep the gate closed.

The Begemann gate is simple in construction, but requires a large headloss z > h. The downstream water level may not touch the gate-leaf, as this would introduce an additional closing torque on the gate.
Figure 9.4. Begemann gate, for regulation of the upstream water level.

Hydraulics. The depth $h$ of the sill and the width $b$ of the structure follows from the broad-crested weir formula ($Q = 1.7 \, b \, H^{3/2}$) and from the $b/h$ relation as applicable for gate-leafs.

The location of the axis determines the sensitivity of the gate for water level variations. The height $a$ of the axis above the target water level is recommended at $a = \frac{1}{2} \, h$. A larger value of $a$ will increase the decrement $\Delta h$. The distance $p$ of the axis and the gate-leaf depends on the location of the counterweight. It is recommended here to place the counterweight above the gate-leaf, see also figure 9.4, so that the value $p \leq \frac{3}{8} \, h$. The Begemann gate can also be transformed into a "hook" gate by placing the counterweight just above the axis, which means that a larger value $p$ is required for the closed position, i.e. $p \approx 6 \, h$.

The closing torques by the gate weight $G$ and by the counterweight $W$ are in equilibrium with the hydraulic torque on the gate, so that for closed position is valid:

$$\frac{1}{2} \, \rho g \, b \, h^2 \left( \frac{2}{3} h + a \right) = p \times (G + W)$$

The weight $W$ of the counterweight can be calculated from the above equation, and for $a = \frac{1}{2} \, h$ and $p = \frac{3}{8} \, h$ follows:

$$W = (1.6 \, \rho g \, b \, h^2) - G$$

The location of the counterweight $W$ is determined by the angle $\alpha$, see also figure 9.4, and determines the decrement $\Delta h$. Model tests have shown (Vlugter 1940) that the angle $\alpha$ should be in the range from $15^\circ$ to $55^\circ$. The adjustment of the counterweight during the installation of the gate will provide the exact location.

Decrement. The decrement of the Begemann gate depends on the angle $\alpha$, and can estimated by the equation $\Delta h = 0.4 - 0.17 \, \tan \, \alpha$, see figure 9.5.

Generally, an angle of $\alpha \approx 45^\circ$ to $55^\circ$ provides the lower decrements of $\Delta h \approx 0.20 \, h$. Thus, the stagnant water depth in the structure will be at $h - \Delta h$. The rating curve of the Begemann gate is shown also in figure 9.5.
9.2.2. Vlugter gate

General. The Vlugter Gate is an automatic gate for a constant upstream water level and operates at a very low headloss $z$ during the design discharge $Q$.

The gate has been developed in Indonesia in the 1930s (Vlugter 1940) and is successfully used in irrigation canals, in drains and in rivers. Winches ("windwerk, lier") are usually applied to open the (automatic) gate to release any floating debris ("drijvend vuil").

Principle. The gate exists of a floating drum that functions as a gate-leaf, and of a counterweight, see figure 9.6. The front face of the drum is vertical, but the drum has a semi-circular rear face. The centre of the circle of the rear face is the same as the rotation axis of the gate so that the hydraulic thrust of the downstream water level passes through the axis and does not lead to a closing torque.

The counterweight is located above the drum to provide a closing torque during the lower discharges, and to provide no torque at the higher discharges as to reduce the headloss over the gate. The location of the counterweight $W$ is also determined by the angle $\alpha$, see also figure 9.6.

The model tests of the Vlugter gate have been based on the angle $\alpha = 45^\circ$. The determination of the weight $W$ of the counterweight is similar to the Begemann gate, but the exact counterweight has to be determined in the field until the gate opens and closes correctly.
9. Design of Hydro-Mechanical Regulators

**Figure 9.6.** Vlugter gate, for regulation of the upstream water level.

**Hydraulics.** The depth $h$ of the sill and the width $b$ of the structure follow from the conveyance formula ($z = 1.0 \, \nu^2/2g$) and the relation $b/h = 5$ as applied in the modeltests. The headloss $z$ will occur during the design discharge $Q$, when $\nu$ is the velocity through the opening of the structure without gate. The height $a$ of the axis above the target water level is recommended at $a = \frac{1}{2} \, h$.

**Decrement.** The decrement $\Delta h$ of the Vlugter gate has been found in the modeltests and amounts to $\Delta h = 0.15 \, h$. Thus, the stagnant water depth in the structure will be at $h - \Delta h$. The rating curve of the Vlugter gate is shown in figure 9.7.

**Figure 9.7.** Rating curve of the Vlugter gate (Vlugter 1940).
9.2.3. AMIL gate

General. The AMIL gate is an automatic upstream water level regulator to maintain a constant water level on the upstream side (Alstom undated), see figure 9.8.

The AMIL gates have been developed by Neyropic/Neyrtec for irrigation, and are commercially manufactured by GEC Alstom ACB (Establishment Bergeron, Le Niemeyer. 10/12 avenue des Olympiades, 94132 Fontenay-Sous-Bois Cedex, France. Fax 33-1-48.73.52.19).

Principle. The gate consists of, see also figure 9.8:
- a balanced radial gate-leaf,
- a float attached to the front side of the leaf,
- a "servo-tab", i.e. a reversed bend of the gate-leaf at the top of the gate,
- two counterweights, both adjustable.

The leaf of the gate is shaped to a trapezium with sides $l_{\text{vert}} : 0.5l_{\text{hor}}$. This provides the best flow conditions with a separation from all edges when the gate starts to open. To avoid jamming, there is a small gap between the side edges of the leaf in closed position and the walls of the structure. Therefore the gate cannot be fully tight.

Figure 9.8. AMIL gate, for regulation of the upstream water level (Alstom undated).
Figure 9.9. Hydraulic parameters of the AMIL gate.

Design of gate-leaf. The AMIL gate is designed so that all the forces acting on the gate makes that the gate maintains the upstream water level. The hydraulic thrust on the gate-leaf passes through the axis and does not effect the equilibrium. A condition is that the target upstream water level ("setpoint") is at the same elevation as the axis, see figure 9.9.

Types. The various types of AMIL gates are geometrically similar to each other and designated by a dimension index $D$. The dimension index $D$ is approximately the width $d$ of the water surface, but expressed in cm, thus $D = 100d$. The AMIL gates are available in a series of increasing widths: D80, D90, D100, D110, D125, D140, D160, D180, D200, D220, D250, D280, D315, D355, D400, D450, D500, D560, D630, D710 and D800. The interval "log $D$" of these gates is constant, and amounts to $\Delta(log D) \approx 0.05$ (Ankum 1995).

Hydraulics. The free flow through the AMIL gate occurs for a headloss $z \geq 12 \times 10^{-5} D$. The rating curves of the AMIL gate can be derived from the manufacturers leaflet, see figure 9.10 and read (Ankum 1995): $Q = 4.4 \times 10^{-5} D^{2.1} z^{0.4}$ (submerged flow), $Q = 3.0 \times 10^{-6} D^{2.5}$ (free flow)
where: $Q$ is the discharge in m$^3$/s, $D$ is the dimension index of the gate, $z$ is the headloss over the gate during the design discharge $Q$ in m.

Design. The design of an AMIL gate under submerged flow for a known design discharge $Q$ and the corresponding headloss $z$, and under free flow for a known design discharge $Q$ can be based on the above formulae (Ankum 1995):

$D \approx 120 Q^{0.48} z^{0.19}$ (submerged flow), $D \approx 160 Q^{0.40}$ (free flow)

The hydraulic dimensions of the structure are also expressed in index $D$: the bed width $b = 56 \times 10^{-4} D$, the upstream water depth $h = 45 \times 10^{-4} D$. Thus, with a headloss $z \geq 0.27 h$.

Decrement. The decrement $\Delta h$ of the AMIL gate amounts to $\Delta h \approx 0.05 h$. Thus, the stagnant water depth in the structure will be at $h - \Delta h$. 
Figure 9.10. Rating curves of the AMIL gates (Alstom undated).

9.3. Hydro-Mechanical Regulators for Downstream Control

9.3.1. AVIS gate

General. The AVIS gate is a downstream water level regulator, to maintain a constant water level on their downstream side. The AVIS gate has been developed in the 1930s and is commercially manufactured by GEC Alstom ACB (Establishment Bergeron, Le Niemeyer, 10/12 avenue des Olympiades, 94132 Fontenay-Sous-Bois Cedex, France. Fax 33-1-48.73.52.19).

The name 'AVIS' has a French background: AV are the first two letters of the word "aval", which means downstream, and S is the first letter of the French word "surface". It illustrates that the gate operates at a free surface flow.

Principle. The AVIS gate consists of a cylindrical gate-leaf with a trapezoidal section, a framework and its bearings, and a float in the shape of a cylindrical sector, the whole forming a rigid assembly, see figure 9.11 and figure 9.12. The gate axis is set at the target water level, i.e. the water level at zero-flow.
The float is not directly floating in the downstream canal but is floating in a 'float chamber'. The float chamber enables the float to stand continuously in calm water so that it is not affected by turbulent flow which passes through the regulator. The float chamber is provided with a slot in the bottom for admission of the water. The rotating velocity of the gate \( \nu \) depends on the head difference \( \Delta y \) between the water level in the chamber and in the downstream canal by the formula, as well as on the (adjustable) opening: \( \nu = c \Delta y \).

A damping tank is constructed on the upstream centre portion of the AVIS gate-leaf. The damping tank is equipped with a wide opening under water, and a small calibrated opening at the top as an air-vent. The damping tank, together with the float chamber, will slow down the movement of the gate in case of sudden downstream water level changes or oscillations. In addition, the damping tank enables the gate to react temporarily but immediately to upstream water level changes.

\[\text{Figure 9.11. AVIS gate for downstream water level control (Alstom undated).}\]
High and Low Head Types. Two groups of AVIS gates are available, one for 'High Heads' and one for 'Low Heads'. The Low Head AVIS gate has a wider gate of lesser height than the High Head gate with the same float, see figure 9.13. The High Head AVIS gate has a higher height $E = 0.92 b$ of the gate leaf than the Low Head gate, which has a height $E = 0.74 b$.

Identification of gates. AVIS gates are identified by their dimension index $R/B$. The dimension index $R$ equals the float radius $r$, but expressed in cm, thus $R = 100 r$. The index $B$ equals the bed width of the structure, in cm, thus $B = 100 b$, where $B \approx 1.89 R$ for the High Head types, and $B \approx 2.13 R$ for the Low Head types.

Producible Gates. The AVIS gates are available in a series of increasing widths:

- **AVIS High Head types:** 56/106, 71/132, 90/170, 110/212, 140/265, 160/300, 180/335, 200/375, 220/425, 250/475 and 280/530;
- **AVIS Low Head types:** 90/190, 110/236, 140/300, 160/335, 180/375, 200/425, 220/475, 250/530 and 280/600.

Headloss chart. The manufacturer of the AVIS gates provides a 'headloss chart' for the selection of the AVIS gates, see figure 9.14. The chart are not very practical in the day-to-day design.

It appears that the relations in the chart can also be presented in the form of analytic formulae. These formulae are easier in the design than the graphs of the chart. Moreover, the cost aspects, which appears to be often determining, can also expressed in a formula.
Figure 9.14. Headloss chart of the AVIS gate (Alstom undated).

Alternative design formulae. The design of an AVIS gate can only be done after the design of the canal has provided: (i) the headloss $z$ over the gate during the design discharge $Q$, and (ii) the variation $V$ in upstream water level.

Thus, the selection of the AVIS type is based on the following conditions:

- The gate may not be overtopped at the highest upstream water level:
  - High Head AVIS gate: $R > 143\ (V + z)$,
  - Low Head AVIS gate: $R > 250\ (V + z)$;

- The capacity of the gate should be sufficient to pass the design discharge $Q$ at free flow:
  - High Head AVIS gate: $R > 66\ Q^{0.40}$,
  - Low Head AVIS gate: $R > 57\ Q^{0.40}$;

- The selected gate $R$ will have an actual headloss $z$ at the design discharge $Q$:
  - High Head AVIS gate: $z = 1.0 \times 10^6\ R^4\ Q^2 + 0.05 \times 10^{-2}\ R$,
  - Low Head AVIS gate: $z = 0.6 \times 10^6\ R^4\ Q^2 + 0.05 \times 10^{-2}\ R$.

If this calculated headloss $z$ is quite different from the original adopted headloss, another calculation run has to be done.

- The costs $C$ of the gate, depends also on the index $R$, and should be at the minimum:
  - High Head AVIS gate: $C \approx 8\ R^{1.8}$, in US dollar of 1990,
  - Low Head AVIS gate: $C \approx 9\ R^{1.8}$, in US dollar of 1990,
Hydraulic dimensions. The hydraulic dimensions of the structure are also expressed in the index $R$:

- **High Head AVIS gate**: the bed width $b$ of the structure $b \approx 1.89 \times 10^{-2} R$, sill level $h$ below the downstream target water level: $h \approx 1.00 \times 10^{-2} R$, the gate radius $\approx 1.60 \times 10^{-2} R$, and the decrement $\Delta h \approx 0.05 \times 10^{-2} R$.

- **Low Head AVIS gate**: the bed width $b$ of the structure $b \approx 2.13 \times 10^{-2} R$, sill level $h$ below the downstream target water level: $h \approx 1.13 \times 10^{-2} R$, the gate radius $\approx 1.80 \times 10^{-2} R$, and the decrement $\Delta h \approx 0.05 \times 10^{-2} R$.

The decrement $\Delta h$ of the AVIS gates amounts to $\Delta h \approx 0.05 h$. Thus, the stagnant downstream water depth in the structure will be at $h + \Delta h$.

**Hand winch.** The larger type of AVIS (and AVIO) gates are provided with hand winches in case hydro-mechanical control is temporarily suspended.

When an empty canal has to be filled, the discharge through the AVIS gates may be controlled manually, so as to avoid formation of a hydraulic jump outside the structure.

**Leakage.** AVIS (and AVIO) gates are water level regulators and not discharge regulators. So, they are not water-tight.

If operating conditions occasionally require the flow to be shut off completely, the AVIS gate is to be changed by the AVIO gate which is provided with an upstream orifice (emergency) gate.

**Level detection.** The float chamber of the AVIS/AVIO gates is linked with the downstream water level that is representative for the gate setting. This is the water level in the stilling basin. Apart from this case, it is also possible to connect a certain water level with the water level in the float chamber by means of a pipe. It is also advisable to use such a pipe when two or more gates are parallel. This allows a common feed of the float chambers whatever dissymmetries may occur. The levels in the float chambers remain the same and the gates are open to the same extent.

### 9.3.2. AVIO gate

**General.** The AVIS and AVIO gates are essentially similar gates. The AVIS gate is made up of a radial gate which closes completely the canal, see figure 9.15. The AVIO gate is a variant of the AVIS gate and has also a radial gate, but it closes only an orifice at an offtake of a canal or reservoir. So the AVIO is capable of standing much higher heads than the AVIS.

The letter S in the name AVIS has been replaced by the letter O, which is the first letter of the French word "orifice", to get the name AVIO.

**AVIS or AVIO?** The choice between the open-type (AVIS) and the orifice-type gate (AVIO) is solely determined by the maximum headloss likely to occur between the upstream and the downstream controlled levels. The AVIO gate is capable of standing much higher heads than the AVIS gate.
**Figure 9.15.** AVIO gate for downstream water level control (Alstom undated).

**Principle.** The damping tank does not exist on AVIO gates because of the connection with the metal-lined opening. Extra provisions have been made on the larger types to help the damping of the gate movements.

**High and Low Head Types.** Two groups of AVIO gates are available, one for 'High Heads', see figure 9.16, and one for 'Low Heads', see figure 9.17. The High Head AVIO gate has a square orifice opening with a width $b = w$, while the Low Head AVIO has a rectangular opening with a width $b = 2w$.

**Identification of gates.** AVIO gates are identified by their dimension index $R/S$. The dimension index $R$ equals the float radius $r$, but expressed in cm, thus $R = 100 \times r$. The index $S$ equals the area of the orifice of the structure, in cm².

The AVIO gates are available in a series of increasing widths:

- **AVIO Low Head types**: 28/6, 36/10, 45/16, 56/25, 71/40, 90/63, 110/100, 140/160, 160/200, 180/250, 200/315, 220/400, 250/500 and 280/630;
Figure 9.16. AVIO gate (High Head type) for downstream water level control.

Figure 9.17. AVIO gate (Low Head type) for downstream water level control.

**Headloss chart.** The manufacturer of the AVIO gates provides a 'headloss chart' for the selection of the AVIO gates, see figure 9.18. The chart are not very practical in the day-to-day design. It appears that the relations in the chart can also be presented in the form of analytic formulae. These formulae are easier in the design than the graphs of the chart. Moreover, the cost aspects, which appears to be often determining, can also expressed in a formula.

**Alternative design formulae.** The design of an AVIO gate can only be done after the design of the canal has provided (i) the headloss $z$ over the gate during the design discharge $Q$, and (ii) the variation $V$ in upstream water level. Thus, the selection of the AVIO type is based on the following conditions (Ankum 1995):

- each gate can stand a certain **maximum upstream water level**:
  - High Head AVIO gate: $R > 25 (V + z)$, Low Head AVIO gate: $R > 50 (V + z)$;
- the **capacity of the gate** should be sufficient to pass the design discharge $Q$ at free flow:
  - High Head AVIO gate: $R > 87 Q^{0.40}$, Low Head AVIO gate: $R > 73 Q^{0.40}$;
- the selected gate $R$ will have an **actual headloss** $z$ at the design discharge $Q$:
  - High Head AVIO gate: $z = 9.8 \times 10^6 R^4 Q^2 + 0.05 \times 10^2 R$,
  - Low Head AVIO gate: $z = 2.4 \times 10^6 R^4 Q^2 + 0.05 \times 10^2 R$.

If this calculated headloss $z$ is quite different from the original adopted headloss, another calculation run has to be done.

- the **costs** $C$ of the gate, in US dollar of 1990, depends also on the index $R$, and should be at minimum: High Head AVIO gate: $C \approx 9 R^{1.8}$, Low Head AVIO gate: $C \approx 11 R^{1.8}$. 
9. DESIGN OF HYDRO-MECHANICAL REGULATORS

Figure 9.18. Headloss chart of the AVIO gate (Alstom undated).

Hydraulic dimensions. The hydraulic dimensions of the structure are also expressed in the index $R$:

- **High Head AVIO gate**: the width $b$ of the orifice $b = 0.90 \times 10^{-2} R$, the height $w$ of the orifice $w = 0.90 \times 10^{-2} R$, sill level $h$ below the downstream target water level: $h = 1.07 \times 10^{-2} R$, the width $b_5$ of the structure: $b_5 = 2.50 \times 10^{-2} R$, the gate radius $R = 1.80 \times 10^{-2} R$, and the decrement $\Delta h \approx 0.05 \times 10^{-2} R$.

- **Low Head AVIO gate**: the width $b$ of the orifice $b = 1.80 \times 10^{-2} R$, the height $w$ of the orifice $w = 0.90 \times 10^{-2} R$, sill level $h$ below the downstream target water level: $h = 1.07 \times 10^{-2} R$, the width $b_5$ of the structure: $b_5 = 2.50 \times 10^{-2} R$, the gate radius $R = 1.80 \times 10^{-2} R$, and the decrement $\Delta h \approx 0.05 \times 10^{-2} R$.

The decrement $\Delta h$ of the AVIO gates amounts to $\Delta h \approx 0.05 h$. 
10. DESIGN OF DIVERSION STRUCTURES

10.1. Objectives of Diversion Structures

10.1.1. Need for Diversion Structures

Irrigation. Irrigation is normally a 'discharge control', where the larger main canal flow is diverted into secondary canals with smaller discharges. Thus, a diversion of discharges is needed at each bifurcation point ("splitspunt"), see figure 10.1.

![Diagram of irrigation and flood control system](image)

**Figure 10.1.** Need for diversion structures in irrigation and flood control.

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Drainage. Drainage is normally a pure 'water level control' without a diversion of discharges. For instance, rainfall enters into the smaller drainage channels which are flowing into the larger drainage channels, just by gravity and without any discharge regulation. Even, drainage pumping stations are operated at 'water level control', as the pumps are started when the water levels are above the target levels.

Flood control. Often, the capacity of a river in a delta is too small, and more protection against floodings ('hoogwaterbescherming') is required. The construction of a new flood diversion channel ('hoogwater afleidingskanaal') might be an option for flood control. Such a channel is attractive when the distance to the sea can be shortened, and thus the gradient. Like in irrigation, a diversion of discharges is needed at the bifurcation point ('splitsingspunt'), see also figure 10.1.

Need for diversion structures. 'Diversion structures' ('verdeelkunstwerken') are required at every offtaking channel when a discharge has to be diverted, as happens in irrigation and in flood control. Otherwise, the tailwater levels will influence the diversion. This happens because of the different depth-discharge relations of the canals ($Q = k A R^{2/3} s^{1/2}$):

- the dimensions of the two offtaking channels are different, and so are the values of the wet cross sectional areas $A$ and the hydraulic radius $R$;
- the gradients of the two offtaking channels are different, and so are the values $s$;
- the maintenance schedules are different for the two canals, and so there are different Strickler coefficients $k$ for the canals, see box 10.1;
- the bed level of the ongoing channels can be lowered (by scouring, by excavation), and so there are lower tailwater levels.

Box 10.1. Effect of the maintenance schedule on the water distribution at a 'free offtake'.

It is normal in irrigation that different canal reaches have different maintenance schedules. Thus, they have different Strickler coefficients $k$ for the canals during the irrigation season. Grass growing and sediment deposits will easily reduce the coefficient $k$ to 50% or 25% of its original value. It means that the tailwater level rises for a given discharge.

Such a rise in tailwater level in the ongoing irrigation canal means that a larger portion of the discharge is diverted from this canal.

Hence, an over-supply is given to the first offtakes in the system. The head-ends of the system receive a larger portion of the flow. As a consequence, problems of water shortage will arise at the tail-end.

Terminology. The function of a diversion structure is to divert a certain flow from an upstream channel reach, i.e. the 'incoming channel' ('onverdeelde kanaal'), into a diversion channel, i.e. the 'offtaking channel' ('aftappende kanaal') and the ongoing channel, i.e. the 'continuing channel' ('doorgaande kanaal'), see figure 10.2.

The regulator towards the offtaking channel is called 'offtake' ('aftapping'), and the regulator towards the continuing channel is called 'control' ('stuw'), see also figure 10.2.
Figure 10.2. Terminology of a diversion structure.

Need for objectives and design conditions. The selection of the type of flood diversion structure requires clear 'operational objectives' ("doelstellingen").

Furthermore, the diversion structure and the channels have an interrelation on each other that should be specified, and should be translated into 'design conditions' ("ontwerp-eisen") and 'design criteria' ("ontwerpuitgangspunten").

10.1.2. Operational Objectives of Irrigation Diversion Structures

Options for water allocation to the tertiary unit. Splitted flow is the traditional allocation method to a tertiary unit in many village-managed systems.

Intermittent ("on-off") flow to the tertiary units is often applied in schemes under dryland crops. Intermittent flow to tertiary units is essential in schemes with only 'day-irrigation', when an interval of 12 hours is required during the supply period.

Adjustable flow to the tertiary units is often applied in schemes under paddy, when the peak discharge is required e.g. during land preparation and is gradually reduced during the off-peak season.

Options for water diversion through the main system. The selection of method for water diversion through the main system has to follow the above selections on the method of water allocation.

Thus, the following options for water diversion through the main irrigation system during the off-peak period are possible, see figure 10.3 (Ankum 1992b):

- a splitted flow to tertiary units aims usually to select also a splitted flow throughout the whole main irrigation system in order to obtain a proportional diversion of the water.

However, it is also possible to change at higher levels to an adjustable flow, an intermittent flow (e.g. pumping station) or a rotational flow;
Figure 10.3. Flow diversion in the main irrigation system.
10. DESIGN OF DIVERSION STRUCTURES

- an intermittent flow to tertiary units dictates either rotational or adjustable flow in the main irrigation system. An intermittent flow in the main system would not match a regulated intermittent flow at the tertiary oftake, and could simply be obtained by a splitted flow at the tertiary oftakes;
- an adjustable flow to tertiary units dictates also an adjustable flow at the higher levels. Intermittent or rotational flow over (sub)secondary canals would be in contradiction to an adjustable flow at the tertiary oftake.

10.1.3. Operational Objectives of Flood Diversion Structures

**Operational objectives.** Many operational objectives for flood diversion are possible, such as, see figure 10.4:

- **proportional diversion,** i.e. all flow, and also the floods, are diverted 'proportionally' ("proportioneel") into a fixed ratio between the ongoing river ("doorgaande rivier") and the flood diversion channel ("hoogwaterkanaal");
- **diversion of floods only,** i.e. the flow above a certain threshold value ("drempelwaarde") is diverted into the flood diversion channel. This threshold value is determined by e.g. the capacity of the downriver (beneden rivier);
- **proportional diversion, but to a maximum,** i.e. the river flow is diverted proportionally until the capacity of the ongoing river is reached. The higher river flow is diverted into the diversion channel.

![Diagram showing different flood diversion scenarios](image)

**Figure 10.4.** Some options for flood diversion.
Diversion above the design capacity. Flood control works are designed for a 'design discharge' ("ontwerpaflue"), e.g. the 1 : 20 years' flood. Higher discharges will certainly occur, but are considered as an 'act-of-God' ("overmacht") for which provisions are not taken.

The floodings during these higher discharges above the design discharge $Q_{\text{max}}$ can be concentrated in a certain area. So it is possible to define an additional operational objective for flood diversion, see figure 10.5:

- protection of the ongoing river against floods above the design discharge,
- protection of the diversion channel against floods above the design discharge.

![Graph of diversion above the design capacity](image)

**Figure 10.5.** Options for flood diversion above the 'design discharge'.

**Design discharges.** The flood diversion structure itself is also designed on a 'design flood' ("ontwerp afvoer"). Higher floods may damage the structure. Typically, the flood diversion structure is damaged when the abutments ("landhoofden") are overtopped and water passes over the unprotected slopes. The structure will also be damaged when the stilling basin ("woelbak") does not dissipate the energy sufficiently. So, the 'return period' of the design discharge of the flood diversion structure determines the risk of failure.

The return period of the design discharge must be selected in combination with the freeboard of the abutment above the flood level. For instance, a 'freeboard' ("waking") of 1.50 m above the 1 : 50 years' design flood may provide the same protection against overtopping, as a freeboard of 0.25 m above the 1 : 1000 years' flood.

**10.1.4. Sensitivity of Diversion**

**Sensitivity.** The concept of 'sensitivity' ("gevoeligheid") evaluates the performance of bifurcations under varying discharges.

The 'sensitivity of a bifurcation' ("gevoeligheid van een splitsing") specifies the variations in the offtaking discharge in relation to the discharge in the continuing channel. In literature, the term 'flexibility' is incorrectly used for this concept, see box 10.2.
Box 10.2. Terminology: 'sensitivity' or 'flexibility' of structures?

The here defined "sensitivity" of a structure is also called "flexibility" in literature (Bos 1989). It is proposed (Ankum 1992b, 1993b) to defer from the term "flexibility" in this concept.

The term "flexibility" means: adjustable to changes (Webster dictionary), and is widely used in irrigation literature in relation to "demand-based" water allocation to the tertiary unit (Merriam 1987). A similar meaning of the term "flexibility" is used on the ICID Congress of 1993 in Question nr. 44.2: "Planning and Design for Flexibility", which was elaborated as: the system should have sufficient flexibility to cope with changes in objectives.

Furthermore, the term "sensitivity" is used in literature for: the accuracy of discharge measurement structures in relation to the variations in water level (Bos 1989).

It is recommended here to maintain the following terminology in flow control:

- the term "sensitivity of a structure" is related to the variations in the offtaking discharge, in relation to the discharge in the continuing discharge in the canal;
- the term "flexibility" should be related to the freedom of users on the frequency, rate and duration of the water allocation of the flow control method.

The term "accuracy of a discharge measurement structure" can be used for the effect of water level changes on the reading of discharge measurement structures.

Definition of sensitivity. The sensitivity 'S' is defined as the ratio in the variation of an offtaking discharge $Q_t$, and the variation in the discharge $Q_c$ in the continuing channel. Thus, the sensitivity 'S' depends on the rating curves of the regulators, see figure 10.6:

- the depth-discharge relation of the offtake is a function of the energy head: $Q_t = \beta H_t^T$, or is a function of the water level $y_t$: $Q_t = \beta y_t^T$.
- the depth-discharge relation of the control is a function of the energy head: $Q_c = \alpha H_c^C$, or is a function of the water level: $Q_c = \alpha y_c^C$.

**Figure 10.6.** Rating curves at the bifurcation.
Hence, the sensitivity $S$:

$$S = \frac{dQ_t}{Q_t} \frac{dH}{dH} = \frac{\beta T H_t^{T-1}}{\alpha C H_c^{C-1}}$$

Thus, the sensitivity $S$ of a structure can be calculated:

$$S = \frac{T \times y_c}{C \times y_t}, \quad \text{or: } S = \frac{T \times y_t}{C \times y_t}$$

**Sensitivity of 'fixed' regulation.** The sensitivity as discussed above, is related to **fixed regulation.** The following rating curves are applied, see figure 10.7:

- **fixed water level regulators:**
  - underflow, $H = Q^2$,
  - overflow, $H = Q^{2/3}$,
  - long-crested weirs, $H = \text{constant}$,

- **fixed discharge regulators:**
  - underflow, $Q = H^{0.5}$,
  - overflow, $Q = H^{1.5}$,
  - 'special' orifices, $Q = \text{constant}$.

![Figure 10.7. Rating curves of regulators.](image)

**Sensitivity of 'manual' and 'automatic' regulation.** The concept of sensitivity can also be utilized for **manual and automatic regulation.** Now, the target value is maintained under all circumstances. It means that the rating curves becomes most simple, see also figure 10.7:

- **manual or automatic water level regulators:**
  - all flow types, $H = \text{constant}$,

- **manual or automatic discharge regulators:**
  - all flow types, $Q = \text{constant}$. 
Manual regulation at fixed times only. It is a common practice in many irrigation schemes to adjust the regulators only e.g. once per 14 days, to match the water requests with the water availability. It means that the regulators perform as 'fixed' regulators in the period between the adjustments. Thus, the sensitivity depends also on the frequency of regulation!

Constant discharge diversion. An offtake of a diversion structure has no sensitivity to any variations in incoming discharges, when $S = 0$, see figure 10.8. This occurs when the offtaking channel maintains at a perfect constant discharge $Q_t$. Such a constant discharge $Q_t$ occurs:
- an offtake which is designed for a perfect constant discharge, independent of the upstream water level, e.g. by a special underflow such as the baffle distributor in irrigation,
- a control that maintains the upstream water level at a kept constant level, e.g. by AMIL gate, long-crested weir, but also by a vertical gate that is operated continuously.

![Figure 10.8. Flow diversion for Sensitivity $S = 0$.](image)

Low sensitivity of a diversion structure. An offtake has a low sensitivity to variations in discharge when $S < 1$. This occurs when the discharge variation in the offtaking discharge are less than that in the continuing channel. A low sensitivity ($S < 1$) occurs at:
- a 'free-offtake' with underflow (gate), that is not operated,
- a control with overflow (weir) and an offtake with underflow (gate), that are both not operated, see box 10.3.

**Box 10.3. Example of diversion structure with a low sensitivity ($S < 1$).**

A flood diversion structure is constructed with overflow (weir) to the ongoing river channel and underflow (orifice) to the offtaking flood channel.

The depth-discharge relation to the ongoing river channel reads $Q_c = \alpha y_c^{1.5}$ with a design water depth $H_c = 0.75$ m. The depth-discharge relation to the offtaking flood channel reads $Q_t = \beta y_t^{0.5}$ and design water depth $y_t = 1.00$ m. The sensitivity $S$ can be calculated at $S = (T H_c) / (C H_t) = (0.5 \times 0.75) / (1.5 \times 1.00) = 0.3$.

It means that the offtake will take a very low variation $dQ$ of the changing river discharge $Q$. 

Proportional diversion of a structure. The diversion structure is exact proportional for \( S = 1 \) and any percentage change in the incoming discharge causes precisely the same percentage change in the offtaking discharge, see figure 10.9. A sensitivity \( S = 1 \) occurs:
- when the same type of flow is used in the offtake and in the control, i.e. both overflow, or both underflow. Moreover, both energy heads \( (H_c \text{ and } H_t) \) should have the same value, because their products should have the same value: \( T \times H_c = C \times H_t \).

As above, the regulators are not operated here.

High sensitivity of a diversion structure. The variation in the offtaking discharge will be more than that in the ongoing channel for a high sensitivity, when \( S > 1 \). A high sensitivity \( S > 1 \) occurs at:
- a bifurcation without structures,
- a 'free-offtake' with overflow (weir), see box 10.4,
- a control with underflow (gate) and an offtake with overflow (weir).

![Figure 10.9. Flow diversion for Sensitivity \( S = 1 \).](image)

**Box 10.4. Example of diversion structure with a high sensitivity \( (S > 1) \).**

A flood diversion structure is constructed with underflow (orifice) to the ongoing river channel and overflow (weir) to the offtaking flood channel.

The depth-discharge relation to the ongoing river channel reads \( Q_c = \alpha y_c^{0.5} \) with a design water depth \( H_c = 1.20 \text{ m} \). The depth-discharge relation to the offtaking flood channel reads \( Q_t = \beta y_t^{1.5} \) and design water depth \( y_t = 0.75 \text{ m} \). The sensitivity \( S \) can be calculated at \( S = (T/H_c) / (C/H_t) = (1.5 \times 1.20) / (0.5 \times 0.75) = 4.8 \).

It means that the offtake will take a very high variation \( dQ \) of the changing discharge \( Q \).

Perfect sensitivity of a diversion structure. A diversion structure has a perfect sensitivity for \( S = \infty \), when all variations in incoming discharge is diverted into the offtake, and a constant flow is diverted into the continuing channel, see figure 10.10. A perfect sensitivity \( S = \infty \) occurs at:
- an offtake that maintains the upstream water level at a kept constant level, e.g. by AMIL gate, long-crested weir, but also by a vertical gate that is operated continuously,
- a control which is designed for a perfect constant discharge, independent of the upstream water level, e.g. by a special underflow such as the 'baffle distributor' in irrigation.
Figure 10.10. Flow diversion for Sensitivity $S = \infty$.

Relevance of 'sensitivity'. The relevance of the concept 'sensitivity' in flood control is limited. Normally, flood control involves diversion at one structure only.

The concept of sensitivity is most important in irrigation schemes with fluctuating river discharges. Irrigation systems consist of a large number of structures, where the ever-changing flow has to be diverted. These systems may have serious problems with the equal distribution of discharge fluctuations, see figure 10.11.

A solution for these equity problems can be found in 'gate-proportional' facilities within the upstream control method. This can be obtained by two supplemental measures (Ankum 1995):

- the construction of the same type of flow for water level regulators and discharge regulators, i.e. both overflow, or both underflow. The selected combination determines the "sensitivity" of the diversion structure;
- the operational order not to regulate these structures during an 'irrigation cycle', i.e. the period between the system adjustments to new discharges, e.g. 14-days.

Figure 10.11. The concept of 'Sensitivity' in irrigation.
10.2. Design Considerations and Design Criteria

10.2.1. Design Conditions

Backwater effects. A diversion structure has an influence on the water levels in the upstream channel. The 'depth-discharge relation' ("Q-h kromme") of the structure (orifice or weir formula) is different from the depth-discharge relation of the channel (Strickler formula). It means that the structure creates backwater effects on the upstream channel: there is either a 'setting-up' ("opstuwning") or a 'drawing-down' ("afzuiging") of the water level, and the uniform flow is only obtained in specific cases.

Thus, an additional design condition ("ontwerp eis") of the diversion structure can be included in the design: "there is uniform flow during the 'bed-forming' or 'dominant' discharge ("bedvormende, dominante afvoer") for which the channel bed is supposed to be in equilibrium", see figure 10.12 It means that:

- the dominant discharge \( Q_{\text{dominant}} \) will create no backwater curves on the upstream channel;
- the lower discharges \( Q < Q_{\text{dominant}} \) will lead to a 'positive' backwater curve ("opstuwning"), with sedimentation before the structure because of a lower energy dissipation \( E = \rho g v s \);
- the higher discharges \( Q > Q_{\text{dominant}} \) will lead to a 'negative' backwater curve or 'draw-down' ("afzuigen"), with scouring before the structure because of a higher tractive force \( T = \rho g y s \).

![Figure 10.12. Two design conditions for a diversion structure.](image)

Super-critical flow. The diversion at a structure is also determined by occurrence of:

- sub-critical flow ("onvolkomen, niet-schietende stroming"). For instance, a sub-critical flow towards the ongoing channel makes that any changes in tailwater levels ("achter waterstanden") lead to changes in the diversion;
- super-critical flow ("volkomen, schietende stroming"). A super-critical flow makes that any changes in the tailwater levels do not have any effect on the diversion of flow, see also figure 10.12.
Thus, an additional **design condition** ("ontwerp eis") can be included in the design: "there is always 'free flow' ("volkomen stroming") towards the diversion channel and/or towards the ongoing river".

### 10.2.2. Hydraulic Formulae

Weir for water level control. A weir is used for 'passive' regulation of the water levels, see figure 10.13. The structure is normally applied for free flow only. The discharge formula of a weir under free flow reads, see also section 7.1 of chapter 7:

\[
Q = c_b H^{1.5}, \quad \text{with: } c = c_d \frac{2}{3} \sqrt{\frac{2}{3} g} = c_d \times 1.71 \text{ m}^{1/2}/s
\]

where \(Q\) is the discharge in m³/s, \(c\) is the weir coefficient in m¹/²/s, \(c_d\) is the discharge coefficient, \(b\) is the width of the crest in m, \(H\) is the energy head above the crest in m.

The width \(b\) of weirs can be reduced by applying a sharp-crested type with a **higher weir coefficient** \(c\) in the discharge formula.

For example, a broad-crest weir may have a weir coefficient \(c = 1.7\text{ m}^{1/2}/s\). And a sharp-crested weir may have a weir coefficient \(c = 1.9\text{ m}^{1/2}/s\) for unrounded crests, \(c = 2.1\text{ m}^{1/2}/s\) for cylindrical crests, or even \(c = 2.3\text{ m}^{1/2}/s\) for the specially designed 'ogee-crests' (USBR 1973). It means that the width of an ogee-crest is only 75% of the width of the broad-crested weir.

The value of the weir coefficient \(c\) influences also the required headloss \(z\) for free flow. For example, a broad-crested weir requires a small head of some \(z \approx \frac{1}{2} H\). Cylindrical crests and ogee-crests require a minimum headloss \(z \approx \frac{1}{2} H\). Unrounded crests require a larger headloss \(z\) for aeration of the nappe ("beluchting van de straal"), thus \(z \gg H\).

Thus, an additional **design condition** ("ontwerp eis") can be included in the design: "there should be a minimum width \(b\) of the weir to reduce the costs, by selecting a weir-shape with a high weir coefficient \(c\)."

---

**Figure 10.13.** Design conditions for a weir.
Orifice for water level control. An orifice opening, such as a 'flood screen' (*bandjirscherm*), may be used for 'passive' regulation of the water level. There is submerged flow for the lower discharges, as the orifice should not create much headloss. During the higher discharges, however, the orifice obstructs the flow and the upstream water level will rise considerably, see figure 10.14.

The headloss formula of an orifice under submerged flow, 'conveyance flow', reads, see also section 6.2 of chapter 6 and section 8.3 of chapter 8:

\[
    z = \alpha \frac{v^2}{2g} \quad \text{with:} \quad v = \frac{Q}{y b}
\]

with the headloss \( z \) in m, a coefficient \( \alpha \), the velocity \( v \) within the orifice, the discharge \( Q \) in \( \text{m}^3/\text{s} \), the (average) water depth \( y \) within the structure, and the width \( b \) of the structure, and the gravity acceleration \( g = 9.8 \, \text{m/s}^2 = 9.8 \, \text{N/kg} \).

The above formula can be re-written into the discharge formulae for the control during submerged flow for an open-water level in the orifice, so \( y_1 < w \):

\[
    Q = \sqrt{\frac{1}{\alpha} \frac{b y_1}{\sqrt{2g}} z}
\]

for orifice flow, so \( y_1 > w \):

\[
    Q = \sqrt{\frac{1}{\alpha} \frac{b w}{\sqrt{2g}} z}
\]

where \( Q \) is the discharge in \( \text{m}^3/\text{s} \), \( \alpha \) is a coefficient, \( b \) is the width of the orifice in m, \( y_1 \) is the upstream water level above the sill in m, \( w \) is the height of the orifice-opening above the sill in m, and \( z \) is the headloss in m.

The coefficient \( \alpha \) depends on the entrance and the exit losses, and can be taken at \( \alpha = 0.5 \) to 1.0, for instance. In principle, the coefficient \( \alpha \) should be kept as low as possible to avoid high headlosses. The lower coefficients \( \alpha \) are obtained by allowing an open-water level in the orifice, as long as regulation is not required.

Thus, an additional design condition (*ontwerp eis*) can be included in the design: "the height \( w \) of the orifice opening should allow an open-water level in the orifice, as long as water level regulation is not required, so \( y_1 < w \)."

![Figure 10.14](image.png)

**Figure 10.14.** Design conditions for an orifice as 'flood screen' for water level regulation.
Orifice for discharge control. An orifice, such as a 'gated offtake', is used for regulation of the discharge. The structure is normally applied for free flow only, see figure 10.15.

The discharge formula of an orifice under free flow reads, see also section 8.4 of chapter 8:

$$Q = C_D \cdot b \cdot w \cdot \sqrt{2gy_1}$$

where $Q$ is the discharge in $m^3/s$, $b$ is the width of the opening in $m$, $C_D \approx 0.50 - 0.55$ is the discharge coefficient, $w$ is the height of the opening in $m$, $y_1$ is the upstream water depth above the sill in $m$, and $g = 9.8 \text{ m/s}^2 = 9.8 \text{ N/kg}$ is the gravity acceleration. The discharge formula can be approximated by:

$$Q = 2.3 \cdot b \cdot w \cdot H^{1/2}$$

The minimum headloss $z$ for free orifice flow is quite complex, see section 8.4 of chapter 8. Normally, a minimum headloss $z \approx \frac{1}{2} H$ is required for free flow. The above formula is valid for $H \geq \frac{3}{2} w$.

However, the design of an orifice for discharge control should not apply the above formula for an orifice-type of flow. The dimensions of the orifice are determined by the design (maximum) discharge. An orifice-type of flow means a vertical contraction by the 'front wall' ("borstwering"), and requires a wider width $b$ of the opening and a more expensive structure.

\[\text{Figure 10.15. Design conditions for an orifice as discharge regulator.}\]
It is better to apply the formula of a *control notch* ("vernauwing") for the design of an orifice for discharge control, and to allow an open-water flow through the orifice during the maximum discharge, see also figure 10.15.

The discharge formula of a 'control notch' under free flow and with vertical wall is closely related to the formula for a broad-crested weir and reads, see also section 7.1 of chapter 7:

\[ Q = 1.8 \, b \, H^{3/2} \]

where \( Q \) is the discharge in \( m^3/s \), \( b \) is the width of the opening in m, \( H \) is the energy depth above the sill in m. Normally, a minimum headloss \( z = \frac{1}{2} \, H \) is required for free flow. The above formula is valid for \( H < \frac{3}{2} \, w \).

Thus, an additional design condition ("ontwerp eis") can be included in the design: "the height \( w \) of the orifice opening should allow an open-water level in the orifice during the maximum discharge, so \( H < \frac{3}{2} \, w \)."

**Sill level of the orifice.** The energy depth \( H \) and the opening \( w \) are both related to the height \( p \) of the sill ("drempel") above the riverbed, see figure 10.16. A high sill level for \( p >> \) means that the energy head \( H \) and the opening height \( w \) are smaller, so the width \( b \) of the orifice should be increased at higher costs. The minimum costs of the structure are often obtained for a sill at bed level, so \( p = 0 \).

Thus, an additional design condition ("ontwerp eis") can be included in the design: "the width \( b \) of the orifice is at a minimum by selecting the sill of the orifice at bed level, so \( p = 0 \)."

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![Figure 10.16. Design conditions for the sill level of an orifice.](image-url)
10.2.3. Side-spillway effect

Side-spillway. The hydraulic design of the side-channel spillway has been discussed in section 7.3. of chapter 7. Two features are important, and are called here the 'side-spillway effect', see figure 10.17:

- there is a constant energy head in the longitudinal channel direction, as there are no energy losses other than friction losses. The energy head is determined by the energy head \( H_{\text{ongoing}} \) of the downstream (ongoing) channel reach.
  
  It means that in upstream direction: (i) the water level drops, and (ii) that the velocity head \( v^2/2g \) increases ("toeneemt");
- there is a loss in energy by sudden flow deflection ("plotselinge verandering van stroomrichting"). It is assumed that these 'bend losses' equal 1.0 \( v^2/2g \).
  
  It means that in upstream direction: (i) the total velocity head \( v^2/2g \) is lost, and (ii) the energy depth \( H_t \) over the spillway crest decreases ("afneemt").

The 'side-spillway effect' makes that the hydraulic calculation has to be done step-by-step in upstream direction, and that the spillway becomes quite long.

![LONGITUDINAL PROFILE](image)

![CROSS-SECTION](image)

Figure 10.17. The 'side-spillway effect' at diversion structures.

'No side-spillway effect'. The above effect of side-spillway can be ignored, and there is 'no side-spillway effect' when, see figure 10.18:

- there is a gentle flow deflection, so that there are no bend losses \( \alpha v^2/2g \) because the value of \( \alpha \approx 0 \),
- the energy depth \( H_t \) over the spillway crest is much larger than the bend loss \( \alpha v^2/2g \), so the bend loss \( \alpha v^2/2g \) can be ignored, as \( H_t >> \alpha v^2/2g \),
- the variation \( \Delta H_t \) of the energy depth \( H_t \) over the spillway crest is small and can be ignored, so \( \Delta H_t << H_t \).

The result of 'no side-spillway effect' makes that the hydraulic calculation is most simple, and that the energy depth \( H_t \) at the offtake equals the energy head \( H_{\text{ongoing}} \) of the downstream (ongoing) channel reach.

Thus, the energy depth \( H_t \) above the offtake-crest is: \( H_t = H_{\text{offtake}} - p \), where \( p \) is the elevation of the offtake-crest above the bed.
Figure 10.18. No 'side-spillway effect' at diversion structures.

10.2.4. Minimum Width of the Structure

Check. A check should be made on sub-critical flow ("stromend water") within the entrance of the diversion structure, see figure 10.19. If there is no sub-critical flow but super-critical flow ("schiënd water"), there will be excessive friction losses within the structure. This leads to a high tractive force $T$, because of $T = \rho g y s$. Furthermore, the headloss by the structure will increase, which has effects on the backwater effects in the incoming channel. Moreover, the hydraulic calculation becomes quite complex.

The above check on the sub-critical flow can be made by determining the water line in the diversion structure, and to calculate the corresponding Froude number $Fr$ from $Fr = v/(gy)^{0.5}$. It is also possible to use broad-crested weir formula, as will be shown below.

Figure 10.19. The minimum width $b_{\text{min}}$ within a diversion structure.
Water line within the structure. The course ("verloop") of the water line within a structure is based on the following considerations, see figure 10.20:

- there is a constant energy head in the longitudinal flow direction, and the energy head is determined by either:
  - for a free-offtake: the energy head equals the energy head $H_{\text{ongoing}}$ of the downstream (ongoing) channel reach,
  - for a controlled-offtake: the energy head equals the energy head $H_t + p$ at the control.
- the water level is determined by the velocity head $v^2/2g$. So, the water level will change for a changing discharge.

An example of the water line calculation within a diversion structure is presented in box 10.5.

![Diagram showing water line calculation within a diversion structure.](image)

**Figure 10.20.** The water level within a diversion structure.

Direct calculation. The minimum width $b_{\text{min}}$ within a diversion structure to avoid supercritical flow can also be calculation directly by using the broad-crested weir formula. Flow over the broad-crested weir has the 'critical depth' with a Froude number $Fr = 1$. Thus, the following relation is valid:

$$Q = 1.7 \, b_{\text{min}} \, H^{3/2}$$

with the discharge $Q$ through the structure in m$^3$/s, the minimum width $b_{\text{min}}$ within the structure, and the energy head $H$ in m. The energy head $H$ follows from the downstream conditions at the control, e.g. $H = p + H_C$. An example of the minimum width within a diversion structure is presented also in box 10.5.
Box 10.5. Example of the water line calculation and the minimum width within a structure.

The setting. A flood diversion structure has to divide a 20-years flood \( Q_{20} = 400 \text{ m}^3/\text{s} \) into an offtaking discharge \( Q_t = 250 \text{ m}^3/\text{s} \) and a continuing discharge \( Q_c = 150 \text{ m}^3/\text{s} \).

The structure has vertical walls at a mutual distance \( b = 30 \text{ m} \), and with a weir as the control. The weir-crest is \( p = 1.50 \text{ m} \) above the bed, and is \( b = 15.75 \text{ m} \) wide. The rating curve of the sharp-crested weir is \( Q = 2.1 b \, H^{3/2} \). The river bed within the structure is at \( 10.00 \text{ m}^+ \).

Design. The energy depth during the design flood of \( Q_c = 150 \text{ m}^3/\text{s} \) is at \( H = p + H_c \).

The water depth \( y_C \) in point C is at \( \% \, H_c \) above the crest. So, the water level at point C is at \( 10.00 + 1.50 + \% \times 2.74 = 13.33 \text{ m}^+ \).

The water depth \( y_B \) in point B is determined by the energy depth \( H = 4.24 \text{ m} \) above the bed, the discharge \( Q_c = 150 \text{ m}^3/\text{s} \) and by the width \( b \) between the vertical walls. The iterative calculation is as follows:

- Assume \( \nu = 0.00 \text{ m/s} \), so \( y_B = 4.24 - \frac{\nu^2}{2g} = 4.24 \text{ m} \), and \( \nu = \frac{150}{[30 \, y_B]} = 1.18 \text{ m/s} \).
- Assume \( \nu = 1.18 \text{ m/s} \), so \( y_B = 4.24 - \frac{\nu^2}{2g} = 4.17 \text{ m} \), and \( \nu = \frac{150}{[30 \, y_B]} = 1.20 \text{ m/s} \).
- Assume \( \nu = 1.20 \text{ m/s} \), so \( y_B = 4.24 - \frac{\nu^2}{2g} = 4.17 \text{ m} \), and \( \nu = \frac{150}{[30 \, y_B]} = 1.20 \text{ m/s} \).

So, the water level at point B is at \( 10.00 + 4.17 = 14.17 \text{ m}^+ \).

The water depth at point A is determined in a similar way for a discharge \( Q_t + Q_c = 400 \text{ m}^3/\text{s} \). It can be anticipated that the water level at point A will be lower than at point B, as the term \( \frac{\nu^2}{2g} \) is now larger. Thus, a water depth \( y_A = 3.51 \text{ m} \) and a velocity \( \nu = 3.79 \text{ m/s} \) is calculated. So, the water level at point A is at \( 10.00 + 3.51 = 13.51 \text{ m}^+ \).

Check. A check should be made on sub-critical flow, as super-critical flow with excessive friction losses has to avoided. The Froude number in point C follows from \( Fr = \frac{\nu}{(g y)^{0.5}} = \frac{3.79(9.81 \times 3.51)^{0.5}}{0.5} = 0.65 \). So, there is sub-critical flow at point A.

The minimum structure width \( b_{\text{min}} \) follows from the broad-crested weir formula:

\[ Q = 1.7 \, b_{\text{min}} \, H^{3/2} \].

Hence, \( 400 = 1.7 \, b_{\text{min}} \times 4.24^{3/2} \). So, the minimum width \( b_{\text{min}} = 26.95 \text{ m} \), which is less than the distance \( b = 30 \text{ m} \). It means: there is sub-critical flow at the entrance!
10.3. Options for Diversion Structures

10.3.1. Regulators

'Free' and 'controlled' offtakes. Diversion structures may have one or two 'regulators' ("regelaars"), i.e. an 'offtake' ("aftapping") and/or a 'control' ("stuw"), see figure 10.21:
- a free-offtake ("vrije aftapping") has only a 'regulator' in the offtaking channel, and no 'control' in the continuing channel;
- a controlled-offtake has both a 'regulator' in the offtaking channel, as well as a 'control' in the continuing channel.

![Diagram showing 'free' and 'controlled' offtakes](image)

Figure 10.21. A diversion structure as a 'free' offtake and as a 'controlled' offtake.

Open offtake. It is also possible that design an 'open offtake' without a diversion structure, see figure 10.22. However, a reliable and accurate diversion is impossible without regulators, as the diversion will depend on the tailwater levels ("achterwaterstanden") in the continuing channel and in the offtaking channel. It means that the diversion changes on e.g. changing cross-sections and channel roughness. This matter is discussed at the beginning of the chapter.

![Diagram showing open offtake](image)

Figure 10.22. No diversion structure is unsuitable for flow diversion.
What is regulated? The regulators can either be (see chapter 5):
- a 'water level' regulator ("peilregelaar"), or
- a 'discharge' regulator ("debietregelaar").
Furthermore, it is also possible to regulate on the 'ratio-in-discharge' ("proporzioneel").

How is regulated? The regulation of the regulators can either be (see chapter 5):
- Passive regulation ("vaste regeling"),
- Manual regulation ("hand-bediende regeling"),
- Hydro-mechanical regulation ("hydro-mechanisch"),
- Electro-mechanical regulation ("electro-mechanisch"), which is not discussed here.
Basically, there are only two manner of regulation, i.e. (i) 'passive', and (ii) 'gated' which covers manual, hydro-mechanical and electro-mechanical regulation.

What flow type? The flow type at the regulators can either be (see chapter 5):
- Overflow ("overlaat"),
- Underflow ("onderspuier").

10.3.2. Configuration of a Diversion Structure

Configuration. The operational objectives of a diversion structure should match the type of 'offtake' and the 'control', i.e. the 'configuration' ("configuratie"). The options are:
- the type of regulator (none, water level, discharge, ratio-in-discharge),
- the manner of regulation (passive, manual, hydro-mechanical),
- the type of flow (overflow, underflow).
Not all configurations appear to be very practical.

Practical configurations for free-offtakes. There are only two practical configurations for free-offtakes, see figure 10.23:
- overflow free-offtake ("overlaat vrije-aftapping"),
- gated free-offtake ("regelbare vrije-aftapping").
The design of these configurations will be discussed in the next chapter.

![Diagram of WEIR and GATES with free flow](image_url)

**Figure 10.23.** Types of free offtakes.
Practical configurations for 'passive' diversion structures. Three configurations are practical for 'passive' diversion structures, see also figure 10.24:
- **overflow offtake with orifice-control** ("overlaat-aftapping met onderspu-stuw"),
- **proportional divisor** ("propportionele verdeler"),
- **proportional divisor with orifice-control** ("propportionele verdeler met onderspu-stuw"),
The design of these configurations will be discussed in the next chapter.

![Diagram of Overflow Offtake with Orifice-Control](image)

**Figure 10.24.** Types of passive diversion structures.

Practical configurations for 'gated' diversion structures. Also three configurations are practical for 'gated' diversion structures, see also figure 10.25:
- **gated offtake with weir-control** ("regelbare-aftapping met overlaat-stuw"),
- **overflow offtake with gated-control** ("overlaat-aftapping met regelbare-stuw"),
- **gated offtake with gated-control** ("regelbare-aftapping met regelbare-stuw").
The design of the first two configurations will be discussed in the next chapter. The third configuration is discussed below.

**Gated offtake with Gated-control.** A 'gated offtake with gated-control' ("regelbare-aftapping met regelbare-stuw") implies the application of two regulators with gates, both at the offtake and at the control, see also figure 10.25.

In irrigation, the configuration of two regulators with gates is widely applied. The regulator at the offtake is equipped with a measuring device ("meetinrichting"). This can be
achieved by hydraulic adjustments of the gate (e.g. Romijn weir, Crump-deGruyter gate, etc.), or by constructing a separate measurement structure in series (e.g. broad-crested weir, Cipoletti weir). The result is that the regulator at the 'offtake' becomes a discharge regulator, and the regulator at the 'control' is a water level regulator. This can be translated into clear instructions for the operator.

A 'gated offtake with gated-control' is normally not a good configuration in flood control, as discharge regulation of floods is troublesome. It is difficult to measure the discharge with a vertical or radial gate, and the application of separate measurement structures in flood control is prohibited by its high costs.

So, the gated offtake of the flood diversion may become a (upstream) water level regulator. Such a configuration of two water level regulators will confuse the operator, as he does not know which of the regulators need adjustment to obtain the target water level. It means, there is an instability of diversion: many gate settings, and so flood diversions, are possible to maintain the target water level.

However, there might be solutions to overcome the above problems with the configuration of 'gated offtake with gated-control' in flood diversion:

- the assessment of a rating curve for the discharge regulator, which is accurate and simple for the operator;
- the application of Crump-deGruyter gates at the discharge regulator;
- to operate the discharge regulator as a 'downstream' gate, i.e. the target water level is at the downstream side of the gate. The effect is that the downstream water level is related to the diverted discharge by the Strickler formula. It is obvious that such relationship is quite inaccurate for changing river dimensions $AR^{2/3}$ and for a changing roughness coefficient $k$.

![Diagram of gated diversion structures](image)

Figure 10.25. Types of gated diversion structures.
11. DESIGN EXAMPLES OF DIVERSION STRUCTURES

11.1. Design of Free-Offtakes

11.1.1. Overflow Free-Offtake

Configuration. An 'overflow free-offtake' ("overlaat vrije-aftapping") consists of one weir located at the offtake and no structure at the control, see figure 11.1. The 'side-channel spillway' belongs to this type and has been discussed in sub-section 7.3.2 of chapter 7.

Figure 11.1. An 'overflow free-offtake'.
Performance. An 'overflow free-offtake' is applied when the water depth in the ongoing channel should not exceed a certain value $y_{\text{max}}$, see also figure 11.1. The weir is a 'passive' water level regulator, and diverts a discharge when the water level exceeds the crest level.

It should be noted that this configuration is based on 'water level control', and that the distribution of flow depends on the water level $y_{\text{target}}$ in the ongoing channel. However, the discharge at this target water level $y_{\text{target}}$ is determined by the depth-discharge relation according to the Strickler formula. Thus, the actual discharge changes for changing values of the wet cross-sectional area $A$ and the hydraulic radius $R$, and for changing values of the Strickler coefficients $k$ during the growing season.

It has been shown in sub-section 7.3.2 of chapter 7, that the side-channel spillway is normally not a good structure for diverting flow, because:
- the length of the offtaking weir may become unrealistic long;
- the target water level $y_{\text{ongoing}}$ cannot be maintained, as an additional head is required for flow over the offtaking weir;
- the diversion of the discharge of the side-channel spillway depends on hydraulic changes in the ongoing channel, such as changing roughness and changing cross-sections;
- the side-channel spillway creates a negative backwater curve (drawdown) in the upstream channel.

Design variables. The design requires that two water levels in the ongoing channel should be known: the target water level $y_{\text{target}}$ at which diversion starts, and the maximum water level $y_{\text{max}}$ during the diversion of the maximum (design) discharge. Furthermore, the cross-sectional areas of the channel are required, so that the velocity $v$ can be calculated for a known discharge $Q$. It is assumed that the energy drop $z_\ell$ at the offtake is always sufficient for free flow.

Two design variables have to be designed, see figure 11.2:
(i) the width $b_1$ of the weir at the offtake to the offtaking channel,
(ii) the crest-height $p_1$ of the weir at the offtake.

Moreover, the following hydraulic parameters are also unknown:
(iii) the energy head $H_{\text{max}}$ during the maximum discharge in the ongoing channel,
(iv) the water depth $y_x$ in the channel, at an upstream distance $x$ from point $Z$,
(v) the water velocity $v_x$ in the channel, at an upstream distance $x$ from point $Z$,
(vi) the energy head $H_x$ above the weir-crest at the offtake, at an upstream distance $x$ from point $Z$,
(vii) the offtaking discharge $Q_x$ over a width $\Delta b$ of the weir, at an upstream distance $x$ from point $Z$.

Design conditions. There are seven design conditions required to determine the two design variables and the five hydraulic parameters, see also sub-section 7.3.2 of chapter 7:
- condition 1. Diversion into the offtaking channel starts only when the water level becomes higher than the target water level $y_{\text{target}}$. So, the crest-height $p_1$ of the weir at the offtake follows from:
  \[ p_1 = y_{\text{target}} \]
- condition 2. The energy depth in the channel during the maximum discharge follows from:
  \[ H_{\text{max}} = y_{\text{max}} + \frac{(v_{\text{max}})^2}{2g} \]
  and remains constant in upstream direction because the friction losses can be ignored.
- condition 3. The water depth $y_x$ in the channel at a location $x$ during the maximum (design) discharge follows from:
  \[ y_x = H_{\text{max}} - \frac{(v_x)^2}{2g} \]
Figure 11.2. Design variables of the 'overflow free-offtake'.

• **condition 4.** The energy head $H_x$ above the weir-crest at a location $x$ depends on the 'bend loss' due to a sudden flow deflection ("plotselinge verandering van stroomrichting"). It is assumed that this 'bend loss' equals $1.0 \cdot \nu_x^2 / 2g$, so that at location $x$:

$$H_x = y_x$$

• **condition 5.** The discharge formula for the offtaking weir of a width $\Delta b$ at a location $x$ is:

$$\Delta Q_x = c \cdot \Delta b \cdot H_x^{3/2}$$

for $z \geq \gamma H_x$

• **condition 6 and condition 7.** The width $b_t$ of the offtaking weir depends on the cumulative offtaking discharge $\Sigma \Delta Q_x$, and by checking it with the required offtaking discharge $Q_t$.

So:

$$b_t = \Sigma \Delta b$$

when $\Sigma \Delta Q_x = Q_t$.

**Design process.** An example of the design process of an 'overflow free-offtake', i.e. the side-channel spillway, is presented in sub-section 7.3.2 of chapter 7.

**Operation.** Operation of the overflow free-offtake is not needed, and is even not possible. Any approaching flow above a certain value, is diverted over the spillway. However, it is unavoidable that the continuing flow to the ongoing channel is increased as well.

**11.1.2. Gated free-offtake**

**Configuration.** A 'gated free-offtake' ("regelbare vrije-aftapping") consists of a gated offtake and no structure at the control, see figure 11.3.

**Performance.** A 'gated free-offtake' is applied when the water depth in the ongoing channel should not exceed a certain value $y_{\text{target}}$, see also figure 11.3. The offtake is a manual or a hydro-mechanical water level regulator, and diverts a discharge when the water level exceeds the target level $y_{\text{target}}$, see figure 11.4.
Figure 11.3. A 'gated free-offtake'.

Figure 11.4. An artist impression of a 'gated water level regulator' (Nedeco 1973).
Like the 'overflow free-offtake', the diversion of discharges depends on the water level \( y_{\text{target}} \) in the ongoing channel. However, the discharge at this target water level \( y_{\text{target}} \) is determined by the depth-discharge relation according to the Strickler formula. Thus, the discharge changes for changing values of the wet cross-sectional area \( A \) and the hydraulic radius \( R_t \) and for changing values of the Strickler coefficients \( k \) during the growing season.

The 'gated free-offtake' is an alternative for the side-channel spillway. The 'side-spillway effect' can be ignored because of the energy head \( H \) is quite large. It means that the width \( b_t \) of the gated free-offtake is much smaller than the length of the side-channel spillway.

The disadvantages of the 'gated free-offtake' are:
- the diversion of discharge by the free-offtake depends on hydraulic changes in the ongoing channel, such as changing roughness and changing cross-sections;
- the gated free-offtake creates a negative backwater curve (drawdown) in the incoming channel.

**Design variables.** The design requires that the maximum (design) water level \( y_{\text{target}} \) in the ongoing channel should be known. Furthermore, the cross-sectional areas of the channel are required, so that the velocity \( v \) can be calculated for a known discharge \( Q \). It is assumed that the energy drop \( z_t \) at the offtake is always sufficient for free flow. There are no 'side-spillway effects'. The sill level of the offtake is at the channel bed level, so \( p_t = 0 \).

Two design variables have to be designed, see figure 11.5:
1. the width \( b_t \) of the gates at the offtake to the offtaking channel,
2. the maximum orifice opening \( w_t \) at the offtake.

Moreover, the following hydraulic parameters are also unknown:
3. the energy head \( H_{\text{target}} \) in the ongoing channel for the target water level,
4. the energy head \( H_t \) at the offtake.

![Diagram of Design Variables](image)

Figure 11.5. Design variables of the 'gated free-offtake'.

**Design conditions.** There are four design conditions required to determine the two design variables and the two hydraulic parameters:
- **condition 1.** The energy depth in the channel for the target water level follows from:
  \[
  H_{\text{target}} = y_{\text{target}} + \left(\frac{v_{\text{target}}}{2g}\right)^2/2g
  \]
  and remains constant in upstream direction because the friction losses can ignored.
- **condition 2.** The energy head $H_t$ above the sill of the offtake equals the energy depth in the channel as there are no 'side-spillway effects':
  \[ H_t = H_{\text{target}} \]
- **condition 3.** The depth-discharge relation of the offtake during the maximum offtaking discharge follows the discharge formula of the 'notch':
  \[ Q_t = 1.8 b_t H_t^{3/2} \text{ for } z_t \geq \frac{1}{5} H_t \]
- **condition 4.** The (minimum) orifice opening $w_t$ at the offtake depends on the critical water depth at the offtake:
  \[ w_t = \frac{2}{3} H_t \]

Figure 11.6. An example of a 'gated free-offtake' on a meandering river.
Design process. An example of the design process of a 'gated free-offtake' is presented in box 11.1. An example of a 'gated free-offtake' is shown in figure 11.6, where it is applied for flood diversion from a large meandering river within a flood plain.

Operation. Operation of the gated free-offtake is required, and is based on 'water level control'. The gates are opened or closed in such a way that the target water level in the ongoing channel is maintained. The actual discharge in the ongoing channel is not known, but can be estimated with the Strickler formula.

**Box 11.1. Design example of the 'gated free-offtake'**.

The setting. A 'gated free-offtake' has to be designed in an existing river. The design river discharge amounts to $Q_{20} = 500$ m$^3$/s. The capacity of the continuing down-river is 300 m$^3$/s, with a uniform water depth $y = 3.93$ m and a velocity $v = 2.32$ m/s. So, the offtaking discharge $Q_t = 200$ m$^3$/s.

**Design.** The design of the gated free-offtake weir is based on the following conditions:

- **condition 1.** The energy depth in the river follows from:
  $$H_{\text{target}} = y_{\text{target}} + (v_{\text{target}})^2/2g = 3.93 + 2.32^2/2g = 4.20 \text{ m}$$

- **condition 2.** The energy head $H_t$ above the sill of the offtake equals the energy depth in the river as there are no 'side-spillway effects':
  $$H_t = H_{\text{target}} = 4.20 \text{ m}$$

- **condition 3.** The depth-discharge relation of the offtake during the maximum offtaking discharge follows the discharge formula of the 'notch': $Q_t = 1.8 \cdot b_t \cdot H_t^{3/2}$ or: $200 = 1.8 \cdot b_t \cdot 4.20^{3/2}$. So, the width $b_t$ of the gated offtake is calculated at $b_t = 12.91$ m.

- **condition 4.** The minimum orifice opening $w_t$ at the offtake depends on the critical water depth at the offtake: $w_t = 2/3 \cdot H_t = 2/3 \times 4.20 = 2.80 \text{ m}$.

### 11.2. Design of Passive Diversion Structures

#### 11.2.1. Overflow offtake with Orifice-control

**Configuration.** An 'overflow offtake with orifice-control' ("overlaat-aftapping met ondersput-stuw") consists of a 'weir' as offtake and a 'flood screen' ("bandjirscherm") as control, see figure 11.7.

**Performance.** The 'overflow offtake with orifice-control' has passive regulators. At lower discharges when the 'flood screen' is not touched by the water, the water is flowing undisturbed into the ongoing channel. At higher discharges, the orifice flow-type and the corresponding contraction, generates extra headlosses. Thus, the control becomes a water level regulator and forces the water to flow also over the overflow offtake. However, the 'passive' flood screen cannot avoid that the discharge $Q_c$ towards the ongoing channel
increases at the same time, due to the increasing (upstream) water levels, see also figure 11.7.

The 'side-channel spillway with flood screen' ("zijdelings overlaat met bandjir scherm") belongs to this configuration, and has been discussed in sub-section 7.3.2 of chapter 7. Certain aspects of this structure will be discussed here.

![Diagram of offtaking channel and flood screen configuration](image)

**Figure 11.7.** An 'overflow offtake with orifice-control'.

Design variables. The design requires that the water level \( y_{\text{target}} \) in the ongoing channel at which diversion starts, is known, as well as the corresponding headloss \( z_c \) over the control.

The 'control' is constructed in an existing channel. The sill of the control is at the channel bed level, so \( p_c = 0 \). The cross-sectional areas of the channel are required, so that the velocity \( v \) can be calculated for a known discharge \( Q \). It is assumed that the energy drop \( z_c \) at the offtake is always sufficient for free flow, and that there are no 'side-spillway effects'.

The maximum water level \( y_{\text{max}} \) or the corresponding discharge \( Q_{c,\text{max}} \) during the diversion of the design (maximum) discharge may be required for a final check of the design. **Four design variables** have to be designed, see figure 11.8:

(i) the width \( b_c \) of the orifice at the control to the continuing channel,
(ii) the width \( b_t \) of the weir at the offtake to the offtaking channel,
(iii) the orifice opening \( w_c \) at the control,
(iv) the crest-height \( p_t \) of the weir at the offtake.

Moreover, the following hydraulic parameters are also unknown:

(v) the energy head \( H_{\text{target}} \) during the target water level in the ongoing channel,
(vi) the upstream energy head \( H_{\text{dom}} \) during the dominant discharge in the incoming channel,
(vii) the energy head \( H_{t,\text{dom}} \) above the sill of the offtake during the dominant discharge,
(viii) the offtaking discharge \( Q_{t,\text{dom}} \) during the dominant discharge,
(ix) the continuing discharge \( Q_{c,\text{dom}} \) during the dominant discharge.

Design conditions. There are nine design conditions required to determine the four design variables and the five hydraulic parameters:

- **condition 1.** The orifice-opening \( w_c \) of the control is slightly above the target water level \( y_{\text{target}} \) at which diversion should start:
  \[ w_c = y_{\text{target}} \]
11. DESIGN EXAMPLES OF DIVERSION STRUCTURES

![Design Examples Diagram](image)

**Figure 11.8.** Design variables of the 'overflow offtake with orifice-control'.

- **condition 2.** The energy depth in the channel at which diversion should start, follows from:
  \[
  H_{\text{target}} = y_{\text{target}} + (y_{\text{target}})^2/2g
  \]

- **condition 3.** The orifice opening \( w_c \) at the control depends on the headloss \( z \) under submerged flow during the target water level in the ongoing channel:
  \[
  z_{\text{target}} = \alpha \sqrt{2g}
  \]
  - with \( v = \frac{Q_c}{(b_c \times y_{\text{target}})} \), for open-water flow at the target water depth \( y_{\text{target}} \), with \( y_{\text{target}} < w_c \);
  - with \( v = \frac{Q_c}{(b_c \times w_c)} \) for orifice flow, with \( y_{\text{target}} > w_c \);

- **condition 4.** The crest-height \( p_t \) of the weir at the offtake follows from:
  \[
  p_t = H_{\text{target}} + z_{\text{target}}
  \]

- **condition 5.** There are no backwater effects by the structure during the dominant discharge, so the upstream energy depth \( H_{\text{dom}} \) follows from:
  \[
  H_{\text{dom}} = H_{\text{uniform}}
  \]

- **condition 6.** The energy head \( H_t \) above the sill of the offtake, equals the energy depth \( H_c \) in the channel as there are no 'side-spillway effects':
  \[
  H_t = H_c - p_t
  \]
  So, the energy head \( H_{t,\text{dom}} \) above the sill of the offtake during the dominant discharge is:
  \[
  H_{t,\text{dom}} = H_{\text{dom}} - p_t
  \]

- **condition 7.** The depth-discharge relation over the weir at the offtake during free flow is:
  \[
  Q_t = c_t \, b_t \, H_t^{3/2}, \text{ for } z \geq \gamma H_t
  \]
  So, the offtaking discharge \( Q_{t,\text{dom}} \) during the dominant discharge follows from:
  \[
  Q_{t,\text{dom}} = c_t \, b_t \, (H_{\text{dom}} - p_t)^{3/2}
  \]

- **condition 8.** The depth-discharge relation of the control for orifice flow is:
  \[
  Q_c = C_D \, b_e \, w_c \, \sqrt{2g} \, y_c = C_D \, b_e \, w_c \, \sqrt{2g} \, \frac{H_c}{w_c} \text{ for } H_c \geq \frac{3}{2} w_c
  \]
  see also figure 8.10 of chapter 8 for the value of \( C_D \);
  So, the continuing discharge \( Q_{c,\text{dom}} \) during the dominant discharge follows from:
  \[
  Q_{c,\text{dom}} = C_D \, b_e \, w_c \, \sqrt{2g} \, \frac{H_{\text{dom}}}{w_c}
  \]
• condition 9. The width \( b_t \) of the weir at the offtake to the offtaking channel, can be solved from the continuity of flow
\[
Q_{\text{dom}} = Q_{\text{t,dom}} + Q_{c,\text{dom}}
\]

• checks. The following checks can be made on the above design:
  - the discharge \( Q_{t,\text{max}} \) into the ongoing channel during the design discharge \( Q_{20} = 500 \) m\(^3\)/s can be calculated by determining first the upstream energy head \( H_{\text{max}} \) from the continuity of flow during the design (maximum) discharge:
\[
Q_{\text{max}} = c_t b_t (H_{\text{max}} - p_t)^{3/2} + C_D b_c w_c \sqrt{2g \ H_{\text{max}}}
\]
and by entering the upstream energy head \( H_{\text{max}} \) into the discharge formula:
\[
Q_{c,\text{max}} = C_D b_c w_c \sqrt{2g \ H_{\text{max}}};
\]
  - the assumed values of \( C_D \) of the orifice formula can be checked with the values of \( C_D \) in figure 8.10 of chapter 8.

Design process. An example of the design process of an 'overflow offtake with orifice-control' is presented in box 11.2.

Operation. Operation of the 'overflow offtake with orifice-control' is not needed, and is even not possible. At lower discharges, the water is flowing undisturbed into the ongoing channel. At higher discharges, the control forces the water to flow over the overflow offtake. However, the discharge towards the ongoing channel increases at the same time.

Box 11.2. Design example of the 'overflow offtake with orifice-control'.

The setting. An 'overflow offtake with orifice-control' has to be designed in an existing river. The design discharge amounts to \( Q_{20} = 500 \) m\(^3\)/s, and the maximum capacity of the down-river is \( \pm 200 \) m\(^3\)/s.

The flood diversion should start at a water depth \( y_{\text{target}} = 3.13 \) m in the ongoing channel with a corresponding discharge \( Q = 120 \) m\(^3\)/s and velocity \( v = 2.04 \) m/s. The headloss of the control should be limited to \( z = 0.10 \) m during the \( Q_{\text{ongoing}} = 120 \) m\(^3\)/s, and follows from \( z = 0.25 \frac{v^2}{2g} \).

The dominant discharge of the upstream river is 300 m\(^3\)/s, with an uniform water depth \( y = 3.93 \) m and a velocity \( v = 2.32 \) m/s.

Design. The design of the overflow offtake with orifice-control is based on the following conditions:

• condition 1. The orifice-opening \( w_c \) of the control is slightly above \( y_{\text{target}} \), so:
\[
w_c \approx y_{\text{target}} = 3.13 \text{ m}.
\]

• condition 2. The energy depth at the river at which diversion should start, follows from:
\[
H_{\text{target}} = y_{\text{target}} + (v_{\text{target}})^2/2g = 3.13 + 2.04^2/2g = 3.34 \text{ m}.
\]

• condition 3. The headloss \( z = 0.10 \) m over the control under submerged flow, so:
\[
\begin{align*}
v &= \frac{Q}{b_c \cdot y_{\text{target}}} = \frac{120}{2.32 } \times 3.13 \\
&= (8g \times 0.10)^{1/2} = 2.80 \text{ m/s}.
\end{align*}
\]

hence: the width of the orifice \( b_c = 120 / (3.13 \times 2.80) = 13.69 \) m.

• condition 4. The crest-height \( p_t \) of the weir at the offtake follows from:
\[
p_t = H_{\text{target}} + z = 3.34 + 0.10 = 3.44 \text{ m}.
\]

• condition 5. No backwater curve at \( Q_{\text{dom}} \):
\[
H_{\text{dom}} = H_{\text{uniform}} = 3.93 + 2.32^2/2g = 4.20 \text{ m}.
\]
(to be continued in the next box)
Box 11.2 (continued). Design example of the 'overflow offtake with orifice-control'.

- **Condition 6.** The energy head \( H_{\text{t,dom}} \) above the sill of the offtake during the dominant discharge equals, as there are no 'side-spillway effects':
  \[
  H_{\text{t,dom}} = H_{\text{dom}} - p_t = 4.20 - 3.44 = 0.76 \text{ m}.
  \]

- **Condition 7.** The offtaking discharge over the weir at the offtake during the dominant discharge is:
  \[
  Q_{\text{t,dom}} = c_t b_t (H_{\text{dom}} - p_t)^{3/2} = 2.1 b_t 0.76^{3/2} = 1.39 b_t
  \]

- **Condition 8.** The continuing discharge through the control during the dominant discharge is:
  \[
  Q_{\text{c,dom}} = C_D b_c w_c \sqrt{2g H_{\text{dom}} = 0.4 \times 13.69 \times 3.13 \sqrt{2g \times 4.20} = 156 \text{ m}^3/\text{s}}
  \]

  With \( C_D = 0.4 \) during the dominant discharge, or with \( C_D = 0.5 \) during the maximum discharge;

- **Condition 9.** The diverted discharges into the diversion channel and the ongoing river during the dominant discharge equals \( Q_{\text{dom}} \), so:
  \[
  Q_{\text{dom}} = c_t b_t (H_{\text{dom}} - p_t)^{3/2} + C_D b_c w_c \sqrt{2g H_{\text{dom}}} = 300 = 1.39 b_t + 156
  \]

  Hence the width \( b_t \) of the offtaking weir is \( b_t = 104 \text{ m} \).

Check. The discharge \( Q_{\text{c,max}} \) into the ongoing channel during the design discharge \( Q_{20} = 500 \text{ m}^3/\text{s} \) follows from the two discharge formulae:

\[
Q_{\max} = c_t b_t (H_{\max} - p_t)^{3/2} + C_D b_c w_c \sqrt{2g H_{\max}}
\]

\[
500 = 2.1 \times 104 (H_{\max} - 3.44)^{3/2} + 0.5 \times 13.69 \times 3.13 \sqrt{2g \times H_{\max}}
\]

\[
H_{\max} = 3.44 \left( \frac{500}{2.1 \times 104} - 0.5 \times 13.69 \times 3.13 \sqrt{2g \times H_{\max}} \right)^{2/3}
\]

\[
H_{\max} = 3.44 + (2.29 - 0.434 \sqrt{H_{\max}})^{2/3}
\]

This equation is solved by iteration: assume \( H_{\max} = 4.20 \text{ m} \) and calculate \( H_{\max} = 4.69 \text{ m} \). And again: assume \( H_{\max} = 4.69 \text{ m} \) and calculate \( H_{\max} = 4.66 \text{ m} \). And finally: assume \( H_{\max} = 4.66 \text{ m} \) and calculate \( H_{\max} = 4.66 \text{ m} \). So, \( H_{\max} = 4.66 \text{ m} \).

The discharge \( Q_{\text{ongoing}} \) into the ongoing channel during the design discharge follows from:

\[
Q_{\text{ongoing}} = C_D b_c w_c \sqrt{2g H_{\max}} = 0.5 \times 13.69 \times 3.13 \sqrt{2g \times 4.66}
\]

So that \( Q_{\text{ongoing}} = 205 \text{ m}^3/\text{s} \), which is acceptable.

11.2.2. Proportional Diversor

**Configuration.** A 'proportional divisor' ("proportionele verdeler") consists of two weirs, one as the offtake and one as the control, see figure 11.9.

**Performance.** A proportional divisor is applied when an incoming discharge has to be splitted into an ongoing channel and an offtaking flood channel by a fixed ratio, see figure 11.10.

The weirs are 'passive' regulators, and the discharge is regulated into a fixed ratio. The water level cannot be regulated. Thus, a proportional divisor is a pure type of discharge control: the primary purpose of the structure is to maintain a certain ratio-in-discharges to a downstream destination.
Design variables. It is assumed here, that the 'control' is constructed in an existing channel. The depth-discharge relation of the ongoing channel should be known. Furthermore, the cross-sectional areas of the channel are required, so that the velocity \( v_{\text{ongoing}} \) and the energy head \( H_{\text{ongoing}} \) can be calculated for a known discharge \( Q \). It is assumed that the energy drop \( z_t \) at the offtake is always sufficient for free flow. There are no 'side-spillway effects'.

Four design variables have to be designed, see figure 11.11:

(i) the width \( b_c \) of the weir at the control to the continuing channel,
(ii) the width \( b_t \) of the weir at the offtake to the offtaking channel,  
(iii) the crest-height \( p_c \) of the weir at the control, and  
(iv) the crest-height \( p_t \) of the weir at the offtake.  
Moreover, the following hydraulic parameters are also unknown:  
(v) the energy head \( H_{\text{ongoing}} \) in the ongoing channel,  
(vi) the energy head \( H_c \) above the weir-crest at the control, and  
(vii) the energy head \( H_t \) above the weir-crest at the offtake.  

\[ H_{\text{ongoing}} = y_{\text{ongoing}} + \frac{(v_{\text{ongoing}})^2}{2g} \]

\[ n = \frac{Q_t}{Q_t + Q_c} \times 100\% \]

where \( n \) is a constant, e.g. \( n = 50\% \) for an equal splitting. So:

\[ Q_{\text{ongoing}} = (100\% - n) \times Q_{\text{incoming}} \]

\[ z_c \geq H_c \]

\[ H_c + p_c = H_{\text{dom}} + \gamma H_c \]

\[ H_{\text{dom}} = \frac{Q_c}{b_c H_c^{3/2}} \]

\[ H_t + p_t = \gamma_1 H_t \]

\[ Q_t = c_t b_t H_t^{3/2} \]

\[ p_c = p_t \]

\[ Q_t = c_t b_t H_t^{3/2} \]

---

**Figure 11.11.** Design variables of the proportional divisor.

**Design conditions.** There are seven design conditions required to determine the four design variables and the three hydraulic parameters:  
- **condition 1.** The ongoing discharge \( Q_{\text{ongoing}} \) follows from the fixed proportional ratio \( n \) between the offtaking discharge \( Q_t \) and the approaching discharge \( Q_{\text{incoming}} = Q_t + Q_c \) reads:

- **condition 2.** The energy depth in the channel for a discharge \( Q_{\text{ongoing}} \) follows from:

- **condition 3.** There is just 'free flow' over the weir of the control during e.g. the diversion of the dominant discharge:

- **condition 4.** There should be 'uniform flow' during the dominant discharge in the incoming channel. So, the energy depth at the control equals the uniform energy depth \( H_{\text{uniform}} \) in the incoming channel during the dominant discharge \( Q_{\text{dom}} \):

- **condition 5.** The depth-discharge relation over the weir at the control during free flow is:

- **condition 6.** Proportional diversion is only possible when the crest-height \( p_c \) of the weir at the control is equal to the crest-height \( p_t \) of the weir at the offtake: \( p_c = p_t \)

- **condition 7.** The depth-discharge relation over the weir at the offtake during free flow is:

\[ Q_t = c_t b_t H_t^{3/2} \], for \( z \geq \gamma_1 H_t \]
Box 11.3. Design example of the 'proportional divisor'.

The setting. A flood diversion structure has to be designed in an existing river. The design river discharge amounts to $Q_{20} = 500 \text{ m}^3/\text{s}$. The dominant discharge of the river is assumed at $Q_2 = 300 \text{ m}^3/\text{s}$, with a uniform water depth $y = 3.93 \text{ m}$ and a velocity $v = 2.32 \text{ m/s}$. The capacity of the continuing downriver is $200 \text{ m}^3/\text{s}$. A discharge of $120 \text{ m}^3/\text{s}$ in the ongoing river leads to a (uniform) water depth $y = 3.13 \text{ m}$ and a velocity $v = 2.04 \text{ m/s}$.

The new flood diversion channel will transport a fixed ratio of 60% of the river discharge. The downriver discharge $Q_c$ should reach the capacity of $200 \text{ m}^3/\text{s}$ at the design discharge $Q_{20}$. There should be (just) 'free flow' over the weir of the control during the dominant discharge $Q_{\text{dom}}$.

Design. The structure is designed with two sharp-crested weirs with $c = 2.1 \text{ m}^{1/3}/\text{s}$, at an equal crest-height $p$ above the bed. Free flow over the weir occurs at the headloss $z \geq \frac{1}{2} H_c$.

- **condition 1.** The ongoing discharge $Q_{\text{ongoing}}$ follows from the fixed proportional ratio $n$ between the offtaking discharge $Q_t$ and the approaching discharge $Q_t + Q_c$ reads:

  $$n = \frac{Q_t}{Q_t + Q_c} \times 100\% = 60\%$$

  So for the dominant discharge $Q_{\text{dom}}$: $Q_{\text{ongoing}} = (100\% - 60\%) \times Q_{\text{incoming}} = 0.40 \times 300 = 120 \text{ m}^3/\text{s}$.

- **condition 2.** The energy depth in the river during the $Q_c = 120 \text{ m}^3/\text{s}$ follows from:

  $$H_{\text{ongoing}} = y_{\text{ongoing}} + (v_{\text{ongoing}}^2/2g) = 3.13 + 2.04^2/2g = 3.34 \text{ m}.$$  

- **condition 3.** There is just 'free flow' over the weir of the control during the diversion of the dominant discharge:

  $$z_c \geq \frac{1}{2} H_c$$

  becomes: $H_c + p_c = H_{\text{ongoing}} + \frac{1}{2} H_c$ or: $\frac{1}{2} H_c + p_c = 3.34 \text{ m}$.

- **condition 4.** There should be 'uniform flow' during the dominant discharge in the upstream river channel. So, the energy depth at the control equals the uniform energy depth $H_{\text{uniform}}$ in the upstream river channel during the dominant discharge $Q_{\text{dom}}$:

  $$H_c + p_c = H_{\text{uniform}} = 3.93 + 2.32^2/2g = 4.20 \text{ m}.$$  

  Subtracting this equation from the above equation of condition 2: $\frac{1}{2} H_c = 0.86$. Thus, $H_c = 1.72$ and the sill level $p_c = 4.20 - 1.72 = 2.48 \text{ m}$.

- **condition 5.** The depth-discharge relation over the weir at the control during free flow is:

  $$Q_c = 2.1 b_c H_c^{3/2}, \text{ for } z \geq \frac{1}{3} H_c$$  

  or: $120 = 2.1 b_c 1.72^{3/2}$, so that the weir at the control has a width $b_c = 25.33 \text{ m}$.

- **condition 6.** Proportional diversion is only possible when the crest-height $p_c$ of the weir at the control is equal to the crest-height $p_t$ of the weir at the offtake:

  $$p_c = p_t = 2.48 \text{ m}.$$  

- **condition 7.** The depth-discharge relation over the weir at the offtake during free flow is:

  $$Q_t = c b_t H_t^{3/2}, \text { or: } 180 = 2.1 b_t 1.72^{3/2}, \text { so that the weir at the control has a width } b_t = 38.00 \text{ m}. \text{ (or alternatively: } b_t = n \times b_c = 1.5 \times 25.33 = 38.00 \text{ m}).$$
Design process. An example of the design process of a proportional divisor is presented in box 11.3.

Operation. Operation of the proportional divisor is not needed, and is even not possible. Any approaching flow is diverted into the fixed ratio $n$.

Application. Applications of the proportional divisors can be found in river engineering and in flood control. Rivers in deltas are often diverting in several river branches. Only the application of a proportional divisor guarantees a fully-proportional diversion.

Proportional divisors are widely applied in irrigation. The most simple type of proportional divisor in irrigation is the Fayoum weir. The Fayoum weir has been developed in Egypt in the 1920s. The Fayoum weir is basically a broad-crested weir with a crest-height $p$ above the bed, and a crest-width $b$ between vertical abutments, see also figure 11.10.

Both underflow? The same performance could have been obtained by a proportional divisor with a configuration of two orifices with underflow.

However, such a configuration requires a higher headloss $z = \frac{1}{2} H$ as the energy head $H$ is large, e.g. $z = 1.50$ m for a canal-depth $y_1 = 3.00$ m. Moreover, the height $w$ of the opening of the orifice should be taken quite small, in order to guarantee underflow also for the smaller discharges. Thus, a proportional divisor with two orifices should not be applied.

### 11.2.3. Proportional divisor with Orifice-control

Configuration. A proportional divisor with orifice-control ("proportionele verdeler met onderspui-stuw") consists of a 'weir' as offtake and a 'weir with flood screen' ("bandjirscherm") as control, see figure 11.12.

![Configuration and Performance](image-url)

**Figure 11.12.** A 'proportional divisor with orifice-control'.
Performance. The 'proportional divisor with orifice-control' is composed of passive regulators. The structure is identical to the proportional divisor at the lower discharges when the water does not touch the flood screen. At the higher discharges, the control functions as an orifice with a high sill.

A proportional divisor with orifice-control is applied when the discharge in the incoming channel has to be split into the ongoing channel and the offtaking flood channel by a fixed ratio, but only to a maximum discharge $Q_{\text{cap}}$ in the ongoing channel, see also figure 11.12. Design variables. It is assumed here, that the 'control' is constructed in an existing channel. The maximum water level $y_{\text{ongoing}}$ in the ongoing channel should be known. Furthermore, the cross-sectional areas of the channel are required, so that the velocity $v_{\text{ongoing}}$ can be calculated for a known discharge $Q$. It is assumed that the energy drop $z_t$ at the offtake is always sufficient for free flow. There are no 'side-spillway effects'. Five design variables have to be designed, see figure 11.13:
(i) the width $b_c$ of the weir at the control to the continuing channel,
(ii) the width $b_t$ of the weir at the offtake to the offtaking channel,
(iii) the crest-height $p_c$ of the weir at the control,
(iv) the crest-height $p_t$ of the weir at the offtake,
(v) the orifice-opening $w_c$ at the control.
Moreover, the following hydraulic parameters are also unknown:
(vi) the energy head $H_{\text{ongoing}}$ in the ongoing channel,
(vii) the energy head $H_c$ above the weir-crest at the control,
(viii) the energy head $H_t$ above the weir-crest at the offtake.

![Diagram](attachment:diagram.png)

**Figure 11.13.** Design variables of the 'proportional divisor with orifice control'.

Design conditions. There are eight design conditions required to determine the five design variables and the three hydraulic parameters:
- **condition 1.** The ongoing discharge $Q_{\text{ongoing}}$ follows from the fixed proportional ratio $n$ between the offtaking discharge $Q_t$ and the approaching discharge $Q_{\text{incoming}} = Q_t + Q_c$ reads:
\[ n = \frac{Q_t}{Q_t + Q_c} \times 100\% \]

where \( n \) is a constant, e.g. \( n = 50\% \) for an equal splitting. So:

\[ Q_{\text{ongoing}} = (100\% - n) \times Q_{\text{incoming}} \]

- **condition 2.** The energy depth in the channel for a discharge \( Q_{\text{ongoing}} \) follows from:

\[ H_{\text{ongoing}} = y_{\text{ongoing}} + \left( \frac{v_{\text{ongoing}}}{2g} \right)^2 / 2g \]

- **condition 3.** There is just 'free flow' over the weir of the control during e.g. the diversion of the dominant discharge:

\[ z_c \geq \gamma H_c \text{ becomes: } H_c + p_c = H_{\text{ongoing}} + \gamma H_c \]

- **condition 4.** For instance, there should be 'uniform flow' during the dominant discharge in the incoming channel. So, the energy depth at the control equals the uniform energy depth \( H_{\text{uniform}} \) in the incoming channel during the dominant discharge \( Q_{\text{dom}} \):

\[ H_c + p_c = H_{\text{uniform}} \]

- **condition 5.** The depth-discharge relation of the control under free flow is:
  - for weir flow, so \( H_c < 3/2 w_c \):
    \[ Q_c = c_c b_c H_c^{3/2}, \text{ for } z \geq \gamma_c H_c \]
  - for orifice flow, so \( H_c \geq 3/2 w_c \):
    \[ Q_c = C_D b_c w_c \sqrt{\frac{2g y_c}{2g H_c}} = C_D b_c w_c \sqrt{2g H_c} \]

see also figure 8.10 of chapter 8 for the value of \( C_D \);

- **condition 6.** Proportional diversion is only possible when the crest-height \( p_c \) of the weir at the control is equal to the crest-height \( p_t \) of the weir at the outtake:

\[ p_c = p_t \]

- **condition 7.** The depth-discharge relation over the weir at the outtake during free flow is:

\[ Q_t = c_t b_t H_t^{3/2}, \text{ for } z \geq \gamma_t H_t \]

- **condition 8.** The proportional diversion should be transformed into an orifice flow at the control for a certain incoming channel discharge \( Q_{\text{river}} \). It means that the nappe touches the flood screen. Hence, the critical water depth \( y_c = 2/3 H_c \) equals the opening-height \( w_c \) during the ongoing discharge \((1 - n) Q_{\text{river}}\). An additional condition may added: the discharge \( Q_c \) into the ongoing channel should not exceed the capacity \( Q_{\text{cap}} \) of the ongoing channel, so:

\[ Q_c \leq Q_{\text{cap}} \]

**Design process.** An example of the design process of a proportional divisor with orifice-control is presented in box 11.4.

**Operation.** Operation of the proportional divisor with flood screen is not needed, and is even not possible. Any approaching flow is diverted into the fixed ratio \( n \) up to the capacity of the ongoing channel is reached.

**Application.** Applications of the proportional divisors can be found in flood control. The proportional divisor with flood screen is also applied in irrigation at the headworks ("watervang, prise d'eau, hoofd-inlaatwerk"). However, the flood screen ("bandjirscherm") is constructed at the outtake, as to limit the intake discharge into the primary irrigation canal during river floods.
Box 11.4. Design example of the 'proportional divisor with orifice-control'.

The setting. The flood diversion structure of the above example was a 'proportional divisor' to divert the incoming river discharge up to $Q_{20} = 500 \text{ m}^3/\text{s}$ in a fixed ratio of 60% into a new flood diversion channel. So, the downriver discharge $Q_c$ should reach the capacity of 200 m$^3$/s at the design discharge $Q_{20}$.

It was calculated that the offtaking weir should have a width $b_1 = 38.00 \text{ m}$, the weir at the control should have a width $b_c = 25.33 \text{ m}$, and that both weirs should have an equal height $p = 2.48 \text{ m}$. The energy head $H$ above the weir crest was calculated at $H_c = 1.72 \text{ m}$ during the diversion of the dominant discharge $Q_{dom}$.

Now, the additional objective is that the diversion of floods between $Q_{20} = 500 \text{ m}^3/\text{s}$ and $Q_{50} = 700 \text{ m}^3/\text{s}$ should be diverted: (i) into the downriver up to $Q_{cap} = 200 \text{ m}^3/\text{s}$ and the remainder into the new diversion channel. Therefore, the above proportional divisor will be transformed into a 'proportional divisor with orifice-control' by constructing a flood screen above the weir at the control.

Design. The additional design variable is now the orifice opening $w$ at the control between the weir crest and the flood screen.

- **condition 1.** The depth-discharge relation for overflow of the control under free flow is:
  
  \[ Q_c = 2.1 \ b_c \ H_c^{3/2}, \text{ so } 0.40 \times 500 = 2.1 \times 25.33 \times 1.72^{3/2}, \text{ and } H_c = 2.42 \text{ m} \]

- **condition 2.** The proportional diversion is into an orifice flow at the control when the critical water depth $y_c = \frac{2}{3} H_c = 1.61 \text{ m}$ touches the flood screen. It means that the opening-height $w_c = 1.61 \text{ m}$.

Check. A check is made whether the downriver discharge $Q_c = 200 \text{ m}^3/\text{s}$ is maintained.

- **condition 2:** the discharge formula of the control during these higher discharges depends on the head $H_c$ above the weir-crest and is related to the orifice flow, so:
  
  \[ Q_c = 2.3 \ b_w \ H_c^{1/2} = 2.3 \times 25.33 \times 1.61 \times 2.42^{1/2} = 93.80 H_c^{1/2} \]

- **condition 3:** the discharge formula of the offtake during these higher discharge remains the overflow formula with $H = H_c$, so:
  
  \[ Q_t = 2.1 \ b_1 \ H_c^{3/2} = 2.1 \times 38.00 \times 2.42^{3/2} = 79.80 \times H_c^{3/2}. \]

Rating curve. The rating curve for the structure cannot calculated directly, but should follow an iteration by assuming the energy head $H$, above the crest:

- for $H_c = 2.50 \text{ m}$: $Q_c = 93.80 \times 2.50^{1/2} = 148 \text{ m}^3/\text{s}$ and $Q_t = 79.80 \times 2.50^{3/2} = 315 \text{ m}^3/\text{s}$, so that the incoming river discharge is $Q_{river} = 463 \text{ m}^3/\text{s}$,

- for $H_c = 2.75 \text{ m}$: $Q_c = 93.80 \times 2.75^{1/2} = 156 \text{ m}^3/\text{s}$ and $Q_t = 79.80 \times 2.75^{3/2} = 364 \text{ m}^3/\text{s}$, so that the incoming river discharge is $Q_{river} = 520 \text{ m}^3/\text{s}$,

- for $H_c = 3.00 \text{ m}$: $Q_c = 93.80 \times 3.00^{1/2} = 162 \text{ m}^3/\text{s}$ and $Q_t = 79.80 \times 3.00^{3/2} = 415 \text{ m}^3/\text{s}$, so that the incoming river discharge is $Q_{river} = 571 \text{ m}^3/\text{s}$,

- for $H_c = 3.50 \text{ m}$: $Q_c = 93.80 \times 3.50^{1/2} = 175 \text{ m}^3/\text{s}$ and $Q_t = 79.80 \times 3.50^{3/2} = 523 \text{ m}^3/\text{s}$, so that the incoming river discharge is $Q_{river} = 698 \text{ m}^3/\text{s}$.

It is concluded that the additional objective has been met by constructing a flood screen above the weir at the control.
11.3. Design of Gated Diversion Structures

11.3.1. Gated offtake with Weir-control

**Configuration.** A 'gated offtake with weir-control' ("regelbare aftapping met overlaat stuw") consists of a gated offtake and a weir as the control, see figure 11.14.

**Performance.** A 'gated offtake with weir control' is applied when the discharge in the ongoing channel should not exceed a certain value $Q_{\text{target}}$, see also figure 11.14. The offtake is a manual or an hydro-mechanical water level regulator, and diverts a discharge when the discharge exceeds the target discharge $Q_{\text{target}}$.

![Diagram of Gated Offtake with Weir-Control](image)

**Figure 11.14.** A 'gated offtake with weir-control'.

**Design variables.** The design requires that the maximum discharge $Q_{\text{target}}$ into the ongoing channel and at which diversion starts, is known. The 'control' is constructed in an existing channel, but should flow under free flow. The cross-sectional areas of the channel are required, so that the velocity $v$ can be calculated for a known discharge $Q$.

The sill of the gated offtake is at the channel bed level, so $p_t = 0$. It is assumed that the energy drop $z_t$ at the offtake is always sufficient for free flow, and that there are no 'side-spillway effects'.

**Four design variables** have to be designed, see figure 11.15:

(i) the width $b_c$ of the weir at the control to the continuing channel,
(ii) the crest-height $p_c$ of the weir at the control,
(iii) the width $b_i$ of the gated offtake to the offtaking channel,
(iv) the orifice opening $w_i$ of the gated offtake.
Figure 11.15. Design variables of the 'gated offtake with weir-control'.

Figure 11.16. A 'gated offtake with weir-control' in irrigation (DHV 1986).
Moreover, the following hydraulic parameters are also unknown:
(v) the energy head $H_{\text{ongoing}}$ during the maximum discharge in the ongoing channel,
(vi) the energy head $H_c$ above the sill of the weir during the maximum discharge,
(vii) the upstream energy head $H_{\text{dom}}$ during the dominant discharge in the incoming channel.

**Design conditions.** There are seven design conditions required to determine the four design variables and the three hydraulic parameters:

- **condition 1.** The energy depth $H_{\text{ongoing}}$ in the channel during a discharge $Q_{\text{ongoing}}$ into the ongoing channel follows from:
  \[ H_{\text{ongoing}} = y_{\text{ongoing}} + \left(v_{\text{ongoing}}\right)^2/2g \]

- **condition 2.** There is just 'free flow' over the weir of the control during the dominant discharge:
  \[ z_c \geq \frac{1}{2} H_c \text{ becomes: } \frac{1}{2} H_c = z_c. \text{ So: } \frac{1}{2} H_c = (H_c + p_c) - (H_{\text{ongoing}}) \]
  **equation 1:** \[ \frac{1}{2} H_c + p_c = H_{\text{ongoing}} \]

- **condition 3.** The energy depth $H_{\text{dom}}$ in the channel during the dominant discharge $Q_{\text{dom}}$ follows from:
  \[ H_{\text{dom}} = y_{\text{dom}} + \left(v_{\text{dom}}\right)^2/2g \]

- **condition 4.** There should be 'uniform flow' during the dominant discharge in the incoming channel. So, the energy depth at the control equals the uniform energy depth $H_{\text{uniform}}$ in the incoming channel during the dominant discharge $Q_{\text{dom}}$:
  **equation 2:** \[ H_c + p_c = H_{\text{uniform}} \]
  The sill level $p_c$ can be calculated by subtracting this equation from the above equation 1.

- **condition 5.** The width $b_c$ of the weir at the control into the ongoing channel is determined by the maximum discharge in the ongoing channel:
  \[ Q_c = c_c b_c H_c^{3/2} \]

- **condition 6.** The width $b_o$ of the offtake is determined by the maximum discharge into the offtaking channel, while the energy head $H_t = H_c + p_c$ at the crest is determined by the capacity of the ongoing channel. So:
  \[ Q_o = 1.8 b_o (H_c + p_c)^{3/2} \]

- **condition 7.** The (minimum) orifice opening $w_t$ at the offtake depends on the critical water depth at the offtake:
  \[ w_t = \frac{2}{3} (H_c + p_c) \]

**Design process.** An example of the design process of a gated-offtake with weir-control is presented in box 11.5.

**Operation.** Operation of the gated offtake with weir-control is quite effective. The discharge $Q_c$ to the ongoing channel is determined by measuring the energy depth $H_c$ above the weir crest and by applying the discharge formula of the weir. The gated offtake is regulated in such a way that the ongoing discharge $Q_c$ does not exceed the target discharge $Q_{\text{target}}$ to the ongoing channel. In fact, the regulation is transformed into a 'water level control', by maintaining the energy head $H_{\text{target}}$ above the crest. In practice, the upstream water level may not exceed the $y_{\text{target}}$.

**Application.** Applications of the gated offtake with weir-control can be widely found in flood control. The configuration is also applied in irrigation at the headworks ("watervang, prise d'eau, hoofd-inlaatwerk"), see figure 11.16, and at secondary and tertiary offtakes.
Box 11.5. Design example of the 'gated offtake with weir-control'.

The setting. A 'gated offtake with weir control' has to be designed in an existing river. The design river discharge amounts to $Q_{20} = 500 \text{ m}^3/\text{s}$. The dominant discharge of the upstream river is 300 $\text{m}^3/\text{s}$, with a uniform water depth $y = 3.93 \text{ m}$ and a velocity $v = 2.32 \text{ m/s}$. The capacity of the down-river is 200 $\text{m}^3/\text{s}$, with an uniform water depth $y = 3.13 \text{ m}$ and a velocity $v = 2.04 \text{ m/s}$.

Design. There are two general discharge formulae:

- the depth-discharge relation over the weir at the control during free flow is:
  \[ Q_c = 2.1 b_c H_c^{3/2}, \text{ for } z \geq \frac{1}{2} H_c \]
- the depth-discharge relation of the offtake during the maximum offtaking discharge follows the discharge formula of the 'notch':
  \[ Q_t = 1.8 b_t H_t^{3/2}, \text{ for } z_t \geq \frac{1}{3} H_t \]

Furthermore, there are seven conditions to solve the seven unknown parameters:

- **condition 1.** The energy depth in the river during the maximum discharge $Q_{\text{ongoing}} = 200 \text{ m}^3/\text{s}$ into the down-river follows from:
  \[ H_{\text{ongoing}} = y_{\text{ongoing}} + \left(\frac{v_{\text{ongoing}}}{2g}\right)^2/2g = 3.13 + 2.04^2/2g = 3.34 \text{ m}. \]

- **condition 2.** There is just 'free flow' over the weir of the control during the dominant discharge:
  
  \[ z_c \geq \frac{1}{2} H_c \text{ becomes: } \frac{1}{2} H_c = z_c. \text{ So: } \frac{1}{2} H_c = (H_c + p_c) - (H_{\text{ongoing}}) \]

  \[ \text{equation 1: } \frac{1}{2} H_c + p_c = H_{\text{ongoing}} \]

  \[ \frac{1}{2} H_c + p_c = 3.34 \]

- **condition 3.** The energy depth $H_{\text{dom}}$ in the river during the dominant discharge $Q_{\text{dom}} = 300 \text{ m}^3/\text{s}$ follows from:
  \[ H_{\text{dom}} = y_{\text{dom}} + \left(\frac{v_{\text{dom}}}{2g}\right)^2/2g = 3.93 + 2.32^2/2g = 4.20 \text{ m}. \]

- **condition 4.** There should be 'uniform flow' during the dominant discharge in the upstream river channel. So, the energy depth at the control equals the uniform energy depth $H_{\text{uniform}}$ in the upstream river channel during the dominant discharge $Q_{\text{dom}}$:

  \[ \text{equation 2: } H_c + p_c = H_{\text{uniform}} \]

  Subtracting this equation from the above equation 1: $\frac{1}{2} H_c = 0.86$. Thus, $H_c = 1.72$ and the sill level $p_c = 4.20 - 1.72 = 2.48 \text{ m}$. 

- **condition 5.** The width $b_c$ of the weir at the control into the ongoing river is determined by the maximum discharge in the ongoing river:

  \[ Q_c = c_c b_c H_c^{3/2}, \text{ or } 200 = 2.1 b_c 1.72^{3/2}, \text{ so } b_c = 42.22 \text{ m}. \]

- **condition 6.** The width $b_t$ of the offtake is determined by the maximum discharge into the offtaking channel, while the energy head $H_t = H_c + p_c$ at the crest is determined by the capacity of the down-river. So:

  \[ Q_t = 1.8 b_t (H_c + p_c)^{3/2} \text{ and } 300 = 1.8 b_t (1.72 + 2.48)^{3/2} \]

  Hence, the $b_t = 19.36 \text{ m}$. 

- **condition 7.** The (minimum) orifice opening $w_t$ at the offtake depends on the critical water depth at the offtake:

  \[ w_t = \frac{1}{3} (H_c + p_c) \text{ so: } w_t = \frac{1}{3} (1.72 + 2.48) = 2.80 \text{ m}. \]
11.3.2. Overflow offtake with Gated-control

**Configuration.** An 'overflow offtake with gated-control' ("overlaat-aftapping met regelbare-stuw") consists of a weir as offtake and a gated regulator as the control, see figure 11.17.

**Performance.** The performance of the 'overflow offtake with gated-control' is basically equal to the 'overflow offtake with orifice-control'. But here, the discharge to the ongoing channel can be regulated.

![CONFIGURATION Diagram](image1)

**Figure 11.17.** An 'overflow offtake with gated-control'.

**Design.** Also the design of the 'overflow offtake with gated-control' is equal to the 'overflow offtake with orifice-control', see figure 11.18. Therefore, reference is made to sub-section 11.2.1 of this chapter for the design process and a design example.

![PLAN Diagram](image2)

**Figure 11.18.** Design variables of the 'overflow offtake with gated control'.
12. DROP STRUCTURES AND STILLING BASINS

12.1. Introduction

12.1.1. Types of Energy Dissipators

Need for energy dissipators. 'Energy dissipators' are required whenever an excess of energy would damage an unlined canal. This may be required, see figure 12.1:

- when the gradient of the terrain is steeper than the maximum permissable slope of the canal. Thus, a 'fall structure' is needed;
- when a 'super-critical flow' may occur in a flow control structure, such as a water level regulator, a discharge regulator, and a discharge measurement structure. Here, a 'stilling basin' is needed.

![Diagram of drop structures and stilling basin](image)

**Figure 12.1.** The danger of scouring requires energy dissipators.

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Manner of energy dissipation. Energy dissipation can occur in different manners: (i) due to friction along a sloping glacis, (ii) by impact of the flow against the floor, and (iii) by turbulence in the stilling basin. So, different types of energy dissipators can be distinguished, see figure 12.2.

![Energy Dissipators Diagram]

Figure 12.2. Classification of energy dissipators.

Types of fall structures. There are three types of fall structures to dissipate the excess energy, see figure 12.3:

- A 'chute' or 'flume' dissipates the major portion of the energy by friction over the 'sloping glacis' and the remainder portion by turbulence in the stilling basin. Thus, the drop of the energy level occurs not only in the stilling basin, but also along the sloping glacis.

  Chutes
  * Energy dissipation by:
    - friction on glacis
    - turbulence in basin

  Vertical Drop Structures
  * Energy dissipation by:
    - impact with floor
    - turbulence in basin

  Inclined Drop Structures
  * Energy dissipation by:
    - turbulence in basin

  Water Level Regulators
  * Energy dissipation by:
    - turbulence in basin

  Q-Measurement Structures
  * Discharge Regulators

  Fall Structures in Canal Reaches

  Stillling Basins at Flow Control Structures

- A 'vertical drop structure' has a vertical wall between the 'control' and the stilling basin. The small portion of energy loss occurs by impact of the jet on the floor. The major portion of energy loss occurs by turbulence in the stilling basin.

  Vertical drop structures can only be recommended for the smaller drops. Uplift pressure for greater drops will require thick floors to prevent uplift.

- An 'inclined drop structure' has a sloping glacis between the 'control' and the 'stilling basin'. The energy loss on the sloping glacis due to friction is normally small and is ignored, so that the energy is dissipated in the stilling basin only. The slope of the glacis is chosen as steep as $2_{Vert} : 1_{Hor}$, but is usually $1_{Vert} : 1_{Hor}$ or $1_{Vert} : 2_{Hor}$.

Stilling basins at flow control structures. It is a good principle to provide any structure where super-critical flow may occur, with a 'stilling basin'. This stilling basin will force the hydraulic jump within the structure, so that excessive scouring of the downstream channel bed can be avoided.
12.1.2. Design of Fall Structures and Stilling Basins

Components of fall structures. Fall structures and all other structures with super-critical flow have two functional parts, each having its own typical design features, see figure 12.4:

- the 'control', such as a 'control notch', a 'fixed weir', or a 'gate';
- the 'stilling basin' such as the USBR standard basins, SAF-basin.

Moreover, a 'sloping glacis' may be applied.
The Control. The water level at the top of a fall is normally regulated by a 'control' to prevent drawdown, which may lead to scouring of the upstream canal, see figure 12.5.

The options for the regulation at the control of a fall structure are:
- 'fixed' regulation, by means of a weir-crest, a notch or a screen,
- 'manual' regulation, by means of gate,
- 'hydraulic regulation', by means of a hydro-mechanical gate.

![Diagram of fall structure control types](image)

**Figure 12.5.** Control types of fall structures.

The stilling basin. The most simple stilling basin is just a lowering of the floor at the downstream end of the structure. An over-depth of e.g. $0.3 \times$ the water depth $y$ is often sufficient. But normally, the design of stilling basins follows hydraulic formulae.

Rectangular stilling basins. The stilling basin should be 'rectangular', which means that vertical walls should enclose the stilling basin, see figure 12.6.

The design formulae of typical stilling basins are two-dimensional. They are based on the unit discharge $q$ in $\text{m}^3/\text{m.s}$, and provide the basin depth and the basin length, as well as the sizes of any chute blocks, baffles and the end sill. Thus, the stilling basin should also be constructed 'two-dimensional' with vertical walls, and not 'three-dimensional' with sloping sides.

Model tests have shown that the hydraulic jump in a trapezoidal basin is much less complete over the total width than it is in a rectangular basin. Side eddies may spoil the action of the basin, and the flow over the sloping sides may by-pass the stilling basin (USBR 1973).

Onion Outlet. One of the main problems in the design of any canal system is the fitting of 'smooth' lining with a Strickler coefficient $k = 60 \text{ m}^{1/3}/\text{s}$ to the 'rough' unlined canal with a Strickler coefficient $k = 30 \text{ m}^{1/3}/\text{s}$.

The same problem arises at the end of a side-lining. The shear stress along the lining has to adjust to the permissible shear stress of unlined side slopes, and 'scour holes' quickly develop. These holes make that the lining progressively collapses. Even an end-coffer is often not an ultimate solution.

The principle of the 'onion outlet' is an applicable solution, see figure 12.7. The onion outlet allows for side eddies which do not damage the side slopes because of the low velocities.

Thus, the vertical walls of the stilling basin should be extended after the stilling basin to form an onion outlet, see also figure 12.7. The side slopes of the canal may or may not be protected by rip-rap for extra safety.
CORRECT DESIGN:
Stilling Basin
with \textit{VERTICAL} Sidewalls
(no by-passing of flow)

WRONG DESIGN:
Stilling Basin
with \textit{SLOPING} Sidewalls
(by-passing of flow)

\textbf{Figure 12.6.} A stilling basin should be enclosed by rectangular walls.

\textbf{Figure 12.7.} 'Onion outlet' of a stilling basin.
12.2. Hydraulics of stilling basins

12.2.1. Froude number

Froude number. The Froude number describes the state of the free-surface flow, and is important for the design of stilling basins. The flow is called 'sub-critical' or 'normal flowing' ("stromend") when \( Fr < 1.0 \). The flow is 'super-critical' or 'shooting' ("schie-tend") when \( Fr > 1.0 \).

The Froude number is defined as the ratio of the stream velocity \( v \) to the wave velocity \( \sqrt{gy} \), and is expressed by the formula:

\[
Fr = \frac{v}{\sqrt{g y}}, \quad \text{or} \quad Fr = \frac{q}{\sqrt{g y^3}}
\]

where: \( Fr \) is the Froude number, \( v \) is the velocity of the water in m/s, \( y \) is the water depth in m, \( q \) is the discharge per unit width in m\(^3\)/s, and \( g = 9.8 \) m/s\(^2\) = 9.8 N/kg is the gravity acceleration.

![Figure 12.8. Parameters of the hydraulic jump.](image)

Hydraulics of inflowing jet. The type of stilling basin to be designed depends on the energy of the incoming flow, the 'jet' ("straal"), expressed by the Froude number of the incoming jet, see figure 12.8.

The water depth \( y_j \) and the velocity \( v_j \) of the incoming flow into the stilling basin can be calculated for a known discharge \( q \) per unit width and the energy head \( H_j \) from the two equations:

\[
y_j \times \sqrt{v_j} = q, \quad \text{and} \quad y_j + \frac{v_j^2}{2g} = H_j
\]

where: \( q \) is the discharge per unit width in m\(^3\)/s, \( H_j \) is the energy depth of the jet above the bed in m, \( y_j \) is the water depth of jet in m, \( v_j \) is the velocity of the jet in m/s, and \( g = 9.8 \) m/s\(^2\) = 9.8 N/kg is the gravity acceleration.

**Iteration.** The above equations can be solved by eliminating \( v_j \), so:

\[
y_j = \frac{q}{\sqrt{2g (H_j - y_j)}}
\]
The value of $y_j$ can be determined through iteration. First, it is assumed that $y_j = 0$, and a new value of $y_j$ is calculated. This value is inserted in the formula and again a new value of

$y_j$ is calculated. The ultimate value of $y_j$ is found after a few iteration cycles, see box 12.1.

Apparently, the Froude number of a falling flow $Q$ depends on the depth $H_j$ of the fall and on the discharge $q$ per unit width:

- **a deeper stilling basin** increases the Froude number, thus $Fr >>$.
- **a more widen stilling basin** increases the Froude number, thus $Fr >>$.

**Box 12.1. Example calculation of the Froude number.**

A flow of $Q = 2.80 \text{ m}^3/\text{s}$ drops at a structure. The structure has vertical walls and has a width of $B = 2.50 \text{ m}$. The upstream water level (=energy depth) is located at $0.00 \text{ m}$, the elevation of the floor in the stilling basin has been designed at $-4.50 \text{ m}$.

The inflowing jet into the basin amounts to $q = Q / B = 1.12 \text{ m}^2/\text{s}$. The energy head above the floor is $H_j = 4.50 \text{ m}$. The water depth $y_j$ and the velocity $v_j$ of the incoming jet can be calculated by iteration: $y_j = q / \{ \sqrt{2g(H_jy_j)} \}$:

- assume $y_j = 0$, and calculate $y_j = 1.12 / \{ \sqrt{2 \times 9.8(4.50-0.00)} \} = 0.119 \text{ m}$,
- enter this value again in the above formula and calculate a new $y_j = 1.12 / \{ \sqrt{2 \times 9.8(4.50-0.12)} \} = 0.121 \text{ m}$,
- continue the iteration, until the "input" $y_j$ equals the "output" $y_j$.

Thus, the water depth $y_j$ of the inflowing jet at the stilling basin is $y_j = 0.121 \text{ m}$, with a corresponding velocity of $v_j = q / y_j = 1.12 / 0.121 = 9.26 \text{ m/s}$. The corresponding Froude number amounts to $Fr = v_j / \sqrt{g \ y_j} = 9.26 / \sqrt{(9.8 \times 0.121)} = 8.5$.

Hence, the Froude number of the flow at the level of the stilling floor at $-4.50 \text{ m}$ amounts to $Fr = 8.5$.

A similar calculation can be carried out for floor depths of -3.00 m and -1.50 m, when Froude numbers of $Fr = 6.2$ and 3.4, respectively, can be calculated.

When the width of the structure is reduced from $B = 2.50 \text{ m}$ to $2.00 \text{ m}$ or to $1.50 \text{ m}$, the Froude number at floor depth of $-4.50 \text{ m}$ will amount to $Fr = 7.6$ and 6.5, respectively.

### 12.2.2. Hydraulic jump

**Hydraulics.** The super-critical flow ($Fr > 1$) is transferred to the sub-critical flow ($Fr < 1$) through a hydraulic jump, see figure 12.9. The hydraulic jump is formed when the tailwater depth $y_3$ is sufficient deep, thus $y_3 > y_{\text{min}}$.

The minimum tailwater depth $y_{\text{min}}$, or the 'conjugate depth', depends on the discharge $q$ per unit width in $\text{m}^3/\text{s}.\text{m}$, and the Froude number $Fr_j$ and the water depth $y_j$ of the jet before the jump:

$$y_{\text{min}} = \frac{3}{2} y_j (\sqrt{1 + 8 Fr_j^2} - 1), \quad \text{with: } Fr_j = \frac{q}{\sqrt{g \ y_j^3}} \quad \text{or: } Fr_j = \frac{v_j}{\sqrt{g \ y_j}}$$

The actual length $L_j$ of an 'undisturbed hydraulic jump' is somewhat ill-defined, but can be estimated by the formula:

$$L_j = 6.9 \ (y_{\text{min}} - y_j) \approx 6.1 \ y_3$$
**Figure 12.9.** Parameters of the hydraulic jump formula.

**Analytical formula.** The **minimum stilling basin depth** $y_{\text{min}}$ to form a hydraulic jump can be calculated analytically, see also box 12.1 and figure 12.10. Thus, the simple formula to check if any hydraulic jump will occur in a stilling basin with water depth $y_{\text{min}}$ is:

$$y_{\text{min}} = 1.04 \cdot q^{0.526} \cdot z^{0.21}$$

where $q$ is the discharge per unit width in m$^3$/s.m, $z$ is the energy drop between up- and downstream in m, and $y_{\text{min}}$ is the minimum water depth in stilling basin required for a hydraulic jump.

The **value of the formula** is the direct calculation of the conjugate water depth $y_{\text{min}}$ without a cumbersome iteration process. It is obvious that this formula cannot be used for the design of stilling basins. Such a stilling basin would create a long hydraulic jump at a water slope of approx. $1_{\text{vert}} : 7_{\text{hor}}$, and leads to an expensive basin. Its length can be reduced considerably by an over-depth and/or baffle-blocks and an effective end-sill.

**Figure 12.10.** Minimum water depth $y_{\text{min}}$ for a hydraulic jump.
Box 12.2. Analytical derivation of the conjugate depth $y_{\text{min}}$ for a hydraulic jump.

The minimum basin depth $y_{\text{min}}$ can be written as a function of the drop $z$ between the upstream energy level and the downstream water level, and as a function of the discharge $q$ per unit width, in m$^3$/s.m.

The conversion is done by using an auxiliary broad-crested weir. The approaching energy level is a distance $H_a$ above this crest, the stilling basin is a distance $D_o$ below this crest. Thus, there are four basic equations:

- the upstream energy level above the auxiliary weir:
  
  $H_a = (g / 1.7)^{2/3}$,

- the upstream energy level above the floor of the stilling basin, at the end of the jump:
  
  $D_o + H_a = y_{\text{min}} + z$,

- the upstream energy level above the floor of the stilling basin, at the beginning of the jump, where $y_j$ is the water depth of the incoming jet:
  
  $D_o + H_a = y_j + (q/y_j)^2 / 2g$,

- the hydraulic jump depends on the water depth $y_{\text{min}}$ in stilling basin, the Froude number $Fr_b$, and the water depth $y_j$ of the incoming jet:
  
  $y_j = \frac{1}{2} y_{\text{min}} (1 + 8 Fr_b^2)^{3/2} - \frac{1}{2} y_{\text{min}^2}$, with $Fr_b^2 = q^2 / (g y_{\text{min}}^3)$

The depth $D_o = y_{\text{min}} + z - H_a$ can be calculated from the above equations, by using three other parameters $p$, $r$, and $x$:

\begin{align*}
  p &= y_j / H_a, \\
  r &= y_{\text{min}} / H_a, \\
  x &= z / H_a.
\end{align*}

Elimination of the unknown parameters leads to two equations in $p$, $r$, and $x$:

- $Fr_b^2 = q^2 / (g y_{\text{min}}^3) = g (2/3 H_a)^3 / (g y_{\text{min}}^3) = (2/3)^3 / r^2 = 0.30 / r^2$ so that: $p = \frac{1}{2} r \sqrt{(1 + 8 \times 0.30 / r^2)} - \frac{1}{2} r$

- $D_o + H_a = y_j = \left(\sqrt{2/3 H_a}^{3/2} / y_j^2\right)^2 / 2g$, or: $D_o / H_a = p - 1 + 0.15 / p^2$

while: $D_o / H_a = x + r - 1$, so that: $x = p - r + 0.15/p^2$

Based on these two equations, the relation between $r$ and $x$ can be found by assuming a value of $r$ and calculating the corresponding value $x$.

The data $r$ and $x$ have been plotted in figure 12.10, and the best-fit curve through these points has the equation $r = 1.38 x^{0.21}$. Thus, the minimum water depth in the stilling basin at which the jump will occur can be calculated by: $y_{\text{min}} / H_a = 1.38 (z / H_a)^{0.21}$.

As $q = 1.7 H_a^{1.5}$, this equation can be rewritten into:

\begin{align*}
  y_{\text{min}} &= 1.38 H_a^{0.79} (z / H_a)^{0.21} = 1.38 (q/1.7)^{0.79 \times 2/3} (z / H_a)^{0.21} = 1.04 q^{0.526} z^{0.21}.
\end{align*}

12.2.3. Standard Stilling Basins

USBR type I basin. The hydraulic formulae as discussed above, lead to the design of a stilling basin for the 'undisturbed-jump'. Such a basin has a basin depth $y_{\text{basin}}$ that equals the conjugate depth $y_{\text{conj}}$. USBR studied the undisturbed jump basin as part of their series of model tests on basins, and denoted this basin as the 'USBR type I basin' (Peterka 1974).

The undisturbed-jump or USBR type I basin would create the hydraulic jump, which is quite long at a water slope of approx. 1 Vert : 7 Hor, and leads to an expensive basin, see figure 12.11. Its length can be reduced considerably by an over-depth and/or baffle-blocks and an effective end-sill.
Figure 12.11. The principle of the 'undisturbed-jump' basin (USBR type I basin).

Improvement of efficiency. The energy dissipation in a stilling basin is improved by one or more of the following measures, see figure 12.12:

- an extra over-depth of the floor,
- an end-sill,
- vertical drop, by impact of the jet on the floor,
- chute blocks, to spread the inflowing jet,
- baffle blocks, to dissipate energy of the jet by impact,

Thus, different standard basins have been developed, such as the USBR type III basin, USBR type IV basin, the SAF basin, the Bucket basin, the Vlugter basin, and the vertical drop basin.

Moreover, the width of the whole structure can be increased, as to increase the Froude number $Fr$ of the inflowing jet by a lower unit discharge $q$.

Figure 12.12. Types of standard stilling basins.
12.3. Types of Stilling Basins

12.3.1. Stilling Basins with baffle blocks

**USBR type III.** The **USBR standard basin type III**, see figure 12.13, is suitable for Froude numbers $F_{r_1} > 4.5$. The water depth in the basin equals the conjugate depth $y_{\text{min}}$, needed for an unaided hydraulic jump. However, the length of the USBR type III basin is considerably less by the use of the blocks.

The chutes blocks, baffle blocks and the end-sill along the floor of the basin produce a stabilizing effect on the jump, which permits shortening of the basin. The chute blocks are triangulate blocks that split and aerate ("beluchten") the jet.

The original design formulae for the USBR type III basin are related to the hydraulic testing results (e.g. USBR 1973, Chow 1959, Henderson 1966) and are less practical for the design engineer.

The design of the USBR type III basin can be simplified (Ankum 1991) by expressing the formulae into a design parameter $H_a = (q / 1.7)^{2/3}$. Thus, the depth $y_b$ of the floor below the tailwater level and the length $L_b$ of the floor follows from:

$$\frac{y_b}{H_a} = 1.38 \left( \frac{z}{H_a} \right)^{0.21}$$

$$\frac{L_b}{H_a} = 3.7 \left( \frac{z}{H_a} \right)^{0.21}$$

with a drop $z$ between the upstream and downstream energy level, and a discharge $q$ per unit width in m$^3$/s.m. The USBR type III basin may only be applied for the ratio $z/H_a > 1.2$.

![Figure 12.13. The principle of the USBR type III basin.](image)

**USBR type IV.** The **USBR standard basin type IV**, see figure 12.14, has been developed for Froude numbers $2.5 < F_{r_1} < 4.5$, when a true hydraulic jump does not fully develop and waves may persist beyond the end of the basin. The sole function of the USBR type IV basin is 'wave suppression' ("golfdemping").

The water depth $y_b$ of the basin is $\pm 10\%$ greater than the minimum depth $y_{\text{min}}$, as required for the unaided hydraulic jump. The length $L_b$ of the stilling basin equals the length of the unaided hydraulic jump. The basin is equipped with large chute-type blocks that help to intensify the roller of the hydraulic jump.

The original design formulae for the USBR type IV are related to the hydraulic testing results (e.g. USBR 1973, Chow 1959, Henderson 1966) and are less practical for the design engineer.
The design for the USBR type IV can be simplified (Ankum 1991) by expressing the formulae into a design parameter \( H_a = (q / 1.7)^{2/3} \). Thus, the depth \( y_b \) of the floor below the tailwater level and the length \( L_b \) of the floor follows from:

\[
\frac{y_b}{H_a} = 1.52 \left( \frac{z}{H_a} \right)^{0.21} \\
\frac{L_b}{H_a} = 9.3 \left( \frac{z}{H_a} \right)^{0.21}
\]

with a drop \( z \) between the upstream and downstream energy level, and a discharge \( q \) per unit width in m³/s.m. The USBR type IV basin may only be applied for the ratio \( 0.30 < z/H_a < 1.2 \).

![Diagram of USBR type IV basin](image)

**Figure 12.14.** The principle of the USBR type IV basin.

### SAF basin

The **SAF basin** has been developed at the St. Anthony Falls (SAF) Hydraulic Laboratory, USA, in the 1940s. The basin is recommended where Froude numbers are in the range \( 1.7 < Fr < 17 \). The principle of the SAF basin is presented in figure 12.15.

The SAF basin is a shallow basin of an extreme shortness. For instance at \( Fr = 9 \), the length of SAF basin is \( L_b = 0.87 y_{\text{min}} \) against \( L_b = 2.7 y_{\text{min}} \) for the USBR type III basin, and against \( L_b = 6.1 y_{\text{min}} \) for the unaided jump. The depth of this SAF basin is \( y_b = 0.85 y_{\text{min}} \) against \( y_b = y_{\text{min}} \) for the USBR type III basin and for the unaided jump.

The original design formulae for the SAF basin are related to the hydraulic testing results (e.g. USBR 1973, Chow 1959, Henderson 1966) and are less practical for the design engineer.

The design of the SAF type can be simplified (Ankum 1991) by expressing the formulae into a design parameter \( H_a = (q / 1.7)^{2/3} \). Thus, the depth \( y_b \) of the floor below the tailwater level and the length \( L_b \) of the floor follows from:

- for \( 0.15 < z/H_a < 2.0 \):
  \[
  \frac{y_b}{H_a} = 1.33 \left( \frac{z}{H_a} \right)^{0.14} \quad \text{and} \quad \frac{L_b}{H_a} = 2.2 \left( \frac{z}{H_a} \right)^{-0.12}
  \]
- for \( 2.0 < z/H_a < 6.5 \):
  \[
  \frac{y_b}{H_a} = 1.17 \left( \frac{z}{H_a} \right)^{0.21} \quad \text{and} \quad \frac{L_b}{H_a} = 2.4 \left( \frac{z}{H_a} \right)^{-0.25}
  \]
- for \( 6.5 < z/H_a < 13.0 \):
  \[
  \frac{y_b}{H_a} = 2.60 \left( \frac{z}{H_a} \right)^{-0.20} \quad \text{and} \quad \frac{L_b}{H_a} = 2.4 \left( \frac{z}{H_a} \right)^{-0.25}
  \]

with a drop \( z \) between the upstream and downstream energy level, and a discharge \( q \) per unit width in m³/s.m. The SAF type basin may only be applied for the ratio \( 0.15 < z/H_a < 13 \).
12.3.2. Bucket-type stilling basins

**Bucket basin.** The 'bucket basin' is a deep basin, for which the water depth in the basin is larger than the minimum depth for a hydraulic jump, see figure 12.16.

The energy dissipation depends not on the formation of a hydraulic jump but on the formation of two rollers: a bucket roller on the surface ("oppervlakte neer") and a ground roller ("bodemneer"). The intermingling movements of these two rollers are effectively dissipating the energy.

A minimum water depth in the basin is required to avoid the sweeping out of the surface roller. A maximum water level is required to form a proper ground roller, to avoid the diving of the flow.

The bucket basin has been model tested by USBR, and data have been presented in the form of curves in relation of the Froude number $Fr_z$, whereas the Froude number $Fr_z$ refers to the flow at the point where incoming jet $y_z$ enters the bucket, at the level of the tailwater surface (e.g. USBR 1973, Chow 1959, Henderson 1966).

The Froude number $Fr_z$ of the incoming jet at the level of the tailwater surface, is not a practical parameter in the design, and can be avoided by expressing the formulae into a design parameter $H_a = (q / 1.7)^{2/3}$. The design of the bucket basin involves determination of the radius $R$ of the curvation of the bucket, and the floor depth $y_b$ below the tailwater level which follow directly from the formulae (Ankum 1991):

\[
\frac{R}{H_a} = 0.93 + 0.113 \left( \frac{z}{H_a} \right) - 0.019 \left( \frac{z}{H_a} \right)^2
\]

\[
\frac{y_b}{H_a} = 2.0 + 0.1 \frac{z}{H_a}
\]

with a drop $z$ between the upstream and downstream energy level, and a discharge $q$ per unit width in m$^3$/s.m. The bucket basin may only be applied for the ratio $1.5 < z/H_a < 7$.

**Vlugter basin.** The **Vlugter basin** has been developed in Indonesia by Vlugter in the early 1940s (Vlugter 1941). The Vlugter basin looks like the bucket basin, but has a horizontal floor. The stilling basin is relative deep and short, and has no blocks but an end-sill, see figure 12.16. The glacis ("achtervlak") has a slope of 45°, and is transferred into a curve with a radius $R$ before reaching the stilling basin.

The model tests as performed by Vlugter provided a series of design formulae, which are based on the design parameter $H_a = (q / 1.7)^{2/3}$. The depth $y_b$ of the floor below the
tailwater level, the length $L_b$ of the floor, the radius $R$ between the glacis and the floor, and the minimum height $a$ of the end-sill above the floor follows from:

$$\frac{y_b}{H_a} = 2.0 + 0.1 \frac{z}{H_a}$$

$$\frac{L_b}{H_a} = \frac{R}{H_a} = 1.0 + 1.1 \frac{z}{H_a}$$

$$\frac{a}{H_a} = 0.15 \left( \frac{z}{H_a} \right)^{0.5}$$

with a drop $z$ between the upstream and downstream energy level, and a discharge $q$ per unit width in $m^3/s$. The Vlugter basin may only be applied for the ratio $0.50 < z/H_a < 5$. Box 12.3 presents an example calculation of a drop structure with a Vlugter basin.

**Figure 12.16.** Bucket-type basins (for $z/H_a = 3$).

**Box 12.3.** Example calculation of a Vlugter basin.

A drop structure with a water level drop of $z = 1.50$ m have to designed in a drain with a design capacity of $Q = 2.2$ m$^3$/s. The drain characteristics are: bed width $b_{\text{drain}} = 1.75$ m, side slopes $I_{\text{Vert}} : I_{\text{Hor}} = 1.5:1$ and a design water depth of $y = 1.16$ m. The drop structure will be equipped with a vertical gate of a width $b_{\text{gate}} = 1.50$ m.

It is decided that the stilling basin will be a Vlugter basin of a width of $b_{\text{basin}} = 1.75$ m. The design parameter $H_a$ depends on the discharge per unit width, which is selected here at the gate width $b_{\text{gate}} = 1.50$ m. Hence, $H_a = (Q/1.7b)^{2/3} = (2.2 / 1.7 \times 1.50)^{2/3} = 0.91$ m. Thus, $z/H_a = 1.50/0.91 = 1.66$.

The design dimensions of the basin are:

- radius $R = 1.1 \times z + H_a = 1.1 \times 1.50 + 0.91 = 2.56$ m $= 2.60$ m,
- length of the basin $L_b = 1.1 \times z + H_a = 2.60$ m,
- depth of basin $y_b = 0.1 \times z + 2.0 \times H_a = 0.1 \times 1.50 + 2 \times 0.91 = 1.97$ m $= 2.00$ m,
- height of end-sill $a = 0.15 \times H_a \sqrt{(H_a/z)} = 0.15 \times 0.91 \times \sqrt{(0.91/150)} = 0.11 \approx 0.15$ m.
12.3.3. Miscellaneous stilling basins

Volume basin. Volume basins are recommended for downstream control gates (AVIO and AVIS) to dissipate the excessive energy. Volume basins are rectangular basins of sufficient volume, see figure 12.17.

The volume \( V \) of the volume basins depend on the energy to be dissipated, and are proportional with the product of the design discharge \( Q \) and the (energy) headloss \( z \). Thus (Alstom undated): \( L \times b \times y_b = 21.2 \times Q \times z^{0.5} \). Furthermore, the optimum proportions for the length \( L \), the width \( b \) and the water depth \( y_b \) of the volume basins are: \( L \approx 3 \times b \approx 4.5 \times y_b \).

The original design formulae for the volume basin can be simplified (Ankum 1991) by expressing the formulae into a design parameter \( H_a = (q / 1.7)^{2/3} \). So, the depth \( y_b \) and the length \( L_b \) of the volume basin follows from:

\[
\frac{y_b}{H_a} = 2.93 \left( \frac{z}{H_a} \right)^{0.25} \\
\frac{L_b}{H_a} = 13.2 \left( \frac{z}{H_a} \right)^{0.25}
\]

with a drop \( z \) between the upstream and downstream energy level, and a discharge \( q \) per unit width in \( \text{m}^3/\text{s}.\text{m} \). The range of \( z/H_a \) for which the volume basin may be used has not been specified.

![Figure 12.17. The hydraulic dimensions of the Volume basin.](image-url)
**Vertical-drop basin.** Vertical-drop basins are characterized by a free-falling jet into the basin. The free falling jet makes an impact ("botting") with the basin floor and is turned into the downstream direction, see figure 12.18. The basin is equipped with an end-sill ("einddrempel").

Upto 50\% of the energy may be dissipated by the impact of the jet and by the turbulent circulation in the pool beneath the jet. The remainder part is dissipated by the hydraulic jump in the basin.

![Diagram of a vertical-drop basin](image)

**Figure 12.18.** Straight drop structure with a "vertical-drop" basin.

Although the vertical-drop basin seems to be an efficient energy dissipator because of the impact, the basin becomes very long in comparison with other stilling basins. The stilling basin should not only provide for the hydraulic jump, but should also accommodate the horizontal distance \( L_d \) covered by the dropping jet, which is quite long:

\[
\frac{L_d}{H_a} = 5.14 \left( \frac{z}{H_a} \right)^{0.857}
\]

For example, a discharge per unit width of \( q = 0.5 - 1.0 \text{ m}^3/\text{s} \cdot \text{m} \) dropping over \( z = 1.00 \text{ m} \) requires a horizontal distance \( L_d \) of \( \pm 5.00 \text{ m} \) for the jet to reach the floor!

Modelltests as reported in the literature (e.g. Chow 1959, Henderson 1966), are less practical for the design engineer. These tests provided a series of design formulae, which are based on the 'drop number' \( D = q^2/(g z_b^2) \) and on the drop \( z_b \) in channel bed.

The original design formulae for vertical-drop basin can be simplified (Ankum 1991) by expressing the formulae into a design parameter \( H_a = (q / 1.7)^{2/3} \). Thus, the depth \( y_b \) of the floor below the tailwater level and the length \( L_b \) of the floor follows from:

\[
\frac{y_b}{H_a} = 1.25 \left( \frac{z}{H_a} \right)^{0.179}
\]

\[
\frac{L_b}{H_a} = 11.6 \left( \frac{z}{H_a} \right)^{0.600}
\]

with a drop \( z \) between the upstream and downstream energy level, and a discharge \( q \) per unit width in \( \text{m}^3/\text{s} \cdot \text{m} \). The range of \( z/H_a \) for which the volume basin may be used has not been specified. However, the Froude number \( F_{rj} \) of the jet after the impact with the floor can be derived as well:
\[ Fr_j = 3.27 \left( \frac{z}{H_a} \right)^{0.389} \]

Considering a range for the Froude number of \( Fr_j > 4.5 \), the vertical-drop basin may be applied for \( z/H_a > 2.3 \). Box 12.4 presents an example calculation of a drop structure with a vertical-drop basin.

**Box 12.4. Example calculation of a vertical-drop basin.**

A drop structure with a water level drop of \( z = 1.50 \) m have to designed in a canal with a design capacity of \( Q = 0.8 \) m\(^3\)/s.

It is decided to apply a vertical-drop basin. The width \( b \) of the drop structure is \( b = 1.00 \) m, so that the design parameter \( H_a \), which is the energy depth above an auxiliary broad-crested weir, follows from \( H_a = (Q/1.7b)^{2/3} = (0.8/1.7 \times 1.00)^{2/3} = 0.61 \) m. Thus, \( z/H_a = 1.50/0.61 = 2.46 \).

The design dimensions of the basin are:
- basin depth \( y_b = 1.25 H_a z/H_a^{0.179} = 1.25 \times 0.61 \times (2.46)^{0.179} = 0.90 \) m;
- basin length \( L_b/H_a = 11.6 (z/H_a)^{0.600} = 11.6 (2.46)^{0.600} = 19.91 \). Thus, \( L_b = 19.91 \times 0.61 = 12.14 \approx 12.50 \) m.

### 12.3.4. Selection of stilling basins

**Most simple basin.** Anyhow, it is wise to construct always a stilling basin behind a structure. Supercritical flow may even occur at unexpected circumstances. The cost of a simple basin will be minimal, and it will safeguard the structure during unexpected operation. Such a simple basin can be created by:
- a floor that is slightly lower (e.g. 0.30 m) than the ongoing channel bed;
- the horizontal floor of the basin will be ended by 'end sill';
- a coffer under the end sill.

**Will a jump occur?** The check whether a basin will force a hydraulic jump follows from the formula:
\[ y_{\text{min}} = 1.04 q^{0.526} z^{0.21} \]
where \( q \) is the discharge per unit width in m\(^3\)/s.m, \( z \) is the energy drop between up- and downstream in m, and \( y_{\text{min}} \) is the minimum water depth in stilling basin required for a hydraulic jump.

**When is a basin certainly needed?** The need for a stilling basin follows from, see also box 12.5:
- a basin is required **to force a jump** when the tailwater depth is less than the 'conjugate depth', thus when \( y_3 < y_{\text{min}} \). Utilize the ratio \( z/H_a \) for the selection and the design of the basin;
- a basin is required when **waves** may occur, even if a basin is not required to force a jump (\( y_3 > y_{\text{min}} \)). Thus, the value of the Froude number \( Fr_j \) have to be checked on 'waves'.
Parameter for selection of the basin. There are several types of stilling basins, and each of these basins can be applied in combination with the different control sections, such as weirs, notches and gates. In fact, it is the incoming jet at the floor which determines what type of basin is required.

This incoming jet has a certain flow, \( q \) in \( \text{m}^3/\text{s.m} \), and excess energy, \( z \) in m. Thus, the hydraulics of the incoming jet for the selection of a stilling basin, can be described by either:

- the Froude number \( Fr_j \) of the incoming jet, where \( Fr_j = \frac{q}{(g y_j)^{0.5}} \), where \( q \) is the discharge per unit width in \( \text{m}^3/\text{s.m} \), \( y_j \) is the water depth of the jet and \( g = 9.8 \text{ m/s}^2 \) = 9.8 \( \text{N/kg} \) is the gravity acceleration. The Froude number depends on the energy depth \( H_j \) of the jet;

- the ratio \( z/H_a \), where \( H_a = \left( \frac{q}{1.7} \right)^{2/3} \), \( z \) is the drop between the upstream and downstream energy level.

It appears that the ratio \( z/H_a \) is the most practical parameter in the selection of stilling basins. However, the Froude number \( Fr_j \) is still needed when a check has to be made on waves.

**Box 12.5. Example of an evaluation on the need of a stilling basin.**

**First try.** A gated structure of \( b = 1.50 \text{ m} \) wide and a discharge \( Q = 1.5 \text{ m}^3/\text{s} \) might require a stilling basin. The upstream energy head \( H_j = 1.00 \text{ m} \), the tailwater depth \( y_j = 0.90 \text{ m} \), thus the energy drop \( z \approx 0.10 \text{ m} \).

The discharge \( q \) per unit width is \( q = 1.0 \text{ m}^3/\text{s.m} \), the value of \( H_a = \left( \frac{q}{1.7} \right)^{2/3} = 0.70 \text{ m} \). The water depth \( y_j \) of the (supercritical) jet is calculated by iteration through the equation \( y_j = \frac{q}{[2g \sqrt{(H_j - y_j)^{1.5}}]} = \frac{1.0}{[2g \sqrt{(1.00 - y_j)^{1.5}}]} \) gives \( y_j = 0.23 \text{ m} \), gives \( y_j = 0.26 \text{ m} \), gives finally \( y_j = 0.26 \text{ m} \).

The minimum tailwater depth for a jump is \( y_{min} = 1.04 q^{0.526} z^{0.21} = 1.04 \times 1.0^{0.526} \times 0.10^{0.21} = 0.64 \text{ m} \). Thus, a stilling basin is not required for a hydraulic jump.

The Froude number \( Fr_j \) of the (supercritical) jet \( Fr_j = \frac{q}{(g y_j^{0.5})} = 2.4 \), which is in the "danger" range.

**Second try.** The opening of the structure is made smaller to \( b = 1.00 \text{ m} \) to decrease the Froude number. The upstream energy head remains \( H_j = 1.00 \text{ m} \).

The discharge \( q \) per unit width is now \( q = 1.5 \text{ m}^3/\text{s.m} \), the value of \( H_a = \left( \frac{q}{1.7} \right)^{2/3} = 0.92 \text{ m} \). The water depth \( y_j \) of the (supercritical) jet is calculated by iteration through the equation \( y_j = \frac{q}{[2g \sqrt{(H_j - y_j)^{0.5}}]} = \frac{1.5}{[2g \sqrt{(1.00 - y_j)^{0.5}}]} \) gives finally \( y_j = 0.45 \text{ m} \).

The minimum tailwater depth for a jump is \( y_{min} = 1.04 q^{0.526} z^{0.21} = 1.04 \times 1.5^{0.526} \times 0.10^{0.21} = 0.79 \text{ m} \). Thus, a stilling basin is still not required for a hydraulic jump.

The Froude number \( Fr_j \) of the (supercritical) jet \( Fr_j = \frac{q}{(g y_j^{0.5})} = 1.6 \), which means that a stilling basin is not needed.

**Froude number.** The Froude number \( y_j \) of the jet cannot be determined directly. The water depth \( y_j \) follows from \( y_j + q^2/(y_j^2 \cdot 2g) = H_j \), or:

\[
y_j = \frac{q}{\sqrt{2g (H_j - y_j)}}
\]

The value of \( y_j \) can only be calculated by iteration for a known energy depth \( H_j \) of the jet and a known discharge \( q \) per unit width in \( \text{m}^3/\text{s.m} \): assume \( y_j = 0 \), calculate a new value of \( y_j \)
and insert this value again in the formula, etc., etc.

Based on the Froude Number \( Fr_j \), the following stilling basins are generally recommended, see figure 12.19:

- **\( Fr_j = 1 \).**
  When the Froude number of the incoming jet is equal to \( Fr_j = 1 \), the flow is at critical depth and a hydraulic jump cannot form. A stilling basin is not needed.

- **\( 1.0 < Fr_j < 1.7 \)**
  The incoming jet is only slightly below critical depth, and the change from this low stage to the high stage flow is gradual and manifests itself only by a slight ruffled water surface. No stilling basin is needed, only some protection with rip-rap, or by masonry or concrete lining.

![Diagram of hydraulic jump stages](image)

**Figure 12.19.** Hydraulic jump related to the Froude number.
• 1.7 < Fr<sub>j</sub> < 2.5
A series of small rollers develop on the surface for the Froude number of Fr<sub>j</sub> > 1.7, which become more intense with increasingly higher values of the number.
A stilling basin is needed to dissipate energy effectively. Generally, a basin with an end-sill is needed.

• 2.5 < Fr<sub>j</sub> < 4.5
An oscillating form of jump occurs, the entering jet intermittently flowing near the bottom and then along the surface of the downstream channel.
This range is difficult for selection stilling basins as the hydraulic jump is not well-stabilized and produces waves that travel long distance through the channel. It is better to avoid this range by increase the Froude number through a wider structure. If not possible, the USBR type IV may be used. However, the wave action cannot be entirely dampened.

• 4.5 < Fr<sub>j</sub> < 13
This range requires the most economical basins, because they can be kept short. The USBR type III is recommended, as well as basins with end-sills.

• Fr<sub>j</sub> > 13
When Fr<sub>j</sub> > 13, the jump is very rough, and will make conditions rather difficult to calm the tailwater. The stilling basin will become relatively expensive.
The Froude number Fr<sub>j</sub> of the jet can be changed by another design of the structure:
- a wider structure will decrease discharge q per unit width in m<sup>3</sup>/s.m, and will lead to a higher Froude number Fr<sub>j</sub>;
- a lower floor will increase the energy head H<sub>j</sub> of the jet, and thus will lead to a higher Froude number Fr<sub>j</sub>.

Ratio z/H<sub>a</sub>. The selection of the basin type can be based on the ratio z/H<sub>a</sub>, i.e., the ratio between the drop z in up- and downstream level, and the parameter H<sub>a</sub> = (q / 1.7)<sup>2/3</sup> where q is the discharge per unit width in m<sup>3</sup>/s.m.
The design formulae for the depth y<sub>b</sub> and the length L<sub>b</sub> of the different basins are given in table 12.1, as well as the validity z/H<sub>a</sub> for each of the standard basins.

The comparison of the depth y<sub>b</sub> and the length L<sub>b</sub> of the different basins are presented in figure 12.20 and in figure 12.21.

<table>
<thead>
<tr>
<th>Table 12.1. Selection of stilling basins.</th>
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<tbody>
<tr>
<td>Valid for drop z</td>
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<tr>
<td>------------------</td>
</tr>
<tr>
<td>&quot;UN-AIDED&quot; JUMP</td>
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<tr>
<td>STANDARD BASINS:</td>
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<tr>
<td>• Vertical-drop basin</td>
</tr>
<tr>
<td>• USBR-basin type III</td>
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<tr>
<td>• USBR-basin type IV</td>
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<tr>
<td>• SAF-basin</td>
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<tr>
<td>• SAF-basin</td>
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<tr>
<td>• Bucket basin</td>
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<tr>
<td>• Sluiter basin</td>
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<tr>
<td>• Volume basin</td>
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</tbody>
</table>

* H<sub>a</sub> is a design parameter, which follows from H<sub>a</sub> = (q / 1.7)<sup>2/3</sup>, and q = Q / B.
Figure 12.20. Depths $y_b$ of stilling basins.

Figure 12.21. Lengths $L_b$ of stilling basins.
Other criteria. Other criteria for the ultimate selection of stilling basins may concern the constructional matters.

The USBR and the SAF basins are attractive from the hydraulic point of view. The SAF basin has a large range of validity, and is very short and shallow. The USBR type IV performs well for small Froude numbers. The USBR type III basin is shorter than the unaided jump. However, these basins require a precise construction work for the chute-blocks and baffle-blocks.

The bucket basin and the Vlugter basin are simple to construct as they do not have chute blocks and impact blocks. The Vlugter basin is quite deep, which may be difficult for construction in areas with high groundwater tables and pervious soils. Moreover, the Vlugter basin has shown to be unreliable with tailwater levels above and below the laboratory tested levels, and is therefore not recommended with fluctuating discharges as with weirs in rivers.

The volume basin and the vertical-drop basin require quite large dimensions. The vertical-drop basin seems easy to construct, but has to be well checked on uplift of the floor by groundwater pressure.
13. REFERENCES


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