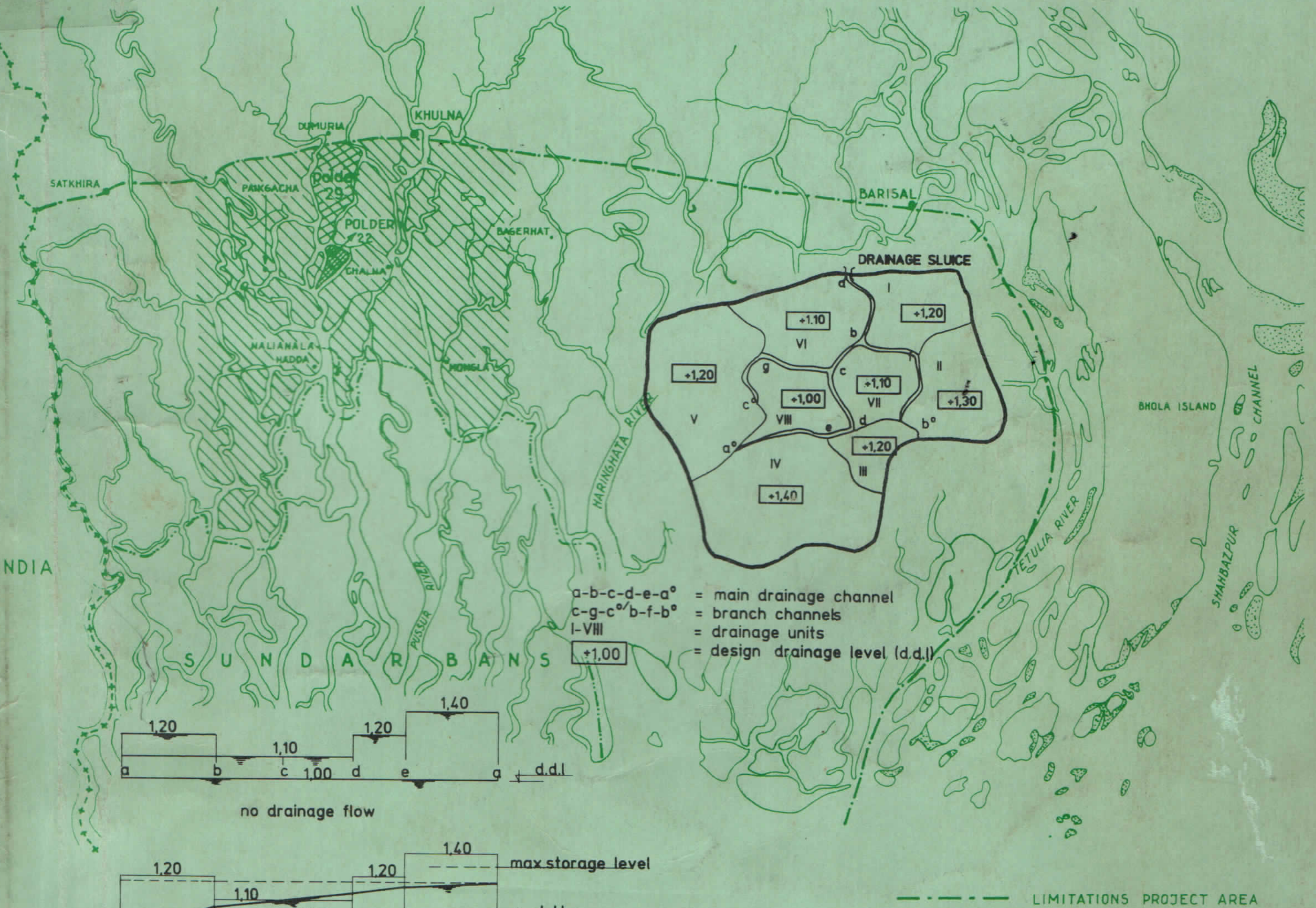
DESIGN MANUAL

FOR POLDERS IN SOUTH-WEST BANGLADESH

Mr A. K. ...



PART 2, VOL. V-VII

DELTA DEVELOPMENT PROJECT
BANGLADESH-NETHERLANDS JOINT PROGRAMME
UNDER BWDB

DHAKA
NOVEMBER 1985

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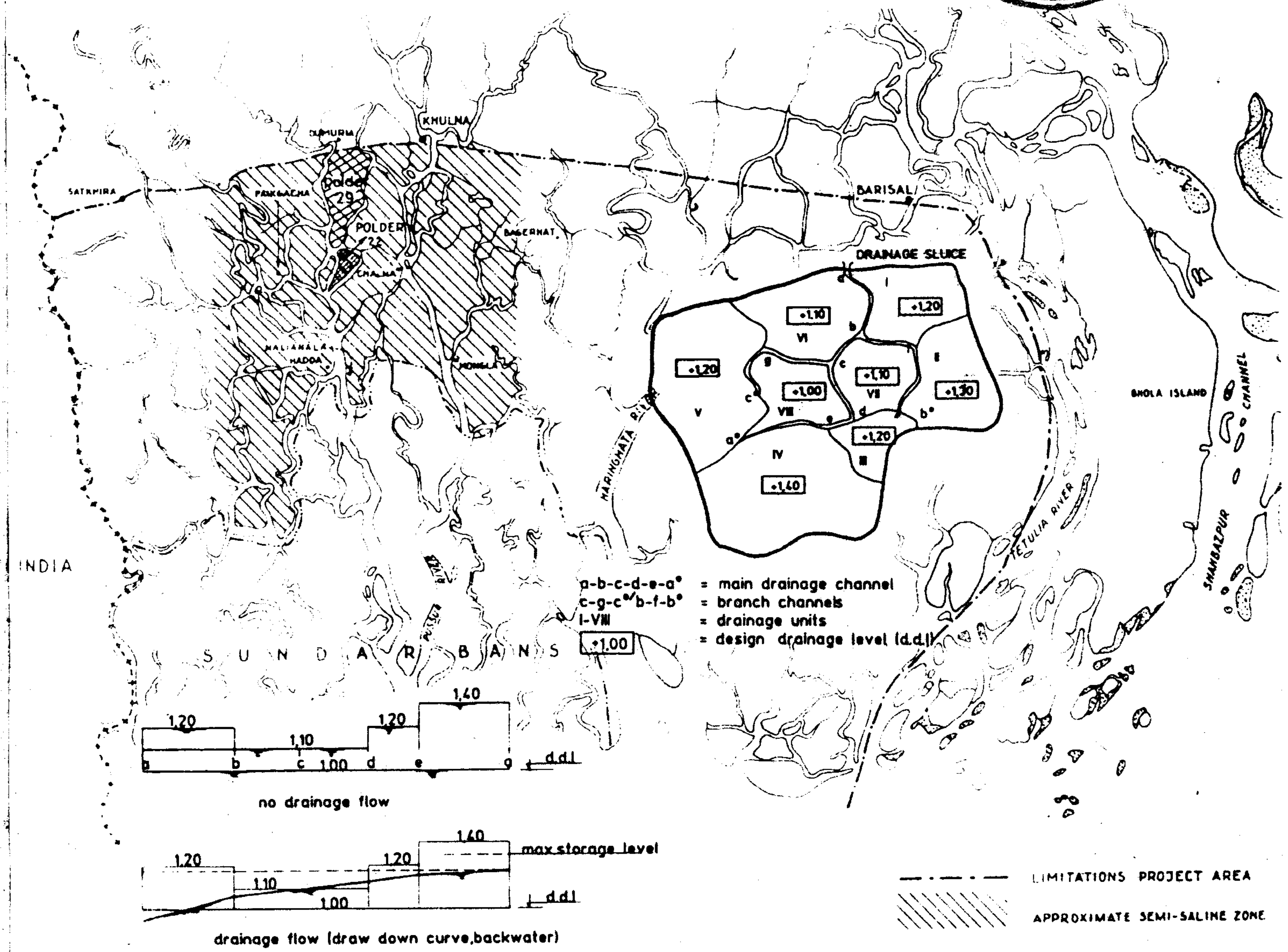

PART 2, VOL. V-VII

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THIS DESIGN MANUAL CONSISTS OF THE FOLLOWING PARTS AND VOLUMES :

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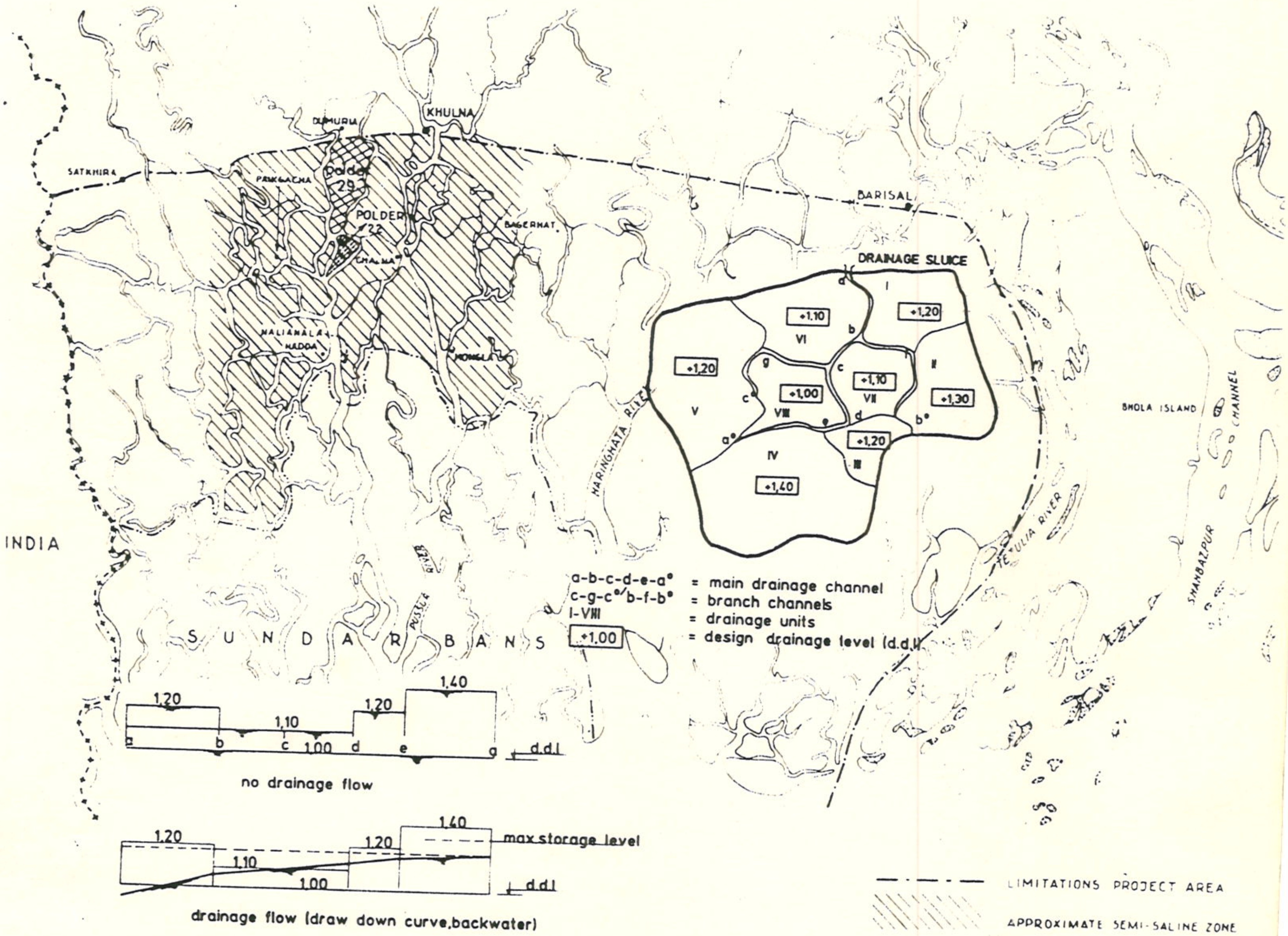

VOL. V

DELTA DEVELOPMENT PROJECT
BANGLADESH-NETHERLANDS JOINT PROGRAMME
UNDER BWDB

DHAKA
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VOLUME V HYDRAULIC COMPUTATIONS

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Chapter 1. Flow through natural and excavated channels

1.1 Manning formula for uniform flow

The flow through channels and pipes depends on the hydraulic gradient (the driving force) and the friction losses which depend on the wall roughness and the dimensions of the channel. The best known and most widely used formula for uniform flow is the Manning formula. It is very suitable for use with pocket calculators that are provided with exponential functions.

$$v = \frac{1}{n} . R^{2/3} . S^{\frac{1}{2}} \quad \text{(for the metric system)}$$

$$v = \frac{1.49}{n} . R^{2/3} . S^{\frac{1}{2}} \quad \text{(for foot-units)}$$

In this manual the metric system will be adhered.

v = average velocity (m/s)

n = Manning's roughness coefficient ($s/m^{1/3}$)

R = hydraulic radius of the cross-section (m).
R is equal to cross-sectional area A divided by wetted perimeter 0.

S = energy loss or hydraulic gradient.

Sometimes the roughness coefficient k of Strickler is used; the relation between n and k is : $\frac{1}{n} = k$

The discharge of uniform flow in a channel or pipe may be expressed as :

$$Q = v.A \quad \text{or,}$$

$$Q = \frac{1}{n} . R^{2/3} . A . S^{\frac{1}{2}}$$

wherein Q = discharge (m^3/s)

A = cross-sectional area of flow (m^2)

1.2 Roughness coefficient

The n-value depends on a number of factors, viz. roughness of bed and side slopes, vegetation, channel irregularity and alignment, hydraulic radius, and obstructions in the channel. Values for the roughness coefficient are given in ANNEX V - 1. For more details reference is made to Ven te Chow (ref. 2).

Cowan (1956) recognized that several factors were affecting the n-value and he proposed that the n-value be computed with :

$$n = (n_o + n_1 + n_2 + n_3 + n_4)\, m_5$$

This method can be applied to drainage channels with a hydraulic radius of less than 4.50 m. The proper values of n_o to n_4 and m_5 for the given conditions can be obtained from Table V-1.1.

Table V - 1.1 Channel characteristics and corresponding n-values

Channel condition		n-values
material involved	earth	n_o=0.020
Degree of irregularity of cross-section (sand bars, eroding shores, holes and bumbs)	Smooth	n_1=0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
Variations of channel cross section (shallow-wide to deep-narrow)	Gradual	n_2=0.000
	Alternating occasionally	0.005
	Alternating frequently	0.010-0.015
Relative effect of obstructions like trees, nets, screens, sills	Negligible	n_3=0.000
	Minor	0.010-0.015
	Appreciable	0.020-0.030
	Severe	0.040-0.060
Vegetation	Low	n_4=0.005-0.010
	Medium	0.010-0.025
	High	0.025-0.050
	Very high	0.050-0.100

Table V - 1.1 Channel characteristics and corresponding n-values (continued)

Channel condition		n-values
Degree of meandering	Minor	m_5=1.000
	Appreciable	1.150
	Severe	1.300

It is no easy matter to decide which n-value to select in a given situation and some field experience is required before an n - value can be estimated.

For natural channels in the Bangladesh delta area an n-value of around 0.035 is commonly used.

1.3 Cross sections

1.3.1 General

Most drainage and irrigation channels have a trapezoidal cross-section or have a profile which can be approximated by a trapezoidal section.

The geometry of the trapezoidal cross-section can be described by : (see Figure V - 1.1).

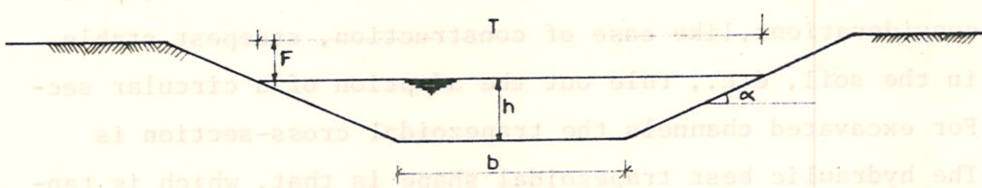


Figure V - 1.1 Trapezoidal channel cross-section.

area : $A = h\ (b + z.h)$

wet circumference : $0 = b + 2h \sqrt{1 + z^2}$

hydraulic radius : $R = \frac{A}{0}$

in which : b = bottom width

h = water depth

α = side slope angle

z = $\cot \alpha$ = side slope

T = topwidth = $b + 2zh$

In most cases an existing natural channel will be used for drainage and/or irrigation purposes. The cross-sections are measured in the field. From the plotted cross-sections the cross-sectional area A and the wetted perimeter O can be measured and the hydraulic radius $R = \frac{A}{O}$ can be computed. With the Manning formula it should be checked if the existing cross-section is sufficient or has to be enlarged. The matter is further elaborated in Chapter 2 of this Volume.

If a new channel has to be made or an existing channel has to be enlarged, aspects as mentioned in the following paragraphs have to be taken into consideration.

1.3.2 Most efficient cross-section

The most efficient cross-section is to be determined considering not only hydraulic but also economic and practical aspects. For a channel having a given roughness n and slope S, which has to carry a certain discharge Q, the best hydraulic section is the section that gives the minimum area A. The best of all possible sections is the circle, or, for channels, the semi-circle. However, practical considerations, like ease of construction, steepest stable slope in the soil, etc., rule out the adaption of a circular section. For excavated channels the trapezoidal cross-section is used. The hydraulic best trapezoidal shape is that, which is tangent to a semi-circle; this is half of a hexagon. For other side slopes, it can be shown that the best hydraulic trapezoidal shape is a trapezoid which is tangent to a semi-circle having its centre on the water surface (see Figure V - 1.2).

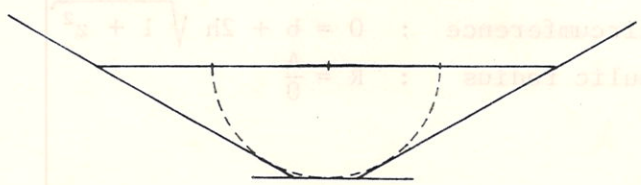


Figure V - 1.2 Channel section of optimal shape.

This means that the ratio of the bottom width b and the water depth h should follow the equation

$$\frac{b}{h} = 2\left(\sqrt{z^2 + 1} - z\right) \qquad (z = \cot\alpha\ ;\ \alpha = \text{side slope}).$$

For a side slope z is 1, 1.5 and 2 this ratio for the best hydraulic trapezoidal cross-section equals 0.83, 0.61 and 0.47 respectively.

The best hydraulic section, however is not always the most economic cross-section. The area A is only the cross-sectional area of flow, the total volume of excavation includes over-excavation for free-board as well and so a minimum value of A need not imply a minimum total excavation. Moreover, considerations such as ease of access and disposal of excavated earth may be more important.

A deep cross-section gives more risk of collapsing side slopes and with variations in the discharge the fluctuations in the water depth are greater.

For the hand-dug canals the most economic cross-section will have a $\frac{b}{h}$ ratio which will be higher than that of the best hydraulic section. Also in low lying areas with high ground water tables the $\frac{b}{h}$ ratio will increase. For drainage channels in these areas the following $\frac{b}{h}$ value as used in the Netherlands are recommended :

$b/h = 1$	for $h < 0.50$ m
$b/h = 2$	for $0.5 < h < 0.75$ m
$b/h = 2.5$	for $0.75\text{ m} < h < 1.0$ m
$b/h = 3$	for $h > 1.0$ m

1.3.3 Side slopes

Steep slopes give minimum land loss but run the risk of collapsing, especially after heavy rainfall or in case of a sudden lowering of the water table of the channel. The side slopes of a canal depend mainly on the kind of material. Table V-1.2 gives a general idea of the suitable side slopes for channels built in various kinds of materials.

Table V - 1.2 : Suitable side slopes

Material	side slopes (vertical/horizontal)
Muck and peat soil	4:1
Stiff clay or earth with concrete lining	2:1 to 1:1
Earth with stone lining or earth for large channels	1:1
Firm clay or earth for small ditches	1:1½
Loose sandy earth	1:2
Sandy loam or porous clay	1:3

1.3.4 Longitudinal slopes

The longitudinal or bottom slope is generally governed by the topography and the energy head available for the flow of water. This will be further elaborated in Chapter 2.

Figure V - 1.3 presents the definitions.

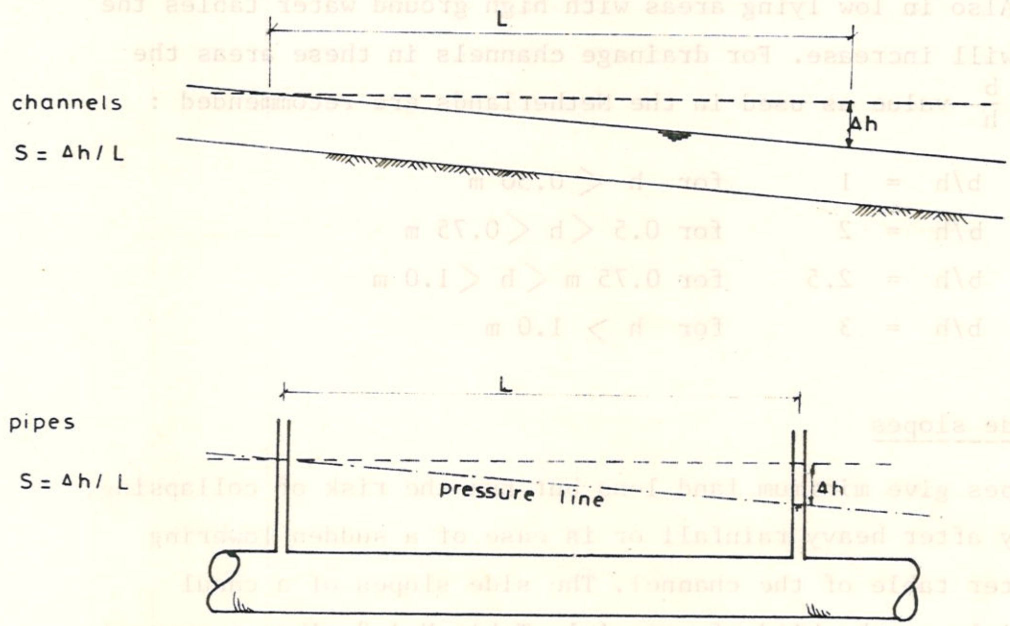


Figure V - 1.3 Hydraulic gradients for channels and pipes.

1.3.5 Free board

The free board (F) of a channel is the vertical distance between the top of the channel embankment and the water surface at the design level. (see Figure V - 1.1). This distance should be sufficient to prevent waves from overtopping the sides. Generally larger freeboards are recommended for canals carrying larger discharges. The following freeboards are recommended for irrigation channels :

Q, (m^3/s)	<0.75	0.75 - 1.5	1.5 - 85
F (m)	0.45	0.60	0.75

These are somewhat less than the freeboard as recommended by the U.S. Bureau of Reclamation:

$$F = C\sqrt{h}$$

in which C = 0.8 for channels upto 0.5 m^3/s, further increasing to 1.35 for channels of 80 m^3/s capacity.

In case of irrigation of rice only, the freeboard can even be reduced more, especially for the smaller channels, because the damage due to overtopping or flooding in case of a breach is less severe. For the low and flat deltaic areas a freeboard of only 0.25 m is suggested for the smaller channels.

1.4 Maximum velocities

Although hardly applicable in the deltaic area it may be mentioned that too high velocities may cause erosion of the channel slopes. The maximum permissible velocities recommended by the American Society of Civil Engineering for straight earthen canals of small slopes and after aging, are given in Table V - 1.3.

Table V - 1.3 Maximum permissible non-eroding velocity (m/s)

Material	Clean water	Water transporting colloidal material
Fine sand, non-colloidal	0.45	0.70
Sandy loam, non-colloidal	0.55	0.70
Silt loam, non-colloidal	0.60	0.90
Alluvial silts, non-colloidal	0.60	1.05
Ordinary firm loam	0.70	1.05
Stiff clay, very colloidal	1.15	1.50
alluvial silt, colloidal	1.15	1.50

In winding channels; the maximum non-eroding velocities are lower; the following reductions are proposed:

slightly winding	5%
moderately winding	13%
strongly winding	22%

If side slopes are protected by vegetal cover, higher maximum velocities are permitted, see Table V - 1.4.

Table V - 1.4 : Permissible velocities (m/s) for channels lined with grass

Cover	side slope range %	erosion resistant soils	easily eroded soils
Bermuda grass	0-5	2.4	1.8
	5-10	2.1	1.5
Annuals	0-5	1.0	0.7

1.5 The minimum permissible velocity

The minimum permissible velocity, or the non-silting velocity, is the lowest velocity that will not induce sedimentation. Its exact value cannot be easily determined.

Since in the deltaic area the hydraulic gradient varies with the fluctuating water levels outside and inside the polders, it is practically not possible to apply the minimum permissible velocity in the hydraulic design of an irrigation or drainage system in these areas.

For completion sake some formulas are mentioned below :

Kennedy formula : $v_{min} = C.\ h^{0.64}$

in which

v_{min} = minimum permissible velocity (m/s)

h = water depth channel (m)

C = coefficient which varies according to the nature of the suspended material (see Table V - 1.5).

Table V - 1.5 Values of C in Kennedy's formula

Nature of suspended material	C
Light loams or very fine sand	0.40
Medium sand (0.4 mm diameter)	0.55

Steevenz formula : $v_{min} = 0.41\ Q^{0.225}$ in which Q is capacity of irrigation canal in m^3/s.

Lacey formulas : $\frac{A}{R} = 4.8\sqrt{Q}$, and $S_{min} = 0.00031\ \frac{f^{5/3}}{Q^{1/6}}$

in which f is a silt factor for which Lacey introduced the relation $f = 12.75\sqrt{d}$ where d is the diameter in centimeters of the predominant type of sediment transported.

1.6 Back water curves

If the flow in a channel is obstructed by a weir, a culvert or another structure, the water surface upstream is raised over a certain distance in the so-called backwater curve (see Figure V - 1.4). For a regular canal with uniform slope S the backwater curve can be approached by the empirical parabolic relationship :

$$\Delta_x = \frac{[(x.S) - 2\cdot\Delta_o]^2}{4\Delta_o} \quad \text{or} \quad \frac{\Delta_x}{h} = \frac{[\,(x.S/h) - (2\Delta_o/h)\,]^2}{4\Delta_o/h}$$

in which : x = distance upstream, (m)

Δ_x = raise in water level at distance x, (m)

Δ_o = raise in water level at distance x = o, (m)

S = bottom slope,

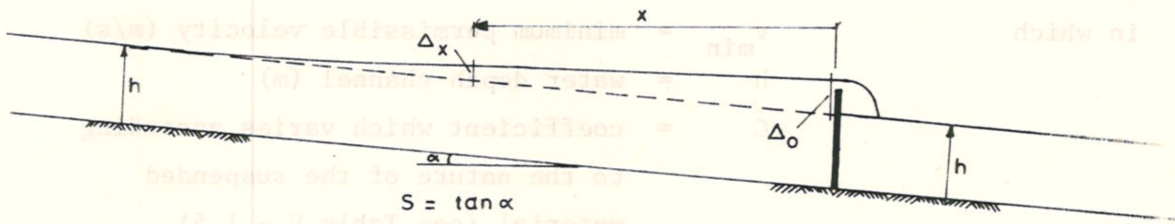


Figure V - 1.4 Backwater curve.

At the distance $x = 2\Delta_o/S$ the backwater effect (raise in water level Δ_x) is zero.

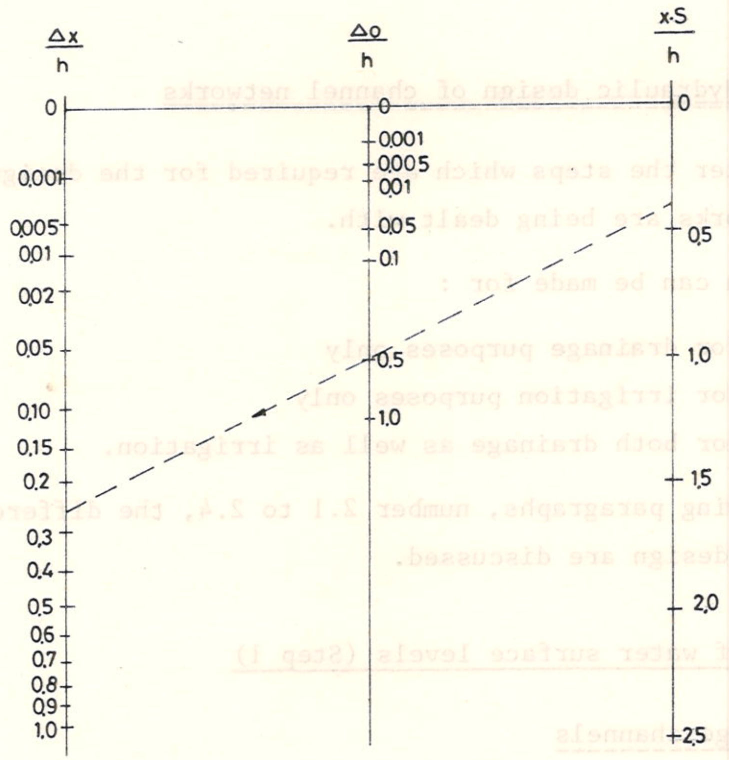
More accurate determinations can be made with the nomograph based on the simplified (van Smaalen) Bakhmeteff method (see Figure V - 1.5).

For larger streams, like the main drainage channel towards a sluice, the backwater curve may be approximated by :

$$\Delta_x = \Delta_o .\ e^{-3.\ S.\ x/h_n}$$

in which h_n is the normal depth belonging to a certain discharge Q and bottom (and parallel energy) slope S.

The latter approximation should only be used in cases where the backwater effect is not more than 10% of the waterdepth. Table V - 1.6 gives a comparison of the different methods to approximate backwater curves.



backwater curve $\left(\frac{\Delta_x}{h}\right)^{0.315} = \left(\frac{\Delta_o}{h}\right)^{0.315} - 0{,}408\,\frac{x \cdot S}{h}$

Figure V - 1.5 Nomograph for determination of backwater curves in a canal section according to van Smaalen-Bakhmeteff (approximation).

Table V - 1.6 : Comparison of backwater formula's

Example :

$S = 0.002$; $h = 1.0$ m; $\Delta_o = 0.5$ m.

	x (m)	10	50	100	200	500
Δ_x	parabolic	0.48	0.40	0.32	0.18	0
Δ_x	nomograph	0.48	0.44	0.37	0.25	0.055
Δ_x	exponential	0.47	0.37	0.27	0.15	0.02

Chapter 2. Hydraulic design of channel networks

In this chapter the steps which are required for the design of channel networks are being dealt with.

A distinction can be made for :

- channels for drainage purposes only
- channels for irrigation purposes only
- channels for both drainage as well as irrigation.

In the following paragraphs, number 2.1 to 2.4, the different steps of the design are discussed.

2.1 Design of water surface levels (Step 1)

2.1.1 Drainage channels

The critical period is the monsoon period when rice is the predominant crop. In general the design water levels in the drainage channels correspond with the average ground levels. (actually the drainage water level in a certain period = groundlevel + required waterdepth in the paddy field in that period).

This drainage level has to be maintained in the whole drainage unit. For larger polders, several drainage units of which each has its specific design drainage level may be designed.

All drainage units will drain the excess water into the main drainage channel(s), either direct via an open connection or via a regulating structure. (check with stop logs or fall-boards).

The main drainage channel(s) will convey the excess water to the sluice. In the situation that no drainage is necessary the water level in the main drainage channel corresponds with the design drainage level of the lowest drainage unit in the polder. During drainage of excess water through the drainage sluice, the water level in the main drainage channel will assume a gradient, depending on the amount of discharge conveyed.

The matter is illustrated in Figure V - 2.1.

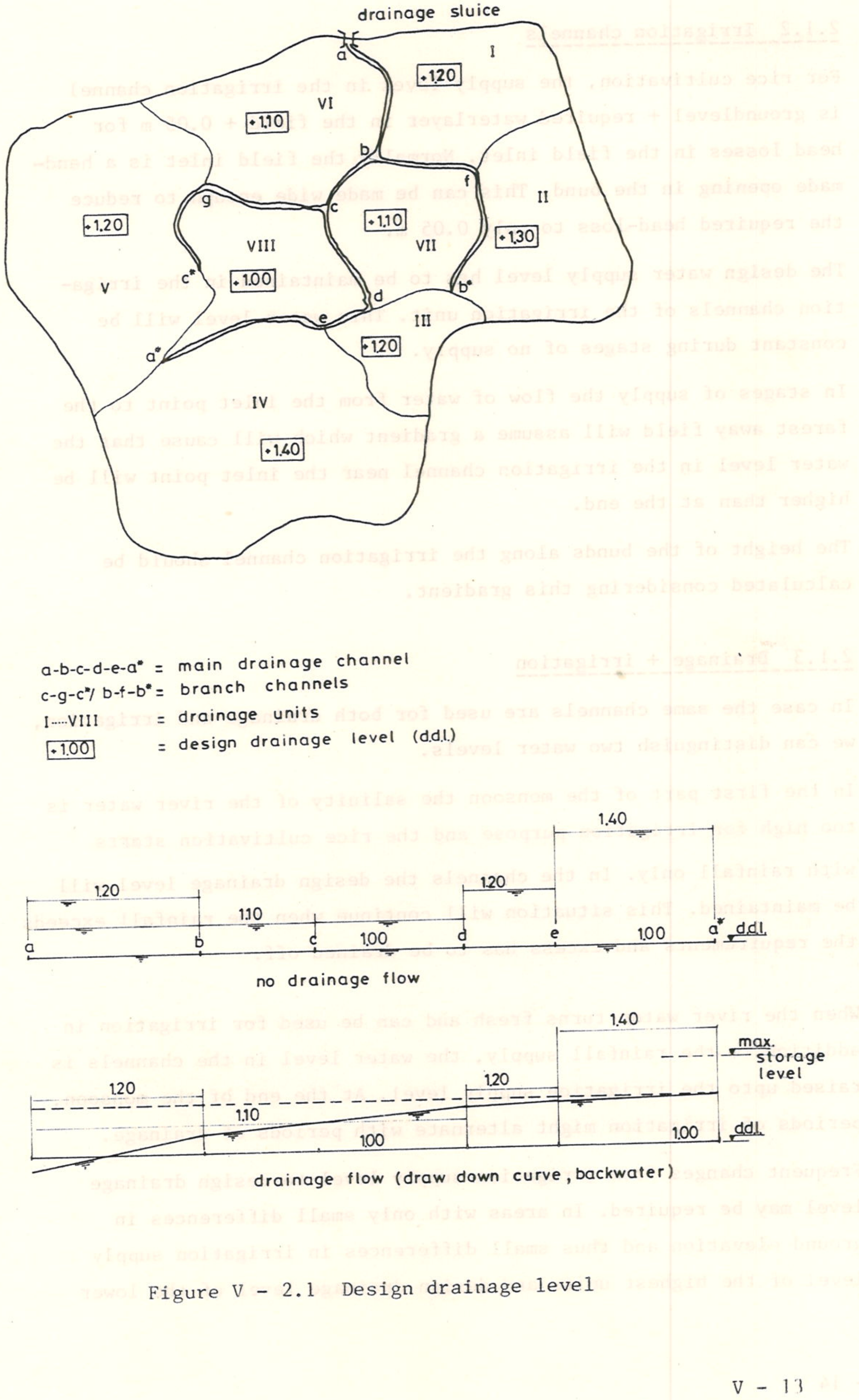


Figure V - 2.1 Design drainage level

2.1.2 Irrigation channels

For rice cultivation, the supply level in the irrigation channel is groundlevel + required waterlayer in the field + 0.05 m for head losses in the field inlet. Normally the field inlet is a hand-made opening in the bund. This can be made wide enough to reduce the required head-loss to only 0.05 m.

The design water supply level has to be maintained in the irrigation channels of the irrigation unit. This water level will be constant during stages of no supply.

In stages of supply the flow of water from the inlet point to the farest away field will assume a gradient which will cause that the water level in the irrigation channel near the inlet point will be higher than at the end.

The height of the bunds along the irrigation channel should be calculated considering this gradient.

2.1.3 Drainage + irrigation

In case the same channels are used for both drainage and irrigation, we can distinguish two water levels.

In the first part of the monsoon the salinity of the river water is too high for irrigation purpose and the rice cultivation starts with rainfall only. In the channels the design drainage level will be maintained. This situation will continue when the rainfall exceeds the requirements and excess has to be drained off.

When the river water turns fresh and can be used for irrigation in addition to the rainfall supply, the water level in the channels is raised upto the irrigation supply level. At the end of the monsoon, periods of irrigation might alternate with periods of drainage.

Frequent changes from irrigation supply level to design drainage level may be required. In areas with only small differences in ground elevation and thus small differences in irrigation supply level of the highest units and design drainage level of the lower units, this could be managable.

However, in areas with more pronounced differences this is only possible if sufficient check structures are available to control the water level changes in the units and in different sections of the (main) channel(s).

In general the dimension of the cross-sections of the channels are determined by the drainage requirements (see also paragraph 2.4). In some stretches along the channels bunds have to be made or raised to prevent flooding of the lower areas during stages when irrigation supply level is to be maintained.

2.2 Required design discharge (Step 2.)

The drainage and irrigation channels first have to be divided into sections with uniform bottom slopes and discharges of the channels. Then the required design discharge is determined for every section. Therefore the area which is drained or supplied via this channel-section is multiplied with the drainage criteria (see Volume IV, Chapter 2) or the irrigation criteria (see Volume IV, Chapter 3). These are expressed in mm/day. To convert the discharge into m^3/s use the conversion factors 1 mm/ha = 10 m^3 and 1 day = 86400 sec.

2.3 Calculation of corresponding water level gradients (Step 3.)

This is done by making use of the longitudinal sections of the channel network. Besides the levels as discussed above, the additional losses for structures in the channel, such as culverts, checks, has to be taken into account.

In principle the hydraulic gradient should correspond with the natural ground slope as closely as possible to prevent extra excavation or fill. Small gradients will result in relatively large cross-sections and steeper slopes may result in larger areas that can not be irrigated by gravity.

In general the minimum slope should not be less than 5×10^{-5} (5 cm/km) for the larger canals. For smaller channels normally steeper slopes are used.

2.3.1 Water level gradient in drainage channels

For the calculation of the water level gradient in the drainage channels the criteria is the drainage level at the end of the drainage channel, farest away from the drainage sluice (at points a*, b* and c* in Figure V - 2.1). The drainage level at this point may vary between the design drainage level (+ 1.00) and the maximum allowable storage level (+ 1.30).

At the point where the drainage level is critical the water level gradient is calculated in downstream direction, taking account of changes in discharges, flow-profiles and any regulating structures. The calculation is carried out with the help of the backwater formulas presented in Chapter 1.6 herefore. Profiles of the drainage channels have to be adopted if the calculations should indicate excessive draw-down effects resulting in too low down stream water levels.

In this way the water level gradient and water levels of the complete system during maximum drainage conditions and average drainage conditions are calculated, i.e. starting value at a* at + 1.30 m and ½ (1.30 + 1.00)=+ 1.15 m. An example of such a calculation is presented in ANNEX V - 3 .

The relation between various discharges and various water levels in the drainage channel near the sluice (H_x) and at the upstream end (H_i) are given in the delivery curve for the drainage channel as determined according to ANNEX V - 3.

2.3.2 Water level gradient in irrigation channels

In general the hydraulic gradient is determined by the principle that the irrigation water has to reach the farest and the highest areas in the irrigation unit at the required supply level. From this required supply level at the farest end of the channel the gradients and water levels are calculated towards the inlet point.

To rise the water levels check structures may be used (see Figure V - 2.2).

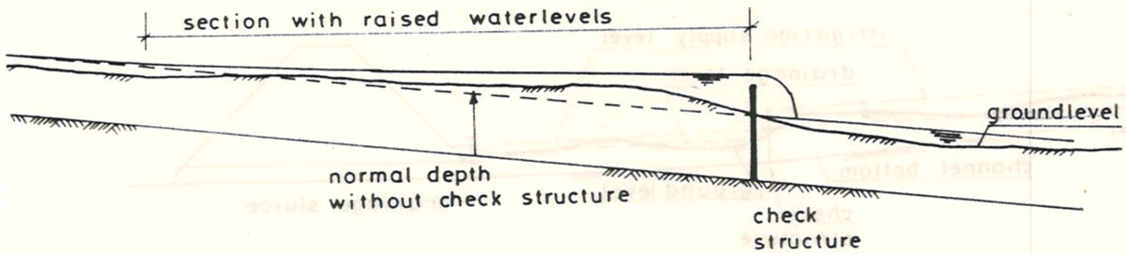


Figure V - 2.2 Check structure raising water levels.

2.3.3 Water level gradients in combined drainage + irrigation channels

In the longitudinal section the water levels and hydraulic gradients for both the drainage as well for the irrigation situation are plotted. For relatively flat areas with a regular slope this will pose no problem and can be achieved by regulating the irriga- inlet and drainage sluices only (see Figure V - 2.3).

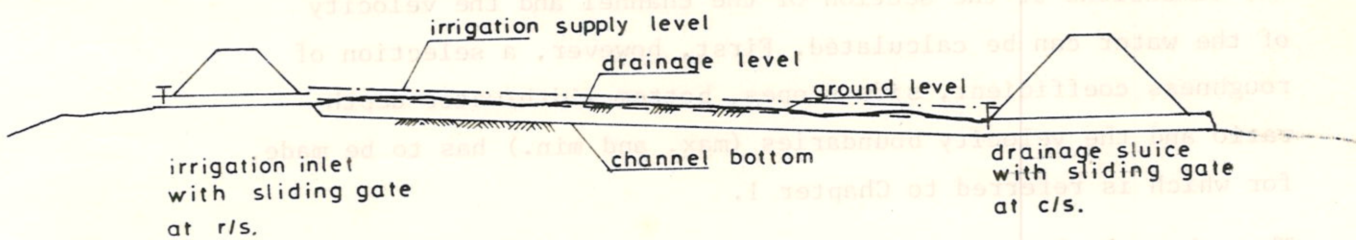


Figure V - 2.3 Hydraulic gradients in case of regular ground slope.

It should be mentioned that this figure and the following figure should be seen in relation with the general lay-out of the drainage/irrigation system as discussed in Volume IV, Chapter 6. An irrigation inlet generally serves only part of the drainage unit and is not located diametrical opposite the drainage sluice.

In case there is no regular slope, check structures may be required to maintain the irrigation supply levels in the higher parts during the irrigation period (see Figure V - 2.4).

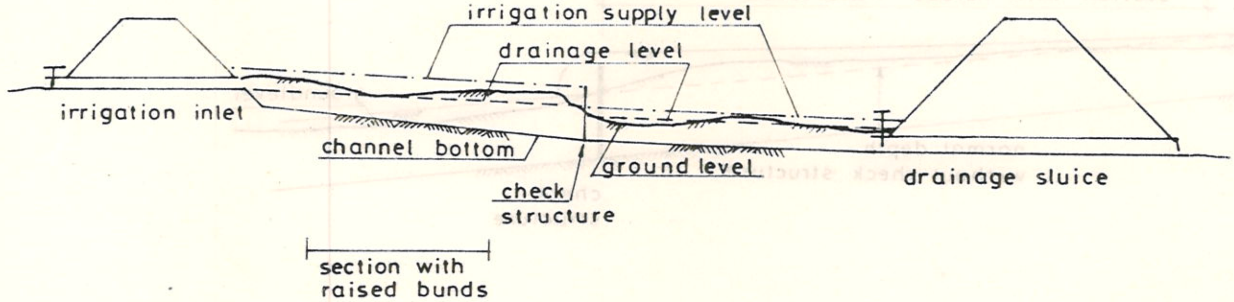


Figure V - 2.4 Hydraulic gradients in case of irregular ground slope.

This check structure should be of a regulating type (see Chapter 3.3) that can be opened or has a weir crest that can be lowered in order to have a drainage flow with a lower water level in the period that drainage is required.

2.4 Required design section, design velocity (Step 4.)

From the determined design discharge (see previous paragraph), the dimensions of the section of the channel and the velocity of the water can be calculated. First, however, a selection of roughness coefficient, side slopes, bottom width/water depth ratio and the velocity boundaries (max. and min.) has to be made, for which is referred to Chapter 1.

Then the calculation of the section and velocity can continue, using :

- nomographs in case of regular cross-sections (see ANNEX V - 2)

- calculator in case of natural channels with irregular cross-sections.

In case of a drainage cum irrigation channel the dimensions are in general determined by the drainage flow because the drainage criteria are much higher than the irrigation requirements and the water level of the drainage flow is lower than the irrigation flow, resulting in a smaller cross-sectional area. Only in the most upstream part, the irrigation flow is the determinant flow. If applicable, check the calculated velocities against minimum and maximum velocity values; if v is too high, reduce the hydraulic gradient and corresponding bottom slope. The gain in head should preferably be used in upstream and downstream sections. If this is not possible it must be absorbed by drop structures. Finally the required freeboard has to be considered for the determination of the crest level of the embankment or bunds along an irrigation channel.

TYPE:

1. Outlet Submerged
   - $h_1 > d$ — full flow
   - $h_2 > d$

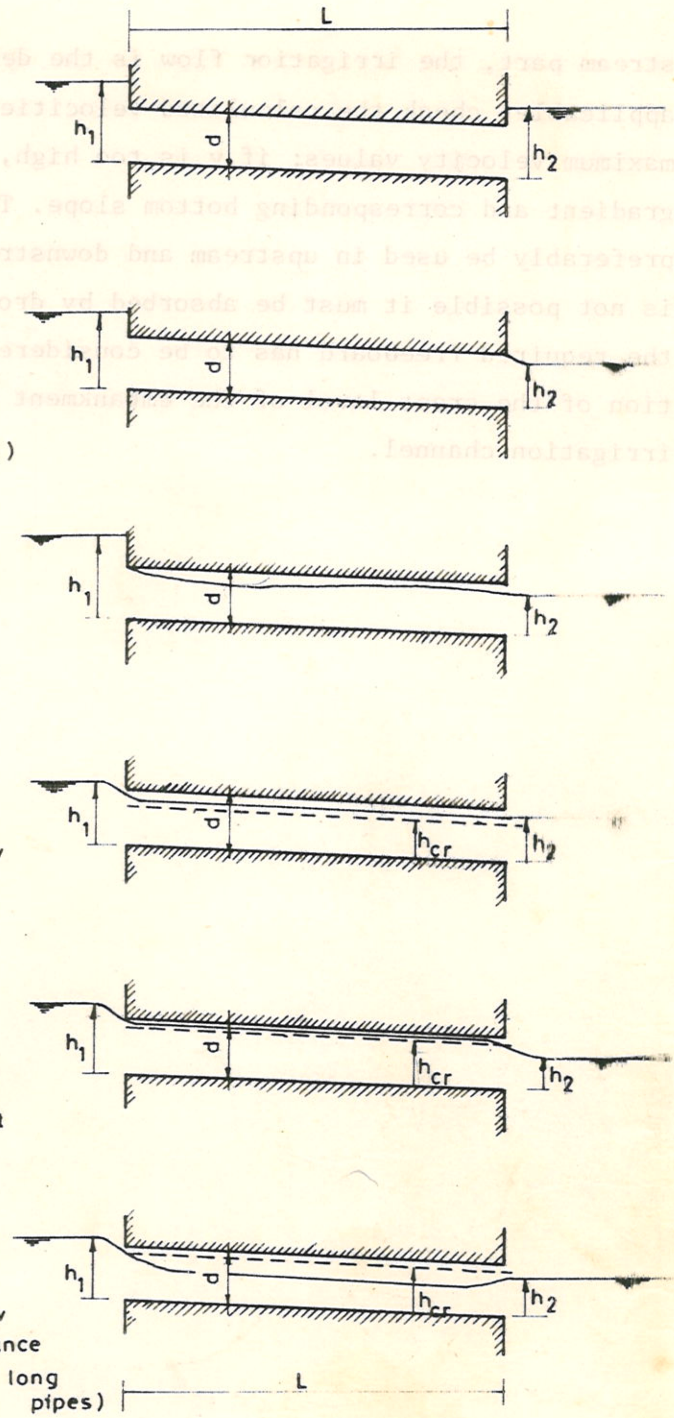
2. Outlet Unsubmerged
   - $h_1 > h^*$ — full flow (only for $\frac{L}{d} \gtrsim 20$, see figure V-3.2)
   - $h_2 < d$

3. Outlet Unsubmerged
   - $h_1 > h^*$ — partly full
   - $h_2 < d$

4. Outlet Unsubmerged
   - $h_1 < h^*$ — subcritical flow
   - $h_2 > h_{cr}$

5. Outlet Unsubmerged
   - $h_1 < h^*$ — subcritical flow control at outlet
   - $h_2 < h_{cr}$

6. Outlet Unsubmerged
   - $h_1 < h^*$ — supercritical flow controle at entrance (for chutes and long pipes)
   - $h_2 < h_{cr}$



h_{cr} = critical depth

$h^* = 1.2$ to $1.5\ d$

Figure V - 3.1 Types of culvert flow.

Chapter 3. Flow through hydraulic structures

3.1 General

The flow through a hydraulic structure, a culvert, or a regulator, may be classified into six types of flow as indicated in Figure V - 3.1 (ref. 2).

The identification of the six flow types may be explained according to the following outlines :

A. Outlet submerged :	Type 1
B. Outlet unsubmerbed:	
1. Headwater greater than the critical value,	
a. culvert hydraulically long:	Type 2
b. culvert hydraulically short:	Type 3
2. Headwater less than the critical value,	
a. Tailwater higher than the critical depth:	Type 4
b. Tailwater lower than the critical depth:	
- slope subcritical:	Type 5
- slope supercritical:	Type 6

For the determination of a certain flow condition through a structure, the following considerations have to be made.

- A culvert will flow full if the outlet is submerged, or, when the headwater is high and the barrel is long.
- The entrance of a culvert will be submerged if the headwater h_1 is more than a critical value $h^* = 1.5$ d, wherein d is the culvert height. Computations have shown that greater accuracy could be obtained by assuming the entrance was not submerged, in case submergence was uncertain (for instance, assuming $h^* = 1.2$ d). Whether a barrel is hydraulically long or short can be determined with the help of Figure V - 3.2.

The discharge for each type of flow can be derived from the basic orifice and weir formula.

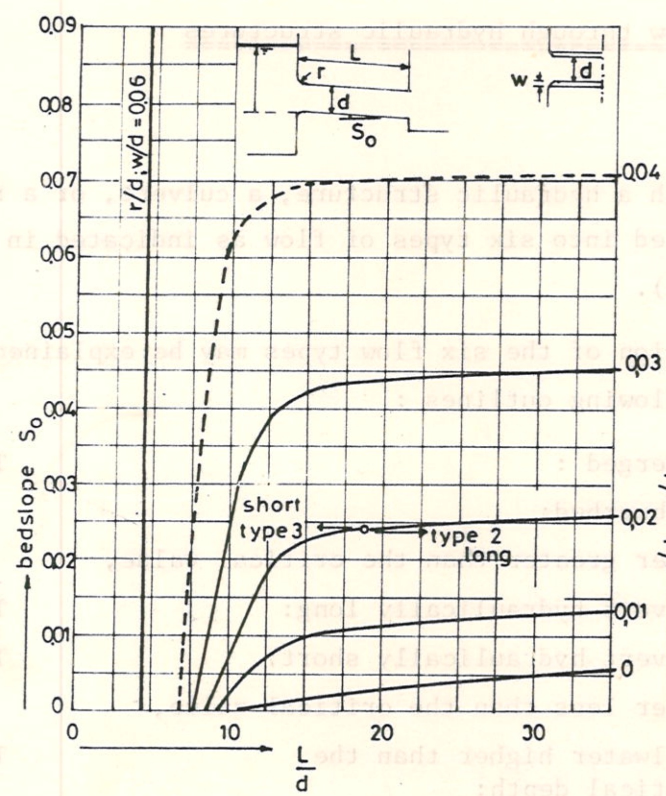


Figure V - 3.2 Criteria for hydraulically short and long box and pipe culverts with concrete barrels and square, rounded or beveled flush entrances from a vertical headwall, either with or without wingwalls. (U.S. Geological Survey, No. 376, 1957).

$$Q = C.A. \sqrt{2g.\ \Delta H}$$

in which Q = discharge in m^3/s

C = coefficient

A = cross section area in m^2

g = gravity acceleration = 9.8 m/s^2

ΔH = energy head loss in m.

In the following Table V - 3.1, the discharges formulas for the six flow conditions are summarized.

Table V - 3.1 Discharge formula

flow condition	discharge	remarks (see also Annex V - IV on energy losses)
Type 1 and Type 2	$Q = C_1 . A . \sqrt{2g . \Delta H}$	for square cornered entrance and outlets: $C_1 = \sqrt{\dfrac{1}{1.5 + \dfrac{2.L.n^2.g}{R^{4/3}}}}$
Type 3	$Q = C_3 . A_b . \sqrt{2g . H_1}$	C_3 = (see Figure V - 3.3 and note 1*)
Type 4	$Q = C_4 . b . h_2 . \sqrt{2g(h_1 - h_2)}$	Only if $h_2 > h_{cr}$ $h_{cr} = \sqrt[3]{\dfrac{q^2}{g}}$ If $h_2 \leqslant h_{cr}$, then type 5 flow C_4 = 0.82 for square entrances
Type 5 and Type 6	$Q = 1.353 \times b \times (h_1)^{3/2}$	for square entrance

Note: 1*) : The main difference with flow type 1 - formula, is the neglection of friction losses in this case.

b = width of barrel (m)

h_1 = headwater above invert level (m)

H_1 = energy head of headwater above invert level (m)

L = length of barrel (m)

n = Mannings roughness coefficient

g = gravity acceleration (= 9.8 m/s^2)

R = hydraulic radius (m)

A = Cross section area of flow (m^2)

A_b = Cross section area of barrel (m^2)

q = discharge per unit width (m^2/s)

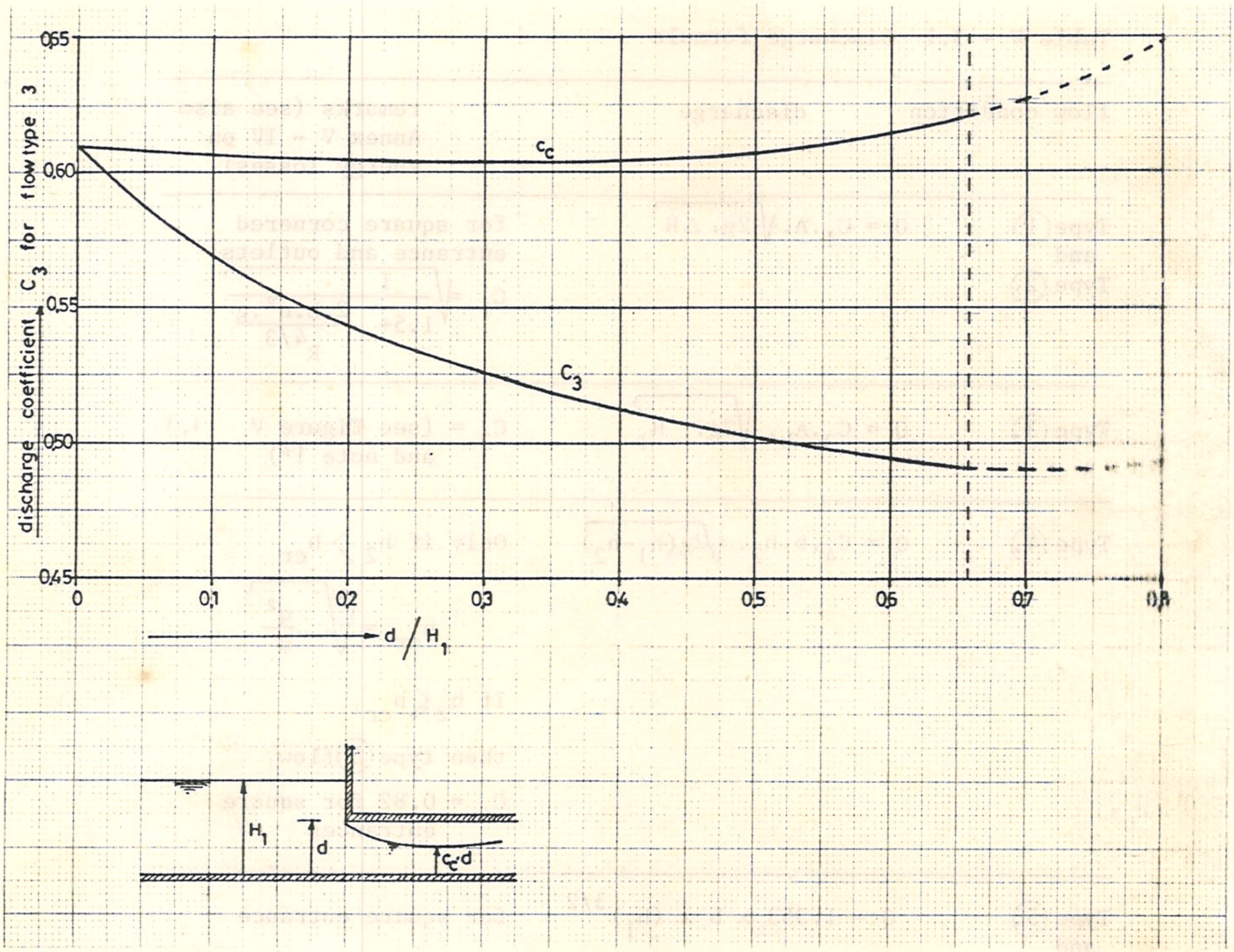


Figure V - 3.3 Discharge coefficient for flow type 3.

3.2 Short culverts

For short culverts (inside the polder under village roads and paths) in which the friction losses can be neglected in the formula

$$Q = C.A.\sqrt{2g.\,\Delta H}$$

the following values for C may be used :

Table V - 3.2 : Discharge coefficient for culverts

Bottom of culvert flush with channel bottom		bottom culverts higher than channel bottom		
entrance sides	C	bottom	sides	C
square	0.80	square	square	0.72
rounded	0.90			

3.3 Check structures

A check structure, or cross regulator, is a structure designed to raise the water level or to maintain a certain water level in a channel. In Chapter 2 and Volume IV, Chapter 6, some examples of the use of check structures have been given. A check structure may be a fixed overflow weir or may be an overflow or orifice provided with slide gates, stoplogs or checkboards (flashboards).

When a constant upstream water level is desired, an overflow type check is normally used. In many cases the channel will function both as irrigation as well as drainage channel. A check structure with removable gate, stoplog or checkboard is recommended in this case to cope with the much higher drainage flow in the period of excess rainfall. Sometimes a check structure can be combined with a culvert by fitting grooves with stoplogs or a slide gate.

The discharge formula for a free flowing weir is :

$$Q = C.\ b.\ H^{3/2}$$

in which Q = discharge (m^3/s)

b = overflow crest length (m)

H = head or waterdepth above the crest measured upstream of the check (m)

C = discharge coefficient ($m^{1/2}/sec$),

for broadcrested weir (stoplogs) : C=1.5-1.0

for sharp crested weir: C=1.9

for rounded weir: C=2.2

A check can also be made by means of a drop structure (see Figure V - 3.4). In this case the discharge can be calculated as :

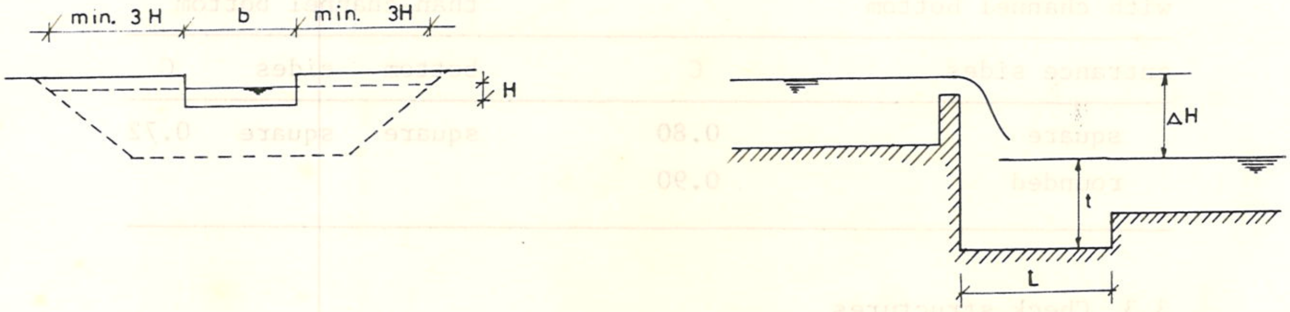


Figure V - 3.4 Drop structure.

$$Q = 1.83\ (b - 0.2\ H)\ H^{3/2}$$

For simple drop structures the dimensions of the depressed floor can be determined with:

$$L = 1.5 \times \Delta H \qquad (\Delta H = \text{drop})$$

$$B \times L \times t = \frac{Q \times \Delta H}{150}$$

wherein Q should be taken in l/s and B is the width of the floor.

In Volume VIII some examples of simple check structures are given.

Chapter 4. Design of Hydraulic Structures

4.1 Tidal inlet structures

4.1.1 Discharge capacity

The tidal inlet structure for the intake of water from a river or estuary is considered to be a long culvert and will have most of the time both a submerged inlet and a submerged outlet. Only at the beginning and at the end of the diurnal irrigation periods the inlet will not be submerged. For discharge calculations this can be neglected.

Long culverts with both the inlet and the outlet submerged will flow full like a pipe and thus flow condition 1 will prevail (see previous Chapter).

$Q = C_1 . A . \sqrt{2g . \Delta H}$ with: C_1 as indicated in paragraph 3.1

For Manning's roughness coefficient ANNEX V - 1 is to be consulted. For concrete lined boxes n = 0.015, for asbestos cement pipes n = 0.013.

For an 8" (20 cm) dia, asbestos-cement pipe the calculation is as follows :

$$D = 0.20 \text{ m}$$

$$A = 0.0324 \text{ m}^2$$

$$R = 0.0508 \text{ m}$$

$$C_1 = \sqrt{\frac{1}{1.5 + \frac{2 \times L \times (0.013)^2 \times 9.8}{(0.0508)^{4/3}}}} = \sqrt{\frac{1}{1.5 + 0.18 L}}$$

if : $L = 26.5$ m

$$Q = C_1 . A . \sqrt{2g . \Delta H}$$

$$= 0.0573 \sqrt{\Delta H} \quad \text{m}^3/\text{s}$$

In Figure V - 4.1 the graphs of Q versus ΔH are presented for various sizes of pipes.

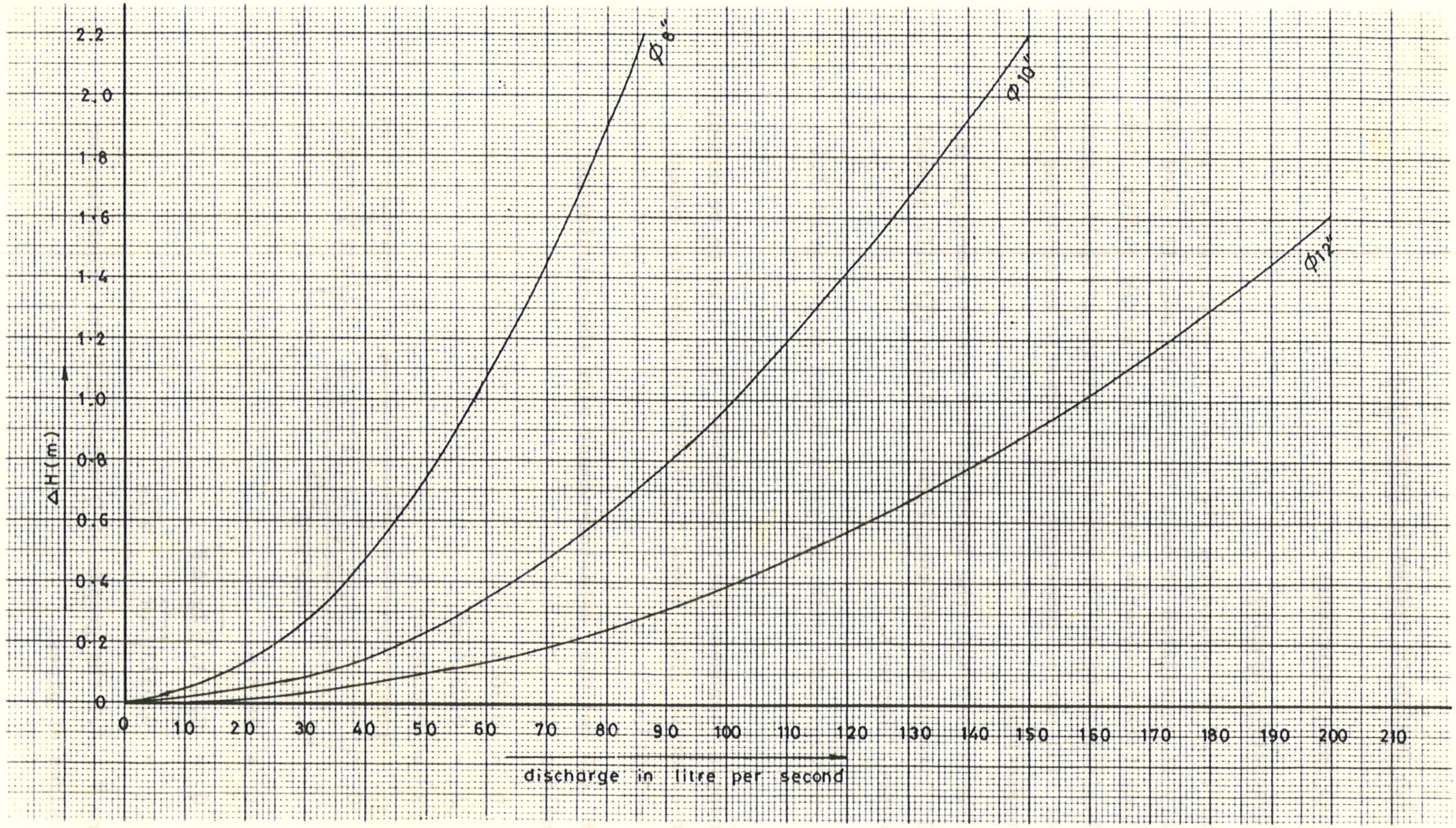


Figure V - 4.1 Pipe culvert discharge for a 26.5 m long barrel, for diameters 8", 10" and 12" pipe.

4.1.2 Area capacity

The area that can be irrigated by an inlet depends on :

- irrigation requirements
- size of the inlet
- outside water level
- ground level of the irrigation unit.

To calculate the area that can be irrigated with a pipe of a certain diameter the following example is worked out. The irrigation requirements are determined according to Volume IV, Chapter 3. It is assumed that the maximum requirements occur in September and October. In Figure V - 4.2 the tidal curves of the outside water levels are shown for these months. These curves are the monthly means of the daily observations. The curve of October is lower and also the irrigation requirements (in case of traditional varieties) are lower in this month. This means that the calculation has to be made for both months.

Assume: Irrigation Supply Requirements :

- September : 156 mm/month = 1560 m^3/ha/month
- October : 144 mm/month = 1440 m^3/ha/month

In Table V - 4.1 the volumes are determined which can be taken in through a 10" pipe inlet with a length of 26.5 m and design water levels in the polder of resp. 0.91 m, 1.07 m and 1.22 m + P.W.D. The volumes are graphically shown as the areas between the tidal curve and the ground level lines. The time axis is divided in portions of ½ hour starting from the centre of the curve. The central portion represents ½ hr. (= 1800 sec.), all the other portions have symetrical portions at the other side of the centre line and the average value of H can be multiplied by 1 hr. (3600 sec) to get the volume at both sides of the centre line.

For a 10" pipe (0.254 m) Q is calculated to be

$$Q = 0.1007\sqrt{\Delta H} \quad m^3/s$$

The areas which can be irrigated in September and October are given in Table V - 4.2 for the three design levels.

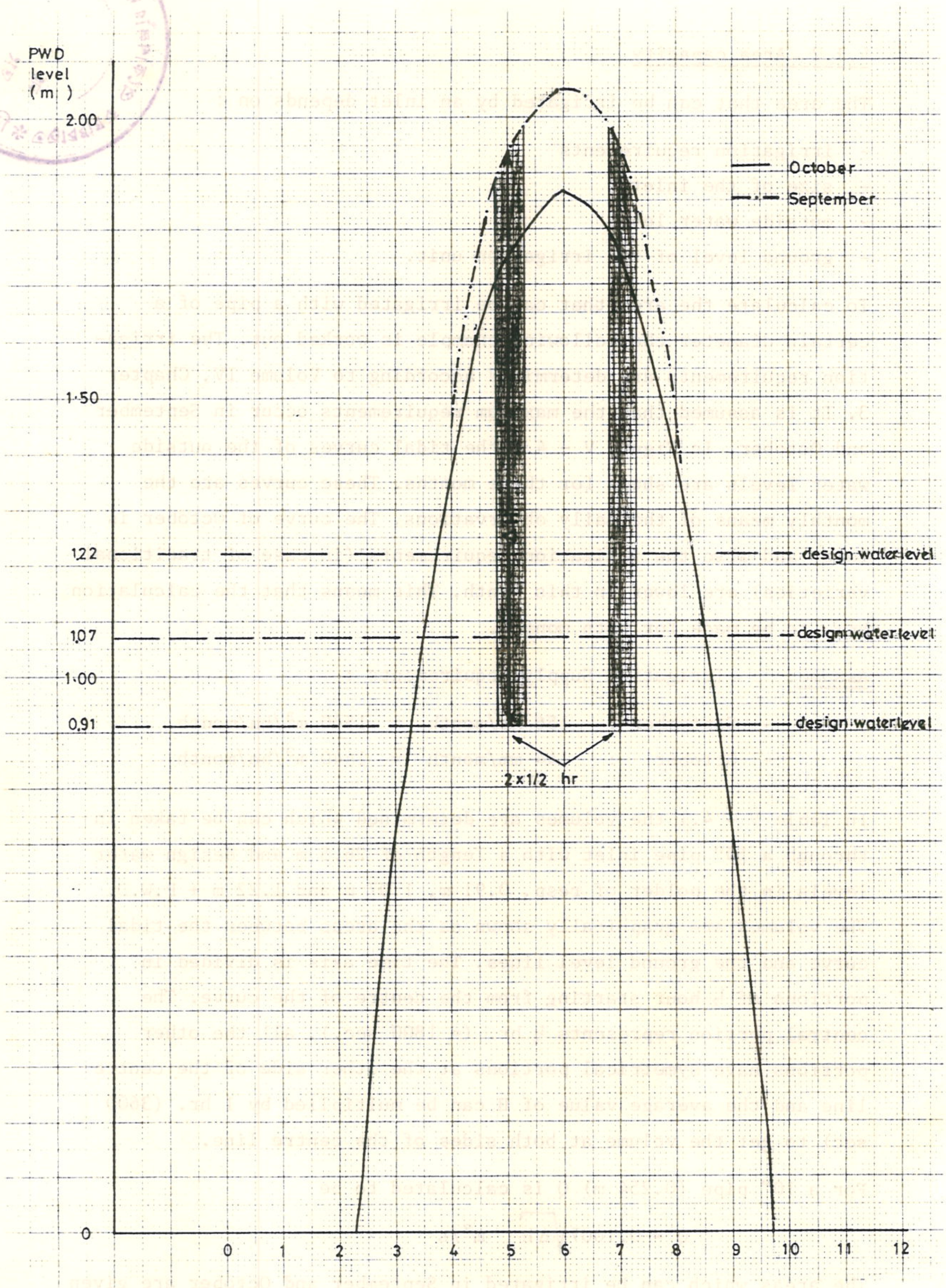


Figure V - 4.2 Mean tidal curve.

Table V - 4.1 Irrigation volumes

10" pipe	SEPTEMBER				OCTOBER			
design water level	ΔH (m)	Q (l/s)	ΔT (s)	$\frac{Q \times \Delta T}{1000}$ (m^3)	ΔH (m)	Q (l/s)	ΔT (s)	$\frac{Q \times \Delta T}{1000}$ (m^3)
0.91 m + P.W.D.	1.13	107	1800	193	0.95	98	1800	177
	1.11	106	3600	382	0.91	96	3600	346
	1.01	101	3600	364	0.84	92	3600	332
	0.88	94	3600	340	0.71	85	3600	305
	0.57	76	3600	274	0.45	67	3600	243
	0.17	42	3600	149	0.16	40	3600	145
	Total per tide 1702 m^3				Total per tide 1548 m^3			
	Total per month 100, 418 m^3				Total per month 91,335 m^3			
1.07 m + P.W.D.	0.97	99	1800	178	0.79	89	1800	161
	0.95	98	3600	353	0.75	87	3600	314
	0.85	93	3600	334	0.68	83	3600	299
	0.72	85	3600	307	0.55	75	3600	269
	0.41	65	3600	232	0.29	54	3600	195
	0.08	28	1800	51	0.08	28	1800	51
	Total per tide 1457 m^3				Total per tide 1289 m^3			
	Total per month 85,948 m^3				Total per month 76,053 m^3			
1.22 m + P.W.D.	0.82	91	1800	164	0.64	81	1800	145
	0.80	90	3600	324	0.62	79	3600	285
	0.70	84	3600	303	0.53	73	3600	264
	0.57	76	3600	274	0.40	64	3600	229
	0.26	51	3600	185	0.14	38	3600	136
	Total per tide 1249 m^3				Total per tide 1059 m^3			
	Total per month 73,747 m^3				Total per month 62,484 m^3			

Table V - 4.2 Irrigation area

design water level	September			October		
	intake	irrigation requirement	area	intake	irrigation requirement	area
(m)+PWD	(m^3)	(m^3/ha)	(ha)	(m^3)	(m^3/ha)	(ha)
0.91	100.418	1560	64	91.335	1440	63
1.07	85.948	1560	55	76.053	1440	52
1.22	73.747	1560	47	62.484	1440	43

From the table it can be concluded that October is the critical month.

The method to calculate the area that can be irrigated by a pipe inlet of a certain diameter gives only an estimate. For accurate calculation, several factors which have a bearing on the intake volume, have to be accounted for.

a) Yearly variations in mean water level

The monthly mean values have been used. In some years, however, the mean water level may be lower or higher. If enough reliable data are available the frequency curves can be determined for the monthly mean water levels and the 90% (lower) mean water level may be used. In this case a higher value should be used for the effective rainfall for the determination of the irrigation requirements. The application of two safety factors will result in too severe requirements. In general the 90% effective rainfall in combination with the mean water level will give the lowest values for the area that can be irrigated.

b) daily variations of the tide

Throughout the month the tidal curve varies. The difference between the maximum water level at neaptide and at springtide may vary. In Figure V - 4.3 the HW levels are shown for Polder 22 in October 1981.

The deviation of the spring tide HWL value and the neaptide HWL value are plus or minus 0.36 m. Since the discharge through the inlet is related to the square root of Δ H, the use of the mean

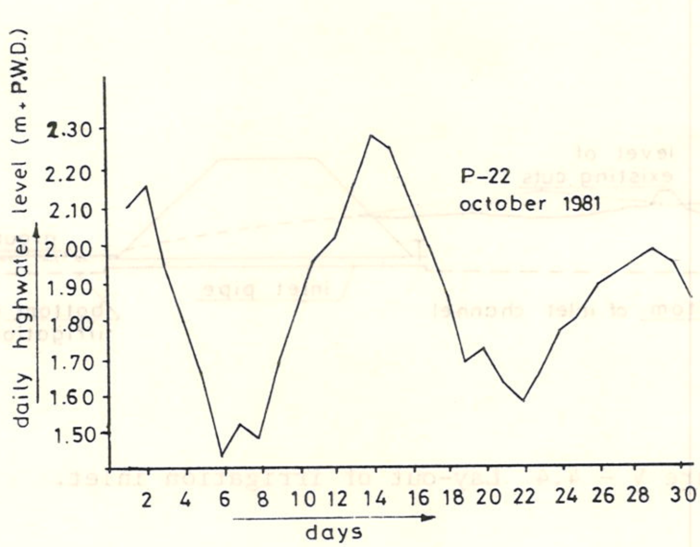


Figure V - 4.3 High water levels at Polder-22 during October 1981.

HWL is not fully correct and inlet quantities should be calculated with due attention to this phenomena.

4.1.3 Location selection

Several factors have to be taken into consideration for the selection of the location of inlet structures. Some of them are being dealt with in this section.

- Inventory of cuts in the embankment and inlet structures made by the farmers themselves. This gives an indication of the actual needs of the farmers. This inventory should be followed by interviews with the farmers to investigate whether inlets at these locations are required and/or additional inlets are needed.
- The inlet should preferably have a direct connection with the river. In general it can be stated that the level of the foreland is 0.3 - 1.0 m higher than the groundlevel inside the polder due to continued siltation on the foreland after the construction of the embankment. The present practise is that water is taken in via the foreland (see Figure V - 4.4). This means that irrigation actually can only start after the outside water level overtops the (higher) foreland (level a). The period of irrigation can be extended if a direct connection is made to

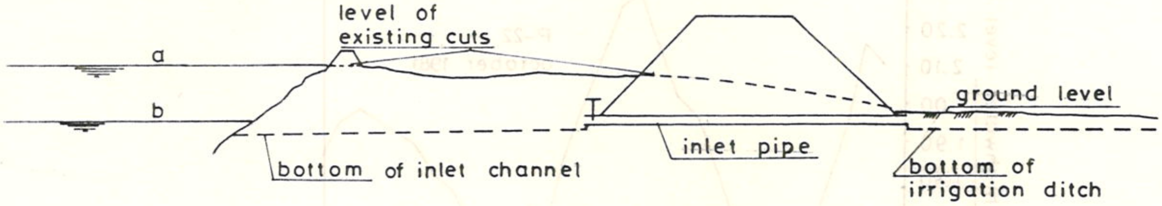


Figure V - 4.4 Lay-out of irrigation inlet.

the river so that irrigation can already start as soon as the river level rises above the inside groundlevel (level b).

- Siltation. If the connection channel between the inlet and the river is long (wide foreland), there is a risk of silting up. This can be avoided by creating a flow during each tide by filling and emptying the bunded foreland through openings close to the polder inlet. The water that leaves the bunded foreland during low tide will flush the silt from the connecting channel (see Figure V - 4.5).

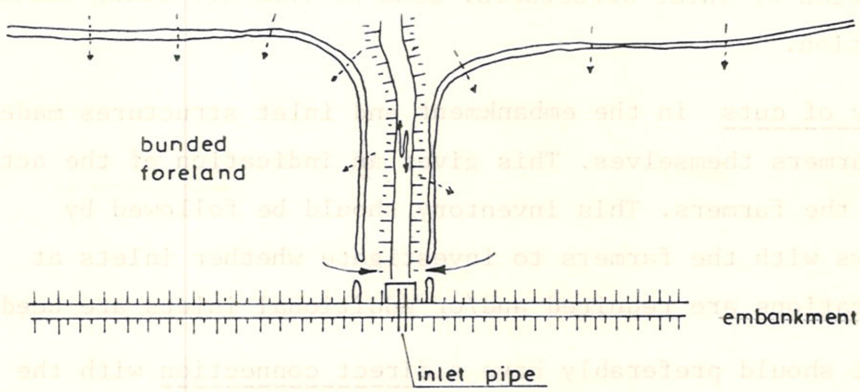


Figure V - 4.5 Flushing of inlet channel.

- Level. The inlet structure should preferably be located at or near the highest part of the irrigation unit, such as to reduce the construction of bunds. After the tidal irrigation interval (twice daily) the irrigation water may flow to the lower parts of the irrigation unit and the water level at the inlet structure may drop below the groundlevel at the inlet structure. Therefore, it is recommended to have the invert-level of the inlet structures about 0.30 m (1 ft) below average groundlevel at the inlet structure, so that the intake at the next irrigation interval can start as soon as the outside water level rises above the polder water level that has been lowered during low tide due to the continued flow to the lower parts of the irrigation unit.

The inlet structure should be provided with a (very light) flapgate at the country side to prevent that water will flow back to the river during low tide.

4.2 Tidal Drainage sluice

4.2.1 Introduction

Once it has been decided according to the guidelines presented in Volume IV, Chapter 6, how many drainage sluices should be made and where these sluices have to be constructed, and the drainage capacity of each of these outlets has been determined, according to the procedures given in Volume IV, Chapter 2, the calculation for each individual drainage unit can commence. It should be realised that the discharge through a drainage sluice depends on the characteristics of the outer water (tidal, non-tidal), the type of drainage-regulator and that the through-flow is determined by criteria for permissable velocities through the structure and in the immediate vicinity of the structure.

In the following paragraphs calculations are presented for the hydraulic design of a tidal drainage sluice that has to convey a certain design drainage discharge as determined by the drainage modules of the area and the size of the area to the drained.

In principle several alternative modes of evacuating the excess water from an embanked area are possible, as is highlighted in Volume VII, Structural Design Aspects.

However, a drainage sluice with lowest possible invert level and a flapgate at the river-side is the most appropriate structure.

For the control of drainage flow and the maintenance of certain water levels inside the polder, the drainage sluice can be equipped with a sliding gate at the country-side. The crest height of this sliding gate in lowered position is determined by various criteria depending on the desired water level to be maintained.

Calculations for the free drainage through the flapgate and/or the controlled drainage through a sliding gate sluice are to be carried out with the formulas presented in Chapter 3 of this Volume.

The boundary conditions to be established before any calculations can start are the following :

- design drainage discharge .
- polder water level (design drainage level) occurring simultaneously with the design drainage discharge.
- hydrograph (tide-curve) of the river.

However in some cases, during certain periods of the year, the drainage sluice may be used as an inlet structure. Especially in areas where there is a combined irrigation-cum-drainage channel system. This may occur in the early monsoon when rainfall is insufficient and water needs are supplemented by letting in fresh river water.

The maximum inlet flow can be calculated after boundary conditions have been established for :

- hydrograph (tide curve) of the river.
- polder water level during stages of inlet-flow through the sluice.

The hydraulic design of the drainage sluice has to be such, that no damage will occur due to extreme velocities during drainage flow as well as maximum inlet-flow.

Another important aspect in the design of a tidal drainage regulator is the discharge capacity of feeder and outfall channel of the sluice and their bottom width in relation to the width of the drainage structure. The regulator width is again determined by the discharge requirements and the characteristics of the river side water levels.

In the calculation process for the hydraulic design of the drainage sluice the following steps are distinguished.

1. Determine location and drainage capacity of the sluice to be build. (see Volume IV, Chapter 2 and 6).
2. Determine boundary conditions of inside and outside water levels during stages of design drainage discharge and during stages that the sluice is used as an inlet-structure.
3. Check if the capacity of the natural available channel, which has to operated as main drainage channel, is adequate.
   If not, enlarge the cross-section of the channel, or choose

other channel or re-consider the required capacity of the sluice and/or location and sizes of the drainage structures.

4. Determine the required invert level of the sluice, considering the different criteria
   - discharge capacity
   - maximum permissable velocities
   - width of the sluice/drainage channel
   - foundation conditions.

5. Make a sketch design of the regulator structure with rough dimensions, considering the foundation possibilities, invert level at the construction site and the drainage requirements.

4.2.2 Location selection

The location of a sluice should be such that optimal use is made of existing khals for the drainage of the considered area. Mostly sluices are built close to existing drainage out-lets, in order to reduce excavation of feeder and outfall channels. However, the natural drainage capacity of an existing khal should correspond with the capacity of the sluice, and the total lay-out of the drainage system in a polder with several drainage structures should be in concert with this principle. The item of drainage lay-out is further elaborated in Volume IV, Chapter 5 and 6.

4.2.3 Boundary conditions for water levels at the drainage sluice

During stages of drainage flow, the polder water level is at or above the design drainage level as determined for the polder.

Actually, during stages of design drainage discharge the water level in the polder may be in between the design drainage level and the maximum allowable water level (design drainage level + allowable storage as determined according to Chapter 2, Volume IV).

For the calculation of the sluice dimensions, based on the average design drainage discharge, the polderwater level is therefore assumed to be half way in between the design drainage level (d.d.1) and the maximum allowable storage level (m.a.st.1).

During the year the design drainage level may vary as the crop grow stage and water requirements vary. Normally, in the monsoon period the design drainage level corresponds with the required water level in the rice-fields (if a rice-crop polder is considered.)

The design drainage discharge through the sluice is to be calculated at average tide conditions of the river water.

Average tide is defined as the average tidal amplitude ranging form M.H.W. (mean high water) to M.L.W. (mean low water) at average river level (F.M.L. = fortnightly mean level) for the considered time of the year.

All water levels M.H.W., M.L.W., H.H.W. and L.L.W. are to be determined via relation curves to be made of gaugereadings from neighbouring gauge stations and the temporary gauge installed at the construction site.

During a certain period of the year the sluice may be used to let in fresh river water into the polder. For the polder water level during these stages a lower level than the design drainage level is to be taken. Although the rainfall will start before the river water turns fresh and can be used for supplementary irrigation, the rain is not enough to fill up the drainage/irrigation channels upto design level. In order to account for most unfavourable conditions to calculate the maximum inlet flow, the polderwater level at this stage is assumed to be 0.30 m below average groundlevel of the lowest drainage unit in the polder.

For the riverwater the H.H.W.S. (highest high water spring) exceeded during 50% of the years of records is to taken.

For maximum velocities during drainage conditions, the riverwater is taken at L.L.W.S. (lowest low water spring tide), exceeded during 50% of the years of record. The polderwater level during this stage is considered to be at maximum allowable storage level (in our example + 0.30 m above design drainage level).

Summarizing the following levels are to be applied in the hydraulic calculations for the drainage sluice.

Criteria	Polder water level	river level
Drainage discharge	half way in between maximum storage level and design drainage level.	average tide conditions (mean tide)
max. inflow velocity	0.3 m below design drainage level	highest high water, HHWS (50%)
max. outflow velocity	maximum storage level.	lowest low water, LLWS (50%)

An important aspect in relation with the determination of the water level boundary conditions is the characteristics of the main drainage and sluice out fall channel. The depth of feeder and outfall channel should be such that at all critical design conditions as mentioned above, the discharge can be conveyed without exceeding maximum velocities. The bottom depth of the feeder channel should at least be 0.5 m below the lowest polder water level occuring in the dry season. The reasoning is that by maintaining a low polder water level the capillairy rise from saline ground water into the top ground layers is reduced. Sluice invert level should therefore allow the drainage at this stage (all be it rather small and during limited periods).

The depth of the outfall channel is determined by the lowest tide conditions (L.L.W.) at which stage the maximum discharge has to be conveyed. Although an outerchannel at this depth would most likely silt up because these low water conditions occur infrequently, the bottom protection behind the sluice has to be designed on these conditions.

4.2.4 Capacity of the drainage sluice and the connecting channels

For the determination of the drainage discharge, e.g. the required drainage capacity of the sluice, the following data are used to illustrate an example :

example:

- gross area : 2500 ha
- drainage module : 30 mm/day

- sluice coefficient : $\frac{T_0}{T} = 1.7$ $\quad T_0$ = tidal cycle = 12 hrs 25 min.
  T = drainage time during one tidal cycle.

The drainage time is estimated at first from the tidal curve and the level of the polder water at the start of the drainage. Later on, after flood routing calculation, it is to be checked whether this has to be corrected.

The total volume of water to be evacuated during 24 hours is :

$$(\text{drainage module}) \times (\text{area}) =$$
$$0.03 \times 2500 \times 10^4 \ m^3 = 750.000 \ m^3.$$

If the tidal cycle is 12 hours 25 minutes and the sluice coefficient is 1.7 then the average discharge through the sluice during the drainage period T should be :

$$Q_{av} = 750.000 \times 1.7 \times \frac{1}{24 \times 360} = 14.8 \ m^3/s$$

Since the drainage discharge during the period T will not be constant but will increase from zero to a certain maximum and will then gradually decrease due to the decreasing polderwater level during T, the maximum drainage discharge is taken to be :

$$Q_{max} = \frac{1}{0.85} \times Q_{av} = 17.36 \ m^3/s$$

The coefficient 0.85 is an estimation based on experience and can later be checked after the floodrouting calculations have yielded the discharge curve of the regulator during the discharge time T.

For the determination of the discharge capacity of feeder and outfall channel extensive surveys have to be made of length, width, depth and natural condition of slopes and bottom growth. A complete description of the channels, obtained by physical site inspection by the responsible designer, complemented with large scale topographic maps, is imperative. Hydraulic calculations using Mannings equation are to be used (see Chapter 1).

The average cross-section of the feeder-cum-outfall channel has to be determined from measured profiles. Cross-sectional profiles should be taken at distances not exceeding five times the width of the channel. From the cross-sections, a long-section should be plotted, showing the average depths of all consecutive cross-sections.

The average cross-sectional profile and slope of the channel can thus be determined as indicated in Figure V - 4.6. It is essential that all soundings should be plotted relative to the same datum.

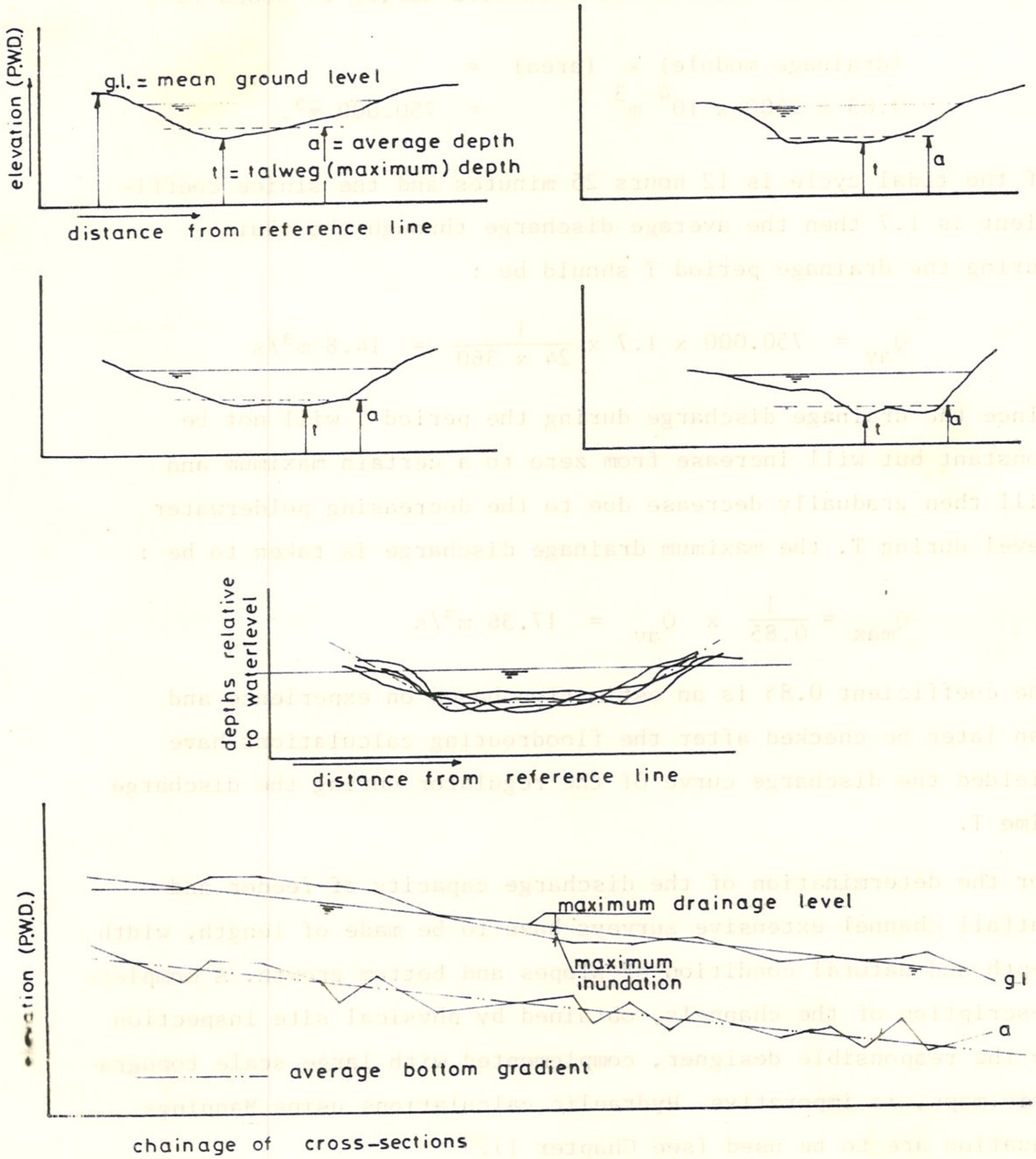


Figure V - 4.6 Cross-section plotting for channel calculations.

If roughness, n, slope S, and average cross-section have been determined, the discharge capacity of the channel can be determined. The maximum drainage discharge capacity of the channel is to be checked with the water level at design drainage level. Since this is the most unfavourable situation to occur.

During the drainage period T, the water level in the main drainage channel will gradually assume a gradient towards the sluice and the water level at the sluice will be lower then the water level at the start of the drainage period.

How to calculate the water levels at the sluice and in the drainage channel during fully developped drainage flow is demonstrated in ANNEX V - 3.

In the same process the average velocities occurring in the drainage channel during the different water level and discharge stages can be checked.

The velocities should at all stages remain below a value at which considerable scour would occure (see Chapter V - 1.4).

For silty clay this velocity is 0.8 m/s.

At the same time the dimensions of the outfall channel should be such that the maximum discharge can be conveyed, even at lowest tide conditions, without causing velocities that exceed above 1.0 m/s. In any case the capacity of the sluice and the capacity of the inflow and outflow channel should be in harmony and should be analysed for all critical flow stages, drainage and flushing.

4.2.5 Dimensions of the sluice, invert level

From the discharge formulas presented in Chapter 3 of this Volume, it can be seen that the discharge through a structure increases if :

1) The wet profile A increases, in case the barrel of the sluice is submerged.
2) The upstream waterhead h, increases, in case the barrel is not submerged.

For our problem this means that the invert level of the structure, which determines the upstream head h, should be as low as possible in order to get a maximum flow through a minimum size structure.

On the other hand the discharge through a structure should be such that flow velocities do not exceed 3,5 m/s since this may create damages to the construction and unwanted turbulences and disturbances.

A third criteria may be the average velocity created in the drainage channel by the maximum drainage discharge, causing scour. Although this could be cured by increasing the channel profile,velocities could also be reduced by raising the invert level.

A fourth and most important criteria is the soil conditions at the construction site, which may prohibit a deep invert level requiring deeper foundations.

In most cases in the Bangladesh Delta Area the second and fourth criteria will be the determining factors for the design of sluice invert level.

If the sluice is also to be used for inlet, then a deep box will give even higher discharges and flow velocities, since the maximum head difference at inlet conditions might be higher than at drainage conditions. However, foundation at a convenient depth may only be possible at very high costs, which are out of proportion to the whole construction cost. Moreover, a deep box will require high and thus expensive wingwalls.

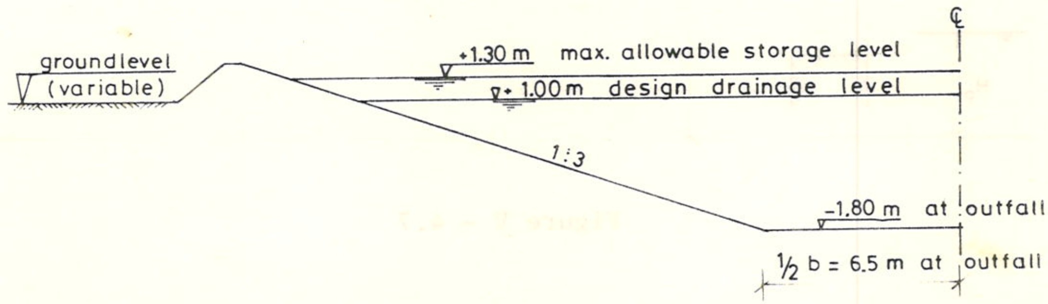
A compromise has to be found, in which proper drainage capacity is combined with adequate foundation and construction costs. If the maximum discharge is determined, then the width of the sluice follows from the maximum allowable velocity in the sluice and the invert level.

If the invert level is not restricted by foundation conditions then the maximum allowable sluice velocity will determine this invert level.

If the invert level of the sluice is not limited by foundation conditions, the velocities occurring in the sluice will be the determining criterium.

In order to demonstrate the calculations procedure to establish the sluice invert level and width, the following information is assumed :

- Drainage area as indicated in ANNEX V - 3.
- Design drainage level : + 1.00 m.
- Maximum allowable storage level : + 1.30 m
- Profile of the main drainage channel as shown below.



- Bottom level of drainage channel near the outfall :-1.80 m.
- average bottom slope of drainage channel : as shown on ANNEX V - 3.
- length of main drainage channel : 6800 m
- drainage area : 2500 ha
- drainage module : 30 mm/day
- tide curve : as shown in ANNEX V - 3, Figure A.V - 3.4.
- area of tertiairy and secondary drainage channels : 105.000 m^2
- area of low pockets which are in open connection with the drainage channel : 1% of 2500 ha + 250.000 m^2.

To determine the flow velocities in the sluice, the upstream water level and the tide have to be known. The tidal curve is known. The water levels just upstream of the sluice have to be determined for the various stages of the flow during the drainage period T, since this flow is gradually changing in time and will generate a back-water curve towards the end of the drainage channel. (see Figure V-4.7).

To know the water level H_x near the sluice in relation to the discharge in the drainage channel and the water level H_i at the upstream end of the drainage channel a delivery curve has to be calculated for the drainage channel for various Q and various values of H_i.

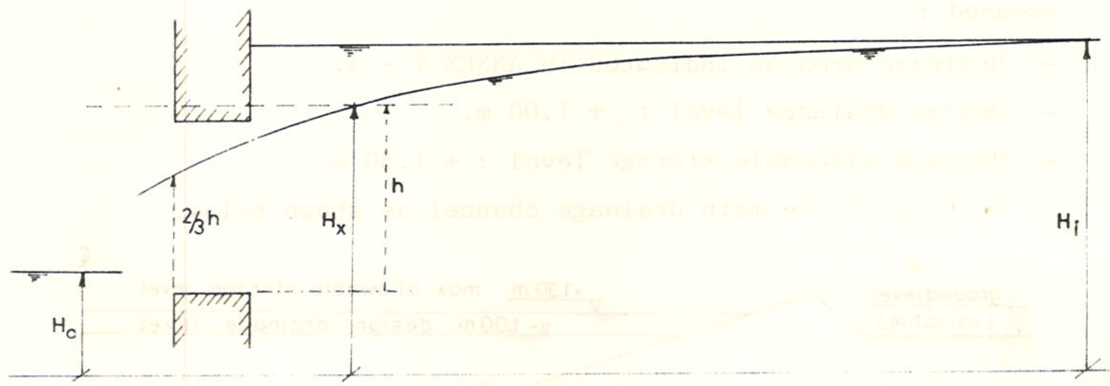


Figure V - 4.7

For our drainage channel the delivery curves have been calculated according to the procedures and diagrams presented in ANNEX V - 3, to which is referred.

The critical condition occurs when the polderwater level is at maximum storage level (+ 1.30 m) and the flow has reached its maximum value ($Q = 17.36\ m^3/s$) during the drainage period T.

The corresponding backwater at the sluice is now taken from the delivery diagram of the drainage channel as shown in ANNEX V - 3, resulting in a water level at the sluice of + 1.28.

In fact, the maximum discharge of $17.36\ m^3/s$ was assumed to occur during average drainage conditions with polderwater level H_i at the upstream end of the drainage channel at the medium between maximum storage level (+ 1.30 m) and design drainage level (+ 1.00 m).

The maximum drainage during extreme conditions with polderwater level H_i at + 1.30 m, will be higher and will at the same time result in a smaller value of H_x at the outfall of the drainage channel. But, as will be shown later on, by estimating Q_{max} at $\frac{1}{0.85} \times Q_{av}$, we will be on the safe side.

It is now assumed that the outside water level has dropped so far that the difference between inside and outside water level at the sluice will cause the sluice discharge to be under

flow condition 5.

The criteria for maximum velocities in the sluice was 3.5 m/s under these extreme conditions.

The maximum permitted water head H_x can now be calculated as follows : (see Figure V - 4.7).

flow condition 5 : $q = C \times (H_x)^{1.5}$

continuity : $q = h \times v_{max}$

critical depth control at box - exit : $h = 2 \times \frac{(v_{max})^2}{2g}$

it follows that : $C \times (H_x)^{1.5} = \frac{(v_{max})^3}{g} = 4.375$

For square cornered entrances of the box, C is taken at 1.35.
For the theoretical unit width discharge with no entrance and friction losses C could be upto 1.70.

For the estimation of the maximum discharge/velocity per unit width in the sluice, take C = 1.60, yielding for H_x = 1.96 m.

The invert level should now be at : 1.28m - 1.96 m = - 0.68 m P.W.D.
If, for instance , initially Q_{max} was chosen at 1.5 x Q_{av} = 1.5 x 14.8 = 22 m³/s, the draw-down at the sluice would have been at + 1.06 m and the invert level would become: 1.06 m - 1.96 m = - 0.90 m P.W.D.

Invert level is now provisionally chosen at a level in between these results, at - 0.80 m P.W.D.

The width of the sluice is now determined assuming average drainage discharge conditions (Q = 14.8 m³/s) and the polder water level at the start of the drainage period T assumed to be half in between maximum stage level and the design drainage level, i.e. at + 1.15 m.

The backwater H_x for this situation is now taken from Figure A.V - 3.2 ANNEX V - 3 and yields for H_x = + 1.13 m.

This level will be reached shortly after the start of the drainage period T, but will then drop further as the continued drainage will cause the water levels in the polder to drop further.

How the water levels will be at the end of the drainage period is approximated as follows :

area : 2500 ha as indicated on Figure V - 2.1.

drainage modules : 30 mm/day

area drainage requirement : 2500 x 10^4x0.03 = 750.000 m^3/day

sluice coefficient To/T = 1.7 hrs = 31.250 m^3/hr

= 8,68 m^3/s

sluice drainage requirement : 14,8 m^3/s during period T.

discharge during sluice period : (T = 7.3 hrs) x 14,8 x 3600 =

= 389.153 m^3.

Supply to drainage channels during the sluice period T :

(T = 7.3 hrs) x 31.250 m^3/hr = 228.248 m^3.

Storage decrease in drainage channels during sluice period T:

389.153 - 228.248 = = 160.905 m^3

The average decrease in polderwater level at the end of the drainage period T is calculated as follows :

- The surface area of the drainage channels is calculated : (see Figure V - 2.1 and ANNEX V - 3).

section :	a-b	b-c	c-d	d-e	e-a*	b-f	f-b*	c-g	g-c*
length (m)	1600	900	1400	600	2300	1200	1400	1400	1100
bottom width (m)	13	13	10	10	9	10	5	10	5
Channel width (m) at surface +1,15m	19,3	26,7	21,4	19,8	17,7	23,0	15,4	21,0	14,0
surface area (m^2)	46880	24030	29960	10880	40710	27600	21560	29400	15400

total area : 247,400 m^2.

- Additional storage area available in secondary and tertiary drainage channels :

  assume surface width at + 1.15 m to be 3 m.

  total length : 29,600 m. (depending on the features of the area under consideration).

  area : 3 x 29.600 = 88,800 m^2.

- Additional storage provided by the low pockets in the area which are in open connection with the drainage channels : 1% of total area = 250,000 m^2.

The total storage are is thus 247.400 + 88,800 + 250.000 = 586,200 m^2.

The average drop of the polderwater level at the end of the drainage period is then : $\frac{160.905}{586.200} = 0.27$ m.
yielding on average level of + 1.15 m - 0.27 m = + 0.88 m.

However immediately after the drainage stops, the water level will still show a gradient, with the level near the sluice below this + 0.88 m and at the end of the drainage channel above this + 0.88 m.

This level at the end of the drainage channel at the end of the drainage period T has to be found by trial and error.

In this example this is worked out with the help of the diagrams of ANNEX V - 3 resulted in a level of + 0.93 m at the end of the drainage channel and + 0.66 m near the sluice for a channel discharge of $Q = 14.8$ m^3/s.
The storage decrease in this situation is :
- from + 1.15 m to + 0.93 is 0.22 m, times the total storage area :

  0.22 x 586.200 = 129.000 m^3

- under the backwater curve starting at + 0.93 m :

section	a-b	b-c	c-d	d-e	e-a*	b-f	f-b*	c-g	g-c*
length (m)	1600	900	1400	600	2300	1200	1400	1400	1100
bottom width (m)	13	13	10	10	9	10	5	10	5
width at surface + 0.93 m	28,0	15,5	20,1	18,5	16,5	21,8	14,1	19,7	12,7
surface area(m^2)	44800	22950	28140	11100	37950	26160	19740	27580	13970

Total area (m^2)	232.400

section	a-b	b-c	c-d	d-e	e-a*	b-f	f-b*	c-g	g-c*
difference between +0.93 m level and backwater curve level starting from +0.93 m	0.22	0.18	0.16	0.12	0.04	0.18	0.16	0.18	0.15
Volume under +0.93 backwater curve (m^3)	9856	4131	4502	1332	1518	4708	3158	4964	2095
total volume (m^3): 36.260									

The total storage decrease is now 129.000 + 36.260 = 165.260 m^3 which is in good accordance with the aforementioned 160.905 m^3.

The polder water level near the sluice during the sluice period T is thus approximated from + 1.15 m at the start of T, to + 1.13 m shortly after the start, to + 0.66 at the end of T.

The average polder water level at the sluice is thus :

$$H_x = \tfrac{1}{2}\,(1.13 + 0.66) = + 0.90 \text{ m}$$

The invert level of the sluice was established at - 0.80 m.

Assumed flow condition 5 during most of the sluice period, with an average headwater of [0.90 - (- 0.80 m)] = 1.70 m, then the average sluice discharge per unit width is $q = 1.35 \times (H_x)^{1.5} = 1.35 \times (1.70)^{1.5} = 2.99 \text{ m}^2/\text{s}$.

The required average sluice discharge is 14.8 m^3/s, yielding a required width of $\frac{14.8}{2.99} = 4.95$ m.

The simplification made in above calculation is to assume that the discharge through the channel-sluice system remains constant during the sluice period T at $Q_{av} = 14{,}8 \text{ m}^3/\text{s}$.

In fact this discharge will be higher than average at the beginning , resulting in a lower level of H_x at the sluice, and lower than average at the end of T, resulting in a higher H_x. However, averaging the values of H_x will yield almost the same average H_x during the sluice period T.

The physical features are further highlighted in ANNEX V - 3.

With a floodrouting procedure, a more accurate determination of the sluice discharge during the drainage period T can be made as is demonstrated in ANNEX V - 3 d), but above method gives a result which will be within 10% of the exact results.

A check is made on the sluice discharge at this invert level and the polderwater level at maximum (+ 1.30 m) with the calculated backwater at the sluice of + 1.28 m.

Sluice discharge : $Q = 1.35 \times 4.95 \times (1.28 + 0.80)^{1,5} = 20.1\ m^3/s$.

Channel discharge was 17.36 m^3/s for the calculated backwater. A correction can be made on this by increasing the channel discharge, which will bring down the backwater and thus decrease the sluice discharge. This can be done graphically by using the information of the delivery curves as presented in ANNEX V - 3.

For various channel discharges the corresponding H_x is plotted against Q with H_i remaining constant at + 1.30 m. This yields line 'a' in Figure V - 4.8.

In the same graph the sluice discharge is plotted against the headwater H_x. Since flow condition 5 was assumed this sluice discharge is :

$$Q = 1.35 \times 4.95\ (H_x + |-0.80|)^{1.5}$$

These plots represent the line 'b'.

Where 'a' and 'b' intersect, sluice discharge and channel discharge are equal. This yields a backwater H_x = 1.25 and a corresponding Q_{max} = 19.6 m^3/s.

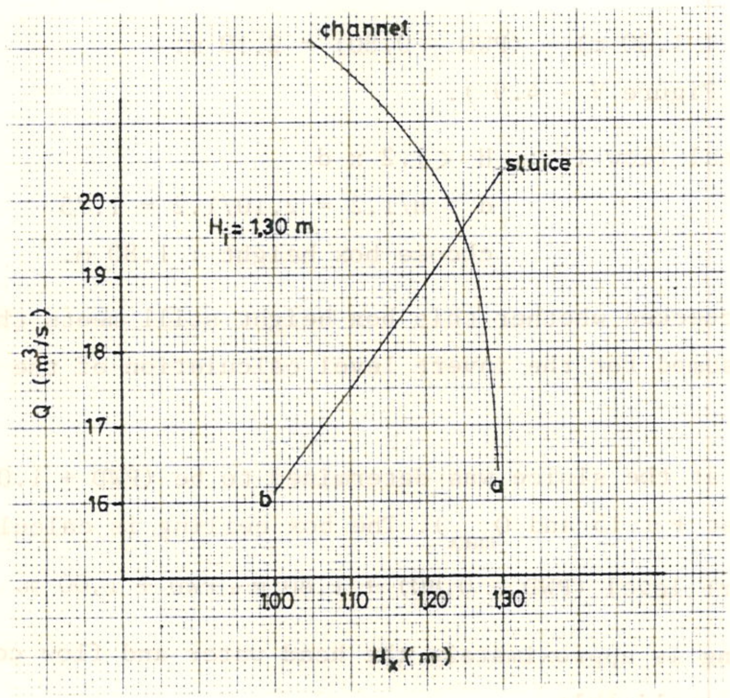


Figure V - 4.8

The maximum velocity in the sluice will then be :

$$q = h \times v_{max} = \frac{(v_{max})^3}{g} \quad ; \quad q = \frac{Q}{4.95} = 3{,}96 \text{ m}^2/\text{s}.$$

It follows that $v_{max} = 3.39$ m/s ($<$ 3.5 m/s !).

If invert level would have been chosen at - 0.90 m the required sluice width would be 4.55 m and the maximum velocities would have been upto 3.5 m/s.

The final choice of invert level and sluicewidth should be made, considering the economy of the whole construction, the foundation possibilities and the maximum velocities occuring in the sluice. It may be more economic to construct a sluice width only three vents and a deeper invert level if foundation does not cause any problems.

On the other hand a four vent sluice might be more expensive in concrete work but due to the smaller invert level, foundation may be cheaper and protection works may be less expensive because of lower extreme velocities.

<u>The height of the box should be such that during maximum drainage conditions supercritical flow in the sluice is avoided.</u>

In our example :

From maximum storage level to invert level is :

(PWD + 1.30 m) - (PWD - 0.80) = 2.10 m.

(see Figure V - 4.9).

No supercritical flow if : $H < 1.2 \times d$

$d > H/1.2 = 2.10/1.2 = 1.75$

choose box height : 1.80 m.

It should be checked whether this box height still meets the flow conditions assumed for the invert level calculation of the previous pages.

The headwater at the sluice was determined to be (PWD + 1.03). (polderwater at + 1.15 and Q_{max}). The box ceiling is calculated to be:

invert level (PWD - 0.80) + 1.80 m = PWD + 1.00 m.

So, box ceiling is approximately the head water and flow condition 5 is indeed applicable.

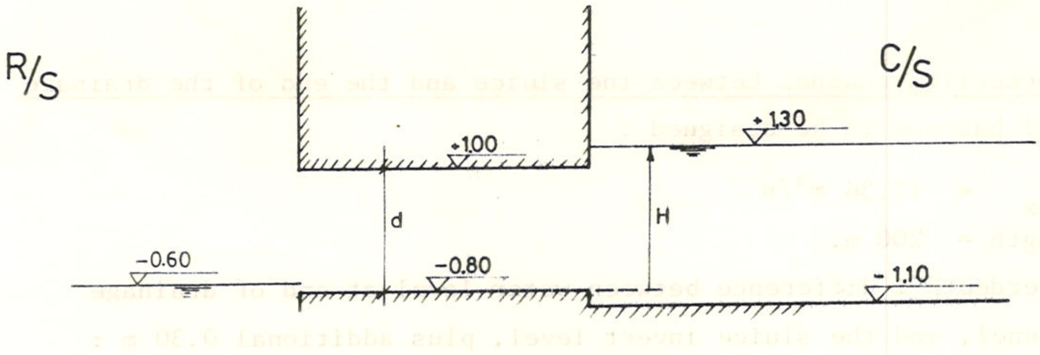


Figure V - 4.9

A sketch lay out of the sluice basic dimensions can now be made (see Figure V - 4.10) in which length of the box, length and height of the wingwalls is evaluated in order to economise on the concrete work.

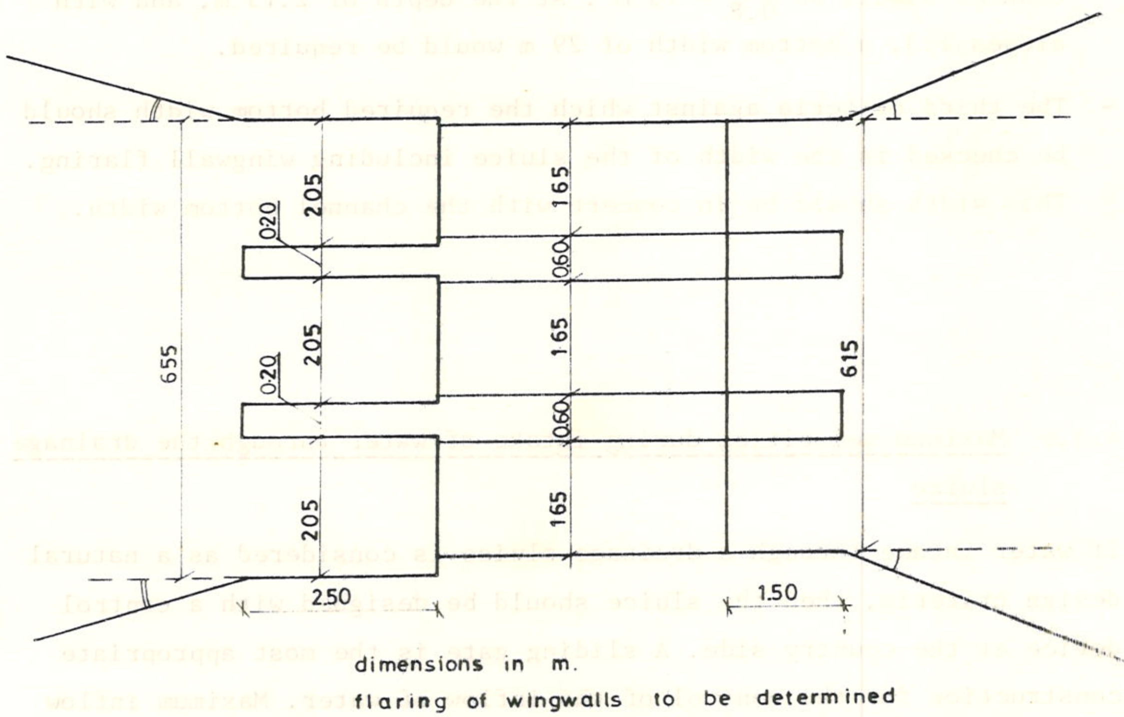


Figure V - 4.10

The connecting channel between the sluice and the end of the drainage channel has now to be designed :

- Q_{max} = 17.36 m^3/s
  length = 200 m.
  waterdepth : difference between water level at end of drainage channel, and the sluice invert level, plus additional 0.30 m :
  $+ 1.03 + |- 0.80\ m| + 0.30 = 2.13$ m.
  Slopes : 1 : 3
  formula: $Q = \frac{1}{h} \cdot A \cdot R^{2/3} \cdot S^{\frac{1}{2}}$
  unknown yet : bottomslope and width.
  choose bottom slope : 8×10^{-4}
  it follows that the bottom width should be : 2.60 m.

- However in case of inlet of water the discharge through the channel will be much higher, say 60 m^3/s. In order to keep velocities within limits, say 0.8 m/s, the profile of the channel should be $\frac{60}{0.8} = 75\ m^2$. At the depth of 2.13 m, and with slopes 1:3, a bottom width of 29 m would be required.

- The third criteria against which the required bottom width should be checked is the width of the sluice including wingwall flaring. This width should be in concert with the channel bottom width.

4.2.6 Maximum velocities during intake of water through the drainage sluice

If water intake through a drainage sluice is considered as a natural design criteria, then the sluice should be designed with a control device at the country side. A sliding gate is the most appropriate construction for the control of the inflow of water. Maximum inflow can be restricted by limiting the maximum clearance that can be given between the bottom of the lifted sliding gate and the invert level.

If intake of water is not a regular operation criteria for the drainage sluice and no specific constructions for the control of inflow are included in the design of the sluice, it still is advisable to check what flow conditions may occur in case the flapgate of the drainage sluice is tied up and the flow is permitted to enter the polder freely through the sluice.

As explained before the boundary conditions for this extreme situation are :

- Outside water level at maximum stage (highest high water spring tide).
- Polderwater level at design drainage level minus 0.3 m.

The calculation process for the determination of the velocities in the drainage sluice and the design of the country side wing-wall and reception bed construction are further on demonstrated using the following basic dimensions of the sluice.

Invert level at - 0.65 m.
Ceiling of the box at + 0.95 m.
Sluice width : 4 vents of 1.50 m wide = 6.00 m.
Water level outside at + 2.70 m (HHWS)
Polderwater level at + 1.20 m.
Crest of the embankment at + 4.20
C/S slope of embankment 1:2
Drainage channel bottom at - 0.90 m.
bottomwidth 13 m.
slopes 1:3.

The flow through the regulator at this stage is shown in Figure V - 4.11.

Assume flow condition (1), both inlet and outlet of the box submerged (see Chapter 3, paragraph 3.1).

$$Q = C_1 . A . \sqrt{2g . \Delta H}$$

For the outside water level, H_o (energy head) may be assumed equal to h_o (waterdepth) since the velocity head $\frac{(v_o)^2}{2g}$ is very small.

For the inside water this may not be true, therefore as a first approximation of the discharge :

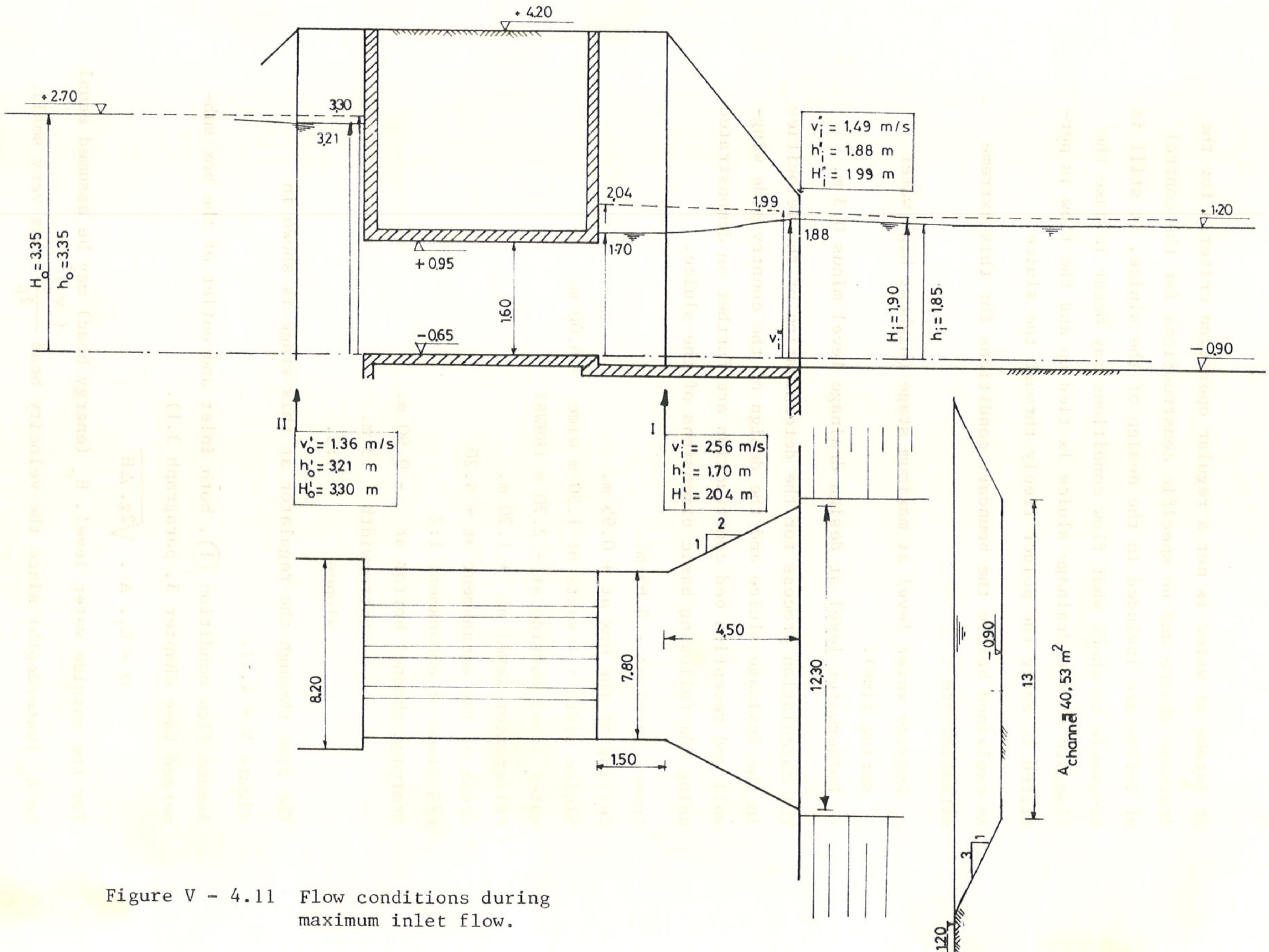


Figure V - 4.11 Flow conditions during maximum inlet flow.

$$Q = 0.82 \times (6.00 \times 1.60) \times \sqrt{2 \times 9.8 \times (3.35 - H_i)}$$

$$= 34{,}85 \sqrt{(3.35 - H_i)} \quad \ldots\ldots\ldots \quad \ldots\ldots\ldots \quad (1)$$

applying Bernouilli :

$$H_i = h_i + \frac{(V_i)^2}{2g} = 1.85 + \frac{(Q_i)^2}{2g.\ (A_i)^2} \quad \ldots\ldots\ldots \quad (2)$$

Substituting (1) in (2) yields :

$$H_i - 1.85 = 0.0377\ (3.35 - H_i)$$

$$H_i = 1.90 \text{ m}$$

and:

$$Q = 42.0 \text{ m}^3/\text{s}$$

The change in energy head from 3.35 m at the river side to 1.90 m at the country side can be expressed in the following energy losses:

$$\Delta H = \Delta H_{con} + \Delta H_{ent} + \Delta H_{exit} + \Delta H_{exp}$$

in which:

ΔH_{con} = contraction losses from the natural channel to the box entrance.

ΔH_{ent} = entrance losses

ΔH_{fr} = friction losses in the box

ΔH_{exit} = exit losses

ΔH_{exp} = expansion losses of the flow from the box exit to the natural channel.

From hydraulic hand books it is assumed that : (ref. 2).

$$\Delta H_{con} = k_{con} \times \frac{(v'_o)^2}{2g} \quad ; \quad k_{con} = 0.06$$

$$\Delta H_{exp} = k_{exp} \times \frac{(v'_i - v_i)^2}{2g} \quad ; \quad k_{exp} = 0.29$$

For contraction and expansion coefficients, see ANNEX V - 4.

For definition of v'_i, v_i and v'_o and v_o, see Figure V - 4.11.

For simplification, v'_o and v'_i are taken equal to the velocity in the box.

$$v'_i = v'_o = \frac{Q}{A_{box}} = \frac{42.0}{6.0 \times 1.6} = 4.38 \text{ m/s} \longrightarrow \Delta H_{con} = 0.06 \text{ m}$$

$$v_i = \frac{Q}{A_{ch}} = \frac{42.0}{40.53} = 1.04 \text{ m/s} \longrightarrow \Delta H_{exp} = 0.16 \text{ m}$$

The total of entrance, friction and exit losses for a square edged concrete box of 1.5 x 1.6 m^2, not longer than 10 m are already expressed in the coefficient $C_1 = 0.82$ of the discharge formula.

So : $\Delta H = 3.35 - 1.90 = 0.06 + \Delta H_{box} + 0.16$

$\Delta H_{box} = 1.23$ m.

and: $Q = 0.82 \times (6.0 \times 1.6) \times \sqrt{2g \times \Delta H_{box}} = 38.7 \text{ m}^3/\text{s}$

With this Q the previous calculations of velocity and energy heads has to be repeated until, by iteration, a best fit Q is found.

For our example it is found that $Q = 39.1 \text{ m}^3/\text{s}$

Now in Figure V - 4.11 the energy line and the water surface can be constructed.

$Q = 39.1 \text{ m}^3/\text{s}$

$$v_i = \frac{39.1}{40.53} = 0.96 \text{ m/s} \qquad \Delta H_{exp} = 0.14 \text{ m}$$

$$v_{box} = \frac{39.1}{6.0 \times 1.6} = 4.07 \text{ m/s} \qquad \Delta H_{con} = 0.05 \text{ m}$$

it yields: $H'_i = H_i + \Delta H_{exp} = 1.90 + 0.14 = 2.04$ m

$H'_o = H_o - \Delta H_{con} = 3.35 - 0.05 = 3.30$ m

At cross-section I : $\left.\begin{aligned} H'_i &= 2.04 = h'_i + \frac{(v'_i)^2}{2g} \\ v'_i &= \frac{39.1}{(h_i + 0.25) \times 7.8} \end{aligned}\right\} \begin{aligned} h'_i &= 1.70 \text{ m} \\ v'_i &= 2.56 \text{ m/s} \end{aligned}$

At cross-section II : $\left.\begin{aligned} H'_o &= 3.30 = h'_o + \frac{(v'_o)^2}{2g} \\ v'_o &= \frac{39.1}{(h'_o + 0.30) \times 8.2} \end{aligned}\right\} \begin{aligned} h'_o &= 3.21 \text{ m} \\ v'_o &= 1.36 \text{ m/s} \end{aligned}$

To check if at any stage supercritical flow will occur, the Froude number at the box entrance is calculated:

$$F = \frac{v}{\sqrt{gh}} = \frac{4.07}{\sqrt{9.8 \times 1.6}} = 1.0$$ at the box exit the

flow is critical. This means that no definite jump will occur and since there is ample expansion possibility for the flow, no additional energy dissipating measures are necessary. The maximum velocity that will occur in the drainage channel to the sluice in case of flushing operation will thus be :

$$v_i = \frac{Q}{A_{ch}} = \frac{39.1}{40.53} = 0.96 \text{ m/s}$$

In case this velocity appears to be too high, the channel profile has to be enlarged and the calculations have to be repeated.

The transition of flow from the box exit to the profile of the bottom drainage channel is provided by the apron and the area with protection.

The velocity at the end of the apron is determined by the flaring angle of the wingwalls and the length of the apron. The wingwall flaring is determined by :

$$\tan \alpha = \frac{1}{3F} \quad (F = \text{Froude number}) = \frac{v}{\sqrt{gh}} = \frac{2.56}{\sqrt{9.82(1.70+0.25)}}$$

$$\tan \alpha = \frac{1}{1.75} \quad : \quad \text{choose } 1/2$$

For our example the length of the apron is 4.5 m (see Figure V - 4.11). The width of the country side apron at the end of the apron is thus 12.30 m (see Figure V - 4.11).

At the end of the apron the flow velocity of $v'_i = 2.56$ m/s will then be reduced to $v''_i = 1.49$ m/s.

Computation goes as follows :

$$H'_i = h'_i + \frac{(v'_i)^2}{2g} = 2.04 \text{ m}$$

$$H''_i = H'_i - \Delta H_{exp}$$

$$\Delta H_{exp} = 0.68 \times \frac{(v'_i - v''_i)^2}{2g}$$ (see ANNEX V - 4).

$$Q = b \times v \times h$$

$$39.1 = 12.3 \times v''_i \times (h''_i + 0.25)$$

From these equations it follows that : $h''_i = 1.88$ m

$v''_i = 1.49$ m

$H''_i = 1.99$ m

In order to reduce velocity further down to an acceptable value of 0.8 m/s, the length of the apron and the retreading wingwalls could be extended further. However, it is more economic to end apron and wingwalls at the toe of the embankment and construct a further smooth transition to the eventual channel profile by means of an adequate bottomprotection that can resist the determined currents.

The bottomprotection should be designed to resist currents upto 1.49 m/s. For details of the bottomprotection, reference is made to paragraph 5.5.4; Volume III, Chapter 5 dealing with revetments. The length of the bottomprotection in the channel direction, measured from the end of the apron should be equal to the bottom width of the drainage channel or 10 times the waterdepth, which ever is the greater.

Note: As shown the velocity in the box will exceed the maximum value of 3.5 m/s, of course it would be possible to design the sluice in such a way that even flushing velocities occuring during mal-operation can be sustained without causing any significant damage.

The frequent reported damages due to scouring of the drainage channel section at the country side of the sluice is probably a result of such mal-operation.

For instance, if the polder water level has decreased to 1.2 m below the ground level, which is quite normal in case of tidal drainage in the winter season, and then the flap gates are fixed in open position at low tide to let in water in the next high tide, the currents in the drainage channel will be very high because the tide is rising faster than the water level in the polder can follow.

To reduce velocities sufficiently the capacity of the drainage channel would have to be far greater than required for drainage.

4.2.7 Maximum velocities during drainage operation

Maximum drainage velocity will occur when the polder water level is at maximum stage and the outer water is at minimum tide level (LLW). In fact this condition will only occur in exceptional circumstances. Normally the polder water level near the sluice will start to fall as soon as the sluice starts to discharge. A backwater curve will develop in the drainage channel.

In our example the following boundary values have been adopted for the water levels (see paragraph 4.2.5 page 51).

country side : + 1.30 m at the end of the drainage channel.

river side : LLW = 0.9 m below PWD

The flow through the sluice in this situation is sketched in Figure V - 4.12).

The flow condition will be 5 and thus: (see Figure V - 4.12).

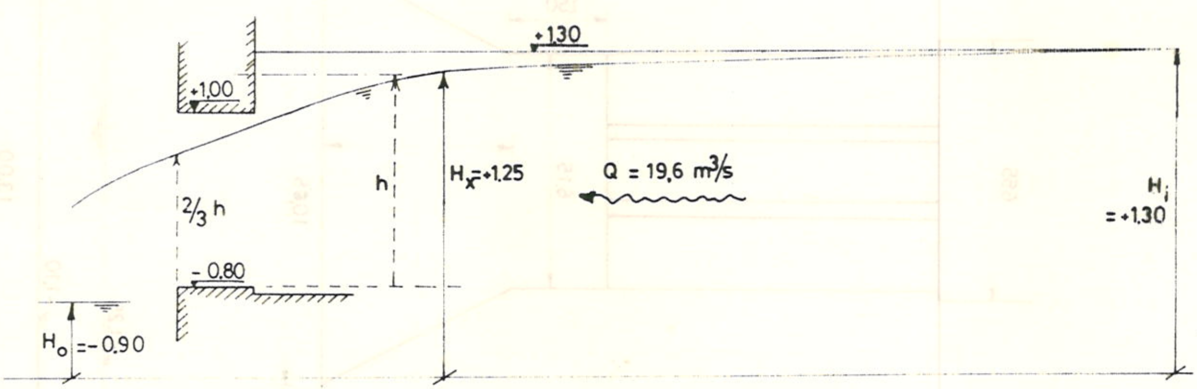


Figure V - 4.12 Flow condition with draw-down curve during maximum drainage situation.

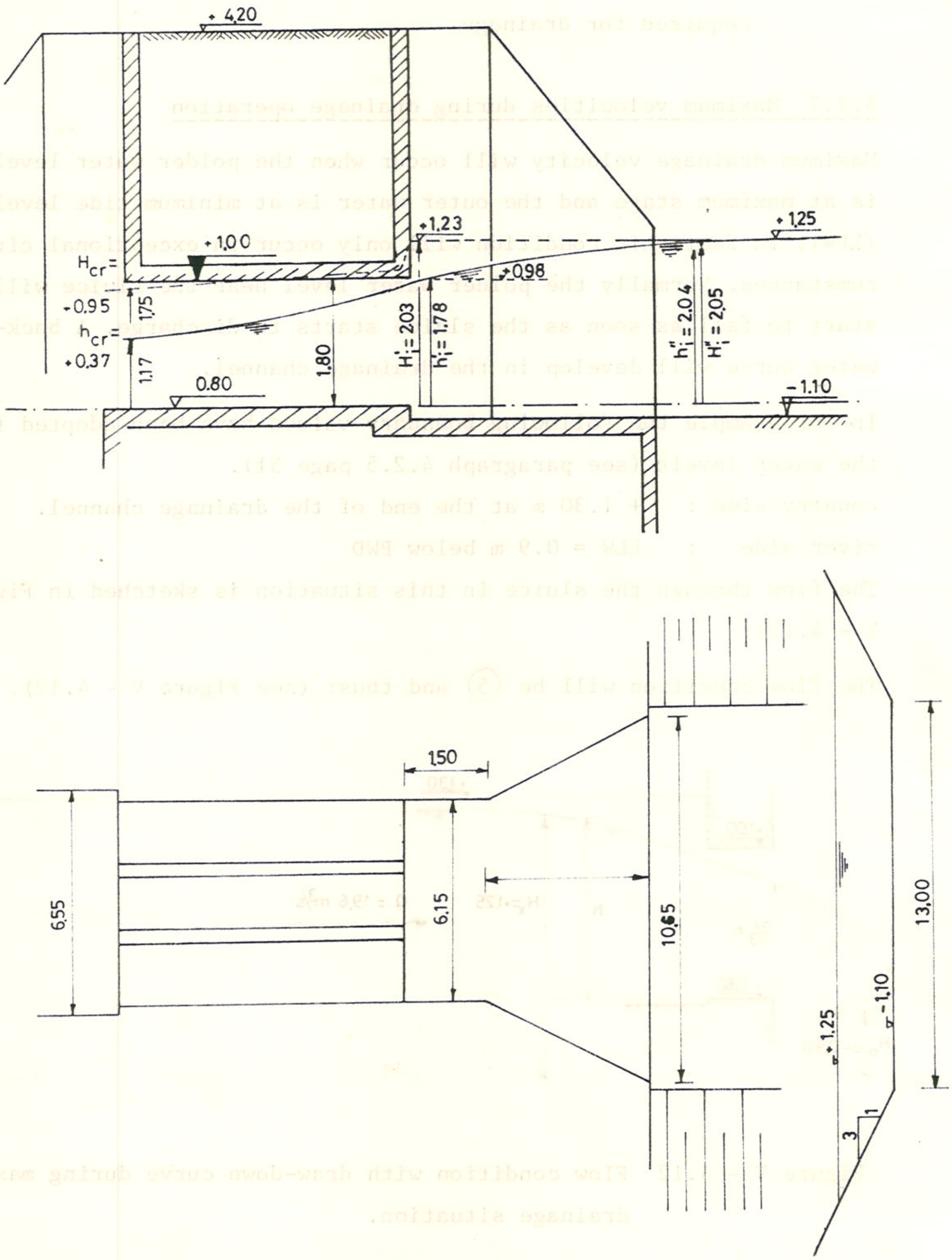


Figure V - 4.13 Flow condition during maximum drainage flow.

In the drainage channel near the sluice : (see Figure V - 4.13).

$$Q''_i = 19.6 \text{ m}^3/\text{s}$$

$$H''_i = 1.25 \text{ m} + |-1{,}10 \text{ m}| = 2.35 \text{ m}$$

$$A = 47.1 \text{ m}^2$$

$$v''_i = 0.42 \text{ m/s}$$

$$\frac{(v''_i)^2}{2g} = 0.01 \text{ m}$$

$$h''_i = 2.34 \text{ m}$$

By calculating the energy head losses in the same way as in paragraph 4.2.6, the velocity and waterdepth at the box entrance can be computed.

$$\Delta H_{con} = k_{con} \times \frac{(v'_i)^2}{2g} = 0.06 \times \frac{(v'_i)^2}{2g}$$

$$H_i = h'_i + \frac{(v'_i)^2}{2g} = H''_i - \Delta H_{con}$$

$$Q'_i = v'_i \times h'_i \times b'_i = 19.6 \text{ m}^3/\text{s}$$

$$v'_i \times h'_i = \frac{19.6}{4.95} = 3.96 \text{ m}^2/\text{s}.$$

From these equations it follows that : $H'_i = 2.03$ m

$h'_i = 1.78$ m

$v'_i = 2.22$ m/s.

It appears that the energy loss due to the contraction of the flow from the channel profile to the profile just in front of the box is only 0.02. Therefore it is not necessary to compute a new discharge for the sluice in which this energy loss is considered. The calculation therefore continues with $Q = 19.6 \text{ m}^3/\text{s}$.

Because of the low tail water the sluice will act as a weir and at the box exit, where the discharge control is, critical depth will occur :

$$q = \frac{19.6}{4.95} = 3.96 \text{ m}^2/\text{s}.$$