$$h_{cr} = \sqrt[3]{\frac{q^2}{g}} = 1.17 \text{ m}$$

$$v_{cr} = \sqrt{g.\ h_{cr}} = 3.39 \text{ m/s}$$

$$H_{cr} = 1.75 \text{ m}$$

The maximum velocity in the box will be : $v_{max} = 3.39$ m/s.

At the box uxit there are two options for the flow to continue :

- 1. The flow makes a drop to reach immediately the head level of the outer water.
- 2. The river side apron is at slightly depressed level (invert level minus 0.3 m) to allow sufficient space for the flap gate operation, and is continuing upto the toe of the embankment.

In the first case a provision has to be designed to still the water jet which falls from the box exit.

In the second case the flow velocity is first decreased on the expanding apron with flaring wingwalls. At the end of the apron it has to be checked whether the discharge per unit width has sufficiently decreased to make a transition to the natural channel with or without a stilling basin or other energy dissipating measures.

The two cases are further elaborated in the following paragraphs.

4.2.8 Drop of flow at barrel exit, stilling basin design

The flow makes the drop to the lower energyhead of the outfall channel straight after the box exit. A stilling basin has to be designed using the graph presented as Figure V - 4.14.

$$q = \frac{19.6}{6.55} = 2.99 \text{ m}^2/\text{s}.$$

Energyhead at box-exit $H_1 = 1.75 + 0.15 = 1.90$ m (see Figure V-15).
(+ 0.15 m is taken to have a convenient reference - plane).
downstream energyhead $H_2 = 0.05$ m.

For the use of the graph of Figure V - 4.14 , the following values

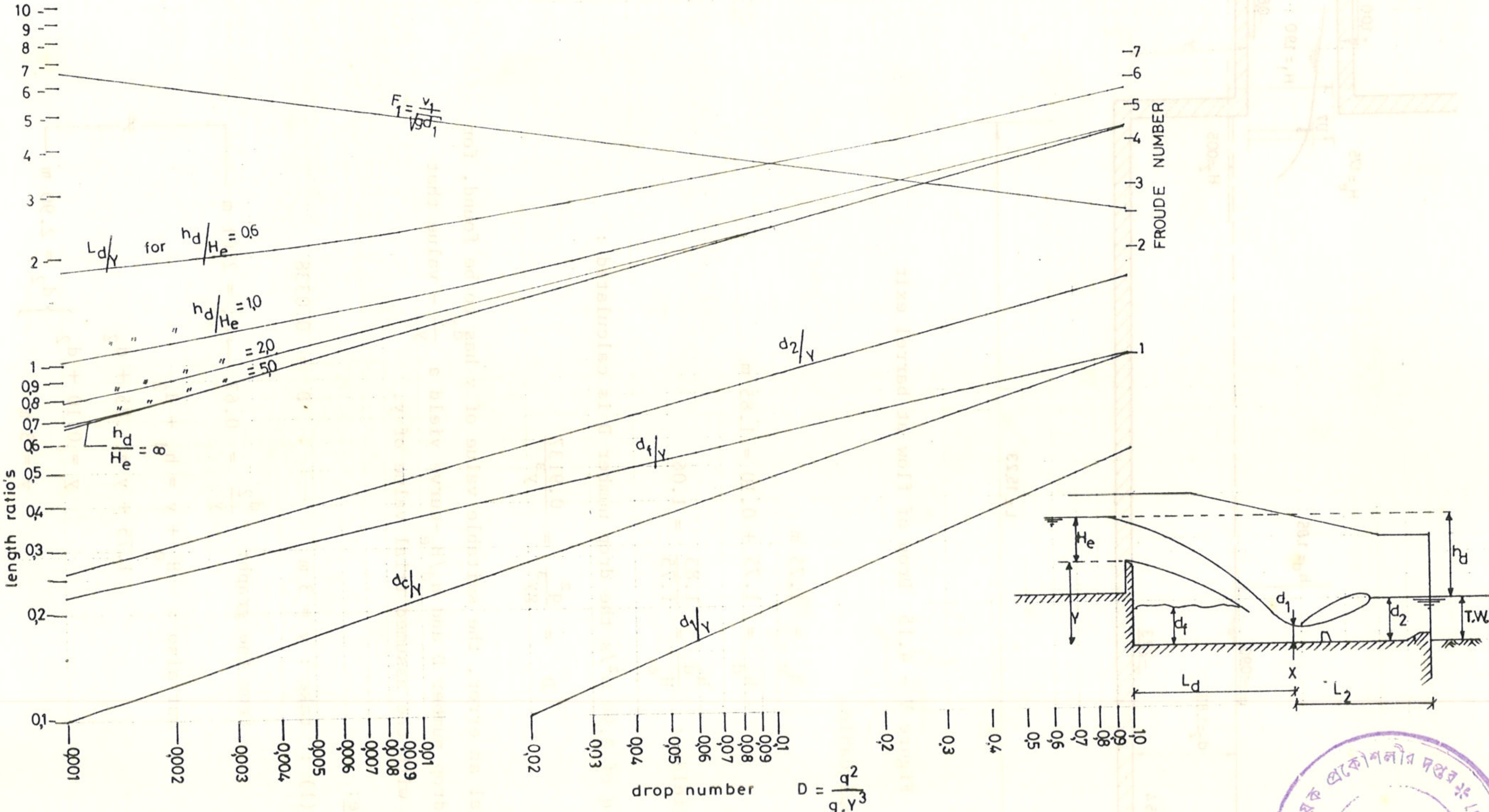


Figure V - 4.14 Hydraulic characteristics of strait drop of flow. (ref. 7)

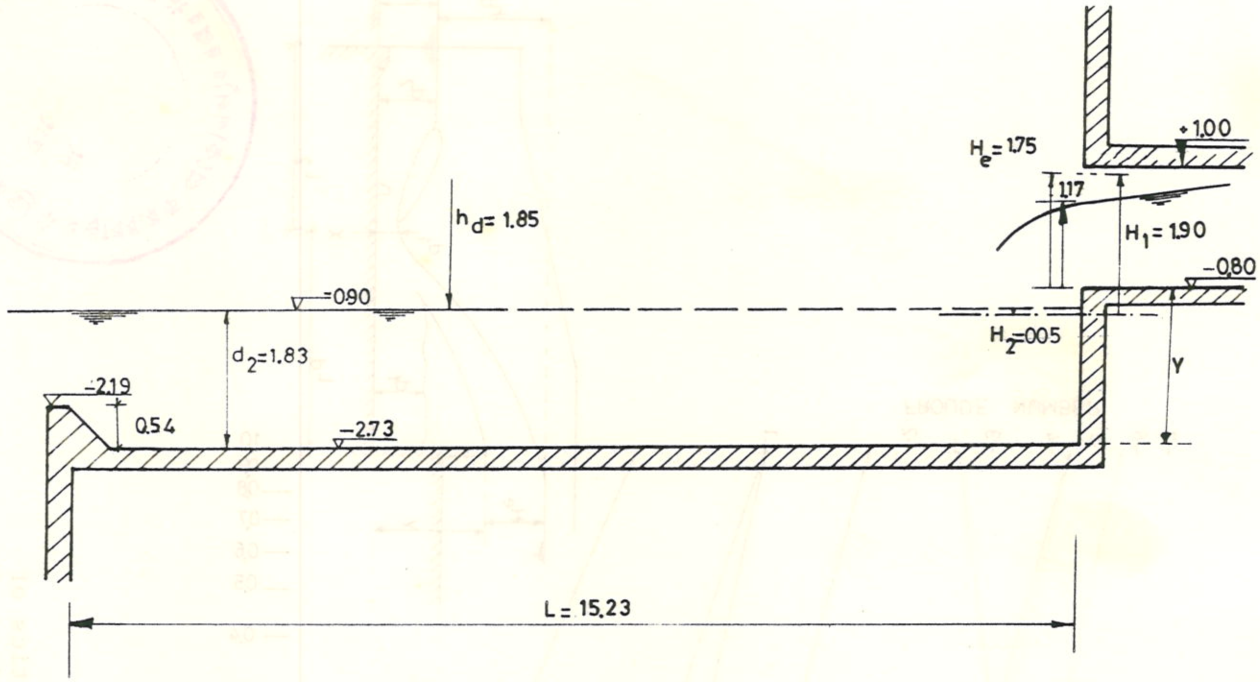


Figure V - 4.15 Drop of flow at barrel exit

are applicable.

$$H_e = 1.75 \text{ m}$$

$$h_d = 1.75 + 0.10 = 1.85 \text{ m}$$

This yields : $\frac{h_d}{H_e} = \frac{1.85}{1.75} = 1.06$

For a q of 2.99 m^2/s the drop number D is calculated :

$$D = \frac{q^2}{gy^3} = \frac{0.9137}{y^3}$$

By trial an error, the suitable value of y has to be found, for which drop number D and h_d/H_e-curve yield a $\frac{d_2}{y}$ - value that agrees with the assumed trial value of y.

<u>example</u>:

trial (1) : take : y = 3 m $\qquad$ D = 0.0338

from the graph: $\frac{d_2}{y} = 0.67 \longrightarrow d_2 = 2.01$ m

but also : $H_e + y = h_d + d_2$

$$1.75 + y = 1.85 + d_2$$

$$\left.\begin{array}{l} y = 0.10 + d_2 \\ y = 3 \text{ m} \end{array}\right\} d_2 = 2.90 \text{ m}$$

$$d_2 = 2.01 \text{ m} \neq d_2 = 2.90 \text{ m}$$

trial (2) : take $y = 2.50$ $\qquad D = 0.0585$

from the graph: $\dfrac{d_2}{y} = 0.775 \rightarrow d_2 = 1.94$ m

but also : $\left.\begin{array}{l} y = 0.10 + d_2 \\ y = 2.50 \text{ m} \end{array}\right\} d_2 = 2.40$ m

$1.94 \text{ m} \neq 2.40 \text{ m}$

trial (3) : take $y = 1.93$ m $\qquad D = 0.1271$

from the graph : $\dfrac{d_2}{y} = 0.95 \longrightarrow d_2 = 1.83$

but also : $\left.\begin{array}{l} y = 0.10 + d_2 \\ y = 1.93 \text{ m} \end{array}\right\} d_2 = 1.83$

$1.83 = 1.83$

If the elevation of the tailwater is at 0.90 m - PWD then the bottom of the stilling basin should thus be at 2.73 m - PWD.

The length of the basin can be found as follows

From the graph of Figure V - 4.14 it is found that $\dfrac{L_d}{y} = 2.68$,

for $\dfrac{h_d}{H_e} = 1.06$ and $D = 0.1271$

It follows that $L_d = 2.68 \times y = 2.68 \times 1.93 = 5.17$ m.

The additional length L_2 can be found from Figure V - 4.16 and the Froude number as calculated for point X in Figure V - 4.14 :

$$F_1 = \frac{v_1}{\sqrt{gd_1}} \longrightarrow \frac{d_1}{y} = 0.223$$

$$d_1 = 0.223 \times 1.93 = 0.43$$

$$F_1 = \frac{q_1/d_1}{\sqrt{gd_1}} = \frac{2.99/0.43}{\sqrt{9.8 \times 0.43}} = 3.39$$

From Figure V - 4.16 it follows : $\dfrac{L_2}{d_2} = 5.5$

$$L_2 = 5.5 \times 1.83 = 10.06 \text{ m}$$

The total length of the stilling basin is then :

$$5.17 \text{ m} + 10.06 \text{ m} = 15.23 \text{ m}$$

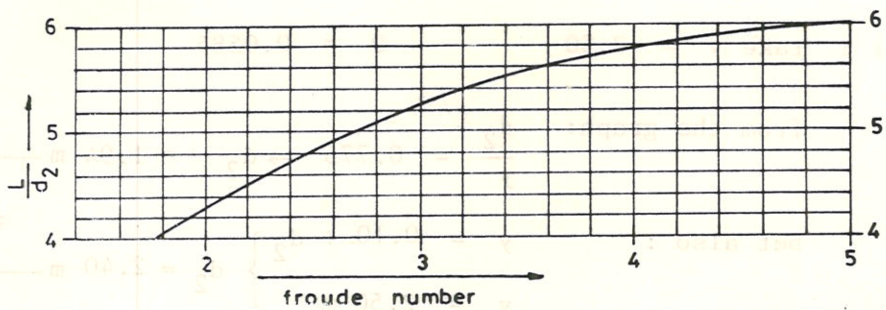


Figure V - 4.16 Length of hydraulic jump.

The height of the sill at the end of the stilling basin should be $1.25 \times d_1$ or, in this case $1.25 \times 0.43 = 0.54$ m.

The lay-out of the construction could then be as shown in Figure V - 4.17.

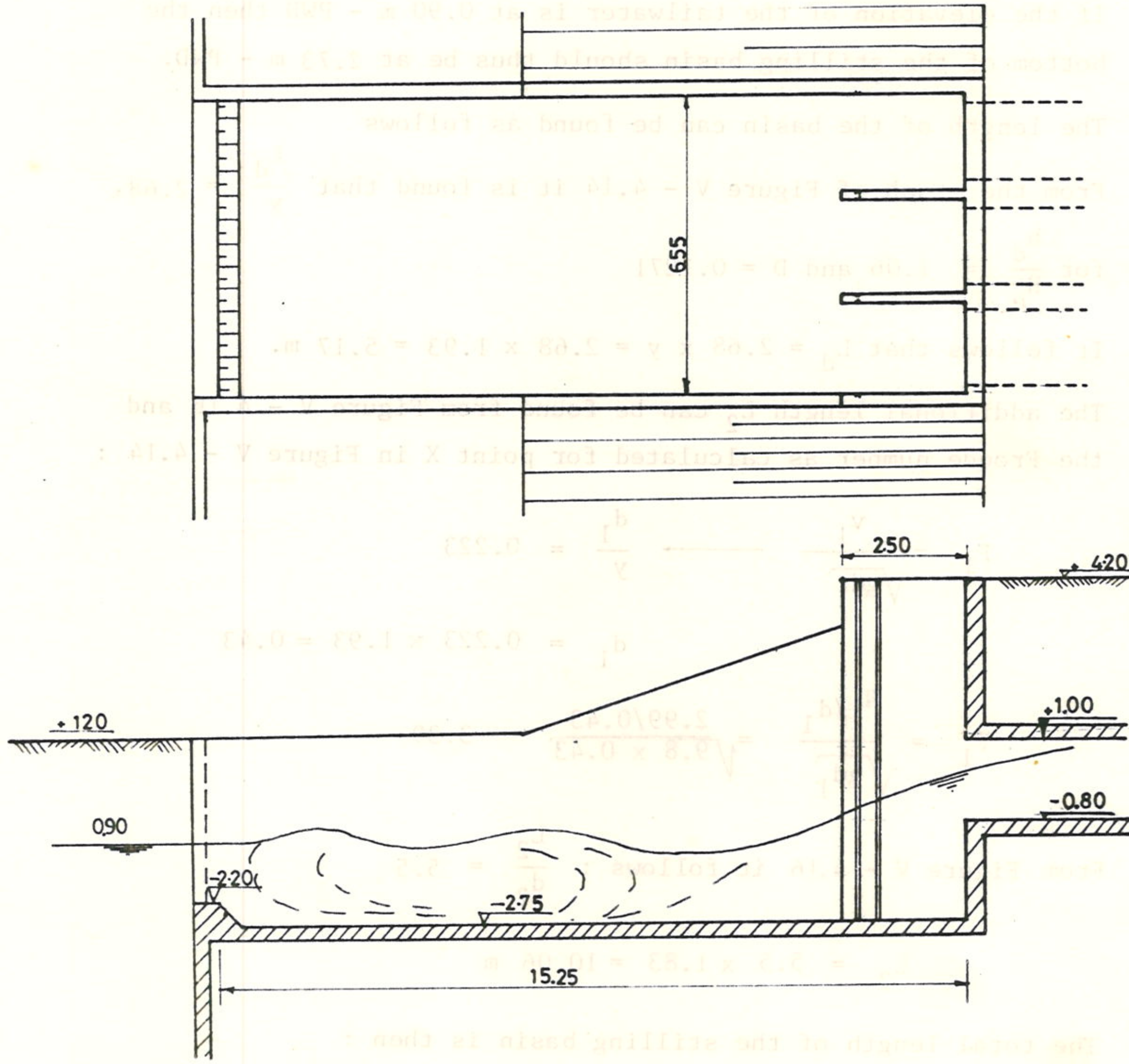


Figure V - 4.17 Lay out of stilling basin

The dimensions of the outfall channel are now to be determined. According to the outline presented in Chapter 1, paragraph 1.3.2, the ratio b/h should be 3 and side slopes 1:3. At the tail water elevation of 0.9 m - PWD this would yield an outfall channel bottom at 2.90 m - PWD and bottom width 6 m. However, the width of the stilling basin is 6.55 (equal to the width of the sluice, see Figure V - 4.17) and the sill depth is at (- 2.73 m + 0.54 m) $\approx$ 2.20 m - P.W.D.

The channel and bottom protection are now designed as follows. The bottom protection beyond the sill will be constructed at the same level as the sill (2.20 - PWD). The water depth will be in this case be 2.20 - 0.90 = 1.30 m. The required profile for a maximum velocity of 0.8 m/s and a discharge of 19.6 m^3/s is $24\frac{1}{2} m^2$. The bottom width of the channel with 1.30 m waterdepth and 1:3 slopes should then be 15 m.

If excavation allows, it would be better to provide a profile which is more in concert with the ultimate natural profile of the unprotected part of the outfall channel. If excavation to a depth of 2.90 m - PWD is possible then the solution could be as shown in Figure V - 4.18.

For the design of the protection layer reference is made to Volume III. In any case the bottom protection should be of a flexible type as to be able to follow any scour that may develop.

As can be seen, a very big excavation has to be made and high wingwalls have to be constructed. Therefore it seems a better solution first to decrease the flow velocity via the apron which is at slightly lower elevation than the box-invert. The widening of the apron by the flaring wingwalls causes the discharge per unit width to decrease. This alternative solution is discussed in the following paragraph.

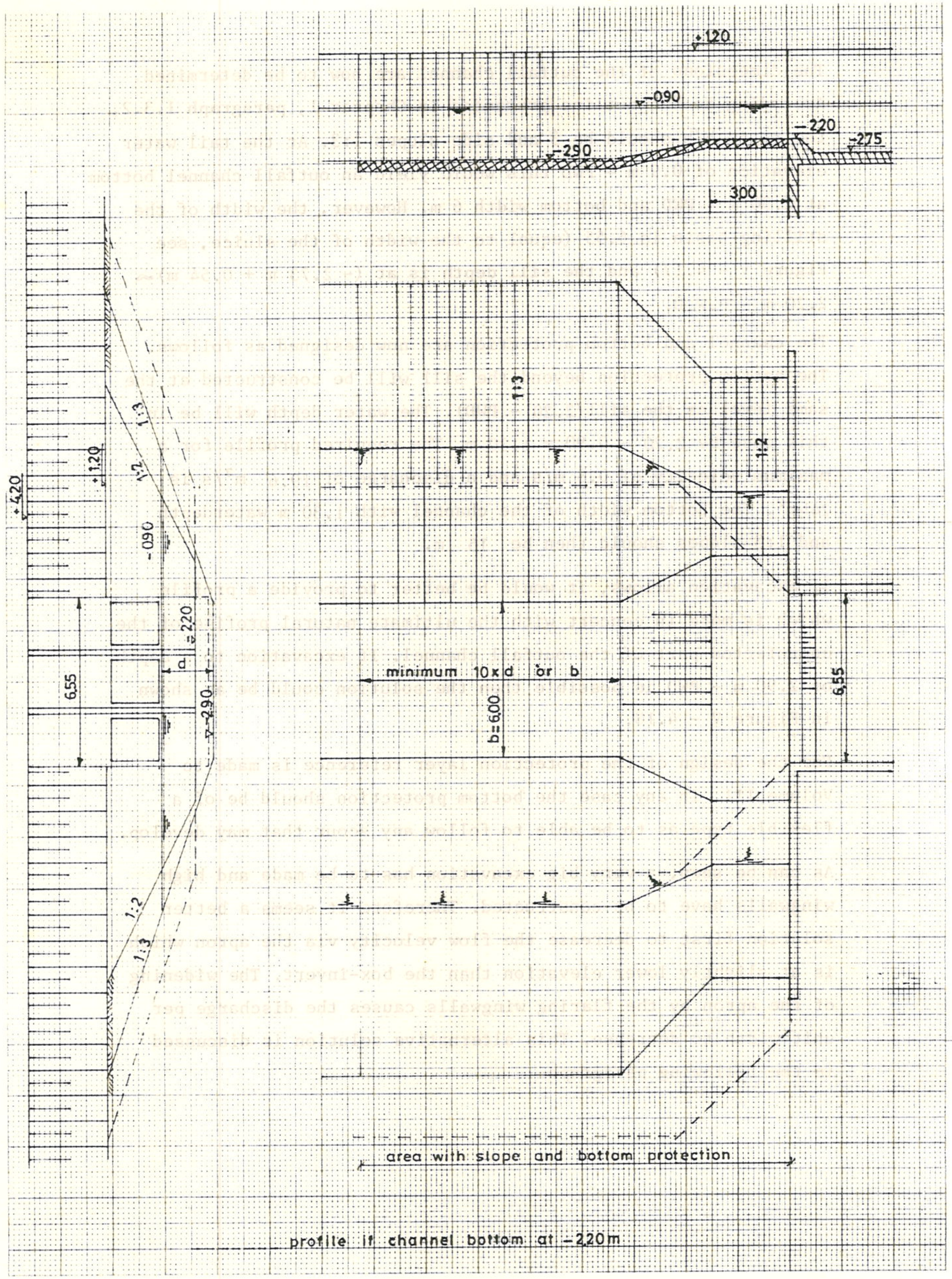


Figure V - 4.18 Lay-out of bottom and slope protection for a sluice with a stilling basin.

4.2.9 Drop of flow at end of apron, baffle block design

At the box exit the floor level is making a drop of 0.3 m and at the same time the profile is widened to 6.55. The profile is further widened between flaring wingwalls upto the end of the apron. Energy losses due to this expansion are analysed as follows (see Figure V - 4.19).

From box-exit to the point were the wingwalls provide further widening :

$$H_{exp} = 0.82 \times \frac{(v_{cr} - v'_o)^2}{2g} \quad \text{...... (see ANNEX V - 4)}$$

$$H'_o = H_{cr} - H_{exp} = 1.75 - 0.82 \frac{(3.39 - v'_o)^2}{2g} = h'_o + \frac{(v'_o)^2}{2g}$$

Continuity of flow : $(h'_o + 0.3) \times v'_o \times 6.55 = 19.6 \ m^3/s$

yields that : $\left.\begin{array}{l} h'_o = 1.48 \ m \\ v'_o = 1.68 \ m/s \end{array}\right\} H'_o = 1.62 \ m$

Checking the Froude number $F = \frac{v}{\sqrt{gh}}$ yields $F = \frac{1.68}{\sqrt{9.8 x (1.48+0.30)}} = 0.40$

so subcritical flow.

Expansion of the flow on the apron continues between the flaring wingwalls.

Required wingwalls flaring : $\tan \alpha = \frac{1}{3F}$ = take 1:1½

Length of apron is 6.80 m, thus the width of the apron at the end is 15.61 m.

The flow at the end of the apron is calculated as follows :

$$H''_o = H'_o - H_{exp} = 1.62 - 0.78 \frac{(v'_o - v''_o)^2}{2g} = h''_o + \frac{(v''_o)^2}{2g}$$

$$1.62 - 0.78 \frac{(1.68 - v''_o)}{2g} = h''_o + \frac{(v''_o)^2}{2g}$$

continuity of flow : $v''_o \times (h''_o + 0.30) \times 15.61 = 19.6 \ m^3/s$

it follows that : $\left.\begin{array}{l} h''_o = 1.57 \ m \\ v''_o = 0.68 \ m/s \end{array}\right\} H''_o = 1.55 \ m$ (PWD+0.75)

At the end of the apron the following situation is now created :

The discharge per unit width is $q = \frac{19.6}{15.6} = 1.25$ **m²/s.**

Considering the edge of the apron as a weir crest, the energy head above the crest is then calculated as :

$$H_1 = h_{cr} + \frac{(v_{cr})^2}{2g} = 1.5 \sqrt[3]{\frac{q^2}{g}} = 0.82 \text{ m.}$$

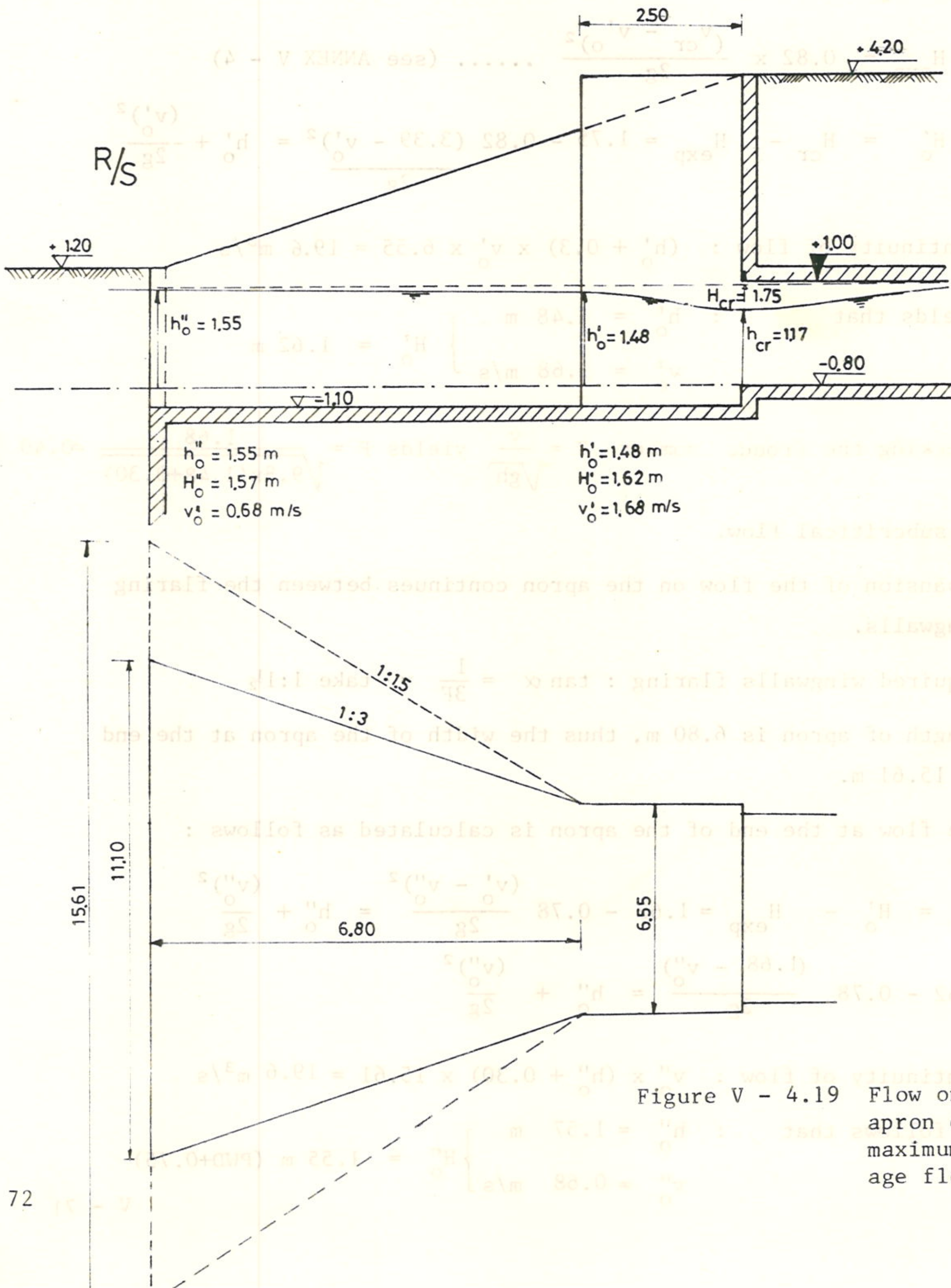


Figure V - 4.19 Flow on r/s apron during maximum drainage flow.

However H_1 was calculated to be 1.57

It is clear that this energy head cannot be attained and consequently a draw - down will form from the barrel exit to the apron and the flow on the apron can be calculated exactly with analyses of energy losses and water level and flow velocities can thus be calculated, but the result will not contribute essentially to the solution of the problem. It is therefore assumed that the flow on the r/s apron is just critical (F = 1) and the flaring of the wingwalls should thus not exceed 1:3.

The width of the apron at the edge becomes thus : $6.55 + (\frac{6.80 \times 2}{3}) =$ 11.10 m and the discharge per unit width :

$$q = \frac{19.6}{11.10} = 1.77 \ m^2/s$$

resulting in an energy head of $H_1 = 1.02$ m at the edge. At the down-stream section. (see Figure V - 4.20), the energy head remains $H_2 =$ 0.20 m.

R/S

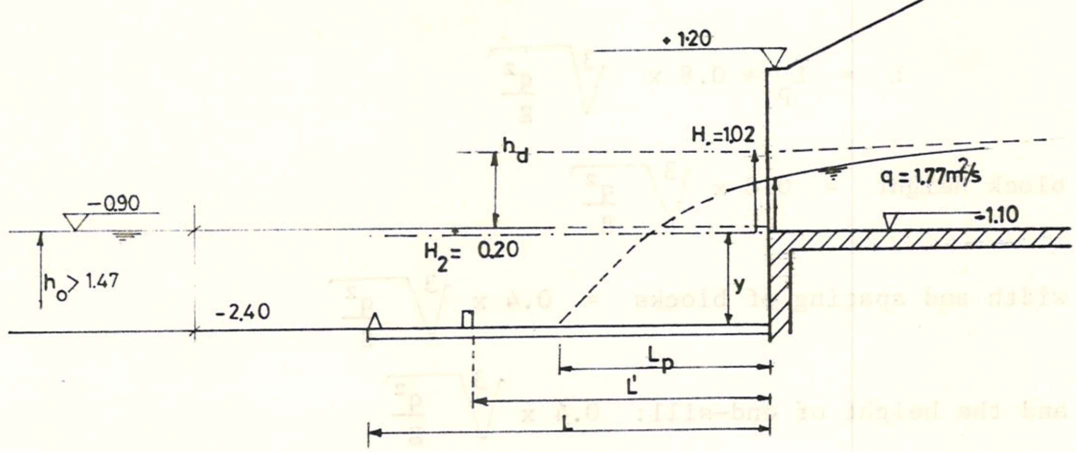


Figure V - 4.20 Flow situation at the end of the apron at lowest tailwater depth.

For the drop which the flow has to make now at the end of the apron, a stilling basin could be designed in the same way as in paragraph 4.2.8. However, since the head-difference and the discharge per unit width are relatively small, a solution can be applied using an impact block basin.

The dissipation of energy is now not taking place by means of a generated hydraulic jump, but by turbulencies induced by collision of the oncoming flow against the blocks.

A condition for the application of the baffle block solution is that the tailwater depth should be :

$$h_o \geqslant 2.15 \times \sqrt[3]{\frac{q^2}{g}}$$

If this condition is not met, then the stilling basin solution should be applied.

Figure V - 4.20 gives the layout of a baffle block solution, which should have the following dimensions :

$$L = L_p + 2.55 \times \sqrt[3]{\frac{q^2}{g}}$$

$$L = L_p + 0.8 \times \sqrt[3]{\frac{q^2}{g}}$$

block height $= 0.8 \times \sqrt[3]{\frac{q^2}{g}}$

width and spacing of blocks $= 0.4 \times \sqrt[3]{\frac{q^2}{g}}$

and the height of end-sill: $0.4 \times \sqrt[3]{\frac{q^2}{g}}$

Figure V - 4.21 shows a nomogram from the U.S. Bureau of Land Reclamation for the relation between q, L_p and y.

For our example: check tailwater : $h_o \geqslant 2.15 \times \sqrt[3]{\frac{q^2}{g}} = 1.47$ m

$$\frac{h_d}{H_e} = \frac{(1.02-0.20)}{1.02} = \frac{0.82}{1.02} = 0.804$$

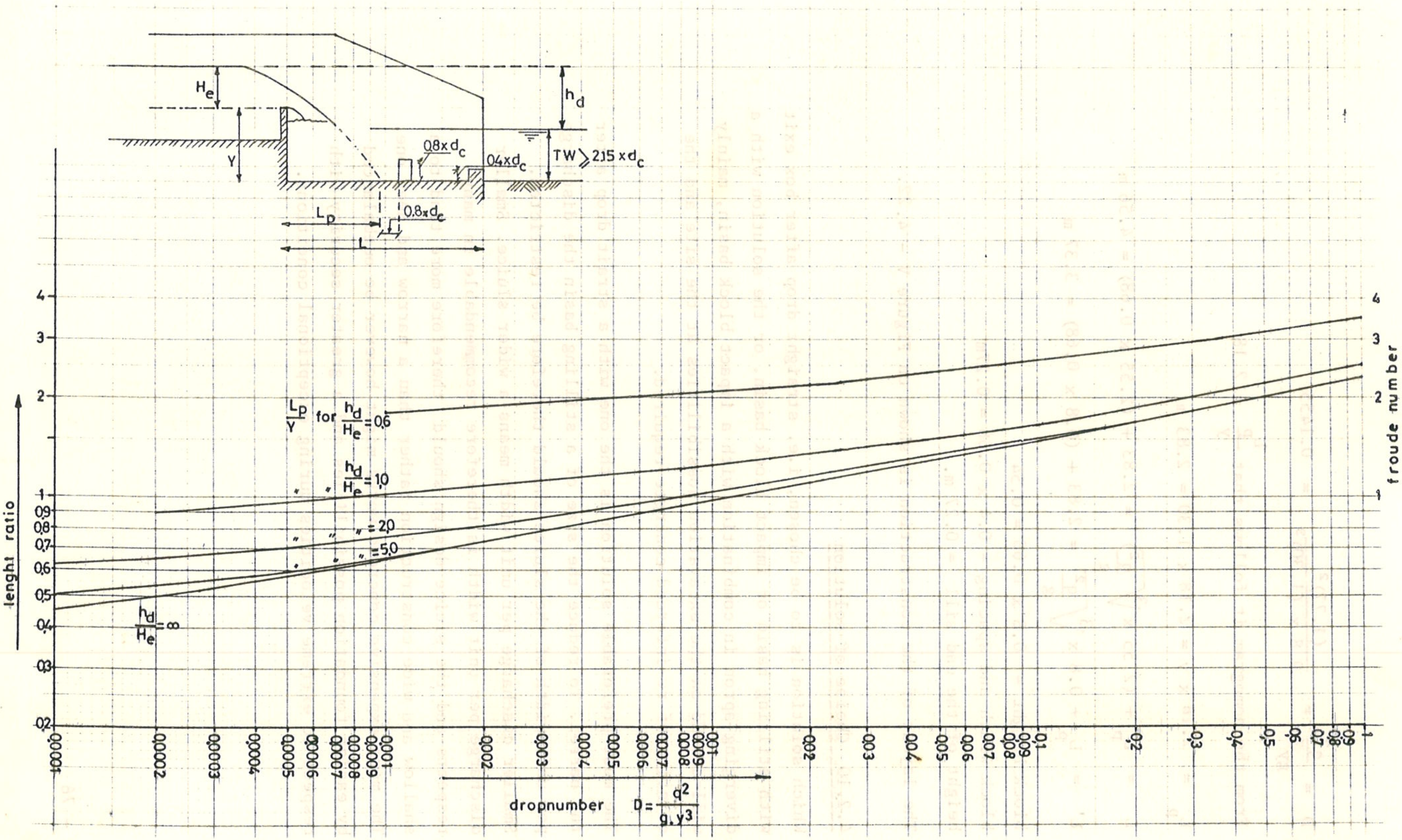


Figure V - 4.21 Hydraulic characteristics of straight drops with impact blocks (ref. 7).

$$D = \frac{q^2}{gy^3} = \frac{(1.77)^2}{9.8 \times (1.30)^3} = 0.1455$$

From the nomogram it follows that $\frac{L_p}{y} = 2.18$

$$L_p = 2.18 \times y = 2.18 \times 1.30 = 2.83 \text{ m}$$

$$L = L_p + (2.55 \times \sqrt[3]{\frac{q^2}{g}}) = 2.83 + (2.55 \times 0.68) = 4.56 \text{ m}$$

$$L' = L_p + 0.8 \times \sqrt[3]{\frac{q^2}{g}} = 2.83 + (0.8 \times 0.68) = 3.37 \text{ m}$$

block height = 0.8 x 0.68 = 0.54

block width and spacing = 0.4 x 0.68 = 0.27m

height of the end sill = 0.27 m.

The layout of the construction is shown on Figure V - 4.22.

4.2.10 Choise of solution

Which solution is to be chosen, i.e. straight drop after box exit with stilling basin or impact block basin, or the solution with a diverging apron in combination with a impact block basin, mainly will depend on the excavation possibilities at the site and the quantity of re-inforced concrete required.

The most preferable solution is the one with a strait drop after the barrel. To reduce the size of a stilling basin the discharge per unit width of the sluice should therefore be restricted. Smaller discharge per unit width means a wider sluice. Smaller discharge per unit width is therefore recommendable in many respects and the sluice design should therefore more tend to a shallow and wide construction rather than a narrow and deep one. The more expensive concrete works might however be outbalanced by easier foundation possibilities and a greater security with respect to extreme velocities during exceptional conditions.

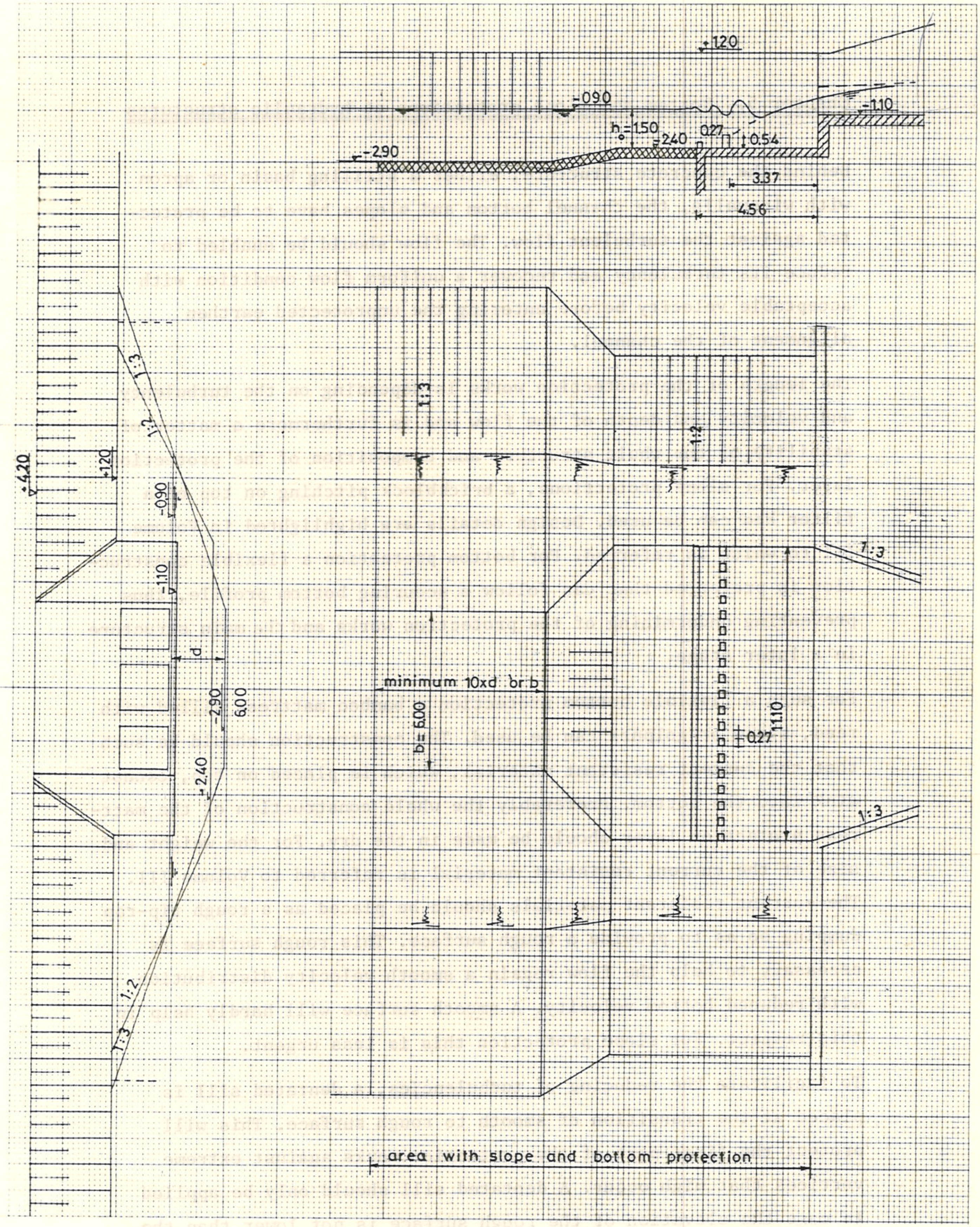


Figure V - 4.22 Lay-out of bottom and slope protection for a drainage sluice with apron and depressed baffle block floor.

Chapter 5. Bottom and slope protections at hydraulic structures

Behind the concrete structure, either a stilling basin or apron with wingwalls, the channel bottom and slopes have to be protected against the turbulent flow. The flow should be enabled to reduce its turbulency and recover a uniform flow condition with acceptable velocity before entering the unprotected earthen alignment of the channel.

The length of the protection works is depending on the turbulency and velocity and depth of the flow and is furthermore a matter of stability of the whole structure and composition of the protection layer. For slope protections, a brickblock pitching on top of a filter bed can be used. Design details are highlighted in Volume III to which is referred. For bottom protection a flexible construction is preferred that can follow a scouring bottom profile, thus preventing undermining of the protection works and the main structure in a later stage.

For such a flexible bottom protection a bamboo mattress filled with reed, straw or golpata can be used. The construction should be such that the current resistant material, which is placed on top, cannot sink into the bottom. Preferably the whole construction of the mattress protection layer should be made in the dry. For the weight and size of the current resistant material is referred to Volume III. The current resistant materials should be placed as a rough rip-rap dumping so as to provide a rough surface. This rough surface is necessary to help the flow regain a smooth velocity distribution with reduced bottom velocity. A smooth surface will hardly help in this respect. For slope protection this is less urgent.

To facilitate the reducing of turbulencies, a dentated sill is placed at the transition of smooth to rough surface. This will protect the first part of the protection works against extreme vortices that migh occur. A dentated sill should only be applied in case the elevation of the rough surface is not lower than the smooth surface.

Dimensions of a dentated sill are shown in Figure V - 5.1. In some cases, the solution of an impact block basin behind the apron may not be possible. For instance at sluices where the old protection layers have been scoured away and the erosion has reached dangerous depths close to the concrete structure. Even if filling up of the erosion hole is possible, this will not form a proper foundation for a impact block basin. In such cases the bottom-protection has to be constructed against the end of the apron or cut-off wall. Depending on the shape of the scour hole, it may be constructed sloping downwards. In Volume III, Chapter 5 an example of such erosion-repair work of the bottom and slope protection is given in Figure III - 5.10.

The bottom area of the channel, that should be provided with a protection layer has to have a minimum dimension in the direction of the sluice axis, equal to the bottom width of the channel or the width of the apron edge or ten times the waterdepth at critical conditions, whichever of the three dimensions is the biggest.

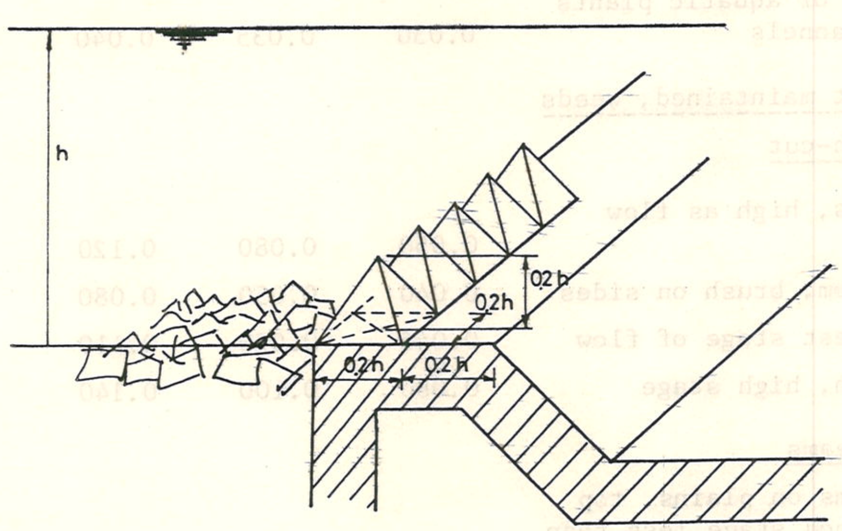


Figure V - 5.1 Dimensions of dentated sill.

ANNEX V - 1. MANNING'S ROUGHNESS COEFFICIENTS (n)

	minimum	normal	maximum
1. Pipes:			
- metal, wood, plastic, cement, precast concrete, asbestos cement	0.010	0.013	0.015
- masonry	0.025	0.030	0.035
2. Excavated channels:			
a. Earth, straight and uniform			
- clean, recently completed	0.016	0.018	0.020
- clean, after weathering	0.018	0.022	0.025
- with short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
- no vegetation	0.023	0.025	0.030
- grass, some weeds	0.025	0.030	0.033
- dense weed or aquatic plants in deep channels	0.030	0.035	0.040
c. Channels not maintained, weeds and brush un-cut			
- dense weeds, high as flow depth	0.050	0.080	0.120
- clean bottom, brush on sides	0.040	0.050	0.080
- same, highest stage of flow	0.045	0.070	0.110
- dense brush, high stage	0.080	0.100	0.140
3. Natural streams			
(minor streams on plains, top width at flood stage less than 30 m)			
- clean, straight full stage, no rifts or deep pools	0.025	0.030	0.033
- same, but more stones and/ or weeds	0.030	0.035	0.040
- clean, winding, some pools and shoals	0.033	0.040	0.045

mannings roughness coefficient	minimum	normal	maximum
- same, with some weeds and/ or stones	0.035	0.045	0.050
- same, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
- Sluggish reaches, weedy deep pools	0.050	0.070	0.080
- very weedy reaches, deep pools	0.075	0.100	0.150

ANNEX V - 2. NOMOGRAPH FOR MANNING'S FORMULA

In the Figures A - V - 2.1 to 2.6 some nomographs are presented for the Manning formula

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

- Figure A - V - 2.1

  With the hydraulic gradient S on the vertical axis and the curve corresponding with the selected value for the roughness coefficient, the value for $\frac{1}{n} . S^{\frac{1}{2}}$ is found on the horizontal axis.

- Figure A - V - 2.2

  With the design discharge on the vertical axis and the value $\frac{1}{n} . S^{\frac{1}{2}}$ as found in the previous step, the value for $A.R^{2/3}$ is found on the horizontal axis.

- Figure A - V - 2.3 to 2.5

  Choose the nomograph for the side slope which corresponds with the selected side slope. With the value for $A.R^{2/3}$ as determined in the previous step, a range of combinations for water depths and bottomwidths is found.

  Select the combination with approximately the best hydraulic section as described in paragraph 1.3 of Volume V. If the waterdepth for economical or practical reasons is determined this value should be used and the corresponding bottomwidth is determined.

- Figure A - V - 2.6

  With this graph the velocity can be determined using the value for the hydraulic radius on the vertical and the curve for the value of $\frac{1}{n} . S^{\frac{1}{2}}$ found in Figure A - V - 2.1.

  Example

  $Q = 0.80 \ m^3/sec$

  $S = 0.0004 \ m/m = 4 \times 10^{-4} \ m/m$

  $n = 0,025$

  side slope 1 : 1.5

- from Figure A - V - 2.1 : $\frac{1}{n} \cdot S^{\frac{1}{2}} = 0.8$
- in Figure A - V-2.2 :

$$\left.\begin{array}{l} Q = 0.80\ m^3/sec \\ \frac{1}{n} \cdot S^{\frac{1}{2}} = 0.8 \end{array}\right\} A{\cdot}R^{2/3} = 1.0$$

From Figure A - V - 2.4 some possible combinations for water depth and bottomwidth for $A{\cdot}R^{2/3} = 1.0$ are determined which are presented in the following table together with the $\frac{b}{h}$ ratio.

Table A - V - 1

Water depth (m)	bottom width (m)	$\frac{b}{h}$
0.90	0.55	0.61
0.80	0.94	0.18
0.70	1.45	2.07
0.60	2.05	3.42
0.50	3.0	6.0

- Figure A - V - 2.5 :

For z = 1.5, the ratio $\frac{b}{h}$ for the best hydraulic section is 0.61 (see paragraph 1.3 of Volume V).

So the first combination (h = 0.90, b = 0.55) gives the best hydraulic section. But for a drainage channel in low lying area a b/h ration of about 2 is more appropriate.

If the combination h = 0.70 m and b = 1.45 is selected the hydraulic radius R is calculated as follows :

Bottomwidth b = 1.45 m

topwidth cross-sectional area = b+ 2 x 1.5 h

= 1.45 + 3 x 0.70 = 3.55 m

$$\text{cross-sectional area} = \frac{\text{bottomwidth + topwidth x h}}{2}$$

$$= \frac{1.45 + 3.44}{2} \times 0.70 = 1.75\ m^2$$

$$R = \frac{A}{O}$$

O = wetted perimeter = the sum of the length of that part of the channel sides and bottom which are in contact with water.

$$= b + 2 \times h \sqrt{1 + z^2}$$

$$= 1.45 + 2 \times 0.70 \sqrt{1^2 + 1.5^2} = 3.97 \text{ m}$$

$$R = \frac{1.75}{3.97} = 0.44 \text{ m}$$

From Figure A - V - 2.6 :

$$\frac{1}{n} \cdot S^{\frac{1}{2}} = 0.8$$

$$R = 0.44$$

$$v = 0.37 \text{ m/s}$$

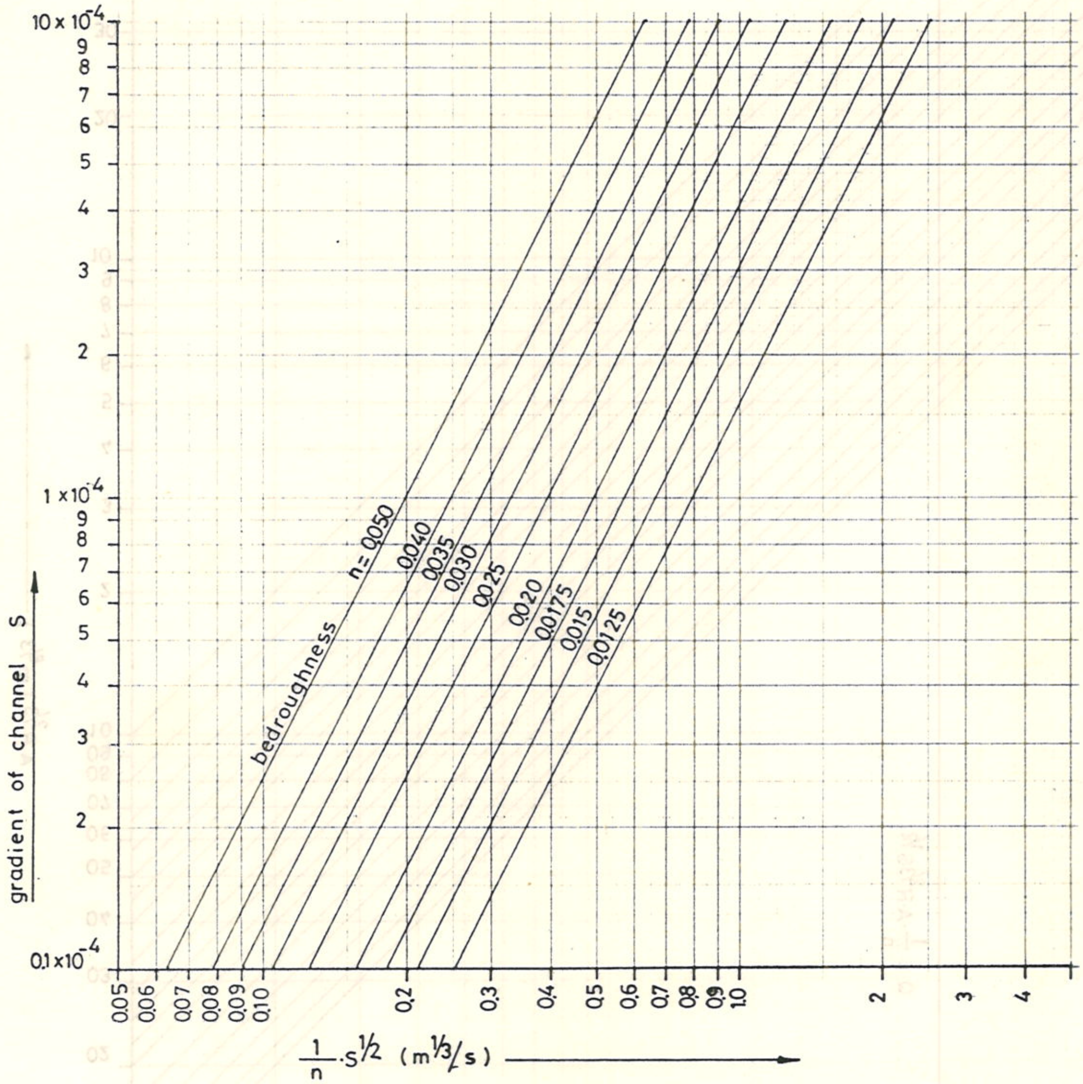
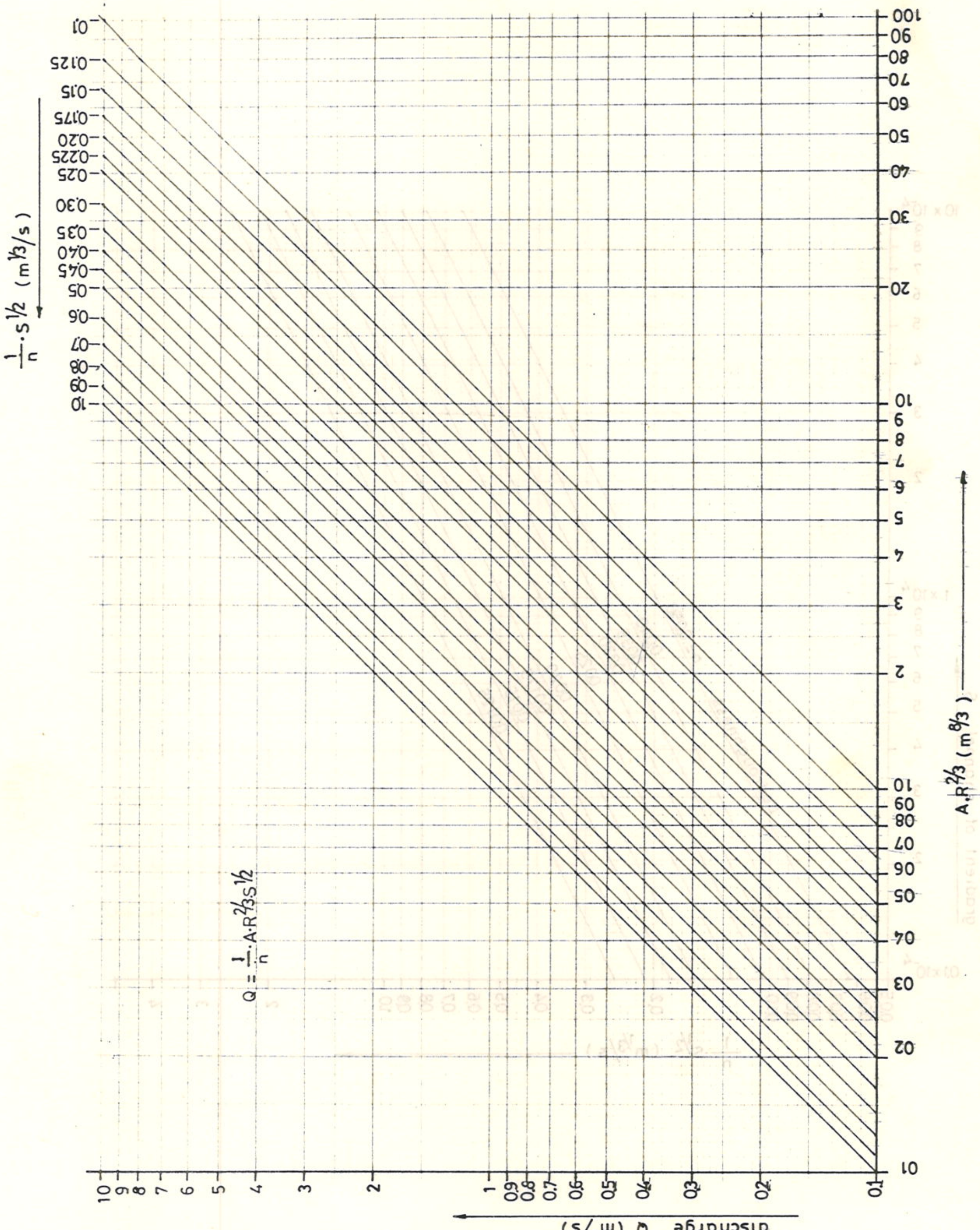

gradient of channel S
10 x 10^-4
1 x 10^-4
0,1 x 10^-4
bedroughness
n= 0,050
0,040
0,035
0,030
0,025
0,020
0,0175
0,015
0,0125
1/n · S^1/2 (m^1/3/s)

Figure A.V - 2.1



$Q = \frac{1}{n} \cdot A \cdot R^{2/3} S^{1/2}$
discharge Q (m^3/s)
$A.R^{2/3}$ ($m^{8/3}$)
$\frac{1}{n} \cdot S^{1/2}$ ($m^{1/3}/s$)

Figure A.V - 2.2

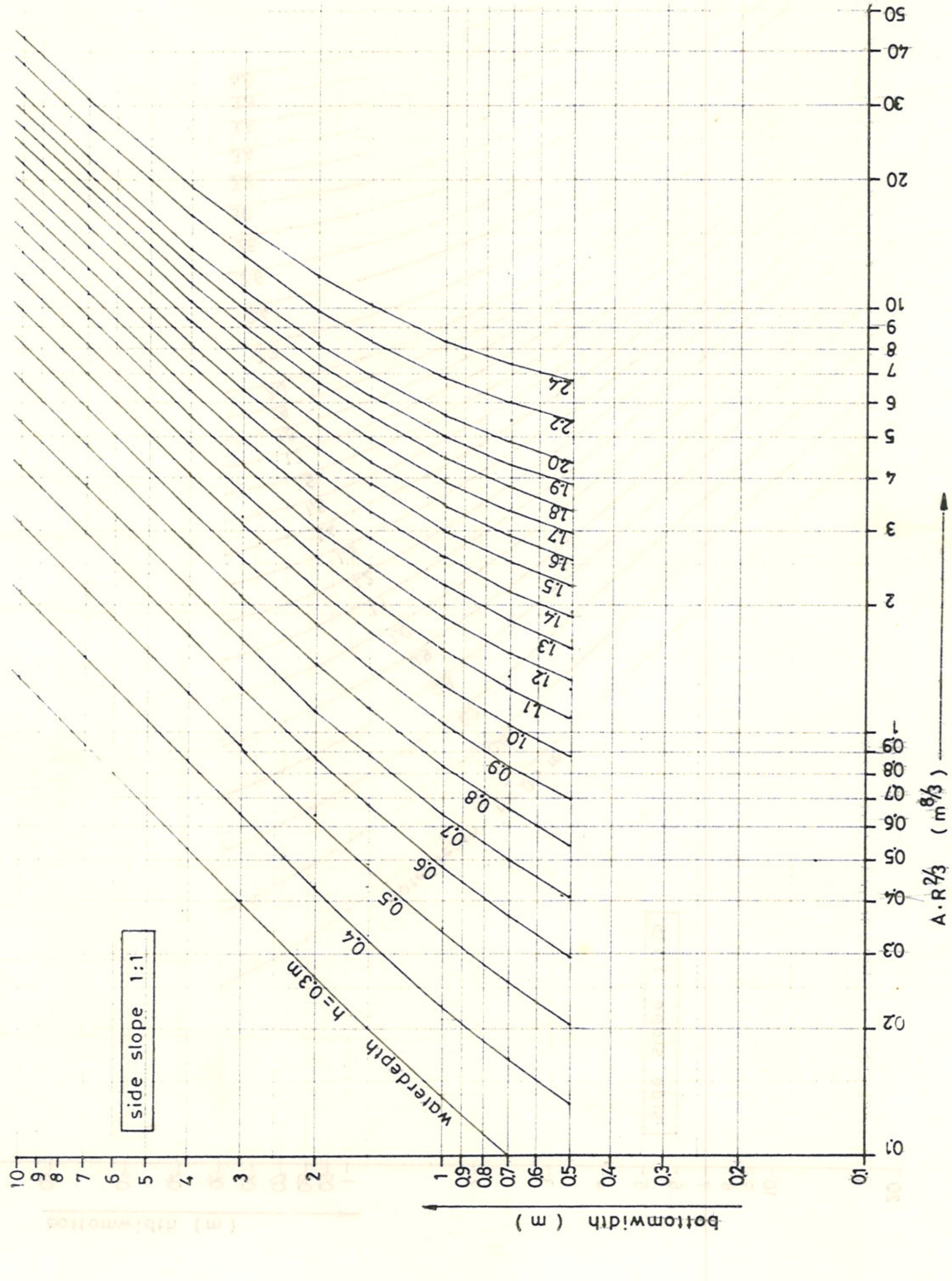

side slope 1:1
waterdepth h=0.3m
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2.0
2.2
2.4
bottomwidth (m)
A·R2/3 (m8/3)

Figure A.V - 2.3

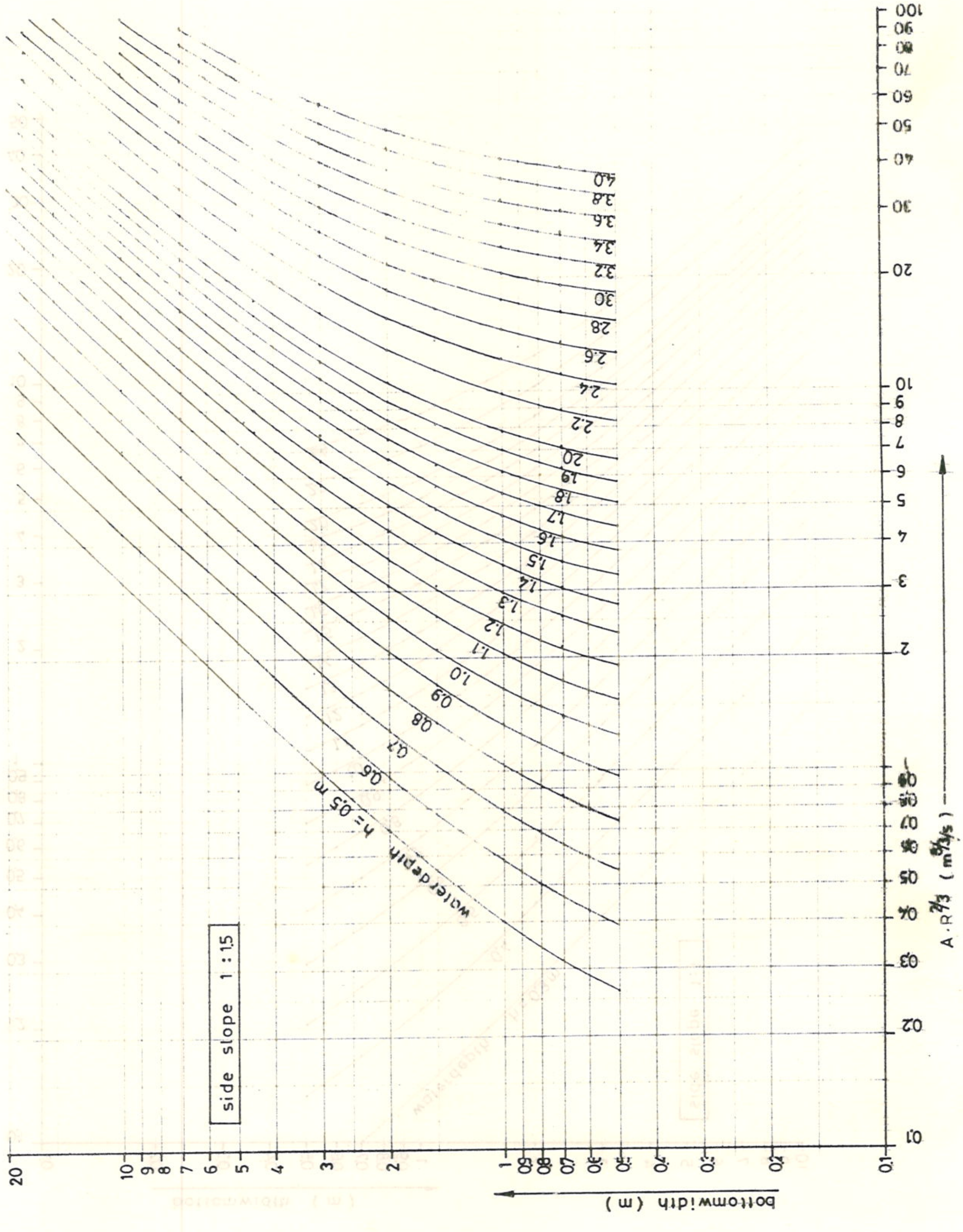
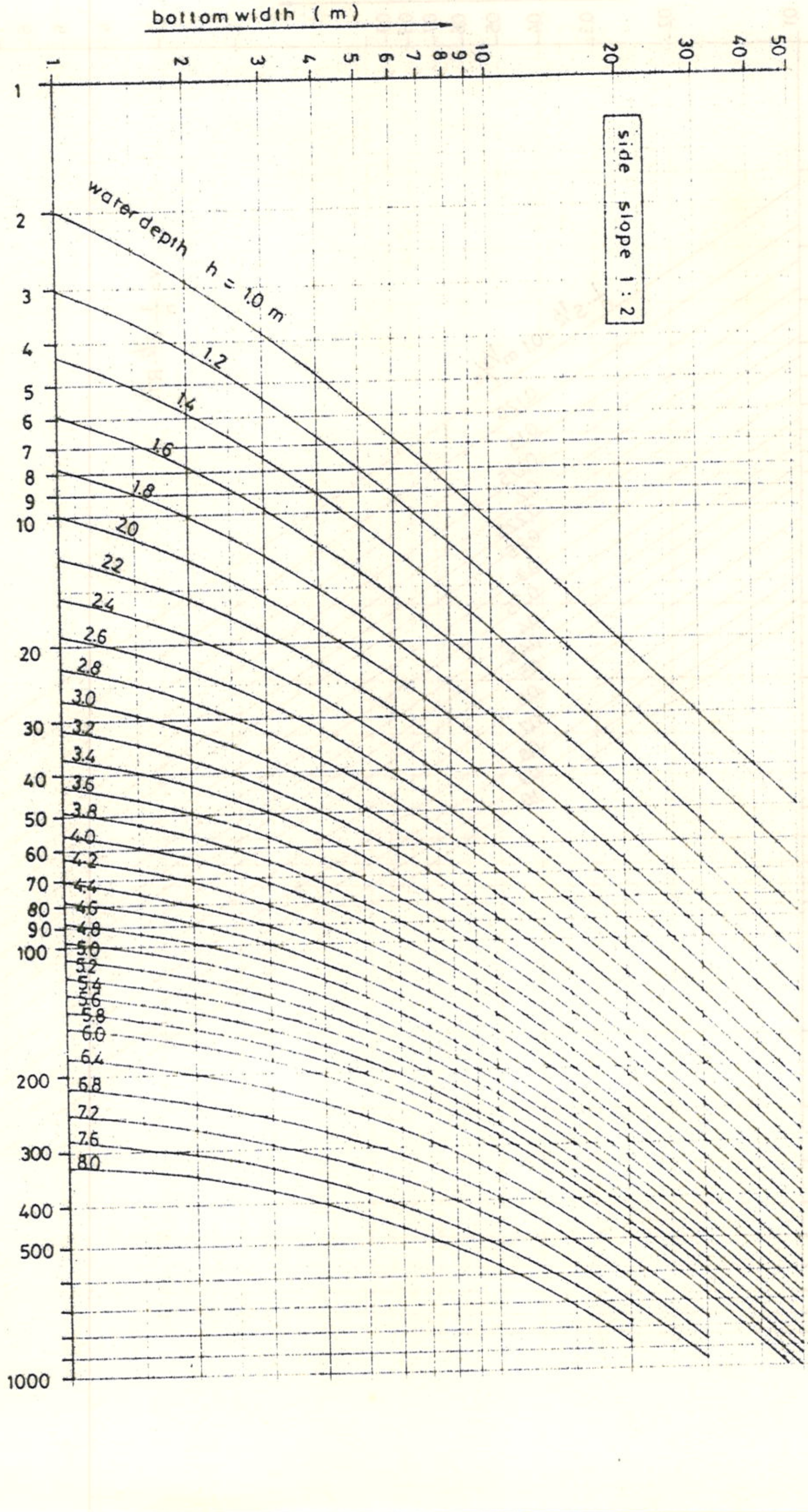
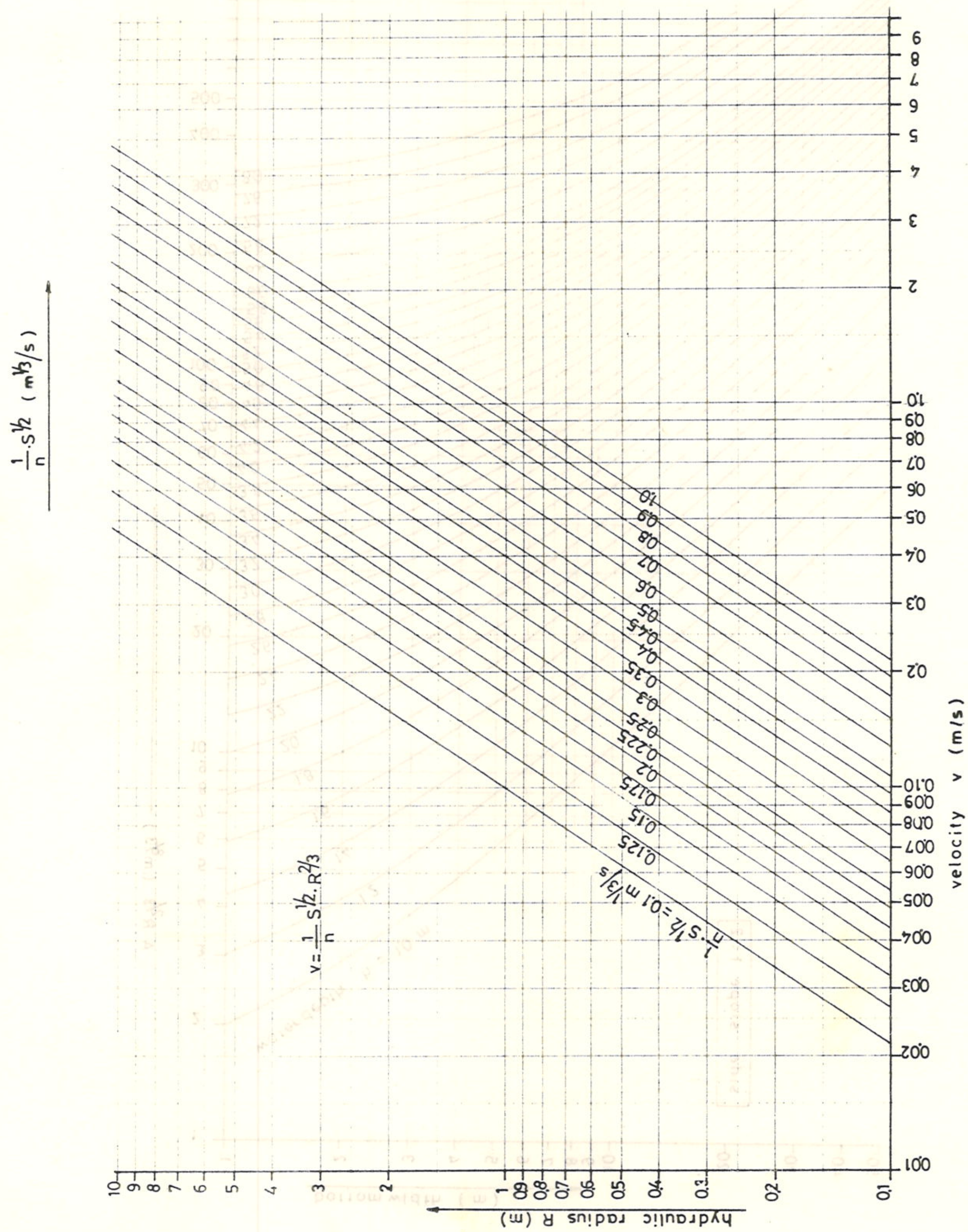


Figure A.V - 2.4

Figure A.V - 2.5


bottom width (m)
A.R$^{2/3}$ (m$^{8/3}$)
side slope 1 : 2
water depth h = 1.0 m
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
4.2
4.4
4.6
4.8
5.0
5.2
5.4
5.6
5.8
6.0
6.4
6.8
7.2
7.6
8.0

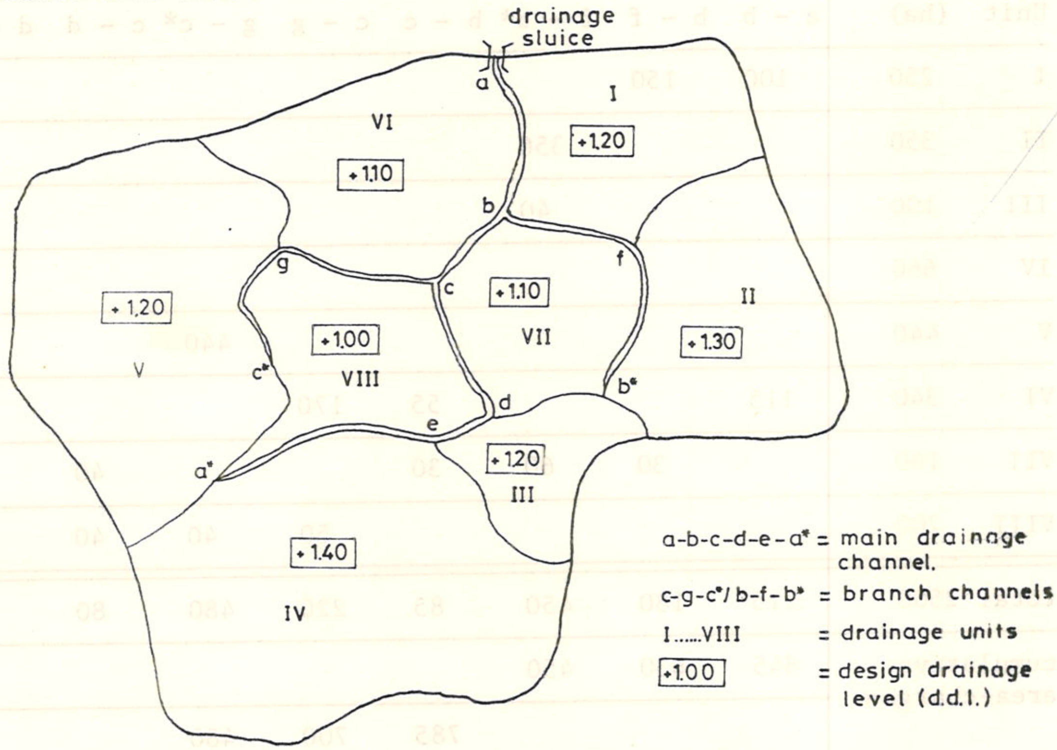


velocity v (m/s)
hydraulic radius R (m)
$\frac{1}{n} \cdot S^{1/2}$ ($m^{1/3}/s$)
$v = \frac{1}{n} S^{1/2} . R^{2/3}$
$\frac{1}{n} \cdot S^{1/2} = 0.1 \; m^{1/3}/s$
0.125
0.15
0.175
0.2
0.225
0.25
0.3
0.35
0.4
0.45
0.5
0.6
0.7
0.8
0.9
1.0

Figure A.V - 2.6

ANNEX V - 3 CALCULATION OF BACKWATER CURVE IN THE MAIN DRAINAGE CHANNEL

Given the polder situation as presented in Figure V - 2.1 as repeated here below



Total polder area : 2500 ha

drainage modules : 30 mm

Drainage flow during discharge period T : Q_{max} = 17,36 m^3/s

(see paragraph 4.2.4). Q_{av} = 14,8 m^3/s

Drainage channel to be checked on maximum flow condition during design drainage level in the channel (highest-discharge-at-lowest-water-level combination).

The area of the polder is subdivided in units which will drain their excess water to the main drainage channel a* - e - d - c - b - a, b* - f - b and c* - g - c.

In the following review it is tabulated which area will drain to which channel section and what will be the drainage flow in each of the channel-sections during maximum drainage flow.

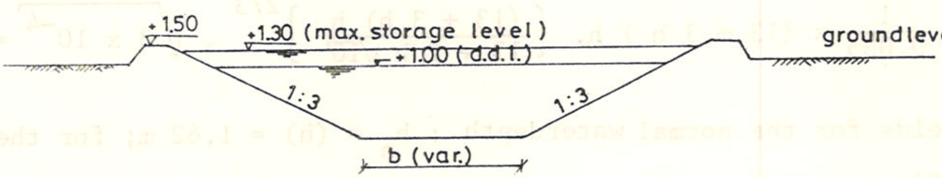
Unit	area (ha)	a - b	b - f	f - b*	b - c	c - g	g - c*	c - d	d - e	e - a*
I	250	100	150							
II	350			350						
III	100			40					60	
IV	660									660
V	440						440			
VI	340	115			55	170				
VII	160		30	60	30			40		
VIII	200					50	40	40	20	50
total	2500	215	180	450	85	220	480	80	80	710
cumulative area-charges		845	630	450						
					785	700	480			
		2500	630	450	1655	700	480	870	790	710 ha
discharge at downstream end		17,36	4,37	3,12	11,49	4,86	3,33	6,04	5,49	4,93 m^3/s
average discharge		16,61	3,75	1,68	11,20	4,10	2,02	5,77	5,21	2,70 m^3/s

The result of the last row may be found as well by plotting the discharges of the end of each section in the lay out of the drainage channel and making the ends fit.

The field data for the drainage channels are presented as follows:

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
average bottomslope	$3x10^{-4}$	$4x10^{-4}$	$3x10^{-4}$	$2x10^{-4}$	$1x10^{-4}$	$5x10^{-4}$	$2x10^{-4}$	$4x10^{-4}$	$1x10^{-4}$
average bottomwidth(m)	13	13	10	10	9	10	5	10	5
roughness coeff. n	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035
average sideslopes	← — — — 1:3 — — — — — — — — →								
average Q_{max} discharge	16,61	11,20	5,77	5,21	2.70	3.75	1.68	4.10	2.02

All channels are assumed to have a profile as indicated in the figure below:



Formulas to be used :

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

2) $(\frac{\Delta x}{h})^{0.315} = (\frac{\Delta_o}{h})^{0.315} - 0.408 . \frac{x. S}{h}$ for backwater curve (see par. V - 1.6).

or nomograph

$A = (b + 3 h)h$

$R = A/(b + 2 h \sqrt{10})$

The average bottom profile of the long section of the drainage channel is plotted together with the design drainage level of the units along the drainage channel.

In the critical drainage situation, with no drainage flow, the water level in the drainage channel will be equal to the design drainage level in the lowest unit (+ 1.00 m, unit VIII).

In the flow situation, shortly after the start of the drainage period T, a draw down curve will have developped towards the sluice, of which the only known boundary condition is the water level at a*, which is assumed to have remained on the level of + 1.00 m (design drainage level).

The draw down curve is now calculated from this point a* onwards in downstream direction.

At first the normal water depths in each channel section are calculated with formula 1). These depths are also plotted in the long section profile of the drainage channel.

example: for section (a - b) :

$$Q = \frac{1}{0.035} \times (13 + 3\ h)\ h. \left[\frac{(13 + 3\ h)\ h}{13 + 2\ h\sqrt{10}}\right]^{2/3} . \sqrt{3 \times 10^{-4}} = 16.61\ m^3/s.$$

This yields for the normal waterdepth : h_n = (h) = 1.62 m; for the other sections:

	a - b	b - c	c - d	d - e	e - a*	b - f	f - b*	c - g	g - c*
h_n (m)	1.62	1.20	1.02	1.08	0.96	0.70	0.80	0.78	1.06

The next step is to calculate the drawdown curve with the help of formula 2), starting at the assumed boundary condition at a*.

The calculation is demonstrated on Figure A.V - 3.1 , yielding the backwater curve for the assumed flow and water level conditions.

The backwater curve should in all sections be above the normal depth to avoid any draw-down effects restricting the discharge capacity.

The profile of the sections should be adjusted in case this condition is not met.

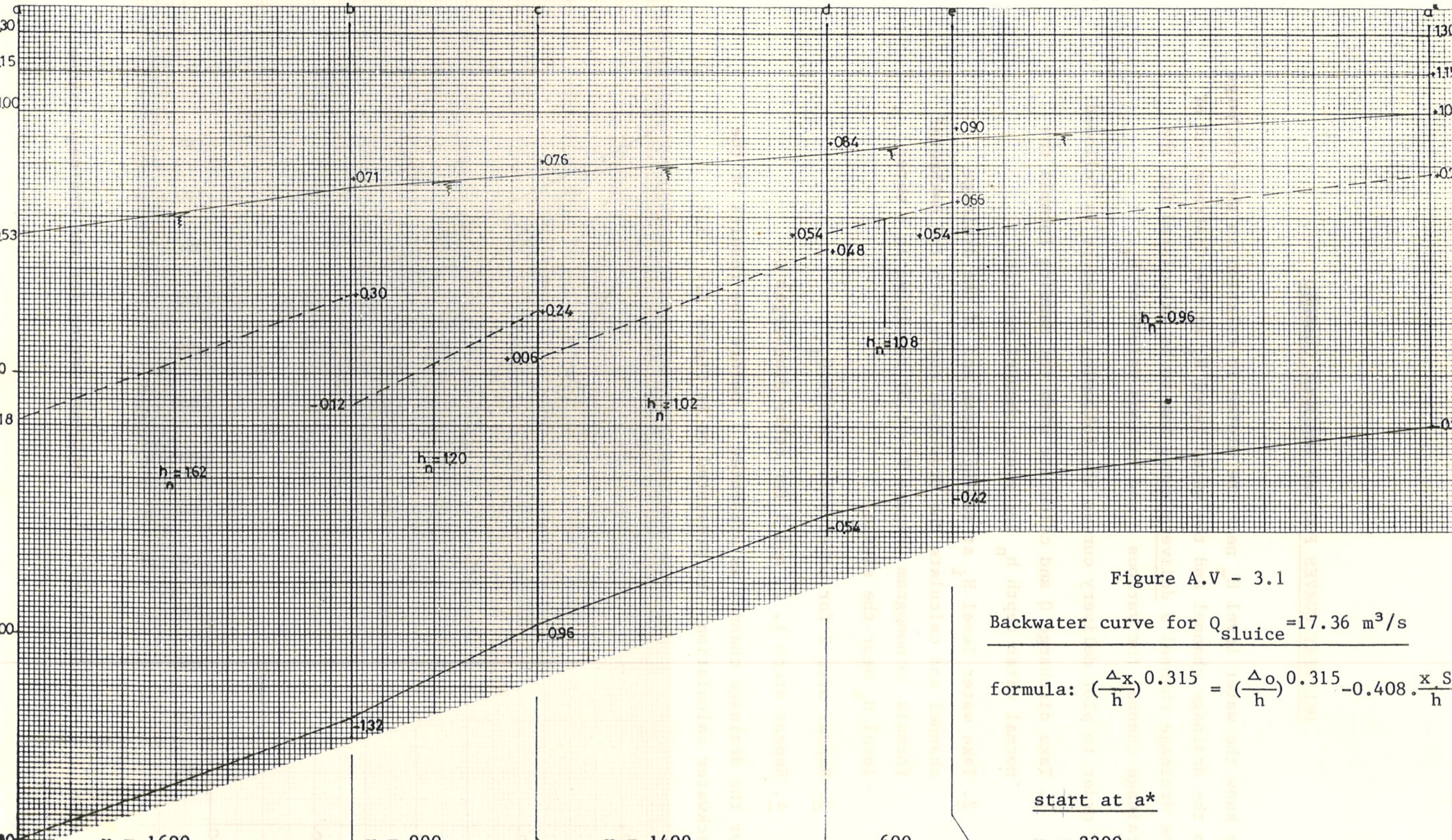


Figure A.V - 3.1

Backwater curve for Q_{sluice}=17.36 m³/s

formula: $\left(\frac{\Delta x}{h}\right)^{0.315} = \left(\frac{\Delta o}{h}\right)^{0.315} - 0.408 . \frac{x.S}{h}$

start at a*

x = 1600	x = 900	x = 1400	600	x = 2300
S = $3x10^{-4}$	S = $4x10^{-4}$	S = $3x10^{-4}$	$2x10^{-4}$	S = $1x10^{-4}$
h = 1.62	h = 1.20	h = 1.02	1.08	h = 0.96
Δx = +0.71-0.30=0.41	Δx = +0.76-0.24=0.52	Δx = 0.84-0.48=0.36	0.90-0.66=0.24	Δx = +1.00-0.77=0.23
Δo = 0.71	Δo = 0.83	Δo = 0.70	0.30	Δo = 0.36
h =-0.18+0.71=+0.53	h = -0.12+0.83=+0.71	h = +0.06+0.70=+0.76	0.54+0.30=0.84	water level downstream is: h=+0.54+ Δ o=+0.90.

DELIVERY CURVES FOR THE DRAINAGE CHANNEL

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.

In order to plot delivery curves the following procedure is followed:

1. Take discharge Q and calculate with the Manning formula the normal water depth h_n for every channel section.
2. Take water level H_i at the upstream end of the drainage channel and calculate the draw-down curve with the Bakhmetef formula or nomogram (Figure V - 1.5) resulting in a water level H_x near the sluice.
3. Repeat step 2 for other water levels H_i.
4. Repeat steps 1, 2, and 3 for other discharges Q.

For the drainage channel as presented before the results of the backwater calculations are presented on in Figure A.V - 3.2

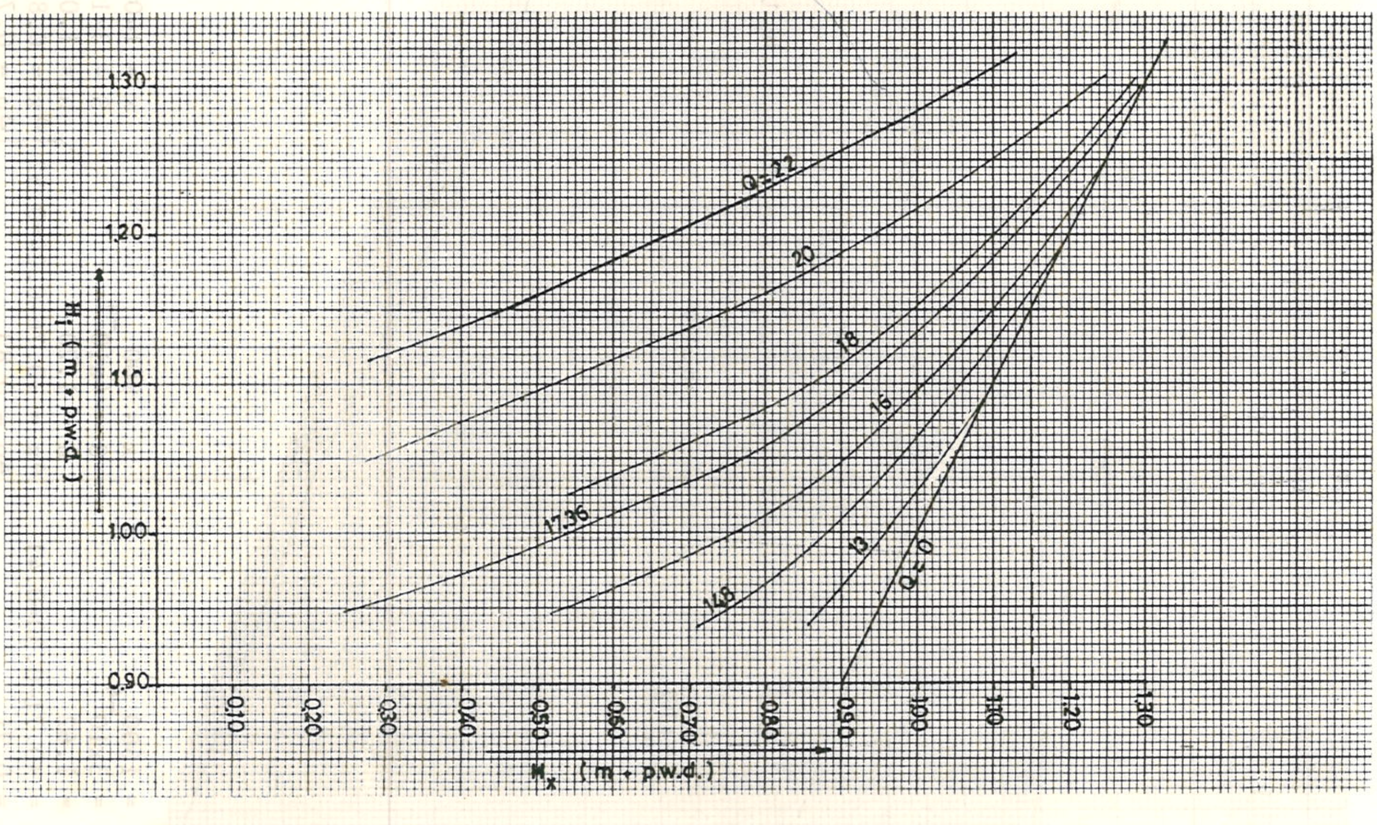


Figure A.V - 3.2 Delivery curves for the drainage channel

WATER LEVELS DURING DRAINAGE PERIOD

In paragraph 4.2.5 the invert level and width of the sluice were determined. The criteria for the invert level was the maximum allowable velocities in the sluice during extreme drainage conditions.

The width of the sluice was determined on the assumption that during the drainage period T the discharge in the channel-sluice system remains constant at $Q_{av} = 14.8$ m^3/s. The backwater H_x near the sluice was taken from the delivery curves of the drainage channel for values of $H_i = 1.13$ and $H_i = 0.93$.

In order to verify the correctress of this simplification the graph of Figure A.V - 3.3 is constructed, in which the sluice discharge is plotted against the upstream head H_x, (assuming flow condition 5) yielding line 'a', and the channel discharge is plotted for various values of H_x and H_i, yielding the lines 'b'. The latter information in fact is a re-plotting of the delivery curves of the drainage channel (see Figure A.V - 3.2).

Where the line 'a' intersects the lines "b", the discharge through the sluice equals the channel discharge.

It is clearly demonstrated that as the polders water level H_i is decreasing, the discharges are decreasing and the headwater at the sluice is decreasing.

For a equi - Q - inter section point, the horizontal axis of the diagram shows H_x and the line section 'c' indicates the difference at that particular Q between H_x and H_i.

It can be seen that for all equi - Q - intersections, the difference between H_x and H_i remains almost constant between 0.09 and 0.12 m.

This means that the average water level gradient in the drainage channel remains practically constant. It also means that the amount of water between the horizontal polderwater level and the backwater curve does not change very much during the drainage period T.

For the end of the drainage period, we therefore may assume that the polderwater level H_i has dropped to the level at which the drainage volume has been evacuated, i.e. to the level of + 1.15 - 0.27 = 0.88 m. The extra water which has been drained in addition because of the backwater from + 0.88 m to approximately + 0.76 m for H_x near, the sluice is then neglected.

The width of the sluice can now be determined in another way as in paragraph 4.2.5.

1. Average polderwater level during the drainage period T is :

   $\frac{1}{2}(+1.15 + 0.88) = 1.02$ m.

2. Average backwater effect to the sluice during the drainage period T is ± 0.10 m, yielding an average headwater at the sluice of + 1.02 - 0.10 m = + 0.92 m.

3. Invert level was established at - 0.80 m PWD.

4. Average headwater for the sluice discharge is thus

   $$+0.92 + \left|-0.80\right| = +1.72 \text{ m.}$$

5. Average sluice discharge :

   $$Q = 14.8 \text{ m}^3/\text{s} = 1.35 \times b \times (1.72)^{1.5}$$

   $$\longrightarrow b = 4.86 \text{ m.}$$

   Initially the sluice width was determined to be b = 4.95 m.

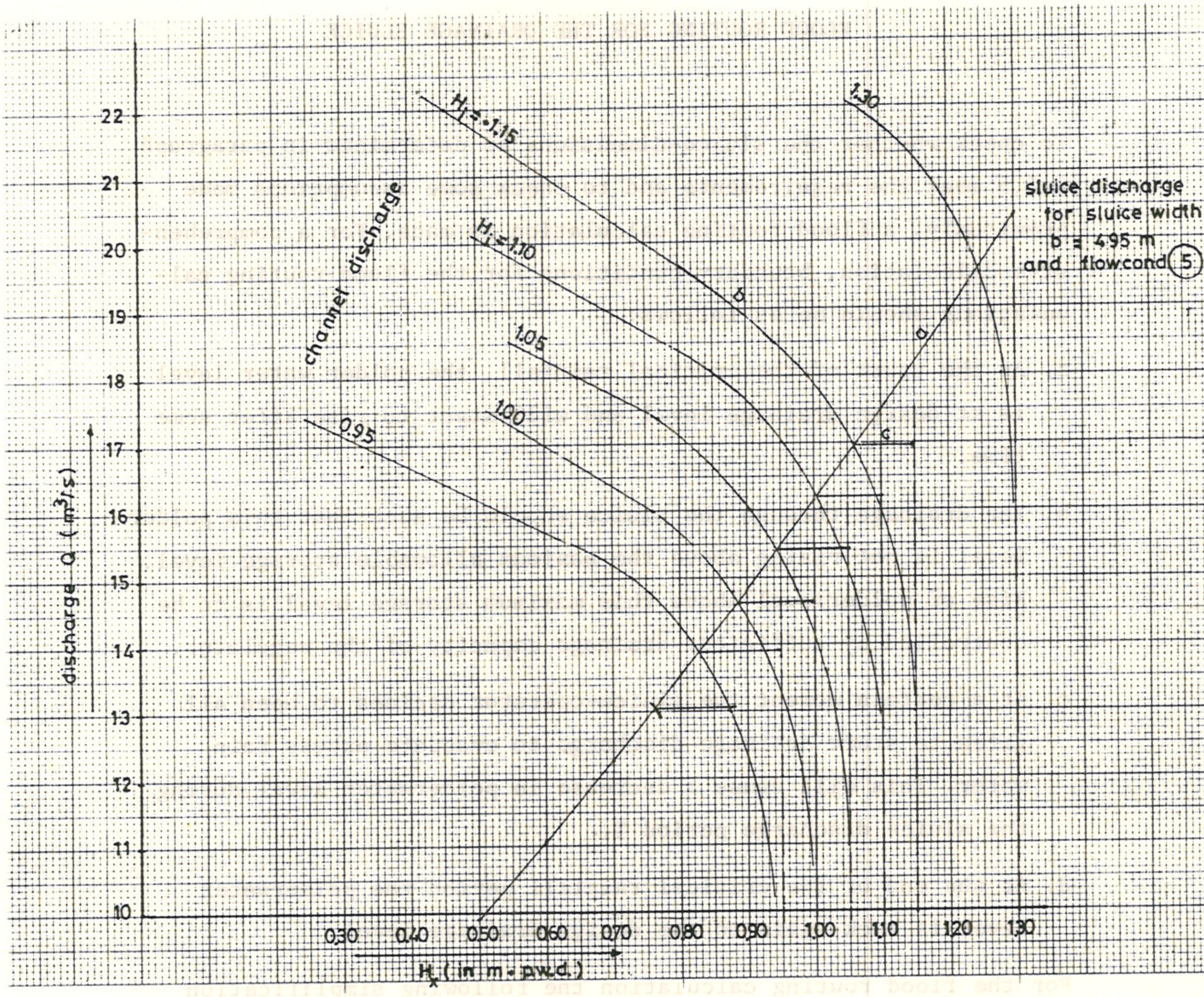

sluice discharge
for sluice width
b = 4.95 m
and flowcond 5
channel discharge
H_i = 1.15
H_i = 1.10
1.05
1.00
0.95
1.30
a
b
c
discharge Q (m³/s)
22
21
20
19
18
17
16
15
14
13
12
11
10
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
1.30
H_x (in m - p.w.d.)

Figure A.V - 3.3

FLOOD ROUTING FOR THE DRAINAGE SLUICE

To check whether the sluice-dimensions as determined in paragraph 4.2.5 meet the requirements and to check some assumptions made previously, a flood routing calculation is made. For a comprehensive calculation, boundary conditions for the flood routing calculations should be as follows:

a. At the start of the critical rainfall, the polder water level is at design drainage level and will start to rise after some time.

b. The drainage through the sluice should be such that this polder level is not rising above the maximum allowable storage level. Rise of polderlevel above this maximum storage level is to be controlled by sufficient drainage capacity of the regulator.

c. At a certain moment the sluice-operator decides to open all gates to start drainage discharge to evacuate excess rain water. This will cause a decrease in polderwater level during the sluice discharge period T.

d. At the end of the critical rainfall period the polderwater level is back again to the design drainage level.

For the flood routing calculation the following simplification is made :

- The drainage capacity is increasing as the polderwater level is rising and will be maximum at the maximum allowable storage level of the polder water and minimum when polder water level is at design drainage level. Therefore during the critical rainfall period the average required drainage discharge is calculated at the stage that the polderwater level is halfway in between design drainage level and maximum allowable storage level.

- It is furthermore assumed that the first tidal drainage period will start at the same time as the critical rainfall period. In practise, the drainage sluice will only be opened if, after the first rains have raised the polderwater level and the forecasts indicate more rainfall.

- During the tidal discharge period T a backwater curve will develop in the drainage canal. The effect of this backwater curve on the sluice discharge is considered in the flood routing.

- The storage decrease in the polder during the drainage period T will result in a drop of water levels in the polder. This drop is considered to be planar and the effect of the backwater curve in the main drainage channel is in this respect neglected.

The relation between storage area provided by the drainage channels and low pockets against the polderwater level H_i is given in Figure A.V-3.4. The Figure is a graphical interpretation of the information on storage area as presented in paragraph 4.2.5, page V-48/50.

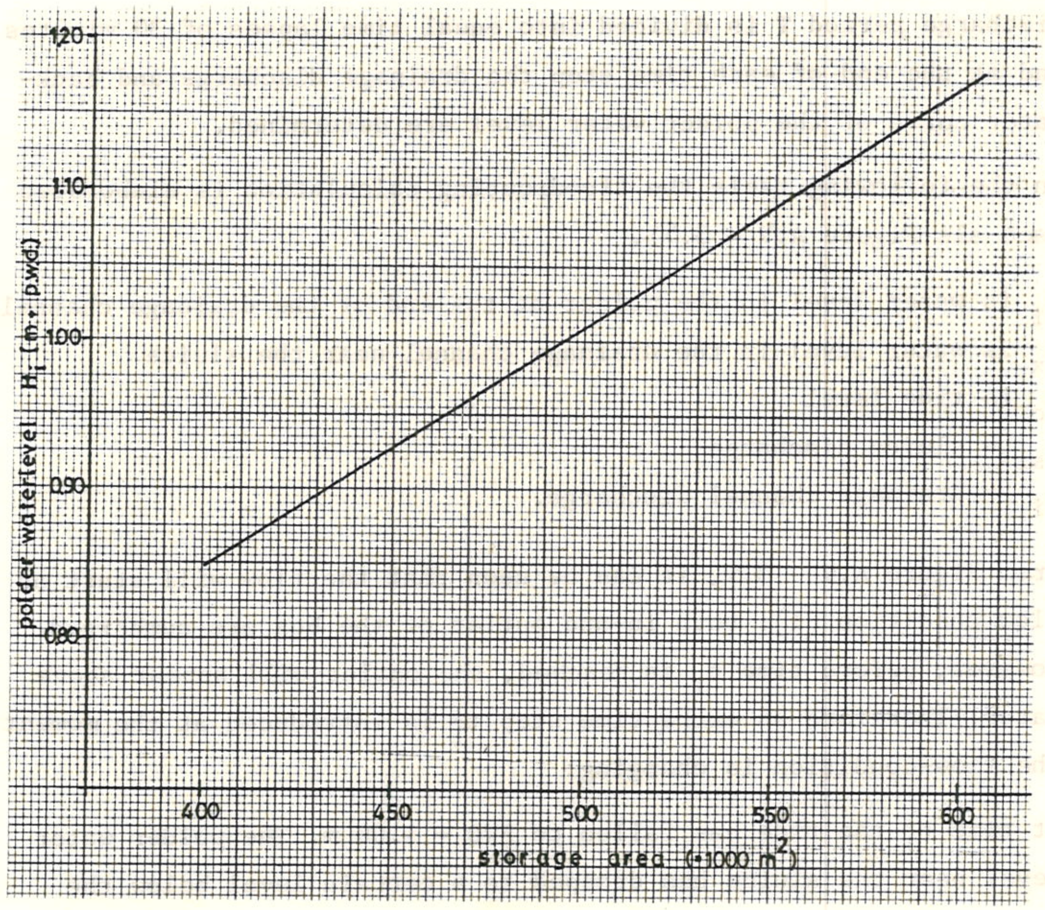


Figure A.V - 3.4

- The total drainage capacity of the sluice during one tidal cycle is compared with the decrease in water storage due to the falling water level H_i in the polder.
- The effect of salinity difference between outerwater and polder water, and the over-pressure required to open the flapgates against the gravity which requires a head-difference of a few centimeters, is not taken into account.

For the calculation example of the floodrouting, the dimensions of the sluice have been taken as calculated in paragraph 4.2.5. To calculate the backwater curve the characteristics of the drainage channel is to be known. In our example the bottom slope S and other features of the drainage channel have been assumed as indicated in ANNEX V - 3. Figure A.V - 3.1.

At time t_o, the tide has fallen to the same level as the polder water level and drainage will start from this moment. The sluice discharge period T is divided into equal time lapses of 30 minutes and at the end of each time step the drainage discharge and backwater curve of the polder water level are determined.

In the following table the floodrouting calculation is elaborated (see als Figure A.V - 3.5).

H_i is water level in the polder at the end of the drainage channel.
H_x is water level in the drainage channel near the sluice
H_o is tide level
Q_{sl} = sluice discharge
Q_i = drainage channel discharge.

From Figure A.V - 3.5 , it can be seen that the flow will start as flow condition 1: both in-and outlet of the box are sumberged. However, before time t_1 is reached, the outlet will appear above water. Therefore an extra time step t_1^* is introduced at the moment the flow-condition is changing.

At time t_1^* the tide has fallen to PWD + 1.00 m. The polder water level near the sluice has dropped to a certain level below the original H_i of 1.15 + PWD (see Figure A.V - 3.5). But it is not known how much. The difference between H_o and H_i, being 0.15 m is formed by a head difference over the sluice, causing a sluice discharge Q_s and a head difference in the drainage channel causing a flow towards the sluice, Q_i. These two discharge have to be equal.

$$Q_s = 0.82 \times d \times B \times \sqrt{2g\,\Delta H} \quad \text{(flow condition 1)}$$

$$= 0.82 \times 1.80 \times 4.95 \times \sqrt{2g \times (H_x - H_o)}$$

$$= 32.3 \times \sqrt{H_x - H_o}$$

The relation between channel discharge and H_i and H_x is presented in the delivery curves of the drainage channel (see Figure A-V-3.3).

The calculation process proceeds as follows:

1. At the beginning of each time step Δt, the tide level outside (H_o), the water level at the end of the drainage channel (H_i), the water level just upstream of the sluice (H_x) and the discharge in this situation ($Q_{s1}=Q_i$) are known.
2. At the end of the time step Δt, the tide level (H_o) is known, the polderwater level (H_i) is estimated, as well as the level (H_x) at the sluice. The corresponding discharge ($Q_{s1}= Q_i$) is determined for this situation.
3. The average discharge in the period Δt is now calculated and multiplied with the duration of Δt to yield the amount of water evacuated through the sluice in the period Δt.
4. The volume of water evacuated in time lapse Δt minus the supply to the drainage channel from the field (= 8,68 m^3/s, see p.V-48) is now divided by the average storage area provided by the drainage channels and low pockets between the level of H_i and + 1.15 m. The decrease in H_i resulting from the drainage during Δt should correpond with the estimated H_i at the end of the time step. (step 2 above). If not, then step 2 should be repeated with a better estimate of H_i and/or H_x.

The first calculation steps (t_o upto t_4) are elaborated hereunder, the whole flood routing calculation is further elaborated in the attached Table A.V - 3.1).

From the results of the floodrouting the following is observed.

1. The estimate drainage period T, assumed to be from the intersection of the level $\frac{1}{2}$(m.a.st.1 + d.d.1) and the estimated polderwater level at the end of the drainage period (see p.V-49) is fairly correct, (T = 7.3 hours).

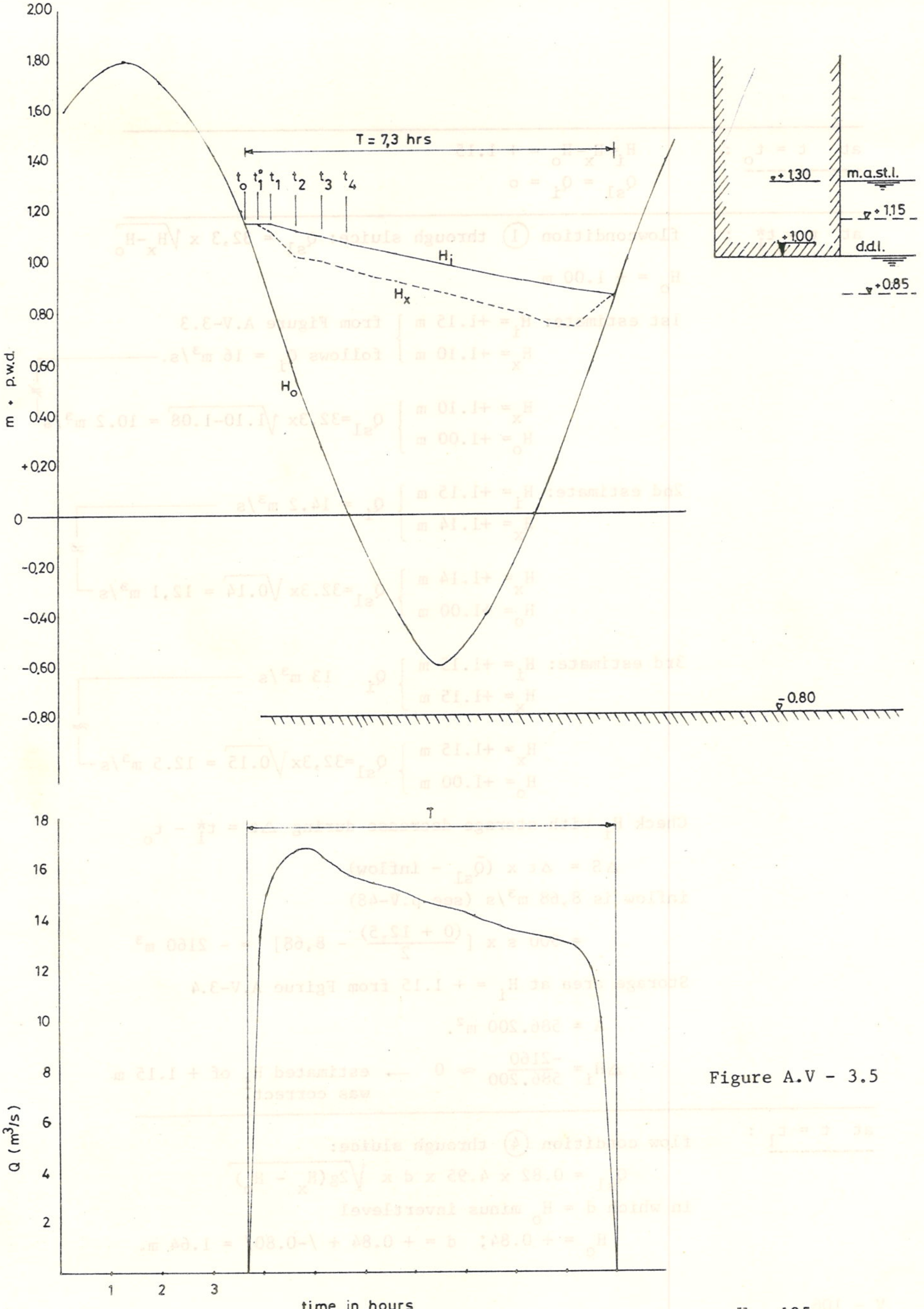
2. The total volume of water discharged through the sluice (361.620 m^3) is some 8% below the requirement. The simplification explained on p.V-50 is the reason for this difference. The average discharge through the sluice during period T (= 13,76 m^3/s) seems to be lower then the average discharge during the stage of critical flow through the sluice (= 14.7 m/s).

3. The maximum occuring discharge is Q = 16,7 m^3/s and is slightly below the assumed maximum of 17,36 m^3/s but fairly in the range if we multiply the actual average discharge of 13.76 with the factor $\frac{1}{0.85}$ yielding for Q_{max} = 16,2 m^3/s (16,7).

4. To bring the sluice upto exact required drainage capacity, the width of the barrels has to be increased by:

$$b = \frac{389.153}{361.620} \times 4.95 = 5.33 \text{ m.}$$

yielding three barrels of 1.80 m wide.



2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
+0.20
0
-0.20
-0.40
-0.60
-0.80
m + p.w.d.
T = 7,3 hrs
t_o t_1^* t_1 t_2 t_3 t_4
H_i
H_x
H_o
-0.80
+1.30
m.a.st.l.
+1.15
+1.00
d.d.l.
+0.85
18
16
14
12
10
8
6
4
2
Q (m³/s)
T
1
2
3
time in hours

Figure A.V - 3.5

at $t = t_o$: $H_i = H_x = H_o = +1.15$

$Q_{s1} = Q_i = o$

at $t = t_1^*$: flowcondition (1) through sluice: $Q_{s1} = 32{,}3 \times \sqrt{H_x - H_o}$

$H_o = +1.00$ m.

1st estimate: $\left.\begin{array}{l} H_i = +1.15 \text{ m} \\ H_x = +1.10 \text{ m} \end{array}\right\}$ from Figure A.V-3.3 follows $Q_i = 16$ m³/s.

$\left.\begin{array}{l} H_x = +1.10 \text{ m} \\ H_o = +1.00 \text{ m} \end{array}\right\}$ $Q_{s1} = 32{,}3\text{x} \sqrt{1.10-1.08} = 10.2 \text{ m}^3/\text{s}$ $\neq$

2nd estimate: $\left.\begin{array}{l} H_i = +1.15 \text{ m} \\ H_x = +1.14 \text{ m} \end{array}\right\}$ $Q_i = 14.2 \text{ m}^3/\text{s}$

$\left.\begin{array}{l} H_x = +1.14 \text{ m} \\ H_o = +1.00 \text{ m} \end{array}\right\}$ $Q_{s1} = 32.3\text{x} \sqrt{0.14} = 12{,}1 \text{ m}^3/\text{s}$ $\neq$

3rd estimate: $\left.\begin{array}{l} H_i = +1.15 \text{ m} \\ H_x = +1.15 \text{ m} \end{array}\right\}$ Q_i 13 m³/s

$\left.\begin{array}{l} H_x = +1.15 \text{ m} \\ H_o = +1.00 \text{ m} \end{array}\right\}$ $Q_{s1} = 32{,}3\text{x} \sqrt{0.15} = 12.5 \text{ m}^3/\text{s}$ $\approx$

Check H_i with storage decrease during $\Delta t = t_1^* - t_o$

$\Delta S = \Delta t \times (\bar{Q}_{s1} - \text{inflow})$

inflow is 8,68 m³/s (see p.V-48)

$= 900 \text{ s} \times [\frac{(0 + 12.5)}{2} - 8{,}68] = -2160 \text{ m}^3$

Storage area at $H_i = +1.15$ from Fgirue A.V-3.4

$A = 586.200 \text{ m}^2$.

$\Delta H_i = \frac{-2160}{586.200} \approx 0 \longrightarrow$ estimated H_i of + 1.15 m was correct.

at $t = t_1$: flow condition (4) through sluice:

$Q_{s1} = 0.82 \times 4.95 \times d \times \sqrt{2g(H_x - H_o)}$

in which $d = H_o$ minus invertlevel

$H_o = +0.84$; $d = +0.84 + /-0.80/ = 1.64$ m.

1st estimate: $\left.\begin{array}{l} H_i = +1.15 \\ H_x = +1.05 \end{array}\right\}$ from Figure A.V-3.3 follows $Q_i = 17.1\ m^3/s$ $\neq$

$\left.\begin{array}{l} H_x = +1.05 \\ H_o = +0.84 \end{array}\right\}$ $Q_{s1} = 0.82 x 4.95 x 1.64 x \sqrt{2g(1.05-0.84)} = 13.5\ m^3/s$

2nd estimate: $\left.\begin{array}{l} H_i = +1.15 \\ H_x = +1.10 \end{array}\right\}$ $Q_i = 16{,}0\ m^3/s$ $\neq$

$\left.\begin{array}{l} H_x = +1.10 \\ H_o = +0.84 \end{array}\right\}$ $Q_{s1} = 6{,}66 x \sqrt{2g(1.10-0.84)} = 15{,}0\ m^3/s$

3rd estimate: $\left.\begin{array}{l} H_i = +1.15 \\ H_x = +1.12 \end{array}\right\}$ $Q_i = 15.3\ m^3/s$ $\approx$

$\left.\begin{array}{l} H_x = +1.12 \\ H_o = +0.84 \end{array}\right\}$ $Q_{s1} = 6.66 x \sqrt{2g(1.12-0.84)} = 15{,}6$

check H_i with storage decrease during $\Delta t = t_1 - t_1^*$

$$\Delta S = \Delta t \times [\bar{Q} - 8{,}68]$$

$$= 900 \times [\frac{(12.5 + 15.5)}{2} - 8{,}68] = 4800\ m^3$$

Storage area at $H_i = +1.15$ m from Figure A.V-3.4

$$A = 586.200\ m^2$$

$$H_i = \frac{-2160 + 4800}{586.200} \approx 0.$$

estimated H_i of + 1.15 m was correct.

---

at $t = t_2$: assume flow condition (4) through the sluice:

$$Q_{s1} = 0.82 \times 4.95 \times d \times \sqrt{2g(H_x - H_o)}$$

in which d = H_o minus invert level.

$H_o = +0.53$; $d = 0.53 + 0.80 = +1.33$ m

$$Q_{s1} = 23{,}9 \times \sqrt{(H_x - H_o)}.$$

1st estimate: $\left.\begin{array}{l} H_i = +1.11 \\ H_x = +1.00 \end{array}\right\}$ from Figure A.V-3.3 follows: $Q_i = 16{,}63\ m^3/s$ $\neq$

$\left.\begin{array}{l} H_x = +1.00 \\ H_o = +0.53 \end{array}\right\}$ $Q_{s1} = 23.9 x\ \ 1.00-0.53 = 16{,}39\ m^3/s$

2nd estimate: $\left.\begin{array}{l} H_i = +1.11 \\ H_x = +1.01 \end{array}\right\} Q_i = 16{,}47 \text{ m}^3/\text{s}$ (from Figure A.V-3.3)

$\approx$

$\left.\begin{array}{l} H_x = +1.01 \\ H_o = +0.53 \end{array}\right\} Q_{s1} = 23.9 \text{x} \sqrt{1.01-0.53} = 16.55 \text{ m}^3/\text{s}$

check if flow condition 4 still prevails:

$$h_2 \geqslant \sqrt[3]{\frac{q^2}{g}} = 1.04 \text{ m.}$$

$h_2 = +0.53 + 0.80 = 1.33 \longrightarrow$ o.k.

check H_i with storage decrease during period $\Delta t = t_2 - t_1$

$$\Delta S = \Delta t \text{ x } (\bar{Q} - 8{,}68) = 1800 \text{ x } \left(\frac{16.55+15.6}{2} - 8.68\right)$$
$$= 13.310 \text{ m}^3.$$

Storage area at H_i = + 1.11 : $A = 561.300 \text{ m}^2$

Storage area at H_i = + 1.15 : $A_o = 586.200 \text{ m}^2$.

Average $\bar{A} = 573.750 \text{ m}^2$

Total storage decrease:

$$\Delta H_i = \frac{\sum \Delta S}{\bar{A}} = \frac{(-2160 + 4800 + 13.310)}{573.750} = 0.028 \text{ m.}$$

Start new trial with H_i = + 1.12

3rd estimate: $\left.\begin{array}{l} H_i = +1.12 \\ H_x = +1.00 \end{array}\right\} Q_i = 17.00 \text{ m}^3/\text{s}$ (from Figure A.V-3.3)

$\neq$

$\left.\begin{array}{l} H_x = +1.00 \\ H_o = +0.53 \end{array}\right\} Q_{s1} = 23.9 \text{x} \sqrt{1.00-0.53} = 16{,}4 \text{ m}^3/\text{s}.$

4th estimate: $\left.\begin{array}{l} H_i = +1.12 \\ H_x = +1.02 \end{array}\right\} Q_i = 16.6 \text{ m}^3/\text{s}$ (from Figure A.V-3.3)

$\approx$

$\left.\begin{array}{l} H_x = +1.02 \\ H_o = +0.53 \end{array}\right\} Q_{s1} = 23.9 \text{x} \sqrt{1.02-0.53} = 16.73 \text{ m}^3/\text{s}.$

check H_i with storage decrease during period $\Delta t = t_2 - t_1$

$$\Delta S = \Delta t \ (\bar{Q} - 8{,}68) = 1800 \text{x} \left(\frac{16.73+15.6}{2} - 8.68\right) = 13.470 \text{ m}^3.$$

Storage area A at H_i = + 1.12 = 567.500 m²

Storage area A_o at H_i = + 1.15 = 586.200 m²

Average storage area $\bar{A} = 576.850\ m^2$

Total storage decrease is :

$$\Delta H_i = \frac{\sum \Delta S}{\bar{A}} = \frac{16.110}{576.850} = 0.03\ m \longrightarrow h_i = \text{o.k.}$$

<u>at $t = t_3$:</u> Assume flow condition 4 through the sluice:

$$Q_{s1} = 0.82 \times 4.95 \times d \times \sqrt{2g(H_x - H_o)}$$

in which $d = H_o$ minus invert level.

$H_o = +0.26$; $d = 0.26 + 0.80 = 1.06\ m$

$$Q_{s1} = 19.05 \sqrt{(H_x - H_o)}$$

1st estimate: $\left.\begin{matrix} H_i = +1.10 \\ H_x = +1.00 \end{matrix}\right\}$ $Q_i = 16{,}3\ m^3/s$

$\left.\begin{matrix} H_x = +1.00 \\ H_o = +0.26 \end{matrix}\right\}$ $Q_{s1} = 19.05 \sqrt{1.00 - 0.26} = 16{,}39\ m^3/s.$ $\approx$

check H_i with storage decrease during period $\Delta t = t_3 - t_2$:

$$\Delta S = \Delta t\ (\bar{Q} - 8.68) = 1800 \times \left(\frac{16.39 \times 16.73}{2} - 9.68\right) = 14.180\ m^3$$

Storage area A at $H_i = +\ 1.10$ is: $555.500\ m^2$

Storage area A_o at $H_i = +\ 1.15$ is: $586.200\ m^2$

Average storage area $\bar{A} = 570.850\ m^2$

Total storage decrease is:

$$\Delta H_i = \frac{\sum \Delta S}{\bar{A}} = \frac{30.290}{570.850} = 0.05\ m \longrightarrow H_i \text{ is o.k.}$$

check flow condition: 4 if $h_2 \geqslant \sqrt[3]{\frac{q^2}{g}}$

$\sqrt[3]{\frac{q^2}{g}} = 1.04\ m.$ $\quad h_2 = +\ 0.26 + 0.80 = 1.06\ m \longrightarrow$ o.k.

<u>at $t = t_4$:</u> Assume flow condition 5 for the flow through the sluice:

$$Q_{s1} = 4.95 \times 1.35 \times (H_x)^{1.5}$$

H_x should be taken to the invert level.

$H_o = +\ 0.01$

1st estimate: $H_i = +1.07$

from Figure A.V-3.3 follows direct that

$Q_i = 15{,}7\ m^3/s = Q_{s1}$

$H_x = +0.97$ m.

check H_i with storage decrease during period $\Delta t = t_4 - t_3$:

$$\Delta S = \Delta t \times (\bar{Q} - 8.68) = 1800 \times \left(\frac{16{,}39+15{,}7}{2} - 8.68\right) = 13.260\ m^3$$

Storage area A at $H_i = +1.07$ is 537.000 m^2

Storage area A_o at $H_i = +1.15$ is 586.200 m^2

Average storage area $\bar{A}$ is 561.600 m^2

Total storage decrease is :

$$\Delta H_i = \frac{\sum \Delta S}{\bar{A}} = \frac{43.500}{561.600} = 0.08\ m \rightarrow H_i = o.k.$$

check flow condition:

$$\sqrt[3]{\frac{q^2}{g}} = 1.01\ m\ , \quad h_2 = +0.01+0.80 = 0.81\ m. \quad o.k.$$

5 prevails.

Table A.V - 3.1 Floodrouting through a drainage sluice

discharges for flowcondition: (1) $Q_{s1} = 0.82 \times b \times d \times \sqrt{2g\ \Delta H}$ for full flow barrel b x d = 1.80 x 4.95

$= 32,3 \times \sqrt{\Delta H}$

(4) $Q_{s1} = 0.82 \times b \times h_2 \times \sqrt{2g(h_1 - h_2)}$ for both outlet and inlet un-submerged.

$= 17,97 \times h_2 \sqrt{h_1 - h_2}$ h_1 = upstream head; h_2 = downstream head.

(5) $Q_{s1} = 1.353 \times b \times (h_1)^{1.5}$ h_1 = upstream head

$= 6,7 \times (h_1)^{1.5}$

t	H_i	H_x	H_o	Q_i	Q_{s1}	Δt	$\bar{Q} \times \Delta t$	($\bar{Q}$-inflow)	ΔS	A_i	$\bar{A}$	$\Sigma \Delta S$	ΔH_i	H_i
t_o	+1.15	+1.15	+1.15	0	0	0		0		586.200				
t_1*	flow condition (1)													
	+1.15	+1.10	+1.00	16,0	10,1									
	+1.15	+1.14	+1.00	14,2	12.1									
	+1.15	+1.15	+1.00	<13	12.5	900	5,625	-2.43	- 2,190	586.200	586.200	- 2,190	0	+1.15
t_1	flowcondition (4)													
	+1.15	+1.05	+0.84	17,1	13.5									
	+1.15	+1.10	+0.84	16,0	15,0									
	+1.15	+1.12	+0.84	15,3	15,6	900	12,645	+5,32	+ 4,790	586.200	586.200	+ 2,600	0	+1.15
t_2	+1.11	+1.00	+0.53	16,6	16,4									
	+1.11	+1.01	+0.53	16,5	16,5	1800		+7,37	13,270	561.300	573.750	15,870	0,028	not ok
	+1.12	+1.00	+0.53	17,0	16,4									
	+1.12	+1.02	+0.53	16,6	16,7	1800	29.070	+7,47	13,450	567.500	576.850	16,050	0,03	+1.15
t_3	+1.10	+1.00	+0.26	16,3	16,4	1800	28,800	7,87	14,170	555.500	570.850	30,220	0,05	+1.15
						check flow condition: $h_2 \geqslant \sqrt[3]{\frac{q^2}{g}}$ = 1,04				h_2 = +0,26+0.80 = +1.06			o.k.	
t_4	flowcondition (5)													
	+1.07	+0.97	+0.01		15.7	1800	28.890	7.37	13,270	537.000	561.600	43,490	0.08	+1.15
						check flowcondition : $h_2 \geqslant \sqrt[3]{\frac{q^2}{g}}$ = 1.01				h_2 = +0.01+0.80 = 0.81			o.k.	
t_5	+1.05	+0.94	-		15.4	1800	27,990	6,87	12,370	524.500	555.350	58,860	0.10	+1.15
t_6	+1.02	+0.91	-		15,0	1800		6,52	11,740	509.500	547.850	67,600	0.12	
	+1.03	+0.92	-		15.1	1800	27,450	6,57	11,830	512.300	549.250	67,690	0.12	+1.15
t_7	+1.00	+0.89	-		14.6	1800	26,730	6,17	11,100	494.000	540.000	78,800	0.15	+1.15
t_8	+0.98	+0.87	-		14.4	1800	26,100	5,82	10,480	481.500	533.850	89,280	0.17	+1.15
t_9	+0.96	+0.84	-		14.1	1800	25,650	5,57	10,030	469.500	527.850	99,370	0.19	+1.15
t_{10}	+0.94	+0.81	-		13.7	1800	25,020	5,22	9,400	457.500	521.850	108,770	0.21	+1.15
t_{11}	+0.92	+0.79	-		13.4	1800	24,390	4,87	8,770	444.800	515.500	117,540	0.23	+1.15
t_{12}	+0.90	+0.76	-		13.0	1800		4,52	8,140	432.500	509.500	125,680	0.25	+1.15
			+0.13			check flowcondition: $h_2 \geqslant \sqrt[3]{\frac{q^2}{g}}$ = 0.89				h_2 = +0.13+0.80 = +0.93 → (4) !				
	flowcondition (4)													
	+0.90	+0.76	+0.13	13,0	13,3									
	+0.90	+0.75	+0.13	13,2	13,2	1800	23,940	4,62	8,320	432.500	509.500	125,860	0.25	o.k.
t_{13}	+0.88	+0.74	+0.37	12,7	12,8	1800	23,400	4,32	7,780	420.000	503.640	133,640	0.27	+1.15
t_{14}	+0.87	+0.82	+0.68	10,3	10,0	1800	20,520	2,72	4,900	414.000	500.100	138,540	0.28	+1.15
t_{15}	+0.87	+0.87	+0.87	0	0	1080	5,400							
							361,620 m³.							

ANNEX V - 4 ENERGY LOSSES

1. Flow through pipes and barrels

The formula and coefficient for determining the discharge through a hydraulic structure can be derived from the basic orifice and weir formula

$Q = CA. \sqrt{2g. \ \Delta H}$, in which : Q = discharge in m^3/s

A = cross sectional area in m^2

g = gravity accelaration = $9.8 \ m/s^2$

ΔH = energy head loss in m

C = coefficient

The total headloss is composed of three component

$$\Delta H = dh_1 + dh_2 + dh_3$$

dh_1 = friction loss, to be calculated with the Manning equation

$$dh_1 = \frac{L \ n^2 \ v^2}{R^{4/3}} = \frac{2 \ L \ n^2 g}{R^{4/3}} \cdot \frac{v^2}{2g}$$

in which L = length of the culvert

n = roughness coefficient

R = hydraulic radius in m = cross sectional area A divided by wetted perimeter 0; for pipes $R = D/4$ in which D is internal diameter.

v = velocity in m/s

dh_2 = entrance convergence loss

$$dh_2 = K_e \cdot \frac{v^2}{2g}$$

K_e = entrance loss coefficient; for square cornered entrances $K_e = 0.5$

dh_3 = exist divergence loss of velocity head

$$dh_3 = K_v \cdot \frac{v^2}{2g}$$

K_v = exit velocity head coefficient;
for square cornered outlets $K_v = 1.0$

for structures with square cornered entrance and outlets.

$$\Delta H = \frac{v^2}{2g} \left(0.5 + 1.0 + \frac{2\,L\,n^2 . g}{R^{4/3}}\right)$$

or

$$\Delta H = \frac{v^2}{2g} . C.$$

$$v = \sqrt{\frac{2g.\,\Delta H}{C}}$$

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

$$Q = C_1\, A. \sqrt{2g.\,\Delta H} \quad \text{in which } C_1 = \sqrt{\frac{1}{C}}$$

2. Energy losses due to sudden transitions in flow-profile

For sub-critical flow passing through sudden transitions, experiments on various designs were made by Formica (ref. 2) as shown in Figure A·V-4.1. In closed conduits, the energy losses in a sudden contraction may be expressed as :

$$\Delta H_{con} = k_{con} \times \frac{v^2}{2g} ,$$

for sudden expansion, the energy loss can be assumed as:

$$\Delta H_{exp} = k_{exp} \times \frac{(\Delta v)^2}{2g}$$

in which K = coefficient

v = velocity downstream of contraction or expansion

Δv = difference in velocity upstream and downstream of sudden expansion

g = gravity acceleration.

Applying these equations to open channels Formica obtained the following values for k_{exp}:

type of design (see Figure A V-3.1)	1	2	3	4	5	6	7	8
k_{exp}	0.82	0.87	0.68	0.41	0.27	0.29	0.45	0.49

According to experimental data obtained by Formica, the values of k_{con} for sudden contractions seem to vary in wide range, generally increasing with the discharge. The approximate medium value of k_{con} for design (1) is 0.10 and for designs (2) to (4) is 0.06.

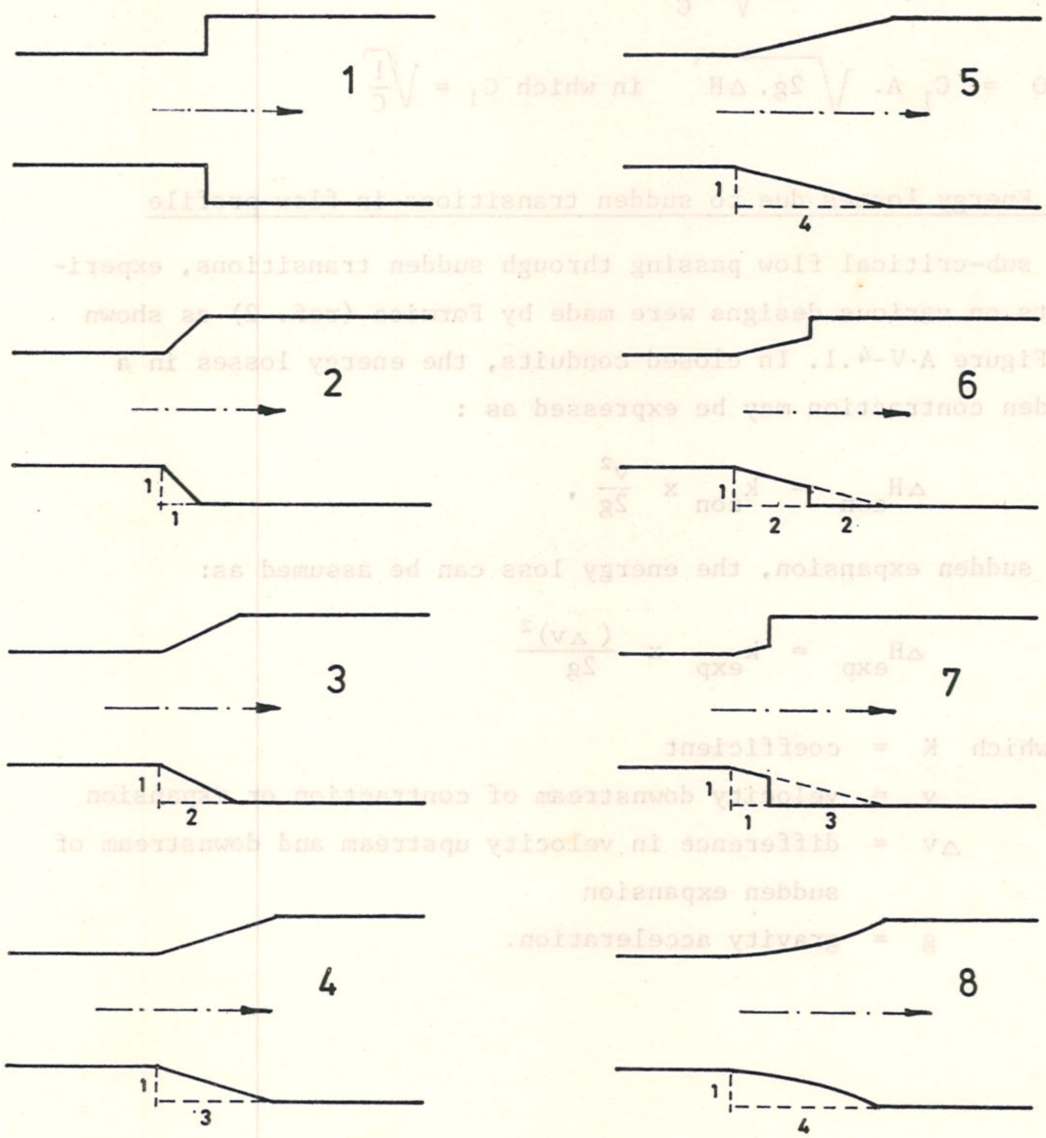


Figure A.V - 4.1 Various designs for sudden transitions for the experiments of Formica (ref. 2).

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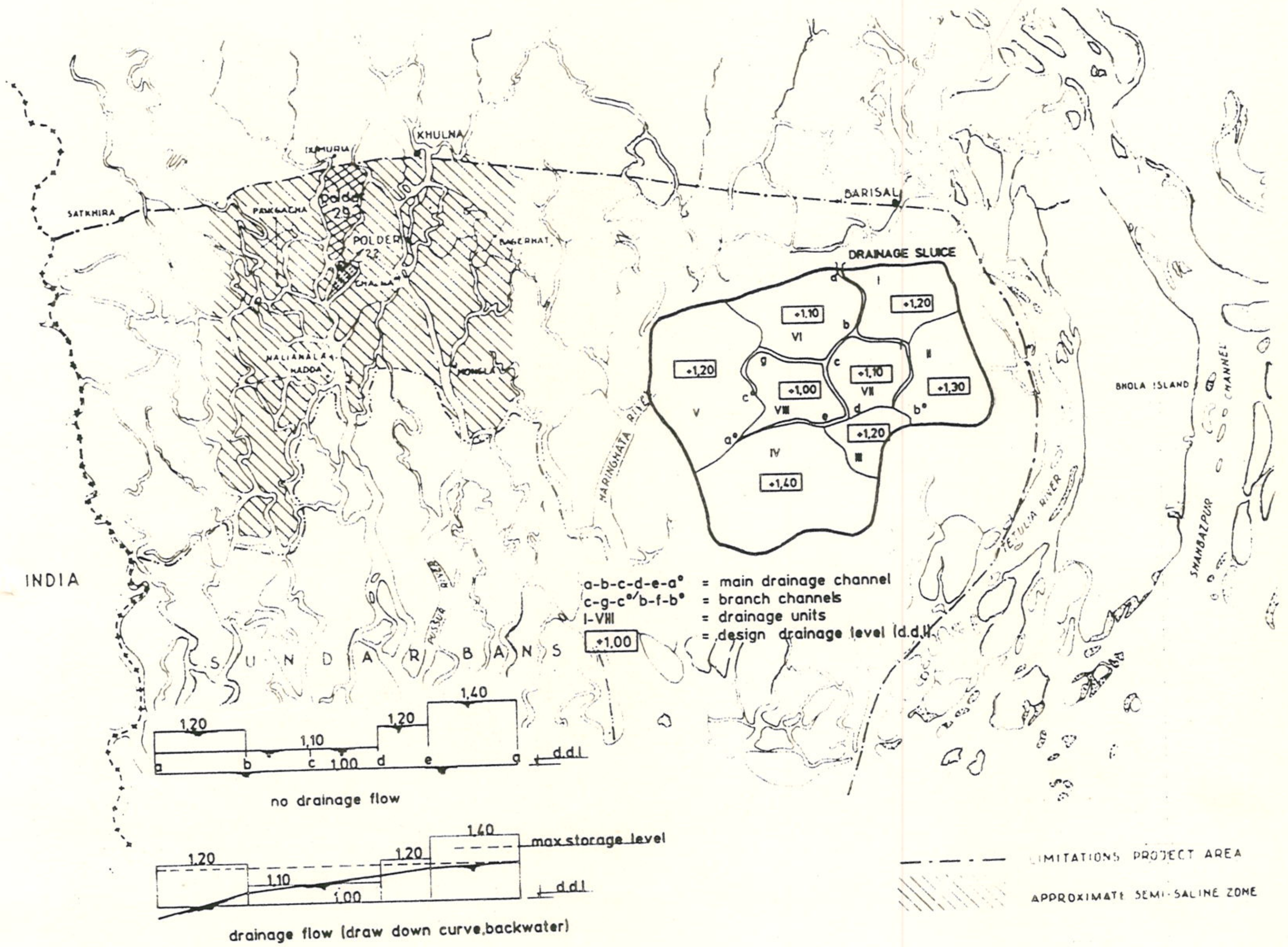
DESIGN MANUAL

FOR POLDERS IN SOUTH-WEST BANGLADESH



VOL. VI

DELTA DEVELOPMENT PROJECT

BANGLADESH-NETHERLANDS JOINT PROGRAMME
UNDER BWDB

DHAKA
NOVEMBER 1985

VOLUME VI. FOUNDATION DESIGN

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VOLUME VI. FOUNDATION DESIGN

Chapter 1. Introduction.

1.1 General.

The basic problems for the designer to solve with respect to foundations and soil mechanics are to design the structure itself strong enough so that :

- all external forces, excerted by water and earth can be resisted,
- the total forces excerted on the subsoil through the foundation are within permissible limits.

The problem therefore can be divided into three major subjects i.e. the soil pressure against and on top of the construction and the foundation design itself.

Furthermore, there is the problem of seepage under the construction and along the sides of it, which has to be blocked by means of cutoff walls. A related problem is the dewatering of excavations to be made for the construction works.

In the following Chapters these subjects will be further elaborated.

For the design works as mentioned above, information has to be available or gathered on the properties, characteristics and strength of the soil. In Volume II, Chapter 6, it is described in which way the soil investigations for a certain construction have to be arranged and what kind of tests are available to asses the soil properties.

Normally the design engineer is not physically involved with the soil investigations and soil testing programme but he will have to base his design on the information presented in the soil report. In the best case he has been involved with arranging and directing the soil investigations. However, the designer should know which tests are required to establish the soil properties needed and which soil properties are to be known for the specific structure he is going to design.

The soil properties which have to be known for the design of foundations for hydraulic structures are :

Table VI - 1.1

		F	W	S
1.	Soil type and description	X	X	X
2.	Unit weight	X	X	-
3.	Relative density of cohesion less soils	X	X	X
4.	Consistency of cohesive soils	X	X	X
5.	Compressibility*)	X	-	-
6.	Shearing resistance	X	X	-
7.	Cohesion	X	X	-
8.	Permeability	-	-	X

F = for foundation design

W = for retaining wall design or horizontal earthpressures

S = design of cutoff walls to prevent seepage

*) = only in case bad subsoil is present, consisting of peat or peat layers.

The laboratory tests to be performed to establish these properties are elaborated in Volume II, Chapter 6.

Before recommendations are made for a certain soil investigating programme, the approximate dimensions and costs of the structure are to be evaluated against the costs of a soil investigation programme.

1.2 Bearing capacity, shearing resistance.

The most important feature for the design engineer to know is the bearing capacity of the soil, being the average contact pressure on the soil at a stage that compression and settlement of the soil is still permissible for the structure, and no failure will occur due to exceedance of the shear strength of the soil.

In case of highly compressive soils, like peat, where settlements will become too big for the construction to bear, it is necessary to test the compressibility of the soil and determine which settlements are resulting from which loads. It may appear that the bearing capacity of the soil is the critical value for the foundation design.

If firm subsoil is available with no peat layers upto a depth of 15 m, then mostly the shear resistance will be the determinant factor. Settlements will in this case be limited. Failure might occur due to subsidence of the subsoil, because the increase of load causes exceedance of shear resistance. This subsidence is assumed to take place along the sides of a triangular shaped soil body under the foundation and along two curved slide planes as indicated in Figure VI - 1.1.

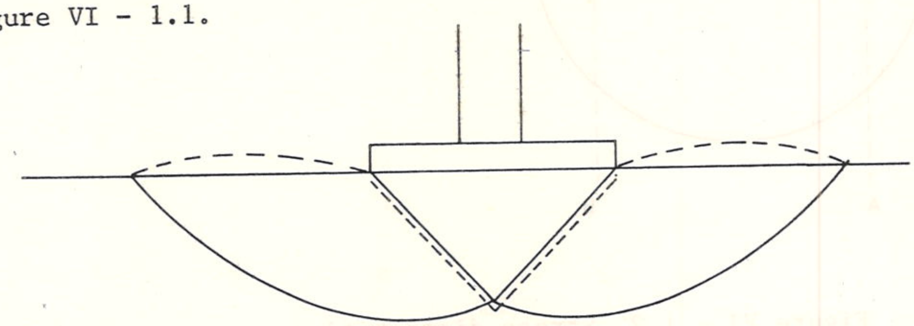

Figure VI - 1.1 Failure of soil under a footing.

It follows that the resistance against failure due to shear will increase as the foundation depth increases. Shearing will in this case be resisted by the load of the earth next to the foundation. Most formulas to calculate the soils bearing capacity are based on this phenomena.

1.3 Stress distribution in the soil beneath a load.

The stress in the soil may be defined as the total force acting on a certain area, devided by the area.

If an infinite plane area is loaded with a unit contact pressure p, the increase of the vertical stress is constant through the whole depth beneath the load and equal to p.

If the loaded area has limited dimensions the problem becomes more complicated. The pressure excerted by the load will be distributed because of the shearing stresses in the planes AA and BB of Figure VI - 1.2 which impede the settlements of the soil mass between these planes. With increasing depth the stresses in a horizontal plane will increase less.

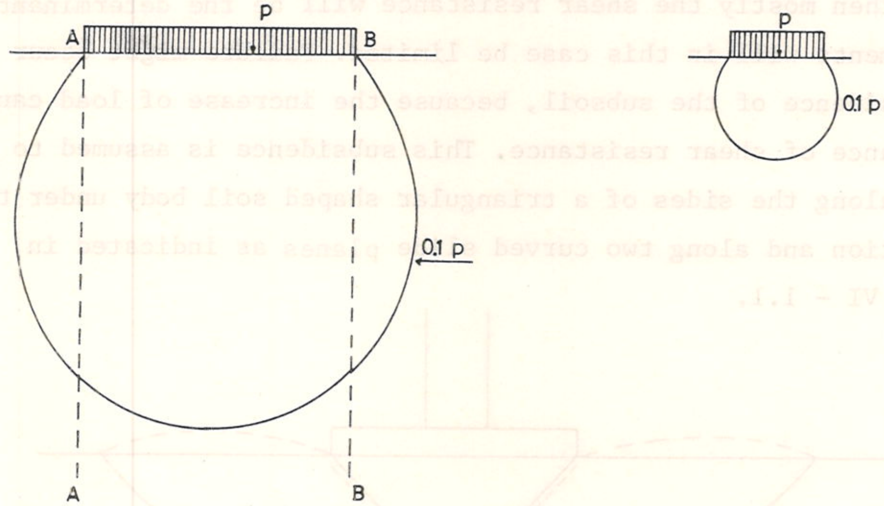

Figure VI - 1.2 Stress distribution.

It follows that the settlement of a loaded foundation is dependent on :

1. the maginitude of the load
2. the compressibility of the soil
3. but also of the size of the loaded area; the greater this area, the greater the depth over which a certain stress increase will extend below the ground surface (see Figure VI - 1.2).

This can also be made clear with a loaded slab of an area A and a certain settlement z. The stresses will be distributed sideways and the area immediately surrounding the slab will thus also get a settlement, though this will be smaller. If a second loaded slab is placed beside the first, the first slab will get a further settlement. So if we place nine equally loaded slabs in a square, the middle one will get the greatest settlement because it will be influenced by the eight surrounding slabs. An oil tank with a flexible bottom founded on a compressible soil will have a greater settlement beneath the centre, while the pressure is distributed uniformly over the bottom area. The bottom will assume a more or less spherical shape (Figure VI - 1.3).

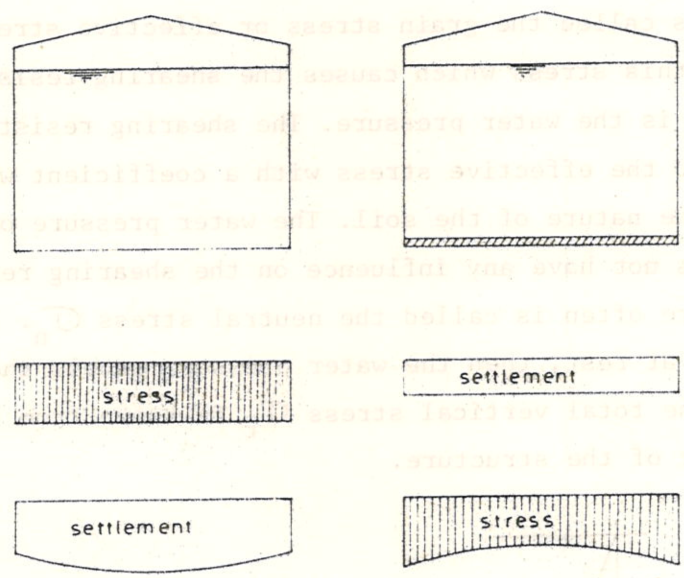

Figure VI - 1.3 Settlement and stress under rigid and flexible foundation plates.

On the other hand, if the bottom slab is perfectly rigid, the settlement is uniform, so the stresses are not uniformly distributed.

For practical purposes it is however assumed that the contact pressure between foundation and soil is uniformly distributed.

At the depth of about two times the smallest dimension of the foundation the stresses are distributed enough to say that the settlement due to the compression of the soil beneath this depth may be neglected. Therefore core borings often are carried out to a depth of about two times the width of the future foundation. It must, however, be born in mind that if there is more than one footing, these footings may influence each other if their intermediate distance is small, so the core borings have to be carried out to a lower level.

The total weight of a structure is transferred to the underlying soil via forces acting at the contact points of the grains and the water pressure. The total vertical stress equals the sum of these components :

$$\sigma_t = \sigma_w + \sigma_e$$

in which σ_e is called the grain stress or effective stress, because it is this stress which causes the shearing resistance of a soil; σ_w is the water pressure. The shearing resistance is proportional to the effective stress with a coefficient which is dependent on the nature of the soil. The water pressure on the other hand does not have any influence on the shearing resistance, and it therefore often is called the neutral stress σ_n. If the groundwater is at rest, then the water pressure equals the piezometric head. The total vertical stress σ_t is known from the total load and weight of the structure.

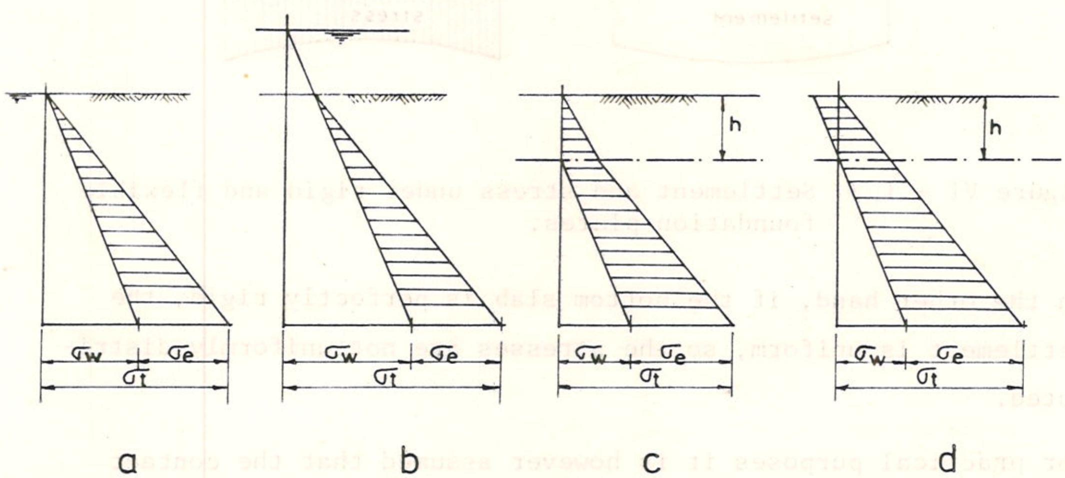

Figure VI - 1.4

In Figure VI - 1.4 the distribution of the total stress, the effective stress and the water pressure are shown, if the phreatic level coincides with the ground surface (a) and if the water level is above the ground surface (b). The effective stresses are the same in case (a) and (b). Case (c) and (d) give the situation if the phreatic level sinks a distance h below the ground surface. If the soil has no capillary action the diagram becomes according to (c),

σ_t decreases with h. $(\gamma_s - \gamma_d)$	γ_d = unit dry weight of soil (natural unit weight)
σ_w decreases with h. γ_w	γ_s = saturated unit weight of soil.
σ_e increases with h. $(\gamma_w - \gamma_s + \gamma_d)$	γ_w = unit weight of water

If the soil over the height h is filled with capillary water, the total vertical stress does not alter, so the effective stress increases with the same amount as the water pressure decreases (see (d)).

A lowering of the water table causes an increase of the effective stresses and, for compressible soils, settlements. This is a problem in polders where the requirements of agriculture lead to a lowering of the water table, which causes settlements at locations with peat layers and means that a further lowering of the water table will be necessary, a.s.o.

1.4 Equilibrium in soil.

If we consider the equilibrium of the soil part ABCD of Figure VI - 1.5, which forms part of an infinite slope, then the forces P, W, and R, that form an equilibrium with the weight G, are all acting under an angle α with the normal of the plane on which they act. They all excert shearing forces on those planes.

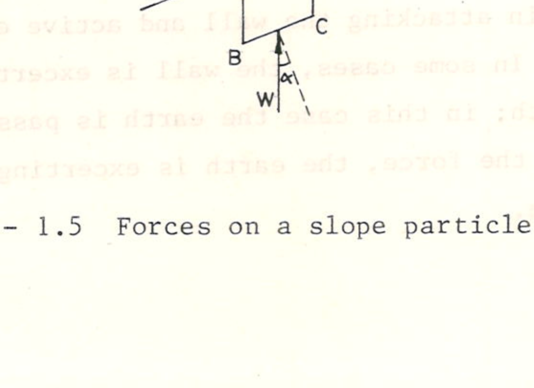

Figure VI - 1.5 Forces on a slope particle.

If the shearing force exceeds a critical value then the soil will subside. For non cohesive soils the critical value of the shearing stress is dependent on the magnitude of the effective stress on that plane. The most direct and simple method to determine the relation between critical shear stress and effective stress is the direct shear test (see Volume II, Chapter 6.3.4). The result may be plotted graphically with the shear stress on the vertical axis and the effective stress on the horizontal axis. The results show a straight line :

$$\tau_{max} = \sigma_e . \tan \varphi + c$$

c = cohesion

φ = angle of internal friction, for sand : $25^o < \varphi < 40^o$; average $\varphi = 30^o$

For a cohesive soil sample (clay) the result will show an almost horizontal line, or, the shearing stress seems independent of the effective stress and is apparently zero. The reason is that if the load is applied quickly, it is taken by the waterpressure and the effective stress will not immediately increase. Because σ_e does not increase, τ does not either. However, if the test is done at a slow rate, giving over-pressed pore water the time to flow away, the result will yield also for clay an angle of internal friction.

1.5 Active, Passive, Neutral earth pressure.

Retaining walls, abutments, sluice wingwalls have to provide lateral support for the adjacent soil masses. These soil masses will excert forces on the wall, which have to be calculated in order to design the wall strong enough.

In most cases the wall has to prevent the soil from falling in. In Figure VI - 1.6, the wall is pushed to the left and the earth is actively involved in attacking the wall and active earth-pressure is excerted. In some cases, the wall is excerting a force against the earth; in this case the earth is passively involved in resisting the force, the earth is excerting a passive earth pressure.

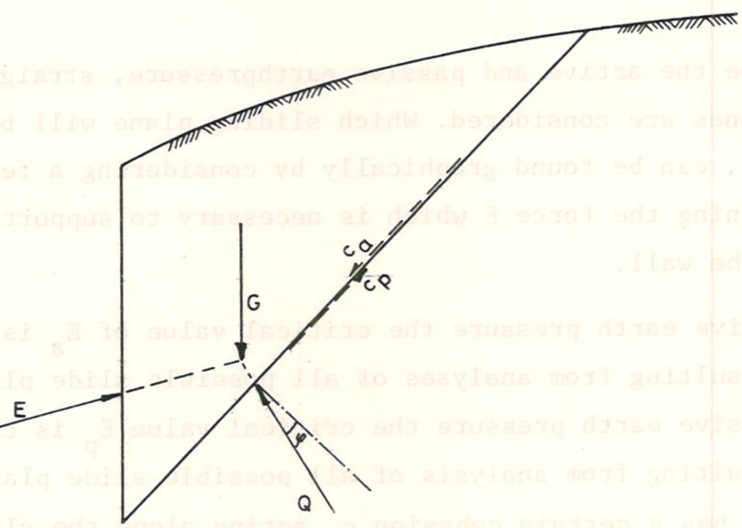

Figure VI - 1.6

To calculate the earth pressure, active, or passive, a theory is adopted assuming straight sliding planes.

The use of earth pressure theory to estimate soil pressure against a construction is valid only if the following three conditions are satisfied:

1. The construction, or part of it, can yield sufficiently by tilting or sliding, so that the full shearing resistance of the backfill can develop.
2. The pore water pressure in the backfill is negligible.
3. The soil properties appearing in the earth pressure equations have values that can definitely be determined.

The amount of deflection necessary for full shearing resistance to develop is in the order of 0.1% of the wall height, and can almost always be expected to develop except where the wall movement is physicially restrained, for instance, earth pressure against the barrel of a sluice. In such cases special care has to be taken. Pore water pressure will usually be negligible if a properly designed drainage system is constructed behind the wall. The third condition is often not so easily satisfied. Frequently constructions must be designed when the only information about a soil to be used for backfill is its general type. In such cases soil properties should be assumed which, from experience, have been found satisfactory.

To determine the active and passive earthpressure, straight sliding planes are considered. Which sliding plane will be determinant, can be found graphically by considering a few cases and determining the force E which is necessary to support or to push back the wall.

For the active earth pressure the critical value of E_a is the minimum, resulting from analyses of all possible slide planes. For the passive earth pressure the critical value E_p is the maximum resulting from analysis of all possible slide planes. If the soil has a certain cohesion c, acting along the sliding plane, then this has to be taken into account.
If the wall is vertical, the groundsurface horizontal and the direction of E horizontal, then the active and passive earth-pressure can be calculated analytically as :

$$E_a = \tfrac{1}{2} \gamma h^2 . \tan^2 (45^o - \varphi/2) - 2. h.c. \tan (45^o - \varphi/2)$$

$$E_p = \tfrac{1}{2} \gamma h^2 . \tan^2 (45 + \varphi/2) + 2. h.c. \tan (45^o + \varphi/2)$$

In which γ = unit weight of the soil

c = cohesion

φ = angle of internal friction

For non cohesive soils (c = 0) the formulas are simplified to :

$$E_a = k_a . \tfrac{1}{2} . \gamma h^2$$

$$E_p = k_p . \tfrac{1}{2} . \gamma h^2$$

In which k_a and k_p are coefficients depending on φ. The most common used methods to determine the values of these coefficients are based on the theories of Coulomb and Rankine. Which of these theories is to be applied will be discussed in Chapter 2.

The value of the earthpressure may vary between E_a and E_p. Sometimes it is important to know the magnitude of the earth pressure in case a wall can not or is not allowed to displace. E.g. abutments of moveable bridges have to be rigid, coupled walls of a sluice barrel can not displace. The earth pressure which has to be resisted in these cases is called the <u>neutral earth pressure</u>. The coefficient for neutral earth pressure k_n has the same meaning as k_a and k_p. Roughly it may be admitted that, k_n for nearly all kind of soils lies between 0.5 and 0.6 (empirically established).

In the above it was assumed that failure will occur along straight sliding planes. If curved sliding planes would have been considered, the resulting earth pressure might have been more unfavourable. In other words: a failure along a curved sliding plane might take place at an earlier stage than along the most unfavourable straight plane. The result would be an E_a which is bigger than in case of straight sliding planes and E_p which is smaller. However, the differences for the active earthpressure are only small and E_a may be determined under the assumption of straight sliding planes. The values determined for E_p under assumption of straight sliding planes must be used with great care.

1.6 Types of foundation.

Foundations have the function to transfer the load of the structure which can be of high stress intensity to the supporting capacity of the soil, which can be rather low.

The transmission of stresses can take place via a shallow foundation, of which the depth of load transfer is less than the least dimension of the footing, or via a deep foundation, (pile or caisson) with depth of foundation exceeding the width.

Which type of foundation should be applied depends fully on the structural load and the characteristics of the foundation soil. The foundation should in any case provide an adequate means to support the structure without causing shear failure or untolerable settlements.

This support can be achieved by spreading the load over a bigger area, or by transferring the load via members to lower levels, making use of the friction and/or end bearing of the members. Spreading the load can be done by making use of spread footings or rafts, resulting generally in a shallow foundation.

For deep foundations, piles, caissons or sheet piling can be used. shallow foundations will be elaborated in Chapter 4, pile foundations will be highlighted in Chapter 5.

1.7 Criteria and Definitions.

Before the design of the foundation can start, the forces acting on the structure and through the structure on the underlying soil, should be analysed with respect to the stability of the structure. Then it should be investigated whether the soil underlying the foundation is capable of supporting the loads superimposed by the structure, considering the performance of the foundation with respect to :

- bearing capacity, or strength of the supporting soil, and
- allowable settlements

A bearing capacity failure is characterized by the structure breaking into the soil because the material is incapable of supporting the load. Settlement is characterized by the sinking of the structure due to compression and deformation of the underlying material. These two behaviours are independent and are investigated separately. Both, however, are caused by excessive foundation pressure.

At first the structures safety with respect to stability will be discussed.

Instability may be caused by overturning forces exceeding the counterbalancing forces or lateral forces exceeding the friction forces acting along the foundation bottom or exceeding the shear stress of the soil which has to resist sliding. In the foundation design, the safety factor against overturning is taken as :

$$S_{fo} = 1.5$$

The safety factor against sliding is :

$S_{fs} = 2$ in case passive earthpressure can develop.

$S_{fs} = 1.5$ in case of absence of passive earth pressure.

Safe soil pressures for bearing are based on the ultimate bearing capacity of the soil. To satisfy the requirements of safety, the safe soil pressure under maximum load is usually limited to ½ or 1/3 the ultimate bearing capacity. That is, the factor of safety against a bearing capacity failure under maximum load should be 2 for non cohesive soils and 3 for cohesive soils.

With respect to settlements that can be allowed for hydraulic structures the following should be considered. Settlements may be uniform or differential, depending upon the load distribution and the soil characteristics. A uniform settlement , with the foundation remaining plane, will not create significant problems as long as the amount of settlement is within tolerable limits. When the foundation pressures are not uniform, or the foundation soil varies in strength, differtial settlement will occur. Differential settlements are defined as the difference in settlements between two points of the foundation. It is generally accepted that the maximum differential settlement may not exceed 3/4 of the computed total maximum settlement. For hydraulic structures the maximum settlement that may occur is set at 5 cm. However it should be kept in mind that the size, type and importance of the structure, will be of influence.

For instance, if a drainage sluice is provided with a sliding gate, which is to be operated along two parallel slots which are 6 m high from bottom to top, and the gate has a play of 2.5 cm in horizontal direction, the maximum allowable tilt of the structure is $\frac{0.025}{6} = 4 \times 10^{-3}$. If the width of the foundation is also 6 m, then the differential settlement should not be more than 2.5 cm, which indicates a maximum total settlement of $\frac{4}{3} \times 2.5$ cm $= 3{,}3$ cm.

Greater settlements may cause problems with operating the sliding gate.

In the following chapter the under mentioned terms will be used.

- <u>Total foundation pressure</u> = total of all forces acting on the base of a foundation, comprising structures dead weight,