2.5 Design of Drainage Sluice

a) The design discharge as calculated under 2.44 - c) was 23.24 m^3/s during the second half of June.

b) The water level boundary conditions are :

in the polder : design drainage level + 1.30 m.

maximum allowable storage level + 1.60 m

low polderlevel at flushing operations + 1.00 m

at the end of the drainage period T (starting at a water level of H_i = + 1.45 m), the average polderwater level will be + 1.15 m.

river water : M.H.W. is + 2.20 m

M.L.W. is + 0.34 m — See ANNEX IX- 5 and 6

F.M.L. is + 1.29 m

H.H.W. is + 2.47 m — See ANNEX IX-7

L.L.W. is + 0.21 m

c) The invert level of the sluice is now determined by the maximum velocity criteria at extreme drainage conditions: i.e. riverwater at L.W.S. (+0.21 m)polderwater at m.a.st.l. (+1.60 m.PWD)

Choose flowcondition :

first (5) : $q = C \times (H_x)^{1.5} \leqslant \frac{(v_{max})^3}{g} = 4.375$

take C = 1.60 $\longrightarrow$ H_x = 1.96 m.

Water level at sluice during maximum discharge is estimated as :

$$Q_{max} = Q_{av} \times \frac{1}{0.85} = 23.24 \times \frac{1}{0.85} = 27.34 \ m^3/s.$$

From the delivery curves of ANNEX IX-16 follows:

$Q = 27.34 \ m^3/s$; H_i = + 1.60 m ; H_x = + 1.20 m.

Choose invert level at level PWD 1.20 - 1.96 = - 0.75 m.

d) The width of the sluice follows from average drainage discharge Q = 23.24 m^3/s at the level of H_i at the start of the drainage period T (H_i = + 1.45 m) and at the end of T (H_i = + 1.15 m).

Assume critical flow through the sluice between the level of H_i = + 1.43 m and H_i = 1.17 m, for which ANNEX IX-16 provides the corresponding values of H_x = + 1.18 and H_x + 0.06 for Q = 23.24 m^3/s.

The average headwater at the sluice during the critical flow-condition through the sluice during the drainage period T is thus ½ (1.18 + 0.06) = + 0.62 m. The sluice invert level was established at - 0.75 m. The average head upstream of the sluice is thus + 0.62 + / - 0.75 / ≈ + 1.40 m.

The average sluice dischargeper unit width is thus:

$q = 1.35 \times (1.4)^{1.5} = 2.24\ m^2/s.$

The required average sluice discharge was 23.24 m^3/s.

Thus the width of the sluice is :

$$\frac{23.24}{2.24} = 10.38\ m.$$

e) Check sluice velocity at maximum drainage discharge with polder water level at H_i = + 1.60 m and riverwater at + 0.21 m.

Assuming flow condition (5), the flow through the sluice can be expressed as :

$$Q_{sl} = 1.35 \times 10.38 \times (H_x + / -0.75 /)^{1.5}$$

This curve is plotted in ANNEX IX-16.

Where $Q_i = Q_{sl}$ and H_i = 1.60, it yields for H_x = + 0.90 m and Q = 29.7 m^3/s.

Check flow condition : (5) o.k. if $h_{cr} \geqslant h_2$

$$\left.\begin{array}{l} h_2 = / - 0.75 / + 0.21 = 0.96\ m \\ h_{cr} = \sqrt[3]{\frac{q^2}{g}} = 0.94 \end{array}\right\} \text{flow condition (5) does not prevail!!}$$

Therefore step c) has to be repeated for another flow condition:

c) Choose flow condition (4):

$$q = 0.82 \times h_2 \times \sqrt{2g\ (h_1 - h_2)} \leqslant 4.375$$

from ANNEX IX-16 ; Q_{max} = 30 m^3/s

H_i = 1.60

H_x = 0.86

h_2 = 0.21 + I.L. (= Invert Level)

$$q = 0{,}82 \times (0{,}21 + / \text{I.L.} /) \times \sqrt{2g(0.86 - 0.21)} \leqslant 4.375$$

I.L. = - 1.28 m PWD.

Check flow condition : $\left.\begin{array}{l} h_{cr} = \sqrt[3]{\frac{q^2}{g}} = 1.25 \\ h_2 = 1.49 \end{array}\right\}$ o.k. (4)

d) The width of the sluice follows now from : average H_x during $Q_{av} = 23.24\ m^3/s$ is + 0.62 m.

I.L. = - 1.28 $\longrightarrow$ $H_x = + 0.62 + / - 1.28 / = 1.90$ m.

average M.L.W. is at + 0.34.

assume flow condition (4) :

$$q = 0.82 \times h_2 \times \sqrt{2g\ (h_1 - h_2)}$$

$h_1 = + 1.90$

$h_2 = + 0.34 + \text{I.L.} = 1.62$ m.

$$q = 0.82 \times 1.62 \times \sqrt{2g(1.90 - 1.62)} = 3.11\ m^2/s.$$

required width : $b = \frac{23.24}{3.11} = 7.46$ m.

Check flow condition : $q = 3.11\ m^2/s$

$\left.\begin{array}{l} h_{cr} = \sqrt[3]{\frac{q^2}{g}} = 1.00 \\ h_2 = + 1.62. \end{array}\right\}$ (4) is o.k.

e) Check sluice velocity at maximum drainage discharge:

assume flow condition (4) :

$$Q = 0.82 \times 7.46 \times h_2 \times \sqrt{2g\ (h_1 - h_2)}$$

$h_2 = + 0.21 + \text{I.L.} = 1.49$

$$Q = 0.82 \times 7.46 \times 1.49 \times \sqrt{2g\ (H_x - 0.21)}$$

This curve is plotted in ANNEX IX-16 and yields for $H_i = + 1.60$ and $Q_i = Q_{sl} = 30.6$, $H_x = 0{,}79$ m.

Step c) could now be repeated again to correct the assumptions on H_x and Q_{max}.

This will ultimately lead to an invert level of - 1.37 and a corresponding sluice width of 7,07 m.

The extreme velocities in the sluice will then be :

$$Q = 0{,}82 \times 7.07 \times h_2 \times \sqrt{2g\,(h_1 - h_2)}$$

$$h_2 = +\,0.21 + I.L. = 1.58$$

$$Q = 0{,}82 \times 7{,}07 \times 1.58 \times \sqrt{2g\,(H_x - 0.21)}$$

yields for $H_i = 1.60$, $Q_i = Q_{sl} = 30{,}7\ m^3/s.$ $H_x = 0{,}78.$

Invert level - 1.39:

$$\left.\begin{array}{l} \text{check flow conditions : } h_{cr} = \sqrt[3]{\dfrac{q^2}{g}} = 1.24 \\ \text{for extreme conditions: } h_2 = 1.60 \end{array}\right\} \text{o.k. } (4)$$

for average drainage flow:

$$q = 0{,}82 \times h_2 \times \sqrt{2g\,(h_1 - h_2)}$$

$$h_1 = +\,0.62 + /\,-\,1.39\,/ = 2.01\ m.$$

$$h_2 = +\,0{,}34 + /\,-\,1.39\,/ = 1.73\ m.$$

$$q = 0{,}82 \times 1.73 \times \sqrt{2g\,(2.01 - 1.73)} = 3{,}32$$

$$\left.\begin{array}{l} h_{cr} = \sqrt[3]{\dfrac{q^2}{g}} = 1.04 \\ h_2 = 1.73 \end{array}\right\} \text{o.k. } (4)$$

check maximum velocity during extreme drainage :

$$q = 0.82 \times h_2 \times \sqrt{2g\,(h_1 - h_2)}$$

$$h_1 = +\,0{,}78 + I.L. = 2.17\ m.$$

$$h_2 = +\,0.21 + I.L. = 1.60\ m.$$

$$q = 0.82 \times 1.60 \times \sqrt{2g\,(2.17 - 1.60)} = 4{,}385\ m^2/s \approx 4{,}375.$$

Therefore final choise for :

invert level	: - 1.35 m
sluice width	; 4 vents of 1.80 m = 7.20 m.

Maximum drainage capacity :

$$Q = 0.82 \times 4 \times 1.80 \times 1.56 \times \sqrt{2g\,(H_x - 0.21)}$$

$$H_i = +\,1.60,\ Q_{sl} = Q_i = 30{,}8 \quad H_x = 0{,}78\ m.$$

f) Determination of the barrel - height

From, m.a.st.l. to invert level is + 1.60 + / - 1.35 / = 2.95 m.

Criteria: $H \leqslant 1{,}2\ d.$ $\qquad d \geqslant \frac{H}{1.2} = \frac{2.95}{1.2} = 2.46$ m.

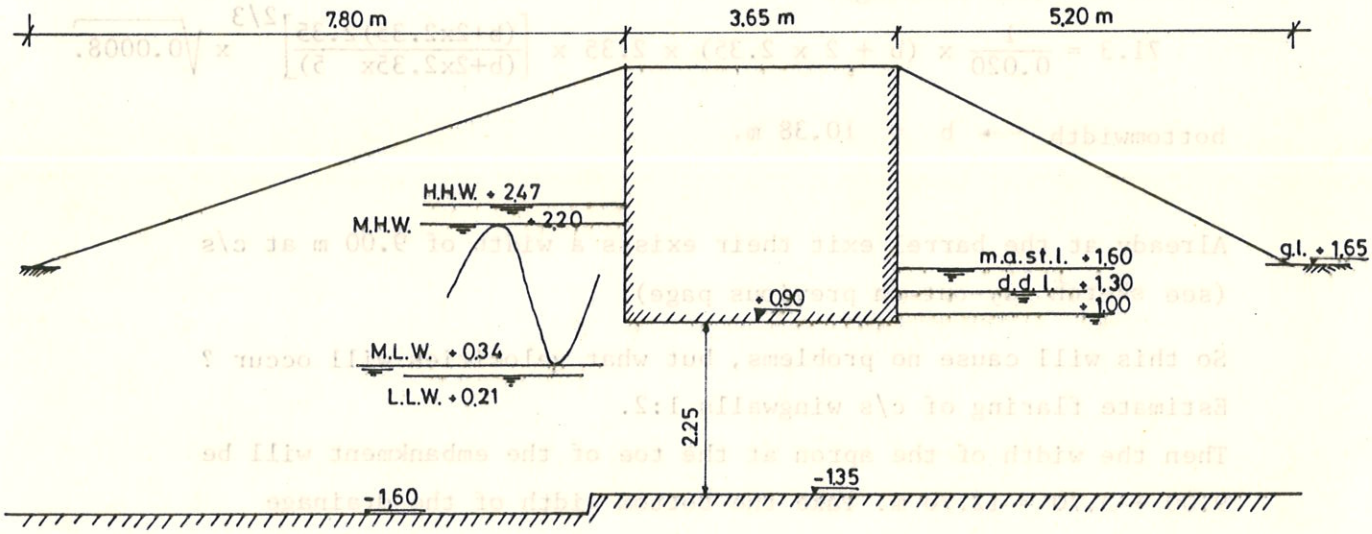
if d.d.l. is chosen :

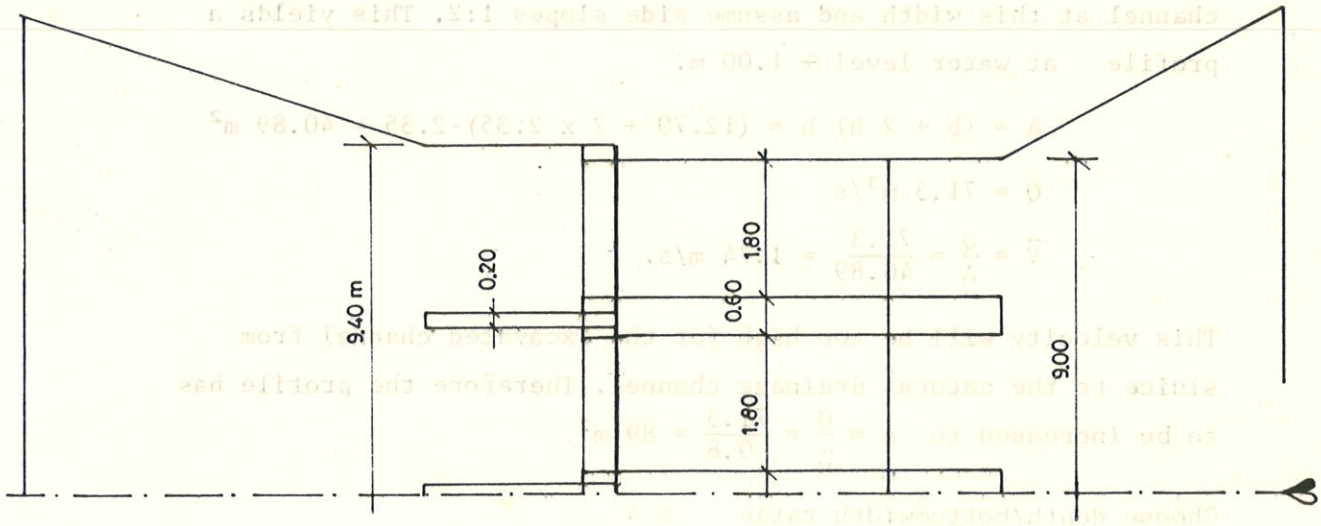
$H = +1.30 + / - 1.35 / = 2.65 \longrightarrow d \geqslant 2.21$

Since draw down effect on the water level in the drainage channel close to the sluice is considerable, it is chosen to adopt the latter size for the barrel height, c.q. d = 2.25 m.

The ceiling of the barrel now comes at + 0.90 m.

g) A sketch lay-out is now made:





h) Check maximum velocities during flushing operations:

Riverwater at + 2.47 m

Polderwater at + 1.00 m.

Asssume full barrel flow : flow condition (1) :

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

$C_1 = 0.82$

$$Q = 0.82 \times (4 \times 1.80 \times 2.25) \times \sqrt{2g(2.47 - 1.00)} = 71.3 \ m^3/s.$$

velocity in the barrel :

$$V = \frac{Q}{b \times h} = \frac{71.3}{4 \times 1.80 \times 2.25} = 4.4 \ m/s.$$

Area of drainage channel required to convey the flushing flow of 71.3 m^3/s. (use Manning).

$$71.3 = \frac{1}{0.020} \times (b + 2 \times 2.35) \times 2.35 \times \left[\frac{(b+2 \times 2.35)\,2.35}{(b+2 \times 2.35 \times \ \ 5)}\right]^{2/3} \times \sqrt{0.0008.}$$

bottomwidth $\longrightarrow$ b = 10.38 m.

Already at the barrel exit their exists a width of 9.00 m at c/s (see sketch lay-out on previous page).

So this will cause no problems, but what velocities will occur ? Estimate flaring of c/s wingwalls 1:2.

Then the width of the apron at the toe of the embankment will be 9.00 + 3.70 = 12.70 m. Take the bottom width of the drainage channel at this width and assume side slopes 1:2. This yields a profile at water level + 1.00 m.

$$A = (b + 2\ h)\ h = (12.70 + 2 \times 2.35) \cdot 2.35 = 40.89 \ m^2$$

$$Q = 71.3 \ m^3/s$$

$$\bar{V} = \frac{Q}{A} = \frac{71.3}{40.89} = 1.74 \ m/s.$$

This velocity will be too high for the excavated channel from sluice to the natural drainage channel. Therefore the profile has to be increased to $A = \frac{Q}{\bar{v}} = \frac{71.3}{0.8} = 89 \ m^2$.

Choose depth/bottomwidth ratio ≈ 4.

$A = (b + 2\,h)\,h = (b + \frac{2.b}{4}) \cdot \frac{b}{4} = 89 \longrightarrow b = 15.40$ m.

bottom depth is then at $+ 1.00 - \frac{15.40}{4} = - 2.85$ m.

If maximum attainable bottom depth is assumed to be - 1.60 , then the bottom width of the connection channel should be .

$$(b + 2 \times 2.6)\ 2.6 = 89 \longrightarrow b = 29.0 \text{ m.}$$

The connecting channel is designed at b = 29.0 m giving a profile of 89 m^2 at water level + 1.00 m and bottom depth - 1.60 m with side slopes 1:2.

Velocity head in the channel is then: $\frac{v^2}{2g} = 0.03$ m.

The velocity head at river side can be neglected since velocities are very small.Energyhead is therefore assumed to be water level (+ 2.47 m) at the r/s and + 1.00 + 0.03 = 1.03 m at c/s.

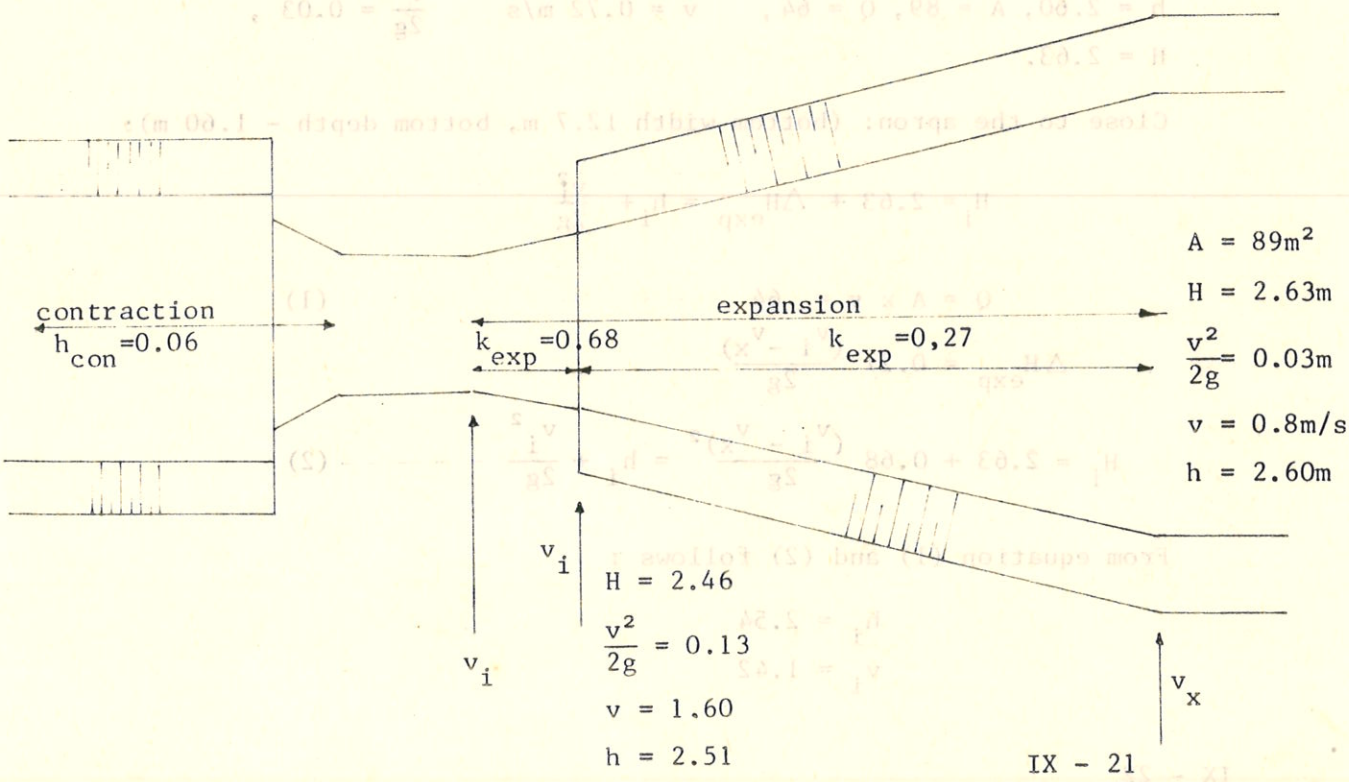
See ANNEX IX-17.

$H_o = + 2.47 + / - 1.35 / = 3.82$ m

$H_x = + 1.00 + 0.03 + / - 1.35 / = 2.38$ m.

$\Delta H_{tot} = 3.82 - 2.38 = 1.44$ m

$\Delta H_{con} = 0.06 \times \frac{(v)^2}{2g}$; $H_{exp} = k_{exp} \times \frac{(v_2 - v_1)^2}{2g}$



first approximation for expansion losses at c/s apron :

$$v'_i = v'_o = \frac{Q}{H_{box}} = \frac{71.3}{4x1.8x2.25} = 4.4 \text{ m/s.}$$

$$v_i = \frac{71.3}{(12.7+2x2.51)2.51} = 1.60 \text{ m/s.}$$

$$\Delta H_{con} = 0.06 \times \frac{(4.4)^2}{2g} = 0.06 \text{ m}$$

$$\Delta H_{exp} = 0.68 \times \frac{(4.4 - 1.60)^2}{2g} = 0.27 \text{ m.}$$

$$\Delta H_{tot} = 1.44 = \Delta H_{con} + \Delta H_{box} + \Delta H_{exp}$$

$$= 0.06 + \Delta H_{box} + 0.27$$

$$\Delta H_{box} = 1.11$$

$$Q_{box} = 0.82 \times (4 \times 1.80 \times 2.25) \times \sqrt{2g(1.11)} = 62.0 \text{ m}^3/\text{s.}$$

We started with a Q = 71.3 m^3/s.
By iteration it is found that the best fit Q = 64 m^3/s. Now on ANNEX IX-17 the water levels can be plotted.

In the drainage channel (bottom width 29 m, bottom depth - 1.60 m):
h = 2.60, A = 89, Q = 64, v = 0.72 m/s $\frac{v^2}{2g} = 0.03$,
H = 2.63.

Close to the apron: (bottom width 12.7 m, bottom depth - 1.60 m):

$$H_i = 2.63 + \Delta H_{exp} = h_i + \frac{v_i^2}{2g}$$

$$Q = A \times v = 64 \quad \text{- - - - - - - - - - - - (1)}$$

$$\Delta H_{exp} = 0.27 \frac{(v_i - v_x)}{2g}$$

$$H_i = 2.63 + 0.68 \frac{(v_i - v_x)^2}{2g} = h_i + \frac{v_i^2}{2g} \quad \text{- - - - - - (2)}$$

From equation (1) and (2) follows :

$$h_i = 2.54$$
$$v_i = 1.42$$

$$\frac{v_i^2}{2g} = 0.11$$

$$\Delta H_{exp} = 0.02$$

$$H_i = 2.65 \text{ (measured from - 1.60 m level).}$$

For the apron the calculation continues :

$Q = 64\ m^3/s$

$v_i = 1.42\ m/s.$

$v_{box} = \frac{64}{4x1,8x2,25} = 3.95\ m/s.$

$\Delta H_{con} = 0,05$

$\Delta H_{exp} = 0,68 \times \frac{(3.91-1.42)^2}{2g} = 0,22$

It yields $H_i' = H_i + \Delta H_{exp} = 2.40 + 0.22 = 2.62$ m (measured from - 1.35 m level).

$$H_o' = H_o - \Delta H_{con} = 3.82 - 0.05 = 3.77 \text{ m.}$$

At x-section I : $H_i' = 2.62 = h_i + \frac{(v_i)^2}{2g}$; $v_i = \frac{64}{h_i' \times 9}$; $v_i' = 3.70$ m/s ; $h_i' = 1.92$ m.

At x-section II: $H_o' = 3.77 = h_o' + \frac{(v_o')^2}{2g}$; $v_o' = \frac{64}{(h_o'+0.25)x9.40}$; $h_o' = 3.60$ m ; $v_o' = 1.81$ m.

Plotting of the water levels shows that where the water leaves the barrels at c/s the elevation is below the ceiling of the box, i.e:

box ceiling at + 0.90

Water level (h_i') at 1.92 - 1.35 = + 0.57.

This means that flow condition (1) will not prevail. Therefore flow will be under flow condition (3). For the discharge this will not make to much difference as can be checked with :

$$Q = C_3 \times 4 \times 1,8 \times 2,25 \times \sqrt{2g \times H_o}$$

$$\frac{d}{H_o} = \frac{2.25}{3.82} = 0.589 \longrightarrow C_3 = 0,495$$

$$Q = 0,495 \times 7.2 \times 2.25 \times \sqrt{2g \times 3,82} = 69,4\ m^3/s.$$

But the flow in the barrel will be contracted to a depth of $c_c \times d$ (see Figure V-3.3) and will be 2.25 x 0.613 = 1.38 m.

The velocity of flow will be $v = \frac{Q}{bxh} = \frac{69.4}{4x1.8x1.38} = 6.98$ m/s

and Froude number : $F = \frac{v}{\sqrt{gh}} = 1.90$.

A turbulent jump-like water surface will develop from approximately halfway the barrel to a distance L downstream.

From Figure V-4.16 follows $\frac{L}{d_2} = 4.2$

and since d_2 = (+1.00 + / - 1.35 /) = 2.35 , L = 4.2 x 2.35 = 9.87 m

i) The protection provided by the barrel and apron/wingwall construction extends up to 7 m if the jump is measured from the middle of the barrel to the c/s toe of the embankment/c.q. apron. It is therefore advisable to extend the floor with another 3 m before the transition to the connecting channel profile is made.

The flaring of the wingwalls of the c/s apron should now be $\frac{1}{3F} = \frac{1}{3 \times 1.9} = \frac{1}{5.7}$; choose 1:6.

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

Length of flaring : 3.70 + 3 = 6.7 m.

flaring angle 1:6.

width of apron at ending: $9.00 + \frac{6.7}{6} \times 2 = 11.23$ m

waterdepth : 2.35 m. (approximately)

profile : 2.35 x 11.23 = 26.40 m^2

Velocity : $\frac{69.4}{26.4} = 2.63$ m/s.

transition to channel with profile of 89 m^2 to be made with mattress construction.

size of bottom armour units : 550 mm. if γ_{st} is assumed to be 1.8 - 1.9.

In this case a combination of brick blocks pitching and mattress construction is chosen.

Velocity at channel section (bottomwidth 29 m, bottom depth - 1.60 m):

h = 2.60 m Q = 69.4 A = 89 m^2, v = 0.78, H = 2.63.

Velocity at channel section (bottomwidth 12.7 m, bottom depth - 1.60 m) : $H = 2.63 + 0.27 \frac{(v - 0.78)^2}{2g} = h + \frac{v^2}{2g}$.

$Q = (12.7 + 2\ h)\ h \times v$, yielding. $v = 1.56$ m/s, $h = 2.51$

From end of apron ($v = 2.63$ m/s) upto where channel bottom is 12.7 m ($v = 1.56$ m/s) a brick block pitching with units of 81 kg, placed in a "dented" pattern are to be applied. (alternating blocks placed on its side, to improve bottom roughness). Make this section long : 10 m from the end of the apron.

Remaining transition zone upto channel bottomwidth of 29 m : apply mattress with unit weights of ballast material :

$v = 1.56$ m/s	diameter 180 mm	($\Delta = 0.8$)
$v = 0.8$ m/s	diameter 50 mm	($\Delta = 0.8$)

Use full size bricks (5 kg) or furnace burns for mattress ballasting.

j) <u>Velocities during maximum drainage operations</u>

Polderwater level $H_i = +1.60$

Q_{max} is determined as $Q = 30.8\ m^3/s$

downstream at the sluice $H_x = +0.78$ m.

water level at r.s. : L.L.W. = + 0.21 m.

The flow through the sluice is shown on ANNEX IX-18.

Flow condition (3) :

$$Q = C_3 \times A \times h_2 \times \sqrt{2g(h_1 - h_2)}$$

$$= 0.82 \times 7.20 \times (1.35 + 0.21) \times \sqrt{2g(0.78 - 0.21)}$$

$$Q = 30.8\ m^3/s$$

$$q = 4.28\ m^2/s$$

$$\left.\begin{array}{l} h_{cr} = \sqrt[3]{\frac{q^2}{g}} = 1.23\ m \\ h_2 = (1.35 + 0.21) = 1.56. \end{array}\right\} \text{ flow condition o.k.}$$

The water levels in the sluice can be calculated.

In the drainage channel :

$$h = (/-1.35/+ 0.78) = 2.13\ m.$$

$$A = (29 + 2 \times 2.38) \times 2.38 = 80.3\ m^2$$

$$\left.\begin{array}{l} Q = 30.8\ m^3 \\ v = 0.38\ m/s \end{array}\right\} \quad \frac{v^2}{2g} = 0.01 \longrightarrow H = 2.14\ m.$$

Before the box entrance :

$$H - \Delta H_{con} = h + \frac{v^2}{2g}.$$

$$\left.\begin{array}{l} 2.14 - 0.06 \frac{(v)^2}{2g} = h + \frac{v^2}{2g} \\ Q = 30.8 = v \times h \times 9.00 \end{array}\right\} \quad \begin{array}{l} v = 1.73 \text{ m/s} \\ h = 1.98 \text{ m.} \end{array}$$

$$\frac{v^2}{2g} = 0.15 \text{ m}$$

$$\Delta H_{con} = 0.01 \text{ m}$$

$$H = 2.13 \text{ m.}$$

At the box - exit :

$$v = \frac{4.28}{1.56} = 2.74 \text{ m/s}$$

$$\frac{v^2}{2g} = 0.38 \text{ m.}$$

$$H = 1.56 + 0.38 = 1.94 \text{ m.}$$

Frounde number : $F = \frac{v}{\sqrt{g\,h}} = \frac{2.74}{\sqrt{9.8 \times 1.50}} = 0.70.$

Choose flaring of r/s wingwalls : $\frac{1}{3F} = \frac{1}{2.1} \approx \frac{1}{2}.$

At the end of the apron the profile has then widened to :
9.40 + 5.30 = 14.70 (see ANNEX IX-18).

Expension losses :

$$H - \Delta H_{exp} = h + \frac{v^2}{2g}$$

$$\left.\begin{array}{l} (1.94) - 0.60 \frac{(v_2 - v_1)^2}{2g} = h + \frac{v^2}{2g} \\ Q = 14.7 \times (h+0.25) \times v = 30.8 \end{array}\right\} \quad \begin{array}{l} v = 1.03 \text{ m/s} \\ \frac{v^2}{2g} = 0.05 \text{ m} \end{array}$$

$$\Delta H_{exp} = 0.25 \text{ m}$$

$$H = 1.69 \text{ m}$$

$$h = 1.64 \text{ m.}$$

$$= (1.64 - 1.35) = +0.29 \text{ m}$$

The actual waterdepth downstream of the sluice is + 0.21 m.

For this drop of flow no specific energy dissipators are required.

1) Outfall channel

Q_{max} = 30.8 m^3/s ; v_{max} = 0.8 m/s.

required profile : 38.5 m^2

if side slopes 1:2 are chosen :

$$A = (b + 2h)\ h = 38.5\ m^2$$

choose bottom depth at - 1.60 m.

water level at + 0.21 m.

yields h = 1.81 and b = 17.65 m

Velocity at apron end : v = 1.03 m/s

Velocity in channel : v = 0.8 m/s.

Waterdepth : 1.81 m.

channel width : 17.65 m

Provide bottom protection with mattress over length of 18 m from r/s apron with ballast material :

v = 1.03 m/s ⟶ diameter = 90 - 100 mm (Δ= 0.8)

unit weight : 1 kg

Apply broken (half) bricks and/or furnace burns.

2.4.3 Irrigation design

a) Available water quantity,quality:

	May	June	July	August	Sept.
rainfall (effective) (90% probability) (Table IV-3.5 - Patuakhali)	81	279	330	324	161
riverwater useable	←		yes		

b) Crop water requirements

From the cropping calendre an average k_c - value for the area is to be determined considering the varieties that are grown.

for instance: k_c-value (from Table IV - 3.4)

	Apr	May	Jun	Jul	Aug	Sep	Oct	
T-Aman		1.10	1.10	1.10	1.05	0.95		
					1.10	1.10	1.10	
HYV-B.Aus		1.10	1.10	1.85	0.95			
average k_c.		1.10	1.10	1.85	1.10	1.10	1.10	(2)
reference evapo-transpiration. (Table IV - 3.1)		129	133	121	114	130		(1)
crop water requirement. (1) x (2) x landuse (=100%)		142	146	224	125	140		

d) Nett Irrigation requirement

(= ET_{crp} - effective rainfall + storage)

For land preparation :

	Apr	May	Jun	Jul	Aug	Sep
ET_c		+ 142	+ 146	+ 224	+ 125	+ 140
percolation (1 mm/day)		+ 30	+ 30	+ 30	+ 30	+ 30
eff. rain.		- 81	- 279	- 330	- 324	- 161
		91	- 103	- 76	- 169	9
M =		2.94				0.3
more then 5 units: $\frac{M \times T}{S}$ =		0,46				0,05

T = duration of presaturation period

S = 200 mm, is water to establish saturation + waterlayer.

From Nomogram

Figure IV-3.2 : $\frac{I}{M}$ = 2,8 I = 2.8 x 2.94 = 8,23 mm/day

= 255 mm/month

Conveyance losses : 0,8 :

Water requirement for landpreparation during May is 225/0,8 = 319 mm.

For the other months when rain exceeds the requirement for crops and percolation an amount of 200 mm per month is applied.

The nett irrigation requirement then becomes.

	Apr	May	Jun	Jul	Aug	Sep
ET_c		142	146	224	125	140
percolation		31	30	31	31	31
landpreparation			200	200	200	200
rain		- 81	- 279	- 330	- 324	- 161
$2/3\ ET_c + 2/3 \times$ perc.+l.p-r =		319*)	38	40	-20	153

*) as determined above.

f) The irrigation supply requirement is the $319 \times \frac{1}{0,8} \times \frac{1}{0,8} = 498$ mm/month for the month of May.

h) The design of irrigation channel network can only proceed after the size of the irrigation unit has been established.

Since no information on this is available, the part of the design will be omitted.

2.6 Design of a tidal inlet structure

a) Irrigation supply requirement as established above : 498 mm/month

$= 0,498 \times 10000\ m^3/_{ha/month} = 4980\ m^3/_{ha/month}$.

b) Tide curve for second half of May is given on ANNEX IX - 10.

c) As an example an irrigation unit in the south-east part of the polder will be designed. Irrigation supply level is chosen at + 0.10 m above the highest groundlevel along the embankment.

From the map this is know to be + 1.62 + 0.10 = + 1.72 m.

For various intake levels, the volumes that can be taken in during one month are now calculated according to the procedure given in paragraph V - 4.1.2.

embankment : crest width 12' = 3.65 m

height + 17' = + 5.20 m

r/s - slope 1:4

c/s - slope 1:2

groundlevel : + 5.3' = + 1.62 m.

length of pipe through embankment at g.l. :

4 x (5.2 - 1.62) + 3.65 + 2 x (5.2 - 1.62) = 25.13 m.

Consider flow condition (1) : both in and outlet submerged:

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

For a 12" asbestos cement pipe of 25.13 m length C_1 is calculated according to Table V - 3.1.

$$C_1 = \sqrt{\frac{1}{1.5 + \frac{2 \times 25.13 \times (0.013)^2 \times 9,8}{(0.0773)^{4/3}}}} = 0.4983$$

D = 0.3048 m
A = 0.074 m²
R = 0.0773 m

$$Q = 0,4983 \times 0.074 \times \sqrt{2g} \times \sqrt{\Delta H}$$

$$Q = 0.1632 \cdot \sqrt{\Delta H}$$

The tide curve is divided in portions of 1800 sec. starting left and right from the moment of high water and for each portion an average value of the riverside water level is determined.

The water level at c/s is assumed stagnant at the irrigation supply level, which is chosen at a level of + 1.50. + 1.65 and + 1.80 m. For the three levels, the volume of water that can be taken in through the pipe, is caculated :

Irrigation Volumes :

12" pipe		May	
Irr. supply level	ΔH (m)	Q (m^3/s)	Q x ΔT
+ 1.80 m	(2.05 - 1.80)	0.0816	294
	(2.00 - 1.80)	0.0730	263
	(1.89 - 1.80)	0.0490	176
		total per tide	733 m^3
		total per month	43.980 m^3.

+ 1.65	(2.05 - 1.65)	0.1032	372
	(2.00 - 1.65)	0.0966	346
	(1.89 - 1.65)	0.0800	288
	(1.73 - 1.65)	0.0400	149
		total per tide	1155 m^3
		total per month	69.300 m^3.
+ 1.50	(2.05 - 1.50)	0.1210	436
	(2.00 - 1.50)	0.1154	415
	(1.89 - 1.50)	0.1019	367
	(1.73 - 1.50)	0.0783	282
	(1.57 - 1.50)	0.0432	93
		total per tide	1593 m^3
		total per month	95.580 m^3.

Summarized :

design water level	intake m^3 per month	irr. requirement m^3/ha/month	area (ha)
+ 1.50	95.580	4.980	19
+ 1.65	69.300	4.980	14
+ 1.80	43.980	4.980	9

From the table it can be seen how many hectares can be irrigated with one 12" pipe for different water levels.

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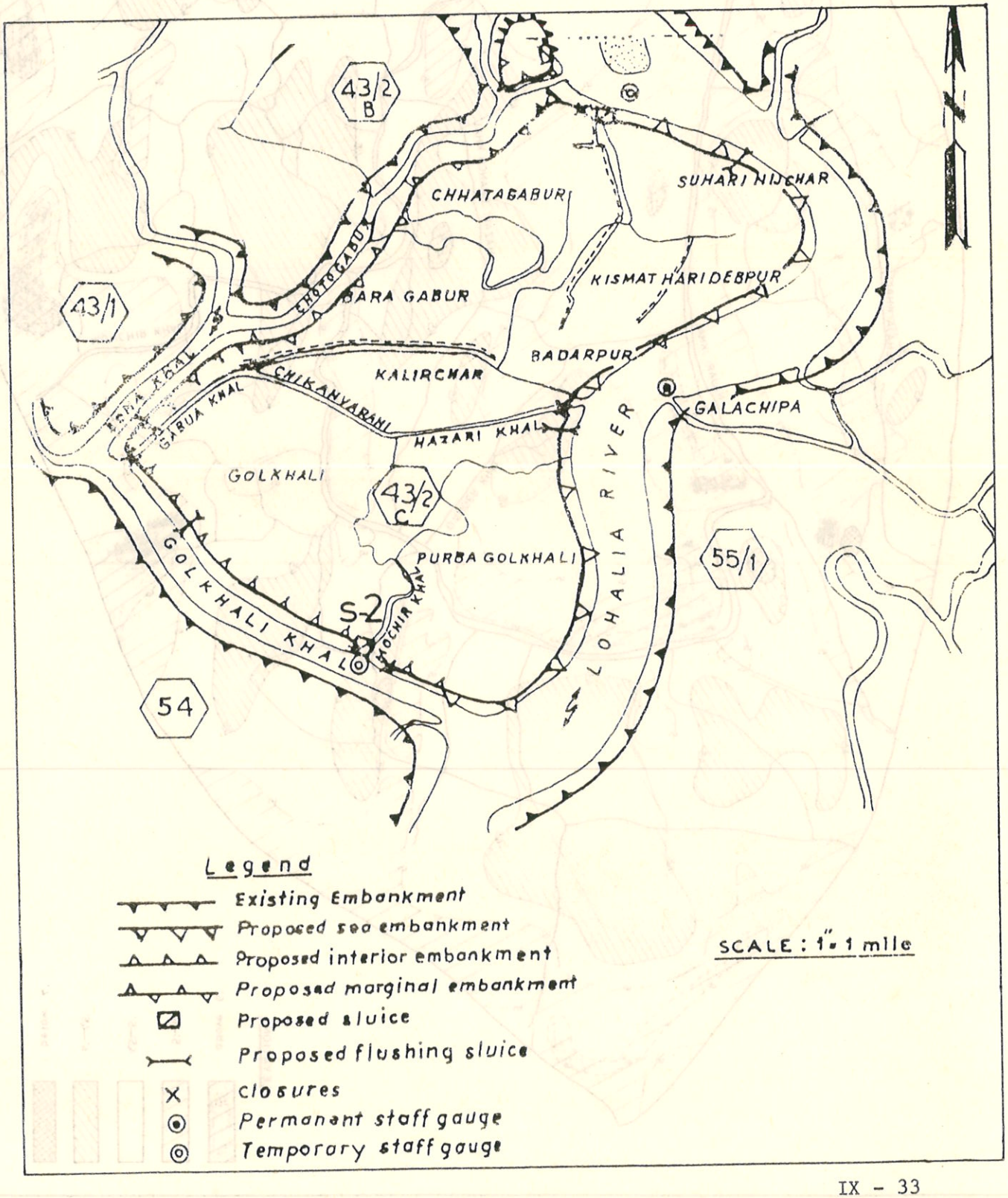
ANNEX IX - 1. AREA MAP



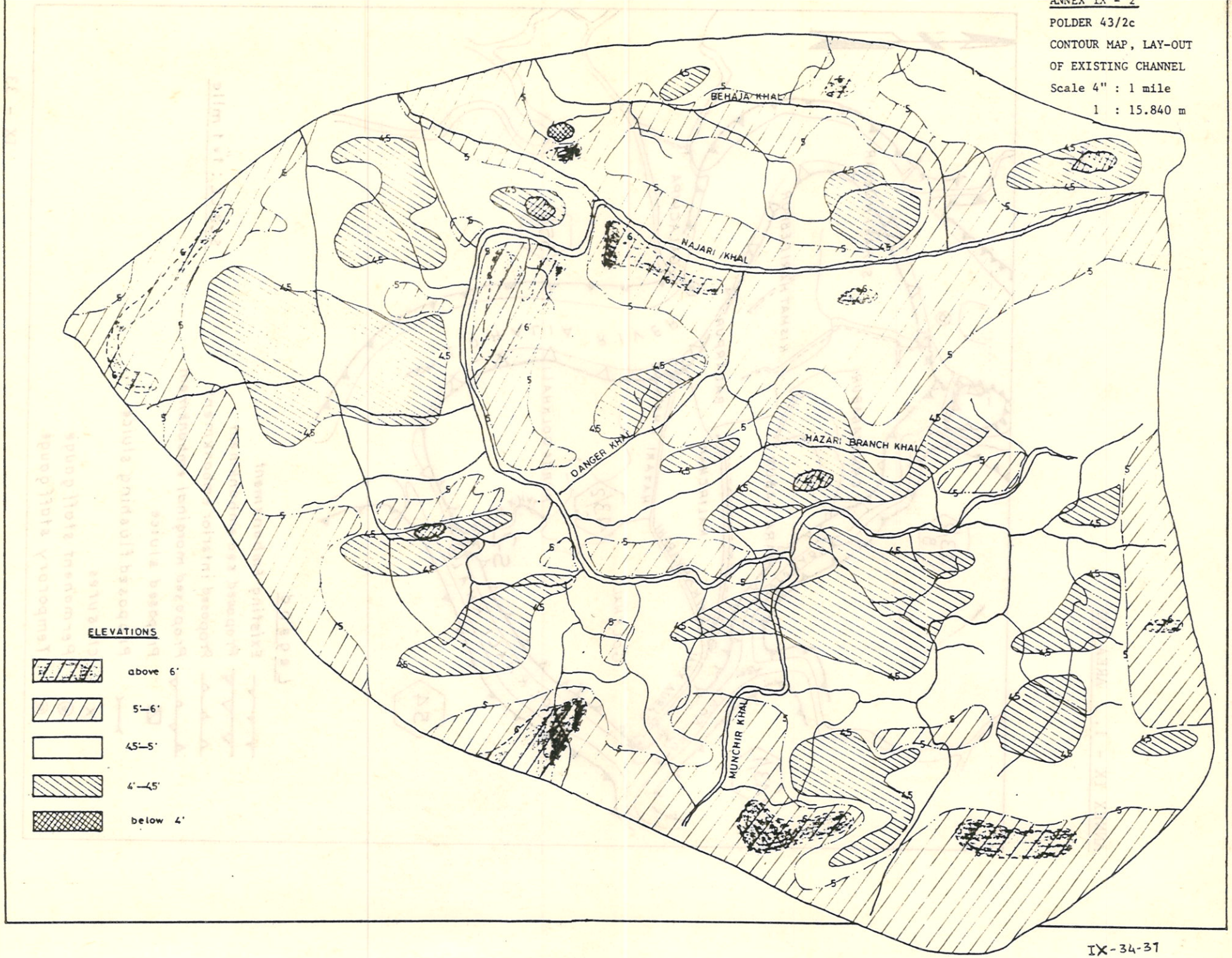
ANNEX IX - 2

POLDER 43/2c

CONTOUR MAP, LAY-OUT OF EXISTING CHANNEL

Scale 4" : 1 mile

1 : 15.840 m



ANNEX IX - 3. DAILY WATER LEVELS, ST. 185 GALACHIPA, 1968 - 1969

ANNUAL MAXIMUM: 2·79 M/~~FT~~

ANNUAL MINIMUM : -0·46 M/~~FT~~

DAILY WATER LEVEL (TIDAL) IN METRE/~~FT~~

RIVER: 70. LOHALIA

STATION: 185, GALACHIPA

SHEET NO. 1

WATER YEAR 1968-69

DATE	APRIL			MAY			JUNE			JULY			AUGUST			SEPTEMBER		
	HWL	LWL	RANGE	HWL	LWL	RANGE	HWL	LWL	RANGE	HWL	LWL	RANGE	HWL	LWL	RANGE	HWL	LWL	RANGE
1	1.98	-0.37	2.35	1.87	-0.27	2.14	2.12	0.08	2.04	2.09	0.40	1.69	1.97	0.53	1.44	1.75	0.59	1.16
2	1.83	-0.43	2.26	1.66	-0.29	1.95	1.97	0.02	1.95	2.06	0.41	1.65	1.81	0.55	1.26	1.77	0.66	1.11
3	1.84	-0.27	2.11	1.65	-0.27	1.92	1.90	0.08	1.82	1.87	0.44	1.43	2.06	0.53	1.53	1.97	0.72	1.25
4	1.74	-0.30	2.04	1.63	-0.20	1.83	1.87	0.08	1.79	2.00	0.43	1.57	2.12	0.69	1.43	2.18	0.53	1.65
5	1.68	-0.27	1.95	1.51	-0.14	1.65	1.84	0.32	1.52	1.97	0.38	1.59	2.16	0.62	1.54	2.19	0.66	1.53
6	1.68	-0.21	1.89	1.51	-0.08	1.59	1.90	0.35	1.55	2.06	0.35	1.71	2.24	0.59	1.65	2.27	0.32	1.95
7	1.49	-0.15	1.64	1.39	-0.02	1.41	2.12	0.23	1.89	2.18	0.29	1.89	2.29	0.50	1.79	2.38	0.29	2.09
8	1.34	-0.12	1.46	1.48	-0.02	1.50	2.27	0.26	2.01	2.21	0.32	1.89	2.30	0.20	2.10	2.39	0.37	2.02
9	1.55	0.00	1.55	1.87	0.02	1.85	2.36	0.29	2.07	2.27	0.23	2.04	2.32	0.23	2.09	2.48	0.69	1.79
10	1.86	0.03	1.83	2.27	0.11	2.16	2.58	0.14	2.44	2.36	0.32	2.04	2.36	0.32	2.04	2.55	0.75	1.80
AV— I	1.70	-0.21	1.91	1.68	-0.12	1.80	2.09	0.18	1.91	2.11	0.36	1.75	2.16	0.48	1.68	2.19	0.56	1.63
11	2.07	-0.12	2.19	2.24	0.17	2.07	2.61	0.23	2.38	2.38	0.29	2.09	2.38	0.35	2.03	2.42	0.78	1.64
12	2.09	-0.08	2.17	2.42	0.08	2.34	2.62	0.32	2.30	2.36	0.23	2.13	2.27	0.41	1.86	2.36	0.79	1.57
13	2.44	-0.03	2.41	2.55	0.12	2.43	2.58	0.35	2.23	2.38	0.38	2.00	2.12	0.29	1.83	2.12	0.81	1.31
14	2.32	-0.06	2.26	2.48	0.14	2.34	2.39	0.41	1.98	2.33	0.29	2.04	2.06	0.34	1.72	2.06	0.67	1.39
15	2.33	-0.32	2.65	2.27	-0.02	2.29	2.29	0.43	1.76	2.27	0.29	1.98	1.97	0.38	1.59	1.97	0.78	1.19
16	2.19	-0.27	2.46	2.12	0.14	1.98	2.21	0.38	1.83	2.24	0.26	1.98	1.81	0.43	1.38	1.87	0.84	1.03
17	2.23	-0.26	2.49	1.75	-0.02	1.73	1.90	0.40	1.50	2.12	0.41	1.72	1.75	0.53	1.22	1.81	0.64	1.17
18	1.92	-0.27	2.19	1.66	0.08	1.58	1.81	0.44	1.37	1.94	0.47	1.47	1.87	0.66	1.21	1.81	0.66	1.15
19	1.57	-0.18	1.75	1.69	0.17	1.42	1.75	0.46	1.29	1.97	0.44	1.53	1.81	0.72	1.09	1.97	0.44	1.53
20	1.45	-0.14	1.31	1.66	0.18	1.48	1.69	0.44	1.25	1.84	0.35	1.49	2.06	0.69	1.37	2.06	0.53	1.53
AV— II	2.06	-0.13	2.19	2.08	0.11	1.97	2.18	0.39	1.79	2.18	0.34	1.84	2.01	0.48	1.53	2.04	0.69	1.35
21	1.23	0.17	1.06	1.63	0.23	1.40	1.94	0.41	1.53	1.81	0.41	1.40	2.30	0.72	1.58	2.18	0.56	1.62
22	1.23	0.11	1.12	1.66	0.23	1.43	1.81	0.32	1.49	2.06	0.43	1.63	2.51	0.69	1.82	2.24	0.23	2.01
23	1.45	0.18	1.27	1.94	0.26	1.68	1.90	0.29	1.61	2.00	0.41	1.59	2.55	0.59	1.96	2.27	0.20	2.07
24	1.81	0.15	1.66	1.90	0.23	1.67	2.21	0.23	1.98	2.06	0.44	1.62	2.58	0.53	2.05	2.51	0.21	2.30
25	1.81	-0.17	1.98	2.00	0.02	1.98	2.36	0.32	2.04	2.00	0.29	1.71	2.51	0.56	1.95	2.47	0.26	2.21
26	1.75	-0.26	2.01	2.01	0.08	1.93	2.36	0.29	2.07	2.36	0.26	2.10	2.27	0.24	2.03	2.39	0.27	2.12
27	1.97	-0.29	2.26	2.06	0.11	1.95	2.35	0.41	1.94	2.64	0.32	2.32	2.24	0.24	2.00	2.12	0.38	1.74
28	2.12	-0.32	2.44	2.03	0.15	1.88	2.27	0.43	1.84	2.79	0.69	2.10	2.09	0.29	1.80	1.97	0.44	1.53
29	2.06	-0.20	2.26	2.24	0.28	1.95	2.09	0.44	1.65	2.42	0.84	1.58	1.97	0.53	1.44	1.87	0.49	1.38
30	2.00	-0.02	2.02	2.21	0.02	2.19	2.00	0.35	1.65	2.21	0.85	1.36	2.03	0.56	1.47	1.81	0.53	1.28
31	—	—	—	2.18	0.02	2.16	—	—	—	2.12	0.72	1.40	2.00	0.50	1.50	—	—	—
AV— III	1.74	-0.06	1.80	1.99	0.15	1.84	2.13	0.35	1.78	2.22	0.51	1.71	2.28	0.50	1.78	2.18	0.36	1.82
MAX.	2.44	0.18	2.65	2.55	0.29	2.43	2.62	0.46	2.44	2.79	0.85	2.32	2.58	0.72	2.10	2.55	0.84	2.30
MEAN	1.83	-0.13	1.96	1.92	0.05	1.87	2.14	0.31	1.83	2.17	0.41	1.76	2.15	0.48	1.67	2.14	0.54	1.60
MIN.	1.23	-0.43	1.06	1.39	-0.29	1.40	1.69	0.02	1.25	1.81	0.23	1.36	1.75	0.20	1.09	1.75	0.20	1.03

ANNEX IX - 4 FORTNIGHTLY MEAN WATER LEVELS

Year 1968-69	No.of H.W.	ΣH.W	MHW	No.of L.W	ΣL.W	MLW	ΣHW + ΣLW	No. of peaks	FML
Fortnightly period									
1. May 2-16	15	29,05	1.93	15	-0.26	-0.017	28.79	30	0.96
2. May 17-31	15	28.62	1.908	15	2.09	0.139	30.71	30	1.02
3. June 1-15	15	33.42	2.228	15	3.59	0.239	37.01	30	1.23
4. " 16-30	15	30.65	2.043	15	5.61	0.374	36.26	30	1.21
5. July 1-15	15	32.79	2.186	15	5.05	0.337	37.84	30	1.26
6. " 16-30	15	32.46	2.164	15	6.87	0.458	39.33	30	1.31
7. July 31 August-14	15	34.82	2.32	15	6.87	0.458	41.69	30	1.39
8. August 15-29	15	32.29	2.15	15	7.80	0.52	40.09	30	1.33

ANNEX IX - 5 FORTNIGHTLY MEANS - SUMMARIZED (water levels in meter)

Fortnight period	1 May 2-16	2 May 17-31	3 June 1-15	4 June 16-30	5 July 1-15	6 July 16-30	7 July 31-Aug. 14	8 Aug. 15-29
1968-69	0.96	1.02	1.23	1.21	1.26	1.31	1.39	1.33
1969-70	0.83	1.08	1.06	0.95	1.28	1.39	1.24	1.18
1970-71	0.98	1.09	1.25	1.27	1.40	1.37	1.33	1.48
1971-72	0.88	1.05	1.21	1.18	1.22	1.32	1.26	1.21
1972-73	0.98	1.14	0.96	1.09	1.25	1.15	1.30	1.31
1973-74	1.02	1.11	1.12	1.28	1.37	1.34	1.33	1.48
1974-75	0.96	1.09	1.21	1.29	1.24	1.21	1.38	1.34
1975-76	0.88	0.99	1.07	1.31	1.28	1.24	1.39	1.44
1976-77	0.99	1.14	1.28	1.31	1.45	1.46	1.49	1.45
1977-78	1.18	1.18	1.35	1.39	1.45	1.61	1.64	1.72
1978-79	1.34	1.71	1.57	1.66	1.67	1.72	1.82	1.92
1979-80	1.21	1.34	1.26	1.53	1.56	1.58	1.70	1.56
F.M.L. 50%	0.98	1.09	1.21	1.29	1.28	1.34	1.39	1.44

ANNEX IX - 6 FORTNIGHTLY MEAN HIGH AND LOW WATERS

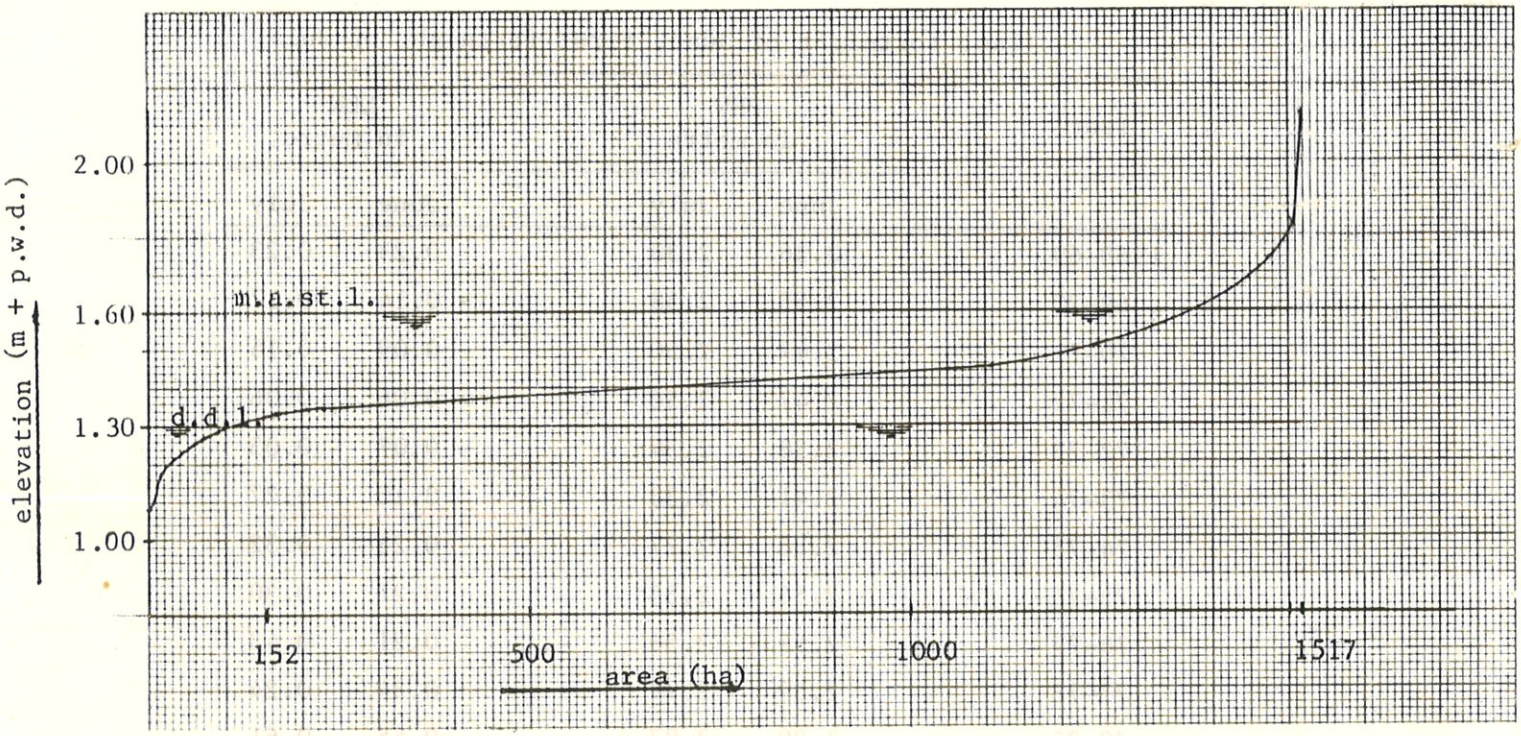
Fortnightly period	MEAN HIGH WATER (MHW) in meter								MEAN LOW WATER (MLW) in meter							
	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8
1968-69	1.84	1.91	2.23	2.04	2.19	2.16	2.32	2.15	0.02	0.14	0.24	0.37	0.34	0.46	0.46	0.52
69-70	2.02	2.34	2.45	2.32	2.21	2.33	2.11	2.12	-.37	-.18	-.32	-.41	.36	.45	.36	.24
70-71	2.02	2.06	2.12	2.26	2.25	2.21	2.03	2.26	-.06	.12	.37	.29	.55	.52	.63	.69
71-72	1.76	2.06	2.36	2.12	2.21	2.28	2.22	2.01	.01	.05	.07	.23	.23	.36	.31	.40
72-73	1.64	2.04	1.98	2.06	2.23	2.07	2.29	2.20	.31	.25	-.06	.13	.26	.23	.31	.42
73-74	2.18	2.07	2.09	2.20	2.23	2.18	2.09	2.30	-.14	.15	.16	.36	.50	.51	.58	.66
74-75	1.94	2.04	2.11	2.15	2.03	2.05	2.21	2.09	-.03	.14	.32	.42	.45	.37	.55	.60
75-76	1.90	2.12	1.96	2.27	2.16	2.07	2.25	2.24	-.13	-.14	.19	.34	.39	.42	.52	.65
76-77	2.12	2.10	2.30	2.20	2.39	2.43	2.50	2.40	-.12	.17	.26	.42	.51	.49	.49	.51
77-78	2.30	2.21	2.38	2.34	2.45	2.63	2.51	2.66	.06	.15	.33	.44	.46	.60	.77	.77
78-79	2.39	2.86	2.67	2.70	2.64	2.73	2.83	2.96	.29	.56	.46	.61	.69	.72	.82	.89
79-80	2.33	2.49	2.40	2.72	2.73	2.64	2.80	2.60	.08	.20	.13	.23	.38	.52	.61	.52
MHW-50%	2.02	2.07	2.23	2.20	2.21	2.21	2.25	2.24	-.03	.14	.19	.34	.39	.46	.52	.52

ANNEX IX - 7 HHW, LLW, SUMMARIZED FOR CRITICAL PERIOD

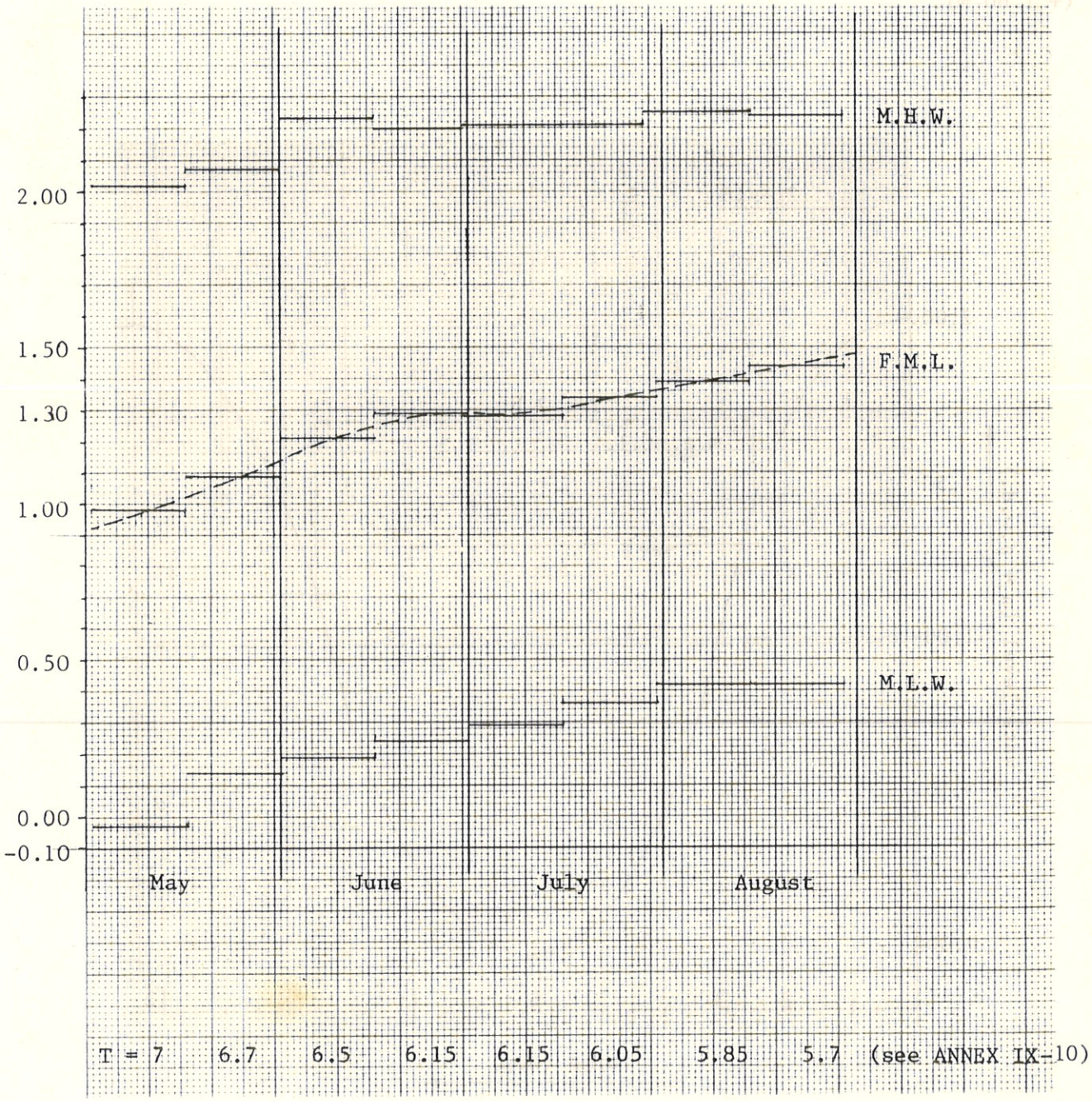
(last fortnight of June, first fortnight of July)

Year	HHW in metre		LLW in metre	
	June	July	June	July
1968-69	2.36	2.38	0.23	0.23
69-70	2.55	2.58	- 0.14	0.17
70-71	2.47	2.50	- 0.09	0.15
71-72	2.47	2.56	0.06	0.06
72-73	2.38	2.38	- 0.07	0.03
73-74	2.41	2.38	0.18	0.27
74-75	2.36	2.27	0.23	0.32
75-76	2.56	2.35	0.21	0.30
76-77	2.59	2.56	0.21	0.43
77-78	2.59	2.99	0.34	0.24
78-79	3.29	2.87	0.55	0.61
79-80	3.08	3.08	0.12	0.24
50%	+ 2.47	2.38	+ 0.21	0.23

ANNEX IX - 8 AREA - ELEVATION CURVE

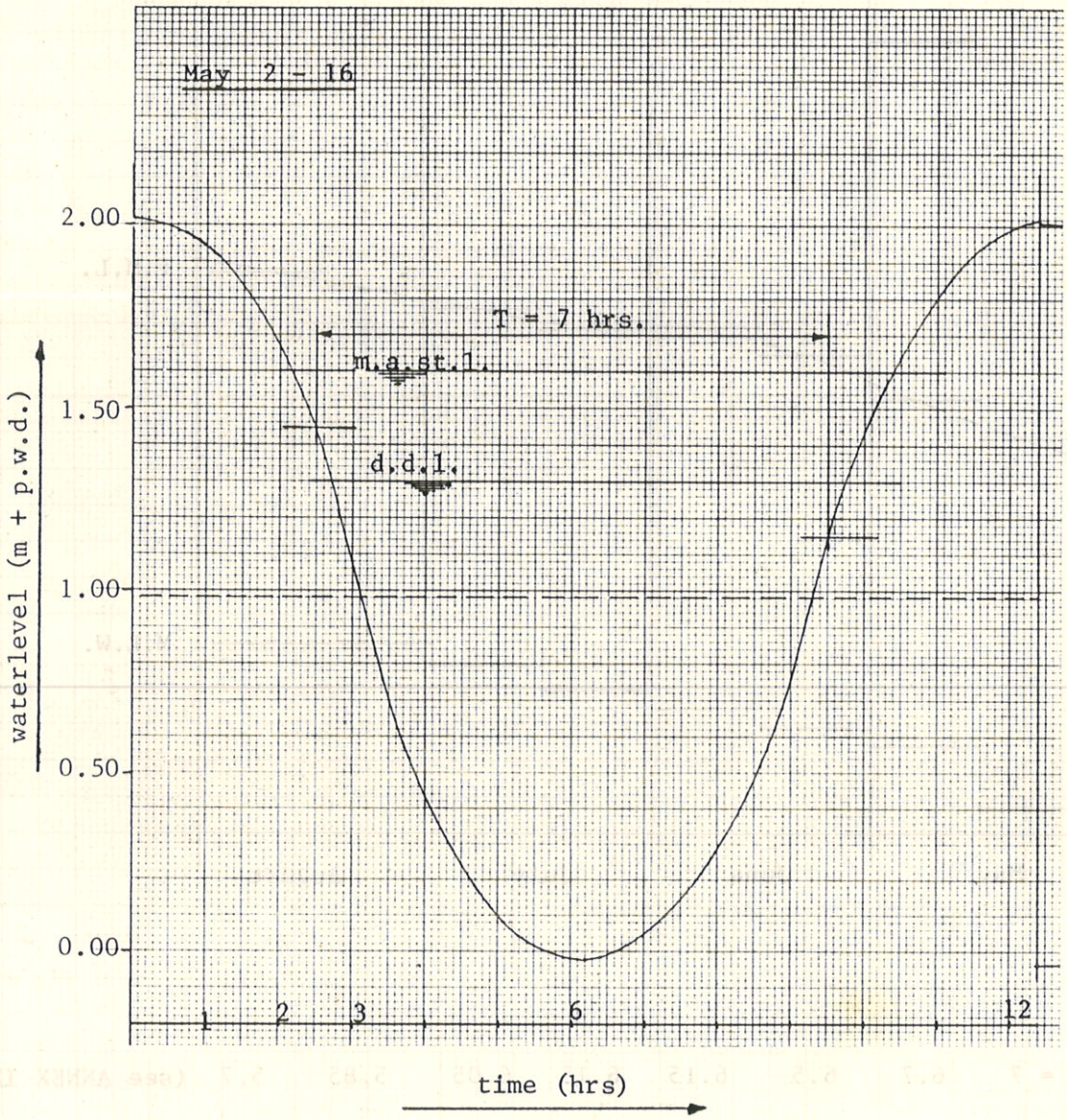


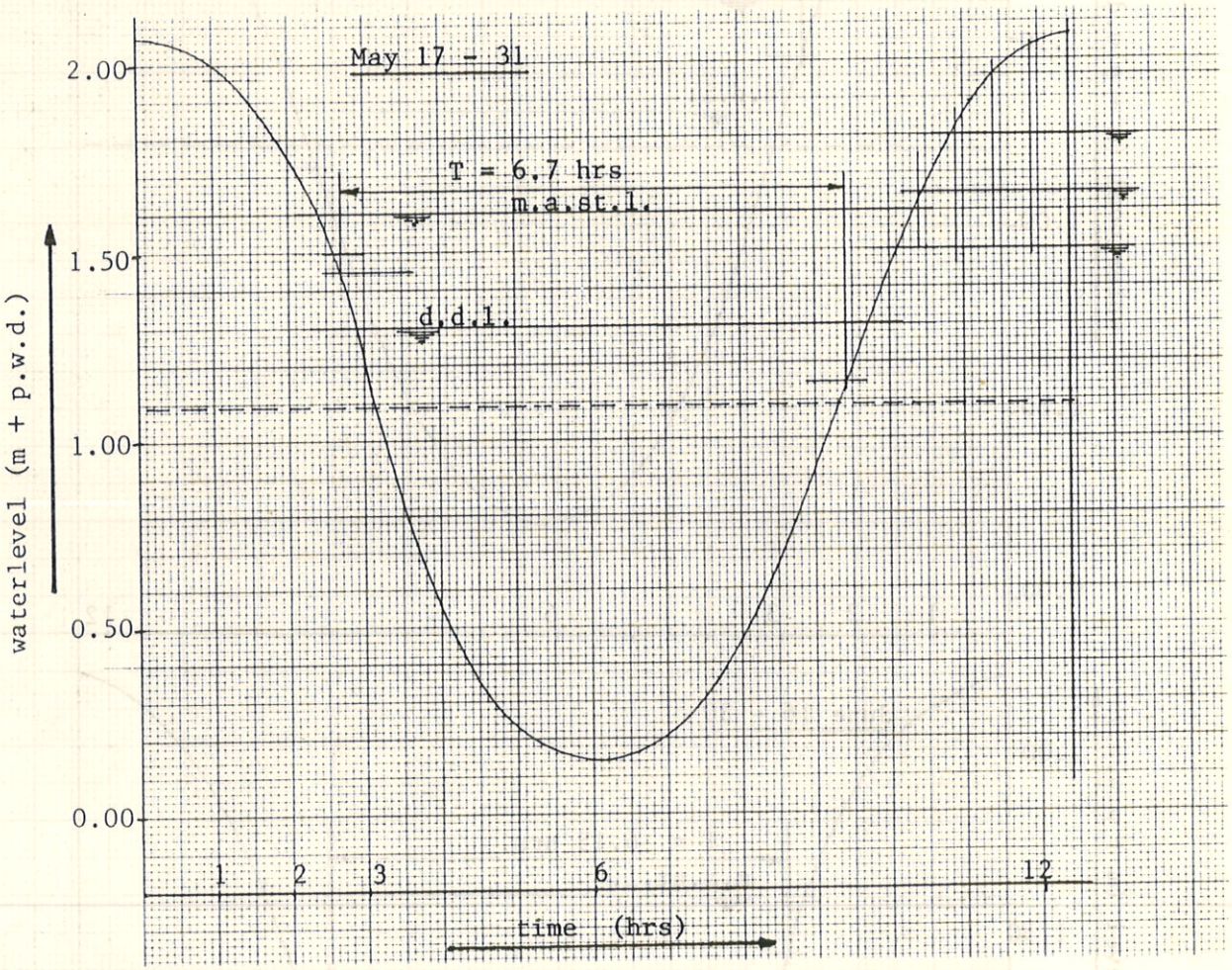
ANNEX IX - 9 AVERAGE TIDES DURING MAY - AUGUST

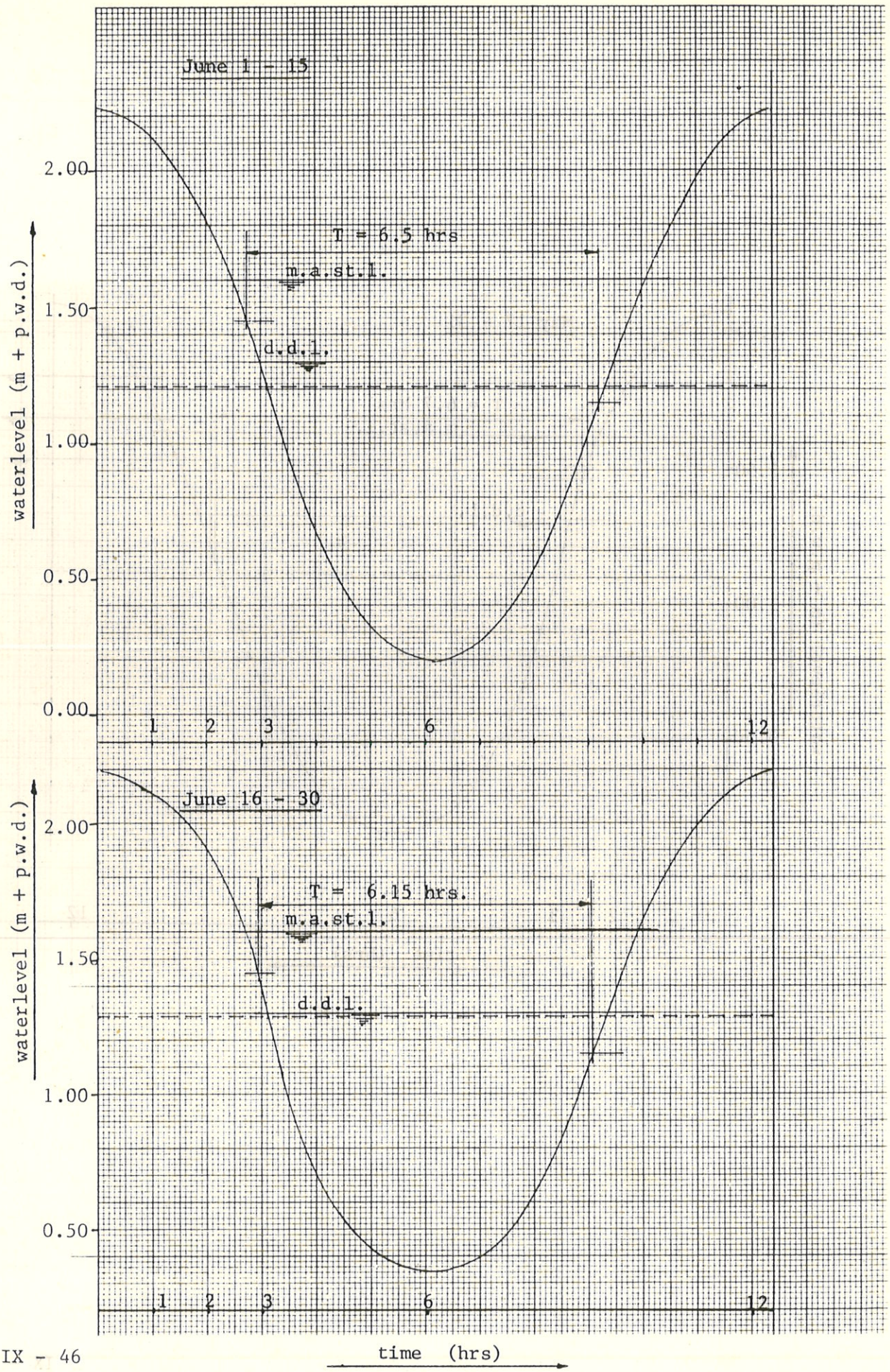


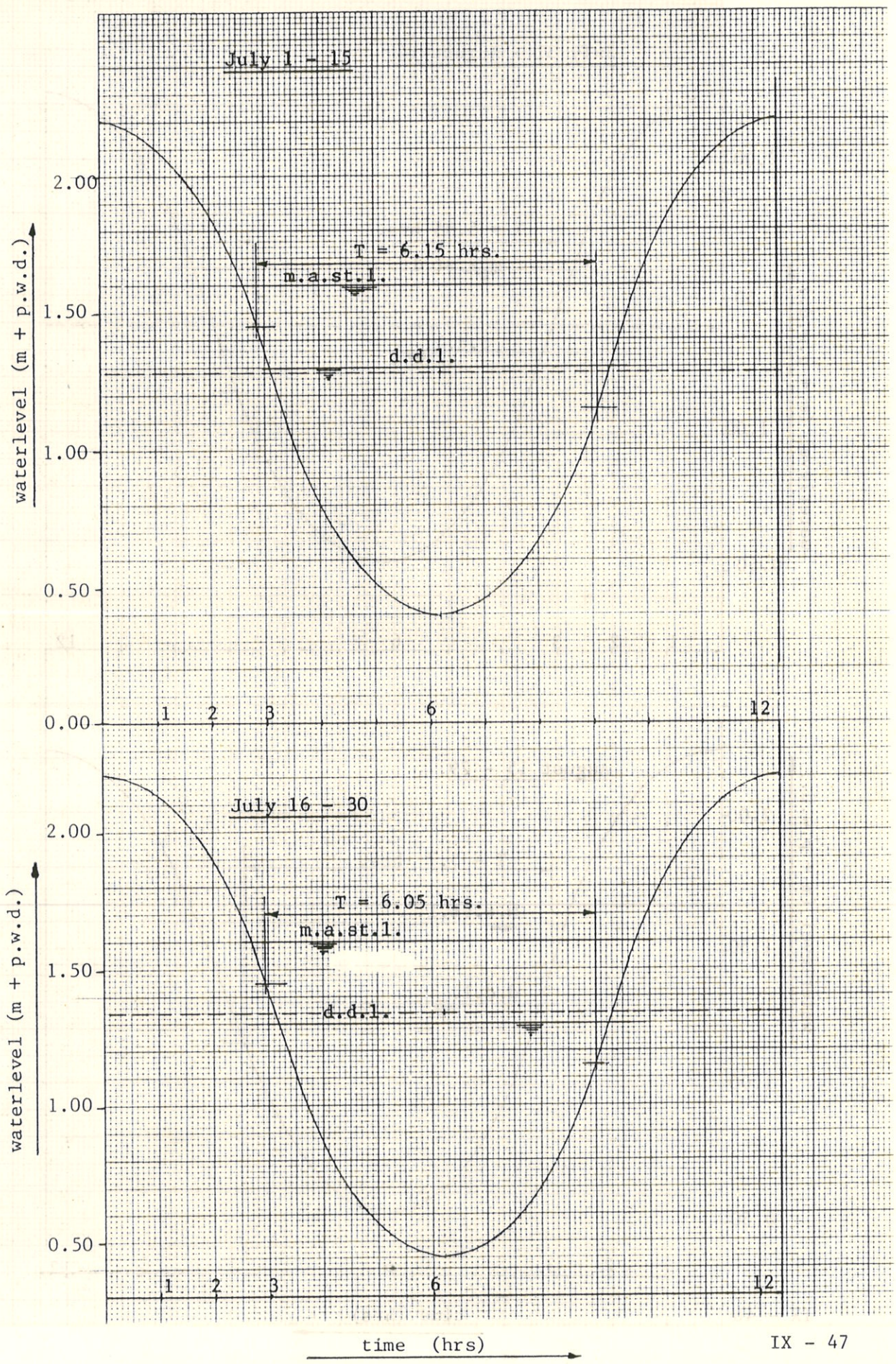
ANNEX IX - 10 TIDE CURVES

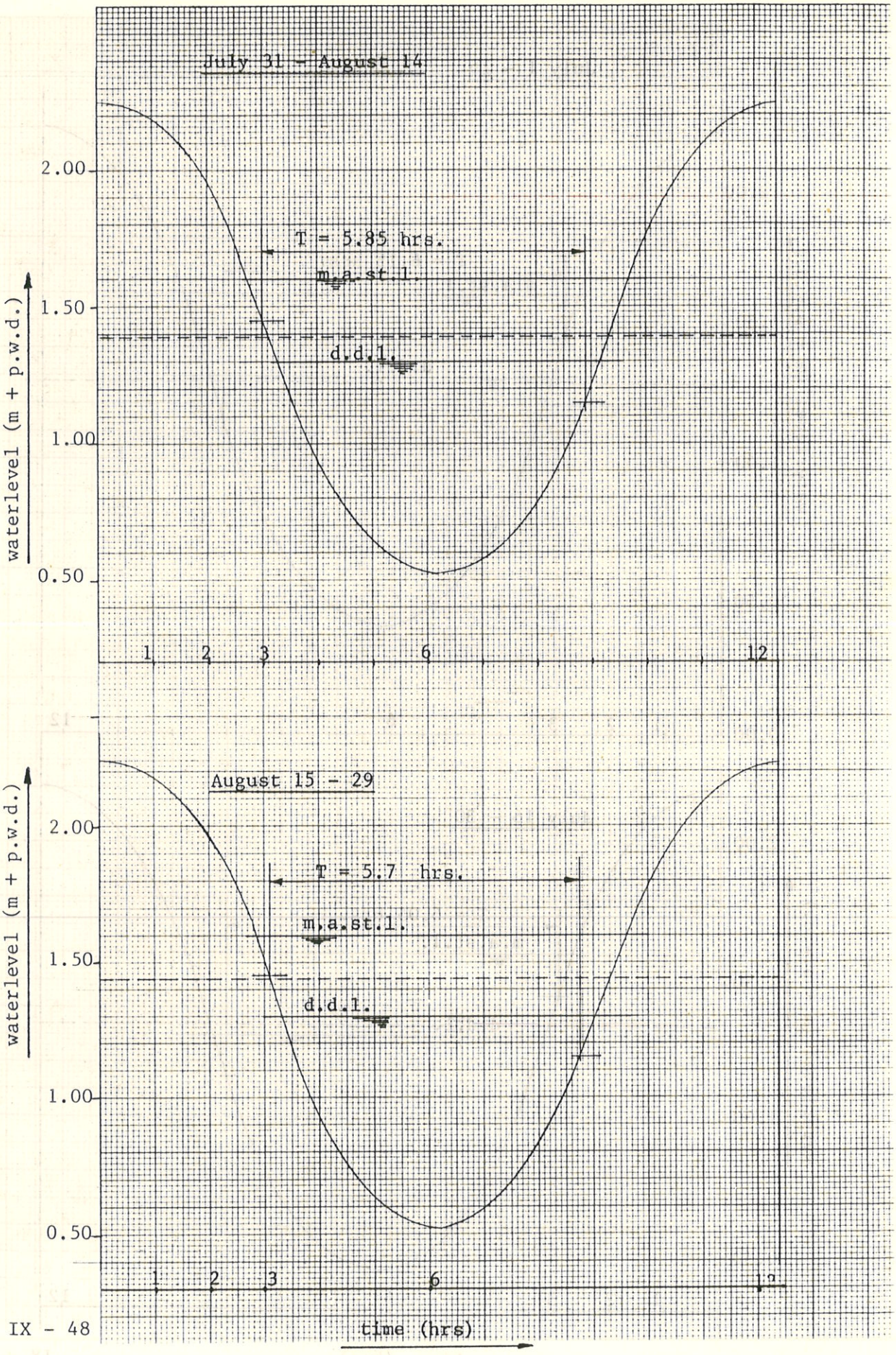
For the fortnightly periods during May - August and the determination of the drainage period T for the assumed d.d.l. (+ 1.30 m) and m.a.st.l. (+ 1.60 m).



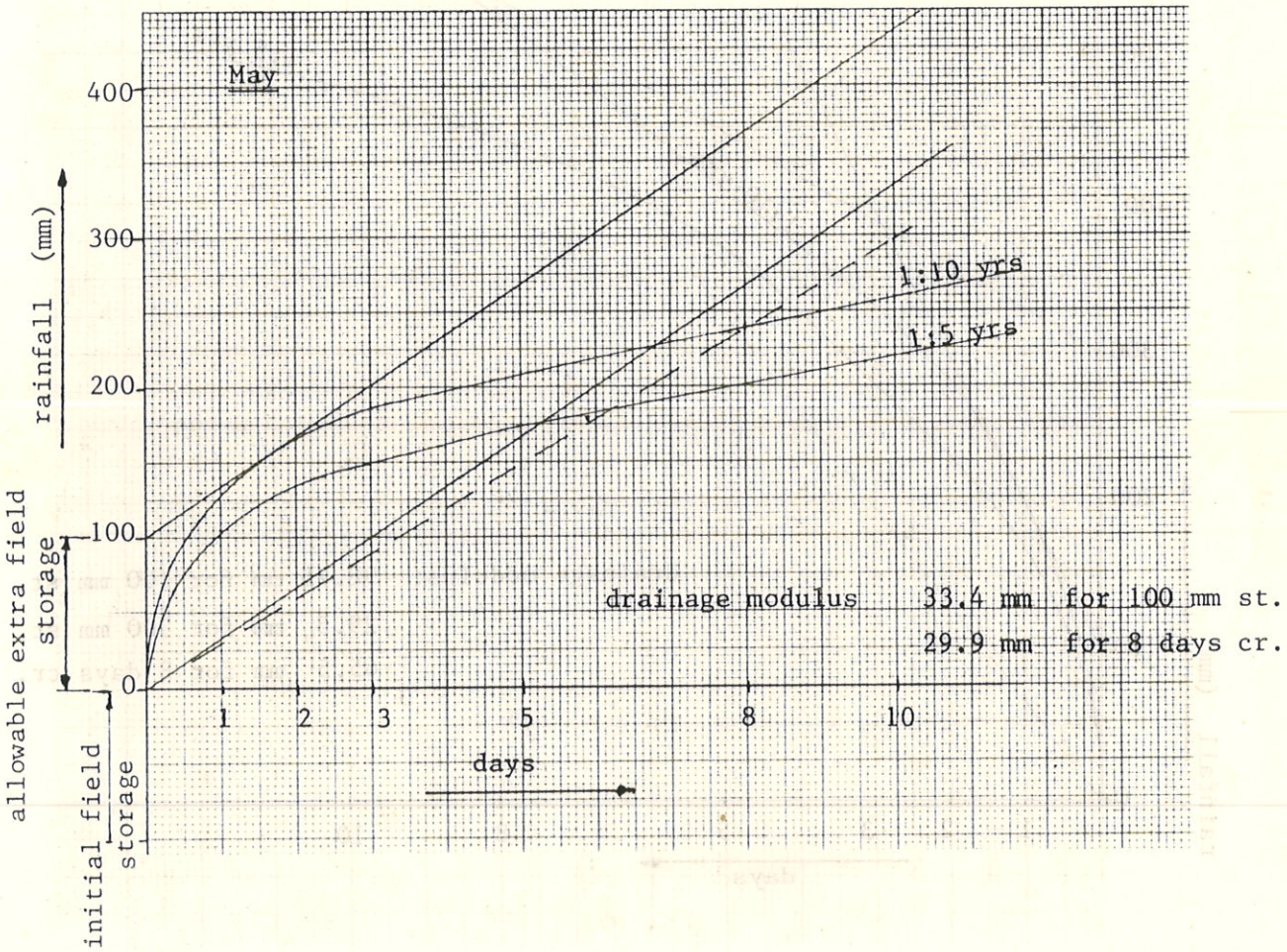

May 17 - 31
T = 6.7 hrs
m.a.st.l.
d.d.l.
2.00
1.50
1.00
0.50
0.00
waterlevel (m + p.w.d.)
1
2
3
6
12
time (hrs)

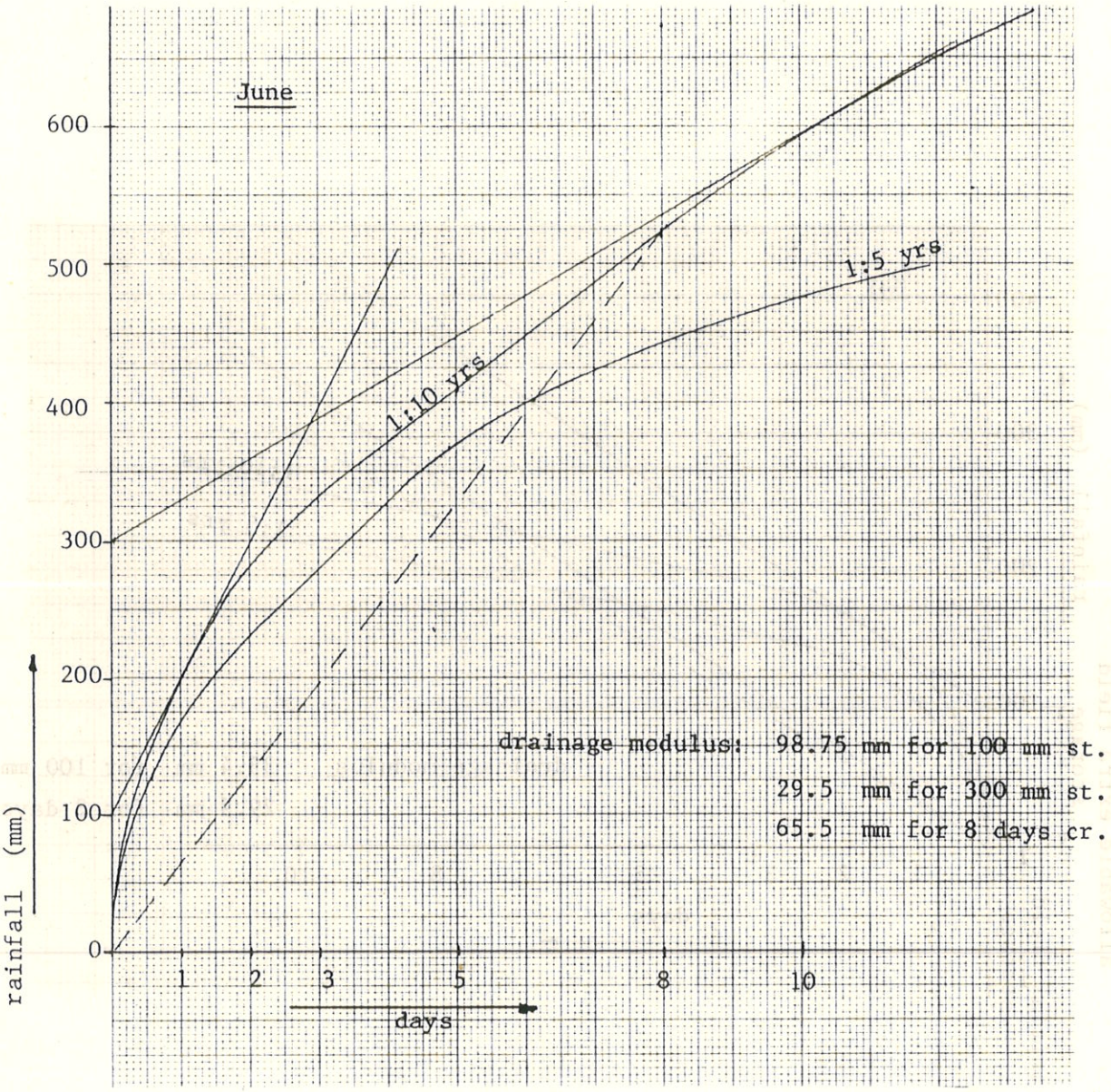

June 1 - 15
T = 6.5 hrs
m.a.st.l.
d.d.l.
2.00
1.50
1.00
0.50
0.00
waterlevel (m + p.w.d.)
1
2
3
6
12
June 16 - 30
T = 6.15 hrs.
m.a.st.l.
d.d.l.
2.00
1.50
1.00
0.50
waterlevel (m + p.w.d.)
1
2
3
6
12
time (hrs)

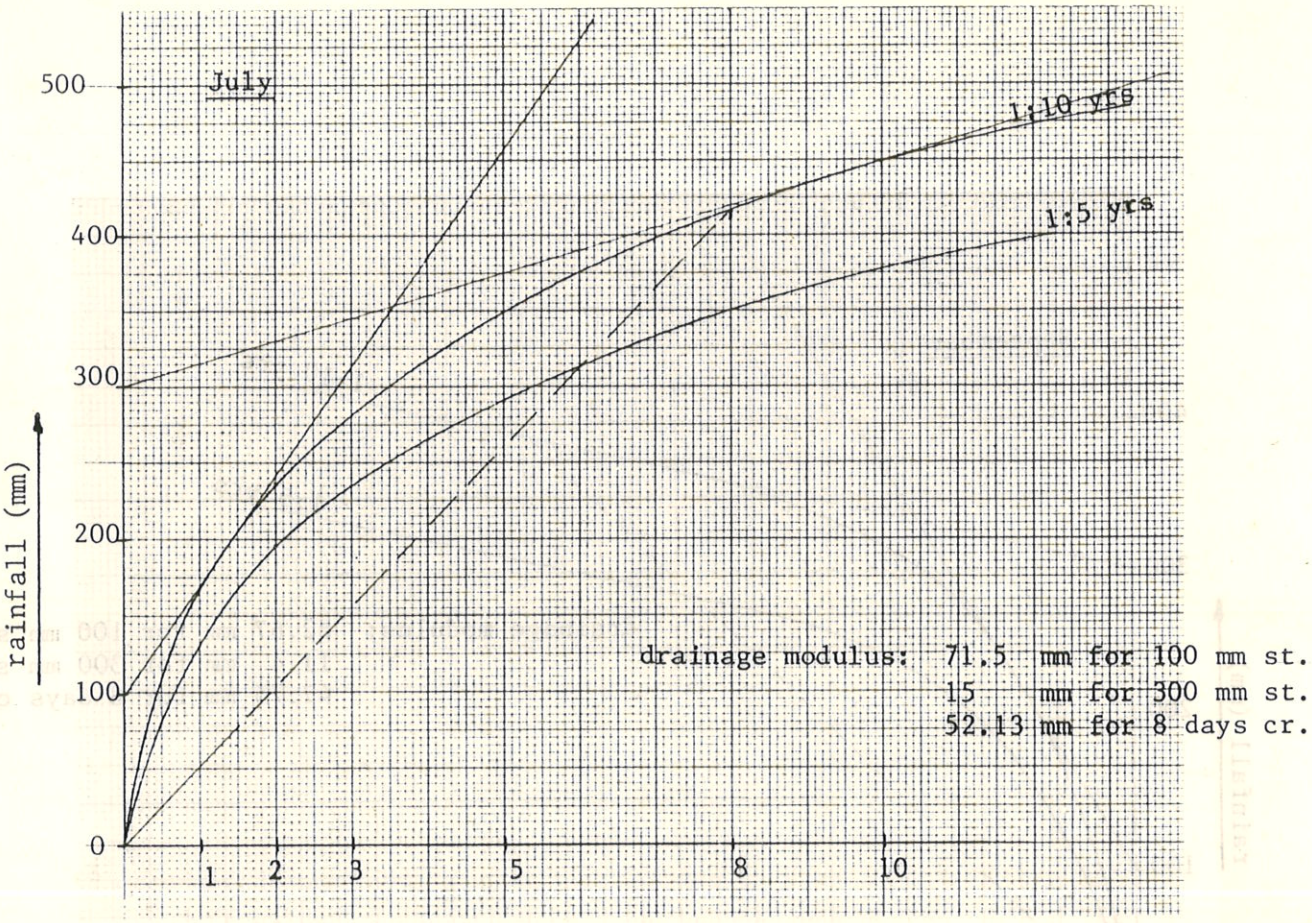

July 1 - 15
2.00
1.50
1.00
0.50
0.00
waterlevel (m + p.w.d.)
T = 6.15 hrs.
m.a.st.l.
d.d.l.
1
2
3
6
12
July 16 - 30
2.00
1.50
1.00
0.50
waterlevel (m + p.w.d.)
T = 6.05 hrs.
m.a.st.l.
d.d.l.
1
2
3
6
12
time (hrs)

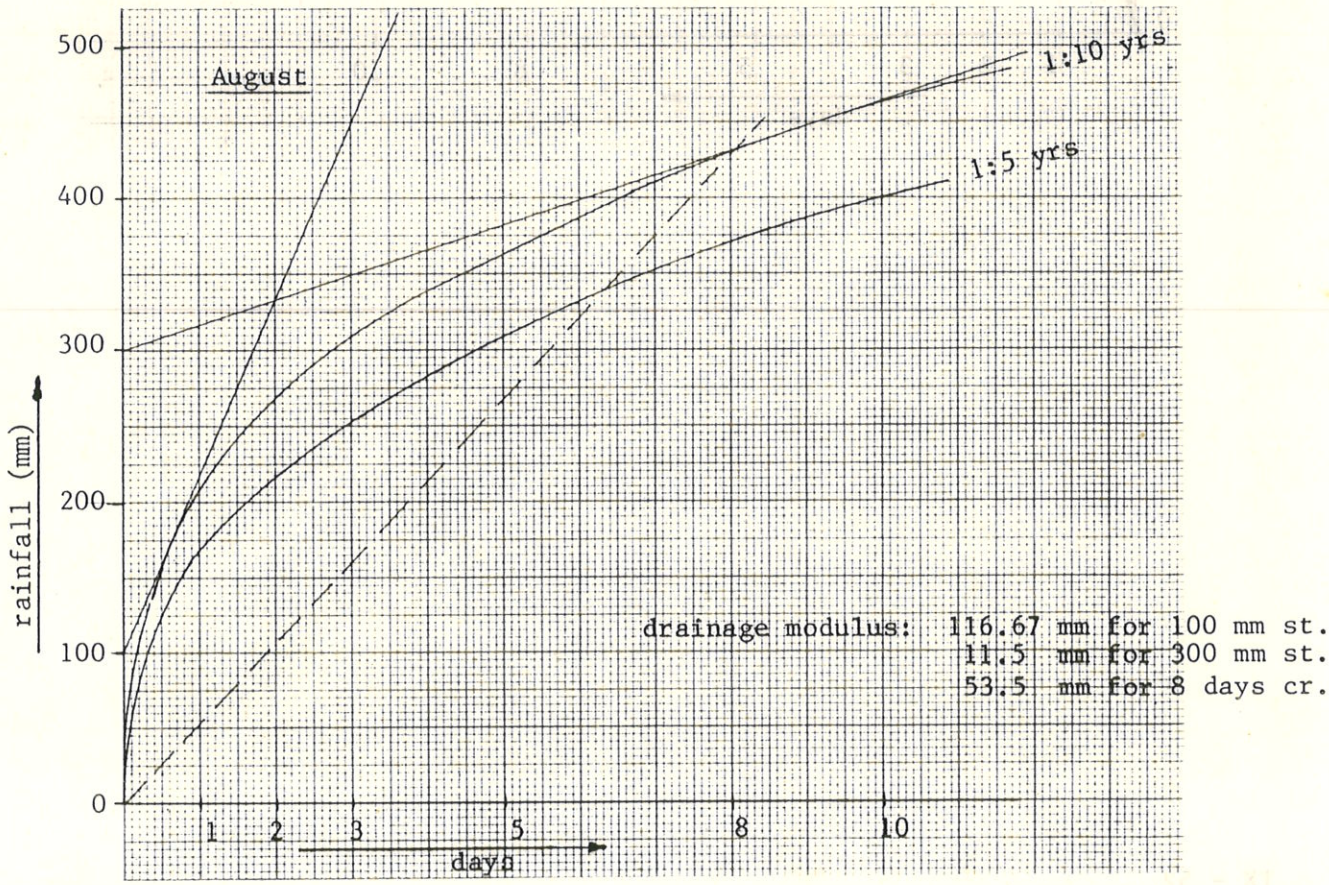

July 31 - August 14
2.00
1.50
1.00
0.50
waterlevel (m + p.w.d.)
T = 5.85 hrs.
m.a.st.l.
d.d.l.
1
2
3
6
12
August 15 - 29
2.00
1.50
1.00
0.50
waterlevel (m + p.w.d.)
T = 5.7 hrs.
m.a.st.l.
d.d.l.
1
2
3
6
12
time (hrs)

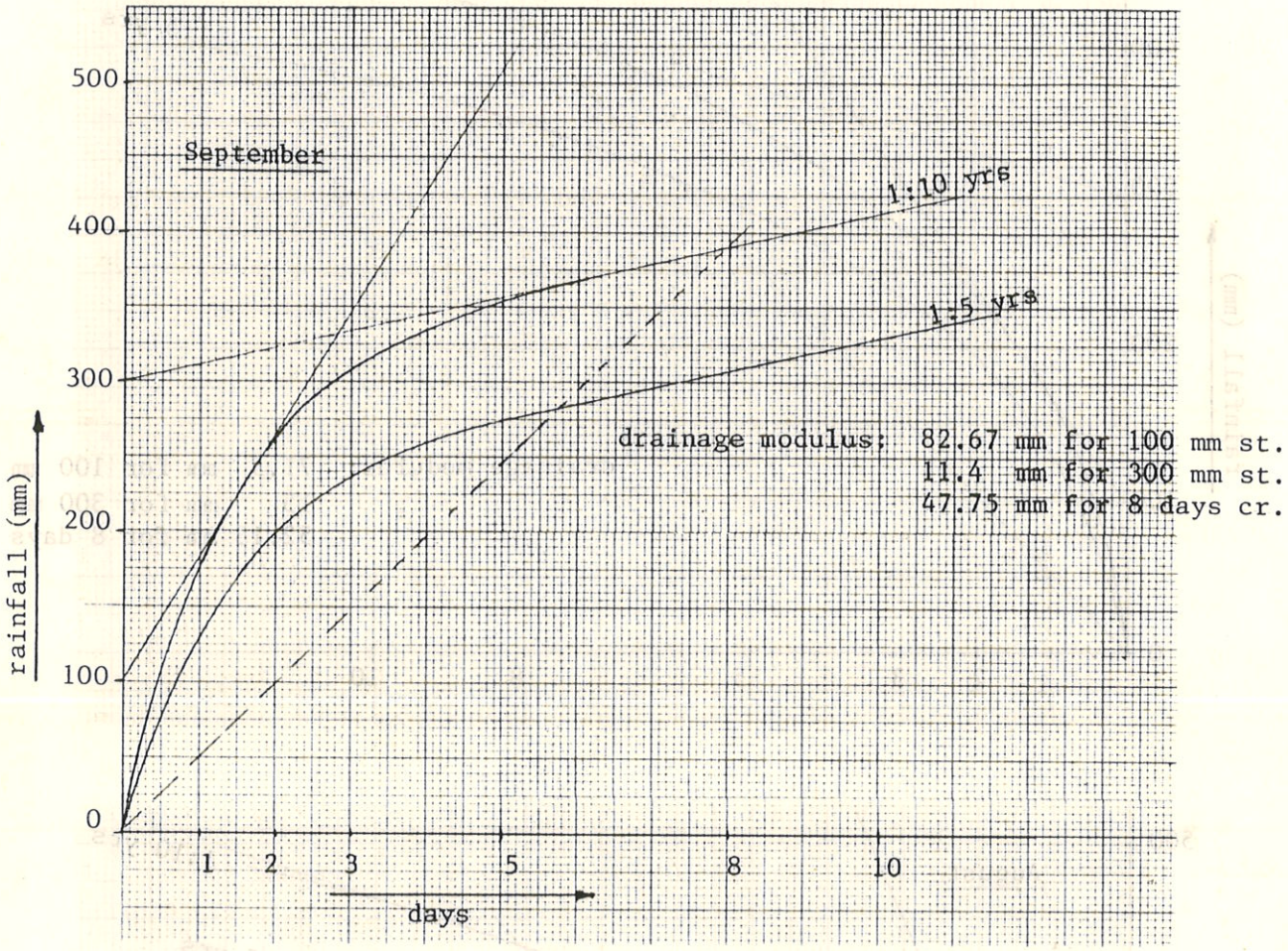
ANNEX IX - 11 RAINFALL DURATION CURVES DETERMINATION OF DRAINAGE MODULUS

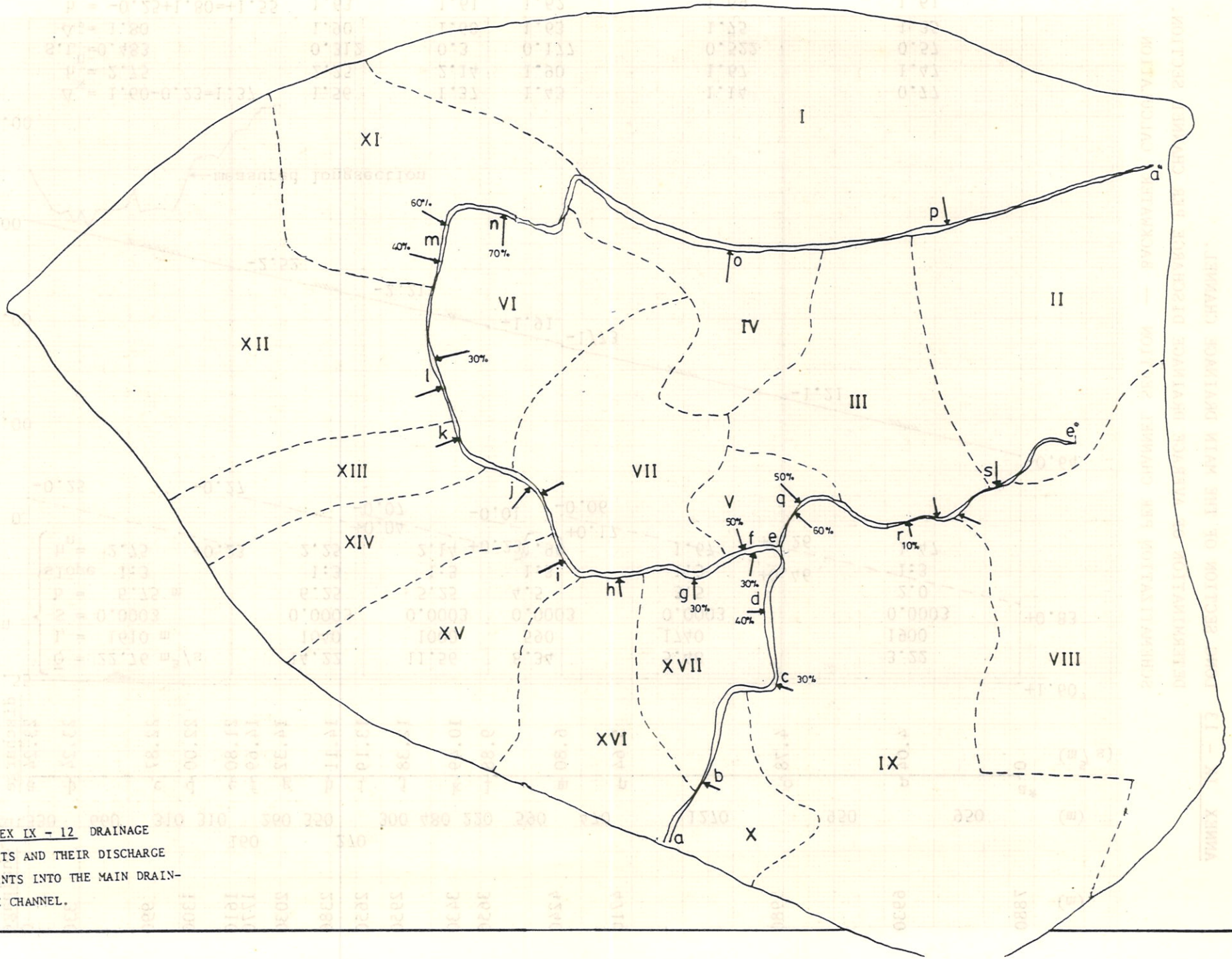



June
600
500
400
300
200
100
0
rainfall (mm)
1:10 yrs
1:5 yrs
drainage modulus: 98.75 mm for 100 mm st.
29.5 mm for 300 mm st.
65.5 mm for 8 days cr.
1
2
3
5
8
10
days


July
1:10 yrs
1:5 yrs
rainfall (mm)
500
400
300
200
100
0
1
2
3
5
8
10
drainage modulus: 71.5 mm for 100 mm st.
15 mm for 300 mm st.
52.13 mm for 8 days cr.


August
1:10 yrs
1:5 yrs
rainfall (mm)
500
400
300
200
100
0
1
2
3
5
8
10
days
drainage modulus: 116.67 mm for 100 mm st.
11.5 mm for 300 mm st.
53.5 mm for 8 days cr.


September
1:10 yrs
1:5 yrs
drainage modulus: 82.67 mm for 100 mm st.
11.4 mm for 300 mm st.
47.75 mm for 8 days cr.
rainfall (mm)
500
400
300
200
100
0
1
2
3
5
8
10
days

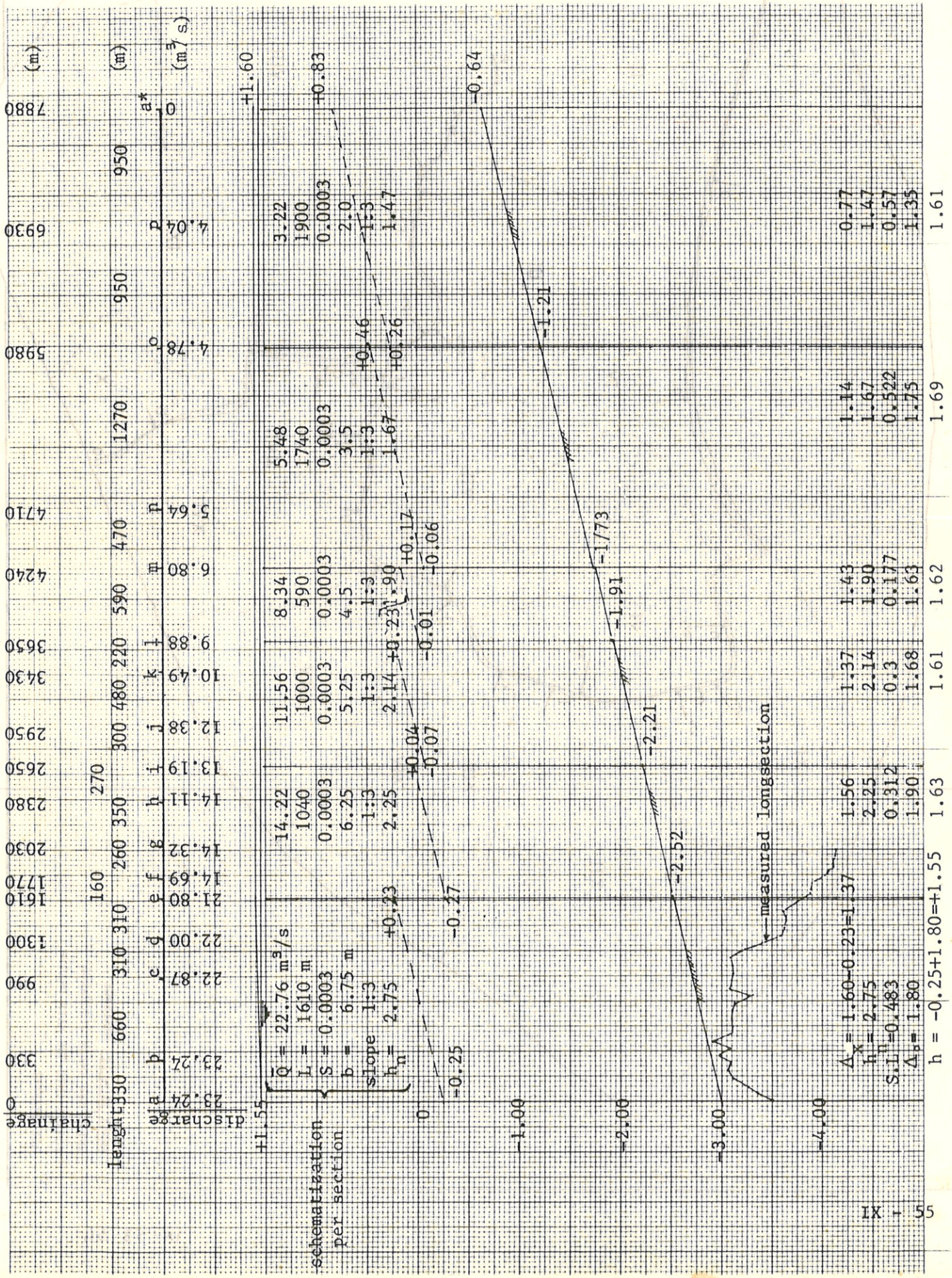


ANNEX IX - 12 DRAINAGE UNITS AND THEIR DISCHARGE POINTS INTO THE MAIN DRAINAGE CHANNEL.

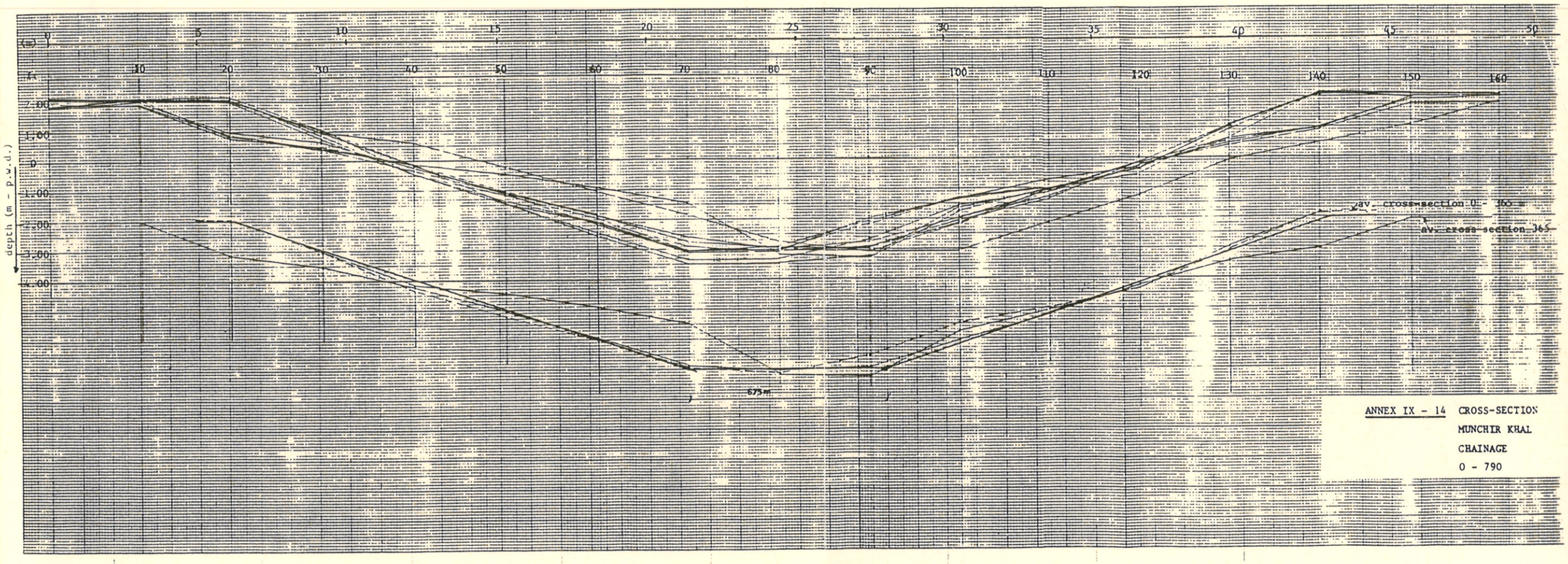
ANNEX IX - 13 LONG SECTION OF THE MAIN DRAINAGE CHANNEL.

DETERMINATION OF AVERAGE DRAINAGE DISCHARGE PER CHANNEL SECTION.

SCHEMATIZATION PER CHANNEL SECTION — BACKWATER CALCULATION.

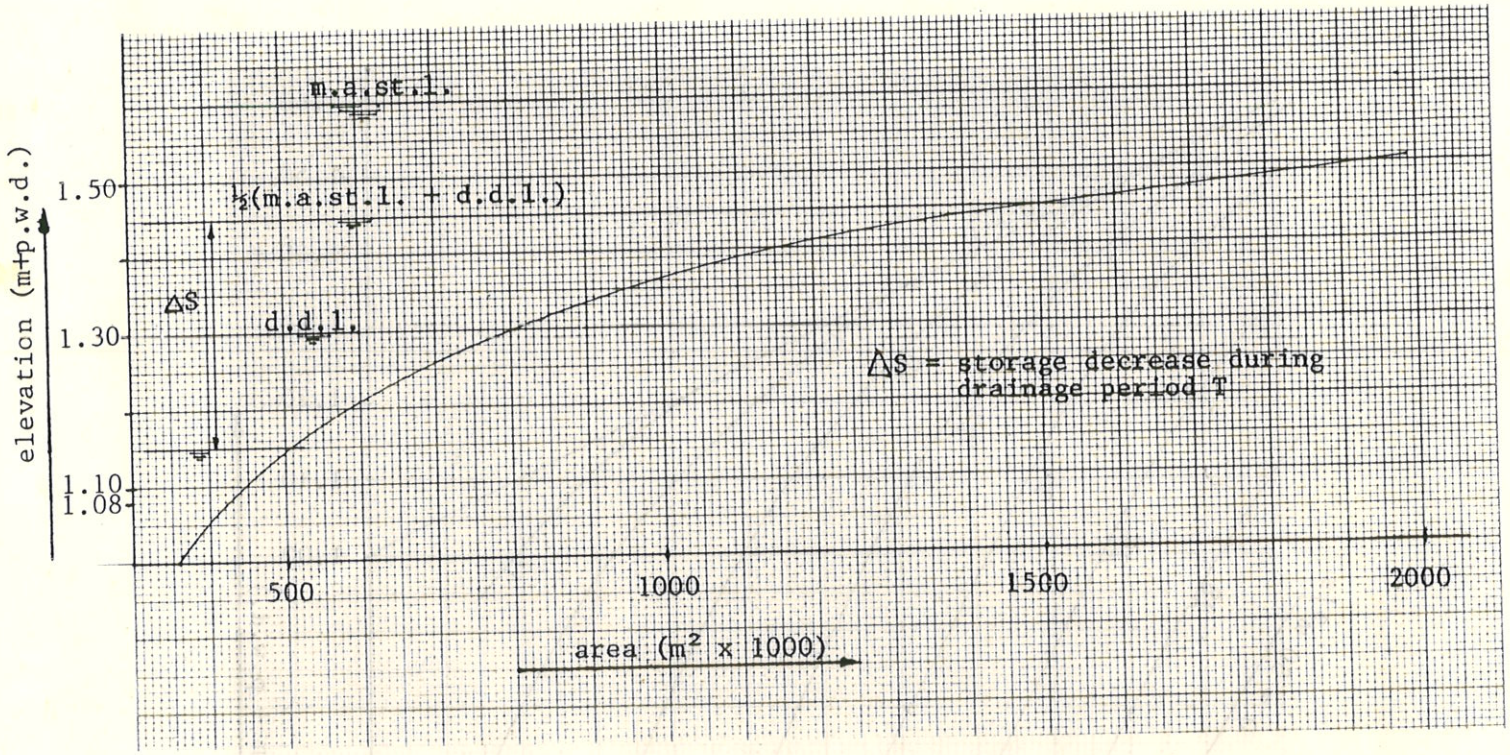


ANNEX IX - 14 CROSS SECTIONS MUNCHIR KHAL, CHAINAGE 0 - 790.

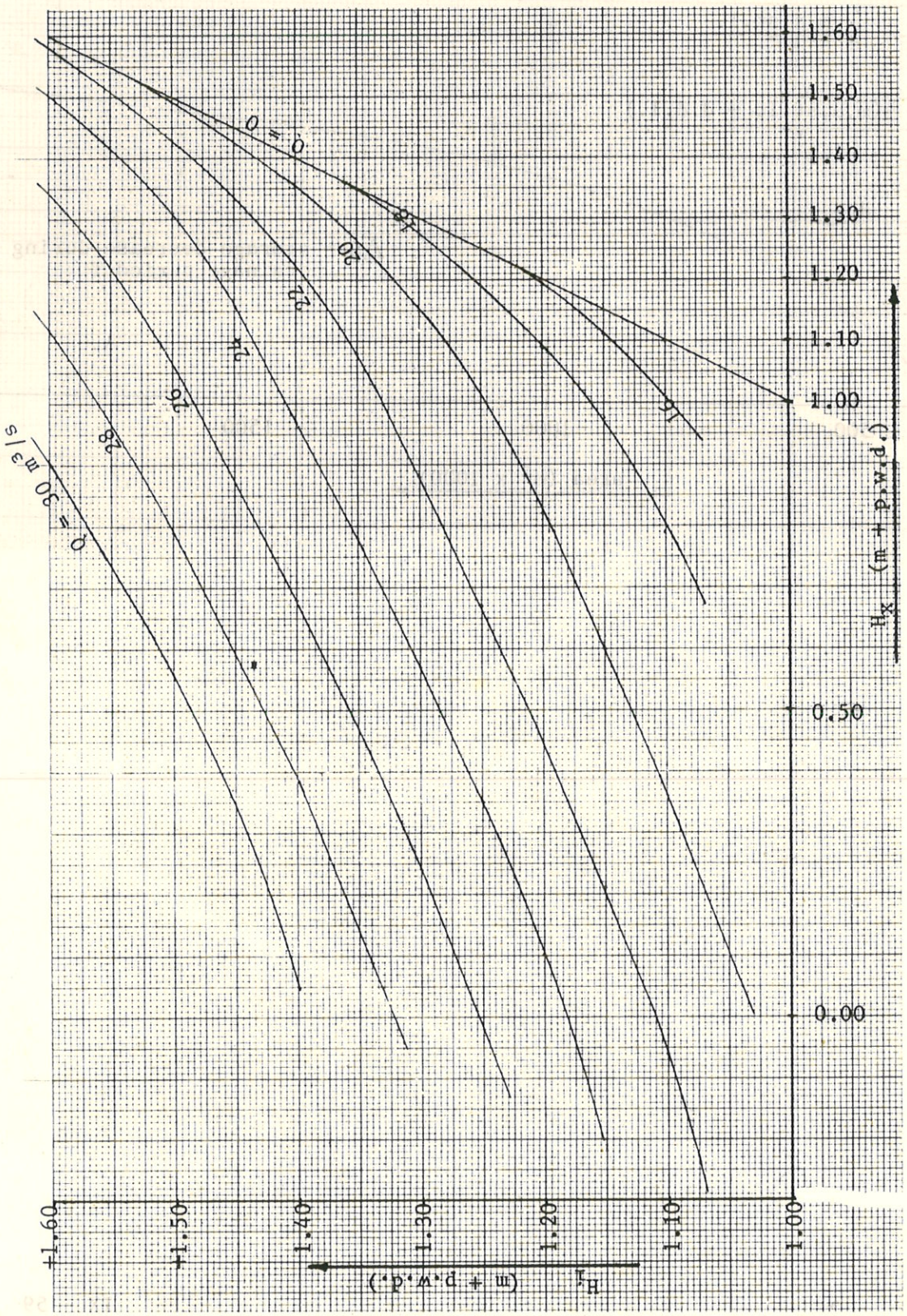


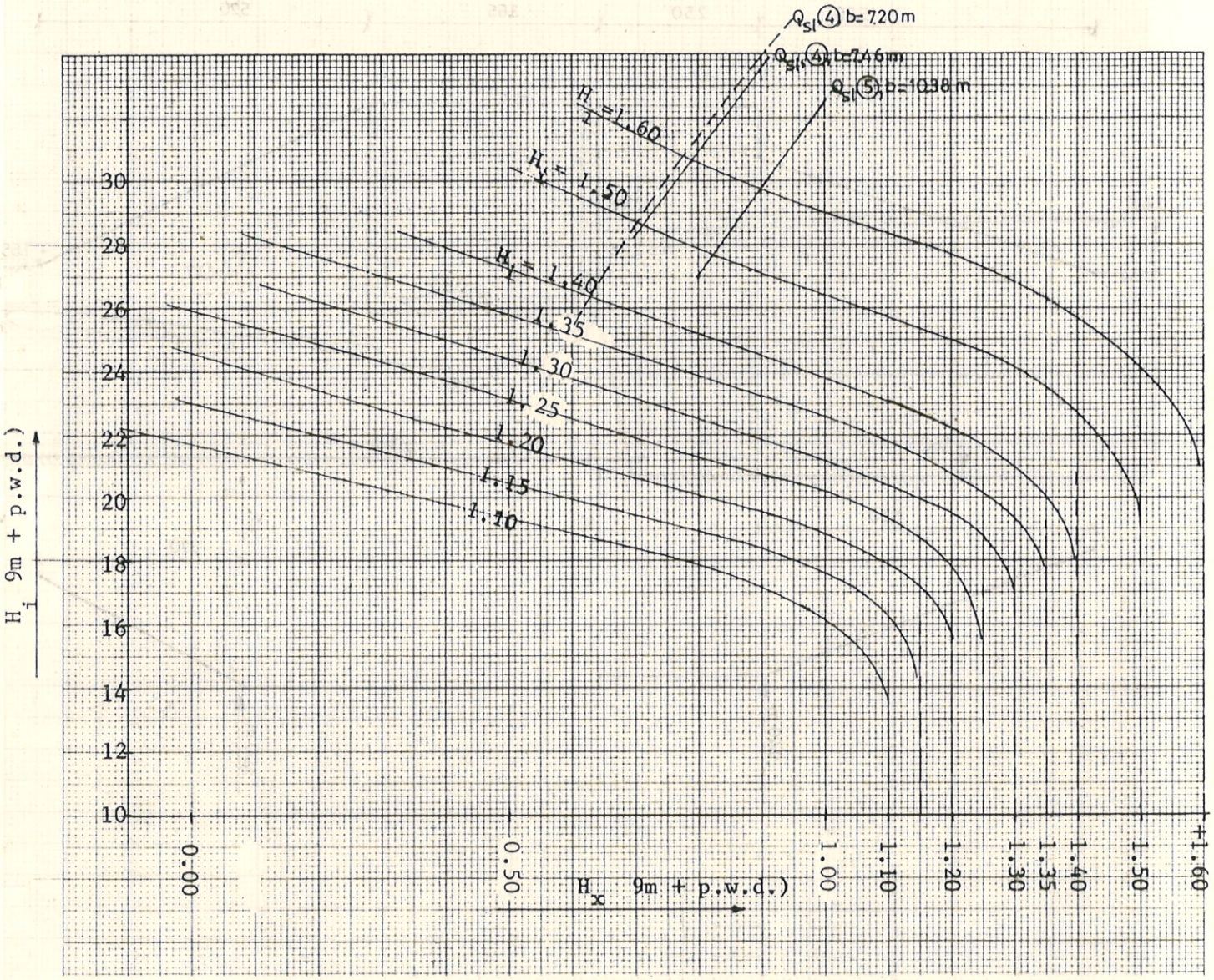
ANNEX IX - 14 CROSS-SECTION
MUNCHIR KHAL
CHAINAGE
0 - 790

ANNEX IX - 15 STORAGE CAPACITY CURVE FOR DRAINAGE CHANNEL AND CONNECTED LOW AREAS.

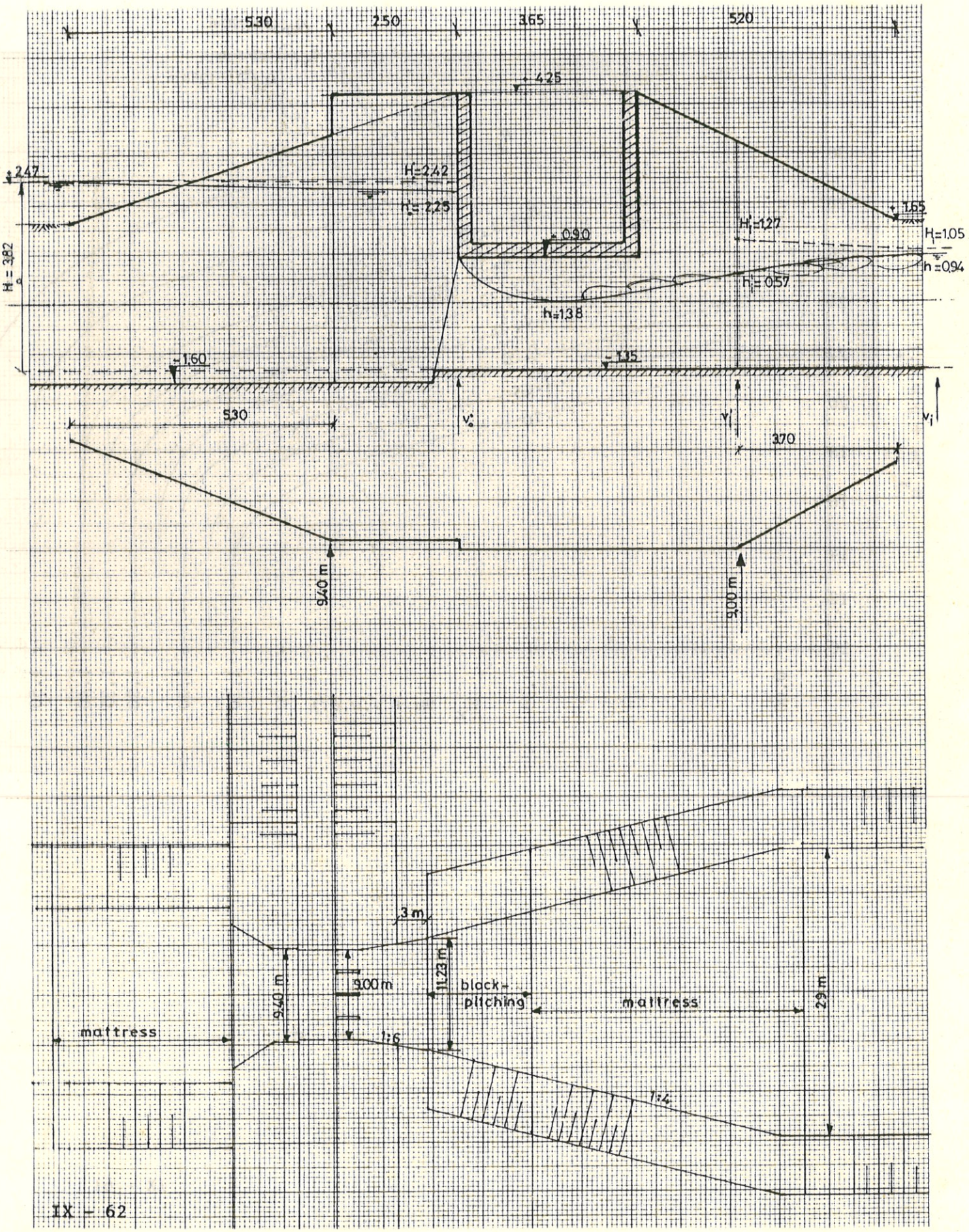
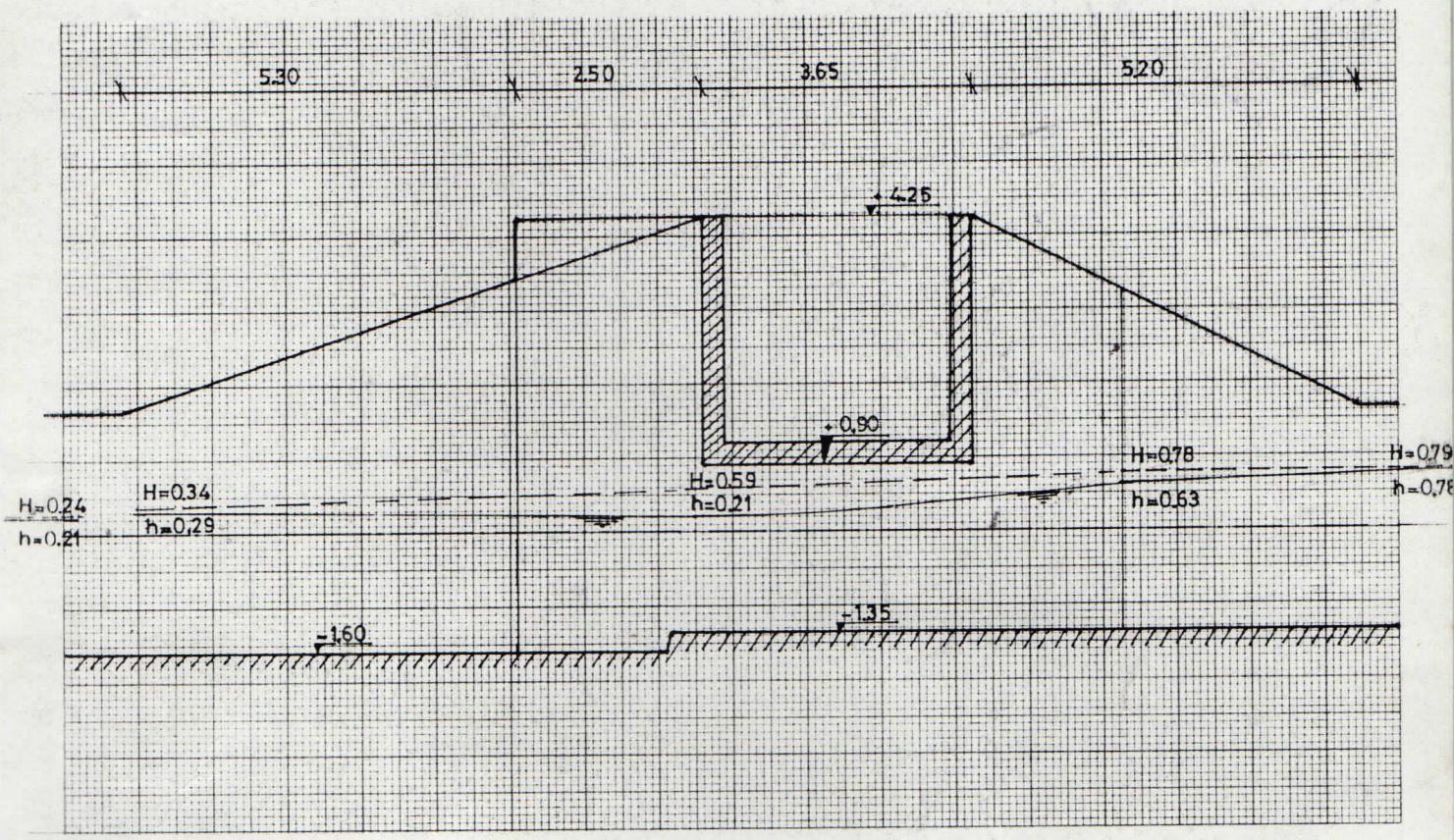


ANNEX IX - 16 DELIVERY CURVES FOR THE MAIN DRAINAGE CHANNEL




Q_sl ④ b=7.20 m
Q_sl ④ b=7.46 m
Q_sl ⑤ b=10.38 m
H_i = 1.60
H_i = 1.50
H_i = 1.40
1.35
1.30
1.25
1.20
1.15
1.10
30
28
26
24
22
20
18
16
14
12
10
H_i 9m + p.w.d.)
0.00
0.50
1.00
1.10
1.20
1.30
1.35
1.40
1.50
+1.60
H_x 9m + p.w.d.)

ANNEX IX - 17 FLOW THROUGH THE SLUICE DURING MAXIMUM FLUSHING OPERATIONS.



ANNEX IX - 18 FLOW THROUGH THE SLUICE DURING MAXIMUM DRAINAGE OPERATIONS.