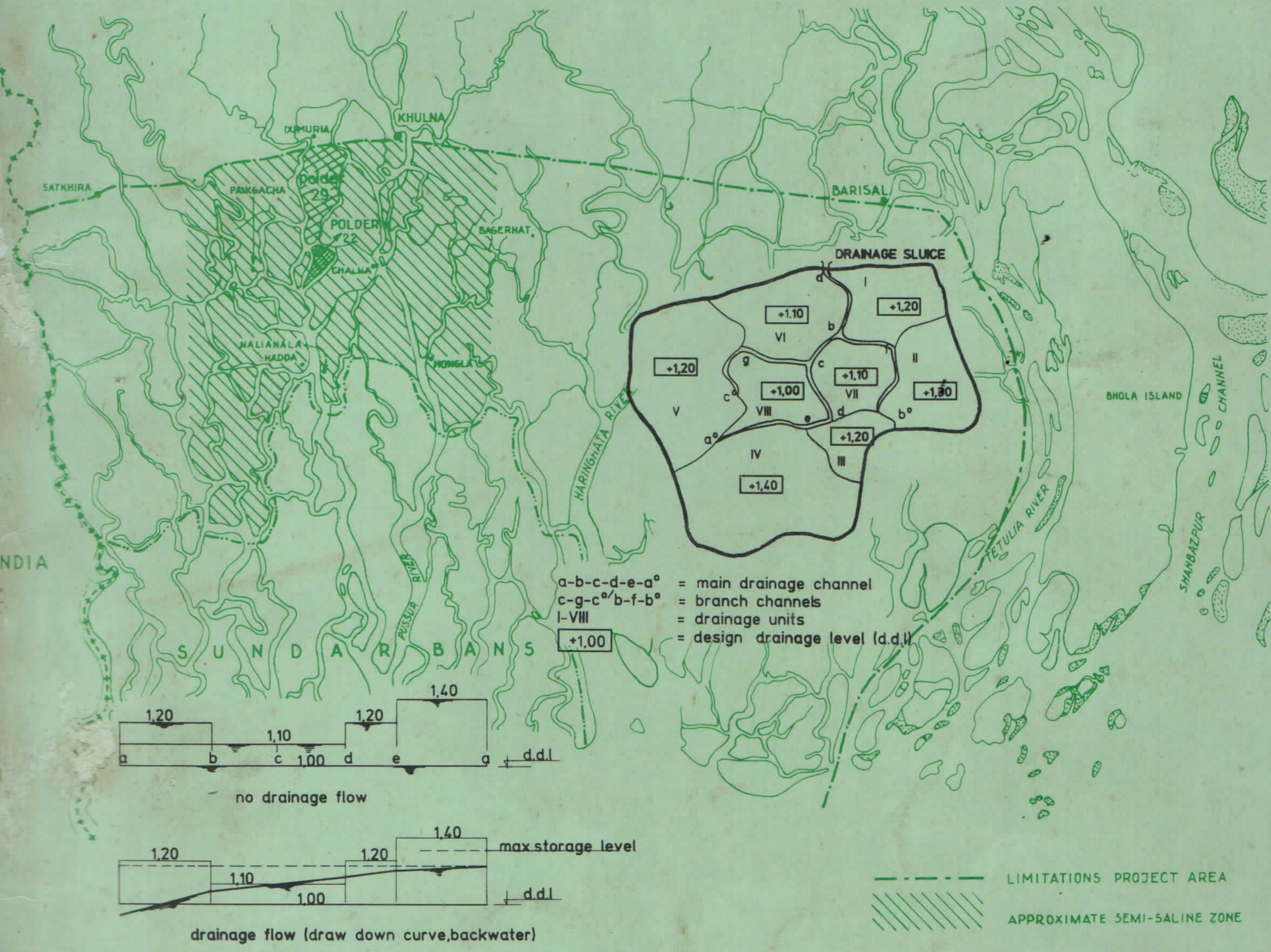
DESIGN MANUAL

FOR POLDERS IN SOUTH-WEST BANGLADESH



PART 4, VOL. IX (SAMPLE DESIGN)

DELTA DEVELOPMENT PROJECT
BANGLADESH-NETHERLANDS JOINT PROGRAMME
UNDER BWDB

DHAKA
NOVEMBER 1985

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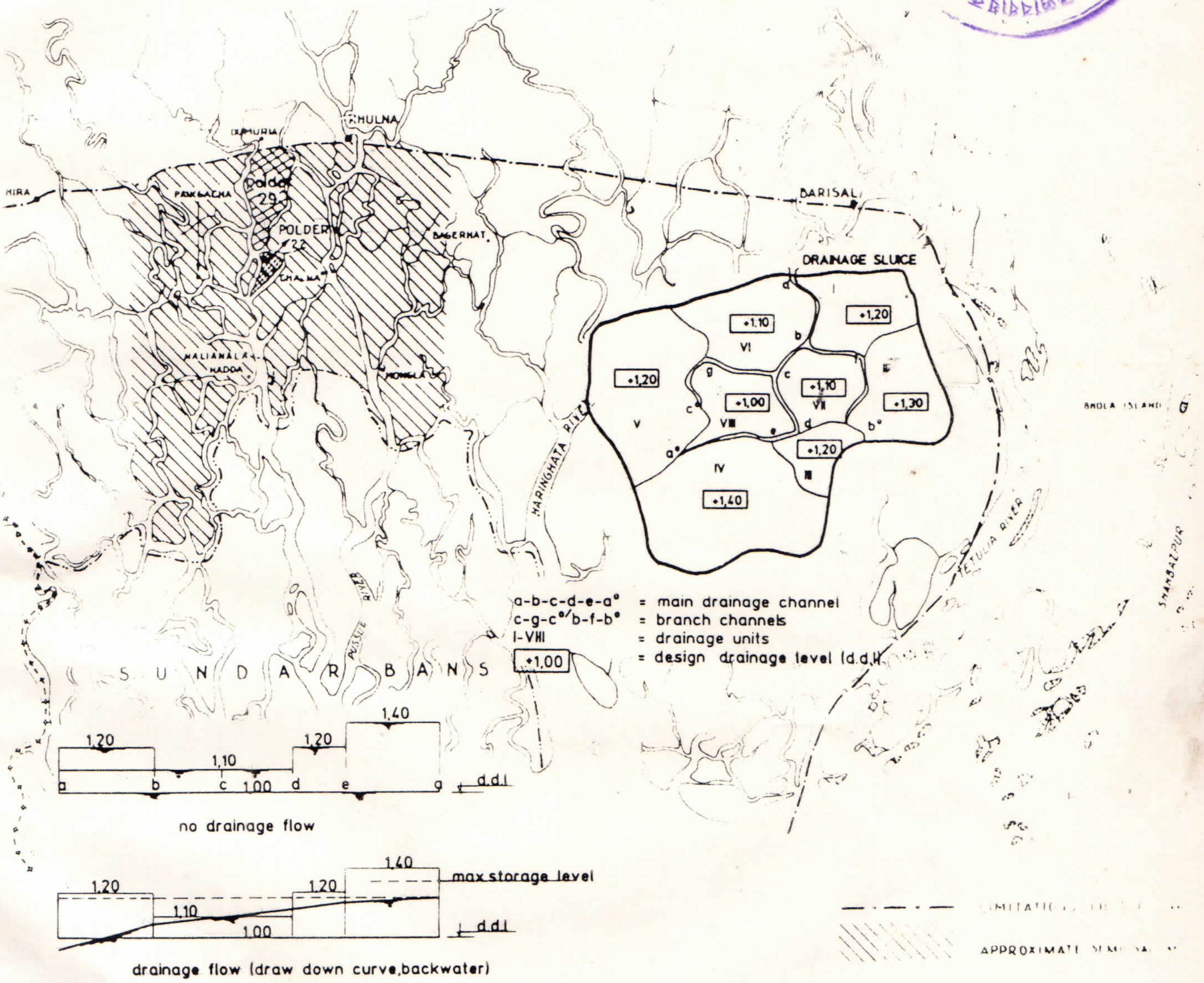

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THIS DESIGN MANUAL CONSISTS OF THE FOLLOWING PARTS AND VOLUMES :

PART 1

VOLUME I — INTRODUCTION AND SUMMARY OF DESIGN PROCEDURES

VOLUME II — SURVEY AND MEASUREMENTS

VOLUME III — DESIGN OF EMBANKMENTS, CLOSURE DAMS

VOLUME IV — IRRIGATION AND DRAINAGE REQUIREMENTS - DESIGN CRITERIA

ANNEX PART 1 — COMMENTS ON AND REMARKS OF DESIGN CIRCLE-I

PART 2

VOLUME V — HYDRAULIC COMPUTATIONS

VOLUME VI — FOUNDATION DESIGN

VOLUME VII — GENERAL AND STRUCTURAL DESIGN ASPECTS

PART 3

VOLUME VIII — BASIC DESIGN DRAWINGS

PART 4

VOLUME IX — WORKED-OUT EXAMPLE.

VOLUME IX WORKED - OUT EXAMPLE

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VOLUME IX WORKED - OUT EXAMPLE

Introduction

In this Volume a complete worked-out example is presented, as far as the basic data provided permitted, of the design of drainage and irrigation structures for a polder in the delta area.

The worked out example is based on the design work carried out by the BWDB Design Department III, who prepared the design of two tidal drainage sluices for Polder - 43/2c,Patuakhali District, a scheme taken up by E.I.P. in 1984-85. For the design of the two sluices, the BWDB - design engineers used the draft manual as the basis.

In the following worked out example it should be noted that a different design drainage level has been assumed than in the BWDB design. The example furthermore only considers the southern part of the scheme, i.e. sluice S-2 in Munchir Khal.

The foundation and structural part of the try-out design of the BWDB design engineer was found to be in good order and procedures were followed very well. Therefore there was no need to elaborate further on this subject in the worked-out example.

2.4.1 Data Collection

a) Information on the area:

from Project Report no. 11: (see ANNEX IX-1)

crops grown : rabi, aus, T-aman.
T-aman grown in nearly entire area.
Damages due to saline water intrusion during spring tides January-May and drainage congestion/fresh water flooding in monsoon period.

project area : 5800 acres = 2347 ha
90% is medium level
low level is 0.30 m below medium
high level is 0.30-0.60 m above medium.
highland : 500 acres = 202 ha: T-aman+rabi
medium land: 5200 acres = 2104 ha:
41% = 863 ha is single crop, T-aman.
36% = 757 ha is double crop, T-aman + aus or rabi.
23% = 484 ha is triple crop, T-aman + aus + rabi.
low land: 100 acres = 41 ha: Boro (12 ha) or
T-aman + rabi (29 ha).

The boro area (12 ha) is submerged during most of the year. Aus seedlings are prepared in April-May, suffer from saline flooding during spring tides.

HYV were for some years grown in 50% of the area, but this practice was abandoned after a few years due to saline constraints (flooding).

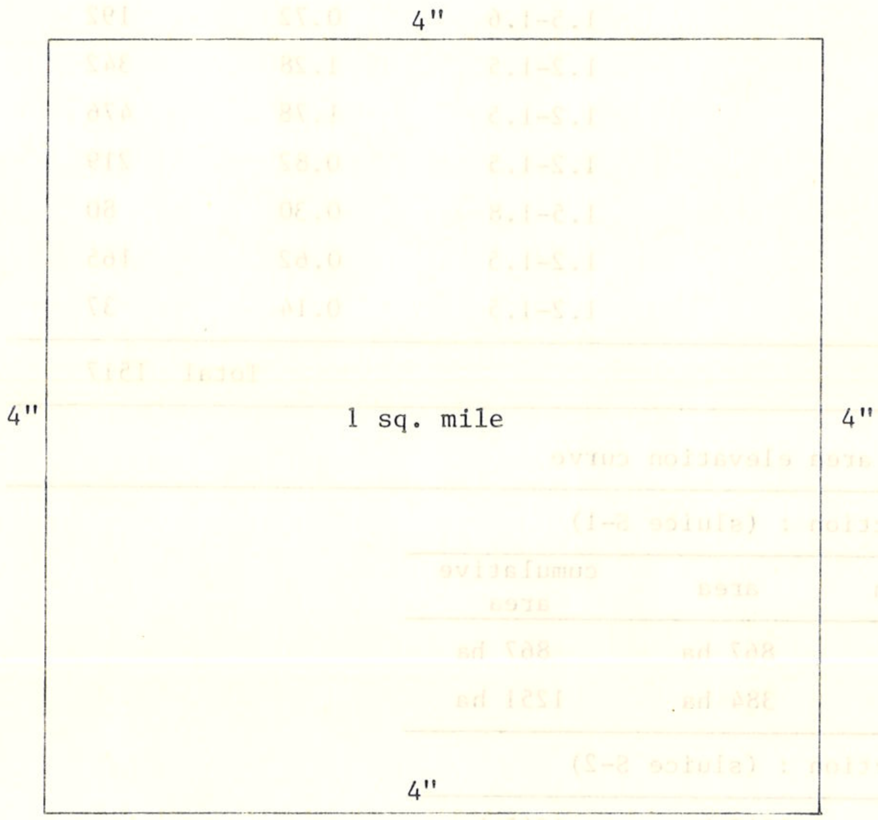
Irrigation is practiced with five low lift pumps and donga over 225 acres = 91 ha.

There exists a large number of internal khals of which 37 running miles (= 59.5 km) have been surveyed.

b) Topographic information

Planimetering of the available topographic maps yields the following information:

- calculation of planimeter constant: scale : 4" = 1 mile.



0.97 revolutions = 1 sq. mile

1 rev. $= \frac{1}{0.97}$ sq. mile $= \frac{1}{0.97}$ x 259 ha = 267 ha.

- planimetering of map: (see ANNEX IV - 2).

Upper section of the map : (sluice S-1)

Symbol no.	planimeter constant	contour in between (m)	revolution no.	required calculated area (ha)
1	267	1.2-1.5	1.23	328
2		1.2-1.5	2.02	539
3		1.5-1.8	0.72	192
4		1.5-1.8	0.72	192
			Total	1251

lower section of the map: (sluice S-2)				
1	267	1.5	0.02	5
2		1.5-1.6	0.72	192
3		1.2-1.5	1.28	342
4		1.2-1.5	1.78	476
5		1.2-1.5	0.82	219
6		1.5-1.8	0.30	80
7		1.2-1.5	0.62	165
8		1.2-1.5	0.14	37
			Total	1517

- data for area elevation curve

upper section : (sluice S-1)

elevation	area	cumulative area
1.5 m	867 ha	867 ha
1.8 m	384 ha	1251 ha

lower section : (sluice S-2)

1.5 m	1245 ha	1245 ha
1.8 m	272 ha	1517 ha

From the contour map it is seen that the total area is within + 1.8 m elevation.

total drainage area for sluice S-1 = 1251 ha

total drainage area for sluice S-2 = 1517 ha.

For area elevation curve (sluice S-2) :

area below 1.8 m:1517 ha.

area below 1.5 m:1245 ha. (rather high!) check maps.

area below 1.35m: 200 ha. (estimated roughly).

area below 1.2 m : 12 ha

lowest point + 1.08 m

highest point + 2.13 m.

c) aerial photos not included in design material

d) detailed topographic survey is available for the proposed sluice construction site.

e) Rainfall and evaporation information from Patuakhali may be used. Duration curves available in the Design Manual.

f) Water levels of station Golachipa (185), River Lohalia of the years 1968-1980 were included in the design material. For the year 1968/1969 the water levels have been analysed and processed to determine the fortnightly mean level. (see ANNEX IX-3 and -4).

On ANNEX IX-5, the fortnightly mean water levels (F.M.L.) for the years of record have been summarized and the mean value (50%) was determined.

On ANNEX IX-6, the same was done for the mean high meters (M.H.W.) and mean low waters (M.L.W.).

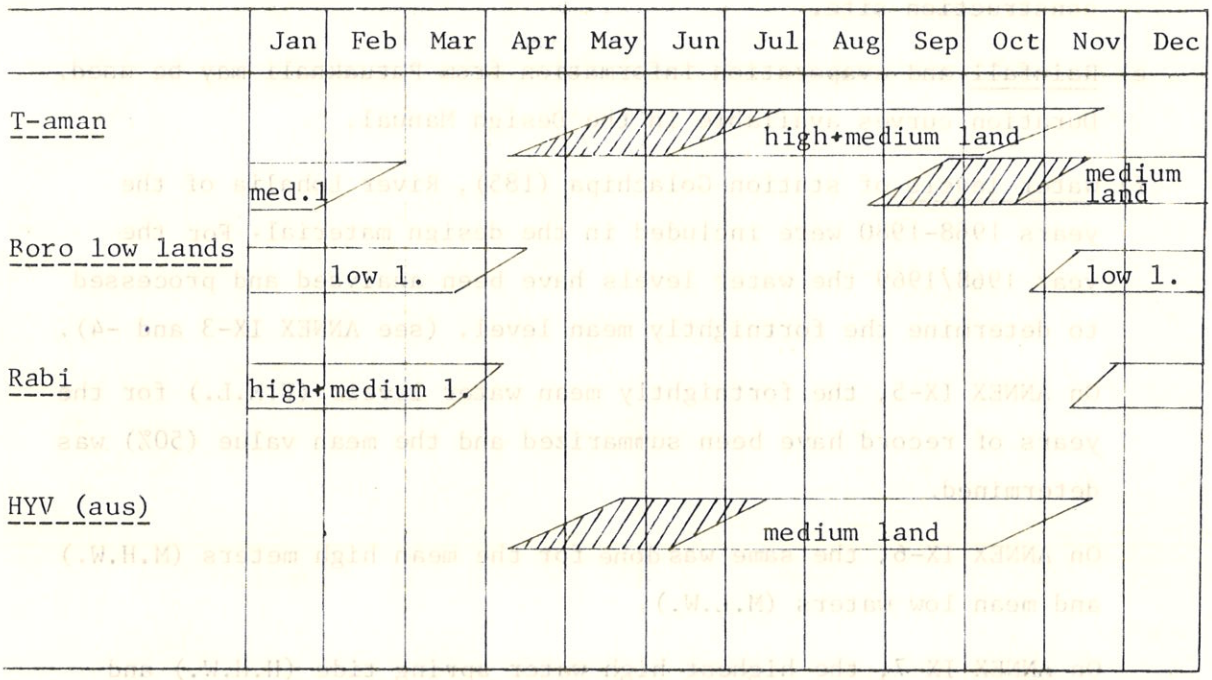
On ANNEX IX-7, the highest high water spring tide (H.H.W.) and the lowest low water springtide (L.L.W.) are summarized for the years of records and the critical period for drainage, as determined later on. (second half of June , first half of July).

Since the Golachipa gauge is very near to the project area, readings from this gauge can be maintained for the tide at the drainage sluices. This was confirmed by the readings of two temporary gauges installed at the proposed sluice sites which were taken during two months and were correlated to the Golachipa gauge reading. (see Project Report no. 11).

g) Salinity
The project report mentions saline river waters from January-May.

2.4.2 Data Evaluation

a) Based on the available information on present agricultural use, the following cropping calendre for the area has been assumed:



From the information of the topographic maps, an area - elevation curve for the S-2 area is prepared and presented in ANNEX IX-8.

Because the area is to be considered as extremely flat, vast areas could be cultivated with HYV. If a d.d.l. of + 1.30 m is assumed, then almost the entire area, except 100 ha could be under HYV.

The m.a.st.l. which should be adopted is then + 1.50 m.

However, the HYV require a very high drainage modulus in order to cope with the restricted capacity of storage of water in the paddy fields. This drainage modulus will result in a high required sluice discharge, which in it self requires a vast storage area because otherwise the water levels in the drainage channel would drop too much during the sluice period T.

The storage area is highly dependent on the features of the agricultural use, and will be influenced by the fact if the lands are bunded and if the drainage channel is bunded.

Since the polder is very flat with little relief, their is definite scope to use the drainage channels also as irrigation supply channels, but this requires all the main channels to be bunded so that the irrigation water can reach all parts of the polder. The bunding of all main channels will however reduce the storage capacity.

On optimum has to be found based on the above considerations.

In the following, it has been assumed arbitrarily that the design drainage level will be at + 1.30 m and the maximum allowable storage level will be at + 1.60 m and that the drainage modulus is determined for long straw varieties which will be grown in the polder.

From ANNEX IV-5 the F.M.L. is plotted on ANNEX IX-9, together with the MHW and MLW levels.

For every fortnightly period, the drainage time T of the average tide curve is determined on ANNEX IX-10. Since no additional information is available on the shape of the tide curve, the best fitting sinus shape for the tide curves is estimated from the information on M.H.W., M.L.W. and F.M.L.

Drainage time T is determined according to point 2.4.4.-d) of Volume I.

2.4.4. Drainage Design of the Polder

a) Determine the drainage modules for each month, May to August for the assumed m.a.st.l. of + 1.60 m. The rainfall duration curves for once in 10 years rainfall and the drainage modulus are plotted on ANNEX IX-11.

The criteria for the determination of the drainage modulus as :

1. The maximum depth of flooding on top of the ground surface should not exceed 200 mm for short straw varieties and 400 mm for long straw varieties. The fieldstorage that can be provided by the rice fields is resp. 100 mm and 300 mm if we consider an initial water level in the rice fields of 100 mm.
2. Starting from the initial water level of 100 mm in the rice fields, the maximum duration of the flooding above this level may not exceed 8 days.

The drainage modulus is now determined for each month and each variety (area cropped with the variety).

b) To yield the sluice discharge the drainage modulus, and area is multiplied with the sluice coefficient $\frac{T_o}{T}$

$$Q_{av} = \text{drainage mod. (mm/day)} \times \text{area (ha)} \times 1.16 \times 10^{-4} \times \frac{T_o}{T} = m^3/s$$

For the sluice S-2 area and the assumptions made for the cropping pattern, the results are :

	May		June		July		August	
	2-16	17-31	1-15	16-30	1-15	16-30	31-14	15-29
drainage modulus:								
- short straw var.	33.4		98.75		71.50		116.67	
- long straw var.	29.9*)		65.5*)		52.13*)		53.5*)	
Sluice period T(hrs)	7.0	6.7	6.5	6.15	6.15	6.05	5.85	5.7
average drainage discharge (m^3/s)								
50% long str. var. / 50% short str. var.	9.87	10.31	17.57	29.14	21.93	22.29	31.74	32.57
100% long str.var.	9.32	9.73	21.99	23.24	18.49	18.80	19.96	20.48

*) 8-day criteria critical.

It appears that the second half of June is the most critical period with a $Q_{av} = 23.24\ m^3/s$ in case the whole polder is considered under long straw varieties.

If we consider 50% of the area under short straw varieties then the total drainage requirement would be 32.57 m^3/s in the second half of August.

In order to be on the conservative side, since only scarse information on the capacity of the drainage channel is available, the design will continue using the design discharge for the case with long straw varieties.

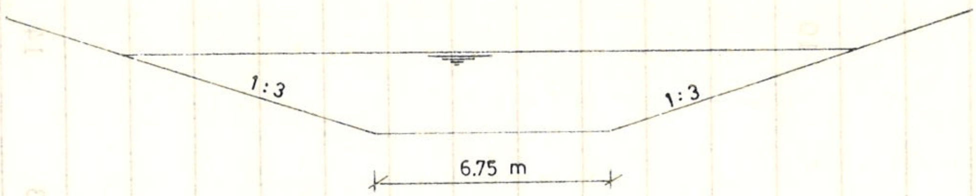
c) Determine the design drainage discharge for each section of the main drainage channel.

Sub-divide therefore the polder area in parts which drain to a certain channel section.

This is done on ANNEX IX-12, and with the table of page 10.

A longsection of the main drainage channel is plotted on ANNEX IX-13 in which the discharge points a .... a* are plotted. For every channel section the mean discharge is now determined.

Total length of main drainage channel as measured from the map is 7880 m for stretch a - a* and 1660 m for stretch e - e*. Only of the first 2000 m (6400 ft) information on the bottom slope is available and of the first 800 m (2600 ft) cross-sections are available which are plotted on ANNEX IX-14 to determine the schematised cross-section for the drainage channel, which is as shown below for the downstream end of the drainage channel.



The schematisation of the whole drainage channel therefore has to be assumed which is done in the following table.

section	a - e	e - i	i - l	l - m	n - o	o - a*
length (m)	1610	1040	1000	590	1740	1900
average bottom slope x 10^{-4}	3	3	3	3	3	3
average bottom width (m)	6.75	6.25	5.25	4.50	3.50	2.00
roughness coefficient (n)	0.045	0.045	0.045	0.045	0.045	0.045
Q_{av} = 23.24 m^3/s	22.76	14.22	11.56	8.34	5.48	3.22

area part	area (ha)	a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s
		discharge point																		
I	264																264			
II	105																			105
III	82																		82	
IV	48															48				
V	19						10											9		
VI	80												24		56					
VII	70										70									
VIII	136																		136	
IX	189			57														113	19	
X	24		24																	
XI	76													76						
XII	177												177							
XIII	40											40								
XIV	53										53									
XV	53									53										
XVI	60								60											
XVII	41				13		14	14												
Total	1517	1517	24	57	13	464	24	14	60	53	123	40	201	76	56	48	264	122	237	105
Cummulative		1517	1517	1493	1436	1423	959	935	921	861	808	685	645	444	368	312	264	464	342	105
Q_{av} = (m^3/s)		2324	2324	22.87	2200	21.80	14.69	14.32	14.11	13.19	12.38	10.49	9.88	6.80	5.64	4.78	4.04	7.11	5.24	1.61

The normal waterdepth for each section is now calculated using Manning's formula and the above information.

normal water-depth (m)	2.75	2.25	2.14	1.90	1.67	1.47

In order to plot the delivery curves for the drainage channel, the normal waterdepths for a range of discharges is furthermore calculated according to the same procedure.

	Q in section *)					
Q sluice	a - e	e - i	i - 1	1 - m	n - o	o - a*
30 m^3/s	h_n=3.11 m	2.54	2.41	2.15	1.87	1.65
28	3.01	2.46	2.33	2.08	1.82	1.60
26	2.90	2.37	2.25	2.01	1.75	1.55
24	2.80	2.28	2.17	1.93	1.69	1.49
22	2.68	2.19	2.08	1.86	1.62	1.44
20	2.56	2.09	1.99	1.77	1.55	1.38
18	2.44	1.98	1.89	1.69	1.48	1.31
16	2.30	1.87	1.78	1.59	1.40	1.25
14	2.15	1.75	1.67	1.49	1.30	1.17
12	2.00	1.62	1.55	1.38	1.22	1.09
10	1.82	1.48	1.41	1.26	1.11	1.01
8	1.63	1.32	1.26	1.13	1.00	0.91

*) The discharges of other stretches are adapted proportionally to the surface drained.

d) From the schematisation of the drainage channel the surface area of the drainage channels is determined at the level of + 1.50 and + 1.30 and at the end of the drainage period T.

section	a - e	e - i	i - l	l - m	n - o	o - a*
length	1610	1040	1000	590	1740	1900
side slope	1 : 3	1 : 3	1 : 3	1 : 3	1 : 3	1 : 3
average bottom lev.	-2.76	-2.37	-2.06	-1.82	-1.47	-0.93
average bottomwidth	6.75	6.25	5.25	4.50	3.50	2.00
surfacewidth at + 1.50 m	32.3	29.5	26.6	24.4	21.3	16.6
area at +1.50	52,003	30,680	26,600	14,396	37,062	31,540
surfacewidth at + 1.30 m	31.1	28.3	25.4	23.2	20.1	15.4
area + 1.30 m	50,071	29,432	25,400	13,688	34,974	29,260

Branch channel e - e*, length 1660 m, estimated surface width at + 1.50 is 24 m, at + 1.30, 22.5 m.

From the Project Report it is known that total length of internal khals in the area is 37 miles (= 59.500 m) for the whole area.

Estimate for the S - 2 area: $\frac{1517}{1517+1251}$ x 59,500 = 32,600 m.

Branch a - a* = 7880 m
Branch e - e* = 1660 m
remaining channels= 23060 m
total 32600 m

Estimate the average surface width of the remaining channels to be 8 m at + 1.50 m level and 7.5 m at + 1.30 m level.

Estimate area of low pockets, which are in open connection with the drainage channels from the topographic map : 41 ha = 410000 m^2 to be submerged at the level of

+ 1.30 and 152 ha (10% of the area is low land) to be submerged and in open connection with the drainage channels at the level of + 1.50 m.

With the above information the storage capacity curve of the drainage channel and connected low lands can be constructed.

at the level of	+ 1.50	+ 1.30	+ 1.00
area of main dr. channel	192,281 m^2	182,825	168,600
branch channel	39,840	37,350	33,600
remaining channels and creeks	184,480	172,950	155,600
low lying areas	1,520,000	410,000	-
total	1,936,600 m^2	803,125 m^2	357,800m^2

It is further estimated that when the drainage level would be reduced to + 1.00 m (below the lowest polder ground level elevation) only the channels will draw water. The surface area at this stage is established by lineairy interpolating the data from the + 1.50 and + 1.30 level. The results are plotted in ANNEX IX-15.

The water level in the polder at the end of the drainage period T is estimated as follows.

area : 1517 ha.
drainage modulus 65½ mm/day
area drainage requirement: $1517 \times 10^4 \times 0.0655 = 993.635$ m^3/day
= 11.50 m^3/s

sluice drainage requirement: 23,24 m^3/s

Storage decrease in drainage channel system:
during one drainage period.
(23,24 - 11.50) x T = 11,74 x 6,15 hrs = 259.924 m^3/tide.

Iterate with the help of ANNEX IX - 15 to the water level H_i at which 259.924 has been evacuated:
Starting from the level of ½ (m.a.st.l. + d.d.l.)

H_i	A	$\bar{A}$	ΔH_i	$\Delta S = \Delta H_i x \bar{A}$	$\gtrless \Delta S$
1.45	1,490,000	-	-	-	-
1.40	1,154,000	1,322,000	0.05	66,100	66,100
1.35	950,000	1,052,000	0.05	52,600	118,700
1.30	803,000	876,500	0.05	43,825	162,525
1.25	680,000	741,500	0.05	37,075	199,600
1.20	584,000	632,000	0.05	31,600	231,200
1.15	507,000	545,500	0.05	27,250	258,450

f)+g) With the previous information, backwater curves for various Q and various stages of H_i are calculated using the schematised information of the drainage channel.

The calculation is demonstrated in ANNEX IX-13.

h) The results of the backwater calculations are presented on ANNEX IX-16, as the delivery curves for the drainage channel.

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 $h_{cr} \geqslant h_2$

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

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 : $\left.\begin{array}{l} h_{cr} = \sqrt[3]{\dfrac{q^2}{g}} = 1.25 \\ h_2 = 1.49 \end{array}\right\}$ 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 :

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

$$h_1 = + 1.90$$

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

$$q = 0.82 \times 1.62 \times \sqrt{2g(1.90 - 1.62)} = 3{,}11 \text{ m}^2/\text{s.}$$

required width : $b = \dfrac{23{,}24}{3{,}11} = 7.46$ m.

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

$\left.\begin{array}{l} h_{cr} = \sqrt[3]{\dfrac{q^2}{g}} = 1.00 \\ h_2 = + 1.62. \end{array}\right\}$ 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) <u>Determination of the barrel - height</u>

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

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

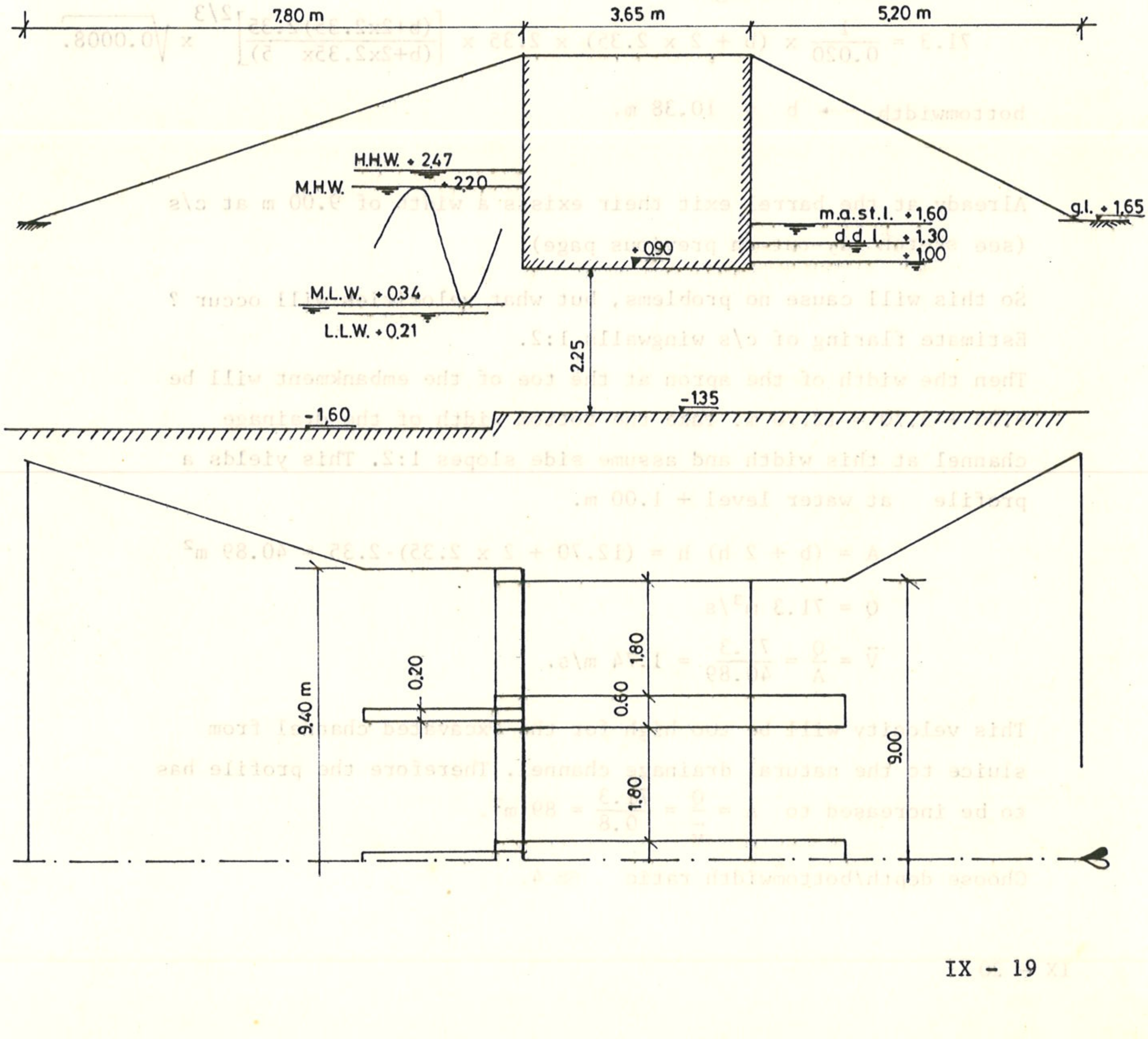
if d.d.l. is chosen :

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

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) <u>A sketch lay-out is now made:</u>



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).

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

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.

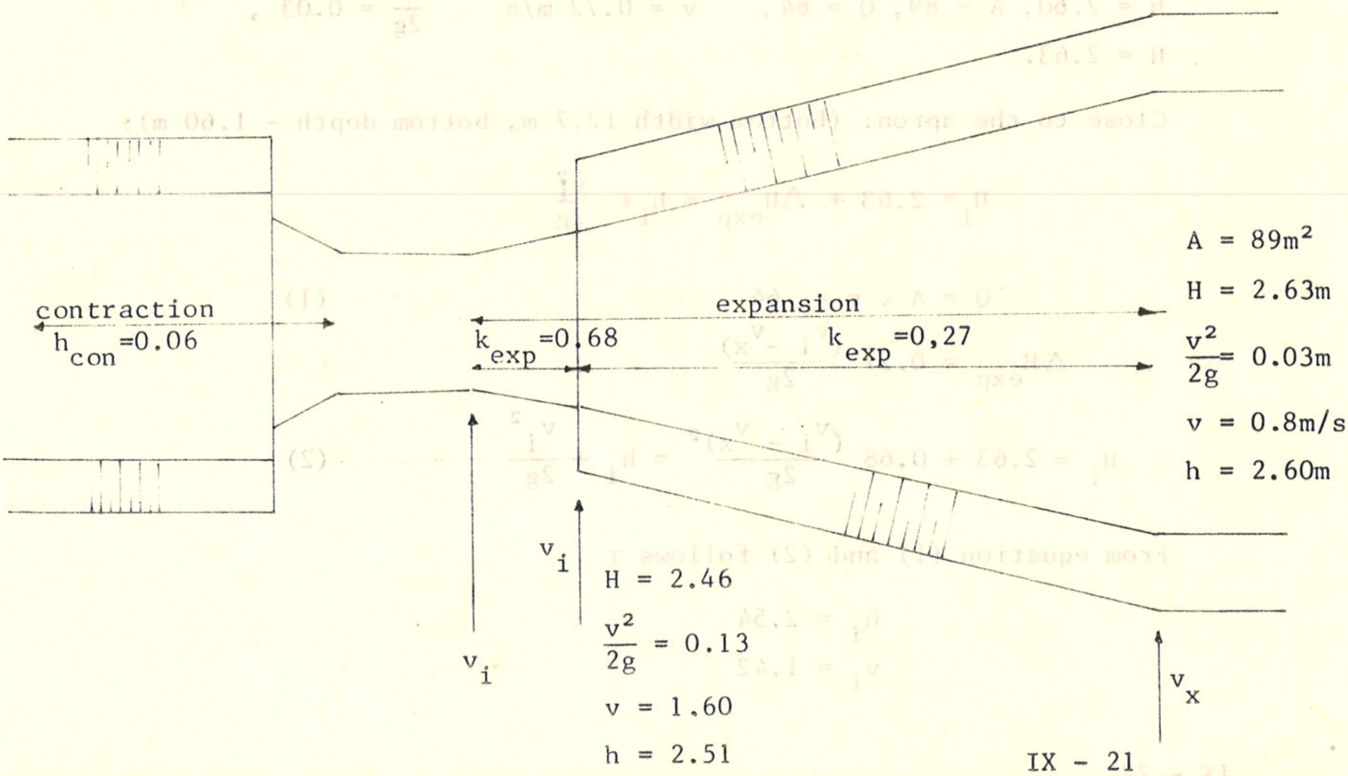
See ANNEX IX-17.

$$H_o = + 2.47 + / - 1.35 / = 3.82 \text{ m}$$

$$H_x = + 1.00 + 0.03 + / - 1.35 / = 2.38 \text{ m.}$$

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

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



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 $Q = 71.3 \text{ m}^3/\text{s}$.

By iteration it is found that the best fit $Q = 64 \text{ m}^3/\text{s}$. 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 :

$$Q = 64\ m^3/s$$

$$\left.\begin{array}{l} v_i = 1.42\ m/s. \\ v_{box} = \dfrac{64}{4x1,8x2,25} = 3.95\ m/s. \end{array}\right\} \begin{array}{l} \Delta H_{con} = 0,05 \\ \Delta H_{exp} = 0,68 \times \dfrac{(3.91-1.42)^2}{2g} = 0,22 \end{array}$$

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

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

At x-section I :

$$\left.\begin{array}{l} H_i' = 2.62 = h_i + \dfrac{(v_i)^2}{2g} \\ v_i = \dfrac{64}{h_i' \times 9} \end{array}\right\} \begin{array}{l} v_i' = 3.70\ m/s \\ h_i' = 1.92\ m. \end{array}$$

At x-section II:

$$\left.\begin{array}{l} H_o' = 3.77 = h_o' + \dfrac{(v_o')^2}{2g} \\ v_o' = \dfrac{64}{(h_o'+0.25)x9.40} \end{array}\right\} \begin{array}{l} h_o' = 3.60\ m \\ v_o' = 1.81\ m. \end{array}$$

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 $2.25 \times 0.613 = 1.38$ 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 $d_2 = (+1.00 + / - 1.35 /) = 2.35$, $L = 4.2 \times 2.35 = 9.87$ 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 : $2.35 \times 11.23 = 26.40$ 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) : $H = 2.63 + 0.27 \dfrac{(v - 0.78)^2}{2g} = h + \dfrac{v^2}{2g}$.

$Q = (12.7 + 2\ h)\ h \times v$, 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 :

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

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

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

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

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

The water levels in the sluice can be calculated.

In the drainage channel :

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

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

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

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 : $F = \frac{v}{\sqrt{g\ h}} = \frac{2.74}{\sqrt{9.8 \text{x} 1.50}} = 0.70.$

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 :

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

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

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

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

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

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

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 $\longrightarrow$ 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	yes	yes	yes	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) <u>Nett Irrigation requirement</u>

(= 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 x perc.+l.p-r		319*)	38	40	-20	153

*) as determined above.

f) The irrigation supply requirement is the 319 x $\frac{1}{0,8}$ x $\frac{1}{0,8}$ = 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.

$$C_1 = \sqrt{\frac{1}{1.5 + \frac{2 \times 25.13 \times (0.013)^2 \times 9{,}8}{(0.0773)^{4/3}}}} = 0.4983 \quad \begin{Vmatrix} D = 0.3048 \text{ m} \\ A = 0.074 \text{ m}^2 \\ R = 0.0773 \text{ m} \end{Vmatrix}$$

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

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

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 - 10 : TIDE CURVES.

ANNEX IX - 11 : RAINFALL DURATION CURVES, DETERMINATION OF DRAINAGE MODULUS

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, BACKWATER CALCULATION

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

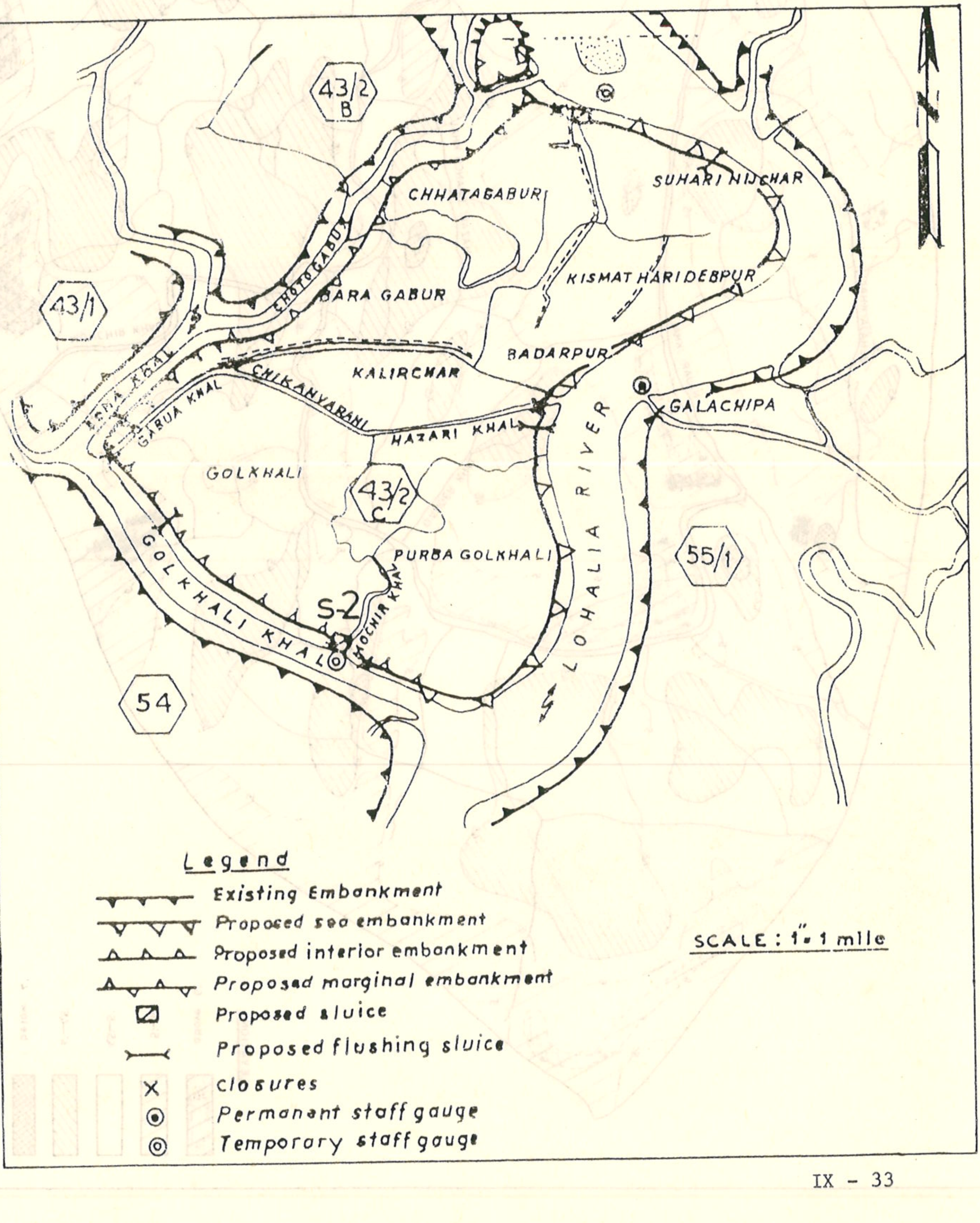
ANNEX IX - 15 : STORAGE CAPACITY CURVE FOR THE DRAINAGE CHANNEL AND CONNECTED LOW AREAS

ANNEX IX - 16 : DELIVERY CURVES FOR THE MAIN DRAINAGE CHANNEL

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

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

ANNEX IX - 1. AREA MAP



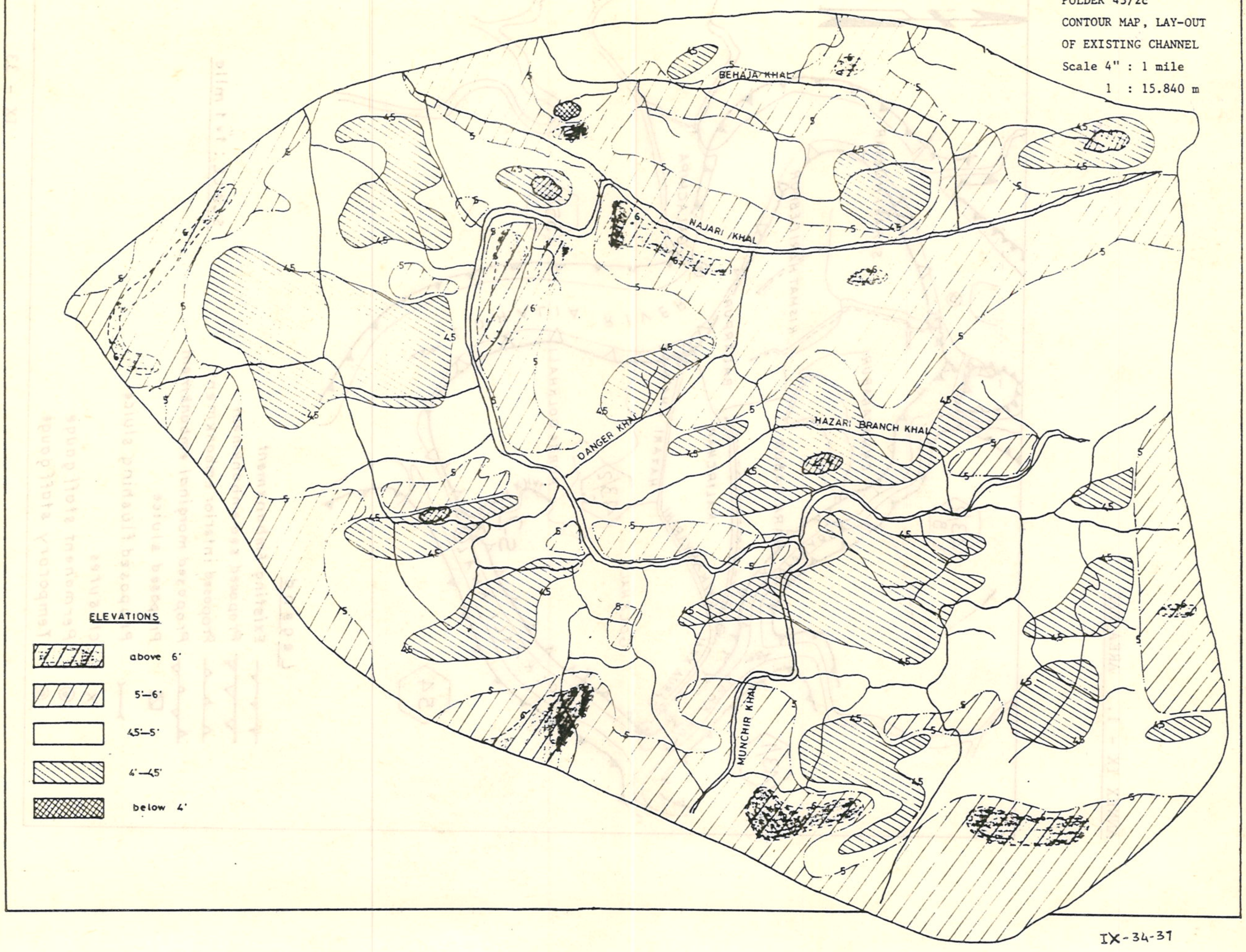
ANNEX IX - 2

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	0.29	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 / Fortnightly period	No. of H.W.	$\sum$H.W	MHW	No. of L.W	$\sum$L.W	MLW	$\sum$HW + $\sum$LW	No. of peaks	FML
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

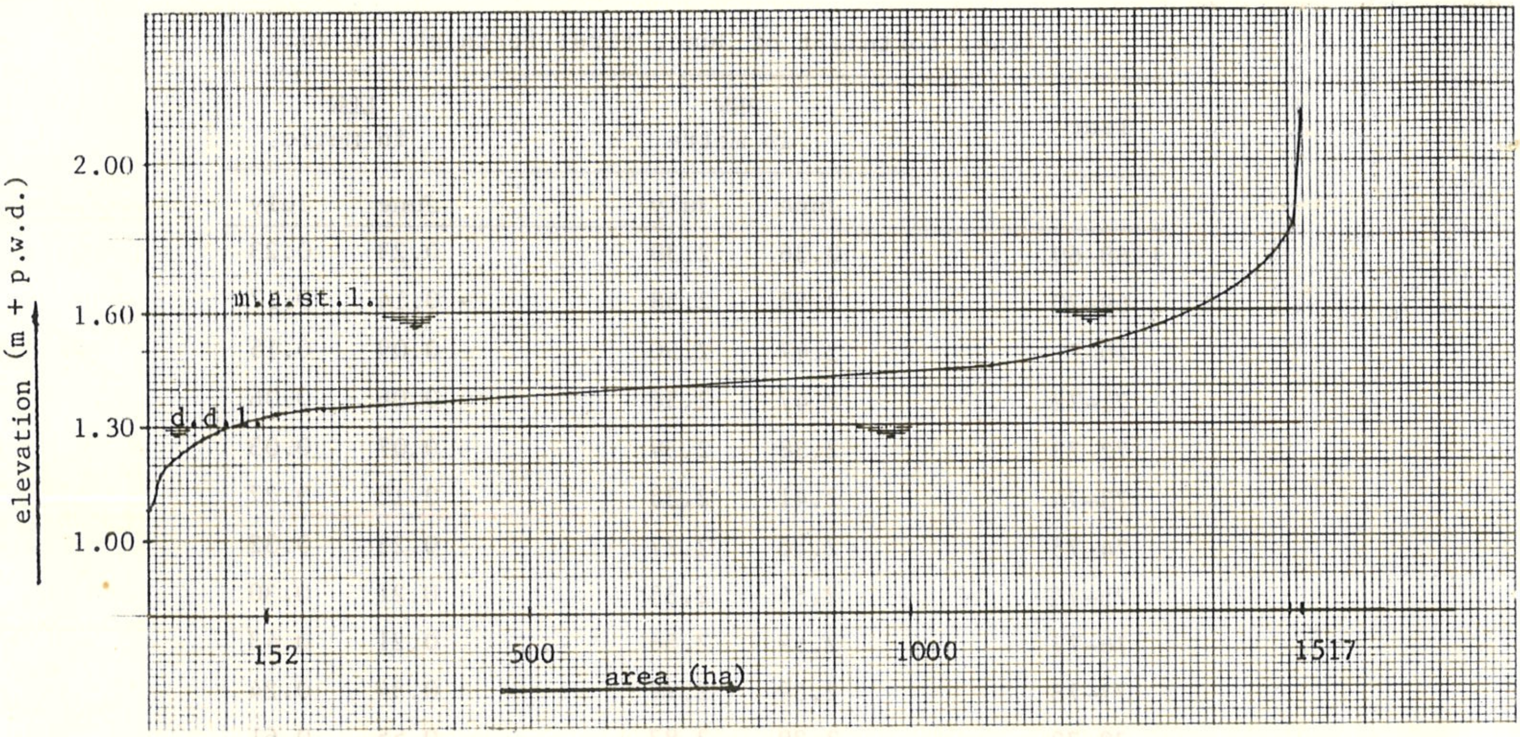
Fortnightly period	MEAN HIGH WATER (MHW) in meter								MEAN LOW WATER (MLW) in meter							
	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8
1968-69	1.84	1.91	2.23	2.04	2.19	2.16	2.32	2.15	0.02	0.14	0.24	0.37	0.34	0.46	0.46	0.52
69-70	2.02	2.34	2.45	2.32	2.21	2.33	2.11	2.12	-.37	-.18	-.32	-.41	.36	.45	.36	.24
70-71	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

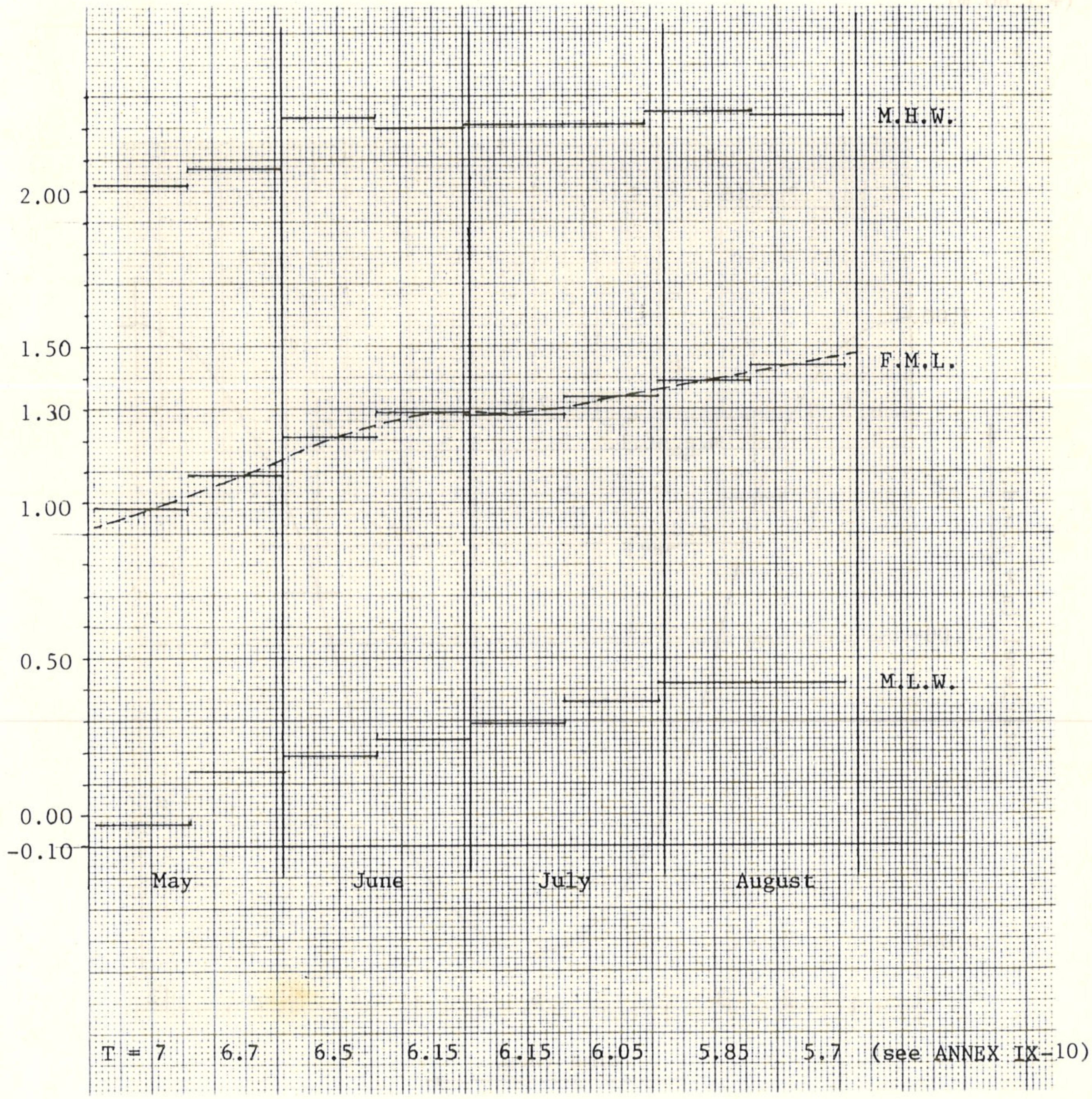
(last fortnight of June, first fortnight of July)

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

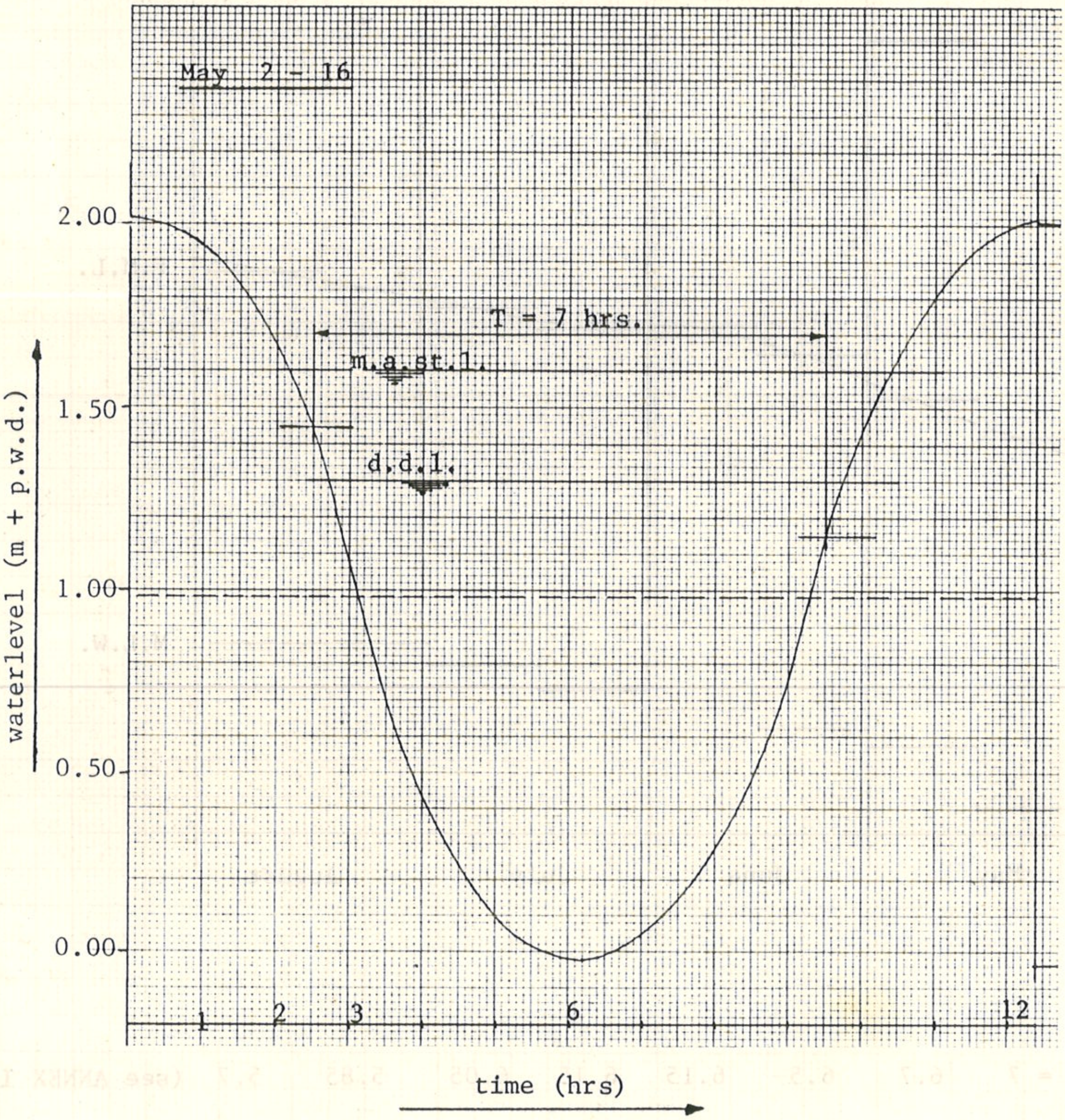


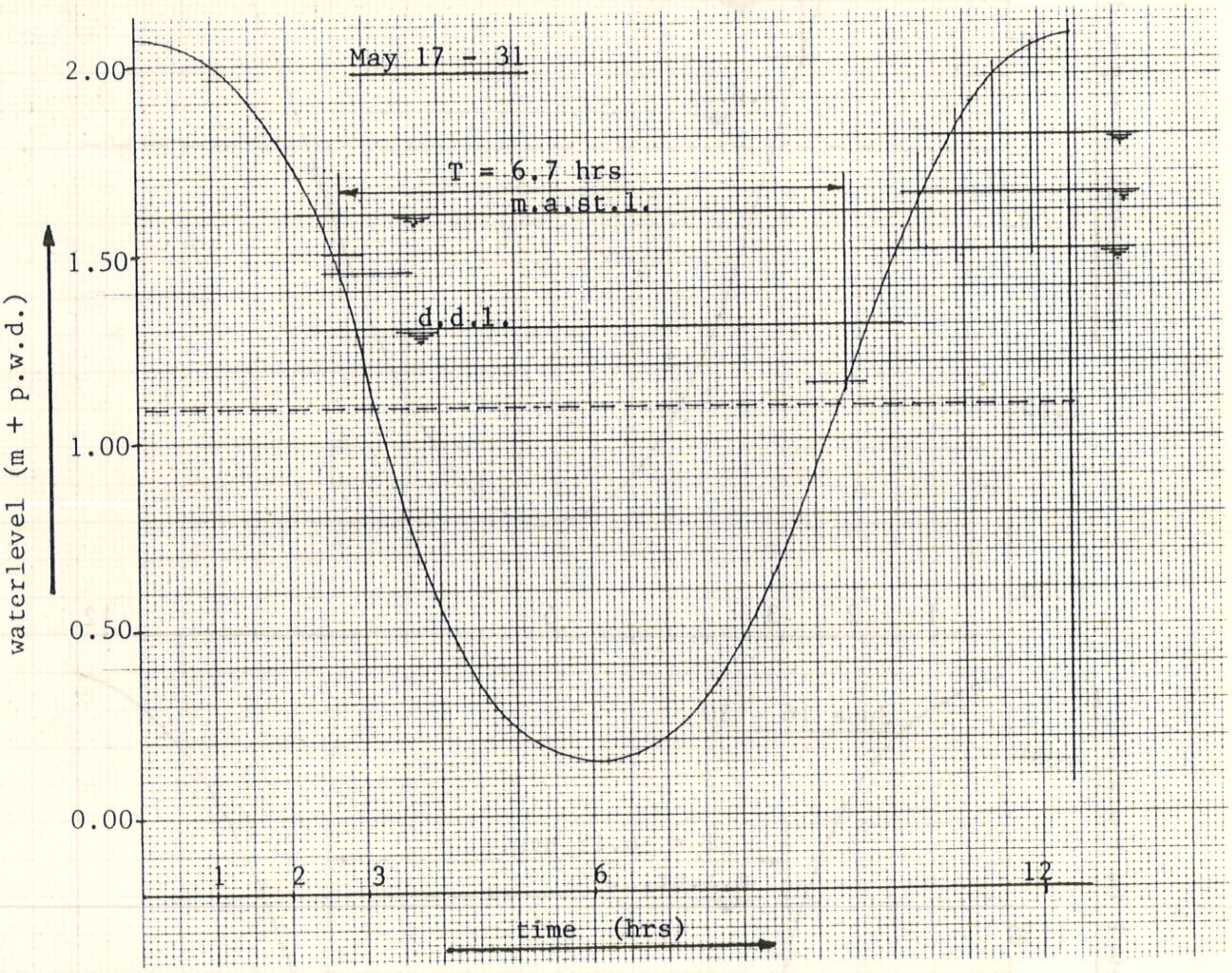
ANNEX IX - 9 AVERAGE TIDES DURING MAY - AUGUST

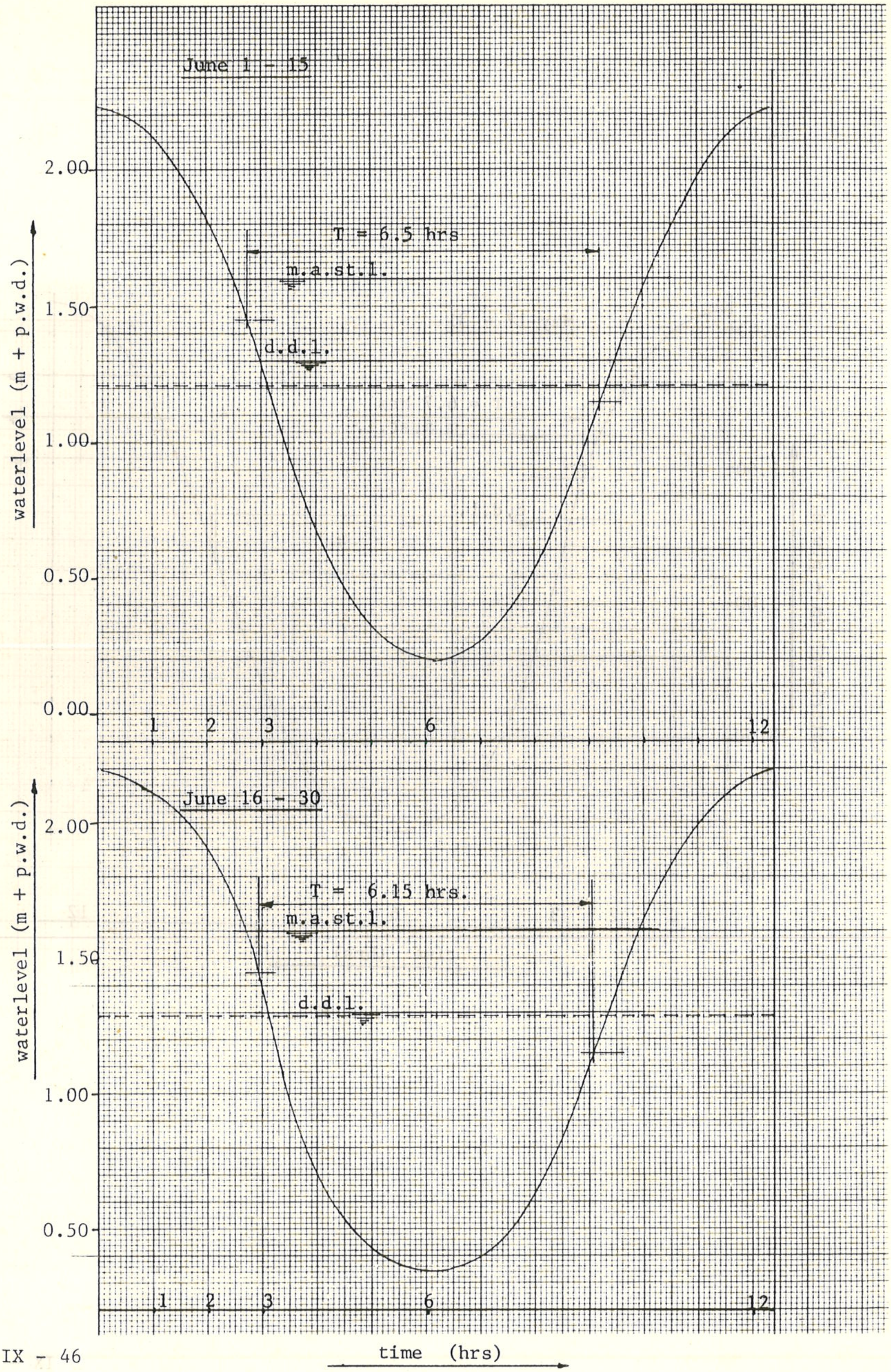


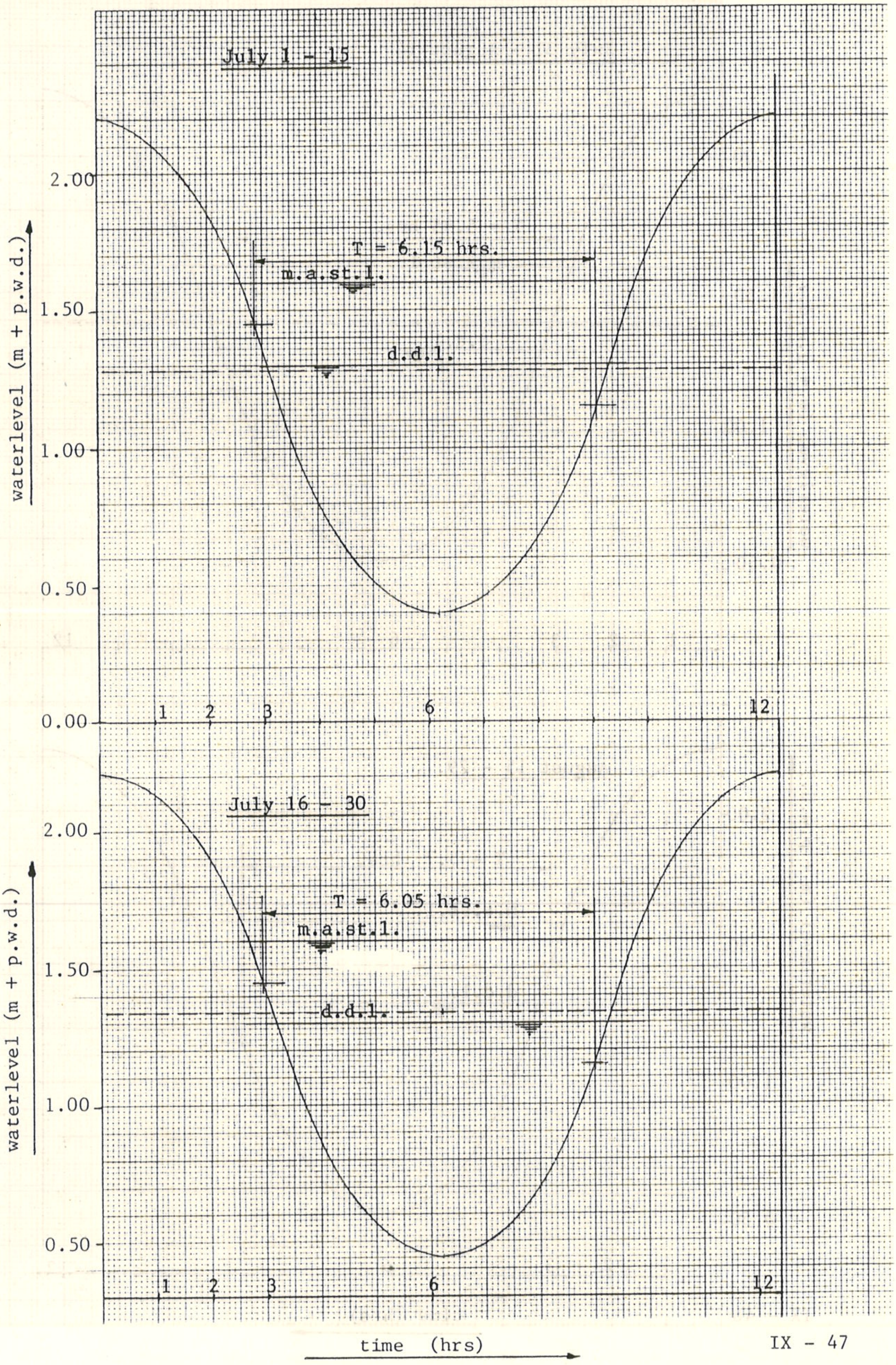
ANNEX IX - 10 TIDE CURVES

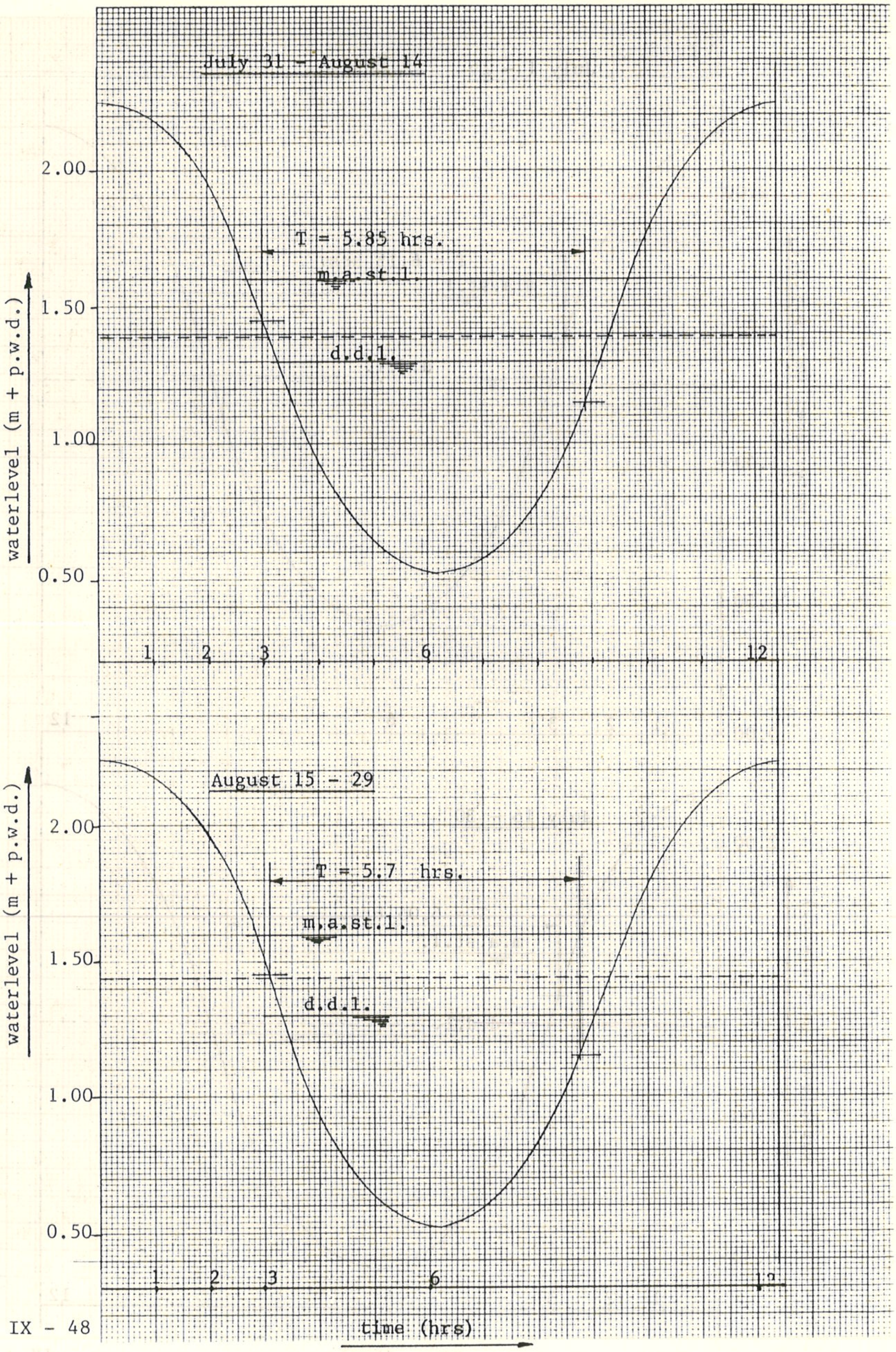
For the fortnightly periods during May - August and the determination of the drainage period T for the assumed d.d.l. (+ 1.30 m) and m.a.st.l. (+ 1.60 m).



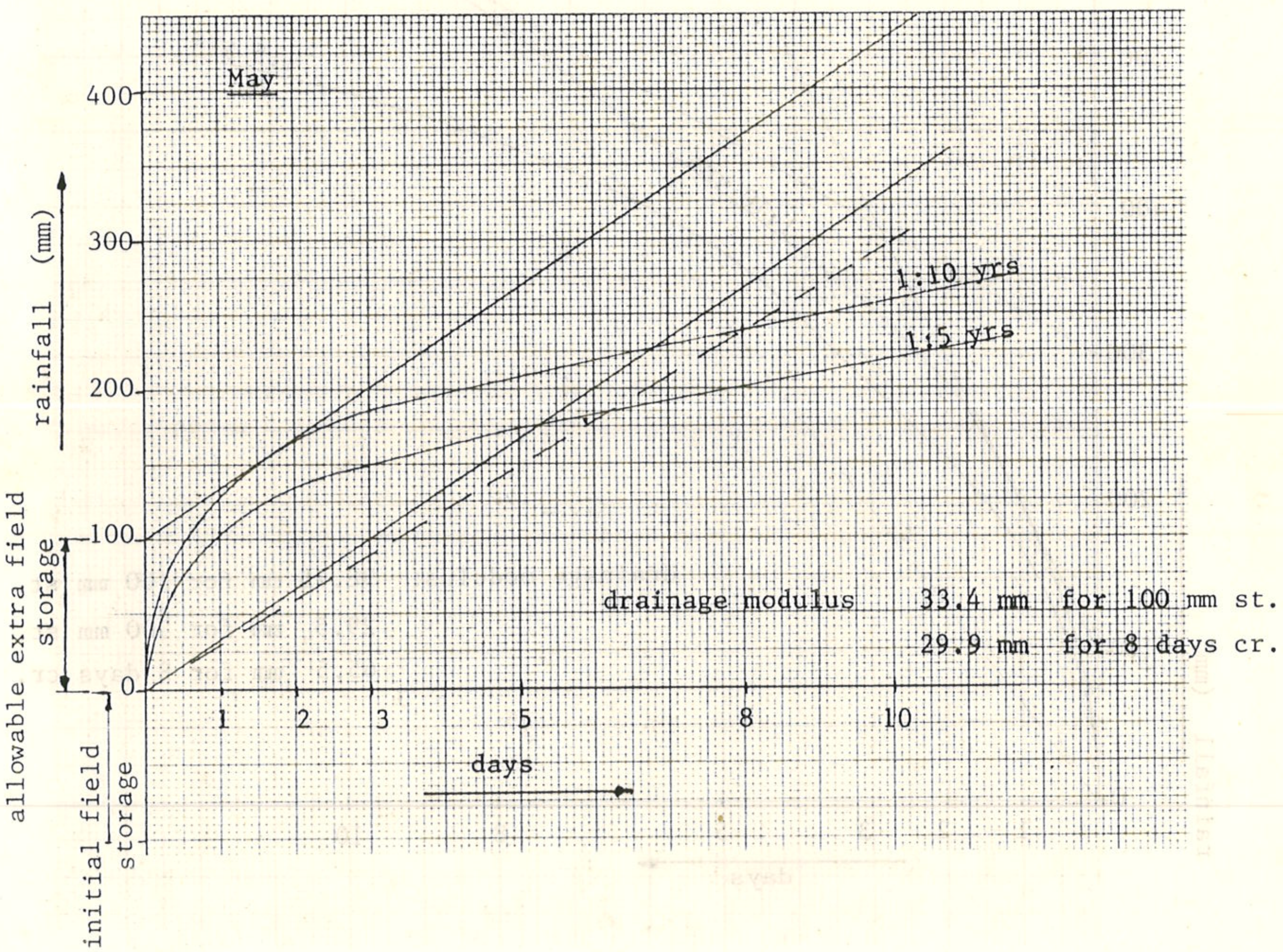

May 17 - 31
T = 6.7 hrs
m.a.st.l.
d.d.l.
waterlevel (m + p.w.d.)
2.00
1.50
1.00
0.50
0.00
1
2
3
6
12
time (hrs)

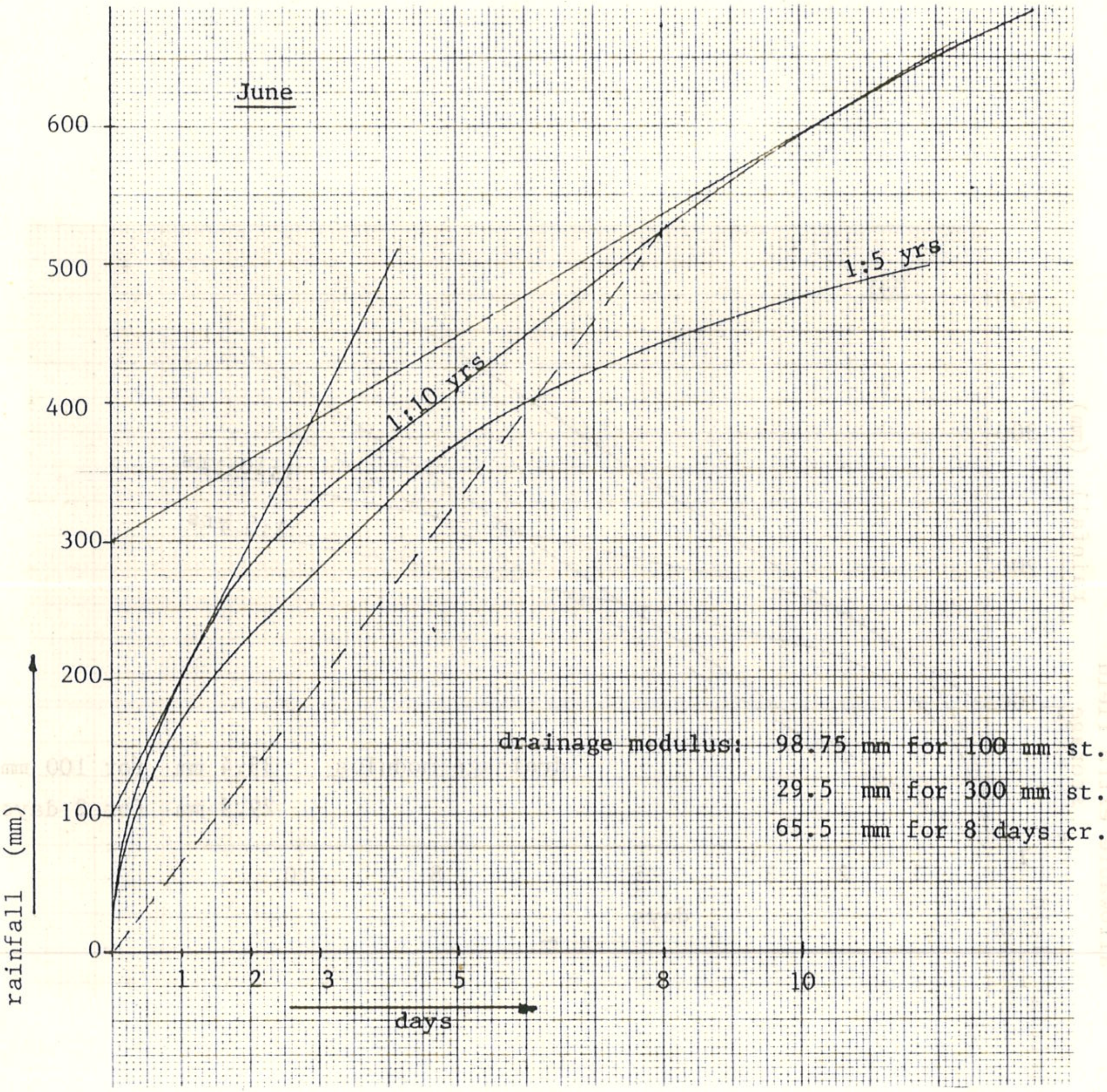

June 1 - 15
T = 6.5 hrs
m.a.st.l.
d.d.l.
waterlevel (m + p.w.d.)
2.00
1.50
1.00
0.50
0.00
1
2
3
6
12
June 16 - 30
T = 6.15 hrs.
m.a.st.l.
d.d.l.
waterlevel (m + p.w.d.)
2.00
1.50
1.00
0.50
1
2
3
6
12
time (hrs)

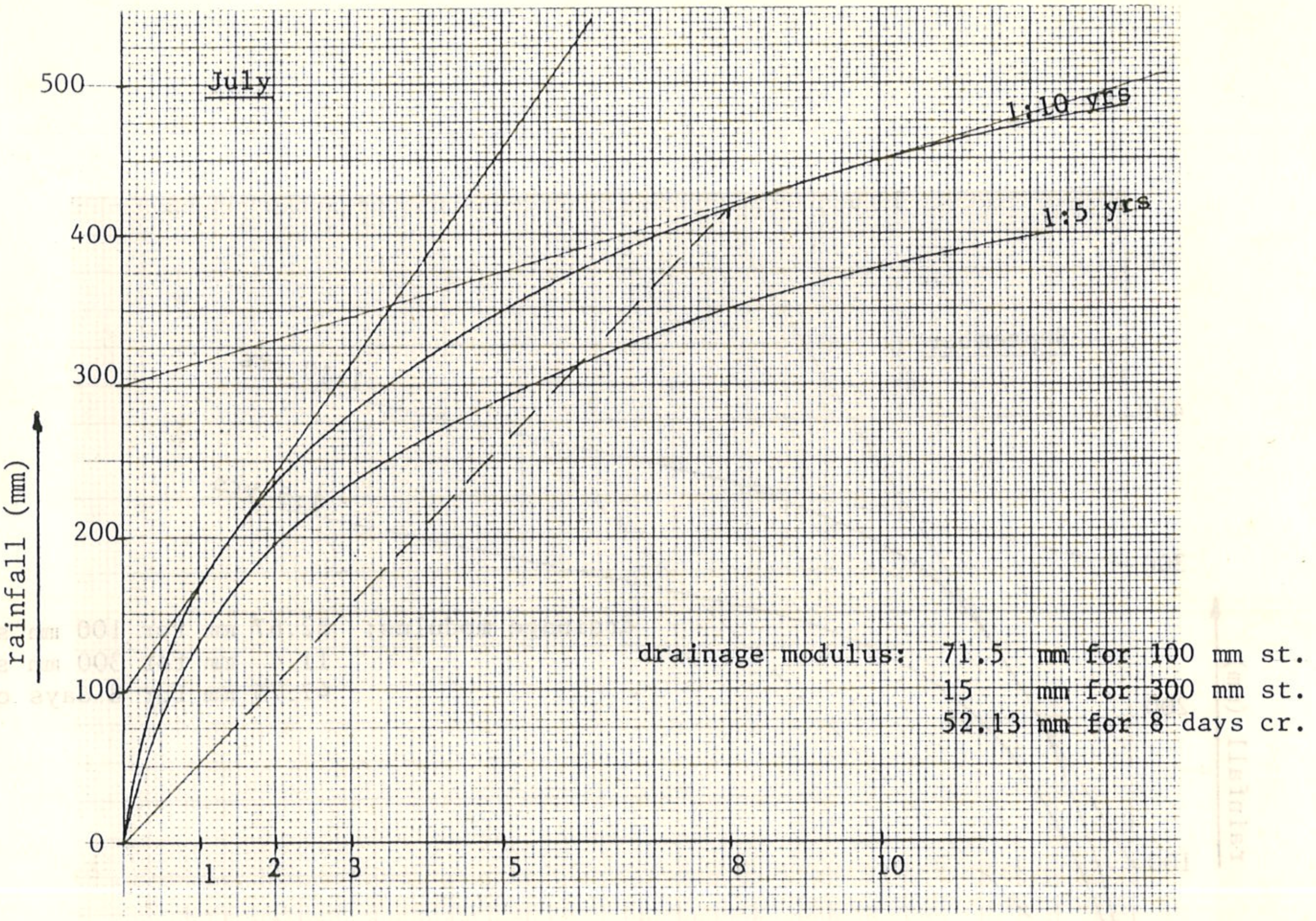

July 1 – 15
waterlevel (m + p.w.d.)
2.00
1.50
1.00
0.50
T = 6.15 hrs.
m.a.st.l.
d.d.l.
0.00
1
2
3
6
12
July 16 – 30
waterlevel (m + p.w.d.)
2.00
1.50
1.00
0.50
T = 6.05 hrs.
m.a.st.l.
d.d.l.
1
2
3
6
12
time (hrs)

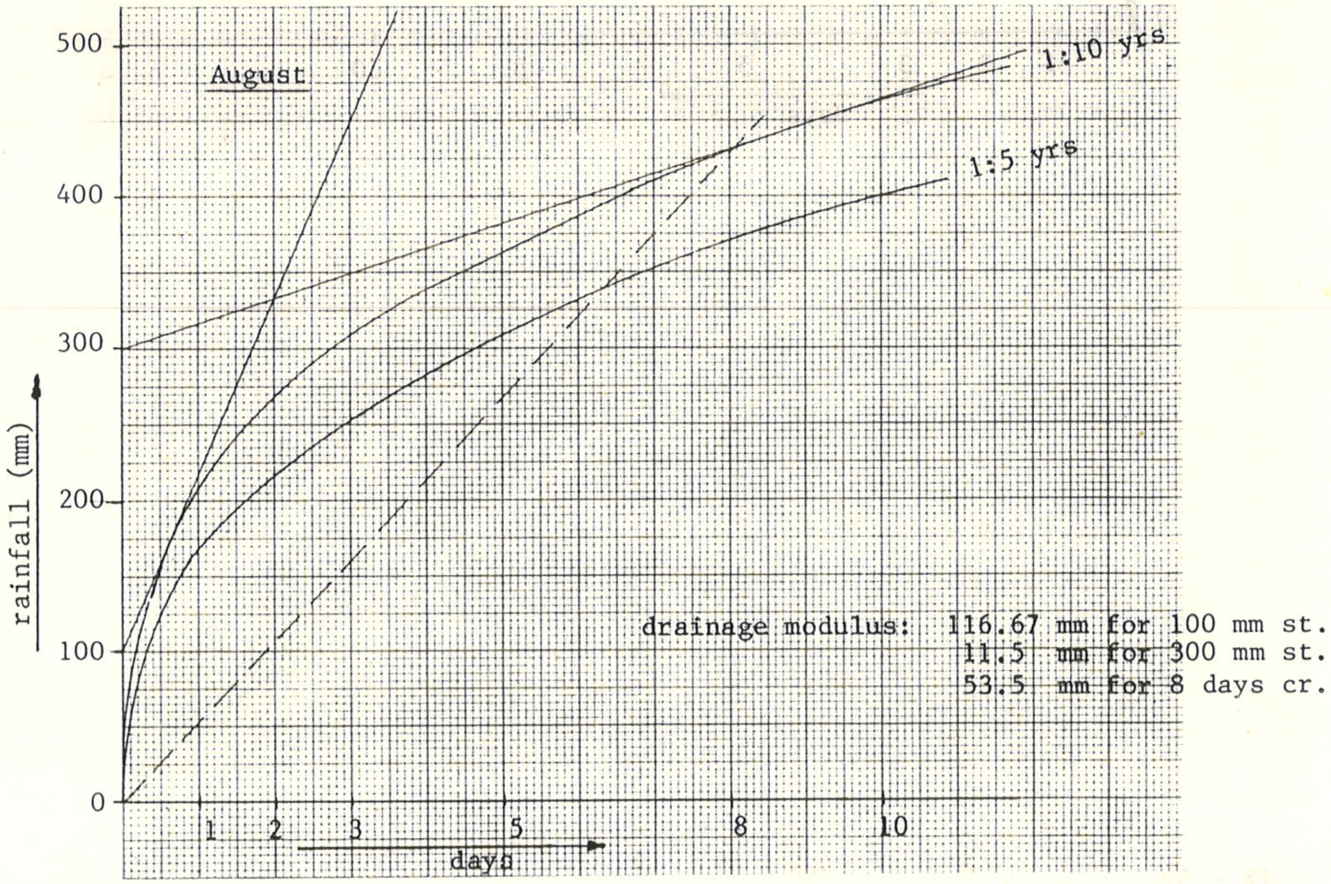

July 31 - August 14
T = 5.85 hrs.
m.a.st.l.
d.d.l.
waterlevel (m + p.w.d.)
2.00
1.50
1.00
0.50
1
2
3
6
12
August 15 - 29
T = 5.7 hrs.
m.a.st.l.
d.d.l.
waterlevel (m + p.w.d.)
2.00
1.50
1.00
0.50
1
2
3
6
12
time (hrs)

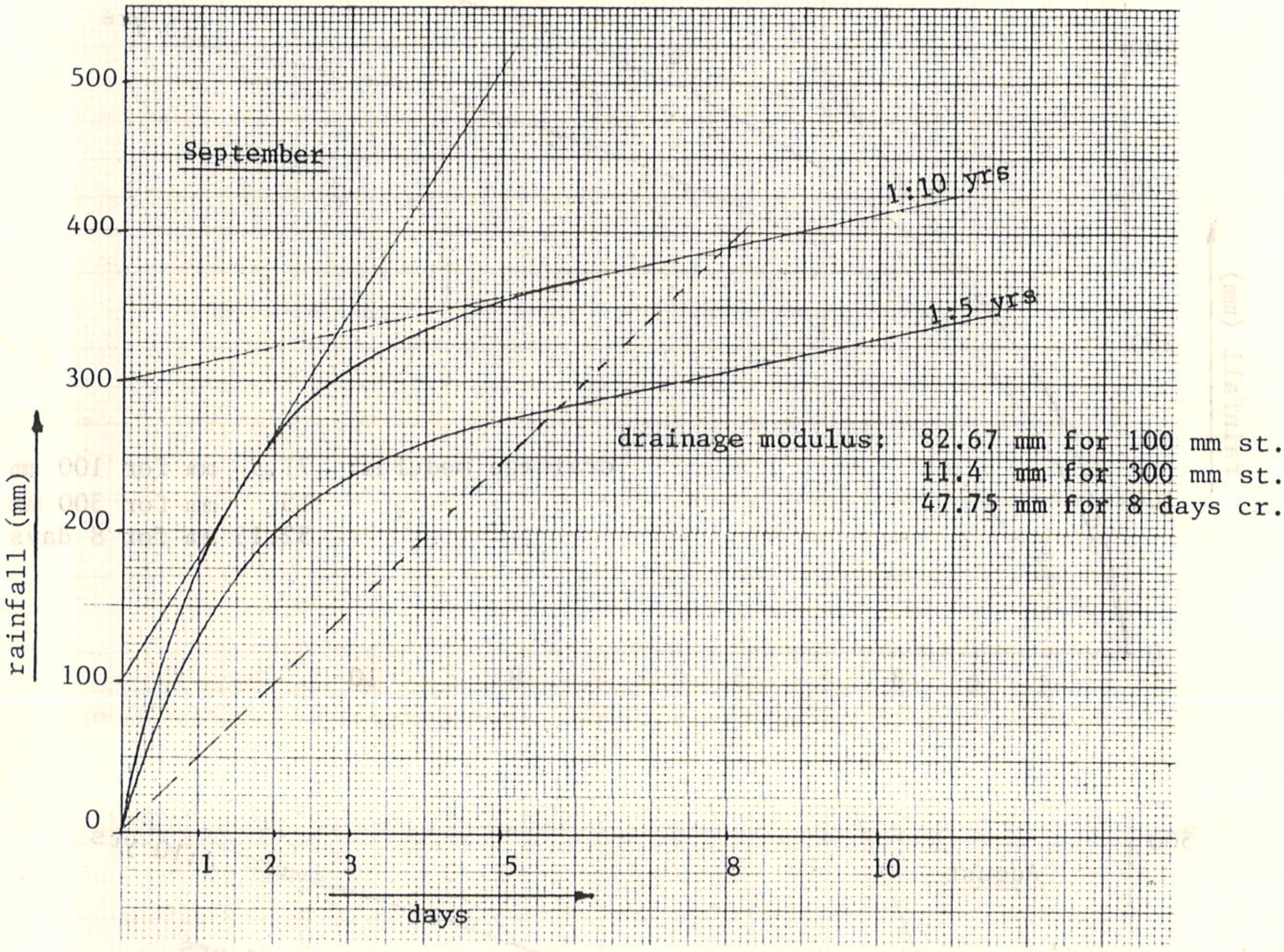
ANNEX IX - 11 RAINFALL DURATION CURVES DETERMINATION OF DRAINAGE MODULUS



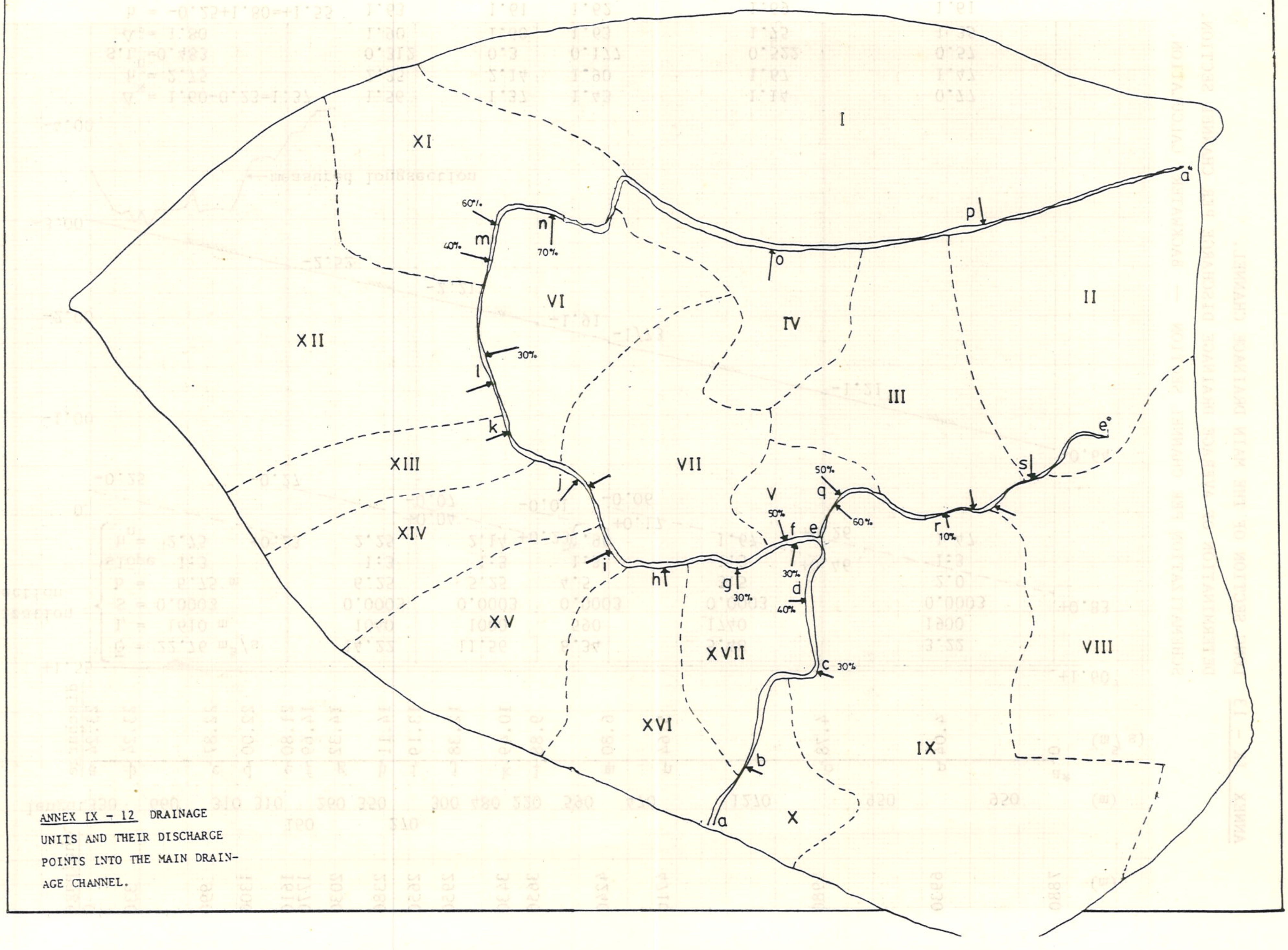

June
1:10 yrs
1:5 yrs
600
500
400
300
200
100
0
rainfall (mm)
1
2
3
5
8
10
days
drainage modulus: 98.75 mm for 100 mm st.
29.5 mm for 300 mm st.
65.5 mm for 8 days cr.


July
1:10 yrs
1:5 yrs
rainfall (mm)
0
100
200
300
400
500
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)
0
100
200
300
400
500
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.



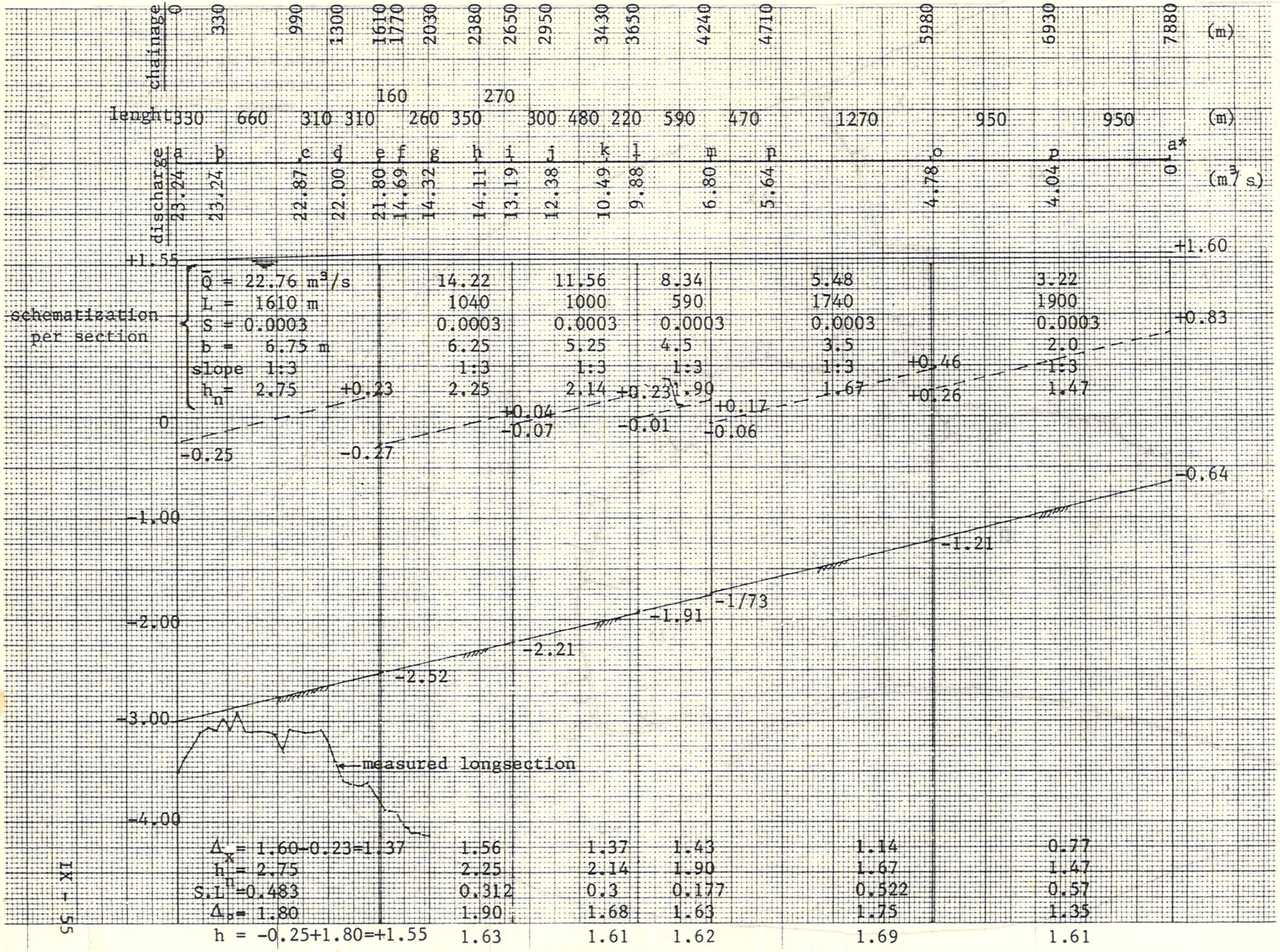
September
1:10 yrs
1:5 yrs
drainage modulus: 82.67 mm for 100 mm st.
11.4 mm for 300 mm st.
47.75 mm for 8 days cr.
rainfall (mm)
0
100
200
300
400
500
1
2
3
5
8
10
days



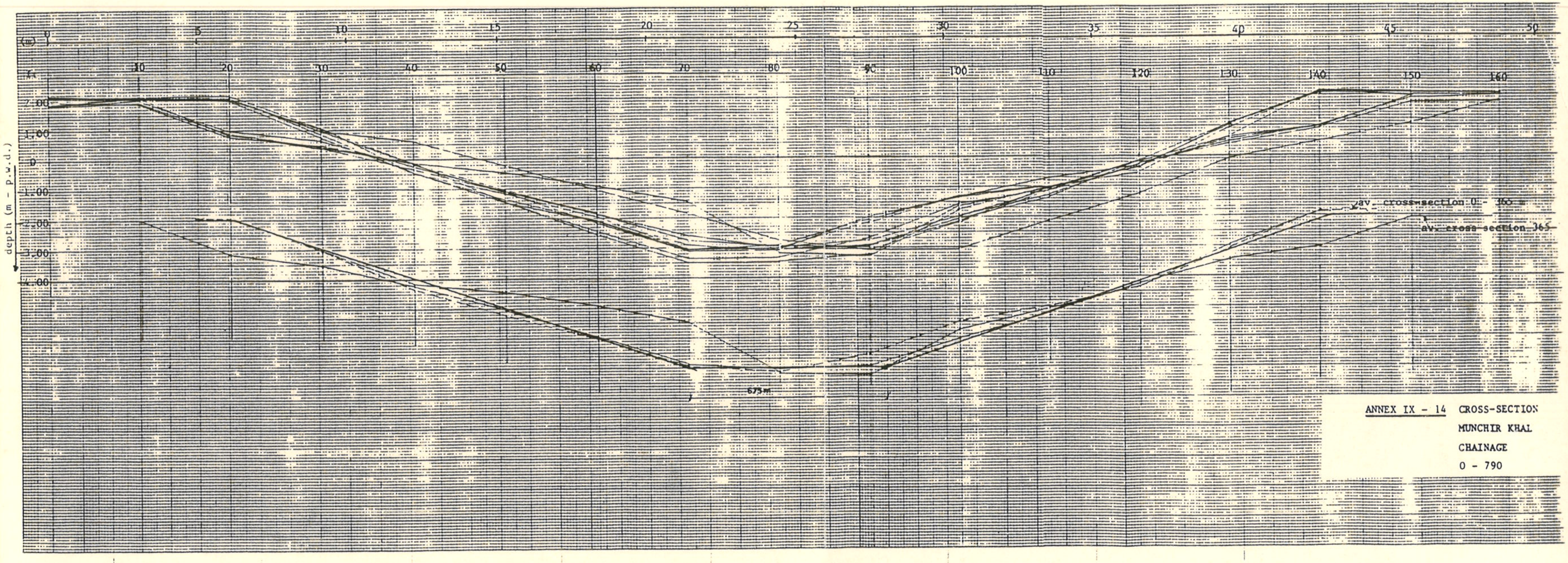
ANNEX IX - 12 DRAINAGE UNITS AND THEIR DISCHARGE POINTS INTO THE MAIN DRAINAGE CHANNEL.

IX-53.54

ANNEX IX - 13 LONG SECTION OF THE MAIN DRAINAGE CHANNEL.
DETERMINATION OF AVERAGE DRAINAGE DISCHARGE PER CHANNEL SECTION.
SCHEMATIZATION PER CHANNEL SECTION — BACKWATER CALCULATION.

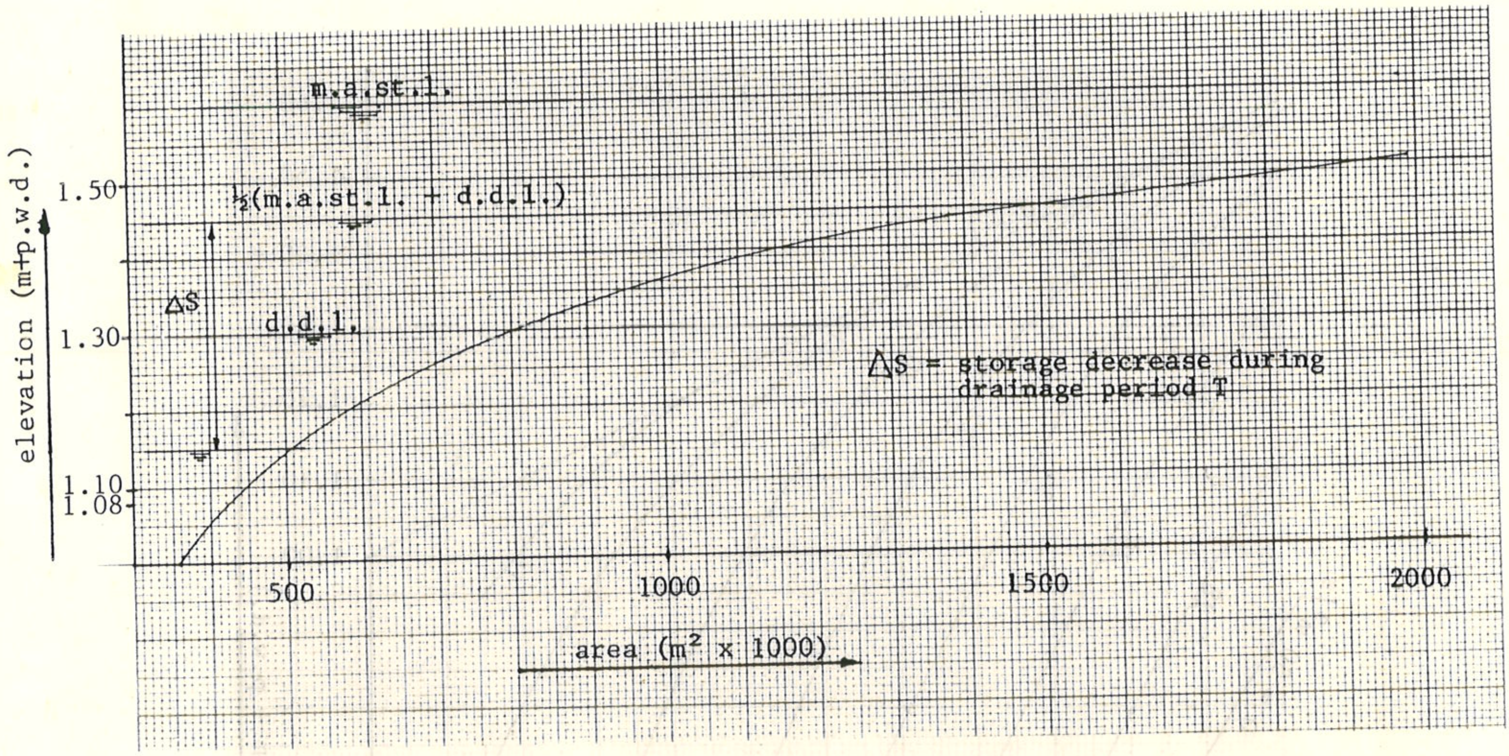


ANNEX IX - 14 CROSS SECTIONS MUNCHIR KHAL, CHAINAGE 0 - 790.

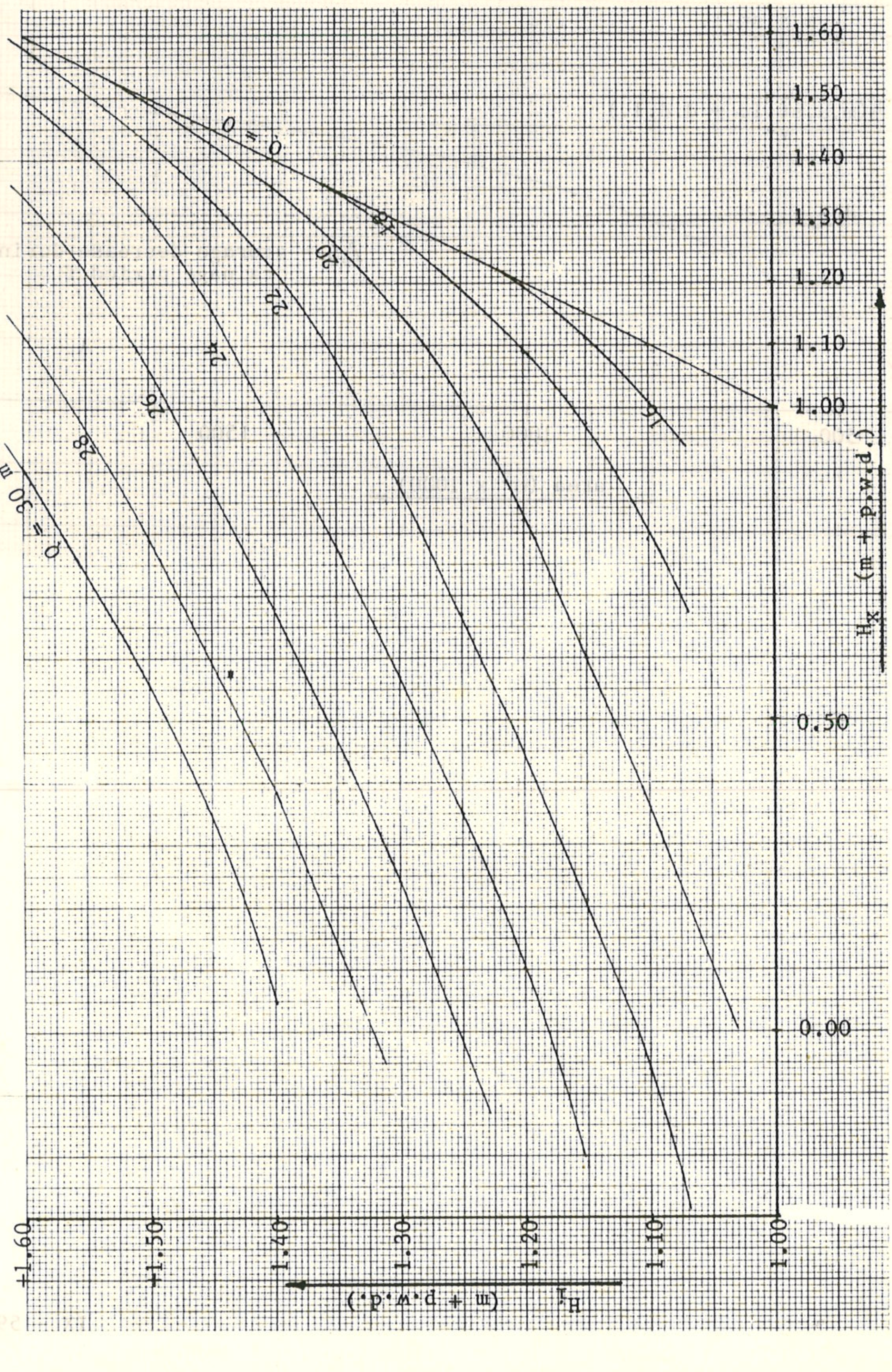


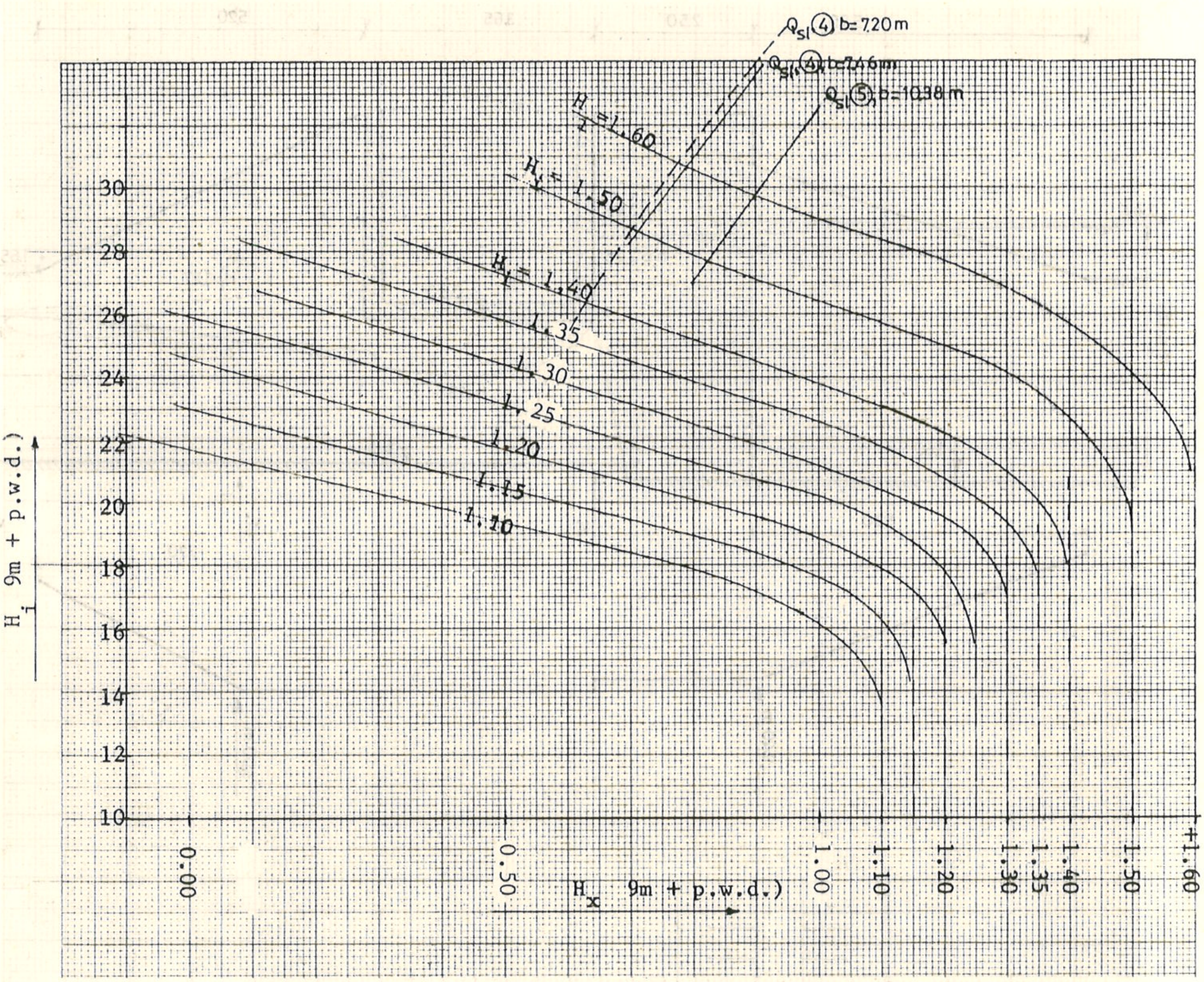
ANNEX IX - 14 CROSS-SECTION MUNCHIR KHAL CHAINAGE 0 - 790

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

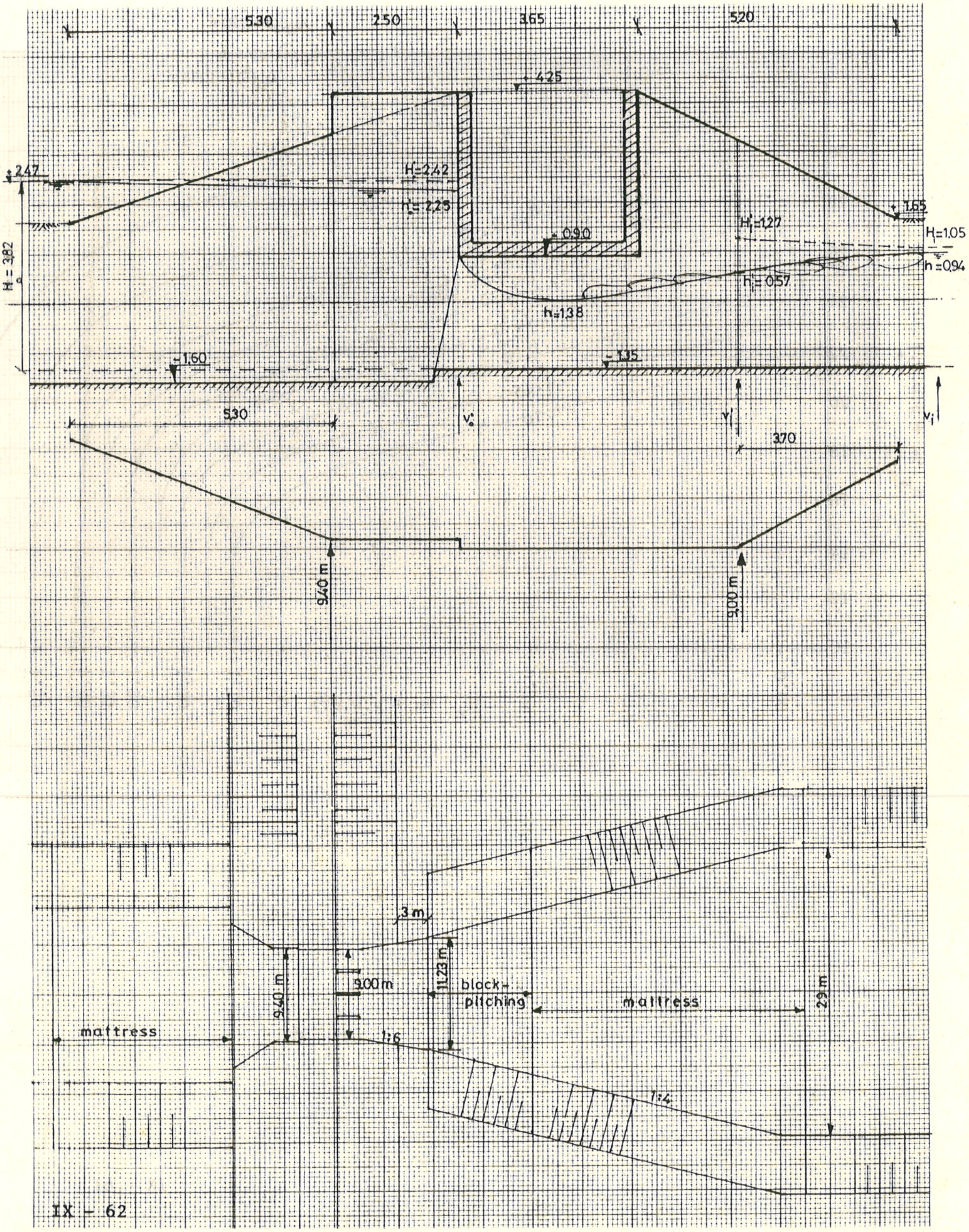


ANNEX IX - 16 DELIVERY CURVES FOR THE MAIN DRAINAGE CHANNEL




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
H_i 9m + p.w.d.)
H_x 9m + p.w.d.)
10
12
14
16
18
20
22
24
26
28
30
0.00
0.50
1.00
1.10
1.20
1.30
1.35
1.40
1.50
+1.60

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



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