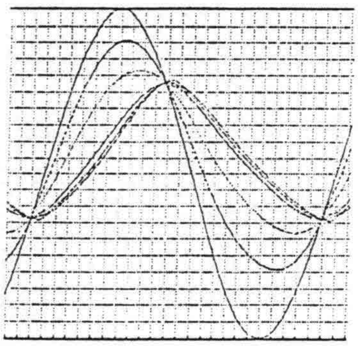
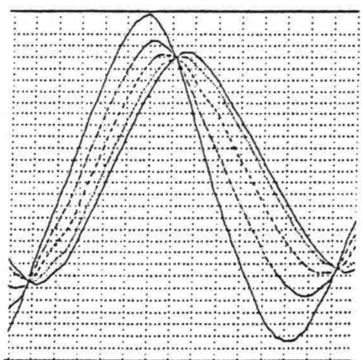
Ir. C. Verspuy

DUFLOW Simulations 1

Tidal Propagation in Networks





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Delft, July 1991

DUFLOW Simulations 1
Tidal Propagation in Networks

International Institute for Hydraulic and Environmental Engineering

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D U F L O W   S I M U L A T I O N S   I

Tidal propagation in networks

CONTENTS

1. INTRODUCTION

To illustrate the operational use of the micro-computer package DUFLOW for the simulation of one-dimensional unsteady flow in channel systems the tidal propagation in an irrigation system (IRRSYS) is considered.
The propagation of periodical shallow water waves in a network system (tidal waves in river systems, estuaries, etc. and also harbor oscillations - "seiches" with periods of 5 till 60 minutes) has to be studied frequently in hydraulic and environmental engineering. The open channel system IRRSYS is based on existing tidal irrigation systems which are constructed in a large number for example in Sumatra and Kalimantan in Indonesie (Ref. [1] and Ref. [2]). Analog simulations of tidal propagation in sea area, lagoons, swamps, etc. can be characterized by a relatively large internal storage area combined with a narrow tidal inlet. In the Netherlands the tidal propagation in the Scheldt Rhine Canal System (October 1986 until March 1987, see scheme in Figure 1 on page 2) can be characterized as analogous to the tidal propagation in IRRSYS, which will be illustrated in Chapter 5. The system IRRSYS is representative for many systems in other countries.

The DUFLOW analysis of IRRSYS in relation with analytic solutions (Ref. [2] and [3]) and with results of measurements (Ref. [4]) is very useful to study some **general** aspects of tidal propagation.

The mean information concerning IRRSYS in Chapter 2 is followed by the presentation of the DUFLOW simulation in Chapter 3 and some attention paid to the interpretation of the results in Chapter 4. Based on the information in chapters 2, 3 and 4 IRRSYS can be used for problem analysis. In Chapter 5 several suggestions are given to predict the influence of parameter variations and to check the personal predictions by DUFLOW calculations (if necessary) to get a good physical insight and to demonstrate how DUFLOW can be used as an important tool for problem analysis with valuable input and output facilities.

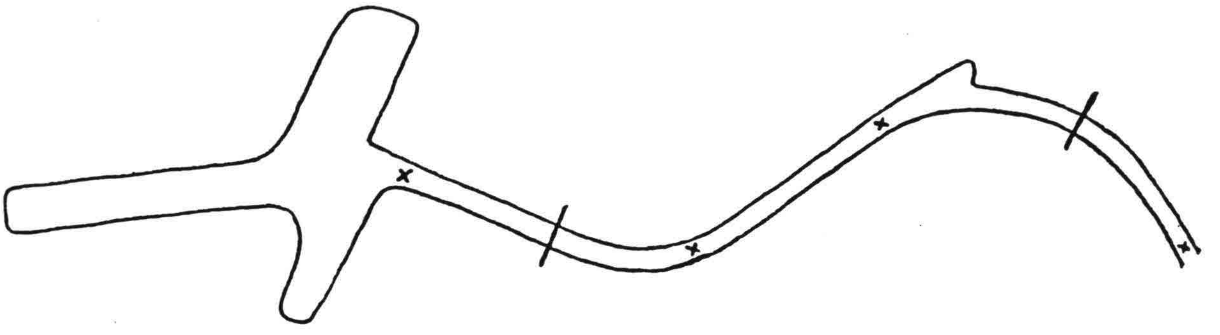


Figure 1

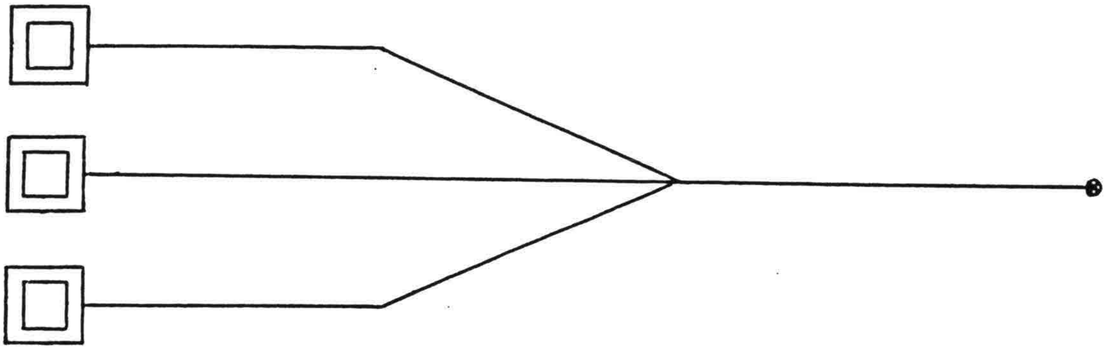


Figure 2

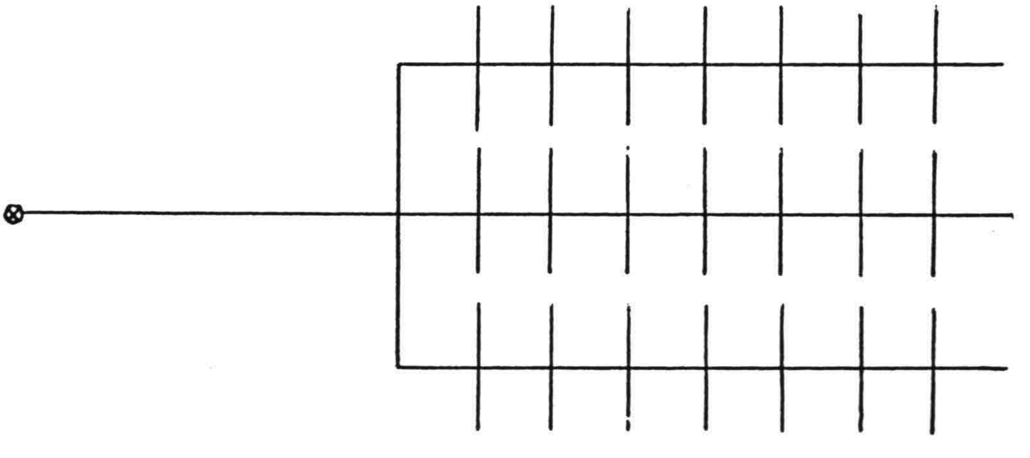


Figure 3

2. IRRSYS INFORMATION

The small scale irrigation system IRRSYS can easily be modelled and is illustrative for different existing systems (see the rough schemes in Figure 1,2 and 3). In Ref. [1] and [2] the problems concerning systems similar to Figure 2 and 3 are analyzed in detail. These systems have in common that a large number of tertiairy canals with a large storage area connected with a number of secondary canals which debouch into a primary canal. The primary canal debouch into a wide river or into an estuary in which the tidal level variations are (almost) independant of the discharge in and out the relatively small irrigation system.

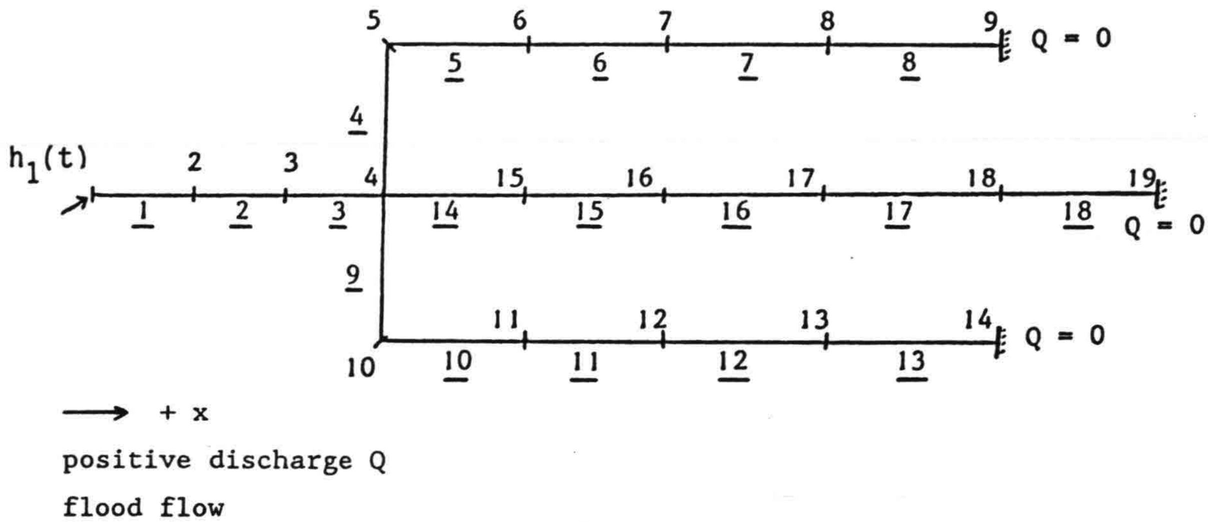


Figure 4 Layout of IRRSYS

The network drawn in Figure 4 consisting of 18 branches is less complicated than that of Figure 3 because the so called tertiary canals are absent. The total storage area of all the tertiary canals is devided and added to the storage area of the secondary canals 5 ... 8, 10 ... 13 and 14 ... 18.

The data of the 18 branches according to the layout of Figure 4 are given in Table 1 on page 4.

branch	length (m)	depth (m)	flow width (m)	storage width (m)
1	1000	3	10	10
2	1000	3	10	10
3	1000	3	10	10
4	2000	3	10	10
5	1000	3	10	80
6	1000	3	10	80
7	1000	3	10	80
8	1000	3	10	80
9	2000	3	10	10
10	1000	3	10	80
11	1000	3	10	80
12	1000	3	10	80
13	1000	3	10	80
14	1000	3	10	80
15	1000	3	10	80
16	1000	3	10	80
17	1000	3	10	80
18	1000	3	10	80

Table 1 Data of canal configuration IRRSYS

The layout of IRRSYS is rather simple :

- primary canal : sections 1, 2 and 3;
- secondary canal : sections 4 and 9 and
  sections 5, 6, 7, 8
  10, 11, 12, 13,
  14, 15, 16, 17 and 18.

The structure of the network of Figure 4 is defined by 18 sections and 19 nodes by means of MENU NETWORK of DUFLOW (see Ref. [5]). The positive direction which is choosen from node 1 to node 2 etc. agrees with the direction of positive discharges (flood flow).

In node 1 the water levels are supposed to be well known and the assumption is made that they are not influenced by the irrigation system. A diurnal tide is considered so the tidal period is 24 hours. At node 9, 14 and 19 the fresh water discharge is supposed to be zero so $Q_9(t) = Q_{14}(t) = Q_{19}(t) = 0$.

As a consequence the **boundary conditions** are rather simple :

- node 1 : $h_1(t) = \hat{h}_1 \cos(\omega t - \kappa)$
  in which $\hat{h}_1 = 1.00$ m , $\kappa = 0$ and
  $\omega = 2\pi/(24*3600) = 0.73 * 10^{-4}$ rad./s.

- node 9, node 14 and node 19 : $Q_9(t) = Q_{14}(t) = Q_{19}(t) = 0$.

Notes : (i) The time levels are given in minutes and measured from the start of the calculations (t = 0). The time steps during the calculations (in seconds) are also given in minutes (see MENU CONTROL DATA, Ref. [5]).

(ii) Start of calculation (t = 0) is defined with year-month-day and hour-minute (see CALCULATION DEFINITION, Ref. [5]) and the initial conditions at that time level in relation with the boundary conditions will cause oscillations in the system over e.g. some hours. The start date and time of writing data to the result file is defined with year-month-day and hour-minute. The end date and time of the simulation (and output) is defined in the same way.

The start time of writing data to the result file is t = 36 hours and the end time of the simulation is t = 72 hours so the results of the calculations which will be discussed in Chapter 3 regard one and a half period (36 hours) :

from t = 2160 minutes until t = 4320 minutes.

The time step and the output interval are equal to 10 minutes so for each time step the results for all nodes of the network are written in the output file (and available for interpretations).

3. DUFLOW SIMULATION IRRSYS

The data of IRRSYS described in Chapter 2 are defined in DUFLOW. The time step and the output interval equals 10 minutes and the following CALCULATION DEFINITIONS (see Ref. [5]) are added : Chezy/Manning : "Chezy" ; Froude : "Yes" ; Theta : "0.55" ; Iteration : "Yes" ; Gauss : "Yes" .
The channel friction is calculated with the formula of De Chezy with the same value of Chezy 's coefficient in both directions (eb and flood) : $C = 40\ m^{1/2}/s$.

The results concerning 18 sections and 19 nodes (see Figure 4) written to the result file IRRSYS.RES are presented by means of the useful output facilities of DUFLOW (see MENU OUTPUT, Ref. [5]).
The main results are inserted in the Annexes 1, 2, 3, ... 12, which will be described in this Chapter.

ANNEXES 1, 2 AND 3.

Annex 1. Water levels in the nodes 1, 2, 3, 4, 5, 6, 7, 8, 9.
Annex 2. Water levels in the nodes 1, 2, 3, 4, 10, 11, 12, 13, 14.
Annex 3. Water levels in the nodes 1, 2, 3, 4, 15, 16, 17, 18, 19.

These three annexes give the water level variation in time of the periodical solution of IRRSYS. The results in Annex 2 are equal to the results of Annex 1 (caused by the symmetry of the system) and almost equal to the results of Annex 3 (caused by the different dimensions of section 14 compared with those of section 4 and 9).
The tidal propagation according to Annex 1, 2 and 3 is characterized by a large dampening and large phase shift over the primary canals (section 1, 2 and 3) of the system. In the other sections of the system (sections 4 ... 18) the influence of acceleration and friction is very small as usual when the canal length is very small compared to the wave length in the system.
When the tidal range (the difference between maximum and minimum water level) at the boundary (node 1) is compared with the tidal range at nodes 4 or 9 we find that the "amplification factor" is about : (0.84 m)/(2.00 m) = 0.42.

This factor is very small related to the ratio of the canal length and the wave length the system ($L/\lambda \approx 1/50$). The small factor 0.42 is caused by the the combination of

- a considerable storage area of the secondary canals;
- the relatively shallow and narrow primary canal (sections 1, 2 and 3);
- a large tidal range (large friction via discharges).

(Note : The dimensions of IRRSYS are based on existing systems, see Chapter 1).

The total storage area of the sections 4 ... 18 is relatively large in relation to the cross-sectional area of the tidal inlet and this total storage area is also relatively large compared with the storage area of the sections 1, 2 and 3. As a consequence the velocities in the primary canal are relatively large and the discharges at nodes 1, 2, 3 and 4 have almost the same values : $Q_1(t) \approx Q_2(t) \approx Q_3(t) \approx Q_4(t)$.

ANNEXES 4, 5 and 6.

Annex 4. Discharge in section 1, 2, 3, 4, 5, 6, 7, 8.
Annex 5. Discharge in section 1, 2, 3, 9, 10, 11, 12, 13.
Annex 6. Discharge in section 1, 2, 3, 14, 15, 16, 17, 18.

The three annexes give the discharge through each of the cross-sections at the nodes of network IRRSYS as a function of time which can be drawn by means of the output facilities of DUFLOW (see OUTPUT DEFINITION : 1B, 2B, 3B, etc.).
Because the discharge is dominated by the large internal storage area and the almost constant internal water level fluctuations the phase shift between different functions Q(t) is relatively small. This aspect makes IRRSYS very useful for problem analysis as will be illustrated below and at most in Chapter 5.
The discharge in sections 1, 2 and 3 (of the primary canal) is almost constant in x-direction and can be estimated rather easily by means of calculations by hand.
In Table 2 the discharges in the primary canal is calculated by multiplying the estimated value of $\frac{\delta h}{\delta t}$ at node 4 and the total

storage area $F = 1.08 * 10^6\ m^2$ for two different time levels :

- maximum flood flow : t = 2830 minutes
- maximum eb flow : t = 3420 minutes

Time level in minutes	F in m^2	$\frac{\delta h}{\delta t}$ in m/s	Discharge Q in m^3/s
2820	$1.08 * 10^{-6}$	0.000031	33
3420	$1.08 * 10^{-6}$	- 0.000022	- 24

TABLE 2 Estimation of the discharge in the primary canal sections

The estimated values in Table 2 agree with the extreme values of the functions Q(t) at the nodes 1, 2 and 3 (see Annex 4).

Note : The small difference between the discharge at node 1 and the discharge at node 3 (at a time level t) can be estimated by multiplying the sum of the storage area ΔF_1 and ΔF_2 of two sections 1 and 2 and the value of $\frac{\delta h}{\delta t}$ at node 2 (at the time level t). More information concerning evaluation calculations which are used in this chapter is given in Appendix 1.

ANNEX 7 Water levels and discharge in section 1.

To understand the impulse and momentum balance concerning a section of a network system, the package DUFLOW offers the facility to draw two waterlevels (at the boundaries) in combination with the (mean) discharge of a section. For section 1 of the primary canal this information is drawn in Annex 7.

The results are used together with evaluation calculations (zie Appendix 1) by means of new DUFLOW facilities which are available with the PC-program ECDUFLOW (see Ref. [6]).

For time level t = 2830 minutes the calculations are given in the Table 3. At this time level maximum flood flow occurs in section 1 with $Q = 32.7\ m^3/s$ and a friction head loss of 0.206 m which is almost equal to the difference in water level $(h_2 - h_1)$ because the contribution of the local acceleration term is zero and the contribution of the advective term is small.

Note : On this time level the mean velocity at node 2 is (somewhat) LARGER than the mean velocity at node 1 and therefore the contribution of the advective term is NEGATIVE which is not a common situation as will be discussed in detail later on (see Annex 12).

program DUFLOW analysis - Details

CALCULATION TIME LEVEL 2830 minutes SECTION 1

L	length	=	1000.00 m
H1	level begin	=	0.965930 m
H2	level end	=	0.756930 m
Hav.	prev timestep	=	0.848605 m
Hav.	next timestep	=	0.872620 m
Q1	discharge begin	=	32.80500 m3/s
Q2	discharge end	=	32.60300 m3/s
Q	average disch.	=	32.70400 m3/s
Qav.	prev timestep	=	32.65250 m3/s
Qav.	next timestep	=	32.70600 m3/s
A1	area begin	=	39.65930 m^2
A2	area end	=	37.56930 m^2
A	average area	=	38.61430 m^2
R	hydr. radius	=	2.178393 m
SW	Storage width	=	10.00000 m
C	de Chezy coeff.	=	40.000 $m^{1/2}/s$
dt	time interval	=	600 sec
SW*L*dH/dt		=	0.2001 m3/s
Q1 - Q2		=	0.2020 m3/s

Friction term:
$$\frac{-Q*|Q|*L}{C^2*A^2*R} = -0.205802 \text{ m}$$

Acceleration term:
$$\frac{-L}{gA} * \frac{dQ}{dt} = -0.000118 \text{ m}$$

Advective term:
$$\frac{Q1^2/A1 - Q2^2/A2}{gA} = -0.003057 \text{ m}$$

SUM of the 3 terms = -0.208976 m
H2 - H1 = -0.209000 m

Table 3 Impulse and momentum balance, section 1, t = 2830 minutes.

Table 4 on the next page gives the results of the calculations for the time level t = 3300 minutes. At that time level (see Annex 7) there is an increasing eb flow in section 1. The contribution of the local acceleration term is only about + 0.0017 m and the contribution of the advective term is larger : + 0.0091 m.
The friction term is dominating again : + 0.229 m. The sum of the three terms agrees with the difference in water level.
In Table 5 at the next page the results of the calculations for time level t = 3600 minutes are given also.
From Annex 7 it can be learned that at time t = 3600 minutes the eb flow is decreasing. After the extreme value of the discharge the sign of the contribution of the local acceleration term have been changed and the value is about - 0.0016 m. The contribution of the advective term is positive (the absolute value is much larger than that of the local acceleration term): about + 0.023 m.

program DUFLOW — analysis - Details

CALCULATION TIME LEVEL 3300 minutes — SECTION 1

L	length	=	1000.00	m
H1	level begin	=	-0.216440	m
H2	level end	=	0.023141	m
Hav.	prev timestep	=	-0.060701	m
Hav.	next timestep	=	-0.132253	m
Q1	discharge begin	=	-24.13500	m3/s
Q2	discharge end	=	-23.53800	m3/s
Q	average disch.	=	-23.83650	m3/s
Qav.	prev timestep	=	-23.52050	m3/s
Qav.	next timestep	=	-24.10700	m3/s
A1	area begin	=	27.83560	m^2
A2	area end	=	30.23141	m^2
A	average area	=	29.03350	m^2
R	hydr. radius	=	1.836058	m
SW	Storage width	=	10.00000	m
C	de Chezy coeff.	=	40.000	$m^{½}/s$
dt	time interval	=	600	sec
SW*L*dH/dt		=	-0.5963	m3/s
Q1 - Q2		=	-0.5970	m3/s

Friction term:

$$\frac{-Q*|Q|*L}{C^2*A^2*R} = 0.229446 \text{ m}$$

Acceleration term:

$$\frac{-L}{gA} * \frac{dQ}{dt} = 0.001716 \text{ m}$$

Advective term:

$$\frac{Q1^2/A1-Q2^2/A2}{gA} = 0.009128 \text{ m}$$

SUM of the 3 terms	=	0.240290	m
H2 - H1	=	0.239581	m

Table 4 Impulse and momentum balance, section 1, t = 3300 minutes.

program DUFLOW — analysis - Details

CALCULATION TIME LEVEL 3600 minutes — SECTION 1

L	length	=	1000.00	m
H1	level begin	=	-0.999050	m
H2	level end	=	-0.548400	m
Hav.	prev timestep	=	-0.768005	m
Hav.	next timestep	=	-0.778095	m
Q1	discharge begin	=	-22.94000	m3/s
Q2	discharge end	=	-22.85500	m3/s
Q	average disch.	=	-22.89750	m3/s
Qav.	prev timestep	=	-23.10850	m3/s
Qav.	next timestep	=	-22.67850	m3/s
A1	area begin	=	20.00950	m^2
A2	area end	=	24.51600	m^2
A	average area	=	22.26275	m^2
R	hydr. radius	=	1.537036	m
SW	Storage width	=	10.00000	m
C	de Chezy coeff.	=	40.000	$m^{½}/s$
dt	time interval	=	600	sec
SW*L*dH/dt		=	-0.0841	m3/s
Q1 - Q2		=	-0.0850	m3/s

Friction term:

$$\frac{-Q*|Q|*L}{C^2*A^2*R} = 0.430145 \text{ m}$$

Acceleration term:

$$\frac{-L}{gA} * \frac{dQ}{dt} = -0.001641 \text{ m}$$

Advective term:

$$\frac{Q1^2/A1-Q2^2/A2}{gA} = 0.022863 \text{ m}$$

SUM of the 3 terms	=	0.451367	m
H2 - H1	=	0.450650	m

Table 5 Impulse and momentum balance, section 1, t = 3600 minutes.

The contribution of the friction term is dominating and the sum of the three terms agrees with the difference in waterlevel $h_2 - h_1$ which equals about 0.45 m at this time level.

In Table 6 the water levels and the discharges at the boundaries of section 1 are presented from the DUFLOW results by means of the PC-program ECDUFLOW. This Table 6 illustrates another new facility (see Ref. [6]) which can be used in relation with the graphical time related output of DUFLOW demonstrated in Annex 7.

program DUFLOW Analysis - Table

Analysis section 1 Page 18

Time min.	H1 (m) H2 (m)	Q1 (m) Q2 (m)	A1 (m²) A2 (m²)	R (m)	C (m½/s)	FRICTION (m)	ACCELERA-TION (m)	CONVEC-TION (m)
3530	-0.954	-24.15	20.46	1.56	40.0	0.4446	-0.0012	0.0256
	-0.486	-23.93	25.14					
3540	-0.966	-23.97	20.34	1.56	40.0	0.4447	-0.0013	0.0254
	-0.498	-23.77	25.02					
3550	-0.976	-23.78	20.24	1.55	40.0	0.4438	-0.0014	0.0251
	-0.510	-23.60	24.90					
3560	-0.985	-23.58	20.15	1.55	40.0	0.4420	-0.0014	0.0247
	-0.520	-23.43	24.80					
3570	-0.991	-23.38	20.09	1.54	40.0	0.4390	-0.0015	0.0242
	-0.530	-23.25	24.70					
3580	-0.996	-23.16	20.04	1.54	40.0	0.4351	-0.0016	0.0236
	-0.540	-23.06	24.60					
3590	-0.999	-22.94	20.01	1.54	40.0	0.4301	-0.0016	0.0229
	-0.548	-22.85	24.52					
3600	-1.000	-22.71	20.00	1.54	40.0	0.4242	-0.0017	0.0220
	-0.556	-22.65	24.44					

Table 6 Analysis-Table for t = 3530, 3540, ... 3600 minutes.

The performance of the water levels and discharges of a section (in this case section 1 of the network) can be related to the performance of the terms of the differential equations. In Table 6 the results for eight successive time levels (including the time level of Table 5) are given. It is possible to present this information for all the time levels of a tidal period.

In Figure 5 on the next page the performance of the four terms of the impulse and momentum balance is drawn from time level t = 2160 minutes until t = 4320 minutes. The time levels of Table 3, 4 and 5 are indicated in this Figure 5.

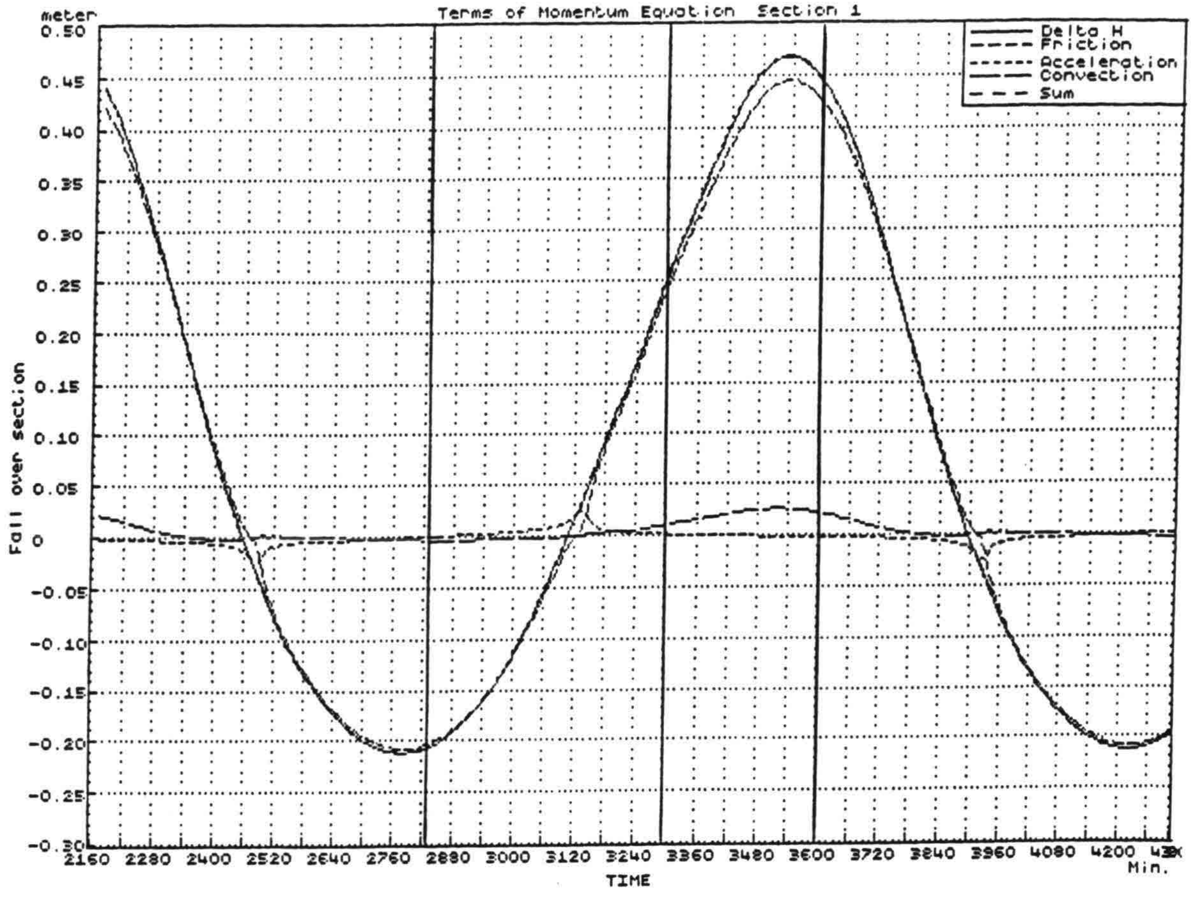


Figure 5 Analysis-graph, terms of impuls and momentum balance.

Figure 5 and the calculations illustrated in Table 3, 4 and 5 make it possible to get a complete view of the balance equations over a whole period. The new DUFLOW facilities by means of ECDUFLOW make it easy to analyse the DUFLOW results in more detail by several graphs (different combinations of terms are possible) of the terms of the differential equations solved by DUFLOW.
The other information of the DUFLOW result file concerning IRRSYS can be presented on the same way. Instead of Annex 7 concerning section 1 the functions h(t) and Q(t) for section 2 (see Annex 8) and for section 3 (see Annex 9) can be analyzed on the same way. The user is invited to analyze section 2 and 3 and to compare these sections with section 1. An anlysis of e.g. section 4 will learn that in this system IRRSYS the friction term in the secondary canals is relatively small (see also Annex 1).

Note : Evaluation calculations according to Appendix 1 are very important to analyse the results of calculations and measurements and to calibrate a model.

The new PC-program ECDUFLOW (see Ref. [6]) is an important support to analyse DUFLOW results by means of calculations and graphical illustrations.

The user is invited to finish the evaluation check and to combine the NUMERICAL results with the GRAPHICAL results of DUFLOW.

ANNEXES 10, 11 AND 12.

For section 1 the mean velocity as a function of time is drawn on Annex 10. The influence of the non-linear character of the system is illustrated by the written information in this graph. The duration of the eb flow is about two hours larger than the duration of the flood flow. The absolute value of the maximum eb flow velocity is much larger than the maximum flood velocity (see Annex 10).

For sections 2, 3, ... etc. similar graphs can be drawn by means of MENU TIME RELATED OUTPUT (see Ref. [5]).
With MENU SPACE RELATED OUTPUT different (combinations of) results can be drawn (see Ref. [5]). In this report only one application of this DUFLOW facility is added in Annex 12 (see below).

Annex 11 give the mean velocities in the sections 1, 2, 3, 4, 5, 6, 7 and 8. From this Annex 11 we learn that the flood velocities in section 3 are larger than the flood velocities in section 1 (and 2). Caused by the relatively large water level differences in the primary canal the cross-section area during flood flow is decreasing strongly in the positive x-direction (downstream). The almost constant (slightly decreasing) discharge and the decreasing cross-section area give an increasing velocity in sections 2 and 3. To illustrate this aspect Annex 12 is added.
During eb flow the water level is decreasing and the mean velocity (negative, eb flow) is increasing in flow direction.

4. INTERPRETATION IRRSYS RESULTS

Some special aspects of the IRRSYS results will be discussed briefly in this Capter 4. The water levels in the system IRRSYS presented in Annex 1, 2 and 3 illustrate three general aspects of tidal propagation :

- distortion of the sinusoidal wave
- the mean water level gradient
- the dampening of the tidal range.

Starting with a sinusoidal boundary condition $h_1(t)$ at node 1 it is obvious that the functions h(t) and Q(t) at different nodes of the network system are deformed that means that these functions are not pure sinusiodal functions in time. The deformation or distortion of sinusiodal functions is well known from tidal analysis (standard determination of tidal constituents from tidal data).
Distortion of sinusoidal waves during their propagation in shallow waters is caused by the non-linear character of the differential equations which can be proved by analytic approach. More generally speaking all the non-linear influences in a tidal system cause tidal constituents with higher frequencies and also components which are constant in time (see Ref. [3]).
Considering the water level variation at node 4 and at node 1 the difference of the mean water level is about 0.16 m. The mean water level inside is much higher than the mean water level at the boundary.
The tidal propagation in a deep prismatic canal closed at one end and with a length of 9 km can easily be analysed. The water level will be almost constant in x-direction ($\frac{\delta h}{\delta x} \approx 0$) so the amplitude (h) and the phase angle ($-\phi$) of a sinusoidal function h(t) will almost be constant in the canal. In that case the discharge Q(t) with a phase angle called $-\kappa$ will be zero all over the canal when the water level reaches its maximum or minimum. So the phase shift ($\phi - \kappa$) will be 90^0 in that case. The mean water level gradient in this case equals zero.
The influence of friction in a system like IRRSYS will cause a much smaller difference between ϕ and κ. The flood flow into the system requires less energy than the the outflow out of the system. Thus a mean water level gradient develops.
In the system IRRSYS the influence of the friction in the primary canal is large which is demonstrated by a large mean water level gradient and by a large dampening of the tidal range.

The ratio between the tidal range at node 4 and the tidal range at the boundary (node 1) is called "the amplification factor" and can be estimated from the results on Annex 1 , which gives:

$$0.84/2.00 = 0.42 .$$

In Appendix 2 the last analytic solution gives an estimation of this amplification factor of about 0.41 .
Other analytic solutions by means of the harmonic methode give almost the same figure which proves that for IRRSYS even a rough model gives a good insight concerning this factor. This aspect is caused by the physical properties of the system : friction dominates in the primary canal (sections 1, 2 and 3) while storage dominates in all the other sections of the network.

5. PROBLEM ANALYSIS DUFLOW - IRRSYS

The insight based on the information in Chapters 3 and 4 will increase by paying attention to parameter variations. In this chapter several suggestions are given to change one or more parameters in the system IRRSYS.
The central question is: "What will be changed concerning the IRRSYS results when a specific change in the system takes place ?"
Considering the analytic solutions in Appendix 2 and the interpretations of the results in Chapters 3 and 4 it is possible to give a PREDICTION of the influences.
The general procedure which have to be followed is given in the following Figure 6 :

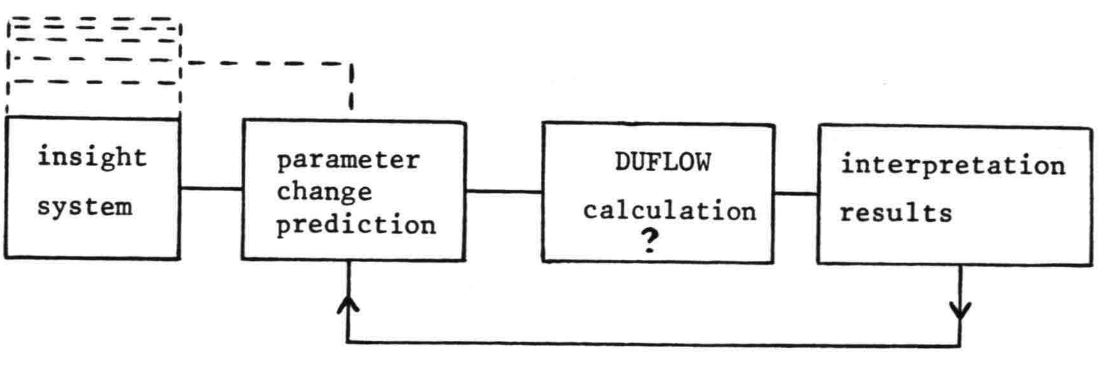


Figure 6 Procedure DUFLOW-IRRSYS analysis.

The FIRST suggestion is to DECREASE the internal storage area up to about 70 % by deleting the central secondary canal (see Figure 7). The prediction can be based on the large influence of the internal storage area (see analytic solution; the discharges in the primary canal will decrease) less dampening will occur so the tidal range at points 4, 5, ... 14 will INCREASE : amplification factor 0.7 ?

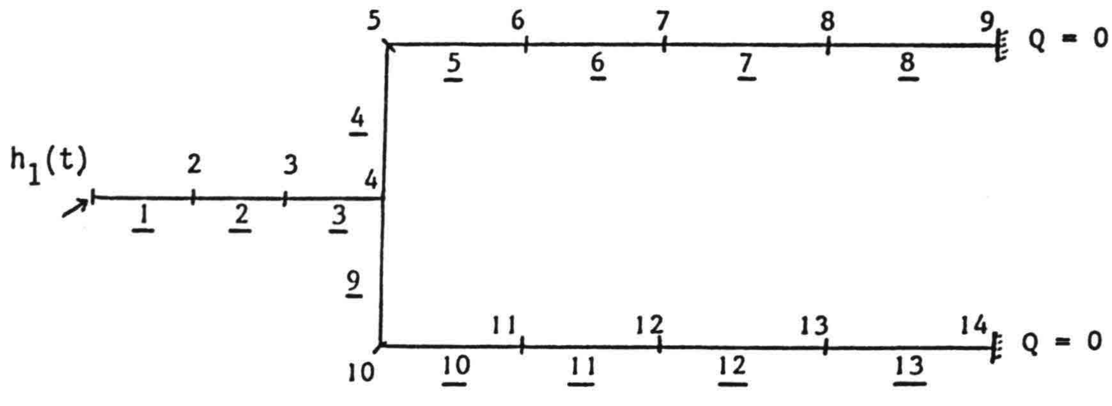


Figure 7 First suggestion

The question is whether the maximum discharge will be 22 m^3/s (instead of 33 m^3/s) or more ?
A rather good prediction is possible by an iterative use of the analytic solution. A new DUFLOW calculation is useful.
The new results will learn that the discharges will not decrease very much caused by an amplification factor of about 0.6.
The DUFLOW solution for the symmetrical new problem can be evaluated in a way similar to that described in Chapter 3.

The SECOND suggestion is to DECREASE the internal storage area up to about 30 % of the original value by deleting one branche of the network in Figure 7: e.g. deleting the sections 9 ... 14.
The user is invited to follow the whole procedure of Figure 6 : assumptions/insight, prediction, DUFLOW–results, interpretations.

Starting from the original network system IRRSYS a lot of similar suggestions concerning :

- the flow width of secondary (or tertiary) canals;
- the mean depth of ,, ,, ,, ,, ;
- Chezy 's coefficient of the secondary canals ;
- increasing internal storage area by 5 % ;

are possible and after a rather good prediction the question whether a DUFLOW calculation is neccesary or not has to be answered. It is not useful of course to make new calculations when (with a good prediction) no influences have to be expected.

Another interesting suggestion is to change the period of the tidal fluctuations at node 1 (boundary condition; semi diurnal instead of diurnal tide) with a factor 1/2.
The user is invited to follow the whole procedure exactly.

Other suggestions concern the properties of the primary canal :

- the flow width of section 1 and/or 2 and/or 3 ;
- the storage width of sections 1, 2 and 3 ;
- Chezy 's coefficient in sections 1 ... 3 ;
- the bottom level in sections 1 ... 3 ; etc.

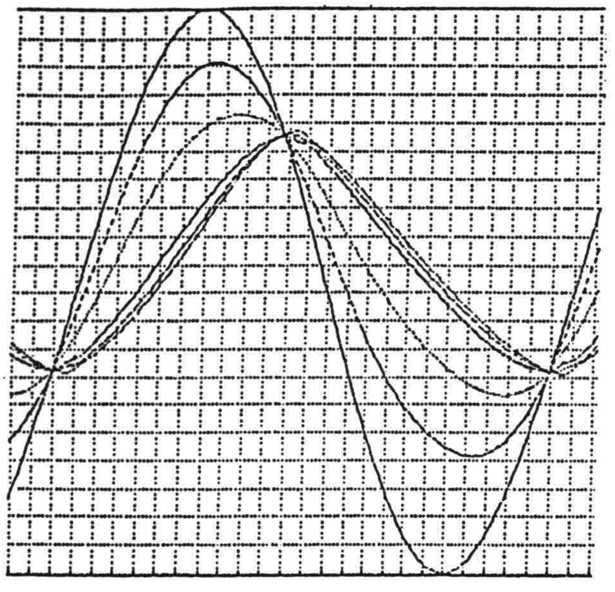
A last suggestion wich makes it useful to follow the entire procedure of Figure 6 is given here by asking the question :

"What will be the influence of a decrease of the section length of the sections 1, 2 and 3 by a factor 1/2 ?"

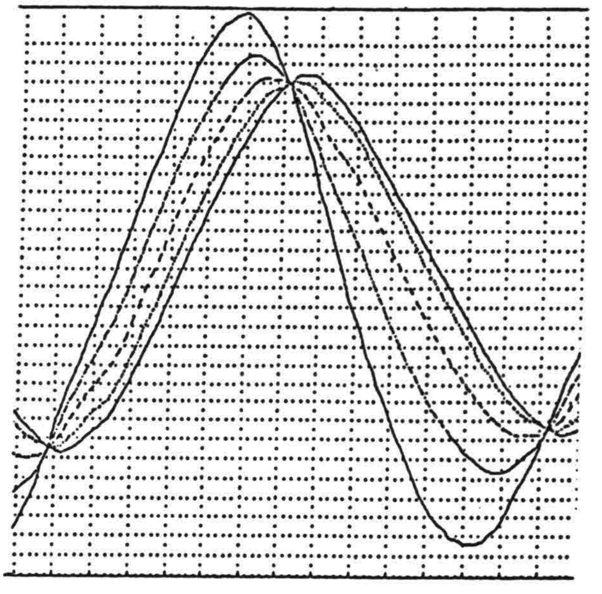
After a good prediction of this influence a DUFLOW calculation is useful to check this prediction and to evaluate the DUFLOW results similar to the interpretation in Chapter 3 and 4.

A final remark concerns the analog simulation of a system in the Netherlands (Ref. [4]) with a DUFLOW model (in 1989), calibrated by means of detailed measurements of 1987, and with maximum discharges in and out the tidal inlet of about 2000 m^3/s , caused by another length scale AND another time scale (semi-diurnal tide).

In that tidal system the amplification factor (see Appendix 2) was "only" 0.6, so the conclusion is that the IRRSYS system based on existing systems is even more spectacular which is illustrated in Figure 8 .



8.a.



8.b.

Figure 8 Illustration IRRSYS - Scheldt Rhine Canal System (1987)

Figure 8.a. gives the characteristic information concerning the water levels (from Annex 1, but without scales) in the IRRSYS network system which is analyzed in this report.
Figure 8.b. gives the similar information concerning the water levels (also without scales, only as an illustration) from the DUFLOW results described in detail in Ref. [4].
From this illustration the physical similarity of the systems will be clear enough.

Delft, July 1991
Ir. C. Verspuy

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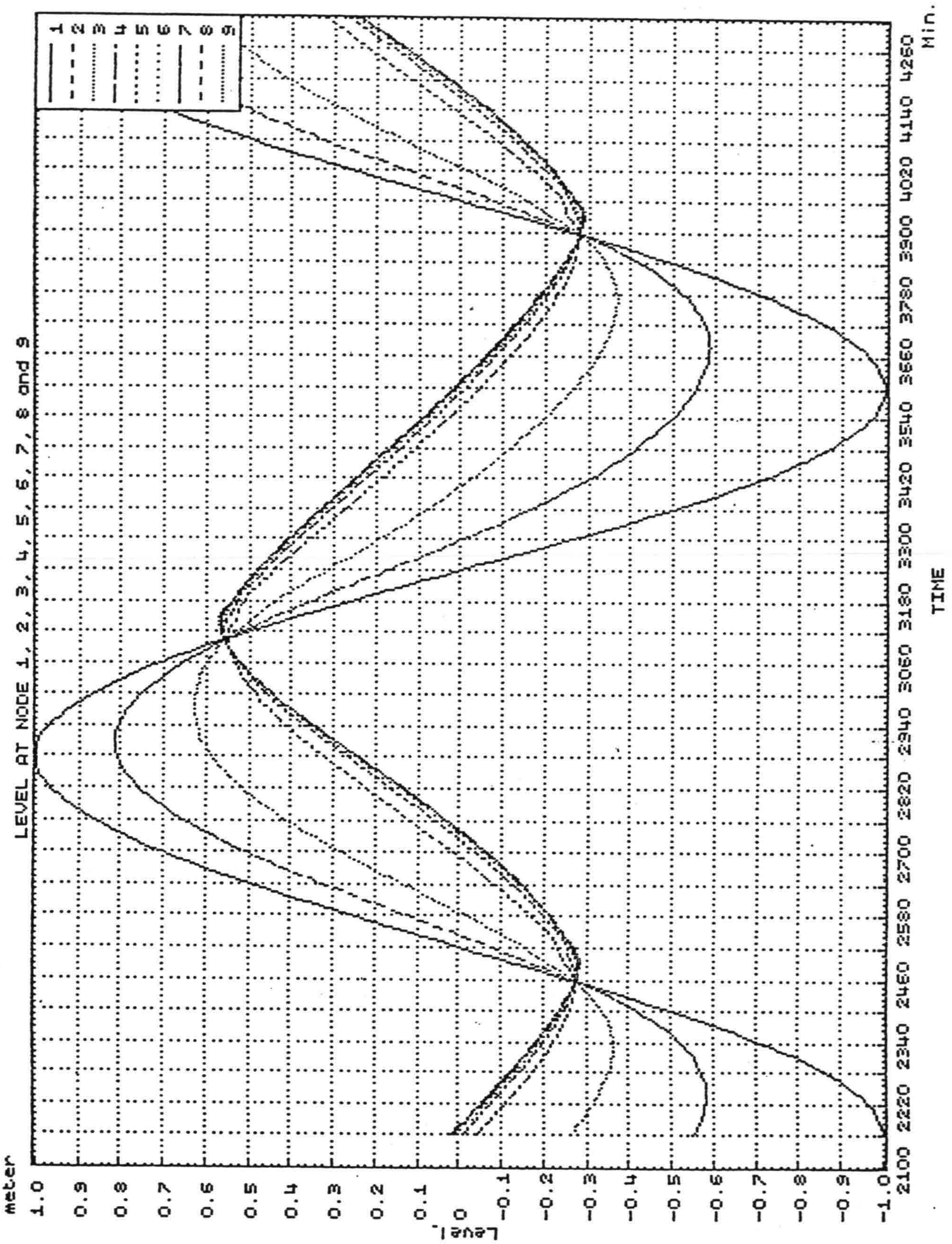
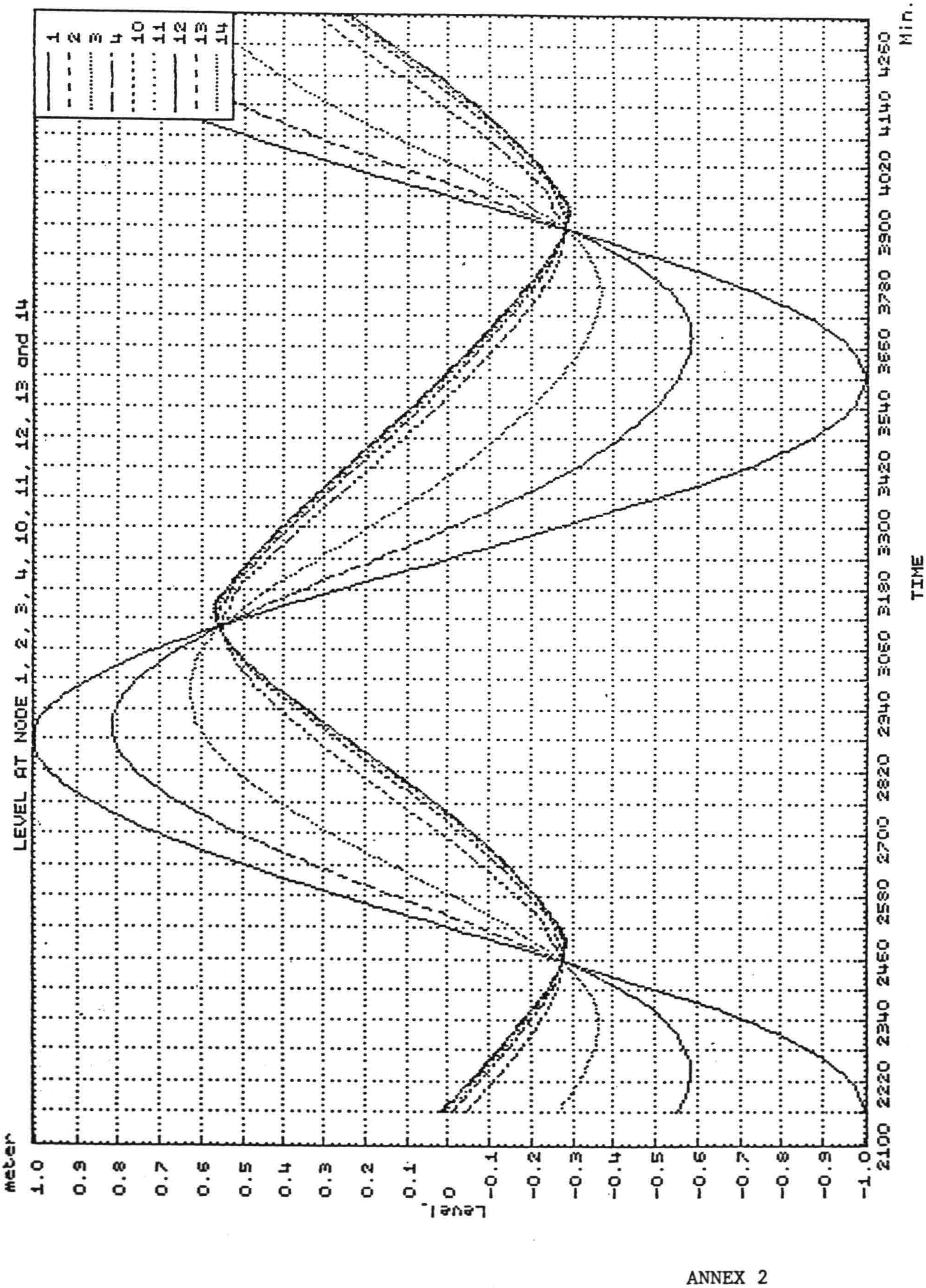
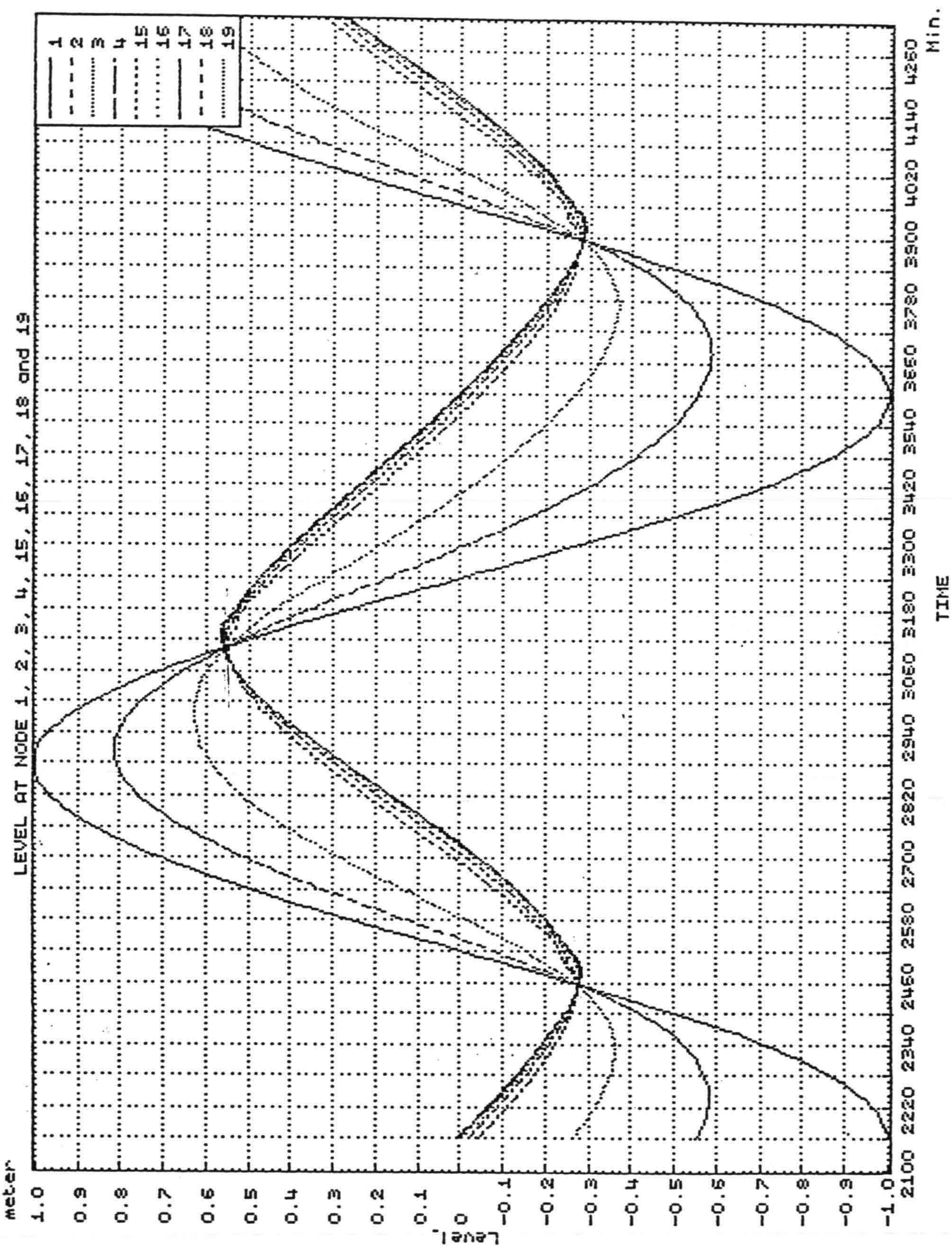

LEVEL AT NODE 1, 2, 3, 4, 5, 6, 7, 8 and 9
meter
Level
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
TIME
Min.
2100
2220
2340
2460
2580
2700
2820
2940
3060
3180
3300
3420
3540
3660
3780
3900
4020
4140
4260
1
2
3
4
5
6
7
8
9

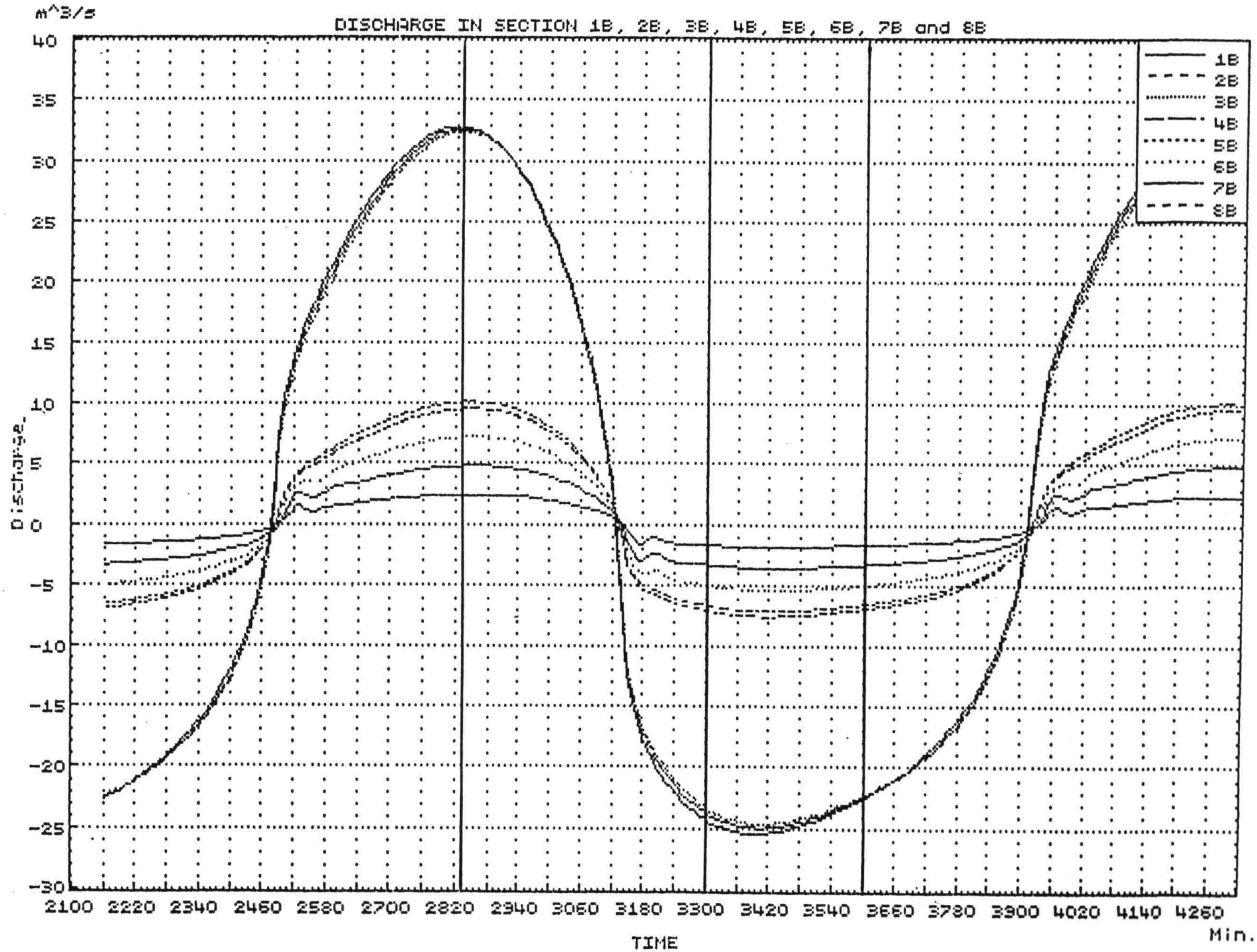
ANNEX 1


LEVEL AT NODE 1, 2, 3, 4, 10, 11, 12, 13 and 14
1
2
3
4
10
11
12
13
14
meter
Level
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
2100
2220
2340
2460
2580
2700
2820
2940
3060
3180
3300
3420
3540
3660
3780
3900
4020
4140
4260
TIME
Min.

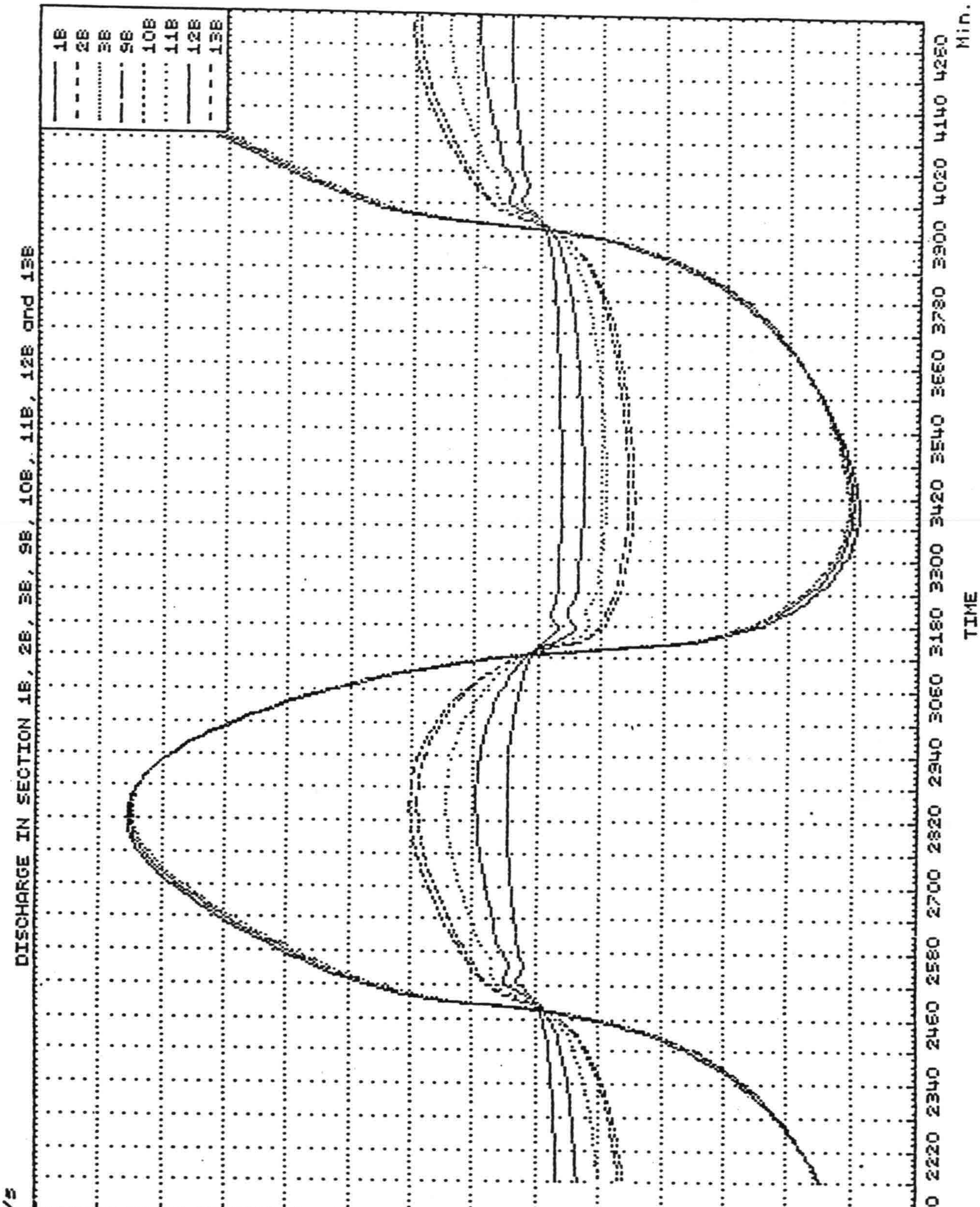

LEVEL AT NODE 1, 2, 3, 4, 15, 16, 17, 18 and 19
meter
Level
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
2100 2220 2340 2460 2580 2700 2820 2940 3060 3180 3300 3420 3540 3660 3780 3900 4020 4140 4260
TIME
Min.
1
2
3
4
15
16
17
18
19

ANNEX 3

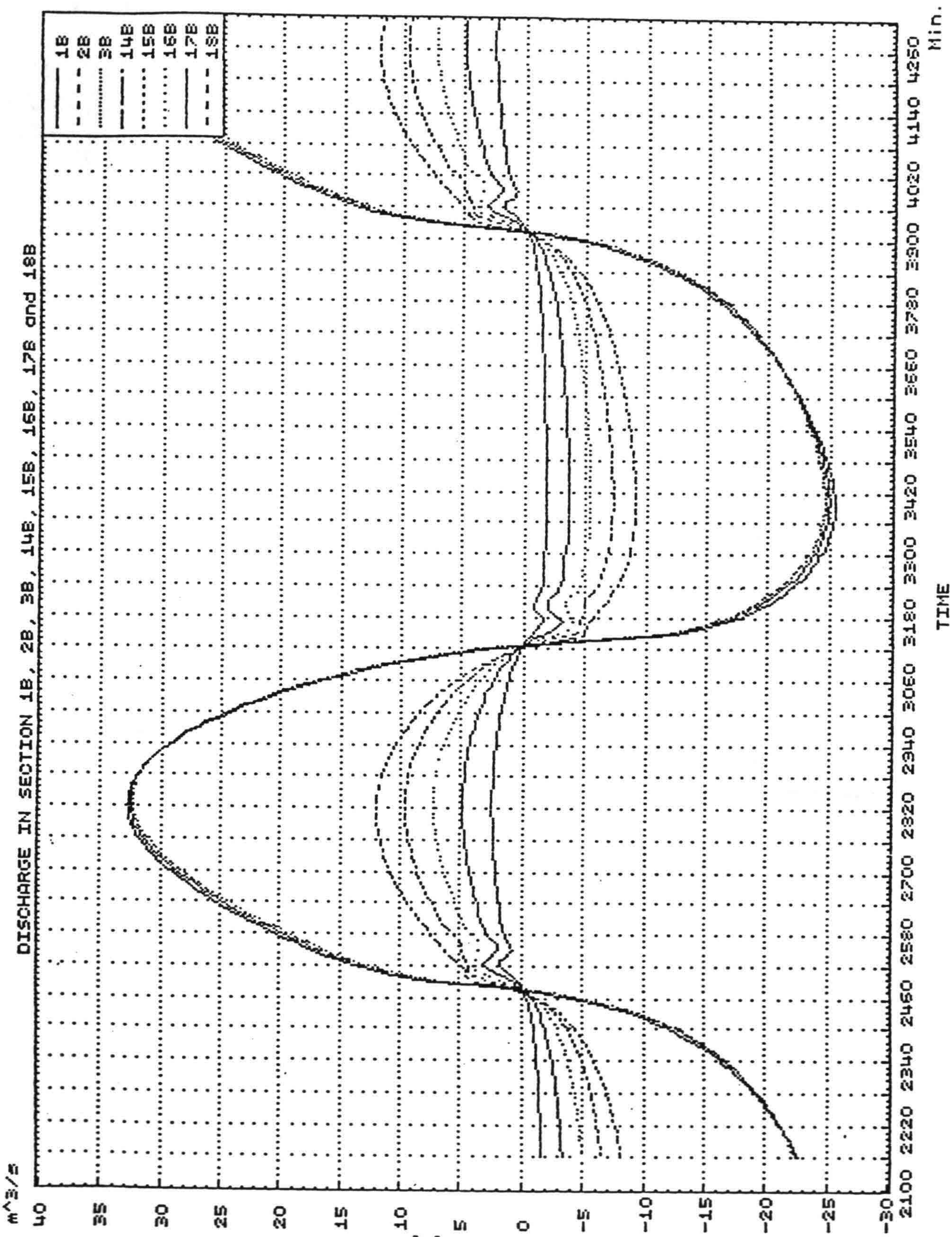
ANNEX 4



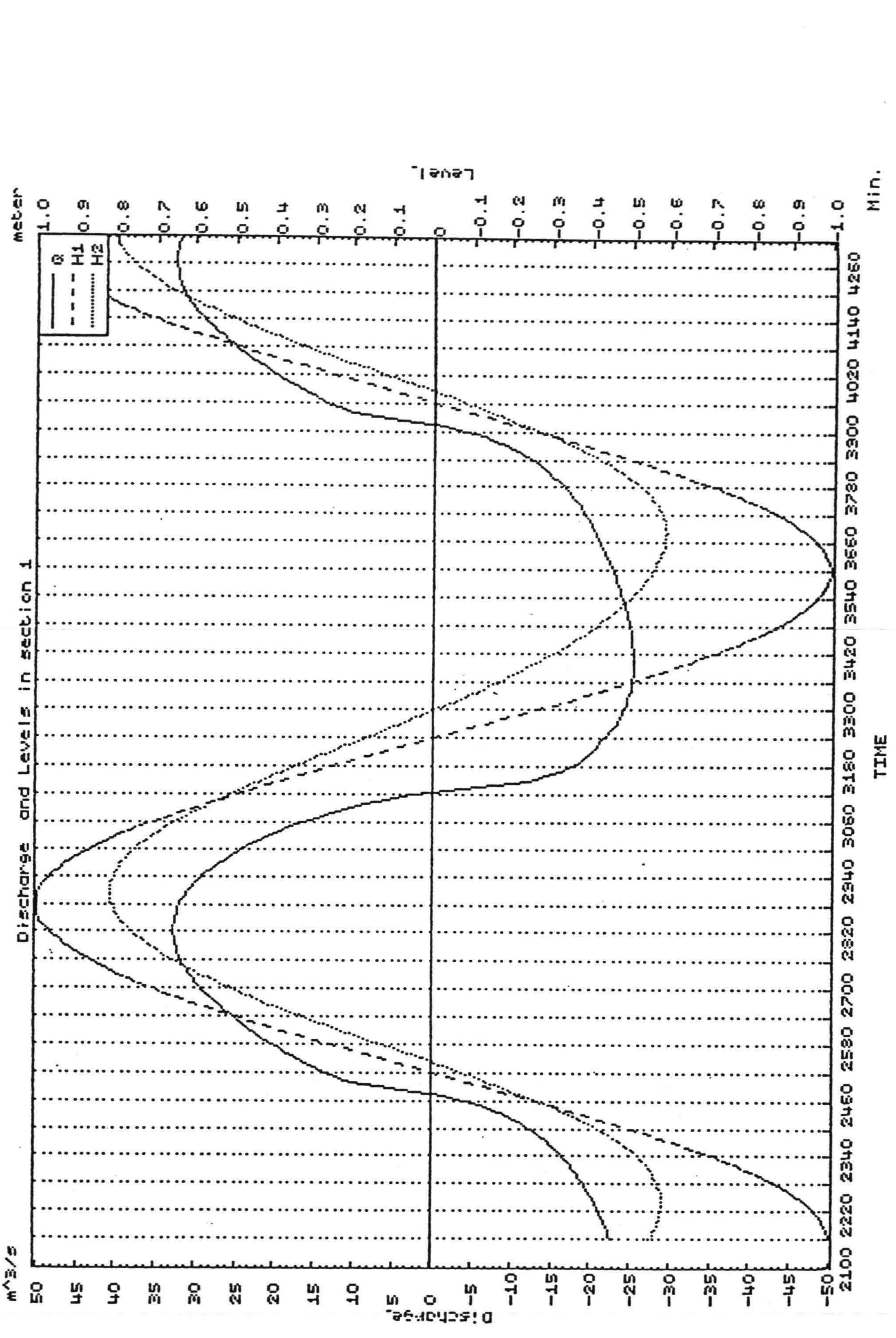
DISCHARGE IN SECTION 1B, 2B, 3B, 4B, 5B, 6B, 7B and 8B
m^3/s
Discharge
40
35
30
25
20
15
10
5
0
-5
-10
-15
-20
-25
-30
1B
2B
3B
4B
5B
6B
7B
8B
2100 2220 2340 2460 2580 2700 2820 2940 3060 3180 3300 3420 3540 3660 3780 3900 4020 4140 4260
TIME
Min.

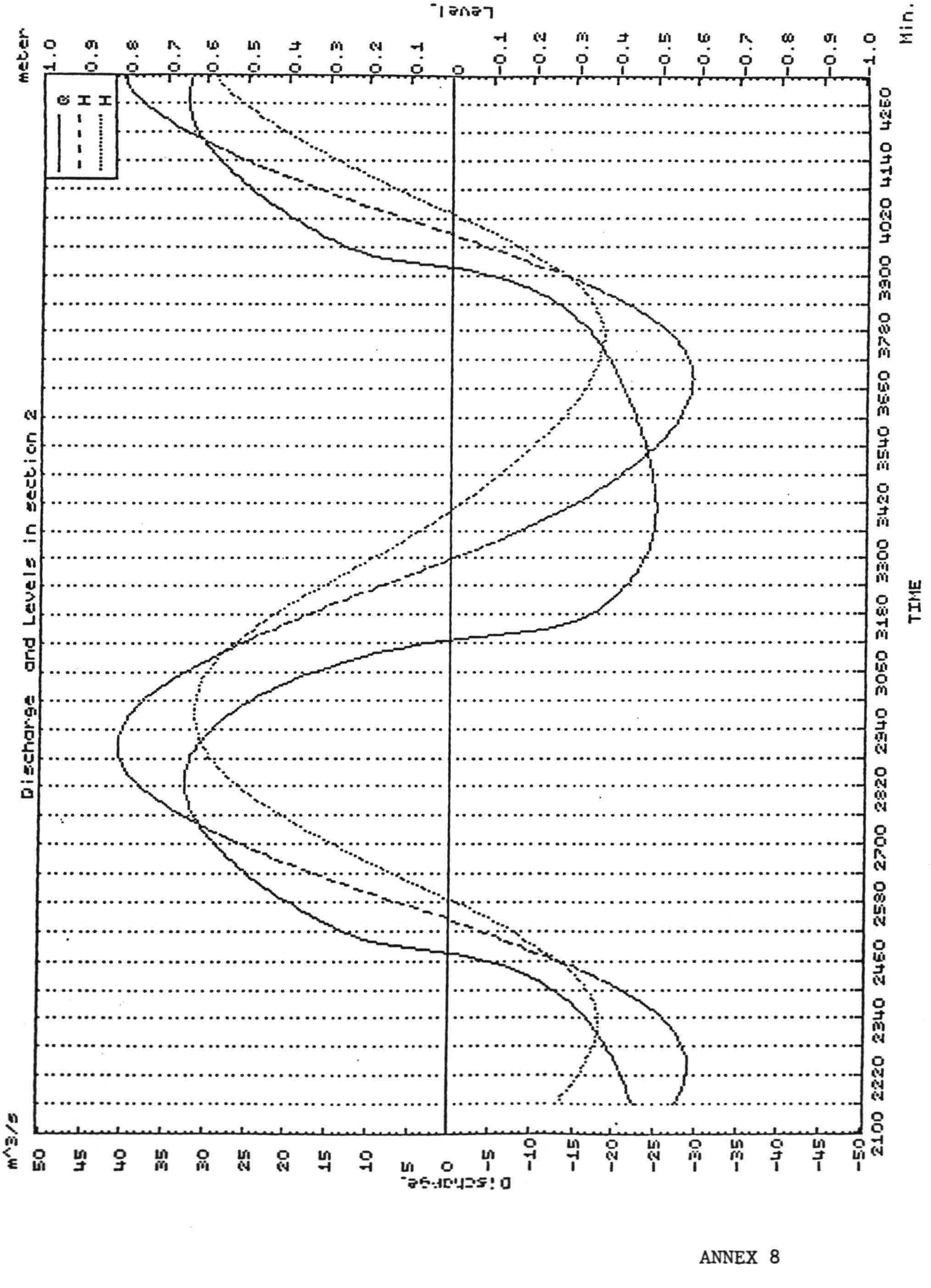

DISCHARGE IN SECTION 1B, 2B, 3B, 9B, 10B, 11B, 12B and 13B
1B
2B
3B
9B
10B
11B
12B
13B
m^3/s
Discharge
40
35
30
25
20
15
10
5
0
-5
-10
-15
-20
-25
-30
2100
2220
2340
2460
2580
2700
2820
2940
3060
3180
3300
3420
3540
3660
3780
3900
4020
4140
4260
Min.
TIME

ANNEX 5

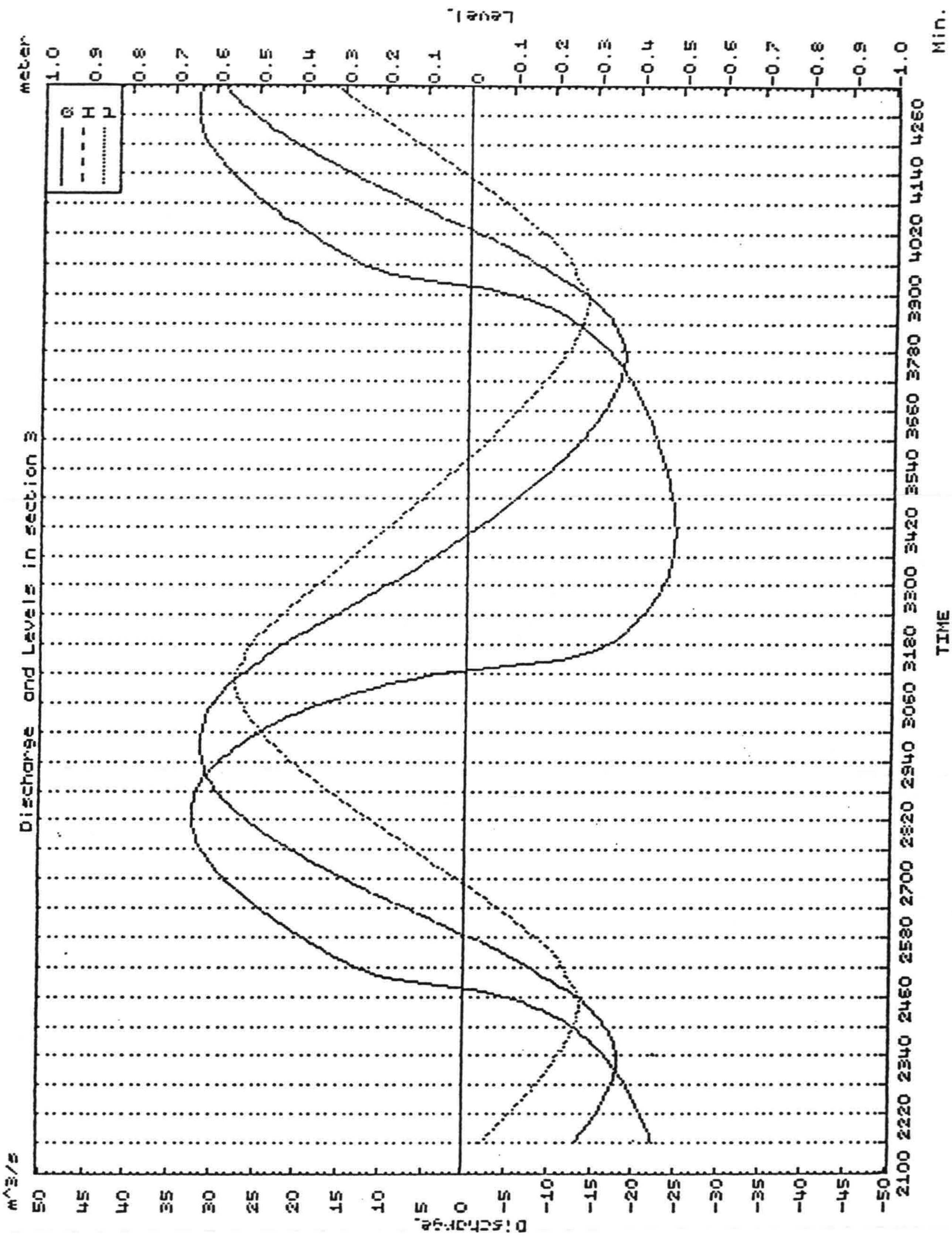

DISCHARGE IN SECTION 1B, 2B, 3B, 14B, 15B, 16B, 17B and 18B
m^3/s
Discharge,
40
35
30
25
20
15
10
5
0
-5
-10
-15
-20
-25
-30
2100
2220
2340
2460
2580
2700
2820
2940
3060
3180
3300
3420
3540
3660
3780
3900
4020
4140
4260
TIME
Min.
1B
2B
3B
14B
15B
16B
17B
18B

ANNEX 6

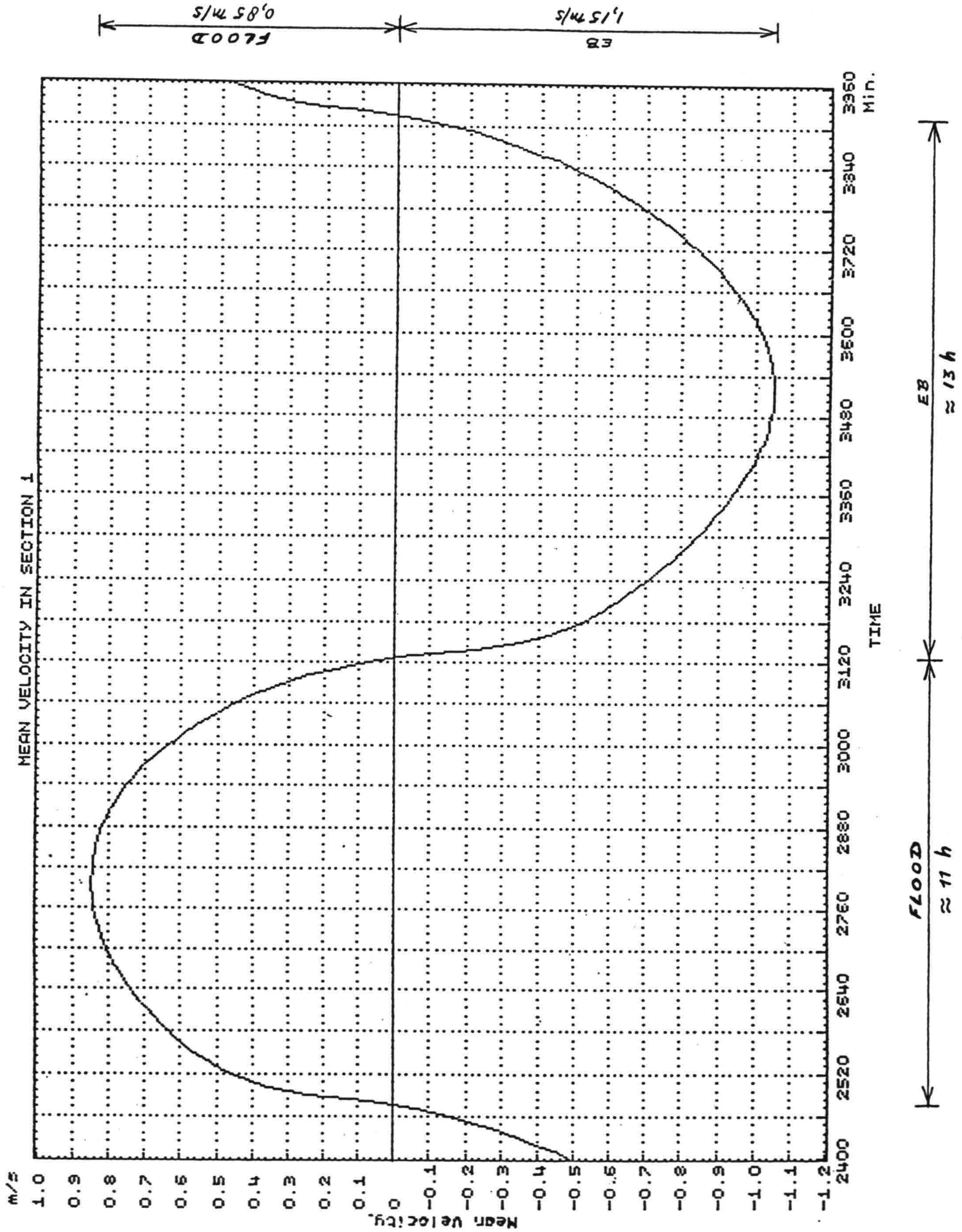

Discharge and Levels in section 1
Q
H1
H2
TIME
Min.
meter
Level
m^3/s
Discharge

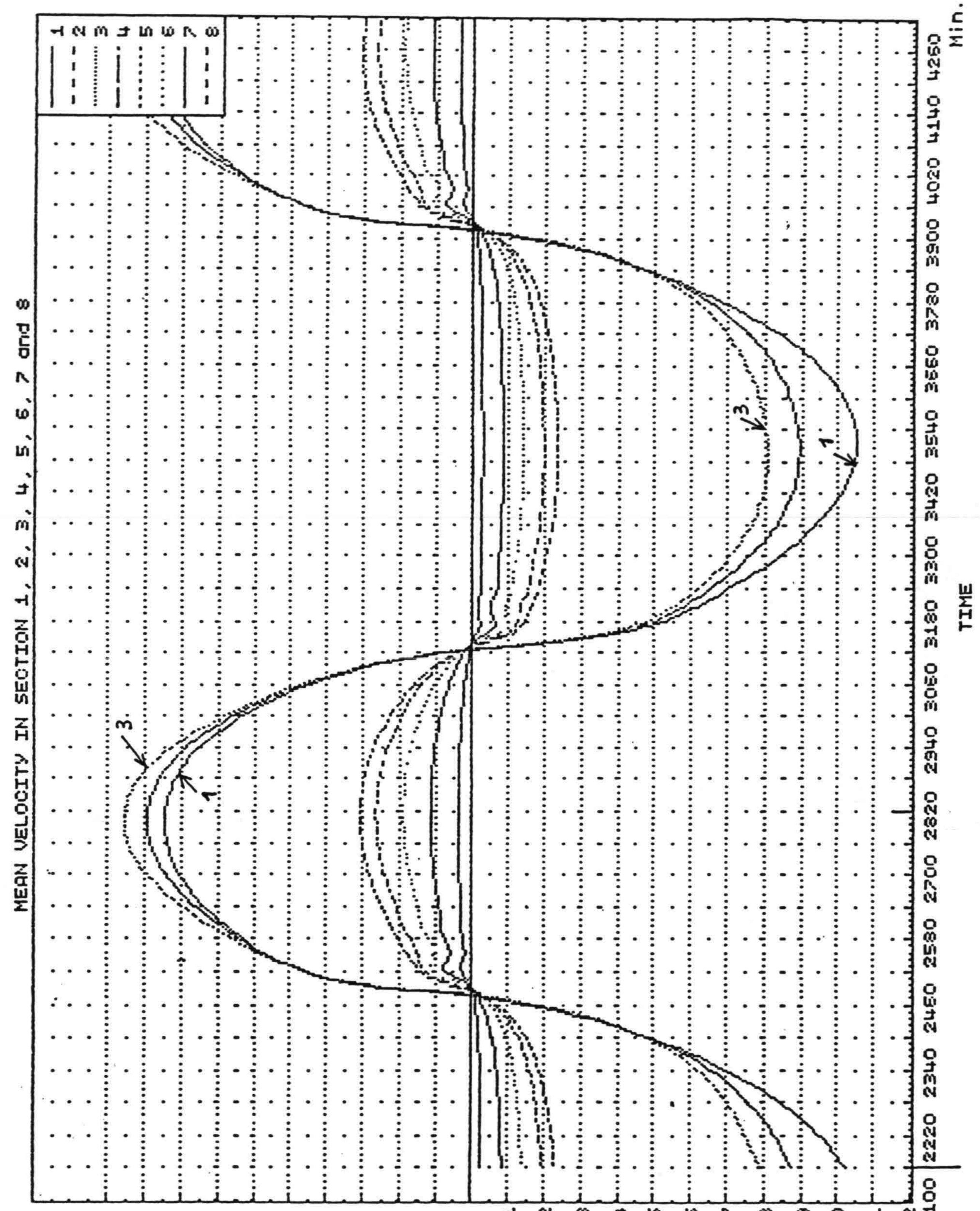


Discharge and Levels in section 2
Q
H
H
meter
Level
m^3/s
Discharge
TIME
Min.
2100 2220 2340 2460 2580 2700 2820 2940 3060 3180 3300 3420 3540 3660 3780 3900 4020 4140 4260
50 45 40 35 30 25 20 15 10 5 0 -5 -10 -15 -20 -25 -30 -35 -40 -45 -50
1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1 0 -0.1 -0.2 -0.3 -0.4 -0.5 -0.6 -0.7 -0.8 -0.9 -1.0


Discharge and Levels in section 3
Q
H
L
meter
Level
Discharge
m^3/s
TIME
Min.
2100 2220 2340 2460 2580 2700 2820 2940 3060 3180 3300 3420 3540 3660 3780 3900 4020 4140 4260
50 45 40 35 30 25 20 15 10 5 0 -5 -10 -15 -20 -25 -30 -35 -40 -45 -50
1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1 0 -0.1 -0.2 -0.3 -0.4 -0.5 -0.6 -0.7 -0.8 -0.9 -1.0

ANNEX 9


MEAN VELOCITY IN SECTION 1
TIME
Min.
Mean Velocity
m/s
FLOOD
0,85 m/s
EB
1,15 m/s
FLOOD
≈ 11 h
EB
≈ 13 h


MEAN VELOCITY IN SECTION 1, 2, 3, 4, 5, 6, 7 and 8
Mean Velocity
m/s
TIME
Min.
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
-1.1
-1.2
2100
2220
2340
2460
2580
2700
2820
2940
3060
3180
3300
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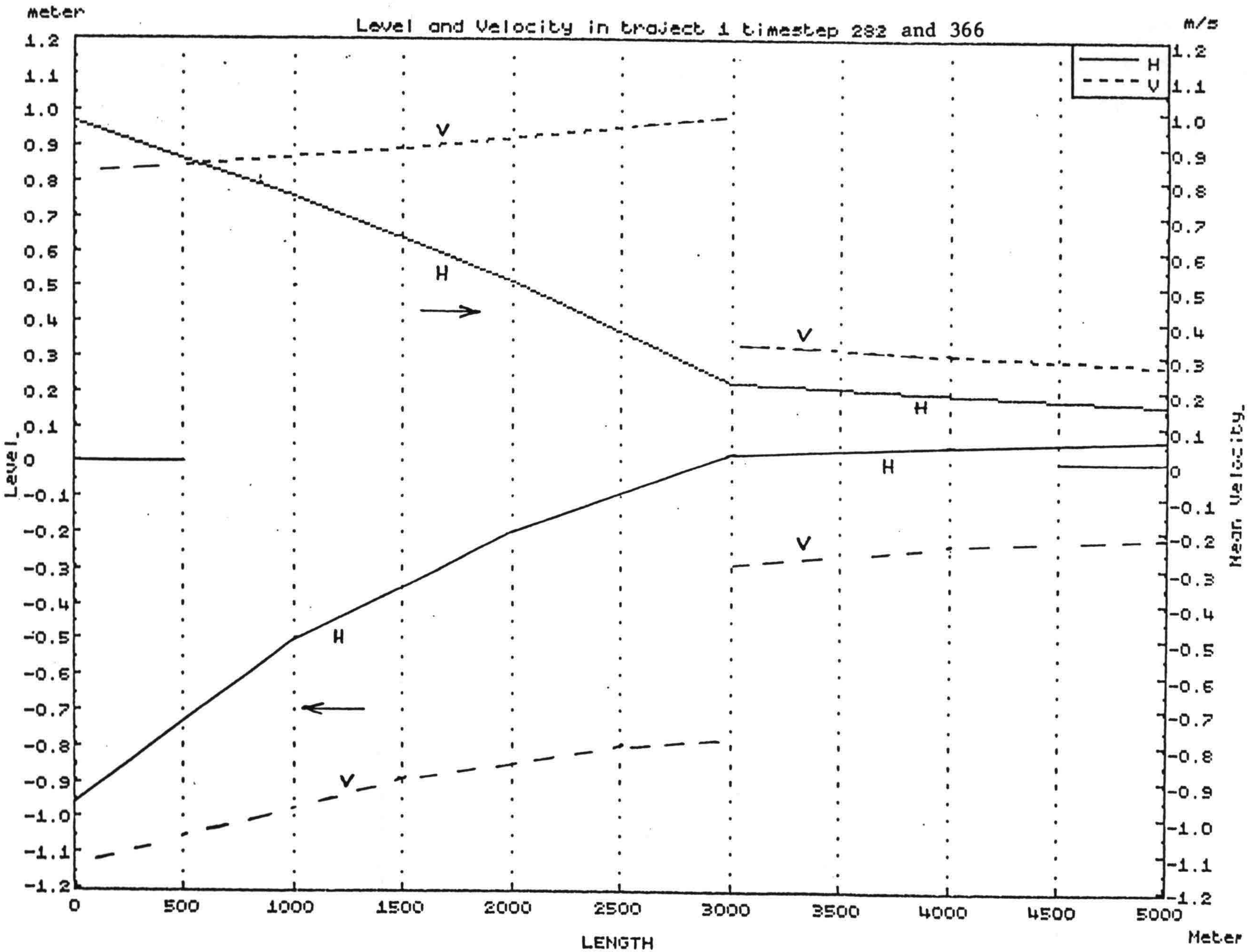
3900
4020
4140
4260
1
2
3
4
5
6
7
8


Level and Velocity in traject 1 timestep 282 and 366
meter
Level
m/s
Mean Velocity
H
V
LENGTH
Meter
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
-1.1
-1.2

APPENDIX 1 Evaluation check by PC–calculations

An arbitrary section as an element of a network system is considered (see Figure 1.1.). Depending on the practical problem the section length (about 0.01 times the wave length) can be 100 m or 10 km. In the network IRRSYS the section length in the primary canal is 1000 m which is relatively large (in that case). In general the influence of the section length (and also the time step) has to be varied and the results of different calculations has to be compared to check the accuracy of the calculations.

The differential equations describing one–dimensional unsteady flow in open channel systems (simulated by DUFLOW, see Ref. [5]) are written in the form :

$$B \frac{\delta h}{\delta t} + \frac{\delta Q}{\delta x} = 0 \qquad (1\text{–}1)$$

$$\frac{1}{g\,A} \left[\frac{\delta Q}{\delta t} + \frac{\delta}{\delta x}\left(\frac{Q^2}{A}\right) \right] + \frac{\delta h}{\delta x} + \frac{Q\,|Q|}{C^2\,A^2\,R} = 0 \qquad (1\text{–}2)$$

Integration of (1–1) and (1–2) in x–direction over the section length (see Figure 1.1.) gives the following expressions :

$$\int_1^2 -B \frac{\delta h}{\delta t}\,dx + \int_1^2 \frac{\delta Q}{\delta x}\,dx = 0 \qquad (1\text{–}3)$$

$$\int_1^2 \frac{1}{g\,A} \left[\frac{\delta Q}{\delta t} + \frac{\delta}{\delta x}\left(\frac{Q}{A}\right) \right] dx + \int_1^2 \frac{\delta h}{\delta x}\,dx + \int_1^2 \frac{Q\,|Q|}{C^2\,A^2\,R}\,dx = 0 \qquad (1\text{–}4)$$

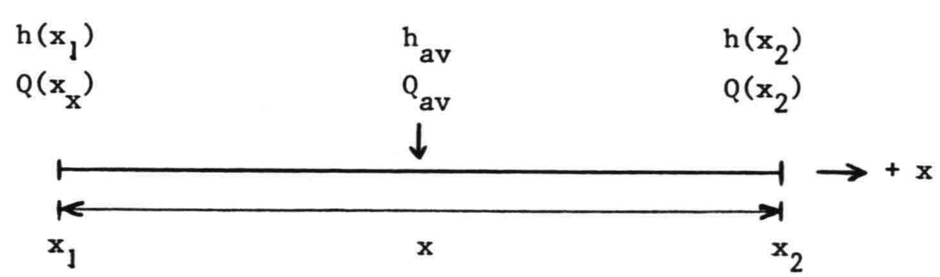


Figure 1.1. A section of a network with section length Δx.

The parameters in (1-3) and (1-4) are functions of x and t. On each time level t the mean values in the section (see Figure 1.1.) is substituted and written as h_{av} , Q_{av} , etc.
The results of the integration using mean values over the section length Δx are written in the form :

$$Q(x_2) - Q(x_1) = - B_{av}\ \Delta x\ \frac{dh_{av}}{dt} = \Delta F\ \frac{dh_{av}}{dt} \qquad (1\text{-}5)$$

$$h(x_2) - h(x_1) = - \frac{1}{g\ A_{av}} \left[\frac{dQ_{av}}{dt}\ \Delta x + Q_2^2/A_2 - Q_1^2/A_1\right] - \frac{\Delta x}{(C^2 A^2 R)_{av}}\ Q_{av}|Q_{av}| \qquad (1\text{-}6)$$

When the functions $h(x_1,t)$, $h(x_2,t)$, $Q(x_1,t)$ and $Q(x_2,t)$ and the dimensions of the section are well known, ΔF, A_{av}, .. etc. can be determined for h_{av} at the time level which is choosen to calculate the terms at the right hand side of the equations (1-5) and (1-6).
Equation (1-5) gives the possibility to check the difference between the discharge at point 2 and the discharge at point 1 at any time level t.
Equation (1-6) gives the balance between the water level difference over the section length Δx and the sum of (three contributions of) :

- the local acceleration term;
- the advective or convective acceleration term;
- the friction term.

The two equations (1-5) and (1-6) together give the possibility to check the results of calculations and also measurements. These evaluation check calculations can be performed by hand.

Evaluation calculations based on (1-5) and (1-6) concerning DUFLOW results (at a larger scale) can also be executed by means of a PC-program which is called ECDUFLOW (see Table 3 ... 6 and Figure 5 in Chapter 3).
More information concerning evaluation calculations with the program ECDUFLOW using the DUFLOW menu structure is given in Ref. [6].

APPENDIX 2 IRRSYS – Results Harmonic Method

The results of three different solutions by means of a PC–program based on the Harmonic Method (see Ref. [3]) are briefly presented first. Finally an analytic solution for the most simple model which can be executed by hand is given.

According to the Lorentz schematization (see Ref. [3]) the differential equations (1–1) and (1–2) of Appendix 1 can be linearized which makes it possible to give an analytic solution existing of two equations for each section of a network system like IRRSYS. For 18 sections the set of 36 equations can be solved when the boundary conditions at the boundaries of the network are substituted. For 18 sections the calculations by means of complex functions are extensive but with the PC–program used here the calculations are executed in about five seconds.
The results are presented in a table consisting the amplitudes and the phase shift of the water levels and the discharges at the nodes of the network system. The solution of the linear differential equations exist of pure sinusoidal functions (so non–linear influences as discussed in Chapter 5 of this report are absent).
The results of the Harmonic Method give a good insight in the tidal propagation at most concerning the dampening, the phase shift and the order of the discharges.

The first solution is found for the most simple model existing of only two sections (see Figure 2.1. at the next page).

Section 1–4 : the flow width and the storage width is 10 m ;
the section length is 3000 m ;
the mean water depth is 3 m ;
Chézy 's coefficient equals $C = 40\ m^{1/2}/s$.

Section $4–4^1$ the flow width is 30 m ;
the storage width is 270 m which is based on the total internal storage area (F) and the section length of 4000 m , so $F = 1,080,000\ m^2$.

From the results, found by the tidal period of 86,400 s and the amplitude at node 1 ($\hat{h}_1 = 1$ m), the amplitude at node 4 and the phase shift at node 4 (which almost agree with the values at node 4^1) are given here :

$$\hat{h}_4 = 0.394 \text{ m and } - \kappa = 293^0 = -67^0$$

These results are presented graphically in Figure 1.
In the result table the phase shift is given as a positive angle.

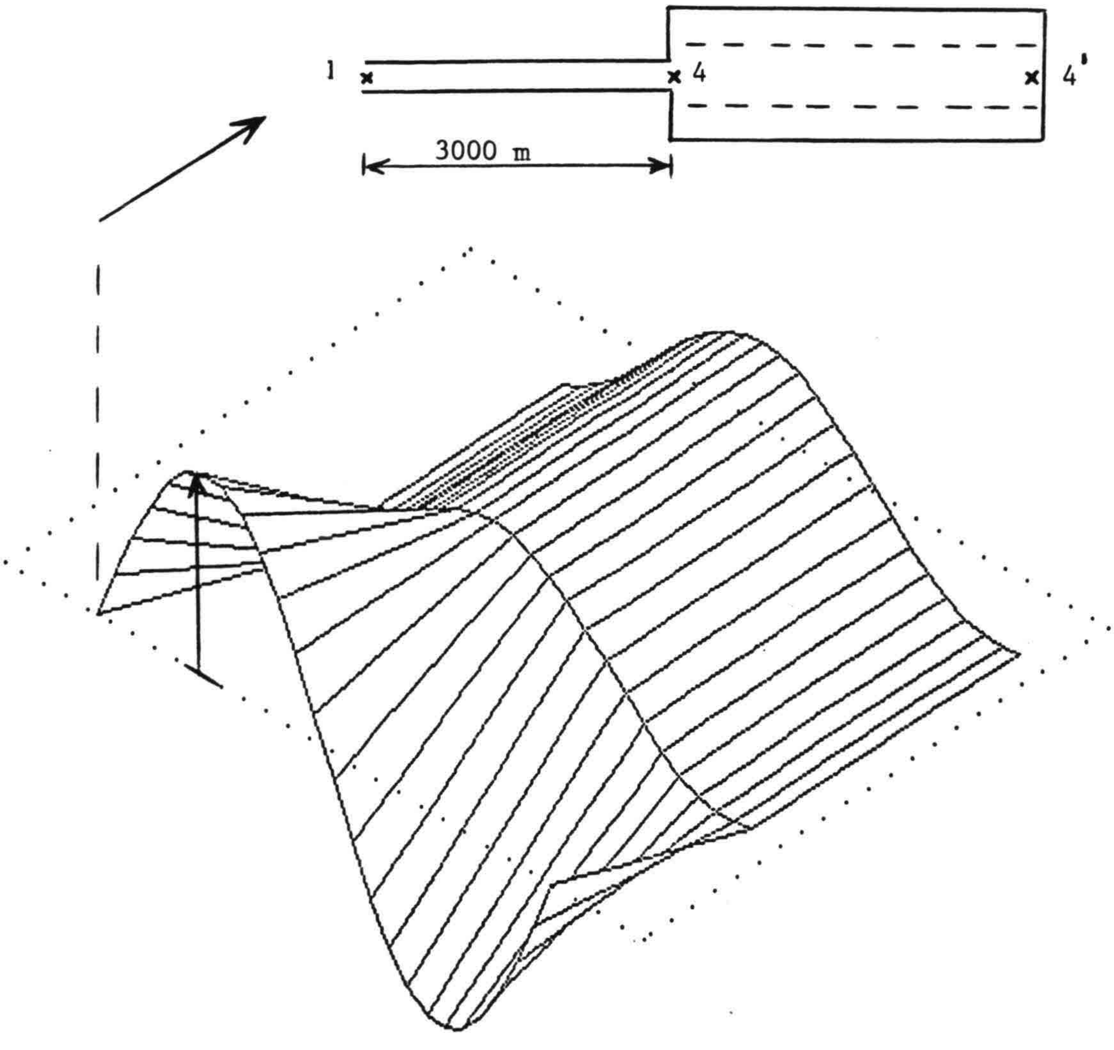
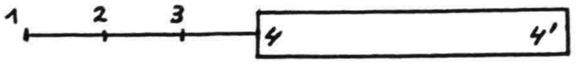


Figure 2.1. IRRSYS model - only two sections

The results of the second model ,with the primary canal existing of three sections of 1000 m , agree with the solution of the first model.These results and the most important information are given on the next page without further comment.

IRRSYS Harmonic Method

3 sections between nodes 1 and 4



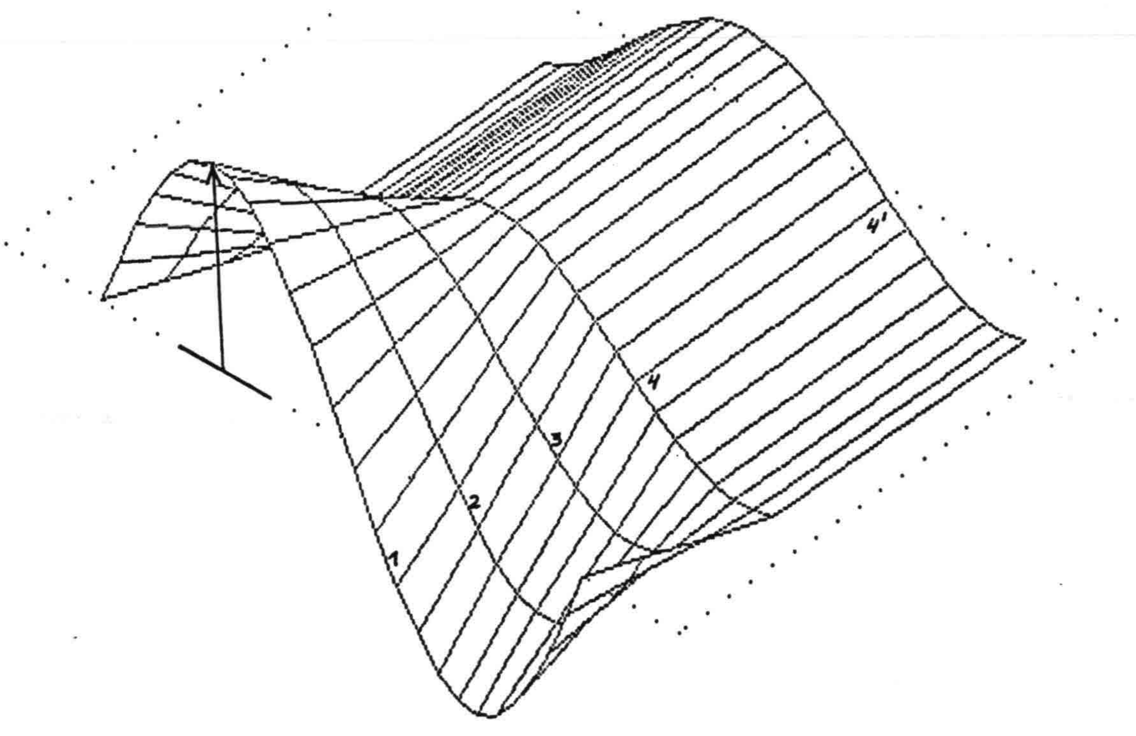
Information and results :

tidal period : 86,400 s

section	flow width	storage width	mean depth	length	C	Q
1	10.00000	10.00000	3.00	1.00	40.00	31.03318
2	10.00000	10.00000	3.00	1.00	40.00	30.75995
3	10.00000	10.00000	3.00	1.00	40.00	30.48561
4	30.00000	270.00000	3.00	4.00	40.00	15.17378

Results :

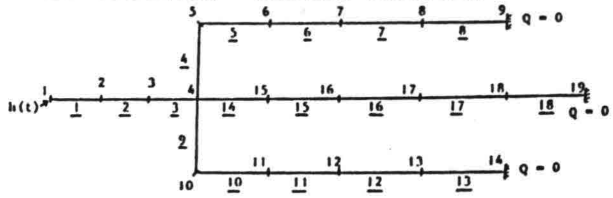
section	h	phase	Q_{begin}	phase	Q_{end}	phase
	1.00000	0.00000				
1	0.72353	350.19333	31.17011	22.37877	30.89625	21.34260
2	0.49188	330.76254	30.89625	21.34260	30.62366	20.70915
3	0.38356	292.96934	30.62366	20.70915	30.34756	20.48178
4	0.38803	289.24956	30.34756	20.48178	0.00000	



The results of the Harmonic Method applied for the entire network existing of 18 branches are given on the next page. The amplitude of h (x,t) in the sections 4 ... 18 is almost constant and the values agree with the results of the first and the second model.

IRRSYS Harmonic Method

18 sections entire network



Period : 86,400 s

Information :

section	flow width	storage width	mean depth	length	C	Q
1	10.00000	10.00000	3.00	1.00	40.00	30.64746
2	10.00000	10.00000	3.00	1.00	40.00	30.37821
3	10.00000	10.00000	3.00	1.00	40.00	30.10833
4	10.00000	10.00000	3.00	2.00	40.00	9.15415
5	10.00000	80.00000	3.00	1.00	40.00	7.77543
6	10.00000	80.00000	3.00	1.00	40.00	5.56157
7	10.00000	80.00000	3.00	1.00	40.00	3.34011
8	10.00000	80.00000	3.00	1.00	40.00	1.11390
9	10.00000	10.00000	3.00	2.00	40.00	9.15415
10	10.00000	80.00000	3.00	1.00	40.00	7.77543
11	10.00000	80.00000	3.00	1.00	40.00	5.56157
12	10.00000	80.00000	3.00	1.00	40.00	3.34011
13	10.00000	80.00000	3.00	1.00	40.00	1.11390
14	10.00000	80.00000	3.00	1.00	40.00	10.02790
15	10.00000	80.00000	3.00	1.00	40.00	7.81292
16	10.00000	80.00000	3.00	1.00	40.00	5.58838
17	10.00000	80.00000	3.00	1.00	40.00	3.35621
18	10.00000	80.00000	3.00	1.00	40.00	1.11927

Results :

section	h	phase	Q_{begin}	phase	Q_{end}	phase
	1.00000	0.00000				
1	0.73585	350.93784	30.78253	22.07942	30.51239	21.01691
2	0.50921	333.68341	30.51239	21.01691	30.24403	20.34083
3	0.38213	299.83440	30.24403	20.34083	29.97262	20.05439
4	0.37969	291.84075	9.42817	18.59097	8.88013	18.14283
5	0.38073	288.95102	8.88013	18.14283	6.67073	17.41932
6	0.38194	287.47890	6.67073	17.41932	4.45241	17.04778
7	0.38275	286.95003	4.45241	17.04778	2.22780	16.91089
8	0.38302	286.89134	2.22780	16.91089	0.00000	—
9	0.37969	291.84075	9.42817	18.59097	8.88013	18.14283
10	0.38073	288.95102	8.88013	18.14283	6.67073	17.41932
11	0.38194	287.47890	6.67073	17.41932	4.45241	17.04778
12	0.38275	286.95003	4.45241	17.04778	2.22780	16.91089
13	0.38302	286.89134	2.22780	16.91089	0.00000	—
14	0.38153	295.05085	11.13285	22.53357	8.92295	21.33513
15	0.38257	292.14722	8.92295	21.33513	6.70289	20.60813
16	0.38378	290.66799	6.70289	20.60813	4.47388	20.23479
17	0.38459	290.13657	4.47388	20.23479	2.23854	20.09724
18	0.38487	290.07759	2.23854	20.09724	0.00000	—

Analytic solution IRRSYS

Another analytic solution based on the similar principles as the harmonic method is briefly described here.
The primary canal of the network IRRSYS (see Figure 4 in Chapter 2) is modelled as ONE section 1–4 with a section length of 3000 m and the total internal storage area is modelled as a large reservoir connected at node 4 (see Figure 2.2.).

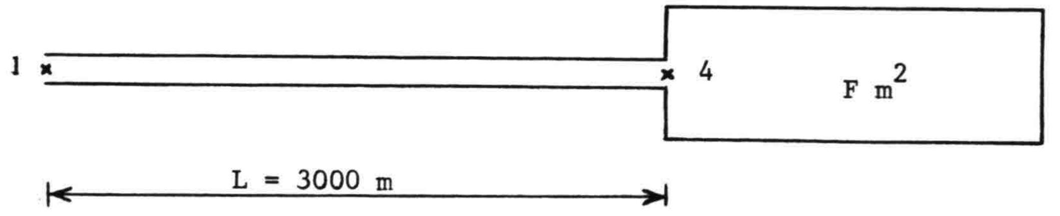


Figure 2.2 The most simple schematization of IRRSYS

The discharge at point 4 is calculated with the relation

$$Q_4(t) = F \frac{\delta h_4(t)}{\delta t} \qquad (2\text{–}1)$$

Because the water level gradient in the sections 4 ... 18 of the network IRRSYS (see Figure 4 in Chapter 2) is relatively small the equation (2–1) gives a good estimation of the discharge at the end of the primary canal. The storage area of the narrow primary canal is relatively small compared to F in (2–1) so the discharge calculated with (2–1) is also a good estimation of the discharge in the entire primary canal. This makes that the momentum equation can easily be integrated in x–direction over the only section 1–4.
We assume that $Q_4(t) = Q_1(t) = Q(t)$ so the reprentative discharge in the primary canal is called Q(t) from now on. Further assumptions are :

- the advective (or convective) term is neglected;
- the cross–section area of the primary canal is constant in x and also constant in time; this constant area is calculated for the mean water level and is called A;

- the friction term is linearized according to Lorentz' method which is applied also in the Harmonic Method so this term is written as : $K * Q(t)$, with $K = (8/(3\pi)) * \hat{Q} * L/(C^2 A^2 R)$.

With these assumptions the general equation (1–6) in Appendix 1 with $L/(gA) = M$ gives the following momentum balance for the entire primary canal which has to be solved together with (2–1) :

$$h_4(t) - h_1(t) = - M \frac{\delta Q}{\delta t} - K Q \qquad (2\text{–}2)$$

Substitution of (2–1) in (2–2) gives :

$$h_1(t) = h_4(t) + KF \frac{\delta h_4(t)}{\delta t} + MF \frac{\delta^2 h_4(t)}{\delta t^2} \qquad (2\text{–}3)$$

The function $h_1(t)$ is given (boundary condition) as a sinusoidal function in time with amplitude $\hat{h}_1$ and phase angle which can be choosen $- \kappa_1 = 0$. The function $h_4(t)$ has to be solved as a sinusoidal function with amplitud $\hat{h}_4$ and phase shift $-\kappa_4$. The analytic solution by means of exponential functions of (2–3) (see handbooks) results in the following expressions :

$$\kappa = \text{artg} \; \frac{\omega K F}{1 - \omega^2 M F} \qquad (2\text{–}4)$$

$$\frac{\hat{h}_4}{\hat{h}_1} = \frac{1}{\sqrt{(1 - \omega^2 MF)^2 + \omega^2 K^2 F^2}} \qquad (2\text{–}5)$$

With (2–4) and (2–5) the phase shift and the amplitude of the function $h_4(t)$ can be calculated by substituting the parameters ω, F, M and K which are representative for the system and are defined before. The discharge Q can be calculated with the solved function $h_4(t)$ by means of equation (2–1).

For the system IRRSYS the following values can be substituted :

$\omega = 0.73 * 10^{-4}$ rad./s , $F = 108 * 10^4\ m^2$, $L = 3000$ m, $A = 30\ m^2$,

and $K = 0.85 * 30 * 3000/(1600 * 900 * 1.88) = 0.0283\ s/m^2$.

Note : For the calculation of K with given L, C, A and R we need a good estimation of the amplitude of Q(t). This means an iteration procedure. In this case we can substitute 30 m^3/s.

The general solution based on the model of Figure 2.2. gives :

$$\kappa = \text{arctg}\,\frac{2.22}{0.942} = \text{arctg}\ 2.357 = 67^{0} \qquad (2\text{-}6)$$

$$\frac{\hat{h}_4}{\hat{h}_1} = 1 \,/\, \sqrt{0.887 + 4.928} = 0.41 \qquad (2\text{-}7)$$

From the results of this simple model a phase shift of 67^{0} and an amplification factor (see Chapter 4) of 0.41 is to be expected.

The results agree with the other results of IRRSYS. The last model is very important however because it makes it very easy to give rather good predictions when ONE of the PARAMETERS L, F, A, C or R is changed. The expressions give a good insight in the role of the different parameters in the system. For example the relative influence of friction, the influence of the large internal storage area, etc. can be "followed" in the analytic approach.

This last analytic solution is very important to analyse and understand systems like IRRSYS.

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