IMPROVEMENT OF PLUIT POLDER SYSTEM IN JAKARTA FLOOD CONTROL PROJECT

by

YULIANTI

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ABSTRACT

Flooding of low lying areas near the sea in Jakarta, Indonesia by storm water runoff has long been a problem. One system of flood control is called the polder system, which consists of a collection of canals and reservoirs.

The object of this study was to develop a method to improve the operation of a polder system using a mathematical model. For this thesis, the Pluit Polder System was used as a case study.

The following conclusions and recommendations were arrived at from this study:

In tropical climates such as that found in the Pluit area, it is important to design a polder system with sufficient capacity to pass the flows from several closely spaced rain storms.

The mathematical model developed in this study can be used to predict the extent of flooding under the assumption that improvements to the components of the Polder have been made.

The choice of alternative improvements depends on the results of a cost benefit study. The cost benefit study was not included in this study. It is recommended that more detailed risk analyses and combinations of storm events be considered in the cost benefit study.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND OF THE STUDY

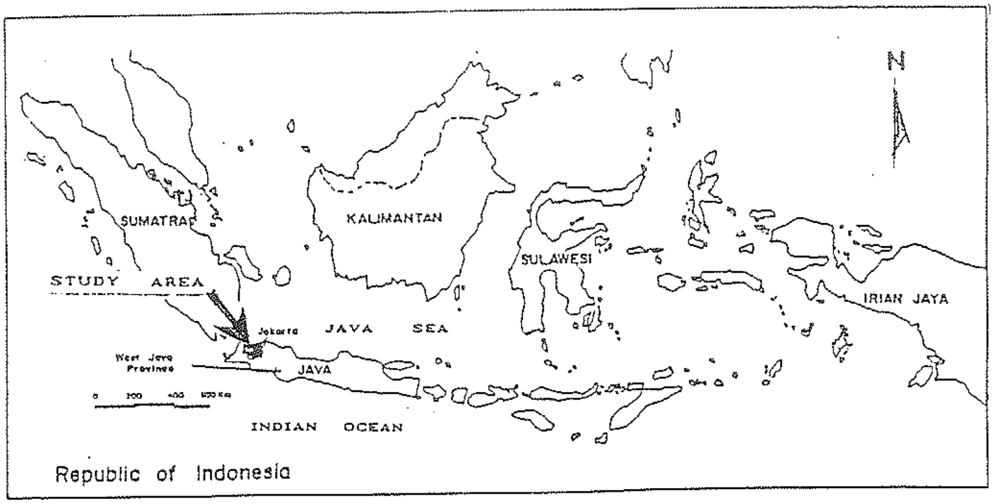
Many cities in low lying areas on the coast of Indonesia have flooding problems due to storm water runoff. Floods are caused by inflows coming from outside the city or caused by heavy rainfall on the city area itself. One system of flood control is called the polder system. A polder system consists of a collection canals and ponds (or polders). The storm water is pumped from these polders to the sea. In many cases polder systems need improvement after a period of time due to changes in the polder area or changes in the area upstream of the polder which influences the capacity of the canals and hydraulic structures in the system. Some examples of these changes are land development, and increases in upstream stream flow. The presence of sediment and waste (or garbage) in the canal results in deposition or erosion in the canal and in the ponds, therefore reducing the capacity of the canal and hydraulic structures. Both changes in flow and waste load cause changes in the polder system.

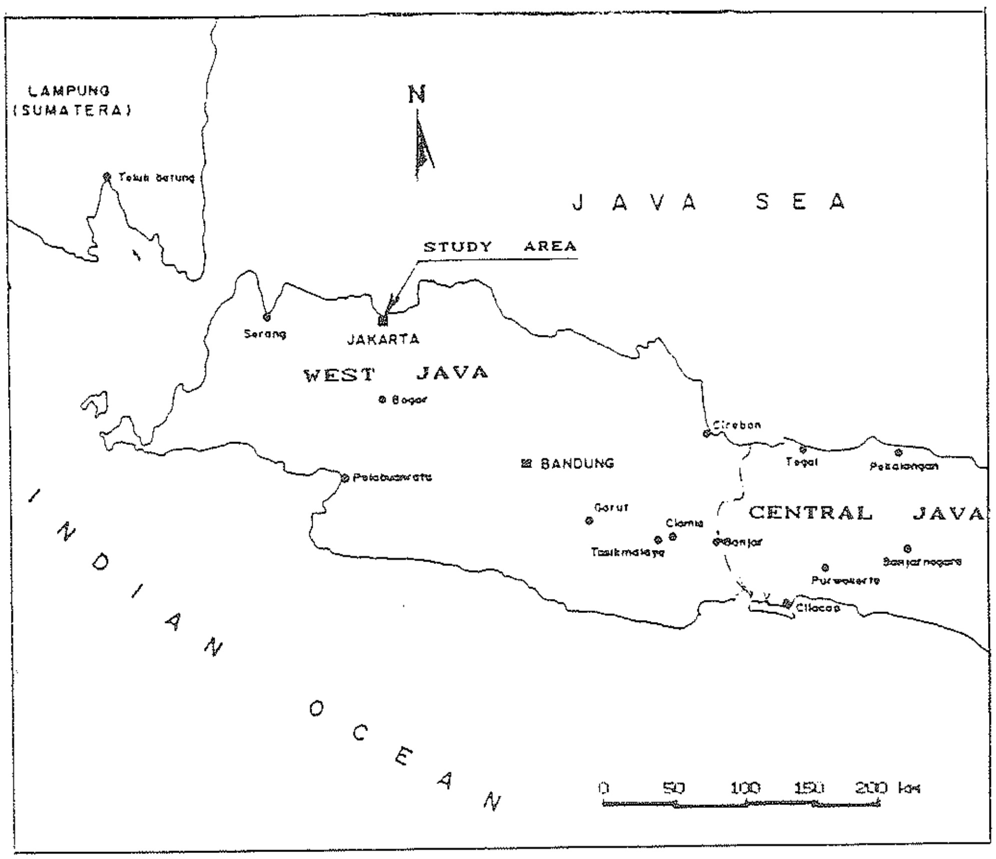
The object of this thesis is to develop a method to improve the polder system of flood control. For this thesis the Pluit Polder System in Jakarta is used as the study case.

Jakarta, the capital city of Indonesia is located in the North part of West Java Province shown in Figure 1.1. Like many cities in Northern Java, Jakarta has a major storm water flooding problem. A polder system has been developed to assist in managing the flooding in low lying areas of the city. There are four polder systems in Jakarta (see Figure 1.2); the Pluit Polder System, the West Sunter Polder System, the East Sunter Polder System and the Marunda Polder System.

The area served by the Pluit Polder System is 2545 hectares used primarily for residential, warehouse and industrial development. These areas are habitually inundated. To overcome flooding problems in this area, the Pluit Polder System was designed by Nedeco and Jakarta Flood Control Project in 1973. The design is based on the 25 year return period flood. In other words this polder system was expected to overcome the 25 year return period single flood. Based on this design, the Pluit Polder System was built in 1977-1981. In fact, flooding still occurs every year in this drainage area. The reasons are:

1. It is common to have two or more successive storms in areas with a tropical climate such as this area. When such storms interact they can greatly increase the resulting flood duration and volumes. The flood volume is very important in the design of a polder system, because once the polder has been filled, this system has very little capacity to pass a flood.





UNIVERSITY OF MANITOBA FACULTY OF GRADUATE STUDIES	
PLUIT POLDER SYSTEM	
GENERAL LOCATION MAP OF THE STUDY AREA	
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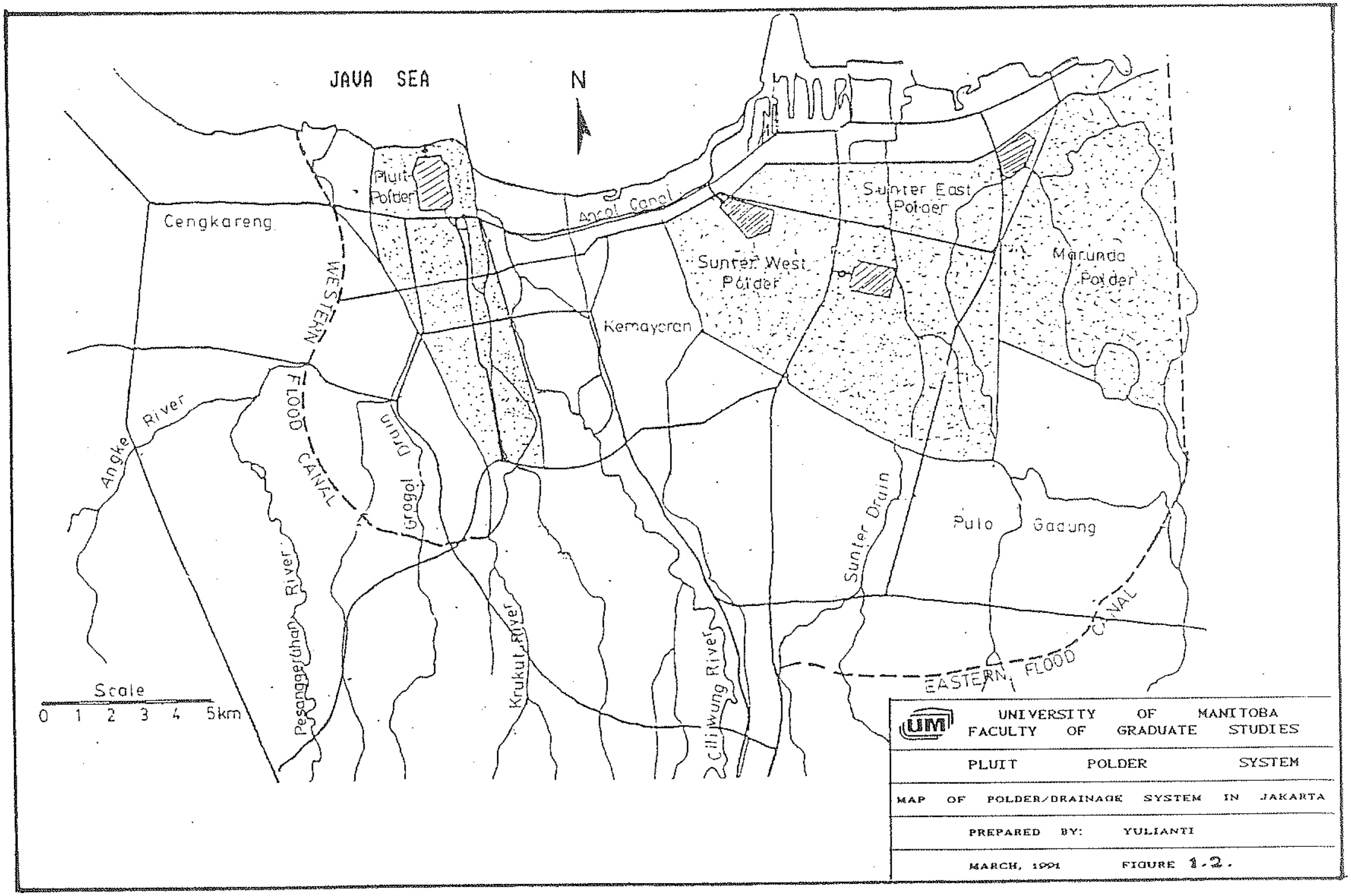
MARCH, 1991	FIGURE 1.1.



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PLUIT POLDER SYSTEM

MAP OF POLDER/DRAINAGE SYSTEM IN JAKARTA

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MARCH, 1991 FIGURE 1.2.

2. The increase in population in Jakarta causes problems for the system. The first problem is related to the rapid expansion of the city, which means increased in land development and increased drainage system. The second problem, related to population growth, is the increase in household waste. The increase in household waste is an important and serious problem here because of the habit of the people in this area of dumping their garbage into the channels and drains. Household garbage as solid waste is transported as suspended or floating matter in the channels. The presence of garbage in the channel reduces the capacity of the channel. Aquatic plants develop from the garbage, increase the channel roughness and therefore decrease the channel capacity in the system. The garbage reduces the capacities of the hydraulic structures to the point where garbage may even plug the structures.

As a result, the developed area upstream of these polders are regularly inundated (yearly flood). The flooded areas stagnate, for as long as several days, especially in the low lying areas. This stagnation increases flood damage to the inundated areas, which are mostly residential, warehouse and industrial areas. To overcome the problem, an effective operating plan and general improvement to the entire system is required.

1.2 THE OBJECTIVES OF THE STUDY

The objectives of this study are:

1. To assess the existing methods of operation of the Pluit Polder System using a mathematical (computer) model.
2. To investigate the adequacy of the existing polder system in passing the design discharge.
3. To research strategies to improve the system by evaluating various flood management alternatives.
4. To recommend remedial measures, together with the most effective operating procedures, as determined by using the mathematical model.

CHAPTER 2

GENERAL DESCRIPTION

2.1 LOCATION OF THE PROJECT AREA

Jakarta, the capital city of Indonesia is located at the mouth of Ciliwung River on a flat plain on the north coast of West Java Province. The study area, Pluit Polder is located in the northern part of Jakarta, immediately adjacent to the coast and lies downstream of a residential area (see Figures 1.2 and 2.1).

2.2 BACKGROUND AND PREVIOUS STUDIES

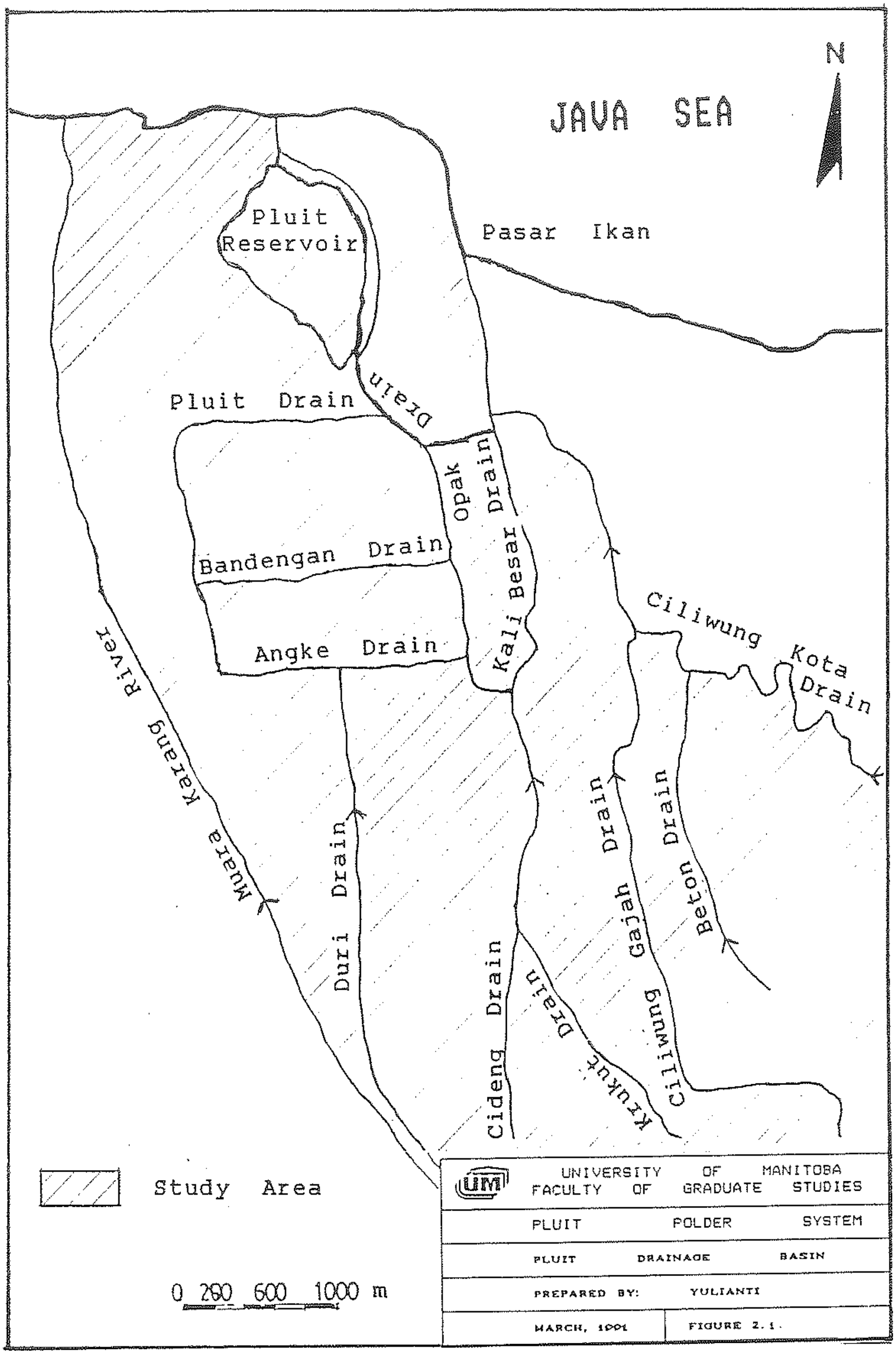
The Pluit Polder System was built in 1977-1981. It consists of a collecting reservoir, a ring canal, a pumping station, an intake to the ring canal, syphons to the reservoir, and outlet gates from the reservoir to the forebay in front of the pumping station (see Figure 2.2). The area served by the Pluit Polder System shown in Figure 2.1 is 2545 hectares including:

Cideng Drainage Area	-	285 hectares
Ciliwung Kota Drainage Area	-	618 hectares
Angke Drainage Area	-	46 hectares
Bandengan and Pluit Drainage Area	-	794 hectares
Kali Besar Drainage Area	-	333 hectares

Beton Drainage Area	-	150 hectares
Krukut Drainage Area	-	319 hectares

The land in the study area is very flat and some of the areas are lower than the channel bank elevation.

A study of drainage and flood control of Jakarta was done by Nedeco Consultants and Jakarta Flood Control Project, 1973. This study was a general analysis of the Pluit Polder system. The Pluit Polder System was built based on this study. Since floods still occurred in the area, a physical model of the inlet to the Pluit Polder System was investigated in 1983 (Ref. Institute of Hydraulic Engineering, 1983). A physical model investigation of the Outlet Pluit Polder System was also carried out in 1987 by the Institute of Hydraulic Engineering in Indonesia (Ref. Institute of Hydraulic Engineering, 1987). The hydraulic parameters that are used for this thesis were obtained from Nedeco Consultants (1973).


JAVA SEA
N
Pluit Reservoir
Pasar Ikan
Pluit Drain
Drain
Opak Drain
Bandengan Drain
Kali Besar Drain
Angke Drain
Ciliwung Kota Drain
Muara Karang River
Duri Drain
Cideng Drain
Krukut Drain
Gajah Drain
Ciliwung
Beton Drain
Study Area
0 200 600 1000 m
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PLUIT POLDER SYSTEM
PLUIT DRAINAGE BASIN
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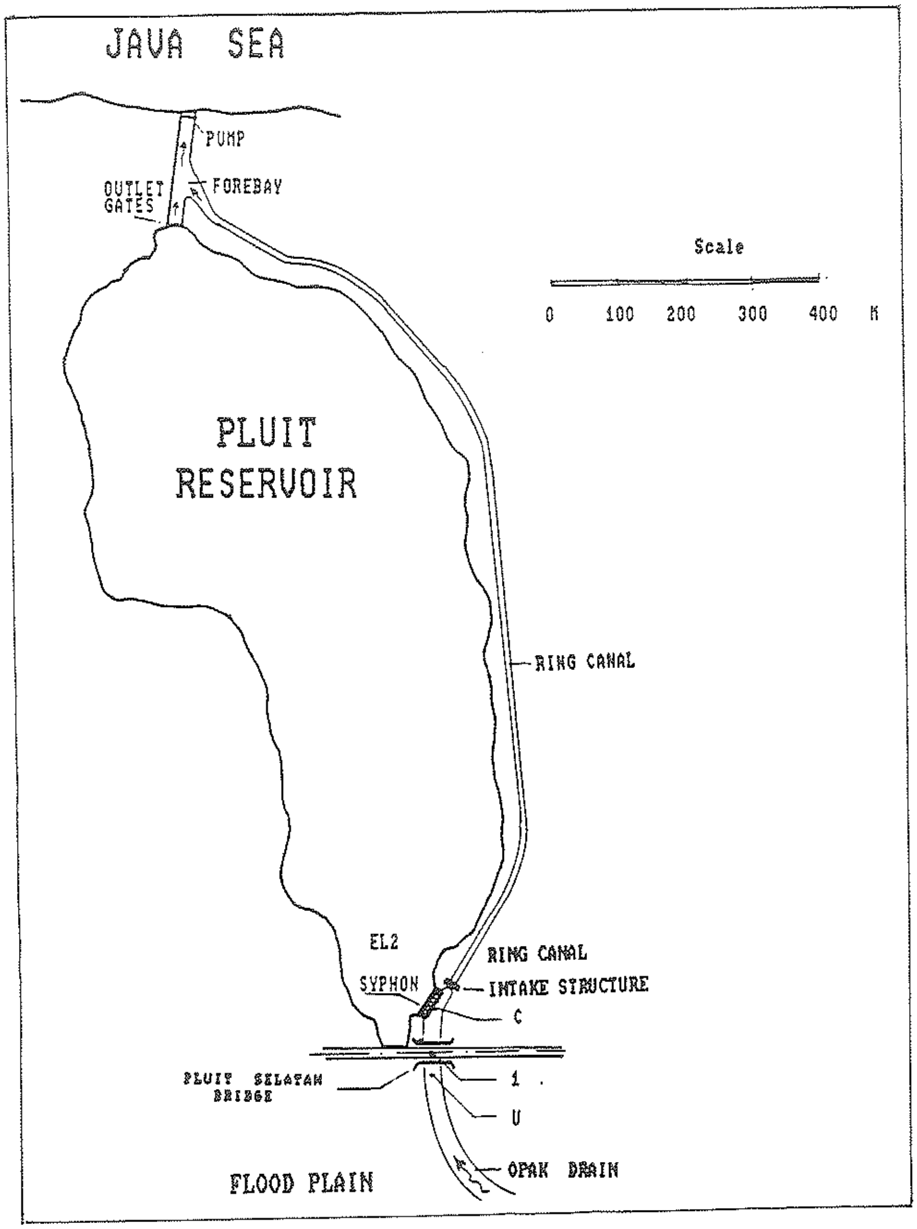
FIGURE 2.1.



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PLUIT POLDER SYSTEM GENARAL LAYOUT OF THE SYSTEM	
PREPARED BY: YULIANTI	
MARCH, 1991	FIGURE 2.2.

III

CHAPTER 3

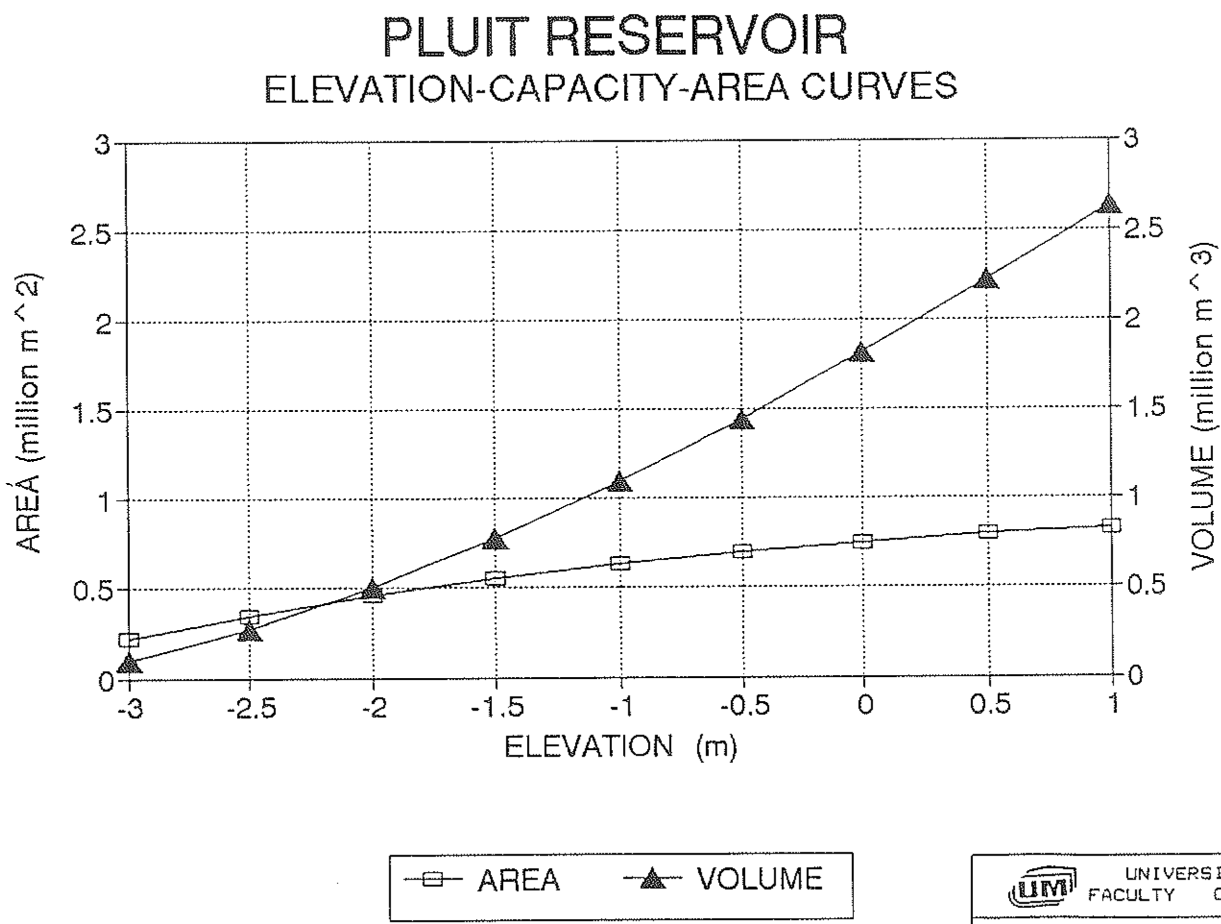
PRESENT CONDITION AND AVAILABLE DATA

3.1 GENERAL ARRANGEMENT

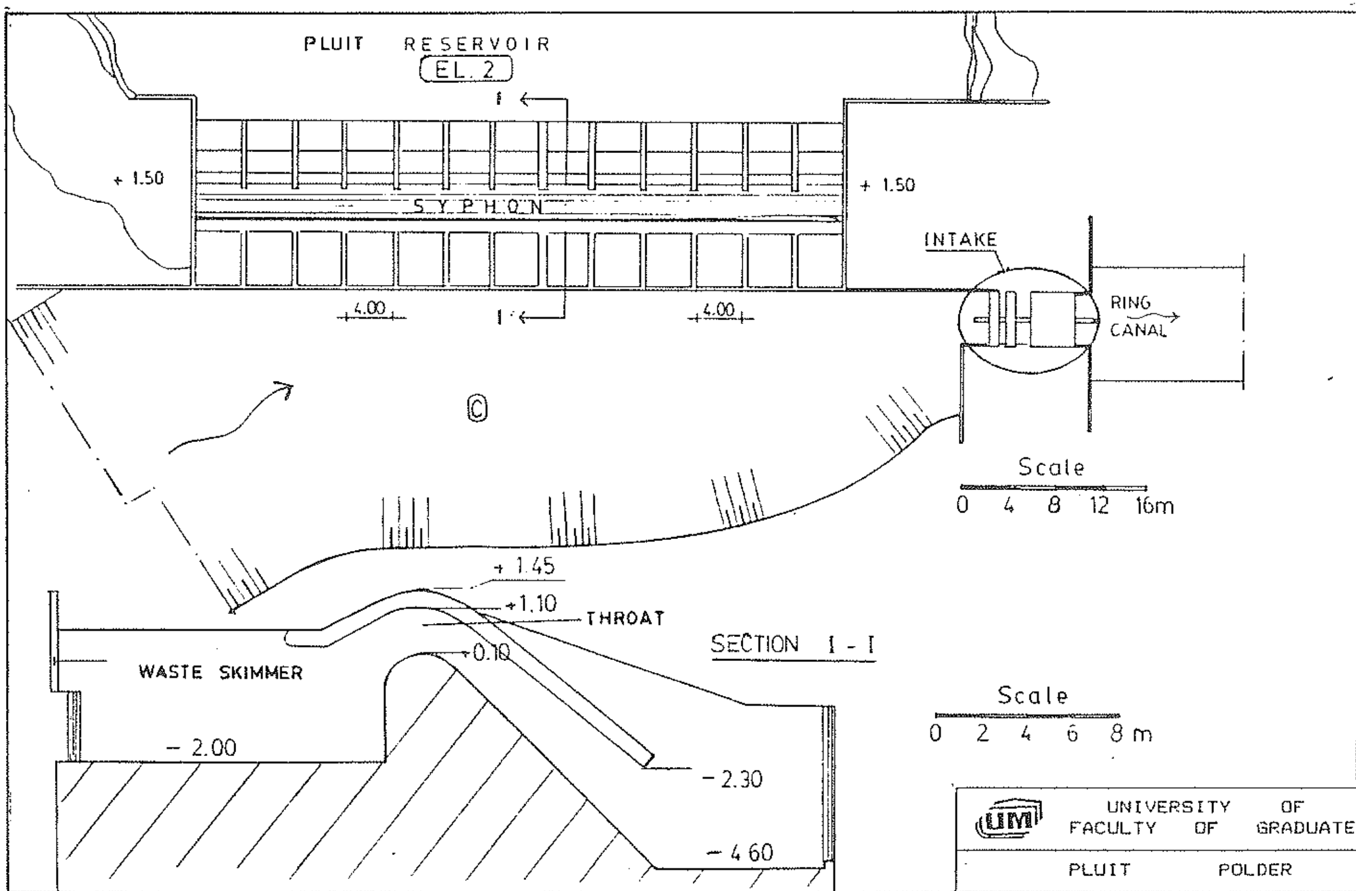
The Pluit Polder System consists of a reservoir, a ring canal, an intake, syphons, outlet gates and a pumping station. The general layout of the system is shown in Figure 2.2. With this system, the upstream flow will be diverted into the ring canal and pumped into the sea. If the flow from upstream exceeds the ring canal capacity or if the water elevation at point C is higher than the crest elevation of the syphon, the remaining flow will flow through the syphon and will be stored in the Pluit Reservoir. The water in the reservoir will then be pumped into the sea as outflow capacity permits. (The detailed explanation of the system operation is given in Sections 5.2 and 5.4.2.)

3.2 DATA AND SPECIFICATION

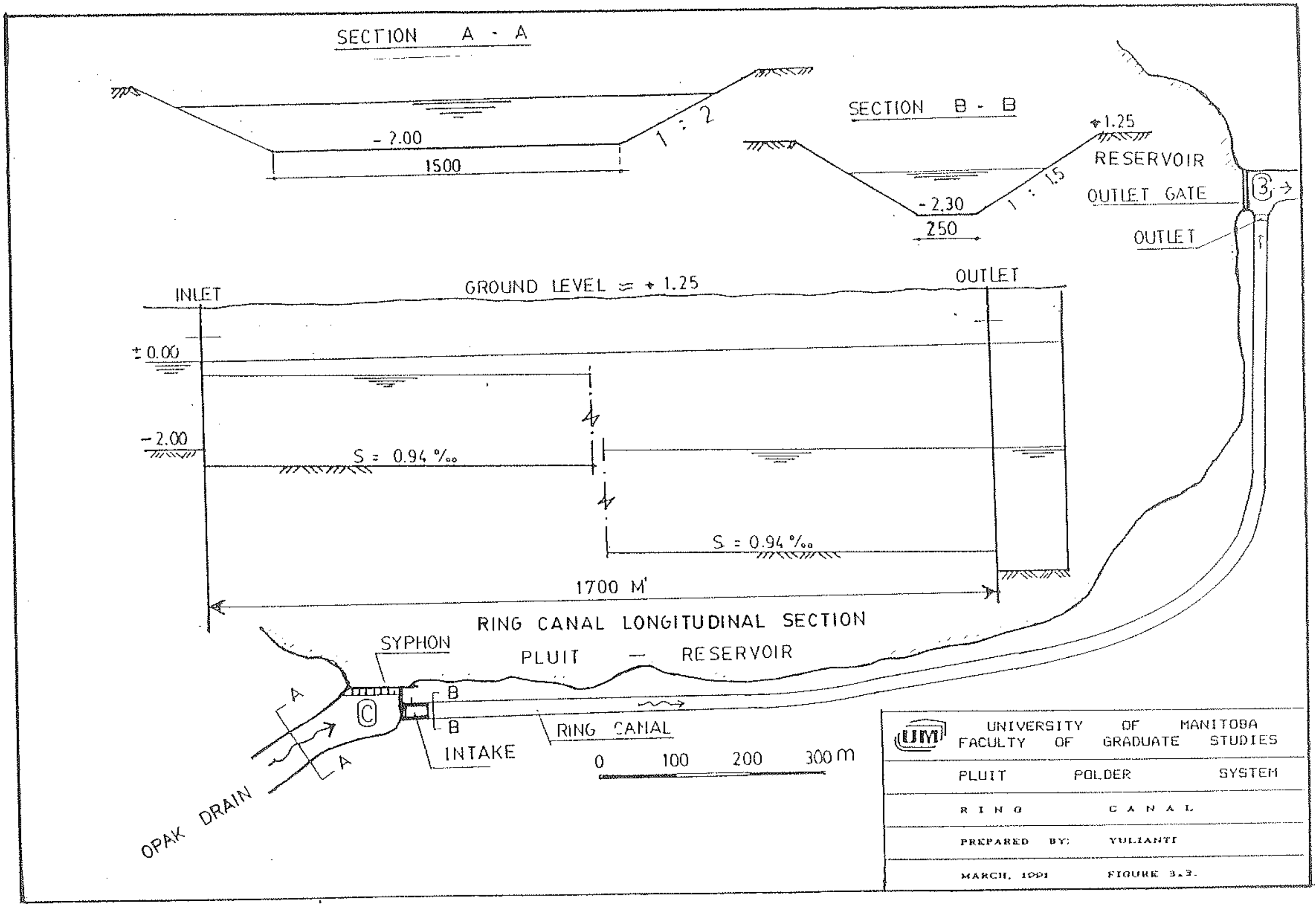
The present condition of the Pluit Polder system has the following specification (see Figures 2.2 and 3.1 - 3.4).



UNIVERSITY OF MANITOBA FACULTY OF GRADUATE STUDIES		
PLUIT	POLDER	SYSTEM
RESERVOIR	CAPACITY	CURVE
PREPARED BY:	YULIANTI	
MARCH, 1991	FIGURE 3.1.	



UNIVERSITY OF MANITOBA FACULTY OF GRADUATE STUDIES	
PLUIT POLDER SYSTEM	
SYPHON	
PREPARED BY: YULIANTI	
MARCH, 1991	FIGURE 3. 2.


SECTION A - A
- 2.00
1500
1 : 2
SECTION B - B
+ 1.25
- 2.30
250
1 : 1.5
RESERVOIR
OUTLET GATE
OUTLET
3
INLET
GROUND LEVEL ≈ + 1.25
OUTLET
± 0.00
- 2.00
S = 0.94 ‰
S = 0.94 ‰
1700 M'
RING CANAL LONGITUDINAL SECTION
PLUIT — RESERVOIR
SYPHON
C
A
A
B
B
INTAKE
RING CANAL
0 100 200 300 m
OPAK DRAIN
UM
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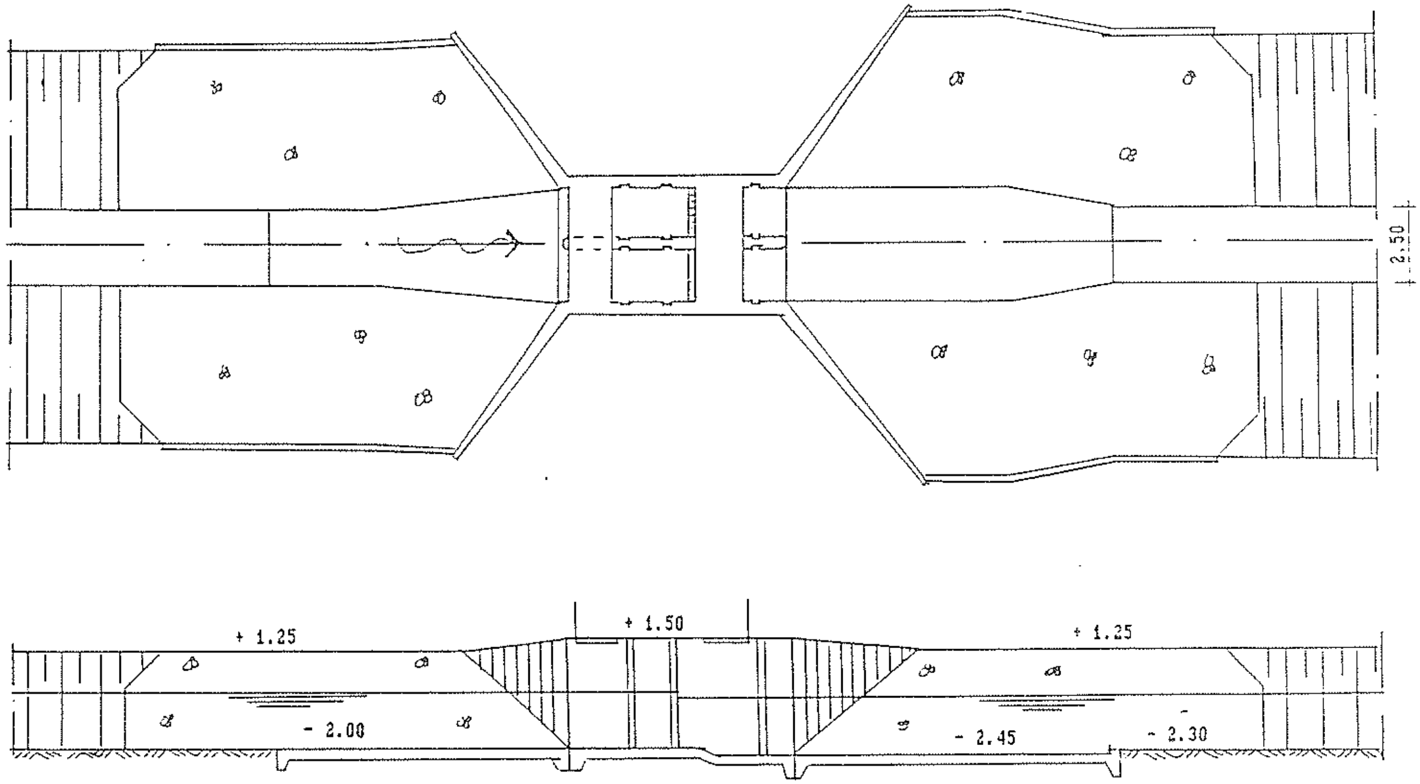
PLUIT POLDER SYSTEM
R I N G C A N A L
PREPARED BY: YULIANTI
MARCH, 1991 FIGURE 3.3.



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PLUIT POLDER SYSTEM

INTAKE

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MARCH, 1991 FIGURE 3.4.

3.2.1 HYDRAULIC DATA

1. Opak Drain

The Opak Drain diverts water from the drains upstream to the inlet structures. The inlet structures are the syphons and the ring canal. The dimensions of the drain are as follows:

Bottom width	-	15.00 m
Slope	-	0.00074
Side slope	-	1:2

During normal conditions, the flow from upstream will be diverted to the ring canal through the Opak Drain and the intake in front of the ring canal. The calculation of the capacity of the Opak Drain is shown in Appendix C.1.

2. Syphon

The function of the syphon is to discharge water from upstream into the reservoir if the capacity of the ring canal is exceeded. The Pluit Syphon is shown in Figure 3.2 and has the following dimensions:

Effective crest length	-	13 @ 4.00 m
Crest level	-	+ 0.10 m
Depth of throat	-	1.00 m
Elevation of the top entrance	-	+ 0.25 m
Elevation of the downstream leg	-	- 2.30 m
Top elevation	-	+ 1.45 m

A waste skimmer is located in front of the syphon to reduce the amount of waste entering the reservoir and more importantly to insure the operation of the syphon. (A detailed explanation of the syphon is given in Sections 5.2.3 and 5.3.5.)

3. Ring Canal

The ring canal, shown in Figure 3.3 diverts water from upstream to the forebay when the water level upstream is higher than + 0.00. This limitation is based on City regulations. The dimensions of the ring canal are:

Bottom width	-	2.50 m
Slope	-	0.00094
Side slope	-	1:1.50
Maximum capacity	-	16 m^3/s
Length of ring canal	-	1700 m

4. Intake

The flow from upstream to the ring canal is regulated through the automatic intake gates. These gates will close automatically if the water level upstream is lower than + 0.00 or if the water level in the ring canal is higher than the water level upstream. If the water level in the ring canal is higher than the water level upstream, backwater flow exists, thus increasing the elevation upstream. The intake is shown in Figure 3.4 and its dimensions are as follows:

Intake width - 2 gates @ 1.75 m

Automatic gates that maintain upstream water level elevation of +0.00

5. Reservoir Outlet Gates

The flow from the Reservoir is diverted to the forebay through the three reservoir outlet gates if the elevation in the reservoir is higher than the elevation in the forebay. The outlet gates are flap gates with the following dimensions:

Net width per gate	-	2.00 m
Height of gate opening	-	2.00 m

6. Pumping station

The water in the forebay is pumped to sea if the elevation in the forebay is higher than -1.90 m due to pump limitation. The following is the pump description:

Number of pump	-	4
Capacity per pump	-	4 m^3/s

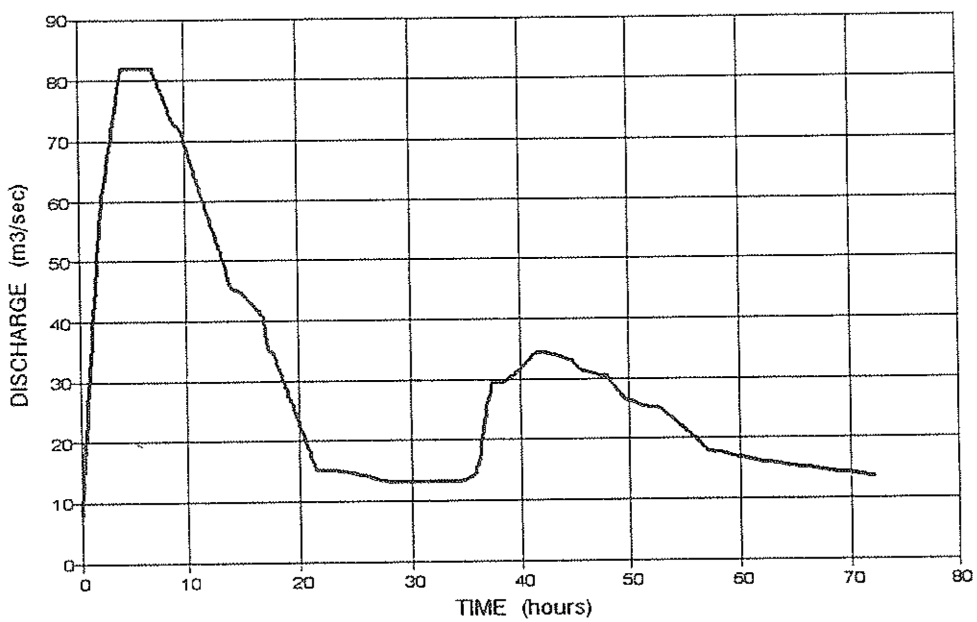
3.2.2 HYDROLOGIC DATA

The following data are available:

1. Three days of Actual streamflow were provided. The available streamflow data records are three days of hourly streamflow records (see Figure 3.5). These data show that the combination of two successive storms in a 3 day period that have actually occurred at that time. The first peak was about 82 m^3/s and the second peak was about 35 m^3/s. Time lag between the two successive storms was about 15 hours. As a random data, these data represent the common streamflow that caused yearly flood in the Pluit Drainage Area. Therefore, these data as an important data was used in the study to simulate the actual condition of flooding in this area.

2. The 25-year and 2-year return period hyetographs of hourly rainfall (Figures 3.6 and 3.7). These data are obtained from the previous study (NEDECO, 1973). The hyetographs represent a single storm event. (The detailed information of the hydrologic data is given in Section 4.2.1.)

FLOOD HYDROGRAPH
3 DAYS ACTUAL RECORDED STREAMFLOW DATA



UM	UNIVERSITY OF MANITOBA FACULTY OF GRADUATE STUDIES
PLUIT POLDER SYSTEM	
THREE DAYS ACTUAL STREAMFLOW DATA	
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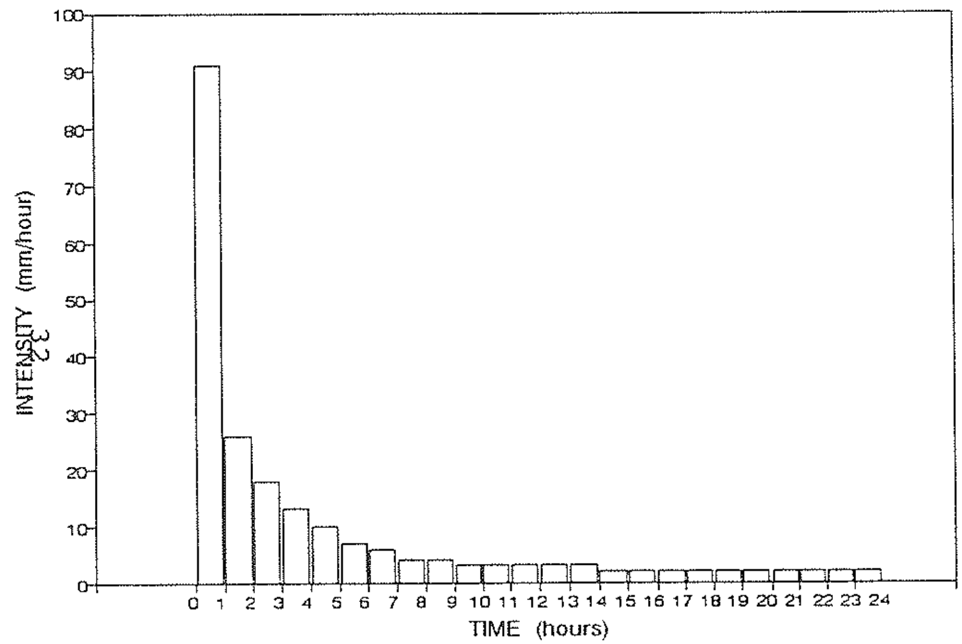
MARCH, 1991	FIGURE 3.5.

RAINFALL HYETOGRAPH
25 Year Return Period



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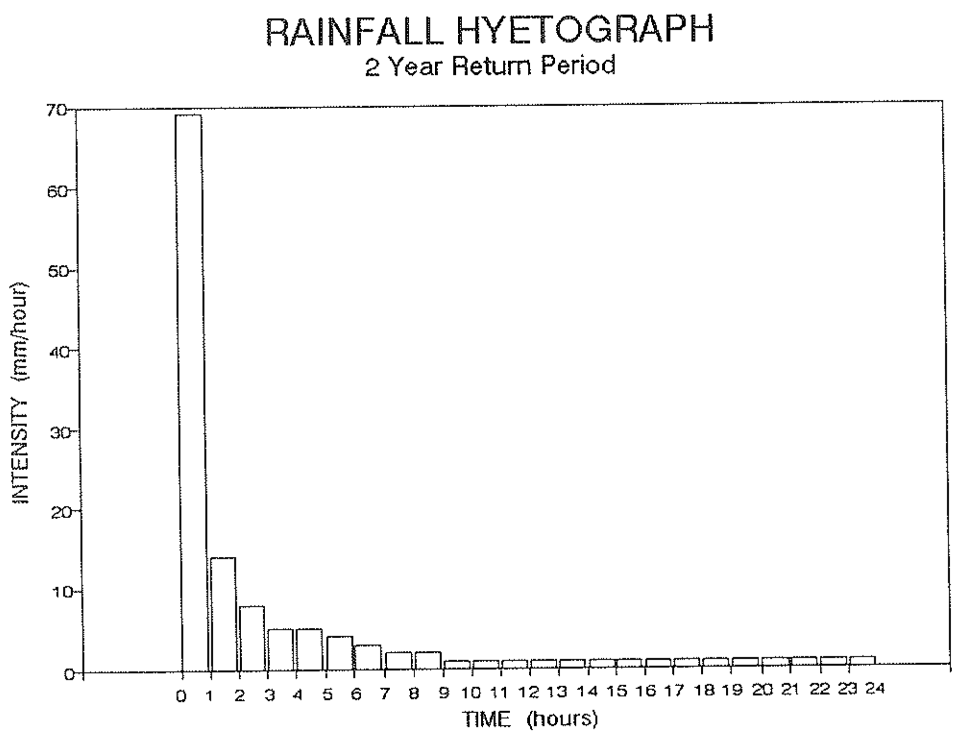
PLUIT POLDER SYSTEM

HYETOGRAPH OF 25 YEAR RETURN PERIOD

PREPARED BY: YULIANTI

MARCH, 1991 FIGURE 3.6.



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PLUIT POLDER SYSTEM					
HYETOGRAPH OF 2 YEAR RETURN PERIOD					
PREPARED BY: YULIANTI					
MARCH, 1991	FIGURE 3.7.				

CHAPTER 4

HYDROLOGY

4.1 CLIMATE

The climate of Jakarta is tropical. The wet season typically occurs from November to April and the dry season occurs from May to October. The months of January and February have the most intense rain. The average yearly rainfall in the project area is about 2000 mm/year. The greater part of the yearly rainfall (approximately 80%) takes place during the wet season, with predominantly northwestern winds. The average recorded temperature in Jakarta is approximately 28 ^{0}C with a maximum of 34 ^{0}C and a minimum of 22 ^{0}C.

4.2 AVAILABLE HYDROLOGICAL DATA

The available hydrological data are:

1. Rainfall data
2. Streamflow data
3. Tidal data

4.2.1 RAINFALL

The rainfall data available are hourly hyetographs of the 25 year and the 2 year return period (Figures 3.6 and

3.7). These data are obtained from NEDECO (1973). The recorded data are accurate representations of a single storm event. However, streamflow data indicate that the assumption that successive storms do not interact is invalid. When such storms interact they can greatly increase the resulting flood durations and volumes. The flood volume is critical because once the polder has been filled, this system has very little capacity to pass a flood. As a result further investigation of two or more successive storms is essential. The risk of two or more successive storm events could be analyze if more continuous rainfall data or streamflow data in this area is available. The rainfall or streamflow data should include the peak and duration of the successive storms and time lag between the storms.

4.2.2 STREAMFLOW

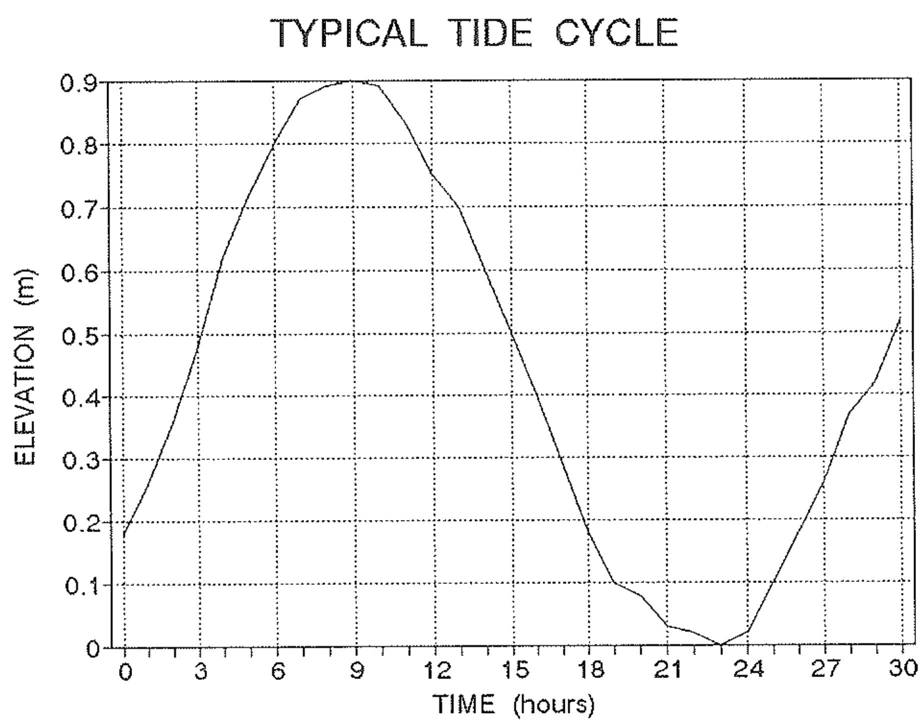
The available streamflow record consists of three day of hourly stream flow data (Figure 3.5). These data are considered to be adequate since they represent the actual streamflow. They are used to simulate the current condition and show that a combination of two successive floods has actually occurred. The first peak flow of this actual data is about the same as the 2-year flood based on calculated runoff using the rainfall data (i.e., the hourly hyetograph). This means that the flood resulting from the first storm using this recorded data can represent the flood with a two year return period.

4.2.3 TIDAL FLOW

The tidal movement in the Java Sea at Jakarta is mainly a single day tide with one high and one low tide for each 24 hours. A typical tide cycle is shown in Figure 4.1 (taken from the Tanjung Priok station). Other tidal data, includes three days of actual recorded tidal data taken at Pasar Ikan (see Figure 2.2) at the same time as the three days of streamflow data shown in Figure 4.2. This data is very important because of the possibility of diverting water directly to the sea without pumping while the tide is low. Therefore the consideration of the effects of tides on the release of collected storm water runoff is an important factor in assessing the operation of the polder. The extreme condition occurs if the peak of the tidal (high tidal elevation) coincide with the peak of the inflow hydrograph. This extreme condition was taken in the simulation model.

4.2.4 SEDIMENT

Sediment data are needed to predict the amount of sedimentation in the canal and reservoir. An ideal channel should not be subject to scour or siltation, and all sediment transported into should be carried out again. In this drainage area, the source of flow is mostly surface runoff from residential and industrial areas where an insignificant



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PLUIT POLDER SYSTEM

TYPICAL TIDE

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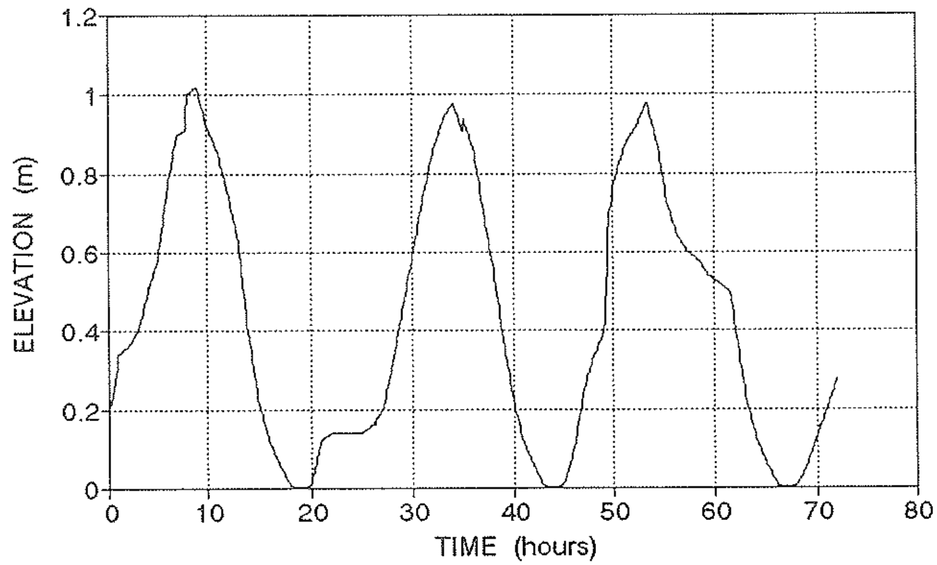
MARCH, 1991 FIGURE 4.1.

PLUIT POLDER SYSTEM
TIDAL DATA



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PLUIT POLDER SYSTEM

THREE DAYS ACTUAL TIDAL DATA

PREPARED BY: YULIANTI

MARCH, 1991 FIGURE 4.2.

volumes of sediment are expected. Therefore sediment is not considered to be a factor in this study. However, sedimentation should be considered in a future study to confirm this assumption.

4.2.5 GARBAGE

Residential garbage is a major problem in this particular drainage area. The presence of garbage in the canal encourages growth of aquatic plants that increase the canal roughness. This results in decreased canal capacity. The presence of garbage is accounted for in this study. As no garbage data is available, reductions in flow capacity of 10% and 25% due to garbage have been assumed to account for garbage in the calculation of the polder system. Since garbage is very important in the system, further field investigation is needed however.

4.3 RUNOFF STUDIES

4.3.1 GENERAL

Two methods can be used to calculate the runoff data from rainfall data, namely, the rational method and the unit hydrograph method. Approaches such as Melchior's, Weduwen's, and Haspers's rational methods are well known in Indonesia for determining runoff. These methods estimate only the peak flow, while the unit hydrograph method estimates the flood volumes in terms of an entire hydrograph as well as flood peaks. In a polder system, where polders provide detention storage, the unit hydrograph method is preferred because of the importance of flood volumes. Many procedures have been developed to construct the unit graph, such as these developed by Snyder Alexeyev, Nakayasu, and the U.S. Soil Conservation Service (SCS) synthetic hydrograph approach, which is internationally known, was used in developing the unit hydrograph in this study. The SCS method is simply a triangle with rainfall duration D, time to peak Tp, time base Tb, and peak flow Qp (see Figure 4.3).

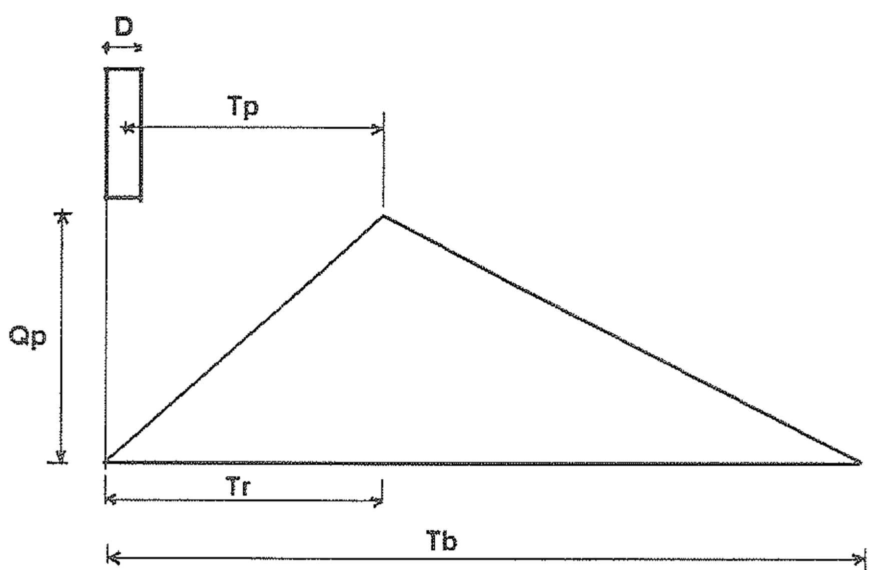


Figure 4.3 SCS Triangular Unit Hydrograph

This method requires:

1. Rainfall data
2. Hydrologic and topographic properties of the catchment area.

For the rainfall data, the total maximum precipitated amount of rainfall recorded from the beginning of the precipitation period should be known for a certain return period (e.q., the 2 and 25 year return periods). The two year return period is used to calculate the capacity of canals and hydraulic structures in the small (micro) drainage system, while the 25 year return period is used to calculate the capacity of the canal and hydraulic structures in the main (macro) drainage canal. With regard to the hydrologic and topographic data, the following information must be available:

1. The size of the catchment area
2. The shape of the catchment area
3. The length of the water course considered
4. The difference in the level between the highest and the lowest point of the water course and
5. The nature of the catchment area.

4.3.2 CALCULATION METHOD

In the calculation of runoff, the rainfall data is taken from a previous study (Nedeco, 1973) in the form of a hyetograph. Two hyetographs data are available, namely, hyetographs with return periods of 2 years and one with a return period of 25 years (Figures 3.6 and 3.7). The hyetograph is divided into portions of rainfall with an "equal" time base. Each portion yields a unit hydrograph. The design hydrographs are obtained by multiplying the ordinates of the unit hydrographs by the value of the design storm, which is computed using the Curve Number Method (see Figure 4.4). Adding the volume of all resulting hydrographs produces the required total hydrograph. The time of concentration, peak time and time base can be calculated using the following equations[14]:

1. Time Concentration (Tc) in hours

$$Tc = (0.869\ L^3\ /H)^{0.385} \qquad (4\text{-}1)$$

where:

L is the length of watercourse in km

H is the difference in height in m

2. Time to Peak (Tp) in hours

$$Tp = D/2 + 0.6\ Tc \qquad (4\text{-}2)$$

where: D is the time interval

3. Time base (Tb) in hours

$$Tb = 2.67\ Tp \qquad (4\text{-}3)$$

4. Peak Discharge (Qp) in m^3/s

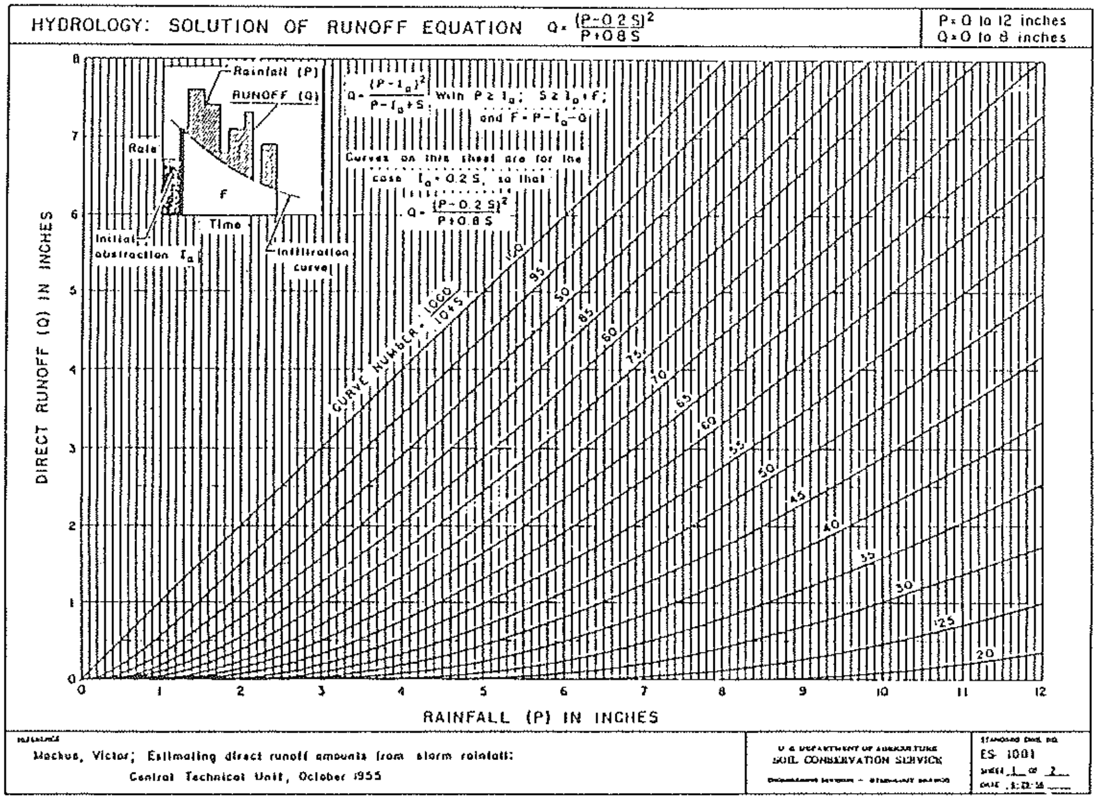
$$Qp = (2.081 \times CA \times P)\ /\ Tp \qquad (4\text{-}4)$$

where:

CA is catchment area (km^2)

P is effective rainfall in mm

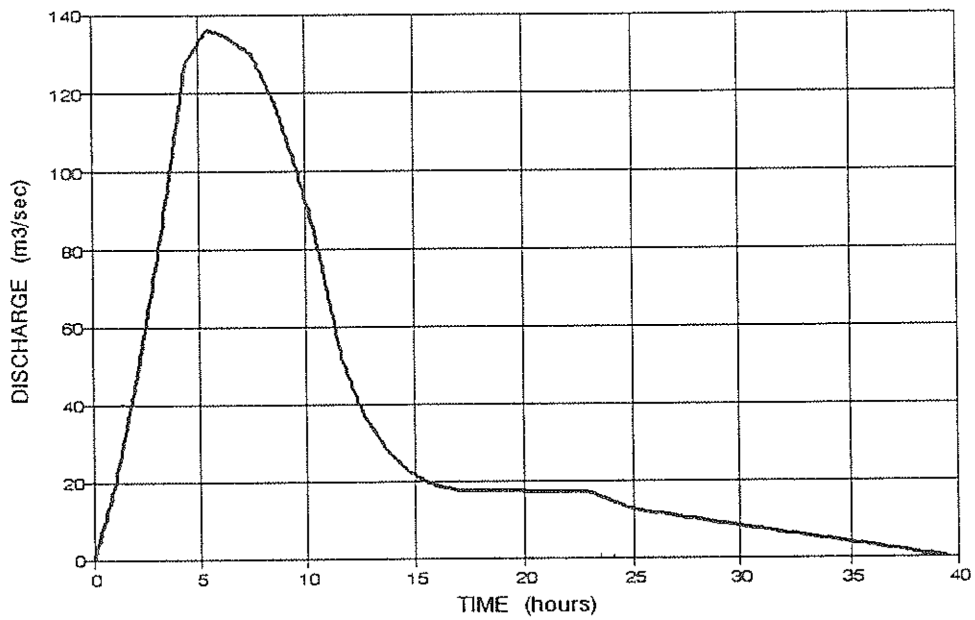
The detailed calculations are shown in Appendix A. The resulting flood hydrographs are shown in Figures 4.5 and 4.6.



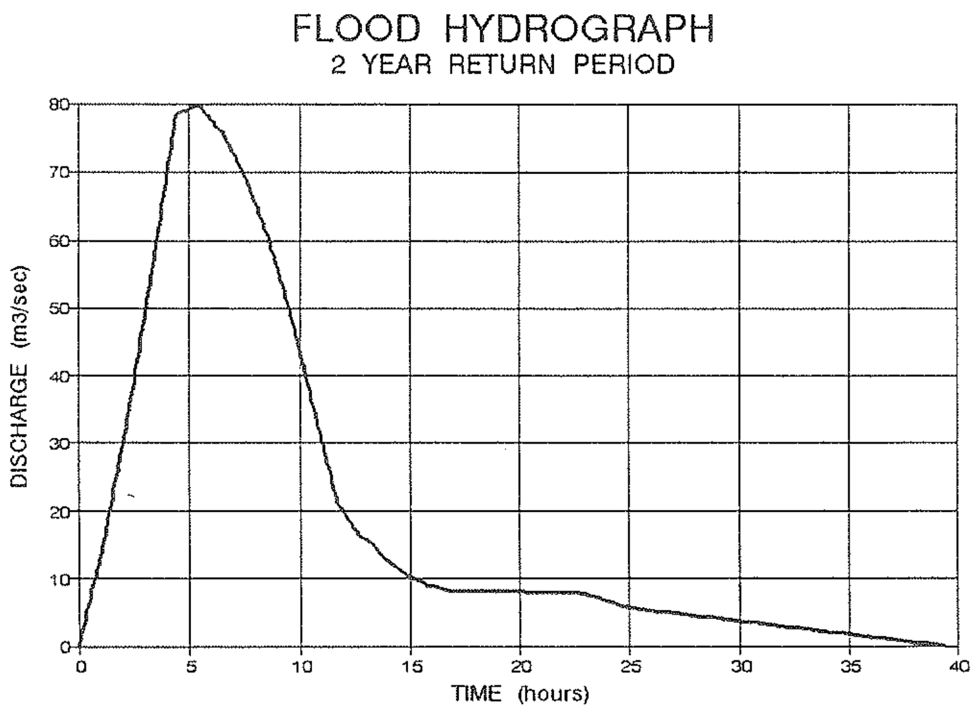
SOURCE : PHILIP B. BEDIENT and WAYNE C. HUBER,
"HYDROLOGY AND FLOODPLAIN ANALYSIS"

UNIVERSITY OF MANITOBA FACULTY OF GRADUATE STUDIES	
PLUIT POLDER SYSTEM	
RUNOFF-RAINFALL RELATIONSHIP FOR DIFFERENT CURVE NUMBER	
PREPARED BY:	YULIANTI
MARCH, 1991	FIGURE 4.4.

FLOOD HYDROGRAPH
25 YEAR RETURN PERIOD



UM	UNIVERSITY OF MANITOBA FACULTY OF GRADUATE STUDIES		
	PLUIT	POLDER	SYSTEM
25 - YEAR		FLOOD	HYDROGRAPH
	PREPARED BY:	YULIANTI	
	MARCH, 1991	FIGURE 4.9.	



UM	UNIVERSITY OF MANITOBA FACULTY OF GRADUATE STUDIES		
	PLUIT	POLDER	SYSTEM
2 -	YEAR	FLOOD	HYDROGRAPH
	PREPARED BY:	YULIANTI	
	MARCH, 1991	FIGURE 4.6.	

CHAPTER 5

METHODOLOGY

5.1 GENERAL

A computer model was developed to simulate the operation of the Pluit Polder System. The development of the computer model is discussed in this chapter. First, the current operating method was simulated with this model by routing the inflow hydrographs through the system. The inflow hydrographs consist of:

1. Three days of actual streamflow data as a random data
2. The calculated flood hydrograph for a 25 year return period
3. The calculated flood hydrograph for a 2 year return period

From these three cases, the current operation of the Polder was simulated showing the reservoir elevations and the extent of flooding. Based on these current simulation results, several operation alternatives were investigated and were evaluated in terms of improvement to the operation of the system. Finally, sensitivity analyses were carried out to assess the accuracy or sensitivity of the model to the possible changes in inflow hydrographs.

5.2 IDENTIFICATION AND DISCUSSION OF SIGNIFICANT FEATURES OF THE PLUIT POLDER SYSTEM

5.2.1 OPAK DRAIN - UPSTREAM CHANNEL

The Opak Drain, as the Main Drain in the Pluit Polder System, carries the water from the Upstream Drains to the inlet structure, syphon and ring canal (Figure 2.2). The Manning Equation was used in the calculation of the flow in the Opak Drain. Using the Opak Drain characteristics, the capacity of the channel was calculated to be 135.08 m^3/s (for the detailed calculation see Appendix C.1). This capacity can be reached only if the channel is clean, (i.e., free of any obstructions to flow). The roughness coefficient ($n=0.02$) applies to a channel excavated in earth under clean conditions (Chow, 1985). In fact, there is always garbage in the Opak Drain. This garbage promotes the growth of aquatic plants which increases the channel roughness. In this study, the capacity reduction due to garbage was taken to be between 10% and 25%. The reduction of 25% of capacity, or $Q = 101.3$ m^3/s, leads to an increase in the roughness number to $n = 0.028$. This roughness number is the same as the roughness number applicable to short grass (Chow, 1985). The capacity reduction of 10% from garbage was also considered in the study under the assumption that there is a regular cleaning program in the Opak Drain (it is not possible to ensure that the Drain is always clean enough to count on no reduction in capacity).

Based on further field investigations, the reduction factor could be adjusted.

5.2.2 THE RAYA PLUIT SELATAN BRIDGE UPSTREAM OF THE RESERVOIR

The Raya Pluit Selatan Bridge acts as a control section in the Opak Drain. The bridge abutments and access road restrict flow in the Opak Channel causing amplification of flood waters upstream and inundation of these areas. Records show that the bridge and its abutments have never been overtopped. Considering the available bridge and channel dimensions, (see Figure 2.2 flood routing was performed to account for the attenuating effect of the bridge on the operation of the Pluit Polder System. The area between the bridge and the inlet structure is also susceptible to flooding. Because the area is not developed, the damage from flooding this area is minimal. (The calculation of the extent of flooding in this area is also included in the program for evaluation of the hazards of future development of this area.)

5.2.3 SYPHON (FIGURES 2.2 AND 3.2)

The syphon in the Pluit Polder System consists of 13, 4.0 m wide channels. The depth of the syphon throat is 1.00 m. The crest elevation is 0.10 m and the total effective crest length is 52.00 m. A waste skimmer, is located in front of the syphon to reduce the amount of incoming waste passed to the reservoir.

The amount of flow through the syphon depends on the water levels upstream and downstream of the syphon. When the water level upstream of the syphon is below the syphon crest, no flow occurs in this condition. When water level upstream of the syphon is higher than the crest of the syphon, the flow through the syphon begins. The flow never goes over the top of the syphon, therefore there is no flow above or over the syphon. There are four types of syphon operation. These are shown in Figure 5.1 and are described as follows:

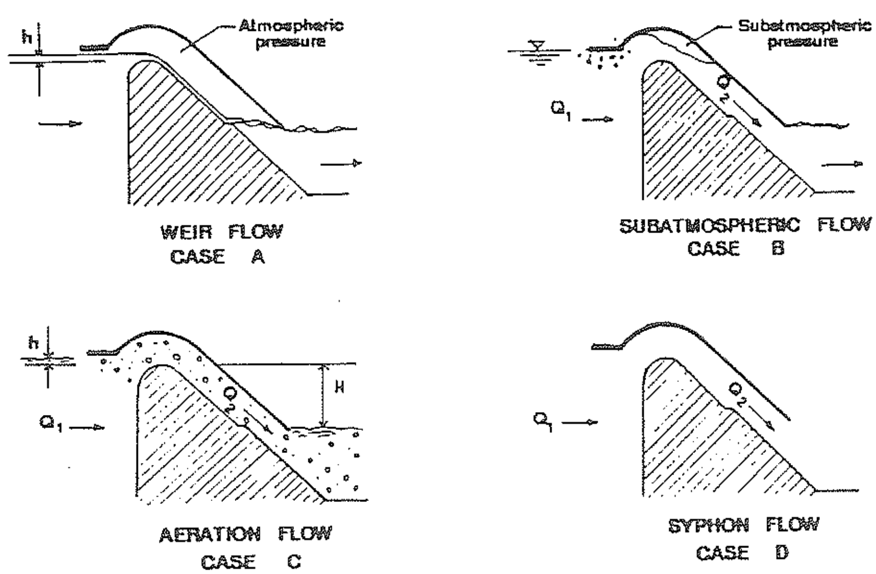


Figure 5.1 Syphon Flow Condition

- Weir Flow (Case A)

  Weir flow occurs when the upstream water level is above the syphon crest but below the top of syphon entrance. Weir flow, can occur when upstream water level completely submerges the syphon entrance but the downstream of the syphon is not filled with water.

- Subatmospheric Flow (Case B)

  Subatmospheric flow occurs when both ends of the syphon are submerged but air is trapped in the throat of the syphon.

- Aeration Flow (Case C)

  Aeration flow occurs when both ends of the syphon are submerged but the Syphon is filled with mixture of air and water. The water levels upstream and downstream in this case are usually higher than in Case B.

- Syphon Flow (Case D)

  Syphon flow occurs when all air has been removed from the syphon. Therefore a vacuum condition occurs in the throat and the throat is fully filled with water.

Detailed explanations of these flow phases are presented in Appendix C.2. In this study only two conditions are considered, the weir flow and the syphon flow (Case A and Case

D), because the other two conditions are short term conditions, which occur in the adjustment of the flow from weir flow to full syphon flow. The weir flow occurs if no vacuum condition can be reached in the throat. The throat is not fully filled with water or the elevation down stream (in the reservoir) is lower than the syphon outlet, allowing air to enter the throat. Under weir flow conditions the flow through the syphon depends only on the upstream energy level. Once the throat is full and the water level downstream is higher than the downstream syphon outlet, so that no air can enter the throat, the discharge is then based on a Full syphon Flow Capacity Equation, which is:

$$Q = u\, A \sqrt{(2g\, H_g)} \tag{5-1}$$

where:

Q = discharge through the syphon in m^3/sec

u = coefficient due to losses

A = cross sectional area of the throat in m^2

g = acceleration due to gravity in m/s^2

H_g = gross head (Head of Water Energy Level at C - Reservoir Energy Level)

A detailed explanation of the weir flow and syphon flow is in Appendix C.2.

5.2.4 RING CANAL INTAKE STRUCTURE AND RING CANAL

The ring canal has a 1.5 m bottom width and a slope of 0.00094, while the side slopes are 1:1.50. The intake at the entrance to the ring canal consists of two 1.75 m wide, automatic gates. The bottom elevation of the Gates is - 2.00 m and the top elevation is +1.25 m. The length of the ring canal is 1.7 km. The maximum design discharge capacity of the Ring Canal is 16 m^3/s. In this study the influence of waste was included in the calculation by reducing the capacity of the ring canal intake structure and ring canal.

There are three conditions of operation of the ring canal intake structure:

- Close the Intake Gates if the upstream water level is lower than + 0.00 m. The automatic Intake Gates will maintain the upstream water level higher than +0.00 m, by closing the gates if the upstream water elevation is less than +0.00 m. This is due to a city regulation which requires the back up of the water level in the drainage system to the normal water level +0.00 m.
- Open the Intake Gates if the upstream water level is higher than +0.00 m and the water level in the forebay (downstream end of the ring canal) is lower than the water level in the ring canal. The flow in the ring canal is considered as uniform flow. The calculation is based on the Manning's Equation.
- Close the Intake Gates if the head differences across the intake is small. This is to prevent water flowing back into the Opak Drain. When downstream water level is higher than the water level in the ring canal, the

condition of flow changes from uniform flow to gradually varied flow, therefore a backwater calculation is then applied. As the water level downstream increases, the difference in water levels upstream and downstream decreases, which leads to a decrease in flow. The ring canal intake gates will close automatically if the head is very small (this is to protect against the flow back from the forebay to the Opak Drain) and hence no flow from upstream exists. At this stage the water from upstream areas will be diverted only to the reservoir through the syphon.

5.2.5 PLUIT RESERVOIR

The Pluit Reservoir has an area of about 83 hectares and a capacity of about 2.6 million m^3. The normal water level is -1.90 m and the maximum reservoir level is +1.00 m. The water elevation in the reservoir depends on the syphon discharge and the outlet discharge from the reservoir to the forebay. reservoir routing is used to calculate the reservoir elevations. If the time required to lower the water level in the reservoir after a flood is short, the reservoir can be expected to be ready for the next flood event. The lowering of the reservoir elevation depends on the outlet capacity, in this case the capacity of the forebay pumps. The larger the forebay pumping capacity, the faster the reservoir elevations can be lowered. The reduction of reservoir capacity due to garbage or sedimentation has not been included in the the computer model of the system. Further field investigations are needed to determine the actual total volume of sediment and garbage entering the reservoir. Knowing the total volume of sediment and garbage, the usable volume in the reservoir can be determined. In this study, as an initial condition, the starting elevation of the reservoir was taken to be -1.90 m reflecting the required normal elevation.

5.2.6 FOREBAY PUMPING STATION

The forebay pumping station consists of four pumps, each with a capacity of 4 m^3/s. Due to pump limitations, the pumps will operate only when the water level at the forebay is higher than -1.90 m.

If no polder system had been introduced, the ideal pump capacity to handle the design flood volumes would be approximately 135 m^3/s, the maximum discharge. The provision of this large pumping capacity is not efficient. The reason being that floods of such large magnitude do not occur frequently. As a result the full pumping capacity would be used infrequently. Since the cost of such a pump installation and operation is high, that the use of storage in a polder system is advantageous.

The pump capacity could be reduced if a pump becomes clogged by garbage, especially plastic waste. This condition is included in the sensitivity analysis by considering operation of three of the four pumps in the system.

5.3 DEVELOPMENT OF THE COMPUTER MODEL

The development of the computer model follows the procedure shown in the flowchart in Figure 5.2 - 5.4. The following subsections refer to the corresponding blocks in the flow chart. The location of structures in the polder system may be identified with reference to Figure 2.2.

5.3.1 THE DATA INPUT (BLOCK A)

The first part of the program consists of Data Input. The Data Input in this program is the hydrological data and the physical dimensions of the hydraulic structures. The hydrological data are given as an inflow hydrograph at Opak Drain at location U in Figure 2.2. The inflow hydrographs are specified given in hourly intervals. However, this time interval is too large to account for the water level fluctuations in the forebay. The area of the forebay is small compared to the area of the reservoir. In the forebay, a small increase in flow discharge results in a large fluctuation in the water level. In reality this large fluctuation will not occur, since the change is gradual. Using a large time interval gives a large fluctuation in the water level in the forebay and therefore less numerical stability. A smaller time interval gives better results, but requires more computer

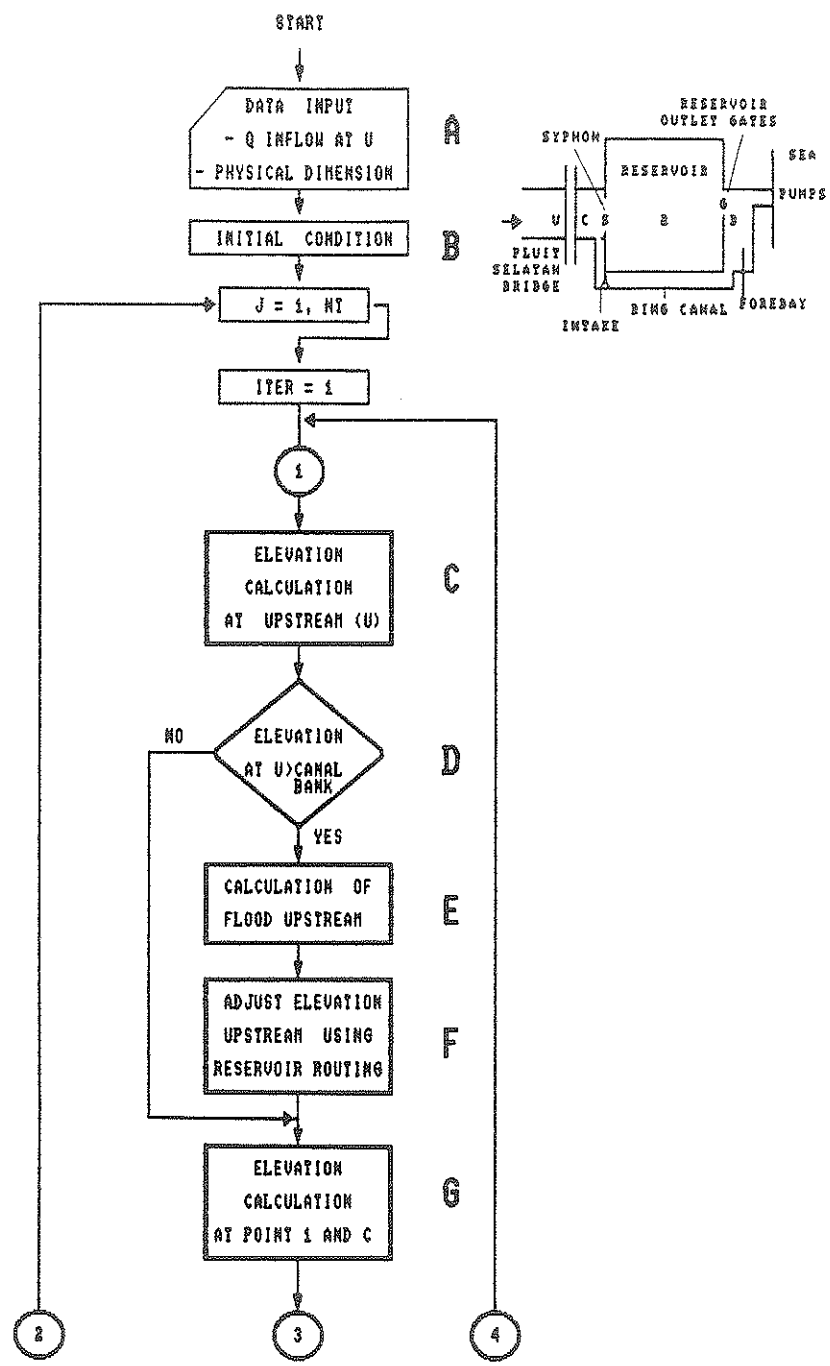


FIGURE 5.2 FLOW DIAGRAM FOR PLUIT POLDER SYSTEM BLOCK A - G

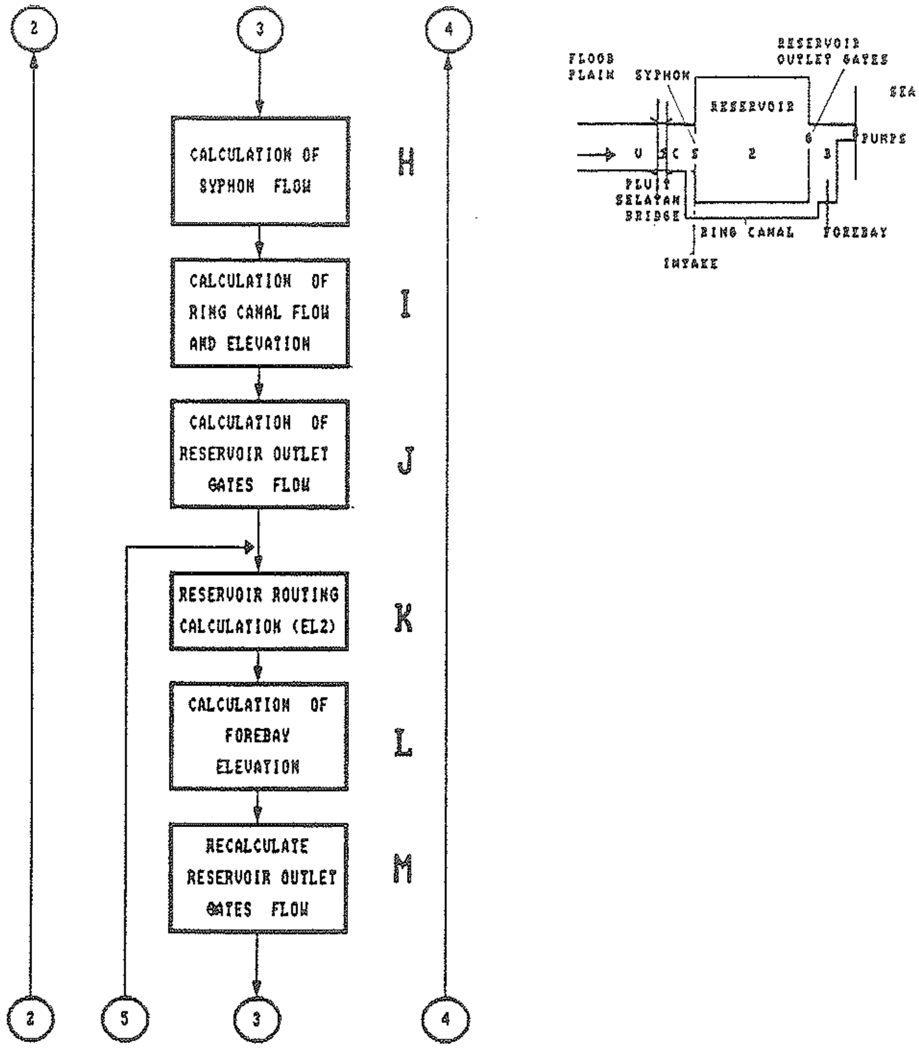


FIGURE 5.3 FLOW DIAGRAM FOR PLUIT POLDER SYSTEM BLOCK H - M

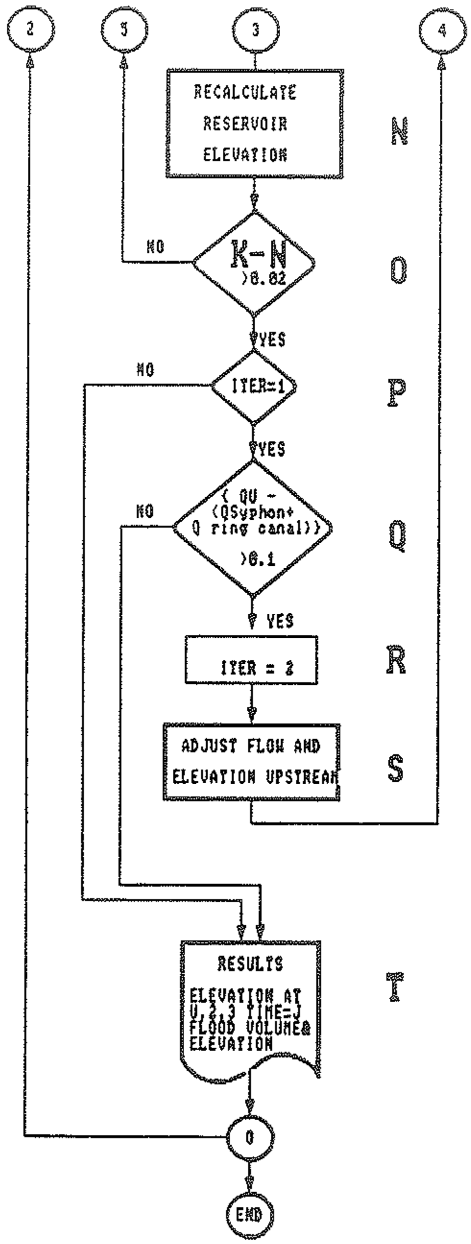


FIGURE 5.4 FLOW DIAGRAM FOR PLUIT POLDER SYSTEM BLOCK N - T

storage and execution time to run the model.

Based on the above consideration, the time interval in the model is taken to be 150 seconds.

Three sets of hydrograph data are considered in the calculation for the Pluit Polder System. These data are:

- Three days of actual record of streamflow data
- The calculated 25 year return period flood hydrograph
- The calculated 2 year return period flood hydrograph.

The data detailing the physical dimensions contain information about:

- The dimensions of the hydraulic structures
- Limitation on elevations
- Fixed coefficients: the roughness coefficients of the Opak Drain and the ring canal, the discharge coefficient of the syphon, and the loss coefficient of the gates.

5.3.2 INITIAL CONDITION (BLOCK B)

It is necessary to know the initial condition of the system prior to running the model to simulate the current operating method of the system. The initial conditions for this system are specified as the initial water elevations in the Opak Drain, the reservoir and the forebay. In addition, the volume and elevation of any flooding at the beginning of the time interval, and discharge through the ring canal, syphon, reservoir outlet gates, and forebay pumps must also be specified.

5.3.3 CALCULATION OF THE WATER ELEVATIONS (UPSTREAM OF THE RAYA PLUIT SELATAN BRIDGE; BLOCKS C, D, E, F).

Using the input data and initial conditions, the next step is the calculation of the elevation upstream of the bridge of the Opak Drain. The elevation of the channel is found using Manning's Equation assuming uniform flow in the channel. The upstream elevation is then checked against the bank elevation. Two conditions upstream of the Pluit Selatan Bridge were considered:

1. **No Flooding**

   If the water elevation in Opak Drain upstream of the Bridge is lower than the bank elevation, no flood has occurred. If no flood has occurred due to the incoming flow, the calculation is based on uniform flow. Therefore Manning's Equation was applied. The Manning's roughness coefficient was taken from Chow (1985). In the Opak Drain, the increase in the roughness due to garbage, reduces the capacity of the channel. Assuming no reduction in capacity due to garbage in the canal, the existing (design) conditions correspond to a roughness coefficient of 0.02. As discussed in Section 4.2.5, the garbage in the channels provides organic nutrients to the water. The increased nutrient load results in plant growth on the channel slope. This growth reduces the discharge capacity of the channel by increasing the roughness coefficient. Considering channel discharge capacity reductions of 10% and 25% (see Appendix C.1), the reduction in capacity due to garbage, the roughness coefficient becomes 0.023 or 0.028 respectively.

2. **Flooding**

When flooding occurs, adjustment to the upstream elevation must occur before calculations can proceed. In this condition, the area upstream of the bridge is assumed to be a reservoir. No topographical data is available for this area. However, it is known that the area is flat. The area, which experiences regular flooding, is taken as the flooded area in the calculations. The boundaries of this area are the streets with higher elevations. Based on the information from the Jakarta Flood Control Project, the inundated area is about 120 hectares. Two flooding conditions that were considered in the model are shown in Figure 5.5 and are described below:

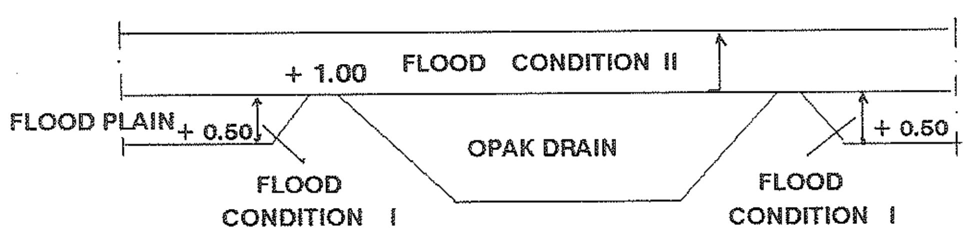


Figure 5.5 Flood Conditions in the Flood Plain

Flood Condition I

Since the elevation of the land is below the level of the channel bank, when flooding first occurs, the flood is due to overflow of the channel, and therefore, any flow greater than the channel capacity contributes directly to flooding. At the condition of flood due to overflow, the elevation upstream of the bridge for the next calculation is the maximum elevation of the Opak Drain.

Flood Condition II

Above the channel bank elevation the characteristics of the flood flows change. At this stage, the area upstream of the bridge is considered as a detention basin or reservoir. The flows are routed through this detention basin. The purpose of this routing is to determine the elevation upstream of the bridge as a function of storage in the basin for a given period of inflow (to calculate the elevation upstream the bridge due to flood, Block F). The basic equations are the Continuity Equation and the Storage Equation. The second order Runge-Kutta technique was used to solve the ordinary differential Equation involved. The Continuity Equation is as follows:

$$dV/dt = Q_{in}(t) - Q_{out}(H) \qquad (5\text{-}2)$$

where:

V = volume of water in storage in the basin

Q_{in} = inflow to the basin

Q_{out} = outflow from the basin

t = time

H = head

The change in volume with time can be expressed in terms of the changes in depth as follows:

$$dV = A_u(H)\,dH \tag{5-3}$$

where:

A_u = upstream area

u = refers to location U in Figure 2.2

H = head

Substituting this Equation (5-3) into the Continuity Equation (5-2) gives:

$$A_u(H)\,dH/dt = Q_{in}(t) - Q_{out}(H) \tag{5-4}$$

The changes in water level:

$$dH/dt = (Q_{in}(t) - Q_{out}(H))/A_u(H) \tag{5-5}$$

$$dH/dt = f(H_n, t_n) \tag{5-6}$$

$$\Delta H = f(H_n, t_n)\ \Delta t \tag{5-7}$$

$$H_{n+1} = H_n + \Delta H \tag{5-8}$$

In the second order Runge-Kutta method, the H is calculated as the average of H at the beginning and the end of the time interval, which is:

$$\Delta H = (\Delta H_1 + \Delta H_2)/2 \tag{5-9}$$

The change in H as a result of change in basin volume becomes:

$$\Delta H_1 = (Q_{in}(t_n) - Q_{out}(H_n))\ \Delta t/\ A_u(H_n) \tag{5-10}$$

$$\Delta H_2 = (Q_{in}(t_n+\Delta t) - Q_{out}(H_n+H_1))\Delta t/A_u(H_n+\Delta H_1) \qquad (5\text{-}11)$$

The inflow was taken from the inflow hydrographs upstream of the reservoir. The outflow was taken as the total flow to the syphon and ring canal intake and governs conditions at the beginning of the time interval. These conditions are calculated based on the value of H at point C in Figure 2.2 immediately downstream of the Raya Pluit Selatan Bridge which is a common point between the ring canal intake and the syphon.

The adjustment of the elevation upstream of the bridge is performed as follows (Block F):

- At the condition of no flood, no adjustment is needed.
- At the condition of flood due to overflow (Flood Condition I), the elevation upstream of the bridge for the next calculation is the maximum elevation of the Opak Drain.
- At the condition of flood above the bank elevation (Flood Condition II), the elevation upstream of the bridge for the next calculation is the Flood Elevation resulting from the reservoir Routing in Flood Condition II.

5.3.4 CALCULATION OF ELEVATION DOWNSTREAM OF THE PLUIT SELATAN BRIDGE (BLOCK G).

To determine the elevation downstream of the bridge (point C in Figure 2.2), the inflow is routed through the Opak Drain from the upstream. First, the elevation at Point 1 in Figure 2.2 (at the location of the Pluit Selatan Bridge) is calculated. The bridge abutment and access road constricts the flow in the Opak Drain, and results in changes in the water level at this section. The calculation is done by using the Energy Equation at points C and 1 (the axis of the bridge pier). Refering to Figure 5.6, the Energy relationships between points C and 1 are:

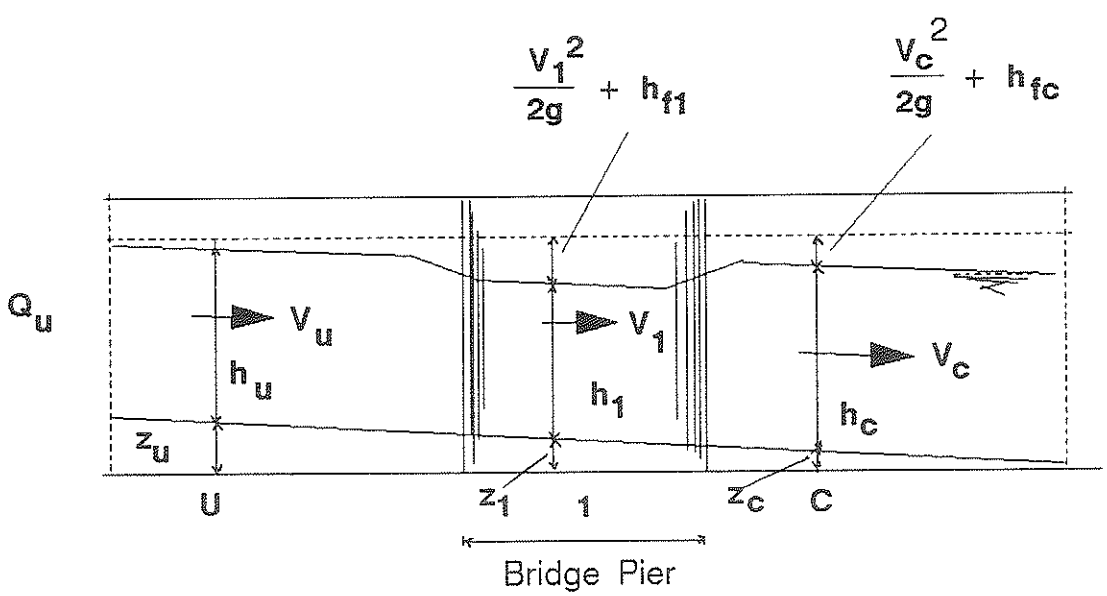


Figure 5.6 Opak Drain at Location of Pluit Selatan Bridge

$$H_u = H_1 \tag{5-12}$$

$$z_u + h_u + V_u^2/2g = z_1 + h_1 + V_1^2/2g + h_{f1} \tag{5-13}$$

$$H_1 = H_c \tag{5-14}$$

$$z_1 + h_1 + V_1^2/2g = z_c + h_c + V_c^2/2g + h_{fc} \tag{5-15}$$

where:

z_u = bed elevation at location U

h_u = water depth at location U

V_u = velocity at location U

z_1 = bed elevation at location 1

h_1 = water depth at location 1

V_1 = velocity at location 1

h_{f1} = energy losses due to contraction

z_c = bed elevation at location C

h_c = water depth at location C

V_c = velocity at location C

h_{fc} = energy losses due to expansion

The elevation at C is determined by solving the above equation using an iterative procedure as follows:

1. Using the known data at location U, determine the water depth at point 1 and calculate the Energy heads at U and 1.
2. Compare the energy heads at U and 1. If these energy heads are not the same, the following adjustment is needed.

3. Adjust the values of h_u, V_u, h_1, V_1, by increasing the water depth at 1 if the energy head at 1 is lower than energy head at U and similarly by decreasing the water depth at 1 if the energy head at 1 is higher than at U.
4. Repeat the calculations until the values agree within a specified error.
5. Next apply the energy equation above to points 1 and C
6. Assume a value of water depth at section C and repeat the calculation procedure from above (1-4) for point 1 and C. In this section h_{fc} is the losses due to enlargement of the channel.

Detailed calculations are given in Appendix C.1. As a result of these calculations, the elevation at section C is known. The elevation at section C is needed to calculate the capacity of the intake to the ring canal and the capacity of the syphon.

Based on the elevation at point C from the above calculations, the capacity of the syphon and ring canal intake can be determined.

5.3.5 CALCULATION OF SYPHON FLOW (BLOCK H AND FLOW DIAGRAM OF SYPHON)

From the calculations in Section 5.3.4, the elevation at Opak Drain, upstream of the syphon (point C) is known. There are two possible cases for the water elevation at point C with respect to flows through the syphon:

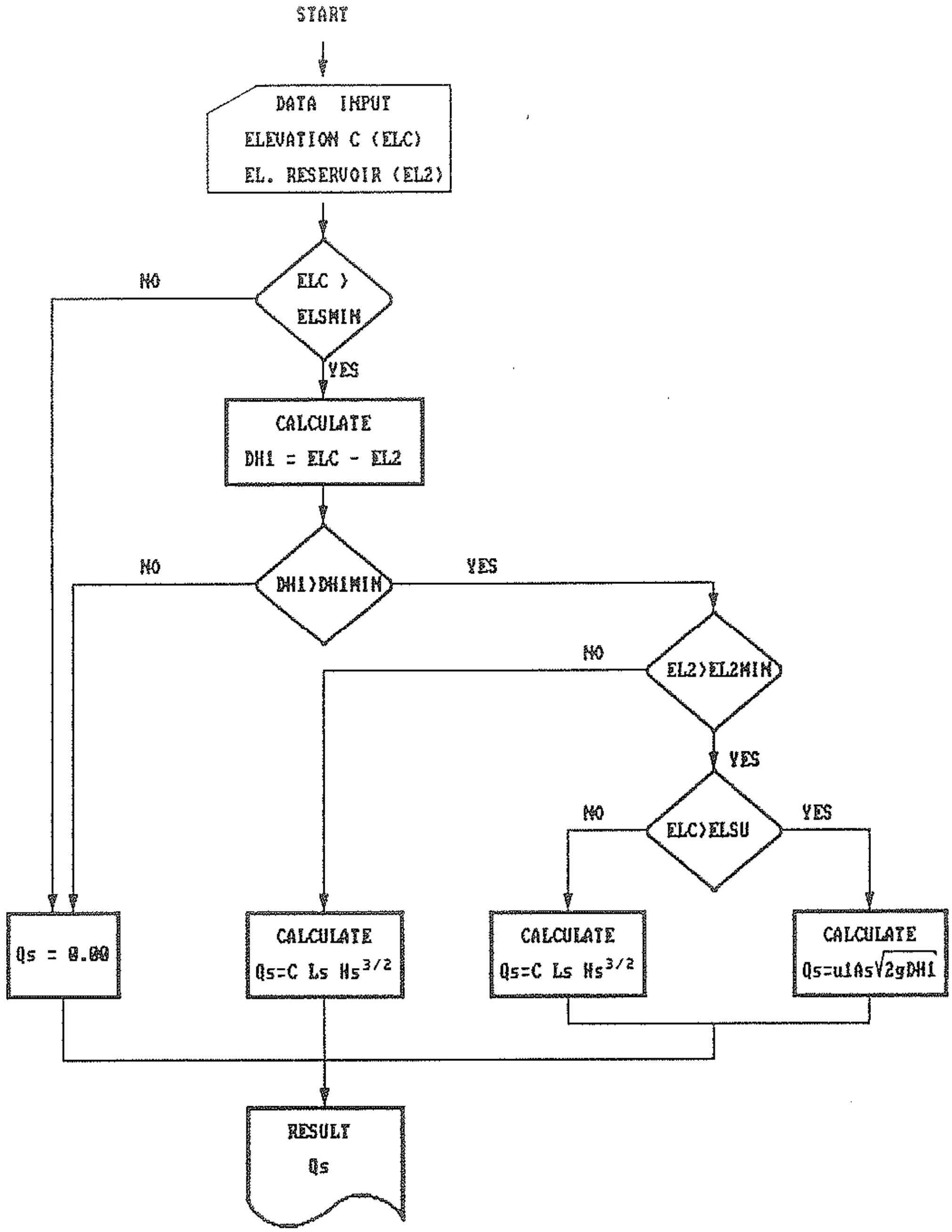


FIGURE 5.7 FLOW DIAGRAM OF SYPHON

CASE 1 (Water Levels At C Lower Than The Syphon Crest)

The first is that the elevation of Point C upstream, is lower than the crest of the syphon. In this case, there is no water flow through the syphon. The crest elevation of the syphon is +0.10 m, therefore at any upstream elevation lower than +0.10 m the syphon discharge (Qs) is equal to 0.00 m^3/s.

CASE 2 (Water Levels At C Higher Than The Syphon Crest)

The second case is for the water elevation at Point C higher than the syphon crest elevation. In this case water flows through the syphon. The amount of flow (Qs) depends on the pressure conditions in the throat. Two flow conditions are considered in this study:

a) Weir Flow (Figure 5.1.A)

If sub-atmospheric pressure does not develop in the syphon, the syphon acts as a weir. Therefore, the calculation for this condition follows the weir calculation. (For details see Appendix C.2.) The discharge equation for flow over a weir is used:

$$Q = C\, L_e\, H^{3/2} \qquad (5\text{-}16)$$

where:

Q = discharge through the syphon (m^3/s)

C = weir coefficient ($m^{1/2}/s$)

L_e = effective crest length (m)

H = total head above crest (El. +0.00) (m)

b) **Syphon Flow (Figure 5.1.D)**

If the air in the throat has been exhausted, the syphon will act as a pure syphon. This condition can only be reached if both the upstream entrance to the syphon and the downstream leg of the syphon are submerged to the point where air cannot enter the syphon. The flow in this condition depends on velocity in the throat and the losses which are described below. (See detailed calculations in Appendix C.2):

The velocity through the throat depends on the difference between upstream and downstream energy levels.

The losses through the syphon are included in the equation as a loss coefficient. This loss coefficient can be determined from the total losses through the entire syphon system. Under these conditions, the syphon is considered to be a closed pipe system. The losses in the system are:

1. Loss at the entrance to the syphon
2. Loss due to friction
3. Loss in the throat of the syphon
4. Loss at the outlet

Based on the above considerations, the syphon flow is calculated using the Full Syphon Flow Equation (Equation 5-1). From Equation 5-1, it is clear that the total capacity of the syphon is dependent on the area of the throat and the differences in energy levels upstream and downstream of the syphon. The larger the head, the larger is the capacity of the syphon. The upstream elevation is dependent on the inflow and the Opak Drain capacity. The upstream elevation should be kept as low as possible especially during high inflow conditions to reduce the possibility of flooding. Low upstream elevations are possible only if the discharge through the syphon is large. Because of the limitation on upstream elevation caused by the damages due to flooding, maximizing the syphon discharge depends on maintaining the reservoir elevation at a low level. As a result, the reservoir elevation should be kept as low as possible. The reservoir elevation issue will be discussed in more detail in Section 5.3.8.

In considering waste (garbage) influences on syphon capacity, the primary mechanism by which waste could reduce the syphon capacity is by the physical clogging of the throat (or the entrance) of the syphon. Such clogging would cause the area of the throat to be reduced, hence reducing the flow through the syphon. As the design of the syphon includes a waste skimmer and there is no evidence that such clogging occurs, this model does not include the clogging of the syphon due to waste loads. Further field investigations are, however, necessary to confirm or deny the existence of such clogging.

5.3.6 CALCULATION OF RING CANAL FLOW (BLOCK I)

From the calculation in Block G (Figure 5.2), the elevation downstream of the bridge (Point C) is known. The water level downstream of the ring canal (in the forebay) at the beginning of the time interval is also known from the initial conditions (Block B) or from previous calculations (previous time interval). Based on the city regulations a water level of el. + 0.00 m in the channel upstream of the intake structure (at Opak Drain) must be maintained. This requirement is to maintain the water level in the drains of the drainage system at the normal water level el. +0.00. Therefore two conditions can be considered:

- <u>Water Level Upstream of the Intake Structures < el. +0.00)</u>

  If the water level upstream of the ring canal intake is zero or lower than zero, the intake gates to the ring canal should be closed. (Since automatic gates were

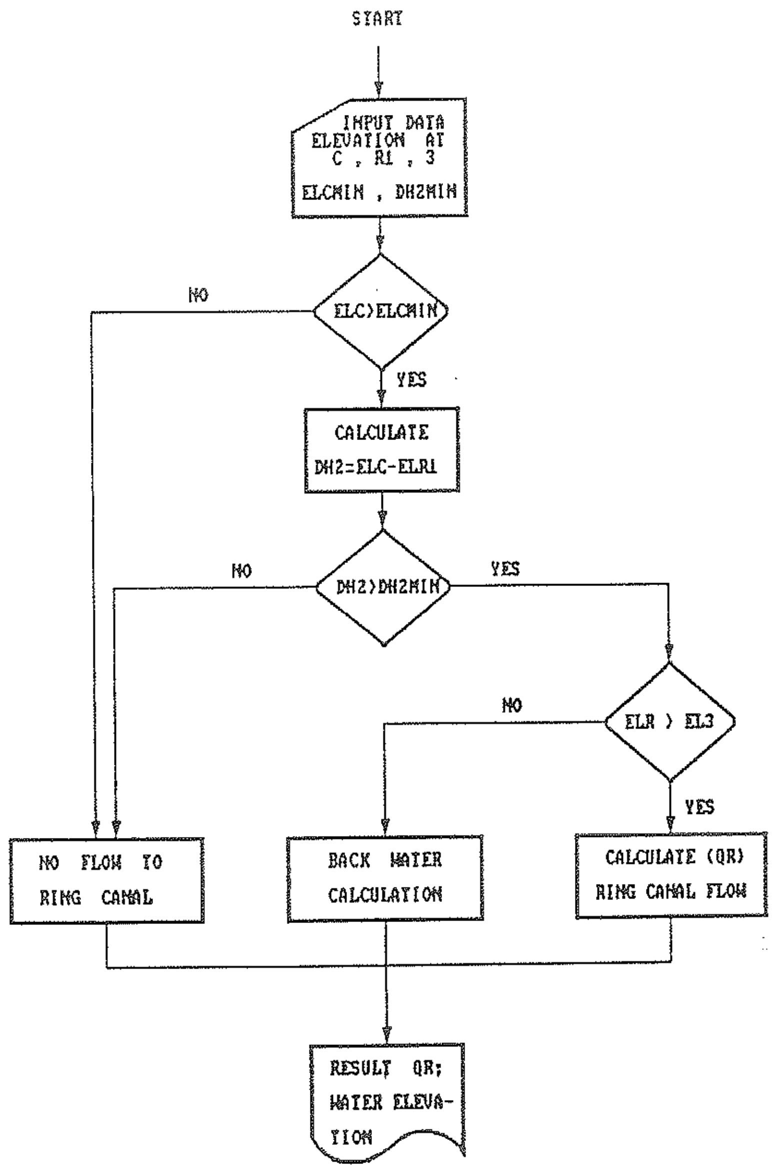


FIGURE 5.8 FLOW DIAGRAM OF RING CANAL

installed in the Pluit Polder System, this is automatically implemented by the gates). This means if El_c <= +0.00, the flow entering the Ring Canal, is $Q_r(1,j) = 0.00\ m^3/s$. Subscript 1,j refers to distance no 1 or more specifically at the beginning of the Ring Canal and at time interval j.

- <u>Water Level Upstream of the Intake Structure > elevation +0.00 m</u>

  If the upstream water level is higher than + 0.00, then two conditions must be taken into account in the calculation of the Ring Canal flow:

  a. <u>The Water Level of The Forebay Is Lower Than The Water Level In The Ring Canal.</u>

     If the water level in the forebay is lower than the water level in the ring canal, the flows and water elevations in the ring canal depends on the upstream flow conditions. In this case it depends on the intake flow or intake capacity. The calculation of water elevation in the ring canal is based on Manning's Equation since the flow can be reasonably approximated by as uniform flow. However, there are conditions under which uniform flow could not be considered:

     1. The condition for which the ring canal intake gates are closed and there is still flow remaining in the Ring Canal. At this

condition intake ring canal discharge is equal to 0.0 m^3/s but there is still remaining flow in the Ring Canal.

2. The condition when the ring canal intake gates are opened. The flow in the ring canal can be low, and therefore a change in water level with time occur. For these conditions the flow changes from uniform flow to rapidly varied unsteady flow. Since this condition occur over a short time interval and there are no significantly influences the system, this condition is neglected in this study.

b. <u>If the downstream water level (in the forebay) is higher than the water level in the ring canal.</u>

If the water level in the forebay is higher than the water level in the ring canal, a backwater calculation then must be applied. This is because a uniform flow condition implies that the same velocity occurs in the entire length of the canal, and the energy slope is the same as the bottom slope. When the downstream water level is higher than the upstream water level, the energy slope is no longer the same as the bottom slope. As a result, uniform flow calculations cannot be used. When the downstream elevation is higher than the upstream elevation, the flow changes from uniform flow to gradually varied flow. In a gradually varied flow regime the depth varies along the channel length, and the dynamic equation of Gradually Varied Flow should be applied. This equation is obtained by differentiating the Energy Equation with respect to distance along the channel. The Energy Equation is:

$$H = z + h \cos \theta + \alpha V^2/2g \qquad (5\text{-}17)$$

where:

H = energy head

z = bed level

h = water depth

θ = channel bed slope

α = coefficient of losses

V = mean velocity

Several methods have been developed to simplify the Differential Equation for Gradually Varied Flow. The Standard Step Method was chosen in this study. The Standard Step Method is used to calculate the water profile iteratively in this situation as follows (refer to Figure 5.9):

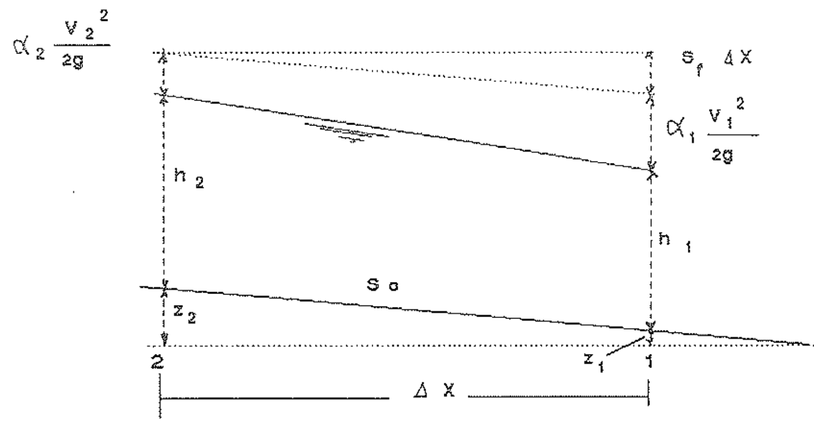


Figure 5.9 Standard Step Method

The distance step Δx and the elevation downstream point 1 is known from the previous calculation. The distance (step) and elevation are used as the starting point. The area, hydraulic radius and the velocity at this point can be calculated using Manning's Equation. The energy level and friction slope can also be determined:

$$H_1 = z_1 + h_1 + \alpha_1 V_1^2/2g \tag{5-18}$$

$$S_{f1} = n^2 V_1^2/ R_1^{4/3} A^2 \tag{5-19}$$

Next, at point 2 a value for the water level is assumed at that point h_2'. The velocity and energy head H_2 is then calculated:

$$H_2' = z_2 + h_2' + \alpha_2 V_2^2/2g \tag{5-20}$$

The energy slope S_{f2} can be calculated using Manning's Equation:

$$S_{f2} = n^2\, V_2^{\,2} / \, R_2^{\,4/3}\, A_2^{\,2} \tag{5-21}$$

$$S_{fav} = (S_{f1} + S_{f2})/2 \tag{5-22}$$

From the average friction energy slope (equation 5-22), the energy head at 2, H_2'', can be determined:

$$H_2'' = H_1 + S_{fav}\,(x_2 - x_1) \tag{5-23}$$

Compare H_2' to H_2'', if the difference is larger than 0.01 m, re-estimate the value of the water level at point 2, repeat the calculation above until it fulfils accuracy requirements. Once the value of H_2' is satisfied, this value becomes H_1 for the next step calculation. If the end result at the ring canal intake shows that the water level downstream is so high that the difference in the water level upstream and downstream of the ring canal intake is small, no flow from upstream occurs. At this condition the ring canal intake gates are closed.

As stated previously, the effects of garbage on the capacity of the ring canal and ring canal Intake has been considered in the system reducing the discharge capacity of the ring canal by 10% and 25%. Further investigation of the amount of waste in this area and the possibility of clogging at the ring canal intake gates should be undertaken.

5.3.7 CALCULATION OF THE FLOW AT RESERVOIR OUTLET GATES (BLOCK J)

To calculate the elevation in the reservoir due to changes in the inflow, the outflow through Outlet Gates must be determined. The flow at the reservoir outlet gate is dependent on the difference in elevation between the reservoir and the forebay. The calculation of the discharge through the Outlet Gates is based on the elevations in the reservoir and the forebay at the beginning of the time interval. The results of this calculation are used as a starting value for the Reservoir discharge for the reservoir routing. The discharge capacity of the outlet gates is determined using the following Equation:

$$Q = u\,A\,\sqrt{(2\,g\,H)} \tag{5-24}$$

where:

Q = discharge through the outlet gates

u = discharge coefficient due to contraction

A = cross sectional area of the gate opening

g = acceleration due to gravity

H = head available (Gross head)

If submerged flow occurs, the Momentum and Energy Equations are used, the calculation is as follows:

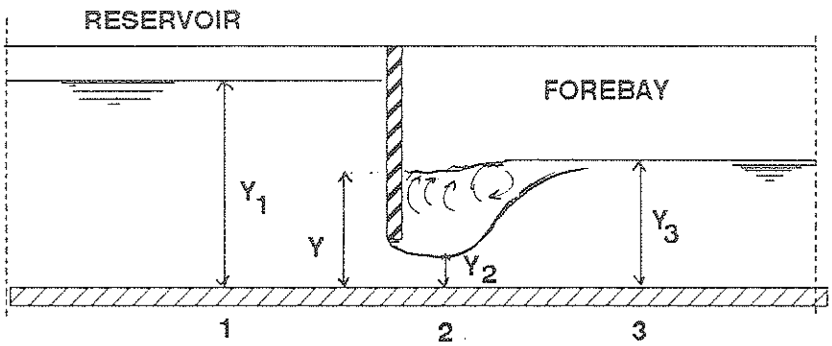


Figure 5.10 Reservoir Outlet Gate

The Momentum Equation between points 2 and point 3:

$$1/2\ \rho\ g\ y^2 - 1/2\ \rho\ g\ y_3^{\ 2} = \rho\ q(V_3 - V) \qquad (5\text{-}25)$$

$$y = y_3 \sqrt{(1 + 2\ F_3^{\ 2}\ (1 - y_s/y_2))} \qquad (5\text{-}26)$$

where:

$$F_3 = V_3/\sqrt{(g\ y_3)} \qquad (5\text{-}27)$$

The Energy Equation between points 1 and 2:

$$y_1 + V_1^{\ 2}/2g = y_2 + V_2^{\ 2}/2g + V_2^{\ 2}/2g \qquad (5\text{-}28)$$

In this model it is assumed that the gates are fully open (opening= 2.00 m) and the coefficient of discharge is 0.75. This coefficient should be further investigated by comparison to field measurements.

5.3.8 THE PLUIT RESERVOIR ROUTING PROCEDURE (BLOCK K - O)

The reservoir routing procedure is used in the calculation of the elevation in Pluit Reservoir. The flow through the syphon is considered as the inflow (Section 5.3.5), while the flow through the Reservoir Outlet Gates is considered as the outflow (Section 5.3.7).

The Reservoir routing uses the following Equation:

$$(S_2 - S_1)/ \Delta t = 1/2\ (I_1 + I_2) - 1/2\ (O_1 + O_2) \qquad (5\text{-}29)$$

where: S = storage

I = inflow (syphon)

O = outflow (Reservoir Outlet Gates)

Δt = routing time period

subscripts 1 and 2 refer to the beginning and end of time period.

The calculation used an iterative method, by adjusting the syphon and reservoir outlet gate capacities to take into account changes in the reservoir level and forebay level. This procedure is repeated until the reservoir elevation does not change the results of the syphon and the reservoir outlet gate discharge calculations.

5.3.9 CALCULATION OF THE ELEVATION OF THE FOREBAY (BLOCK L)

The elevation of the forebay depends on the flow in the ring canal, the discharge from the reservoir outlet gates, and the capacity of the forebay pumps.

The flow through outlet gates is determined as discussed in Section 5.3.7 (Block J), while the ring canal flow is known from the results in Section 5.3.6.

The maximum capacity of the forebay pumps is 4 x 4 m^3/s. Due to pump operating characteristic limitations, the pumps are operated if the water level in the forebay is higher than -1.90 m. Further, the pumps are subject to clogging and damage if the solid waste (plastic, etc) enters them. The total pump capacity is then reduced. The amount of the reduction depends on the amount of pump damage. This condition is accounted for in the computer model with sensitivity analyses and discussed in Section 6.

The equation for the elevation at the forebay is as follows:

$$EL3(J+1) = EL3(J) + (Q_O+Q_R-Q_P) \times \Delta t / A3 \qquad (5\text{-}30)$$

where:

EL3(J)	=	elevation at the Forebay at time (j)
Q_o	=	discharge through Reservoir Outlet Gates
Q_R	=	discharge through Ring Canal
Q_p	=	discharge from the Forebay Pumps
Δt	=	time interval
A3	=	forebay area

This result is used to recalculate the Reservoir Outlet Gate discharge for the Reservoir routing procedure (see Section 5.3.8).

5.3.10 CHECK THE ELEVATION UPSTREAM AND DOWNSTREAM OF THE PLUIT SELATAN BRIDGE (BLOCK P - S)

At the end of the program, the elevation at Section C (upstream of the syphon), and, at Section U is checked. This check is performed because of the possibility that the combined discharge of the syphon and the intake ring canal is lower than the inflow upstream. This difference in flow will increase the elevation upstream of the syphon. When the elevation upstream of the syphon increases, the syphon discharge increases and reservoir elevation increases. The intake ring canal discharge will also increases. This means that due to excess flow, the elevation in the entire system changes. Adjustment due to this increment is important. This adjustment could be done by recalculating all of the above elevations using iterative procedures:

1. Recalculate Block C - Block O
2. Check if the adjusted elevation upstream syphon remain the same after the recalculation
3. Repeat those procedures (1 and 2) until it fulfills accuracy requirements

The excessive calculations required for the above iterative procedures made this approach impractical. Because the results would not differ significantly after the first iteration, while more computer storage and execution time is needed to run the model. Instead, the following approach was used:

- take the average value of the elevation upstream due to the excess water and elevation at the beginning of calculation (Elevation C at $(t+\Delta t/2)$)
- use the average value of water elevation above to route the model (follow 4 in the flow diagram and repeat from Block C)

5.3.11 THE RESULTS (BLOCK T)

The results of the calculation at the end of each time interval are:

- The flood elevation and flood volume
- The water elevation upstream and downstream of the Pluit Selatan Bridge
- The available reservoir capacity or the elevation of the reservoir
- Water elevation in the forebay
- The discharge through the syphon, ring canal and reservoir outlet gates

All of the results at the end of the time interval are used as the initial conditions for the next time interval.

5.4 SIMULATION OF CURRENT OPERATION METHODS

5.4.1 INPUT DATA

The hydrologic data used in the current situation is as follows:

A. Three days of actual streamflow data and tidal data

B. The calculated inflow hydrographs for the 25 year return period storm and the 2 year return period storm

The three day observed hydrograph is the most important piece of data, since it represents the actual flow that occurred in this area. It has a long time period, therefore the flows produced by this flow data are important in the simulation study of the Polder system. This data also represents a multiple hydrograph, which is typical in the Pluit area. The original design of the system was based on a single rainfall event hydrograph while the actual data demonstrate that several closely occurring rainfall events produce a series of runoff hydrographs, which are more difficult for the system to handle. Therefore these data are used throughout the investigation of Pluit Polder System.

The hydraulic data used to simulate the current operating method are based on that available from previous studies. The reduction of the Opak Drain capacity due to garbage is included in the simulation model (See Section 5.4.2).

5.4.2 CURRENT SYSTEM OPERATING RULES

The investigation of the current situation is based on the following operating rules:

1. Ring Canal Intake Gates and Ring Canal
   - The intake gates are opened if the water level upstream of the gates is higher than +0.00
   - The intake gates are closed if the water level upstream of the gates is +0.00 and lower.
2. syphon
   - Inflow to the reservoir depends on the difference in water levels between the Opak Drain and the Pluit Reservoir. The calculation is shown in Appendix C.2.
3. Reservoir Outlet Gates
   - The gates are opened if the water level of the Pluit Reservoir is higher than that of the forebay.
   - The gates are closed if the water level of the forebay is higher than that of the Pluit Reservoir.
4. Forebay Pumping Station
   - The pumps are run at the maximum capacity of 16 m3/s if the water level of the Forebay is higher than -1.90 m
   - The pumps are stopped if the water level of the Forebay is lower than -1.90

5.4.3 WASTE CALCULATIONS

Since the waste dumped into the canals reduces the capacity of the system, some assumptions have been made in the calculation of the capacity of the hydraulic structures. These assumptions are as follows:

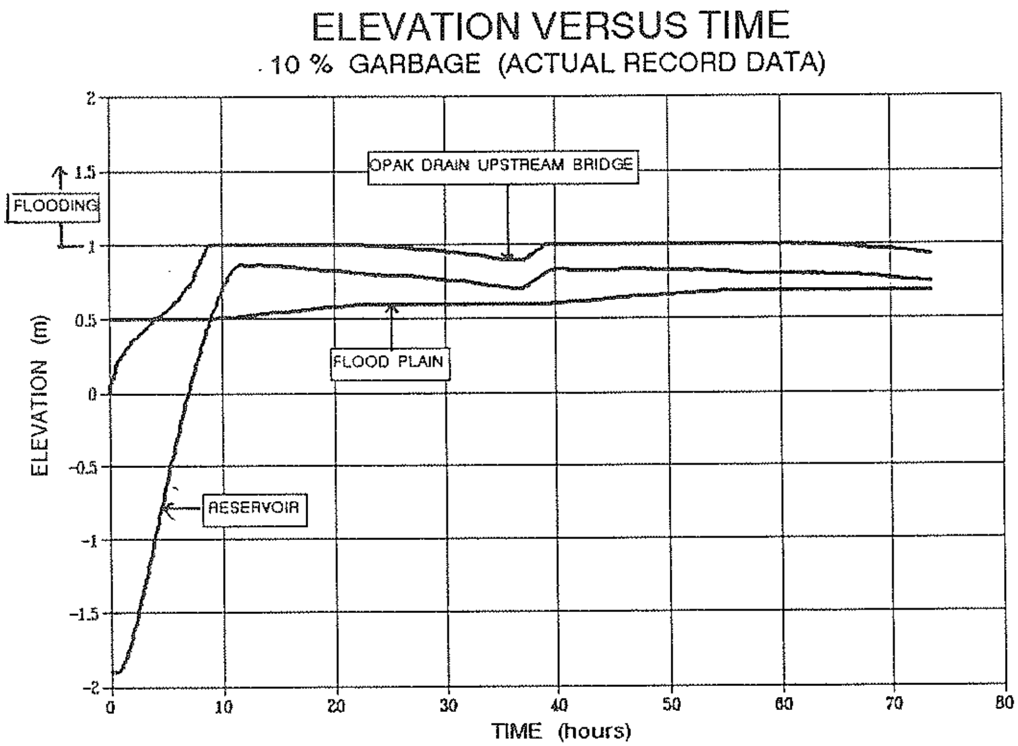
- Reduction of the Opak Drain discharge capacity by 10% and 25%. This reduction is accounted for by increasing the roughness coefficient. This approach is based on the fact that waste promotes the growth of aquatic plants, which in turn increase the channel roughness.
- No reduction in the syphon capacity has been made due to garbage. There is a skimming waste structure upstream of the syphon and the openings in the syphon are large (1 m x 4 m). Hence, the chance of such blockage is considered to be minimal.
- Reduction in the ring canal capacity by 10% and 25%, based on the same reasoning used for the Opak Drain.
- The possibility of pump stoppage due to plastic waste, (a condition which has actually happened) is investigated in one of the sensitivity runs. It is assumed that one pump fails causing the total capacity of the pumps to be reduced by 4 m^3/s.

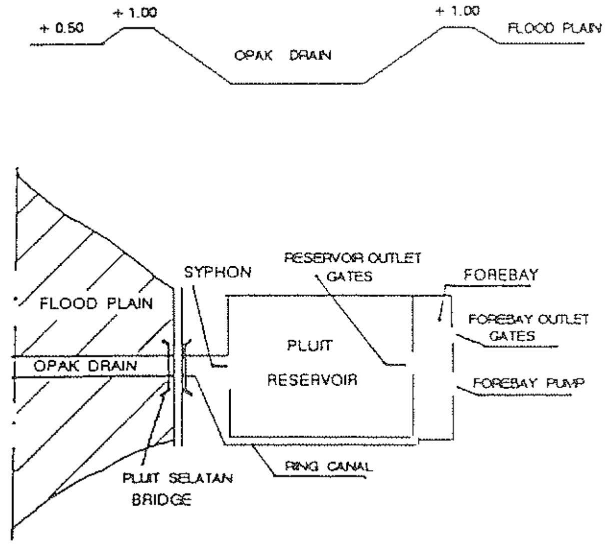
These assumptions should be investigated in the prototype in the future by measurement devices at appropriate points of the hydraulic structures.

5.4.4 ANALYSIS OF THE RESULTS OF CURRENT OPERATION SIMULATION

Using the hydrological data and waste assumptions, a series of simulation runs were performed. Relationships between flooding elevation, reservoir elevation, forebay elevation, tidal elevation and time are derived in these model runs.

The results of the simulation of the current conditions are shown in Figures 5.11 - 5.18. Figure 5.11 and 5.12 shows the resulting flood and relation between reservoir, forebay, and tidal elevation due to 10 % garbage and considering 3 days actual data (inflow hydrograph 1). Figure 5.13 shows the condition of flooding using 25% garbage and considering Inflow Hydrograph of 3 days actual data (Inflow Hydrograph 1). The water elevation in the Opak Drain increases with the increase of the upstream streamflow until it reaches the bank elevation +1.00 m at T=8.5 hours. Flooding in the flood plain areas begin at T=8.5 hours (elevation +0.50 m) and increases until elevation +0.84 m at T=20 hours. This is flooding in the flood plain areas which is lower than the Opak Drain bank elevation due to the first storm. This flooding will remain the same during the lag time between the first storm and the second storm. Since not much water could be pumped from the reservoir, the reservoir elevation is still high when the





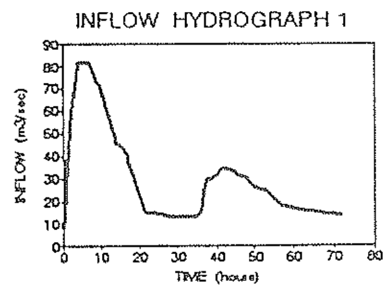
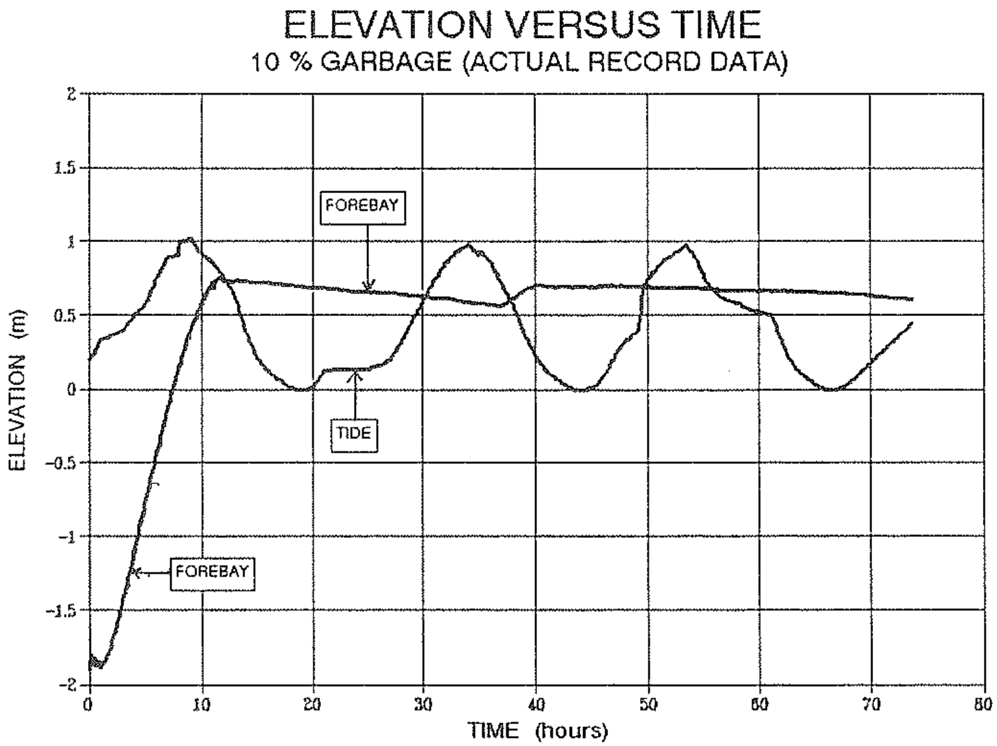
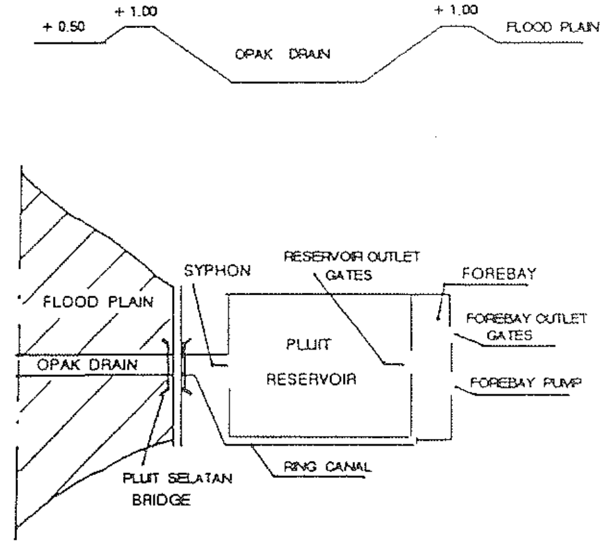


Figure 5.11 Elevation versus Time for Current Condition 10% Garbage (Actual Recorded Data)





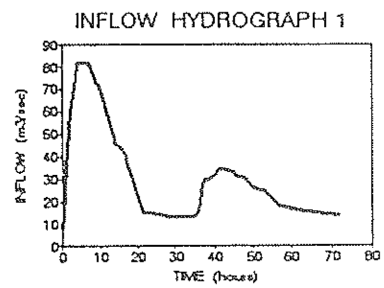
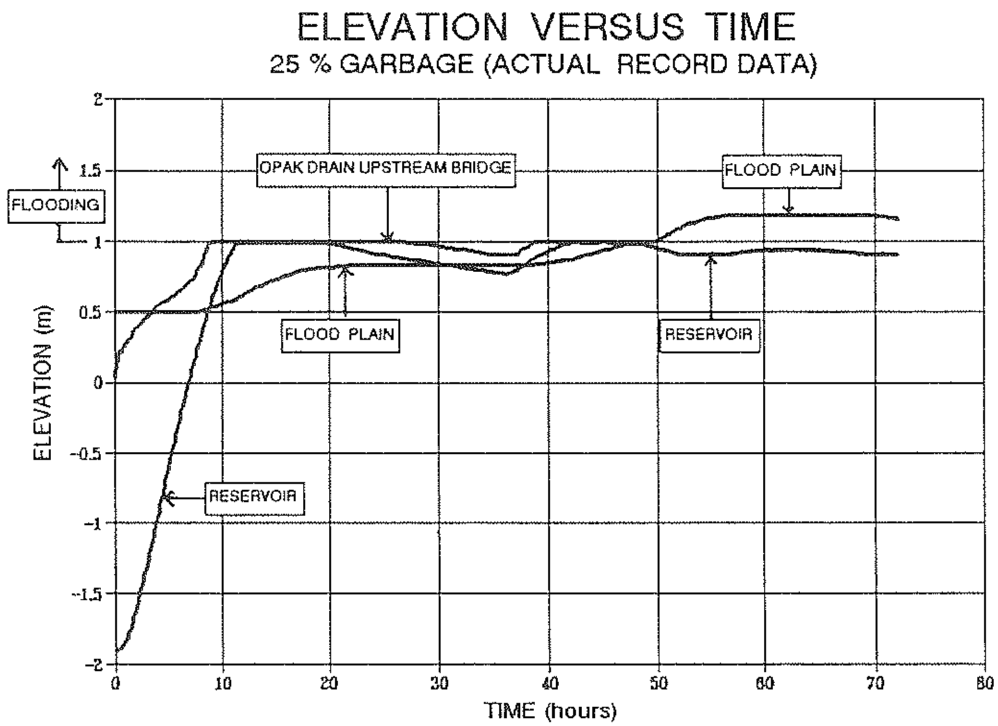
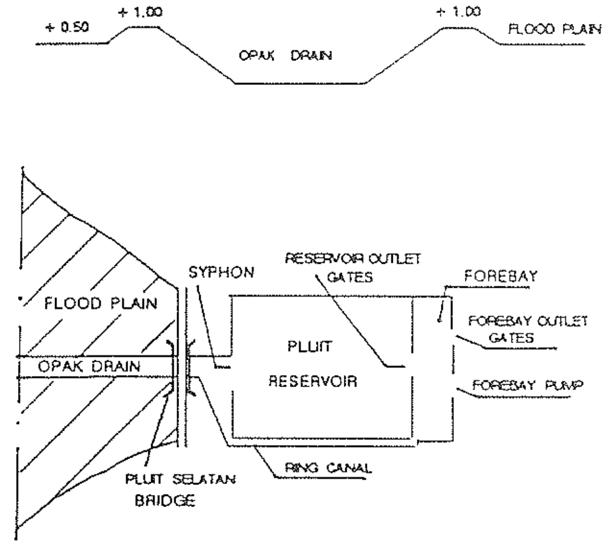


Figure 5.12 Elevation versus Time for Current Condition
10% Garbage (Actual Recorded Data)





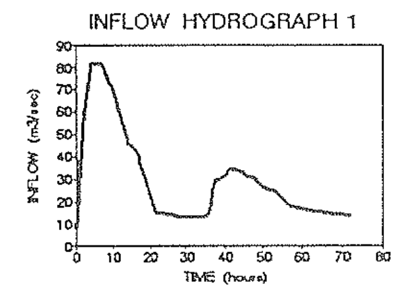
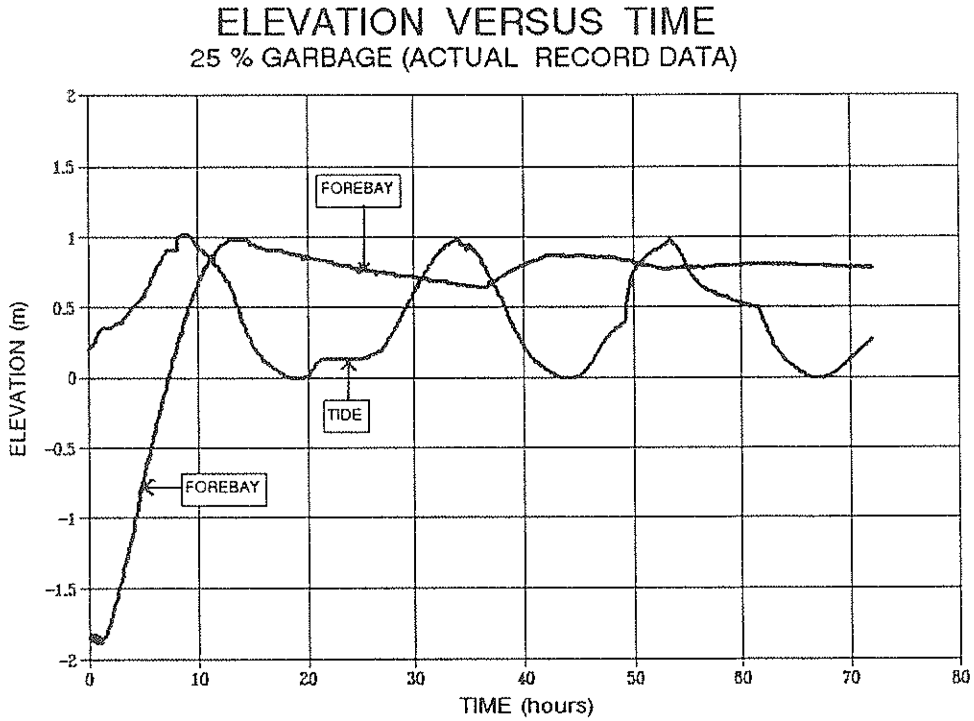
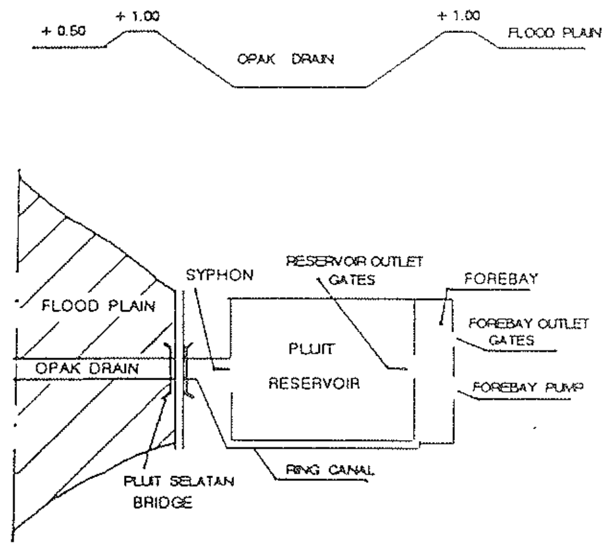


Figure 5.13 Elevation versus Time for Current Condition 25% Garbage (Actual Recorded Data)





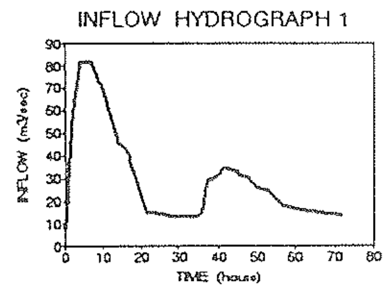
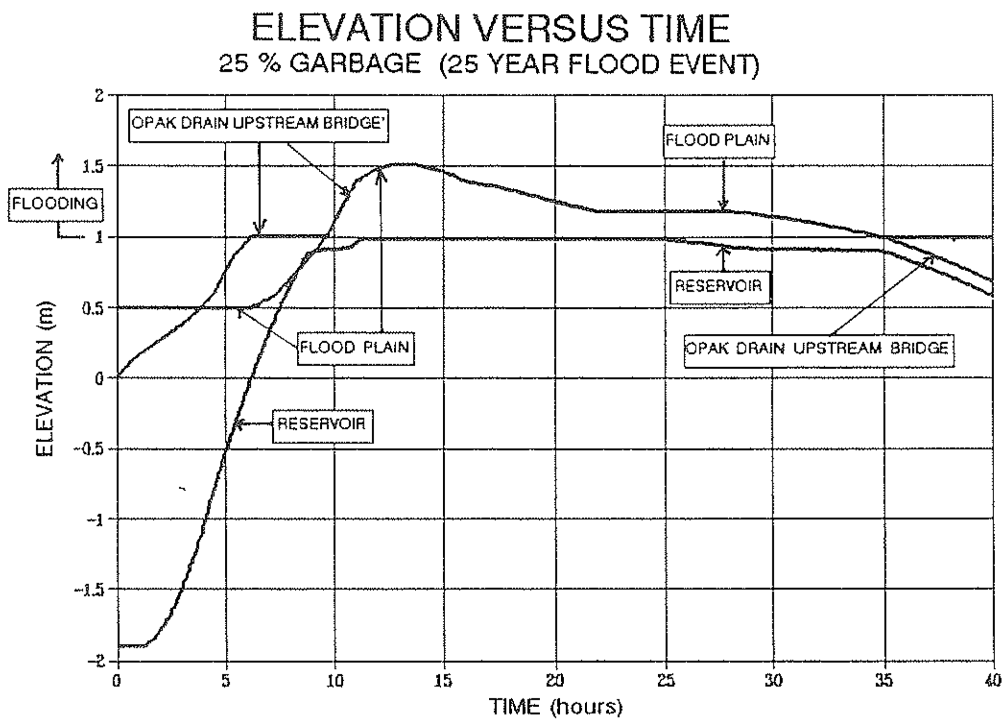
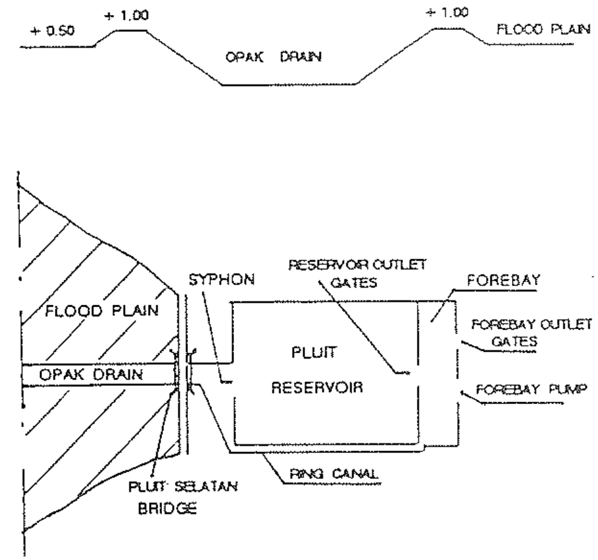


Figure 5.14 Elevation versus Time for Current Condition
25% Garbage (Actual Recorded Data)





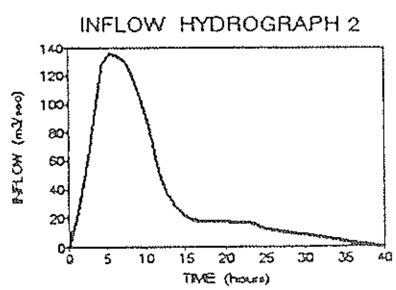
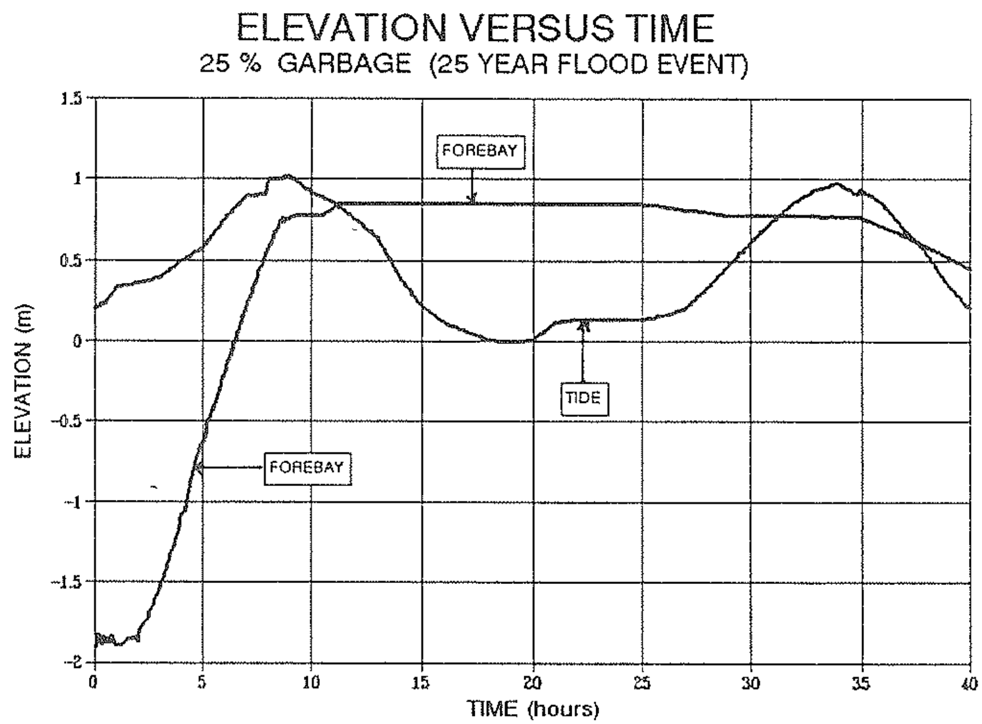
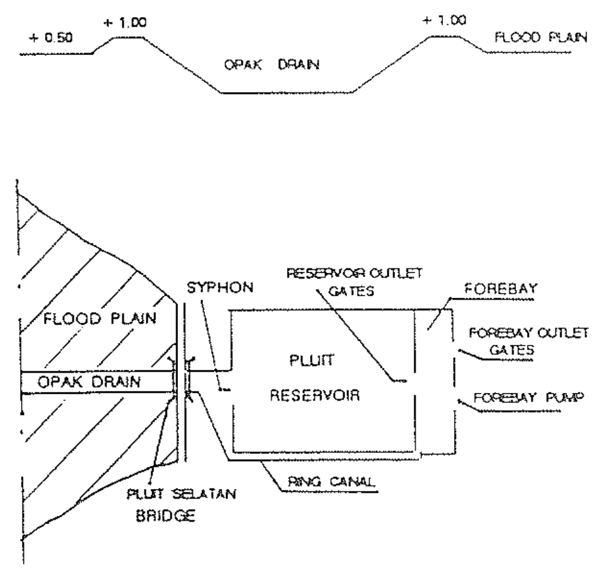


Figure 5.15 Elevation versus Time for Current Condition
25% Garbage (25 Year Flood Event)





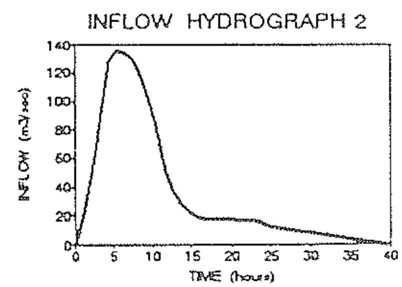
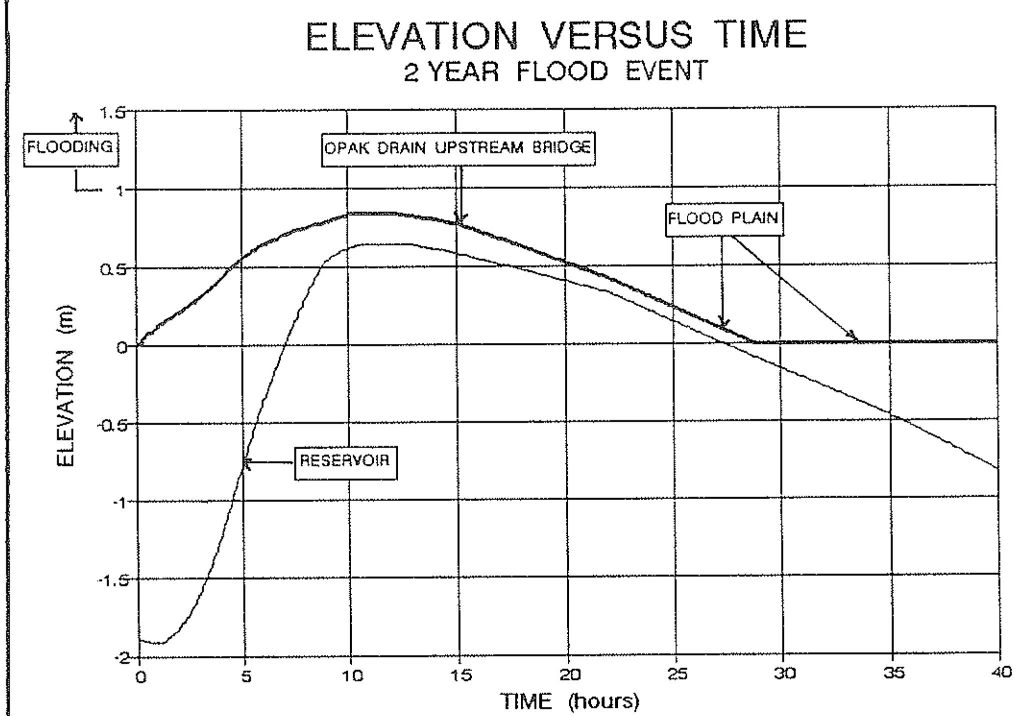
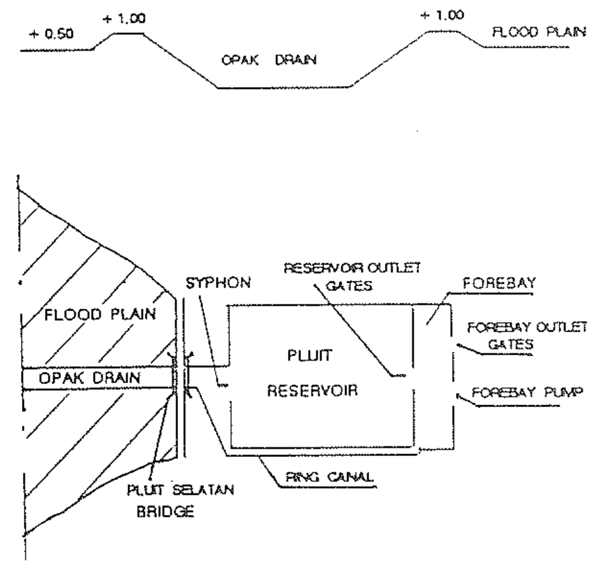


Figure 5.16 Elevation versus Time for Current Condition 25% Garbage (Actual Recorded Data)





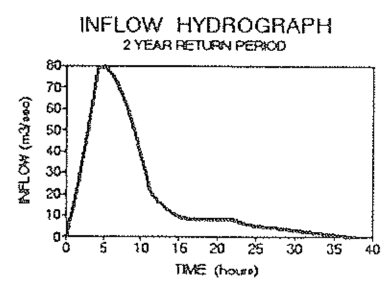
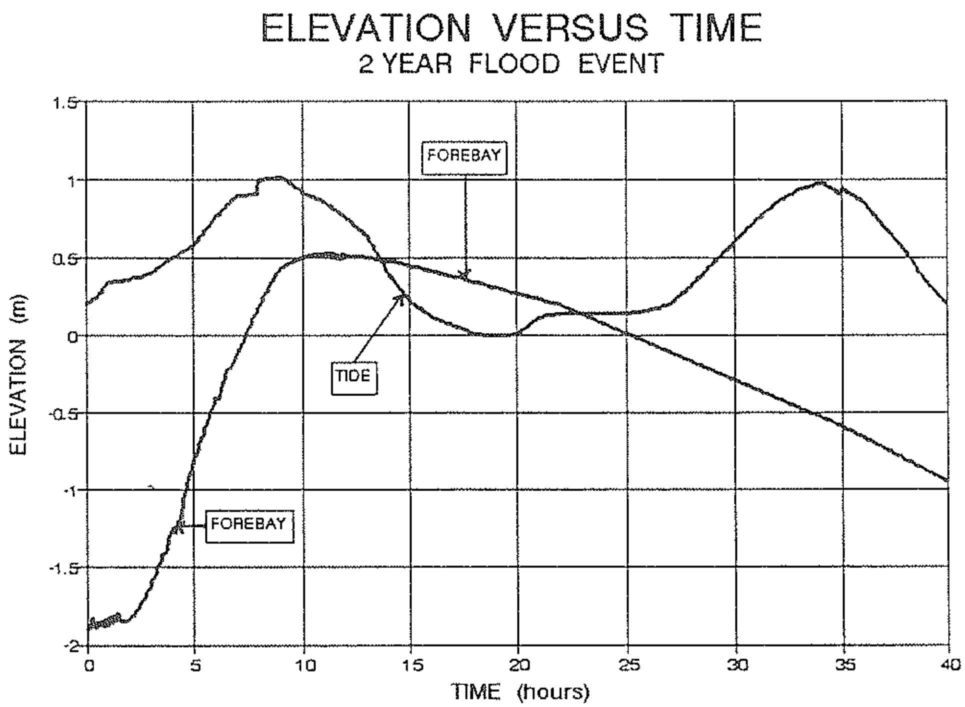
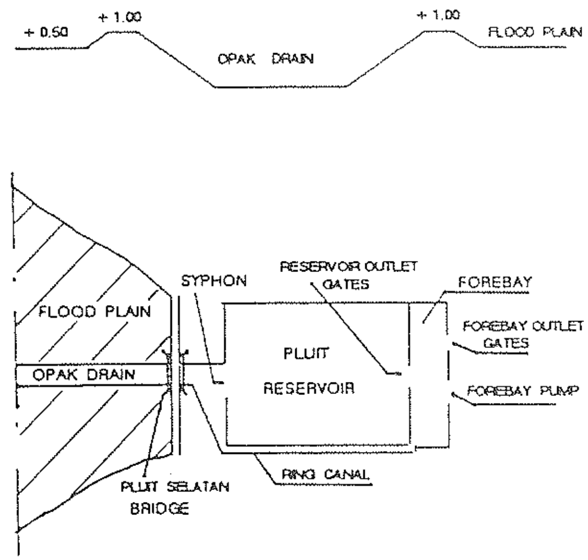


Figure 5.17 Elevation versus Time for Current Condition 25% Garbage (2 Year Flood Event)





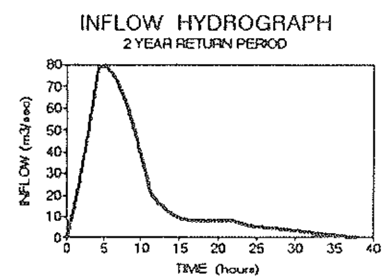


Figure 5.18 Elevation versus Time for Current Condition 25% Garbage (2 Year Flood Event)

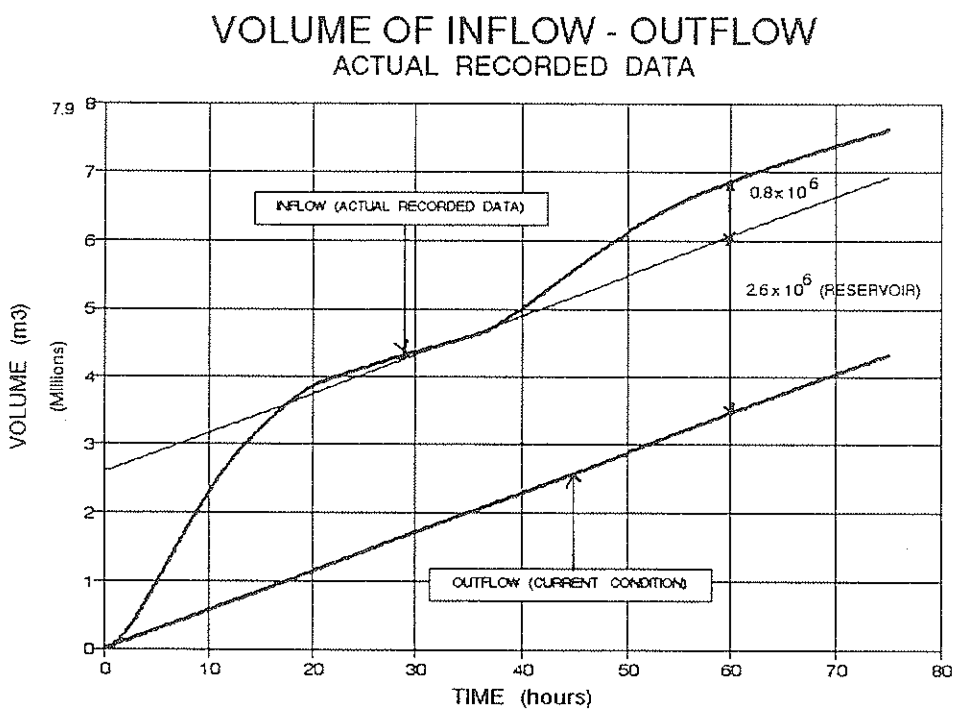


Figure 5.19 Volume of Inflow and Outflow

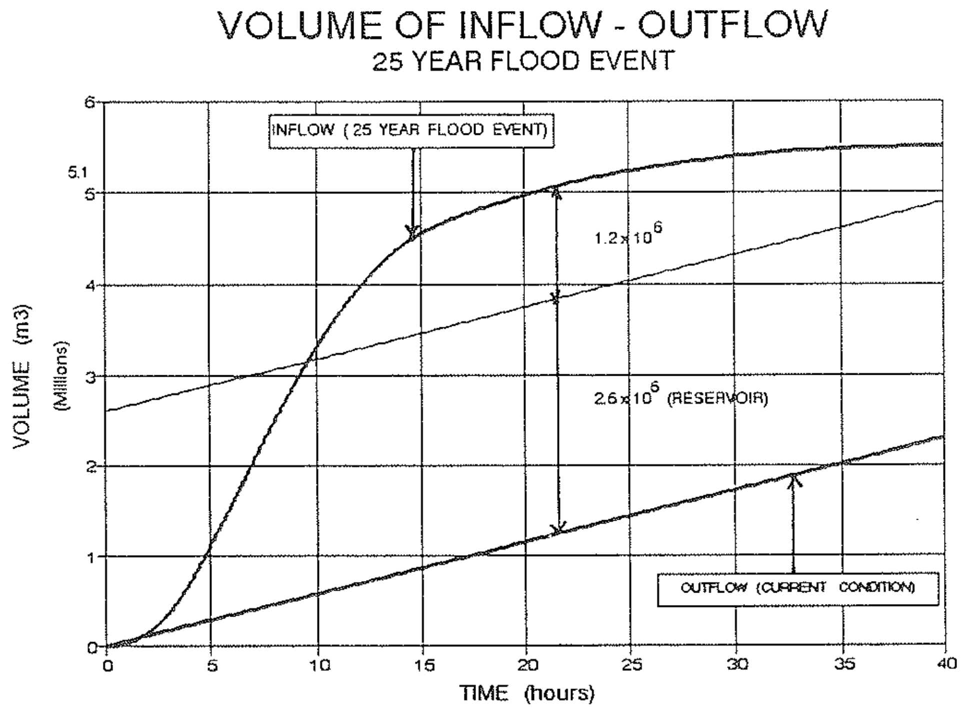


Figure 5.20 Volume of Inflow and Outflow
Current Condition (25 Year Flood Event)

second storm occur. At T=39 hours, flooding increases due to the second storm until it reaches the maximum flood elevation + 1.20 m at T= 59 hours. This flood of about 0.70 m represents the yearly flooding that usually occurs in the flood plain. The reservoir elevation at the end of the second storm is still high so that no capacity exists in case another storm occurs. Figure 5.14 shows the relationship between the forebay elevation and tidal elevation with time. This represent the extrem condition since the tidal elevation is high during the first storm (the peak of the first storm and the tide is coincide). After T = 12 hours, the tidal elevation decreases, the head between the tidal and forebay elevation increases. This means that there is a possibility to divert water directly to the sea during low tide period and large head.

Using 25 year return period hydrograph as a single storm, the result in Figure 5.15 shows that flooding begin at T- 6.0 hours. This flooding causes by the insufficient Opak Drain capacity. The flood increases as the upstream inflow increases and the reservoir capacity decreases. This flood condition is flood above the Opak Drain bank elevation. The flooding in the flood plain reaches the maximum elevation of about + 1.50 m at T= 13 hours. This flooding reduces as the upstream flow reduces until it reach the Opak Drain bank level + 1.00 m at T=34.5 hours. This remaining flood in the flood plain areas which is lower than the Opak Drain level is reduced by normal drainage paths. This flood was never happend until now. The Pluit Raya bridge as a control point is never flooded. Figure 5.16 shows the relation between tidal elevation and forebay elevation with time. This represent the extreem condition that the peak of the storm is coincide with the peak of the tide. There is a possibility of diverting water directly to the sea between T=12 hours and T= 30 hours.

Figures 5.17 and 5.18 shows the results of simulation using 2 years flood events. Using the three days inflow hydrograph, no flooding occurs in the flood plain.

From these results, the following observations and conclusions are derived and used to investigate improvements to the system overall:

1. All previous investigations used <u>single</u> storms to produce the design condition. Because of the humid tropical climate characterized by a pronounced rainy season, the results obtained for the Pluit Polder System simulation study demonstrate that all polders must be designed to handle combined events caused by closely spaced rain storms. Since the polder system deals with detention storage, flood volume is very important. In this case not only the peak inflow, but also the duration of the inflow influences the entire system, which supports the observation that a combination of two flood events with lower return periods should be

considered in addition to a single high return period flood event.

2. In the case of combined storm events, the second flood occurs because the elevation in the Reservoir is still high from the first event when the second storm occurs, therefore the capacity of the syphon is relatively very low. As a result, the flows backup upstream of the Raya Pluit Selatan Bridge, which acts as a control. The resulting increased water elevations cause added and prolonged flooding.
3. The reduction of Opak Drain capacity due to waste causes flooding in low lying areas upstream of the Pluit Reservoir because the inflow is larger than the capacity of the channel. The overflow occurs before it reaches the inlet structure, syphon and ring canal.
4. The tide elevation is lower than the forebay elevation for significant periods of time. As a result, there is the possibility of releasing water from the forebay through an automatic gate directly to the sea during low tide periods. Since the lowest recorded tide elevation is +0.00, it is not possible to divert water through the forebay gates during the restricted forebay level (el. -1.90) due to pump limitation.

5. The key improvement to the operation of the Pluit System lies in the outlet system. By increasing the capacity of the outlet, either by increasing the forebay pump capacity or by implementing automatic tidal gates, a faster reduction in reservoir elevation becomes possible. This faster reduction in the reservoir elevation reduces the risk that the reservoir might be full when another storm occurs. A lower reservoir elevation gives a higher capacity for the syphon. As a result of the larger syphon capacity larger upstream flows can be passed. Such an increase of the outlet capacity should prove to be economic because very little cost is involved in providing automatic flap gates at the outlet from the Forebay.
6. An increase of Opak Drain capacity is needed to pass the expected flows. This increase in capacity could be gained if regular dredging is done. To reduce the waste entering the Reservoir and Ring Canal, a waste screen could be installed at the control section which is under the bridge. This waste screen would be useful <u>only if</u> regular cleaning is carried out. Otherwise this waste screen will further constrict flows and cause more upstream flooding.

7. The results of cumulative inflow and outflow are shown in Figures 5.19 and 5.20. From these results the storage requirements can be derived. This calculation was done by using the actual three days streamflow data and the flood hydrograph for 25 years return period. Based on the actual recorded data, the volume of reservoir required is 3.4 million m^3, with the current outlet conditions (Qpump= 16 m^3/s). Based on the flood hydrograph for the 25 year return period storm, the volume of the reservoir required to accommodate this flood is 3.8 million m^3. From the Reservoir Storage Curves (Figure 3.1) the available reservoir volume at current conditions is 2.6 million m^3. This means that the current reservoir volume is 1.2 million m^3 smaller than the required volume.

5.5 ALTERNATIVES FOR MODIFICATION OF THE OUTLET SYSTEM

5.5.1 OUTLET MODIFICATION ALTERNATIVES

The Outlet system consists of (see Figures 5.21 and 2.2):

- The Forebay Pump
- The Forebay Outlet Gates
- The Reservoir Outlet Gates
- the Forebay Ring Canal Outlet

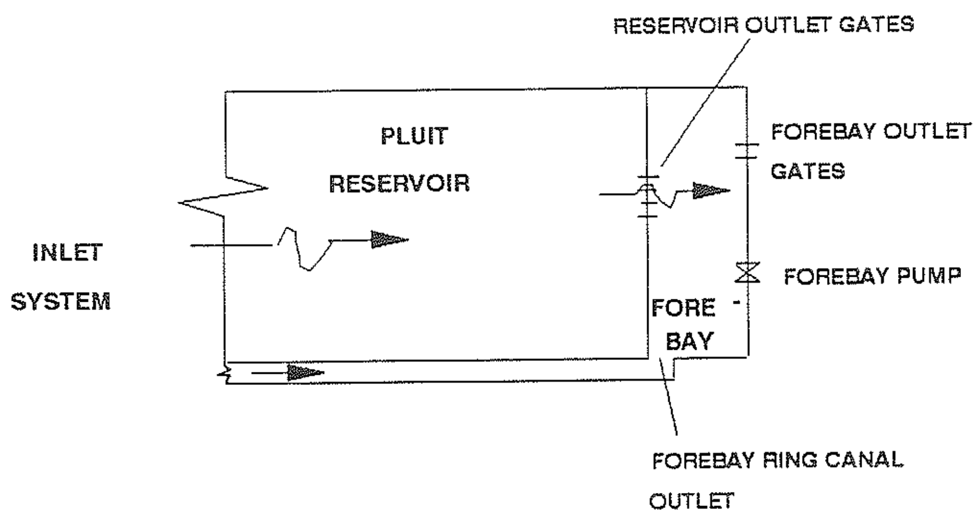


Figure 5.21 Outlet System Structures

In this outlet modification alternatives, no alternatives for improvement of the forebay ring canal outlet were considered, the reason being that the flow from the ring canal depends on the ring canal intake capacity and there is no outlet gate at the end of the ring canal. Therefore any improvement to the forebay ring canal outlet capacity would depend on improvement of the ring canal intake or the ring canal itself. The alternatives for improvement of the ring canal are included in the Combination of Alternative Modification (Section 5.7).

The following inflow hydrographs were used in the simulation model:

- **Inflow Hydrograph 1 (Figure 3.5):**

  This Hydrograph is three days of actual recorded stream flow data. As discussed in Subsections 4.3.1 and 5.4.4, a key component of the Polder System operation involves the use of detention storage. As a result, flood volume is very important to successful operation of the Polder System. Large flood volumes can result from combined events, which by themselves would not exceed the capacity of the polder system. Therefore the polder system must be designed to handle combined events caused by closely spaced rainstorms. The three days of actual streamflow data contains two flood events caused by closely spaced rain storms. It is therefore important to consider these types of events in the study. These data also support the observation of multiple storms in the Pluit area. Since combined storm floods are very important, that flooding occurs for far more frequently than would be anticipated by the size of the individual rain storms, other possible combined storm floods are considered in this study. These are described in Section 7, Sensitivity Analysis.

- **Inflow Hydrograph 2 (Figure 4.5):**

  This hydrograph is a calculated flood hydrograph for a 25 year return period. All previous investigations used this single storm hydrograph to produce the design condition. However, stream flow data analyzed above indicate that the previous assumption that successive storms do not interact is invalid.

Based on the results discussed in Subsection 5.4, the following alternative improvements to the current operation of the system were simulated using the mathematical model.

Alternative 1: Increase Forebay Discharge Pump Capacity

Increasing the Forebay Discharge Pump Capacity results in a faster reduction of reservoir levels. A lower reservoir level gives a higher syphon flow and, hence, larger upstream flows can be passed. Based on Inflow Hydrograph 1 and the current Pluit Reservoir capacity (2.6 millions m^3, Figure 5.19), the required pump capacity is about 21 m^3/s. The current Pump capacity is 16 m^3/s assuming that garbage does not reduce pump capacity. Based on Inflow Hydrograph 2 and the current Pluit Reservoir capacity, the required pump capacity is about 32 m^3/s (Figure 5.20). Since the flood based on Inflow

Hydrograph 2 does not occur frequently, it may not be economic to install the system with large pumps (32 m^3/s).

In the Pluit Polder System, however, it is probable that two or more successive, relatively small floods will occur within a short period of time. Using the current pump capacity of 16 m^3/s might lead to the condition that the reservoir is full when another storm occurs. The risk inherent in the possibility that the reservoir is full for significant periods of time makes increasing the forebay outlet capacity to divert water from the reservoir or from the forebay to the sea necessary. This could be done either by increasing pump capacity or introducing other outlet structures as described in Section 5.4.4 Item 4. Both possibilities were included in this study. The effects of total pump capacities of 20 m^3/s and 24 m^3/s were simulated in this study. These pump capacities are based on the required pumping capacity for Inflow Hydrograph 1 (21 m^3/s). An increase of 4 m^3/s was taken based on the current Pluit Pump Capacity (4 pumps, each with capacity of 4 m^3/s).

The amount by which the pump capacity is increased should be determined on economic grounds. The installation, operations and maintenance costs of the pump should be taken into account. Economic calculations were not included in this study and should be considered in future studies.

Alternative 2: Introducing Automatic Forebay Outlet Gates

Refering to Figure 5.22, the Automatic Forebay Outlet Gates would works as follows:

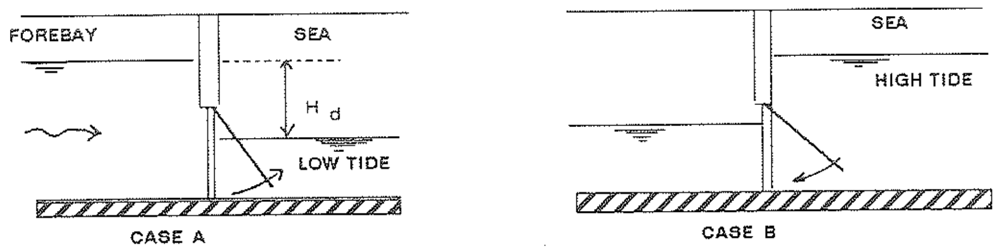


Figure 5.22 Forebay Outlet Gates

Case A. If the water level in the forebay is higher than the tide water level, the gates would automatically open because of the differential head(H_d). The water would flow directly from the forebay to the sea. The capacity of the forebay outlet gates depends on the gross head of the upstream water elevation (water elevation in the forebay) and the downstream water elevation (Tidal Elevation). The outlet capacity is calculated based on the following equation:

$$Q_{outlet} = C_c \; A \sqrt{(2 \; g \; H_d)} \tag{5.31}$$

where:

C_c = coefficient due to contraction

A = area of gates opening

H_d = differential Head

The detailed calculation is shown in Appendix C. In this study the width of the forebay outlet gates is taken to be 2.00 m per gate. Two alternative configurations of outlet gates were simulated, 2 gates that are 2.00 m wide and 3 that are 2.00 m wide.

Case B. If the Forebay water level is lower than the tide water level, the Forebay Outlet Gates would automatically close. In this case, there is no flow from the forebay to the sea.

Alternative 3: Combination of Increased Forebay Pump Capacity and Automatic Forebay Outlet Gate(s).

For this alternative, the required increase in pumping capacity was determined based on the required pump capacity of the first storm of Inflow Hydrograph 1. The peak of the first storm of Inflow Hydrograph 1 is approximately the same as a 2 year return period flood event. Since the required total pump capacity is about 21 m^3/s (refer to Alternative 1) and the current pump capacity is 4 @ 4 m^3/s (16 m^3/s), the improvements to pump capacities to 5 @ 4 m^3/s, or 20 m^3/s and; 6 @ 4 m^3/s or 24 m^3/s, were considered. The combined inflow hydrographs were applied since they are typical inflow hydrographs in the Pluit drainage area. After the first storm, the water level in the Reservoir is still high. Therefore, a forebay outlet gate is introduced to enable the outlet system to lower the reservoir elevation during low tide and to prepare for the next storm. In this study the combination of pump capacities of 20 m^3/s and 24 m^3/s and automatic forebay outlet gates of 2 @ 2.00 m and 3 @ 2.0 m were investigated.

A combination of increasing forebay discharge pump capacity and adding automatic forebay outlet gates may be more effective than increasing only pump capacity or introducing only forebay outlet gates. The reasons are:

- Increasing only the pump capacity to fulfil the requirement of the possibility of the two or more successive storms requires the installation of large capacity pump. High costs for installation, operation and maintenance are normally associated with such pumps.
- Introducing the forebay outlet gates alone without increasing the pump capacity, is not believed to be capable of overcoming the flooding problem. The forebay outlet gates would work <u>only</u> when the tide elevation is lower than the water elevation in the forebay. If a storm occurs while the tide is high, the outflow from the forebay occurs only through the current pumping system, which has insufficient capacity.

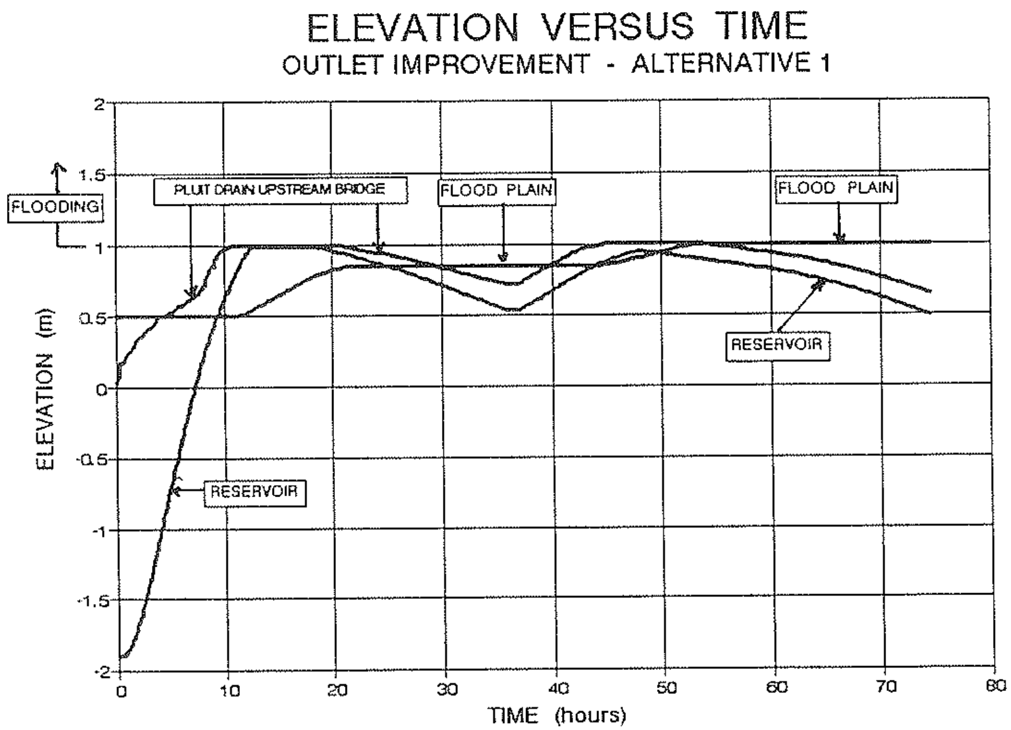
Alternative 4: Combination of Increased Forebay Pump Capacity, Automatic Forebay Outlet Gates and Increase in Reservoir Outlet Gates.

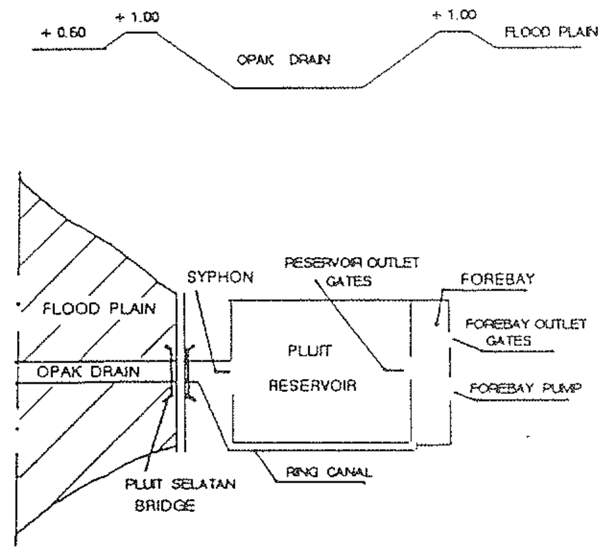
The increase in forebay pumping capacity and the introduction of forebay outlet gates increases the outlet capacity from the forebay to the sea. The elevation in the forebay depends on the flow from the reservoir and ring canal, the larger the flow that enters the forebay, the higher the forebay elevation.

The flow from the Reservoir Outlet Gates can be increased by adding to the capacity of the Reservoir Outlet Gates. In this study, the capacity of the Reservoir Outlet Gates is increased by increasing the number of Reservoir Outlet Gates. This improvement was simulated with Reservoir Outlet Gates of 4 @ 2.00 m (the current Reservoir Outlet Gates are 3 @ 2.00 m).

5.5.2 ANALYSIS OF THE RESULTS OF THE PROPOSED IMPROVEMENTS TO THE OUTLET SYSTEM

Inflow Hydrograph 1 and Inflow Hydrograph 2 were routed through the system for each of the aforementioned improvement alternatives. From these inflow hydrographs, the operation of the Polder was simulated to determine reservoir elevations and the extent of flooding in the flood plain upstream of Pluit Selatan Bridge. The results of the simulation of the alternatives are shown in Figures 5.23 - 5.42. From these





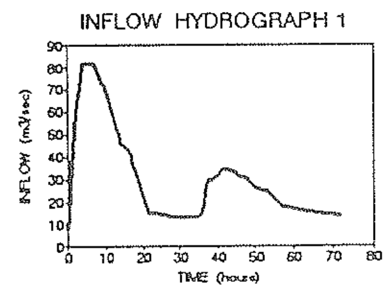
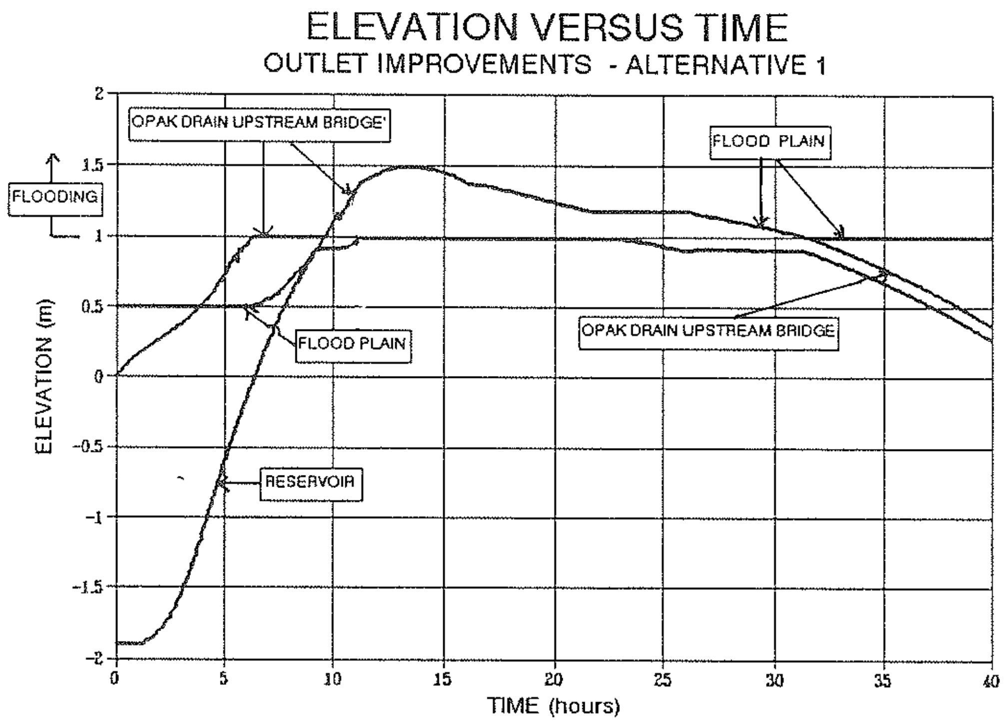
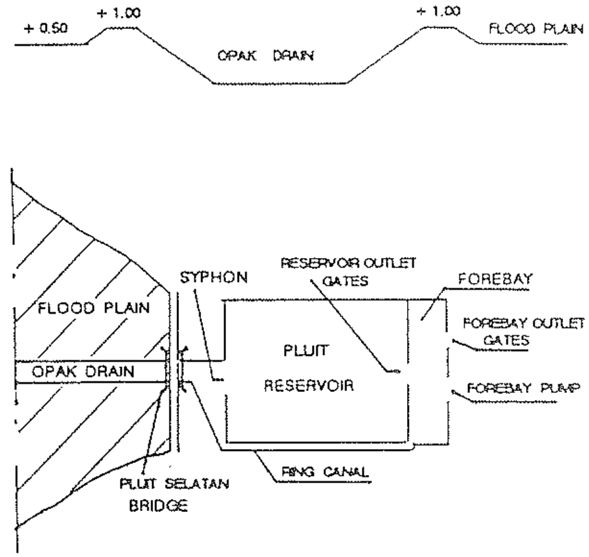


Figure 5.23 Elevation versus Time for Outlet Improvement Alternative 1 (Pump Capacity = 20 m3/sec using Inflow Hydrograph 1)





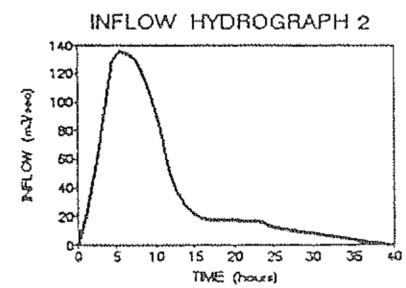
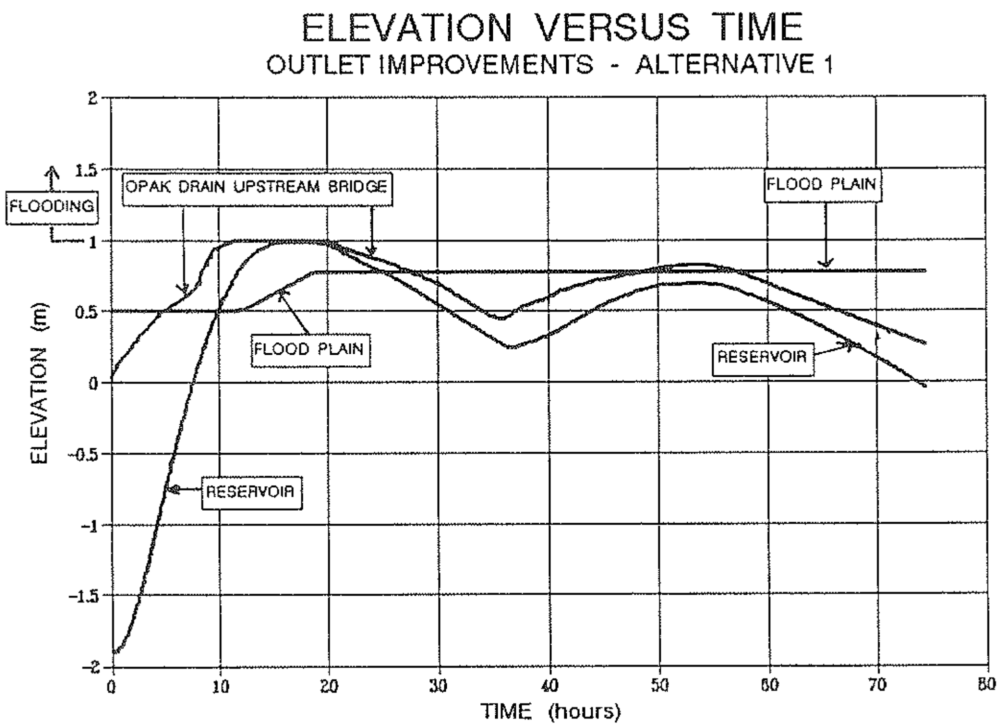


Figure 5.24 Elevation versus Time for Outlet Improvement Alternative 1 (Pump Capacity = 20 m3/sec using Inflow Hydrograph 2)



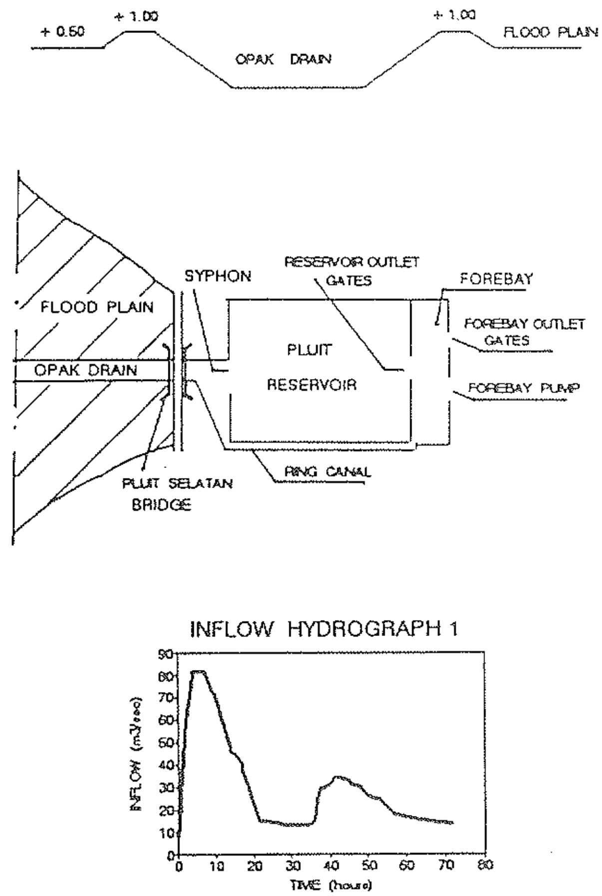
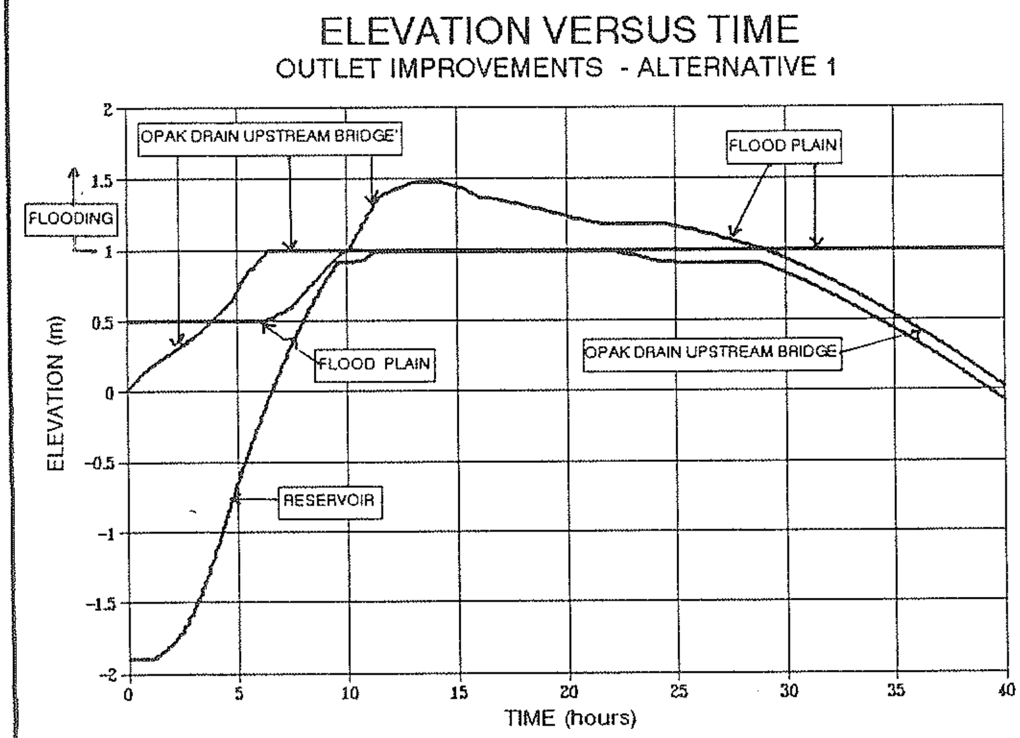
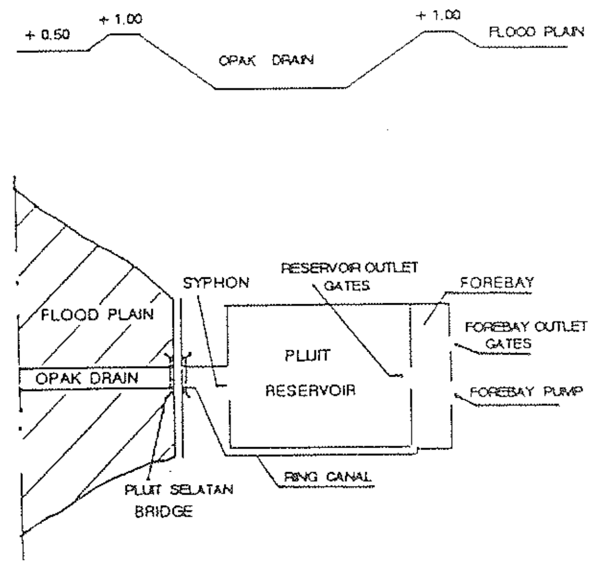


Figure 5.25 Elevation versus Time for Outlet Improvement Alternative 1 (Pump Capacity = 24 m3/sec using Inflow Hydrograph 1)





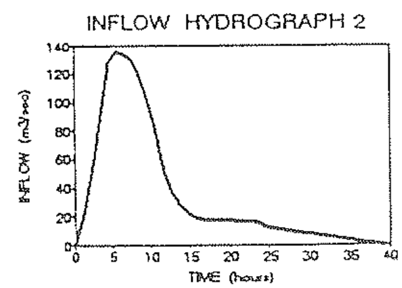
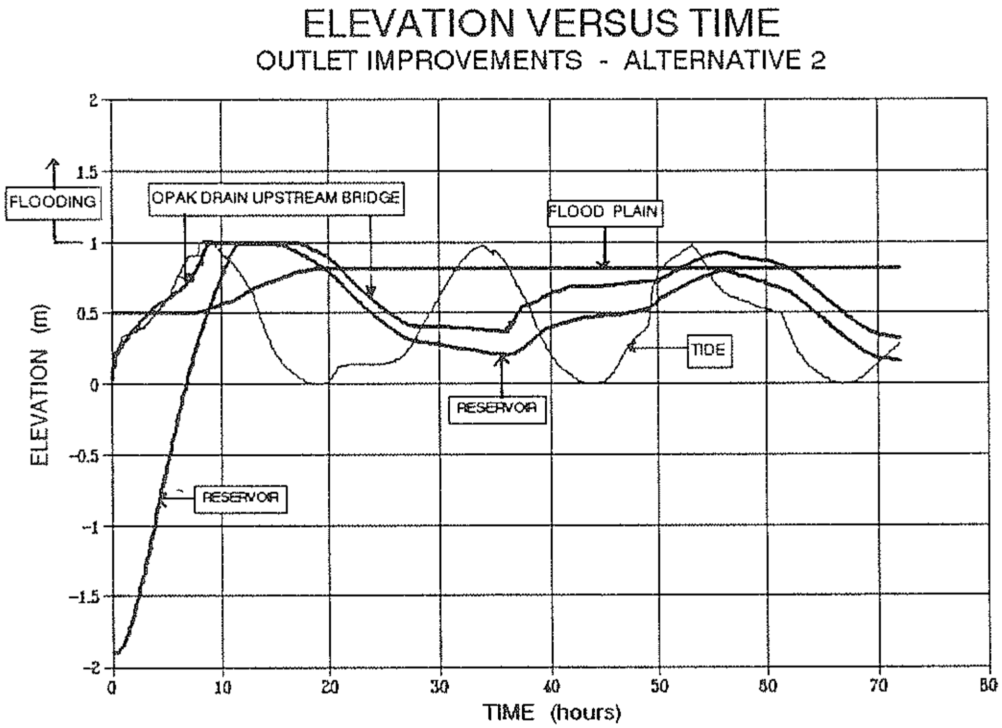
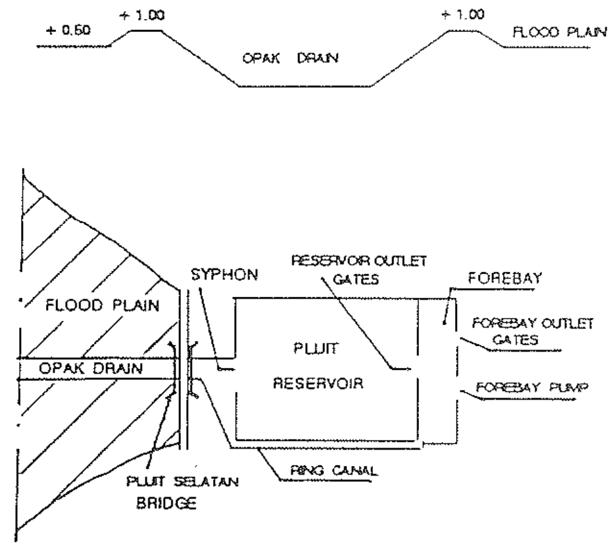


Figure 5.26 Elevation versus Time for Outlet Improvement Alternative 1 (Pump Capacity = 24 m3/sec using Inflow Hydrograph 2)





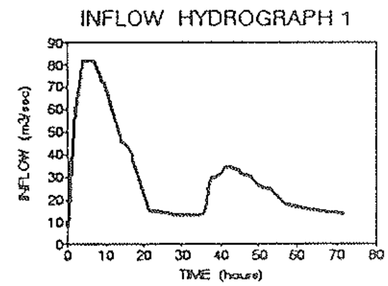
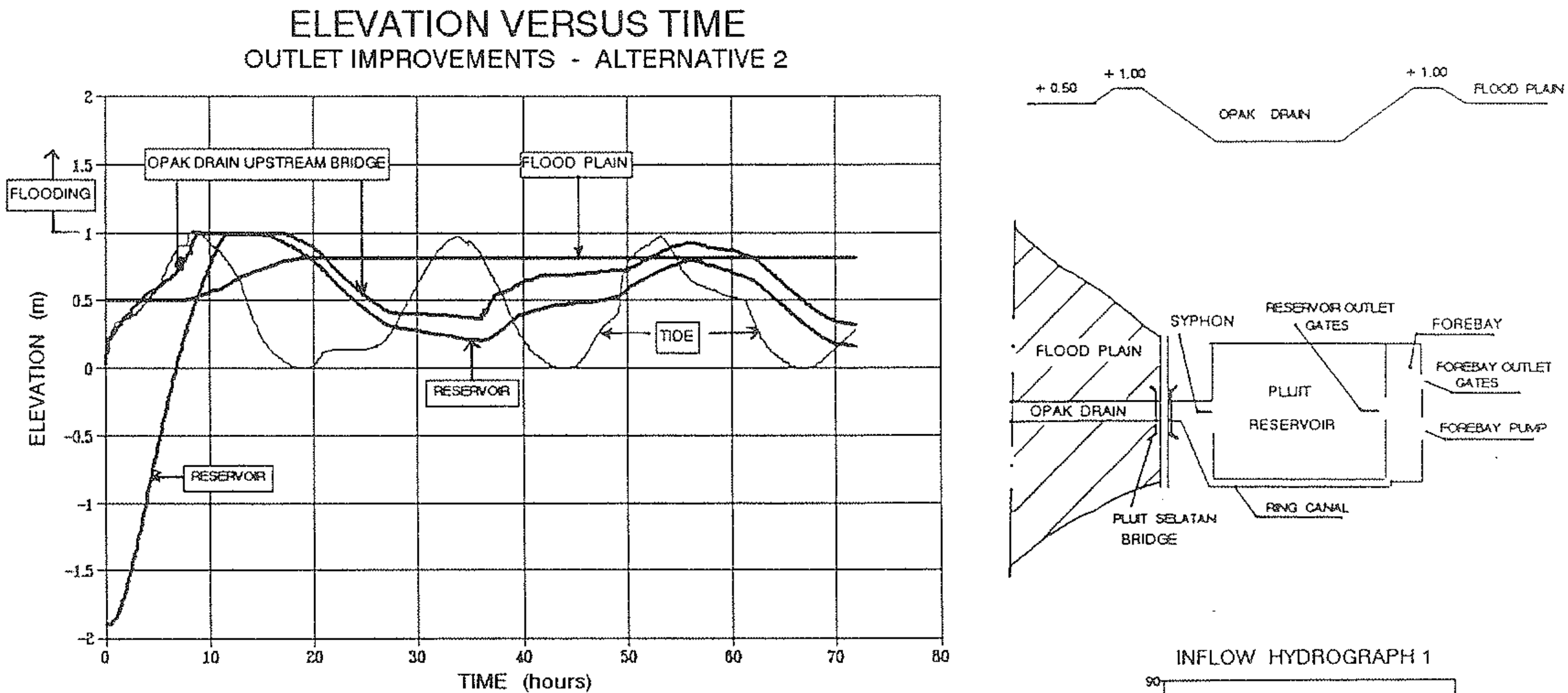


Figure 5.27 Elevation versus Time for Outlet Improvement Alternative 2 (Forebay Outlet Gates 2x2 m using Inflow Hydrograph 1)



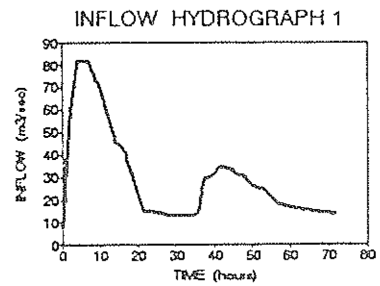
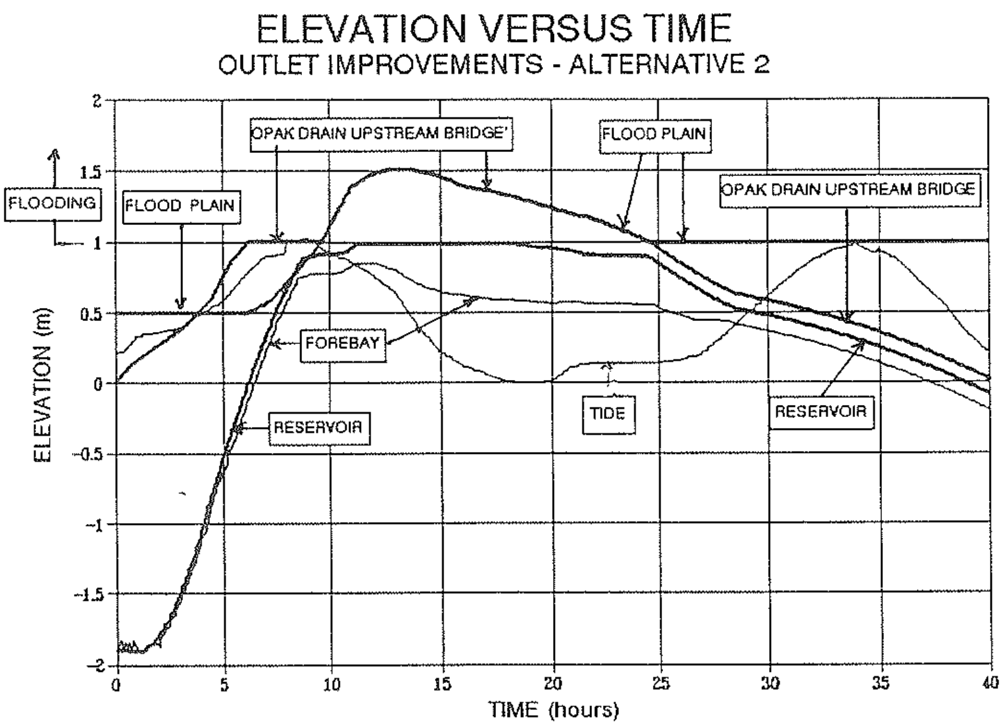


Figure 5.28 Elevation versus Time for Outlet Improvement Alternative 2 (Forebay Outlet Gates 3x2 m using Inflow Hydrograph 1)



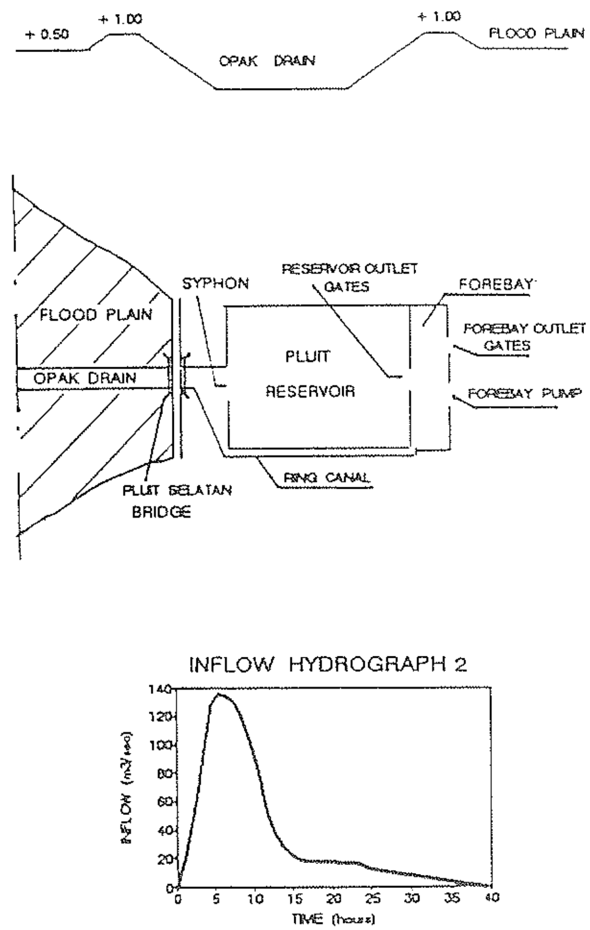
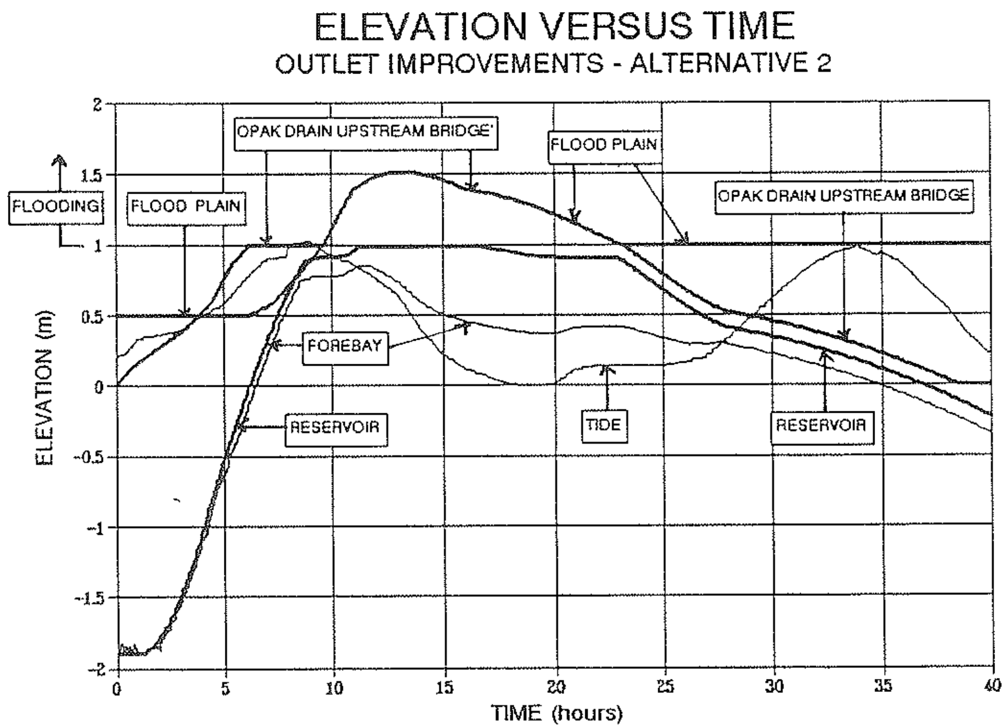
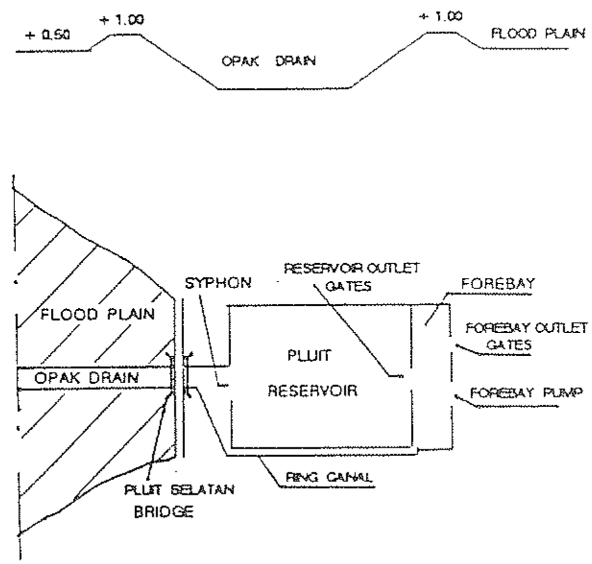


Figure 5.29 Elevation versus Time for Outlet Improvement Alternative 2 (Forebay Outlet Gates 2x2 m using Inflow Hydrograph 2)





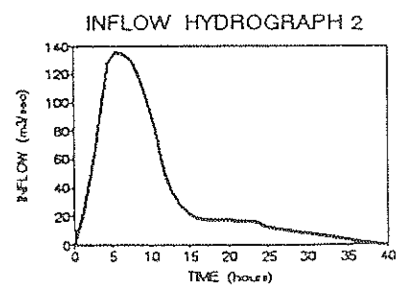
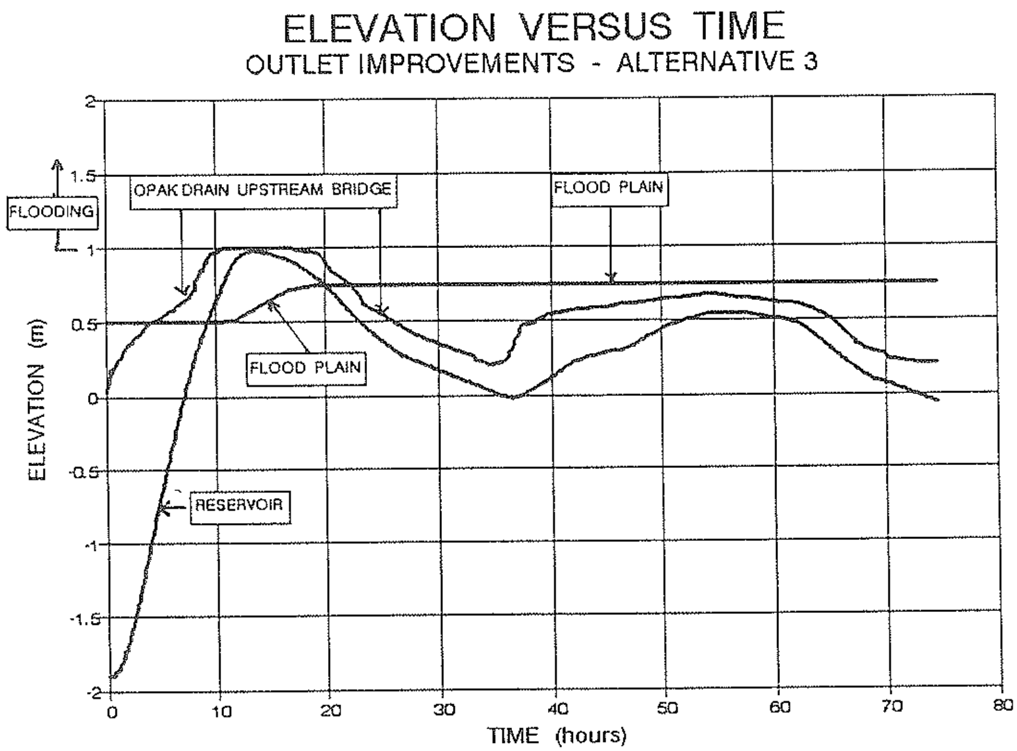
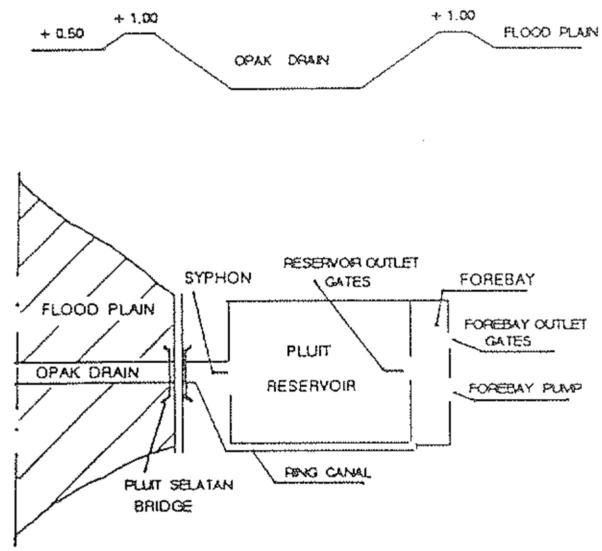


Figure 5.30 Elevation versus Time for Outlet Improvement Alternative 2 (Forebay Outlet Gates 3x2 m using Inflow Hydrograph 2)





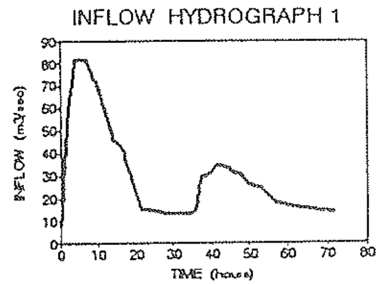
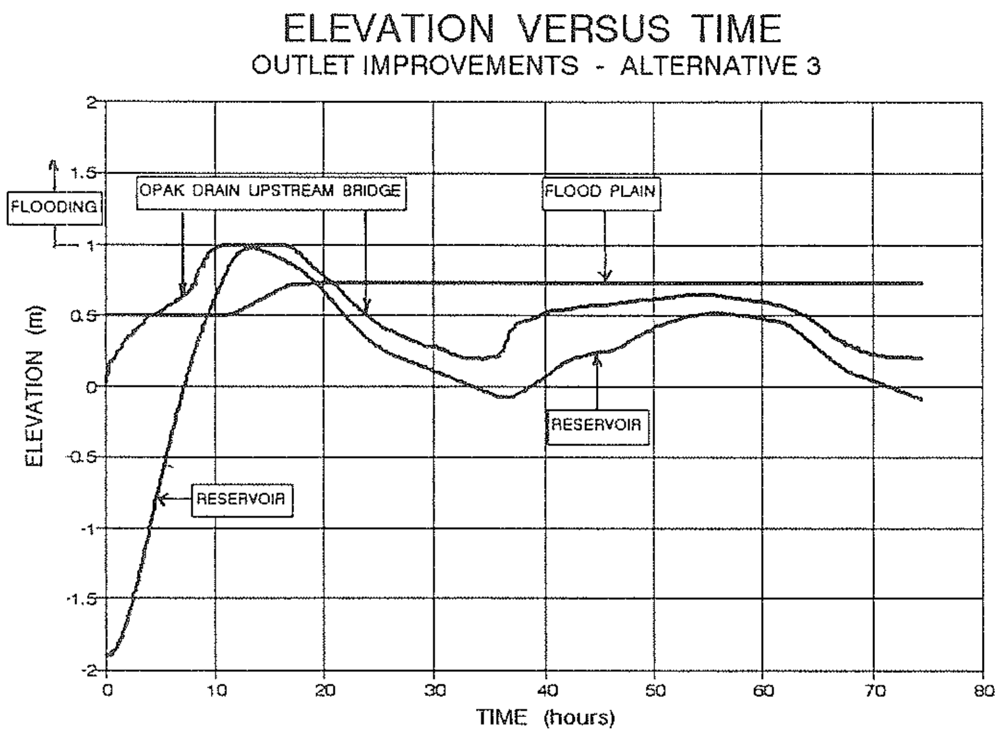
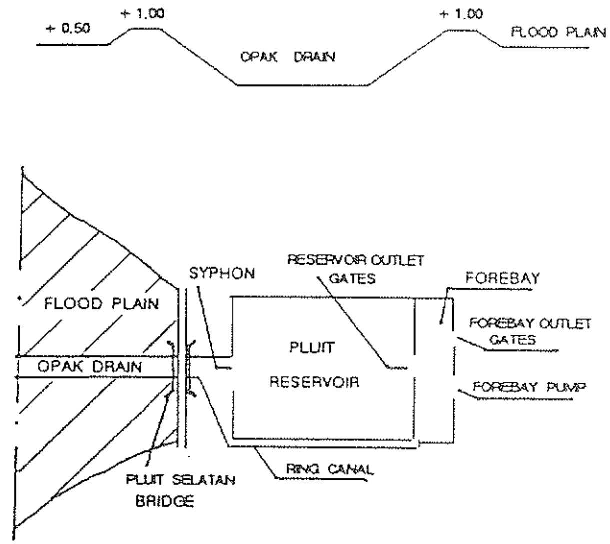


Figure 5.31 Elevation versus Time for Outlet Improvement Alternative 3 (Pump capacity = 20 m3/sec; Forebay Outlet Gates 2x2 m using Inflow Hydrograph 1)





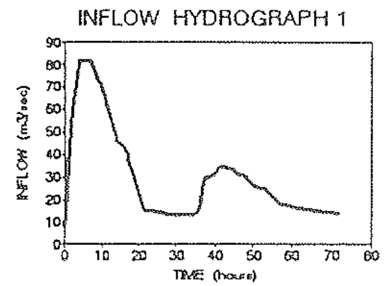
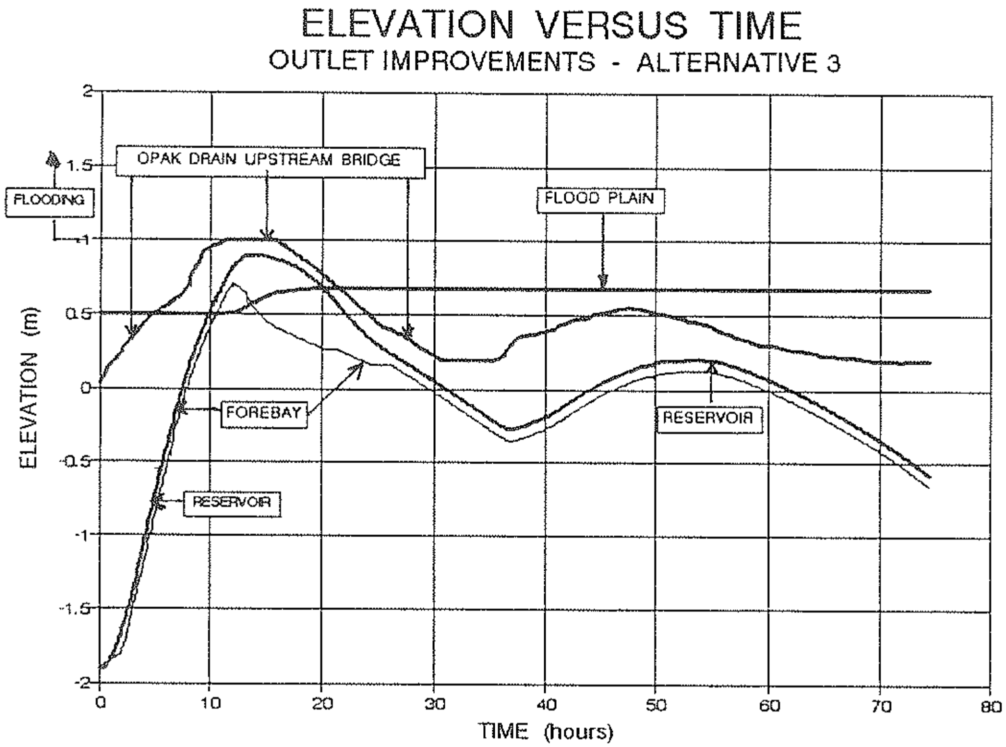
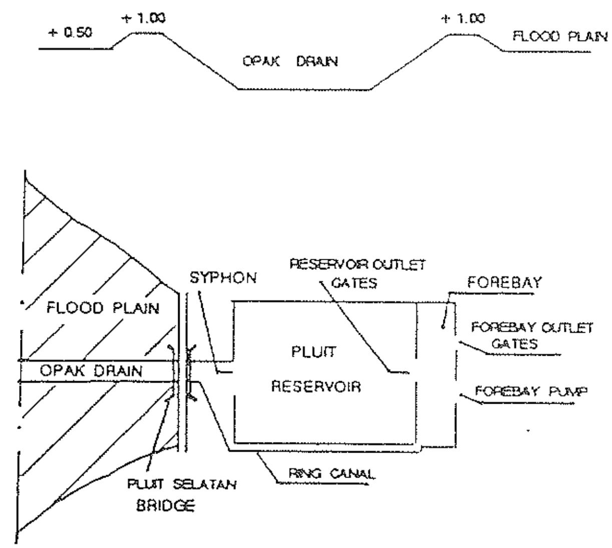


Figure 5.32 Elevation versus Time for Outlet Improvement Alternative 3 (Pump capacity = 20 m3/sec; Forebay Outlet Gates 3x2 m using Inflow Hydrograph 1)





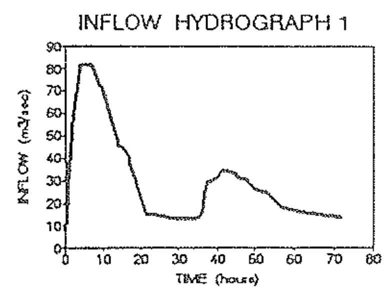
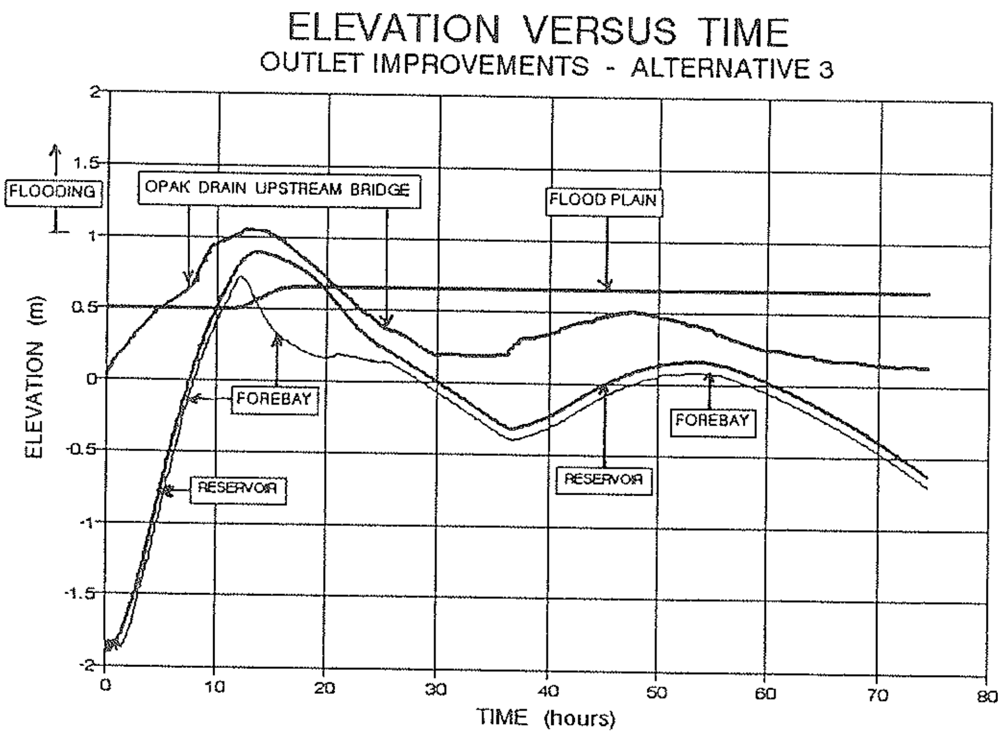
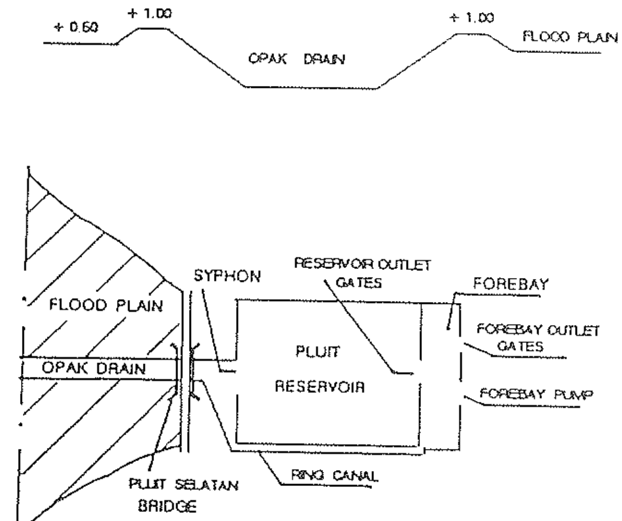


Figure 5.33 Elevation versus Time for Outlet Improvement Alternative 3 (Pump capacity = 24 m3/sec; Forebay Outlet Gates 2x2 m using Inflow Hydrograph 1)





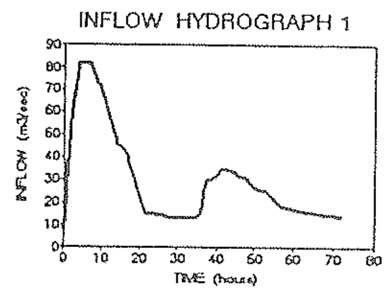
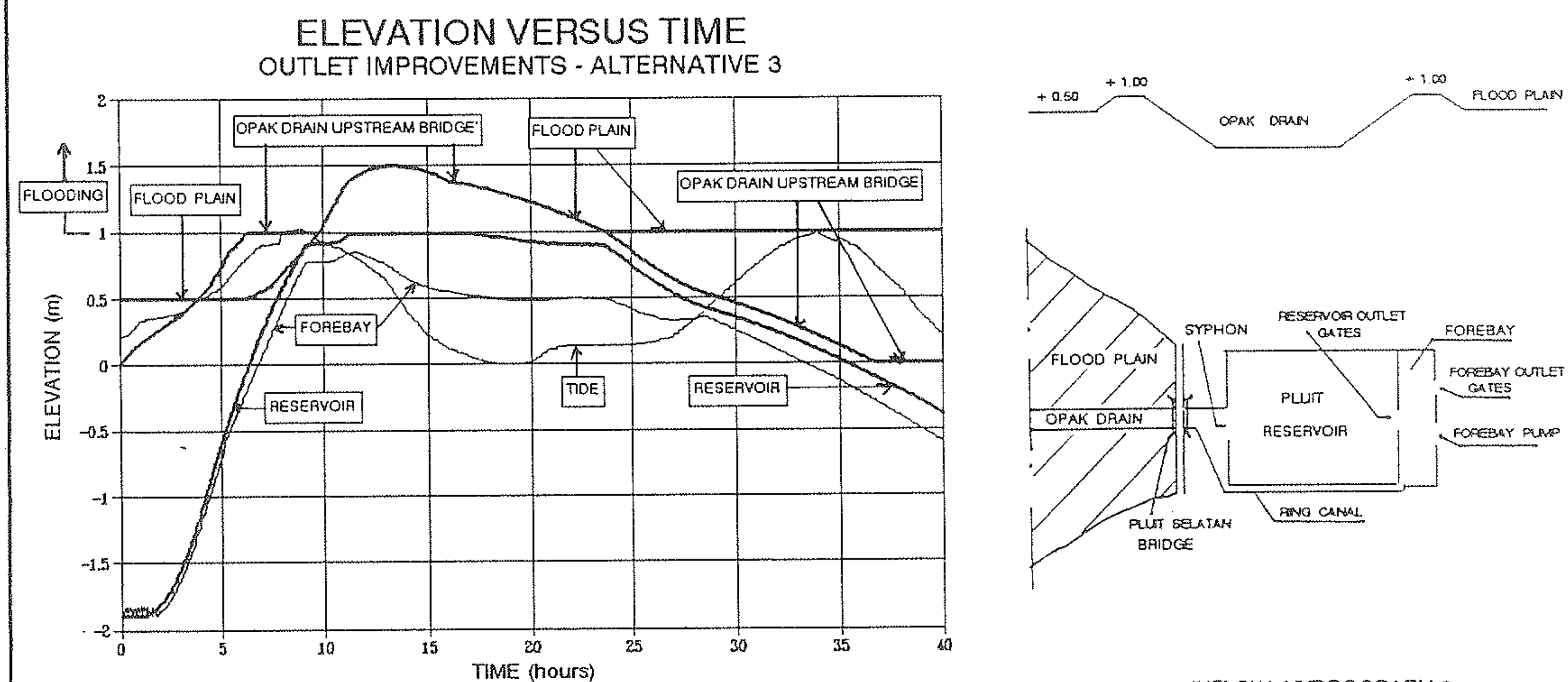


Figure 5.34 Elevation versus Time for Outlet Improvement Alternative 3 (Pump capacity = 24 m3/sec; Forebay Outlet Gates 3x2 m using Inflow Hydrograph 1)



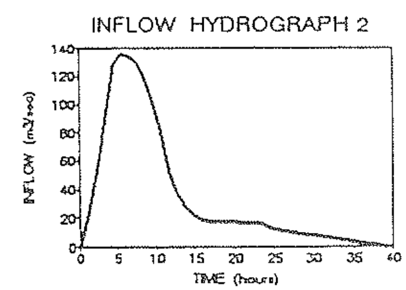
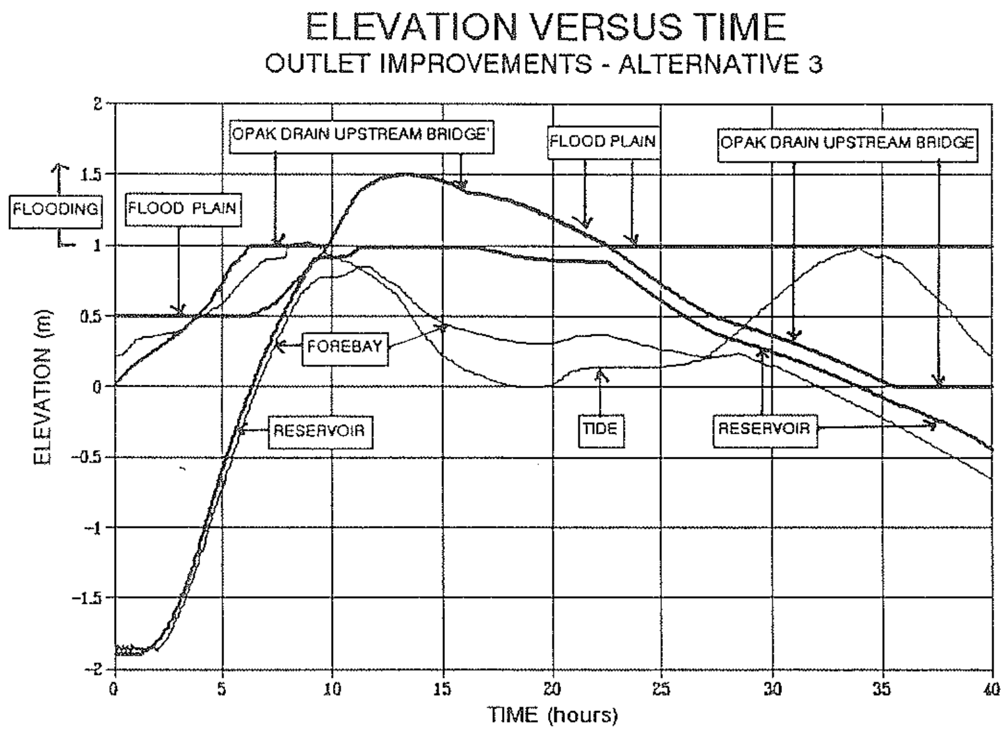
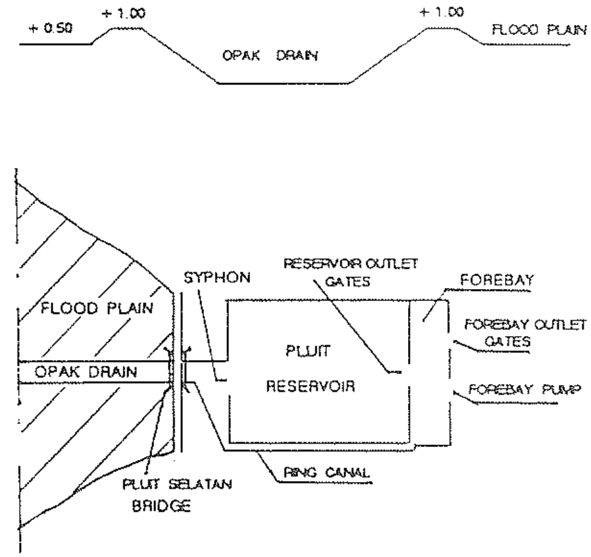


Figure 5.35 Elevation versus Time for Outlet Improvement Alternative 3 (Pump capacity = 20 m3/sec; Forebay Outlet Gates 2x2 m using Inflow Hydrograph 2)





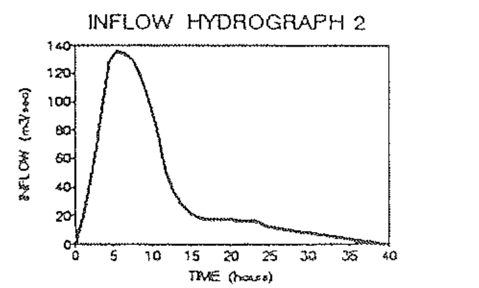
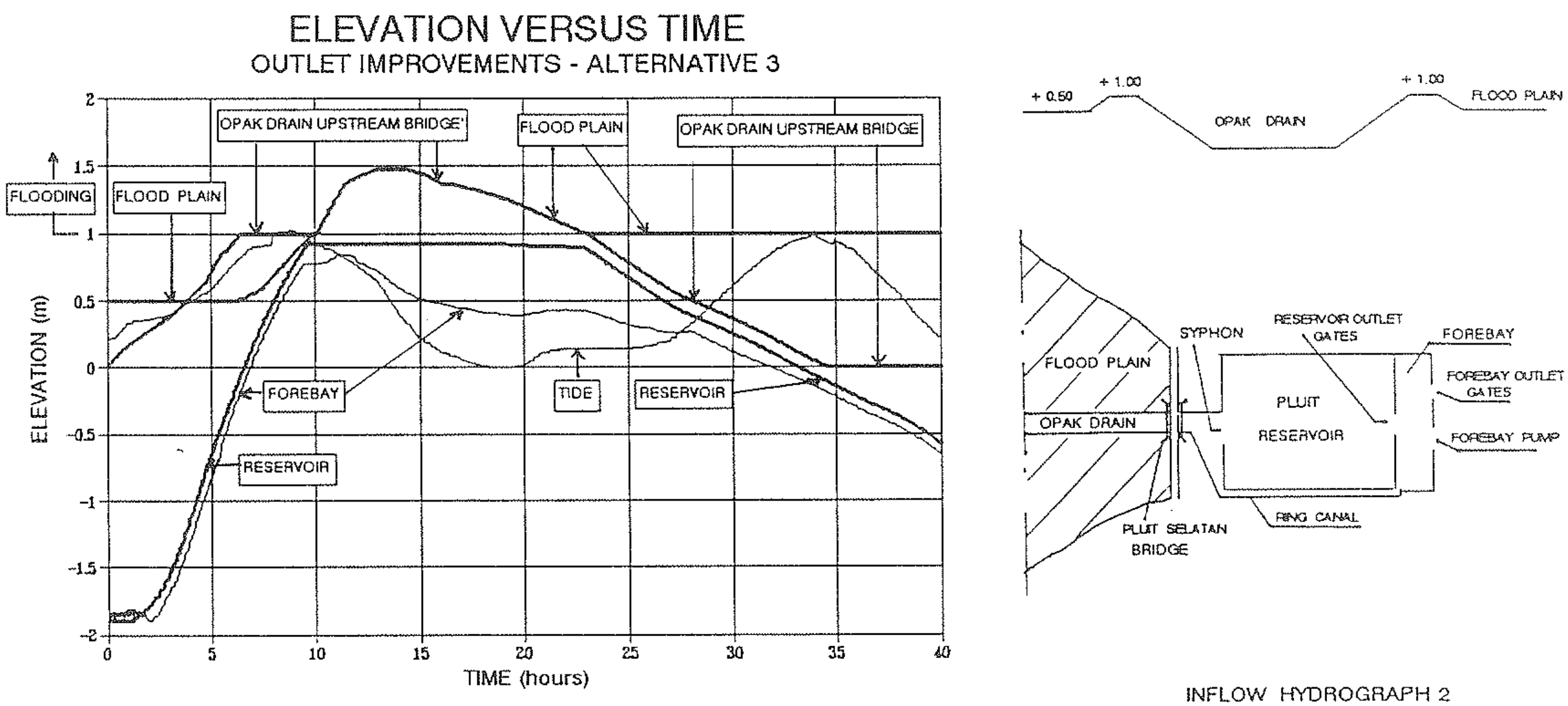


Figure 5.36 Elevation versus Time for Outlet Improvement Alternative 3 (Pump capacity = 20 m3/sec; Forebay Outlet Gates 3x2 m using Inflow Hydrograph 2)



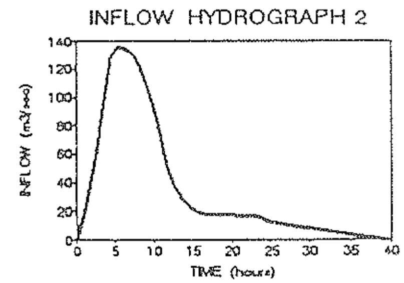
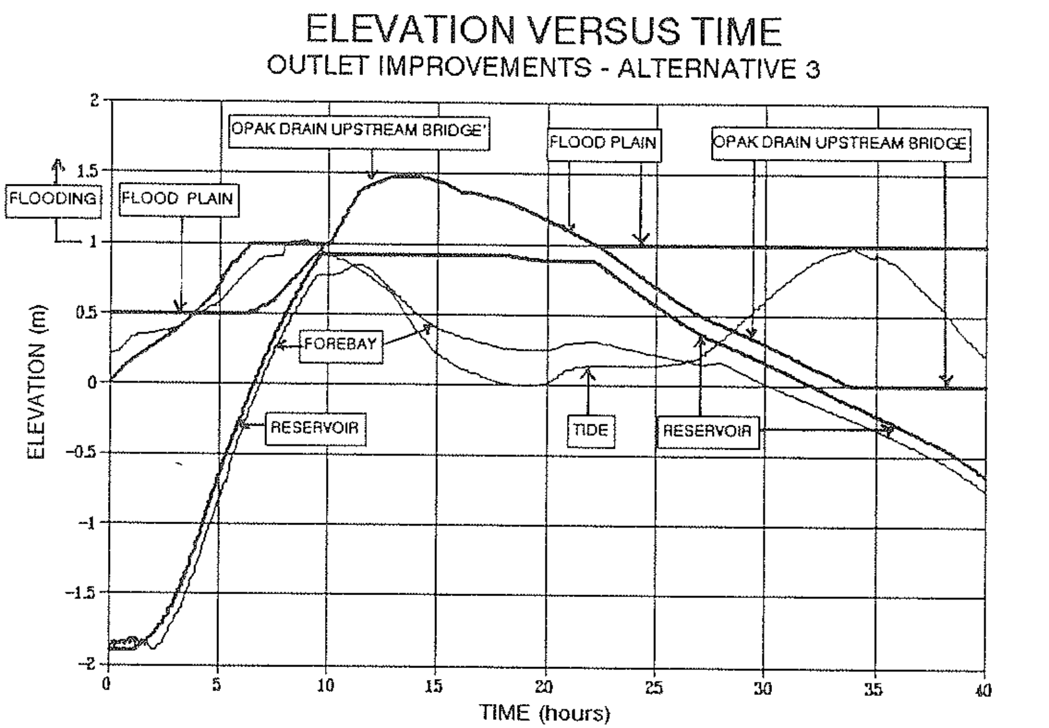
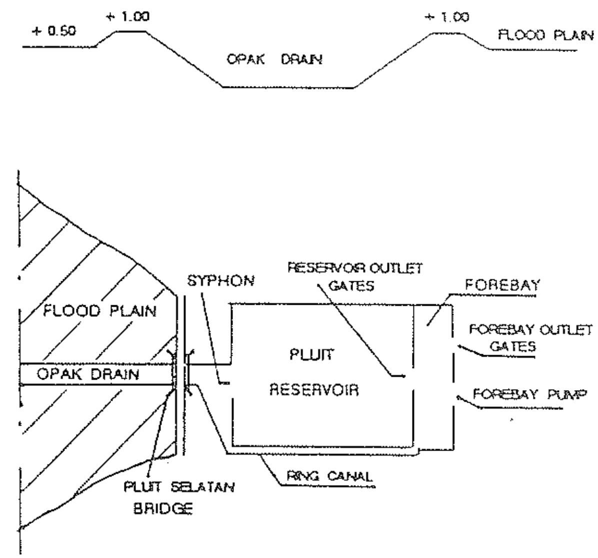


Figure 5.37 Elevation versus Time for Outlet Improvement Alternative 3 (Pump capacity = 24 m3/sec; Forebay Outlet Gates 2x2 m using Inflow Hydrograph 2)





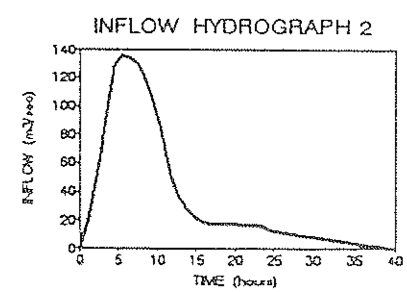
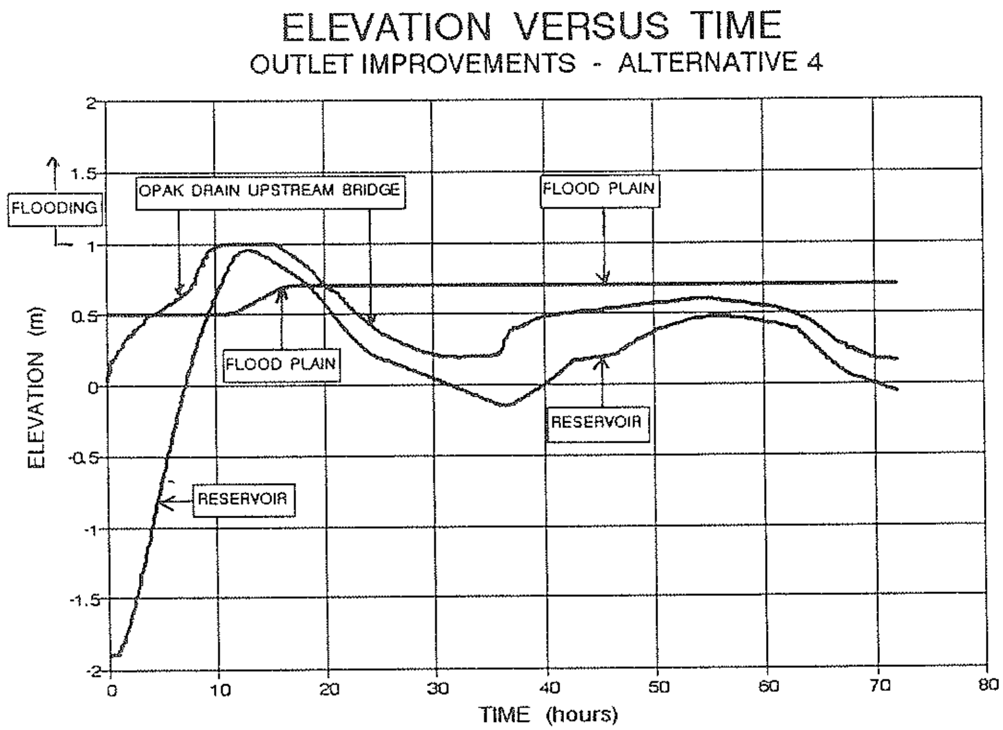
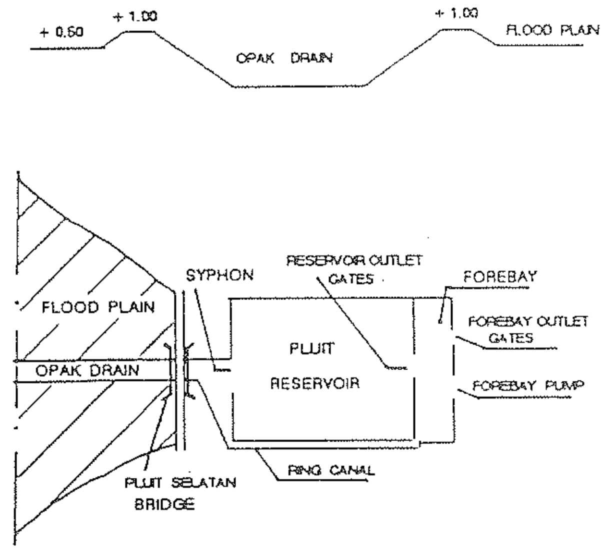


Figure 5.38 Elevation versus Time for Outlet Improvement Alternative 3 (Pump capacity = 24 m3/sec; Forebay Outlet Gates 3x2 m using Inflow Hydrograph 2)





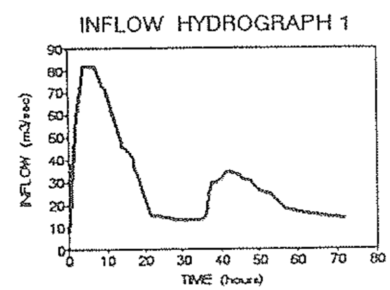
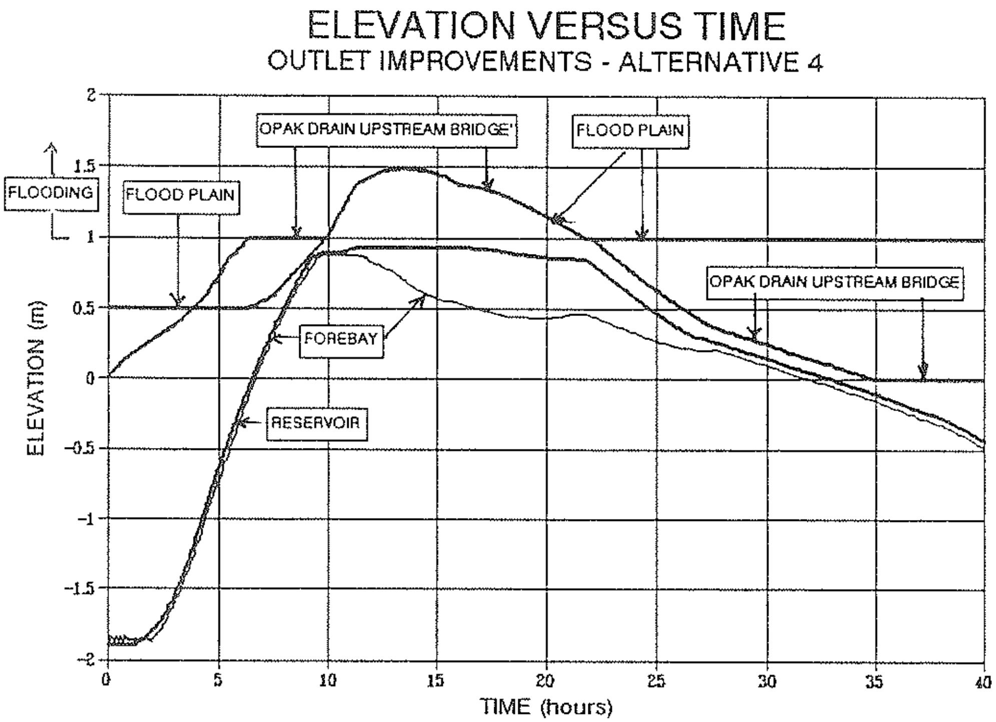
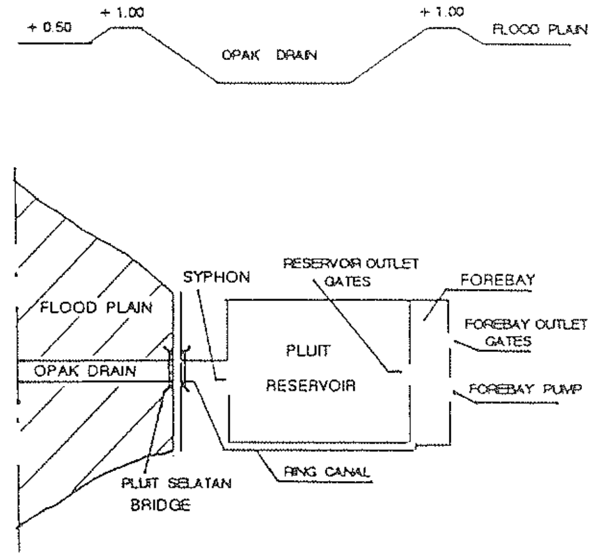


Figure 5.39 Elevation versus Time for Outlet Improvement Alternative 4 (Pump capacity = 20 m3/sec; Forebay Outlet Gates 3x2 m; Reservoir Outlet Gates 8 m using Inflow Hydrograph 1)





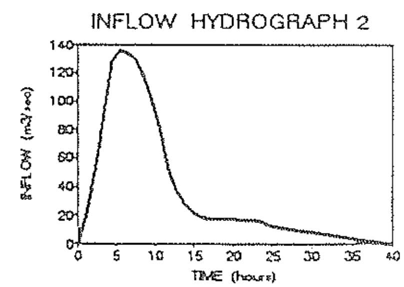
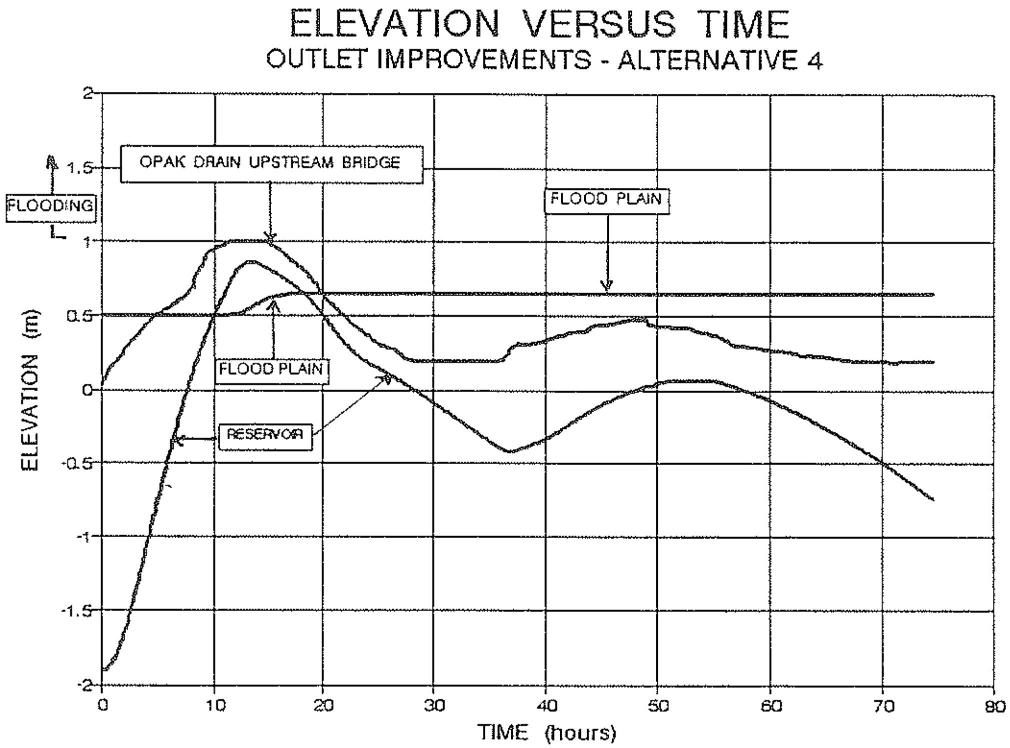
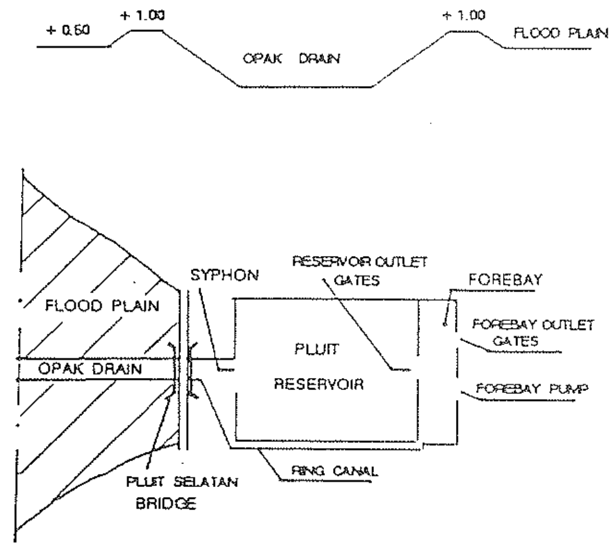


Figure 5.40 Elevation versus Time for Outlet Improvement Alternative 4 (Pump capacity = 20 m3/sec; Forebay Outlet Gates 3x2 m; Reservoir Outlet Gates 8 m using Inflow Hydrograph 2)





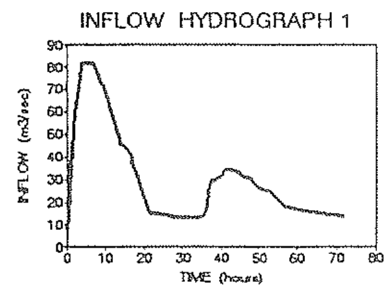
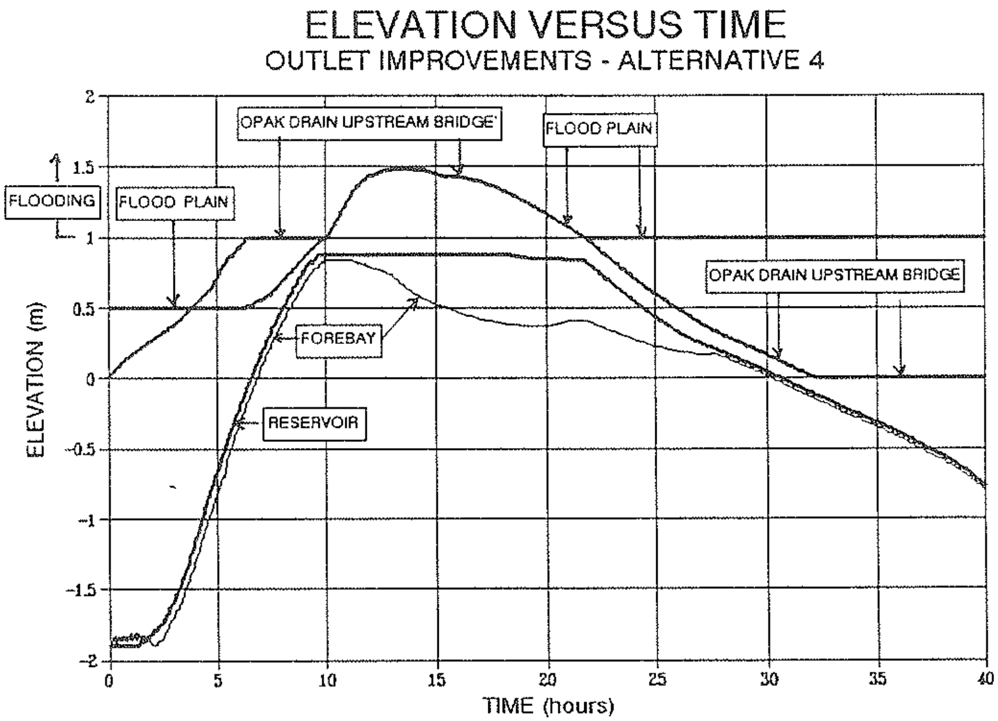


Figure 5.41 Elevation versus Time for Outlet Improvement Alternative 4 (Pump capacity = 24 m3/sec; Forebay Outlet Gates 3x2 m; Reservoir Outlet Gates 8 m using Inflow Hydrograph 1)



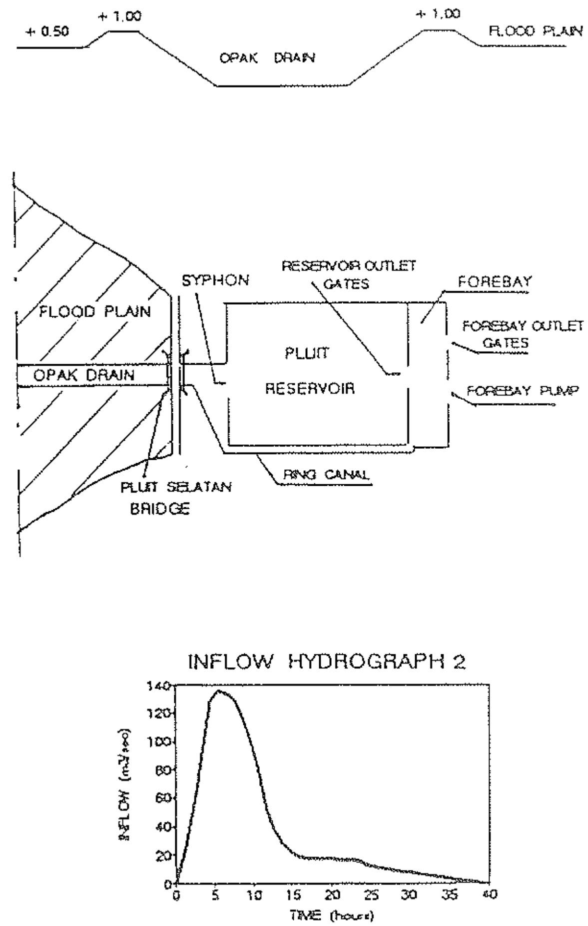


Figure 5.42 Elevation versus Time for Outlet Improvement Alternative 4 (Pump capacity = 24 m3/sec; Forebay Outlet Gates 3x2 m; Reservoir Outlet Gates 8 m using Inflow Hydrograph 2)

results, the following observations and conclusions are derived:

Alternative 1: Increase the Forebay Discharge Pump Capacity

As seen in Figures 5.23 - 5.26 improving the pump capacity results in a decrease in Reservoir elevation. This decrease in reservoir elevation increases the syphon capacity and decreases the total flood volume. The amount of reduction in flooding depends on the amount of increase to the pump capacity. The investigation shows that when increasing the pump capacity to 20 m^3/s (from 16 m^3/s) considering inflow hydrograph 1 (Figure 5.23), flooding still occurs as a result of the first storm. Since the reservoir elevation is still high after the first storm, flooding also occurs as a result of the second storm. This improvement does reduce, however, the amount of flooding in the Flood Plain upstream of Pluit Selatan Bridge. From the results of the simulation, the total volume of flooding is reduced 221,000 m^3, from 798,000 m^3 to 577,000 m^3. Using Inflow Hydrograph 2 (Figure 5.24), this increase in pump capacity results in a reduction of flood volume of 19,000 m^3 from 1,179,000 m^3 to 1,160,000 m^3. By increasing the pump capacity to 24 m^3/s and using Inflow Hydrograph 1 (Figure 5.25), the flooding upstream of the Pluit Selatan Bridge still occurs, but the volume of flood water is reduced from 798,000 m^3 to 325,000 m^3.

Under these conditions increased capacity in the Opak Drain would also be needed because the Inflow Hydrograph 1 also causes the water level to reach the top of the Opak Drain banks. Using Inflow Hydrograph 2 (Figure 5.26), flooding still occurs. This is because the Opak Drain Capacity is smaller than the streamflow and therefore overflows the Opak Drain banks. Increased capacity of the Opak Drain would be necessary. The reduction in volume of flooding after this improvement is 38,000 m^3, from 1,179,000 m^3 to 1,141,000 m^3.

Alternative 2: Introducing Automatic Forebay Outlet Gates

Introducing automatic forebay outlet gates with the existing forebay discharge pumps reduces the flooding in the Flood Plain upstream of the Pluit Selatan Bridge caused by Inflow Hydrograph 1 (Figures 5.27 and 5.28). The reduction of flooding occurs when the tidal elevation is lower than the Forebay elevation (at time about 15 hours). At the beginning of the first storm, the tidal elevation is higher than the water elevation in the forebay. As a result, no water flows through the forebay outlet gates. After time = 12 hours, the tidal elevation is lower than the water elevation in the forebay and flow through the forebay gates occurs. This improvement also reduces the water elevations in the reservoir and forebay. The reduction of reservoir elevation results in no increase in flooding as a result of the second storm. Introducing automatic forebay outlet gates in addition to existing discharge pumps would not reduce the amount of flooding in the flood plain upstream of the Pluit Selatan Bridge due to the storm created by Inflow Hydrograph 2 (Figures 5.29 and 5.30). This is because the tidal elevation was high during the storm. Therefore no flow through the forebay automatic gates would occur. However, it would reduce the water elevations in the reservoir and forebay in the event that another storm would occur shortly after.

In assessing the effectiveness of the automatic forebay gates, a primary concern is the probability that the tidal elevation is above the forebay elevation during the peak runoff period. The flow through the forebay outlet gates depends on the tidal elevation. It is impossible to predict the occurrence of the storm runoff with the high tidal elevations. Should this combination of events occur, there would be no flow through the forebay gates. This risk is significant because the tidal elevation is high 50 percent of the time. (It should be noted that the tide was high during the beginning of Inflow Hydrograph 1.)

Alternative 3: Combination of Increased Forebay Pump Capacity and Automatic Forebay Outlet Gates.

The results in Figures 5.31 - 5.38 shows that the combination of increased forebay pump capacities and introduction of automatic forebay outlet gates gives better results than using Alternative 1 or Alternative 2. The Reservoir elevation at the end of the storm is low so that capacity exists in case another storm occurs.

Using Inflow Hydrograph 1 (Figures 5.31 - 5.34), the flooding upstream resulting from the first storm was reduced by the increase in pump capacity. This condition occur because at the beginning of the first storm, the tidal elevation is higher than the water elevation in the forebay. Therefore no flow through the forebay gates occurs. The amount of reduction in flooding depends on the amount of increase to the pump capacity (as discussed in the result of Alternative 1). After the peak flood from the first storm passes, the tidal elevation is lower than the water elevation in the forebay and the flow through the forebay gates occurs. This results in a lower reservoir level at the beginning of the second storm. Therefore no increase in flooding occurs as a result of the second storm. The lowering in the reservoir elevation after the first storm and during the second storm depends on the increase in forebay pump capacity and the dimension of the forebay outlet gates. The following table shows the reservoir elevations and the corresponding available reservoir volumes after the second flood for each Alternative for improvement.

Table 5.1 Reservoir Elevation (T = 72 hours, after Inflow Hydrograph 1)

Pump m^3/s	Forebay Outlet Gates m	Reservoir Elevation m	Reservoir Volume Available m^3
20	2 @ 2	0.01	780,000
20	3 @ 2	-0.02	781,000
24	2 @ 2	-0.45	1,119,000
24	3 @ 2	-0.49	1,156,000

Using Inflow Hydrograph 2, flooding in the Flood Plain upstream of the Pluit Selatan Bridge still occurs. However, using this improvement alternative, the forebay elevation and reservoir elevation was reduced prior to the flood reaching the inlet Works, which results in the benefit that the amount of flooding is reduced upstream and the duration of flood is shortened. Flooding still occurs because the Opak Drain capacity is smaller than the streamflow. The reduction in Reservoir elevation is very important in reducing the hazard from further successive storms. Improvement of the Opak Drain capacity is still necessary to overcome the flooding, since the inflow is larger than the Opak Drain channel capacity. The following table shows the reduction in reservoir elevation and the corresponding available reservoir volume after the second flood using the improvement alternative.

Table 5.2 Reservoir Elevation (T= 37,9 hours, after INFLOW HYDROGRAPH 2)

Pump capacity m^3/s	Forebay Outlet Gates m	Reservoir Elevation m	Volume Reservoir Available m^3
20	2 @ 2	- 0.20	931,000
20	3 @ 2	- 0.26	976,000
24	2 @ 2	- 0.36	1,043,000
24	3 @ 2	- 0.43	1,113,000

Alternative 4: Combination of Increase Forebay Pump Capacity, Automatic Forebay Outlet Gates and Increase in Reservoir Outlet Gates

A further increase in outlet capacity can be gained by increasing the size of reservoir outlet gates from 3 @ 2.00 m to 4 @ 2.00 m. The use of forebay outlet gates of 3 @ 2.0 m results in a larger difference in water elevation between the reservoir and forebay during low tide periods. As a result, more flow from the reservoir could be diverted into the forebay. This leads to a lower reservoir elevation, and increases syphon capacity, and reduces the amount of flooding. The following combinations of alternative improvements considering reservoir outlet gates were simulated:

Combination A: Pump capacity of 20 m^3/s and Forebay Gates 3 @ 2.00 m and increase Reservoir Outlet Gates from 3 @ 2.00 m to 4 @ 2.00 m (4 gates, each gates 2.00 m width)

Combination B: Pump capacity of 24 m^3/s and Forebay Gates 3 @ 2.00 m and increase Reservoir Outlet Gates from 3 @ 2.0 m to 4 @ 2.0 m.

The results (Figures 5.39 - 5.42) show that the increased size of reservoir outlet gates increases the water elevation in the Forebay and therefore increases the capacity of the Forebay Outlet Gates.

Using Inflow Hydrograph 1, and Combination A gives the following results:

- a reduction in the reservoir elevation, so that at the end of the second storm the available reservoir volume is 940,000 m^3 compared to 905,000 m^3 without increasing outlet gates. Using combination B, the elevation in the reservoir at the end of the second flood becomes 1,322,000 m^3 compared to 1,247,000 m^3. These improvements also reduce the water elevation in the Opak Drain.

Using Inflow Hydrograph 2, flooding upstream occurs when the streamflow is larger than the Opak Drain capacity. However, these improvements reduce the reservoir elevation and forebay elevation which is desirable to prepare the reservoir for another storm. Applying Combination A in the simulation model reduces water elevation in the reservoir to El. -0.27 m, or a corresponding available reservoir volume of 976,000 m^3. While using combination B, the reservoir elevation becomes El.-0.55 m or a corresponding available reservoir volume of 1,210,000 m^3.

Comparison of volume of flooding as a result of each Alternative Modification of the Outlet System are discussed in Section 6, Evaluation of the Alternatives.

5.6 ALTERNATIVES FOR MODIFICATION OF THE INLET SYSTEM

5.6.1 INLET MODIFICATION ALTERNATIVES

The Inlet System consists of:

- the Opak Drain
- the syphon
- the ring canal intake structure and the ring canal

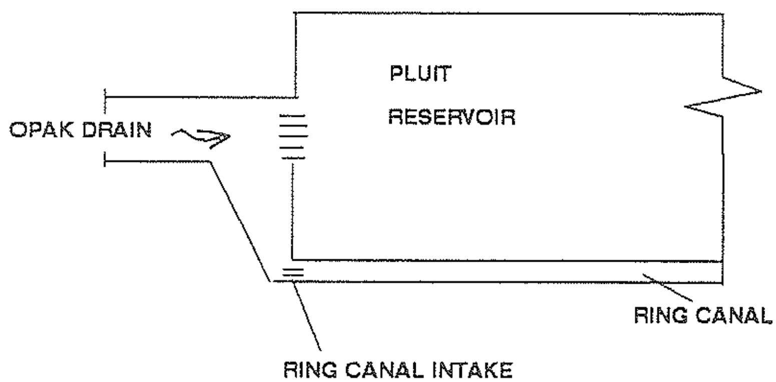


Figure 5.43 Inlet System

In this inlet modification, no improvement alternatives for the syphon are considered. The reason for this choice is that the flow through the syphon depends on the water elevation in the Opak Drain and in the reservoir. Once the reservoir is full, an increase in syphon capacity could result in flooding over the reservoir banks.

This flooding could affect areas that would not otherwise be flooded and could damage the reservoir walls and outlet structures. The results in Section 5.4 show that using the current outlet system, the reservoir fills in a short time implying that the syphon capacity is already sufficiently large enough. Therefore no increase in the syphon capacity is believed to be necessary. However, improvement of the syphon capacity will be considered in Section 5.7 (Combination of Inlet and Outlet System) where the improvements in Outlet System give the possibility for more water flow to the reservoir through the syphon.

The ring canal capacity could be improved by cleaning the channel by cutting and by removing vegetation during dry periods. Such cleaning is not included as an alternative in this improvement of the inlet system because based on the results of Section 5.4, the elevation in the forebay increases faster than the current pumping capacity can lower the forebay elevation. As a result, increasing the ring canal capacity would not improve the system performance. However, improvement of ring canal by cleaning is considered in Section 5.7 (Combination of Alternatives Modification), where improvements to the outlet system to give the possibility of more flow to the Forebay are addressed.

The same inflow hydrographs are used for improvements to the inlet system as were used in the discussion of the Outlet System (Section 5.5).

Based on the above discussion and the results discussed in Subsection 5.4, the improvement to the inlet system only relates to improvement to the Opak Drain. The following alternative improvements to the current operations of the system were simulated using the mathematical model.

Alternative 1: Increased Opak Drain Capacity by Increasing the Bank Elevation

The Opak Drain capacity could be improved by dyking the banks thus increasing the bank elevation. The increase in the bank elevation increases the cross sectional area of the channel and therefore increases the discharge capacity of the channel. In this study an increase in the bank elevation to +1.40 is simulated. This elevation is based on the required elevation to pass the flows generated using the Inflow Hydrograph 2 while limiting the water elevations behind the Pluit Selatan Bridge and at the syphon, so that no flooding occurs at the bridge and no overflow occurs above the syphon.

Alternative 2: Increase the Opak Drain Capacity by Cleaning

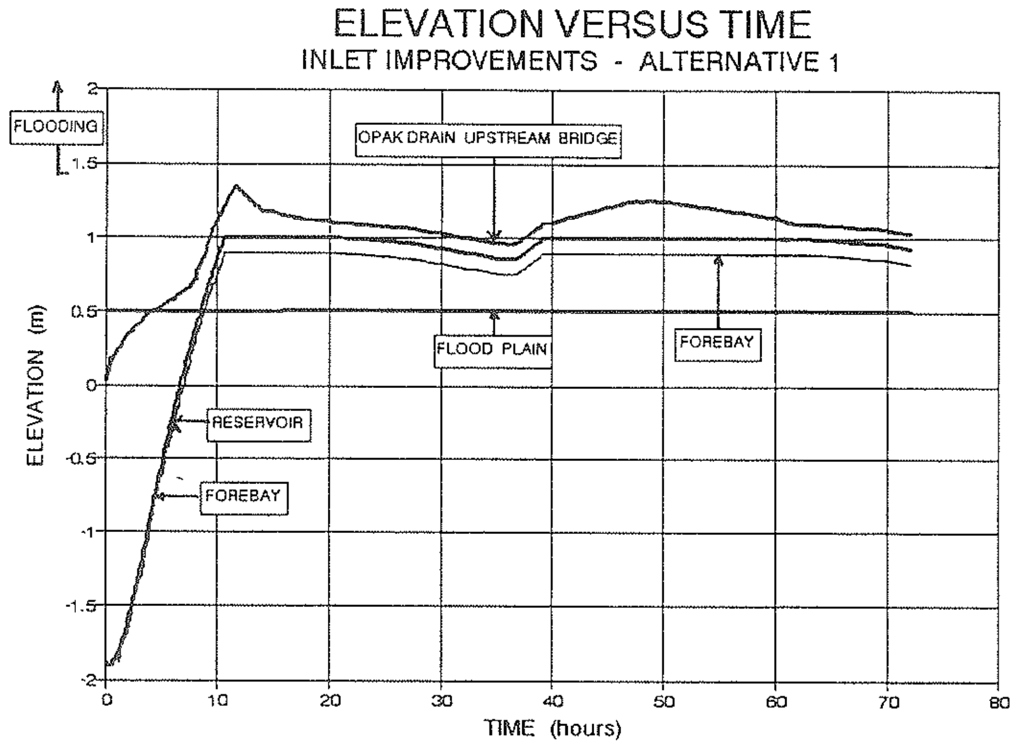
Cleaning of the Opak Drain to remove vegetation and garbage during dry periods could be done by cutting or by removing vegetation from the channel. By cleaning the Opak Drain, the roughness coefficient would be reduced. This reduction in the roughness coefficient would increase the channel capacity. Since it is not possible to clean the channel completely, and ensure that the channel remains clean during the rainy season, a 10% reduction of channel capacity due to garbage is considered in this study. This reduction in channel capacity increases the roughness coefficient to 0.023.

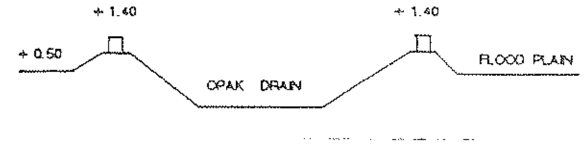
5.6.2 ANALYSIS OF THE RESULTS OF THE PROPOSED IMPROVEMENTS TO THE INLET SYSTEM (OPAK DRAIN)

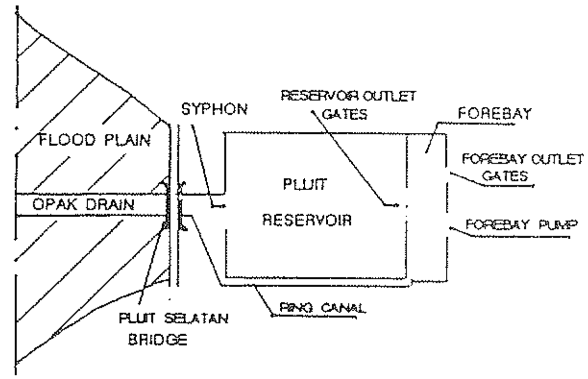
Inflow Hydrograph 1 and Inflow Hydrograph 2 were routed through the system for each of the aforementioned alternative improvements. From these Inflow Hydrographs, the operation of the Polder was simulated to determine Reservoir elevations and the extent of flooding in the floodplain upstream of Pluit Selatan Bridge. The results of the simulation of the alternatives are shown in Figures 5.44 - 5.47 and are discussed below:

Alternative 1: Increase the Discharge Opak Drain Capacity by Increasing the Bank Elevation to +1.40 (0.90 m in height)

The result in Figure 5.44 shows that increasing the bank elevation could overcome flooding in the flood plain upstream the Pluit Selatan Bridge caused by Inflow Hydrograph 1. The







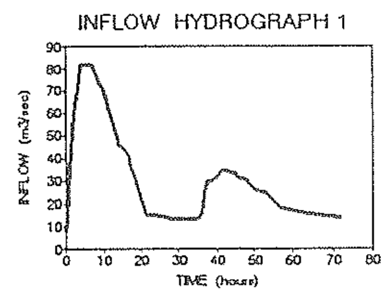
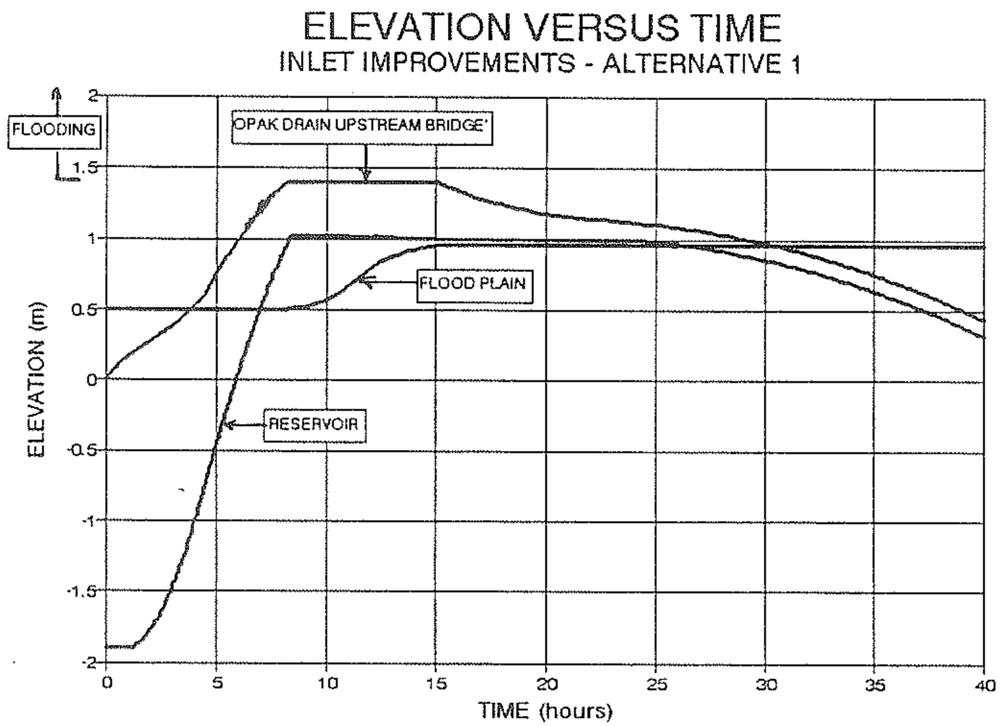
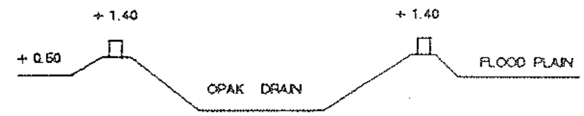
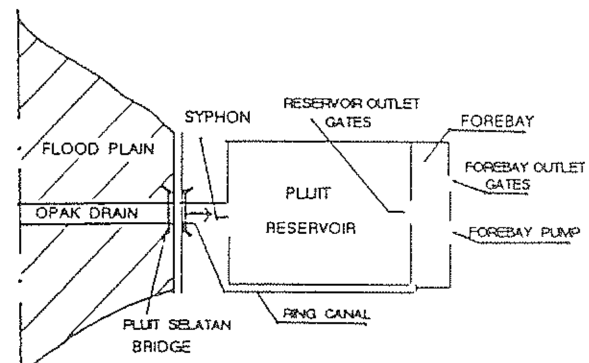


Figure 5.44 Elevation versus Time for Inlet Improvement Alternative 1 (Inflow Hydrograph 1)







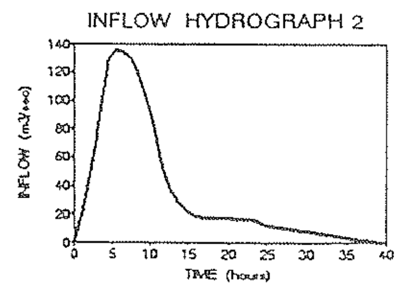
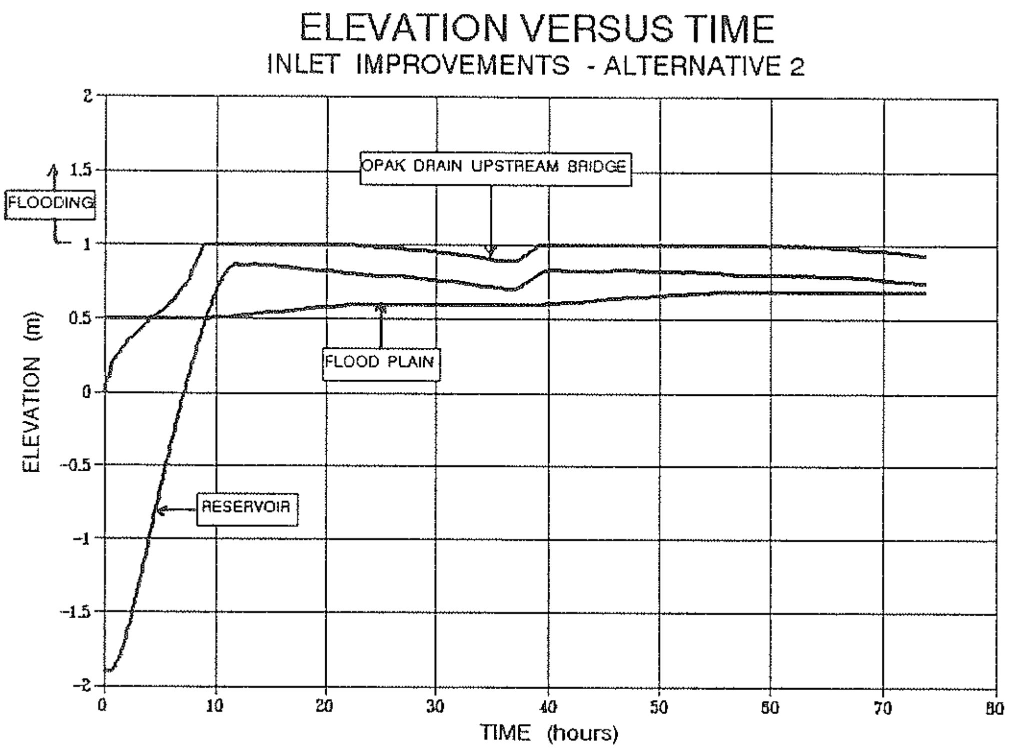
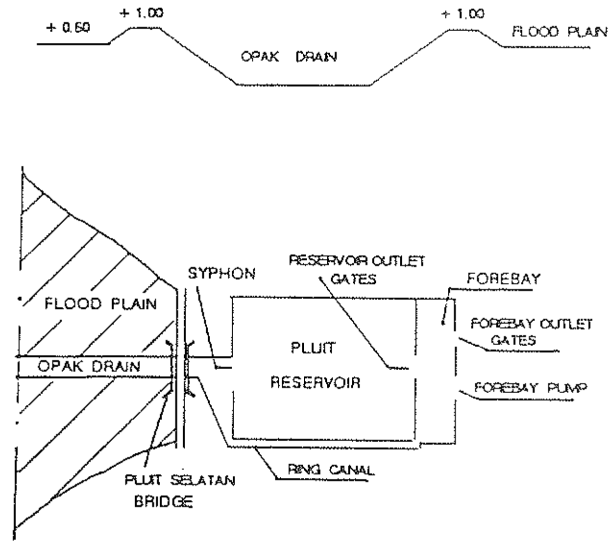


Figure 5.45 Elevation versus Time for Inlet Improvement Alternative 1 (Inflow Hydrograph 2)





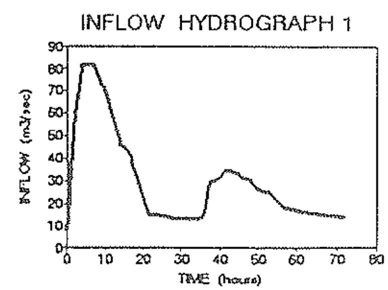
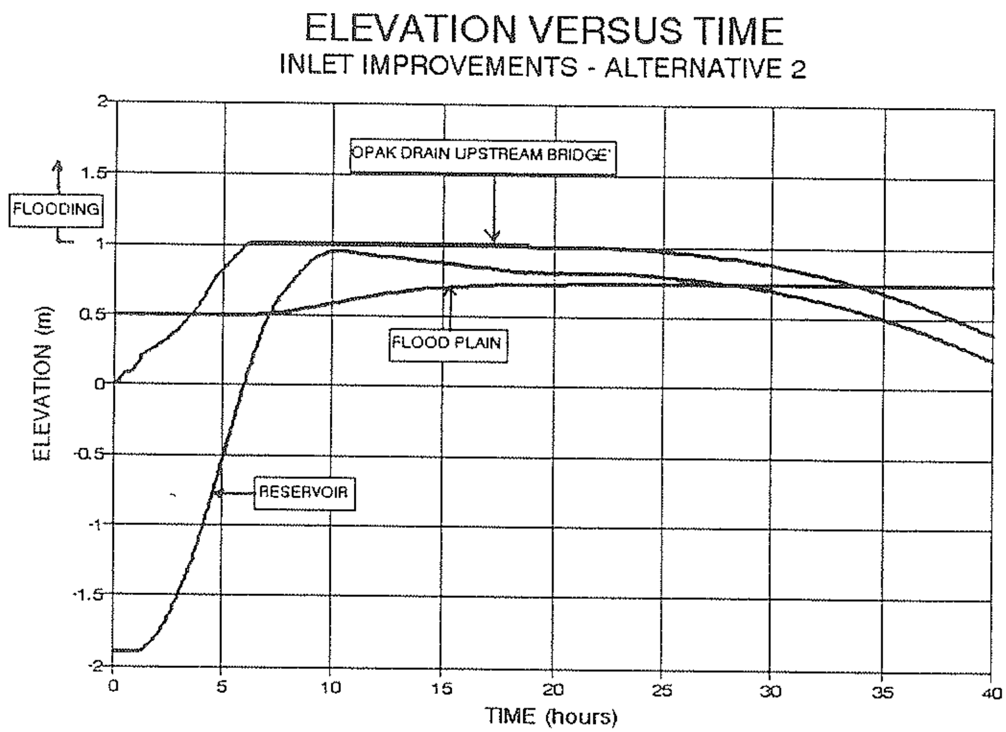
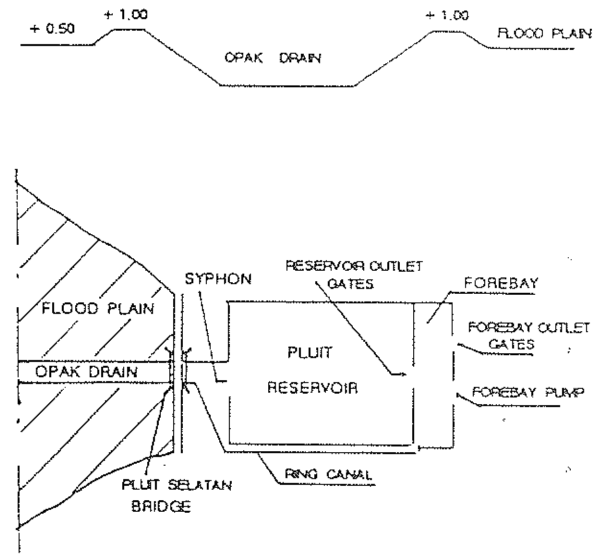


Figure 5.46 Elevation versus Time for Inlet Improvement Alternative 2 (Inflow Hydrograph 1)





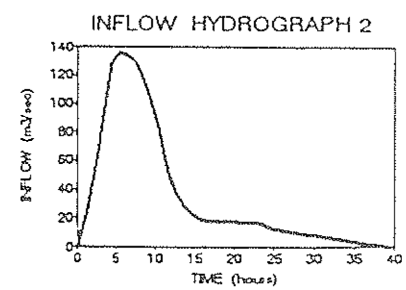


Figure 5.47 Elevation versus Time for Inlet Improvement Alternative 2 (Inflow Hydrograph 2)

channel capacity is sufficient to pass the flow from Inflow Hydrograph 1. However, this improvement leads to higher reservoir and forebay elevations. As a result, of the increased in reservoir elevation, the reservoir would be full when the second storm occurs.

As seen in Figure 5.45 using Inflow Hydrograph 2, improving the Opak Drain capacity would reduce the volume of flooding in the Flood Plain upstream of the Pluit Selatan Bridge during the storm. This result occurs because the channel could pass greater flows. This improvement does not entirely solve the flooding problem. The increase in flow in the Opak Drain means an increase in the water elevation in the Opak Drain. The flow through the syphon would increase because of the increase in the Opak Drain elevation. As a result, the reservoir elevation and forebay elevation would increase. If no improvement to the outlet system is considered, the outflow capacity would still be limited by the current forebay pump capacity. As a result, of the increase in forebay elevation, flow from the Pluit Reservoir would decrease and the reservoir would fill. The resulting increase in reservoir elevation decreases the flow through the syphon. The elevation in the Opak Drain would then increase causing flooding upstream of the Pluit Selatan Bridge. As discussed above, this improvement leads to higher reservoir and forebay elevations. If another storm occurs before the reservoir elevation can be reduced, increased flooding would occur.

Alternative 2: Increase the Discharge Opak Drain Capacity by Cleaning

Cleaning the Opak Drain reduces the roughness coefficient, and therefore increases the channel capacity. As a result, a larger discharge could be passed by the Opak Drain.

The result in Figure 5.46 shows that increase in Opak Drain capacity due to cleaning could reduce total flooding in the Flood Plain caused by Inflow Hydrograph 1 upstream of the Pluit Selatan Bridge by 581,000 m^3 as the channel capacity is sufficient to pass the flows from Inflow Hydrograph 1. However, because of the limitations on outflow capacity, the water elevation in the Opak Drain increases causing flooding upstream of the Pluit Selatan Bridge. Using Inflow Hydrograph 2, this alternative improvement gives a lower volume of flood in the flood plain. However, other improvements are still needed to reduce the flood caused by Inflow Hydrograph 2.

5.7 COMBINATION OF ALTERNATIVE MODIFICATIONS

From the previous discussion the interaction between various parts of the Pluit system is apparent. Improvements to isolated parts of the system do not entirely solve the flooding problem. As a result, Combinations of Upstream and Downstream Alternative Improvements should be considered.

5.7.1 COMBINATIONS

The following combinations of Alternative Improvements were analyzed:

Combination 1

- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation +1.40 m
- Increase the forebay pump capacity to 20 m^3/s.

Combination 2

- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation +1.40 m
- Increase the forebay pump capacity to 24 m^3/s.

Combination 3

- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation +1.40 m
- Increase the forebay pump capacity to 20 m^3/s.
- Introduce of the automatic forebay outlet gates of 2 @ 2.00 m.

Combination 4

- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation +1.40 m.
- Increase the forebay pump capacity to 24 m^3/s.
- Introduce of the automatic forebay outlet gates of 2 @ 2.00 m.

Combination 5

- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation +1.40 m.
- Increase the forebay pump capacity to 20 m^3/s.
- Introduction of the forebay outlet gates of 3 @ 2.00 m.
- Increase the reservoir outlet gates from 3 @ 2.00 m to 4 @ 2.00 m.

Combination 6

- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation +1.40 m.
- Increase the forebay pump capacity to 24 m^3/s.
- Introduction of the forebay outlet gates of 3 @ 2.00 m.
- Increase the reservoir outlet gates from 3 @ 2.00 m to 4 @ 2.00 m.

Combinations 1 to 6 are based on the results in Sections 5.5 and 5.6:

Using Inflow Hydrograph 1, an improvement in the Outlet System (Structures) would reduce the total volume of flooding upstream of the Selatan Bridge and reduce the water elevation in the reservoir and forebay. The reduction in flooding depends on the increase in the forebay pump capacity, the introduction of automatic forebay gates and the increase in the dimensions of the reservoir outlet gates. However, an increase in the Opak Drain Capacity is still needed because the Inflow Hydrograph 1 causes the water level to reach the top of the Opak Drain. Inflow Hydrograph 2 causes flows that are higher than the capacity of the current Opak Drain. Therefore, improvements to both Outlet and Inlet structures are necessary for this case.

Combinations involving only increased Opak Drain capacity and forebay outlet gates were not considered in this study because the forebay outlet gates only work when the water level in the forebay is higher than the tidal level. Using the current pump capacity as explained in Section 5.5, this combination does not provide any increased protection from flooding if the storm occurs during a high tide period.

Combination 7

- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation +1.40 m and cleaning regularly.
- Increase the Forebay Pump capacity to 24 m^3/s.
- Introduce of forebay outlet gates of 3 @ 2.0 m.
- Increase the reservoir outlet gates from 3 @ 2.00 m to 4 @ 2.00 m.
- Increase syphon capacity by adding a syphon similar to that existing.
- Increase ring canal capacity by cleaning regularly.

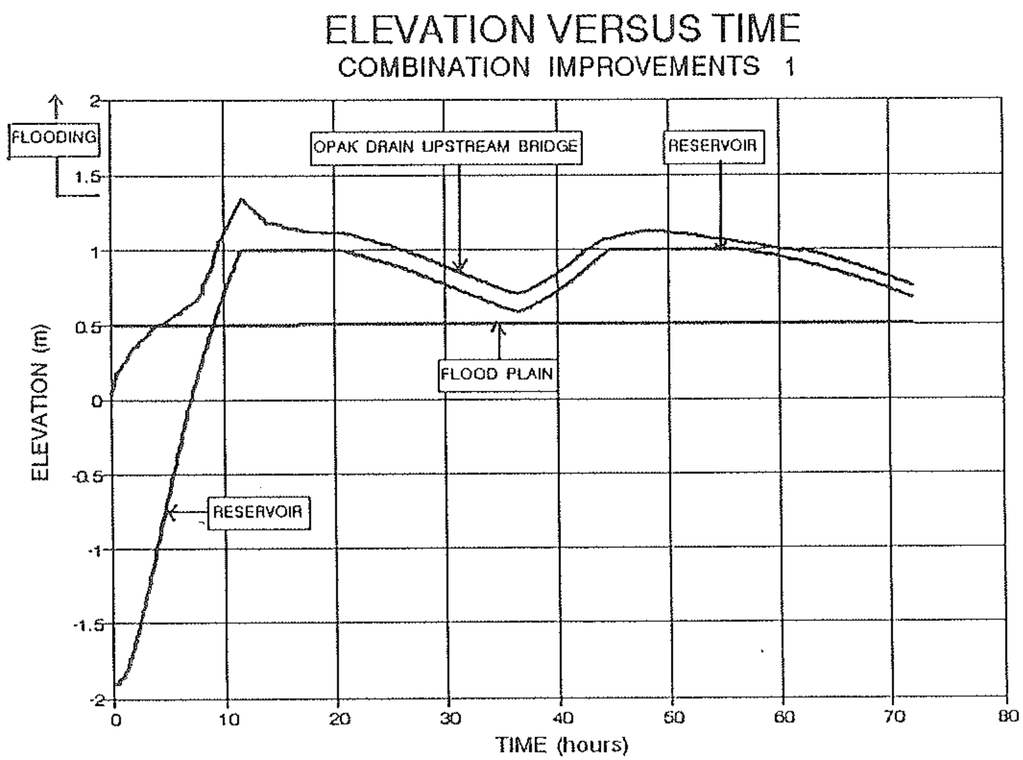
Combination 7 includes all identified alternative improvements of the outlet and inlet systems. In this combination, the alternative of increasing the syphon capacity and the ring canal capacity are also included. Improving the Opak Drain by increasing the bank elevation and by regular cleaning leads to a larger discharge. This means more inflow would pass through the channel. The flow in the channel would have to be diverted through the syphon and or ring canal, therefore a larger capacity of the syphon and or ring canal could be considered. The increase in the syphon capacity is only useful if the outlet system is improved as without such improvements current capacity is sufficient.

5.7.2 ANALYSIS OF THE RESULTS OF THE PROPOSED COMBINATIONS OF IMPROVEMENTS (THE OUTLET AND INLET SYSTEM)

Inflow Hydrograph 1 and Inflow Hydrograph 2 (as used before in the investigation of Outlet Alternatives Modification) were routed through the system for each of the aforementioned combinations of improvement alternatives. From these inflow hydrographs, the operation of the Polder was simulated to determine reservoir elevations and extent of flooding in the flood plain upstream of the Pluit Selatan Bridge. The results of the simulation of the alternatives are shown in Figures 5.48 - 5.61 and are discussed below, while the volume of Flooding and comparison of volume of flood are discussed in Section 6, Evaluation of the Alternatives.

Combination 1: Increasing the Opak Drain Capacity, Increasing the Forebay Pump Capacity

The improvements of the Opak Drain and the pump capacity would be sufficient for Inflow Hydrograph 1 (Figure 5.48). However, the reservoir elevation is still high after the first storm. This condition poses a hazard if another storm occurs. Using Inflow Hydrograph 2, which is the flood Hydrograph for 25 year return period, flooding upstream in the Flood Plain occurs. The forebay pump capacity is too low to pass Inflow Hydrograph 2. As discussed in Sections 5.5 and 5.6 insufficient outflow capacity leads to a high reservoir



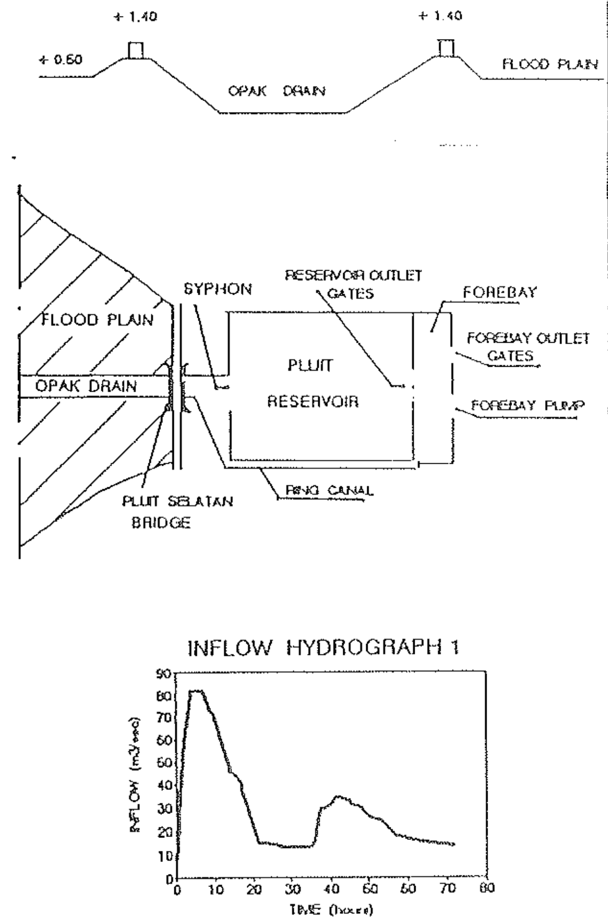
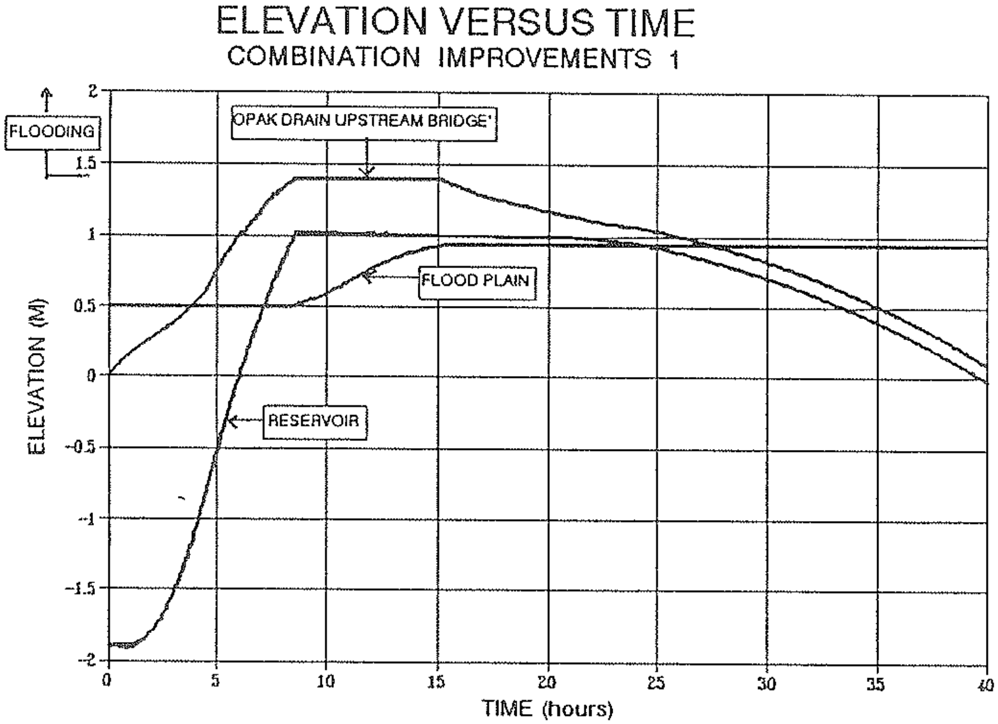
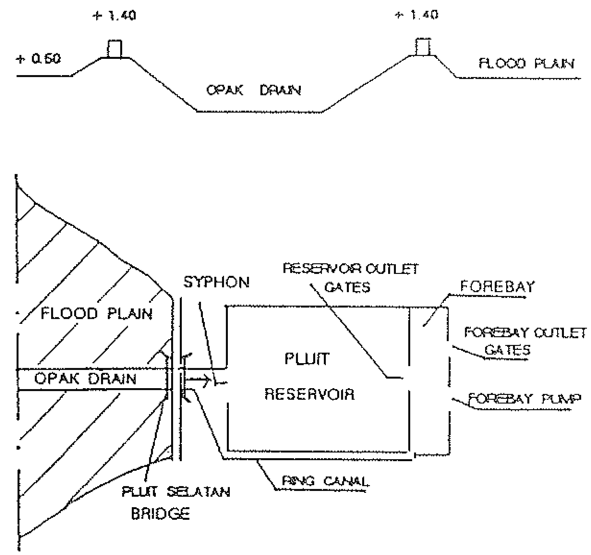


Figure 5.48 Elevation versus Time for Combination Improvement (Combination 1 using Inflow Hydrograph 1)





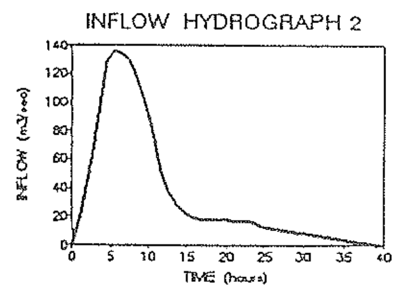
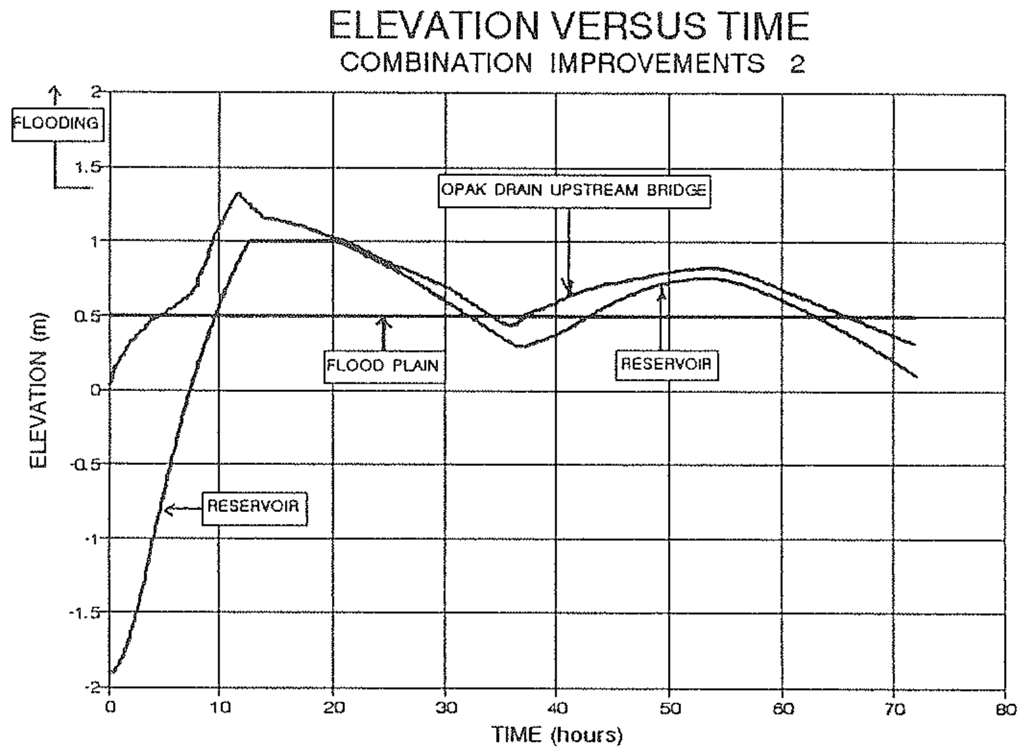
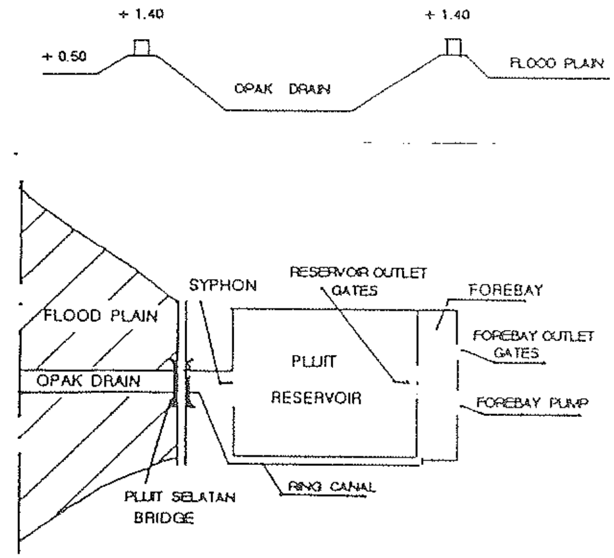


Figure 5.49 Elevation versus Time for Combination Improvement (Combination 1 using Inflow Hydrograph 2)





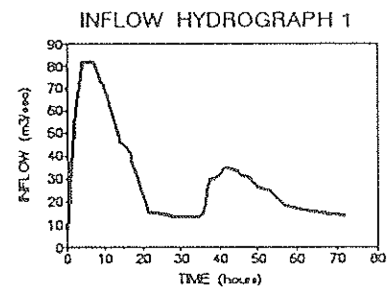
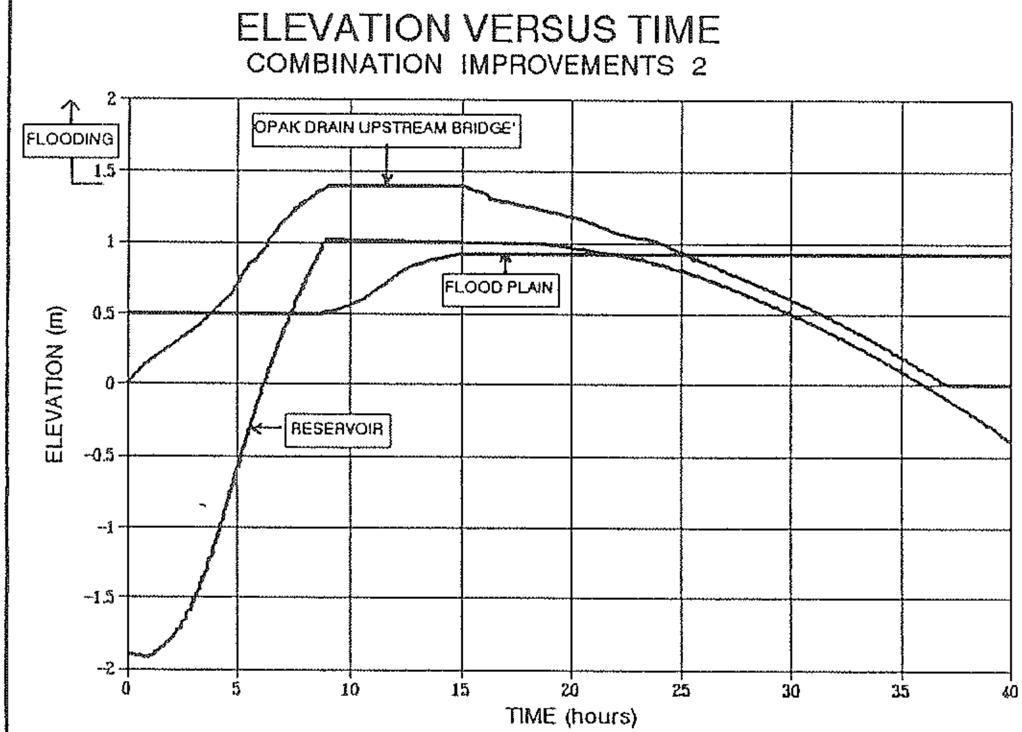
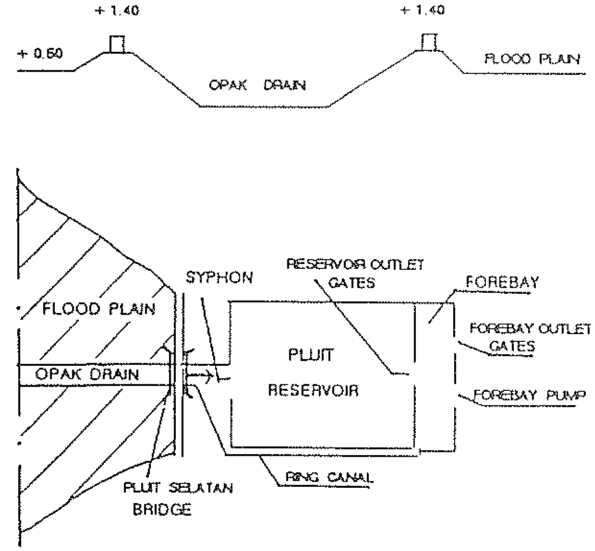


Figure 5.50 Elevation versus Time for Combination Improvement (Combination 2 using Inflow Hydrograph 1)





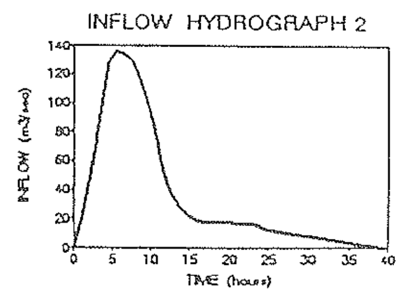


Figure 5.51 Elevation versus Time for Combination Improvement (Combination 2 using Inflow Hydrograph 2)

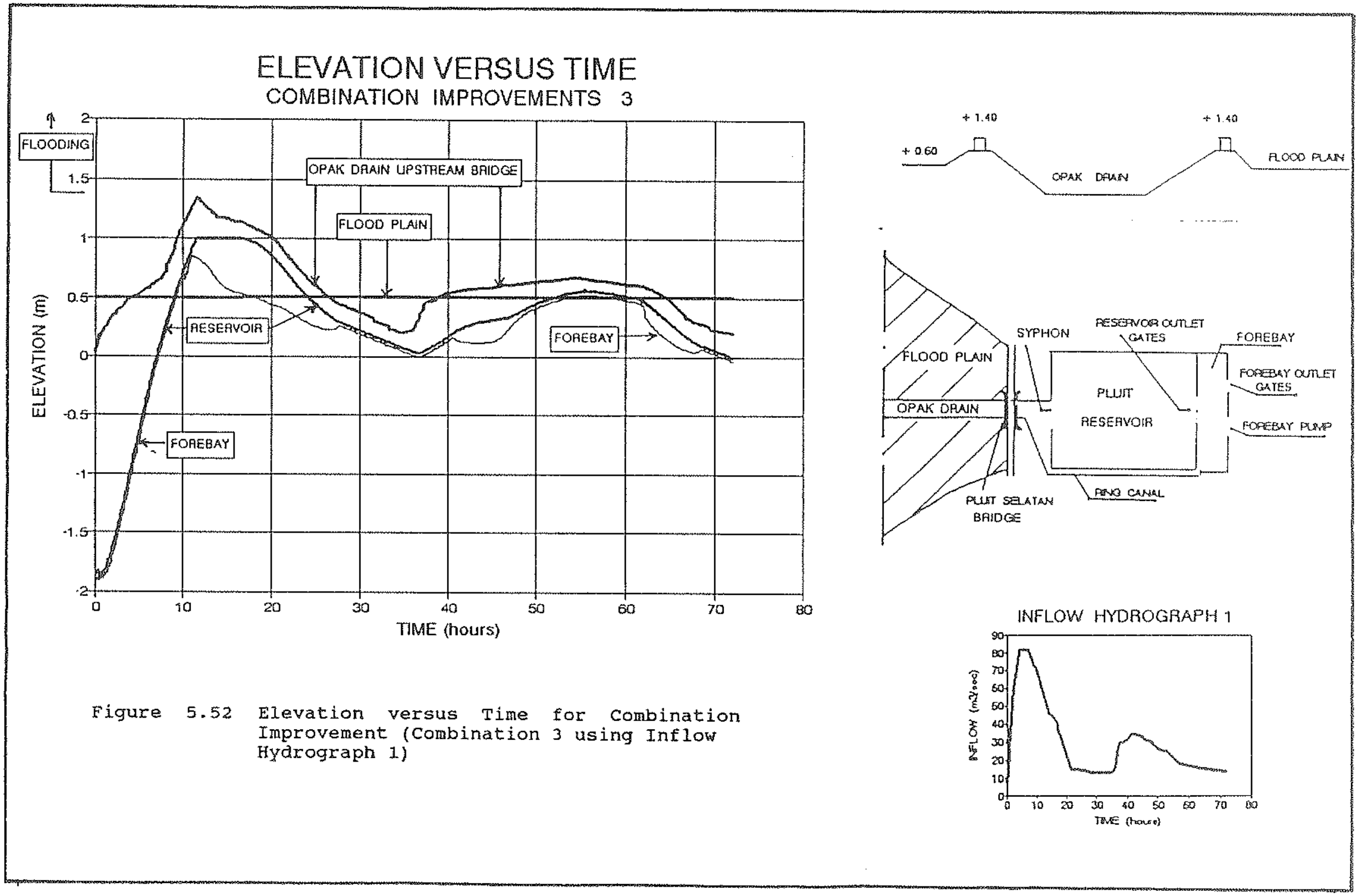
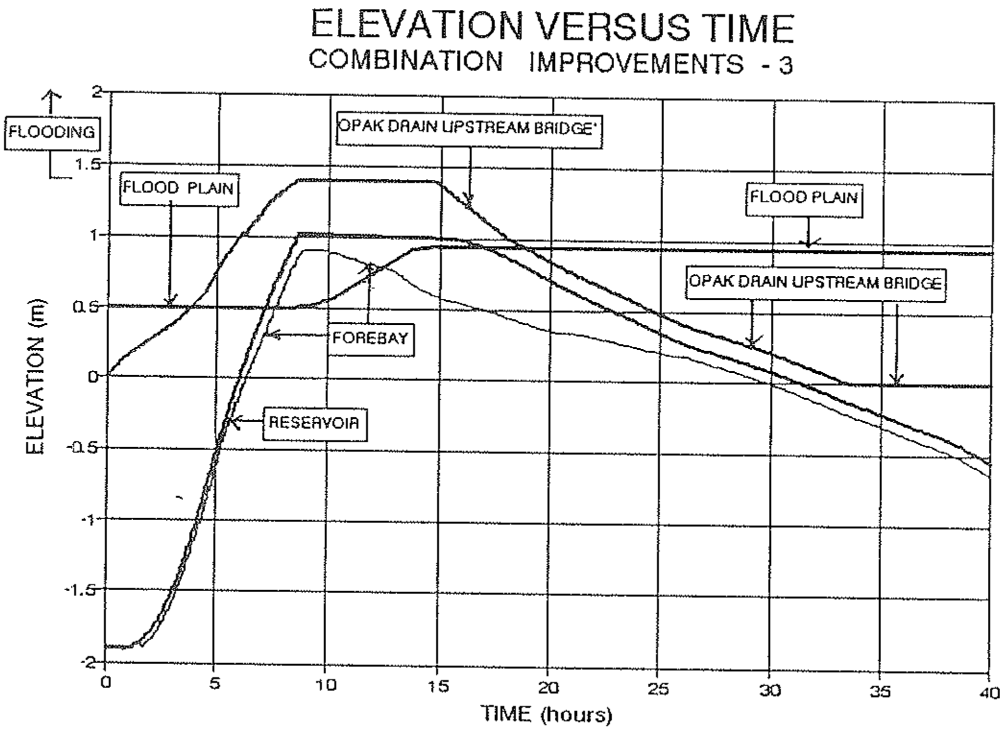
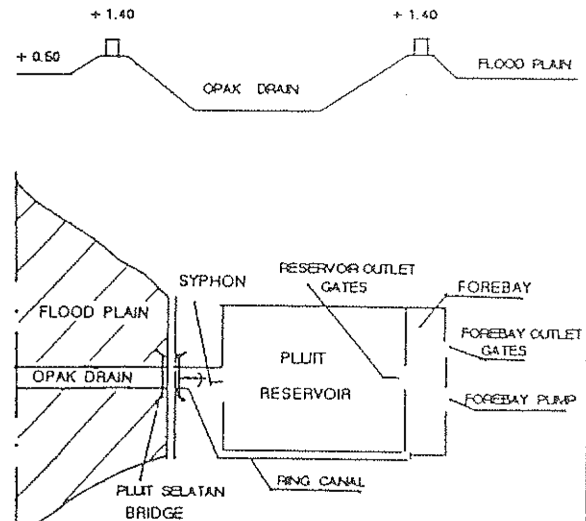


Figure 5.52 Elevation versus Time for Combination Improvement (Combination 3 using Inflow Hydrograph 1)





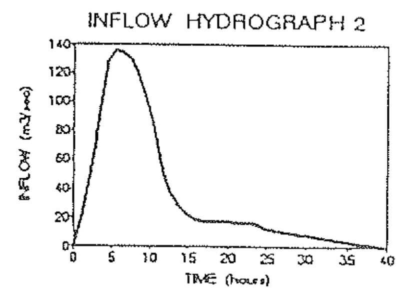
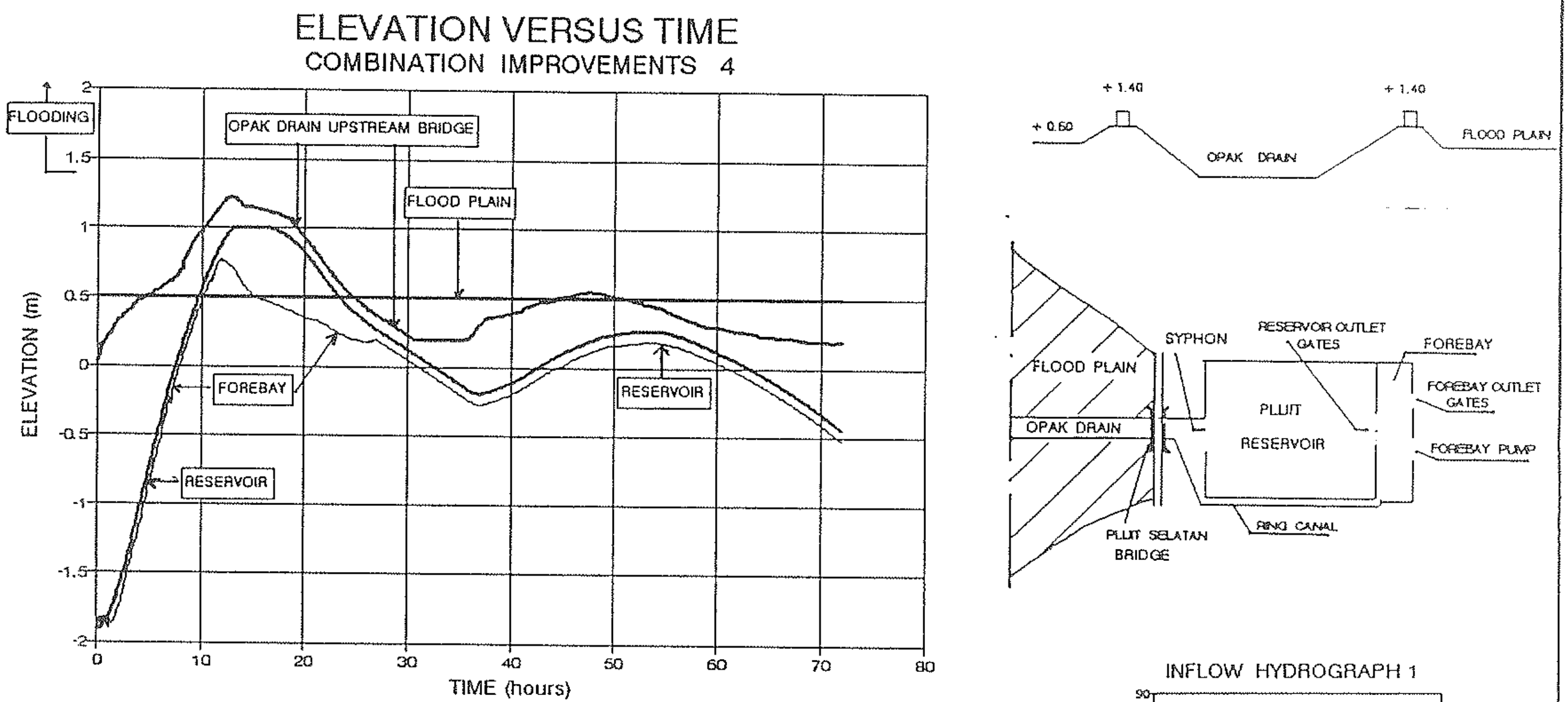


Figure 5.53 Elevation versus Time for Combination Improvement (Combination 3 using Inflow Hydrograph 2)



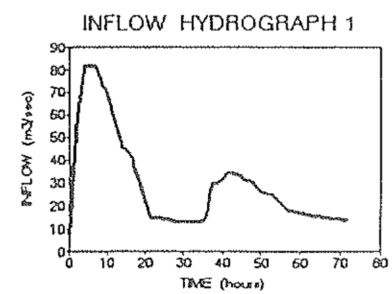
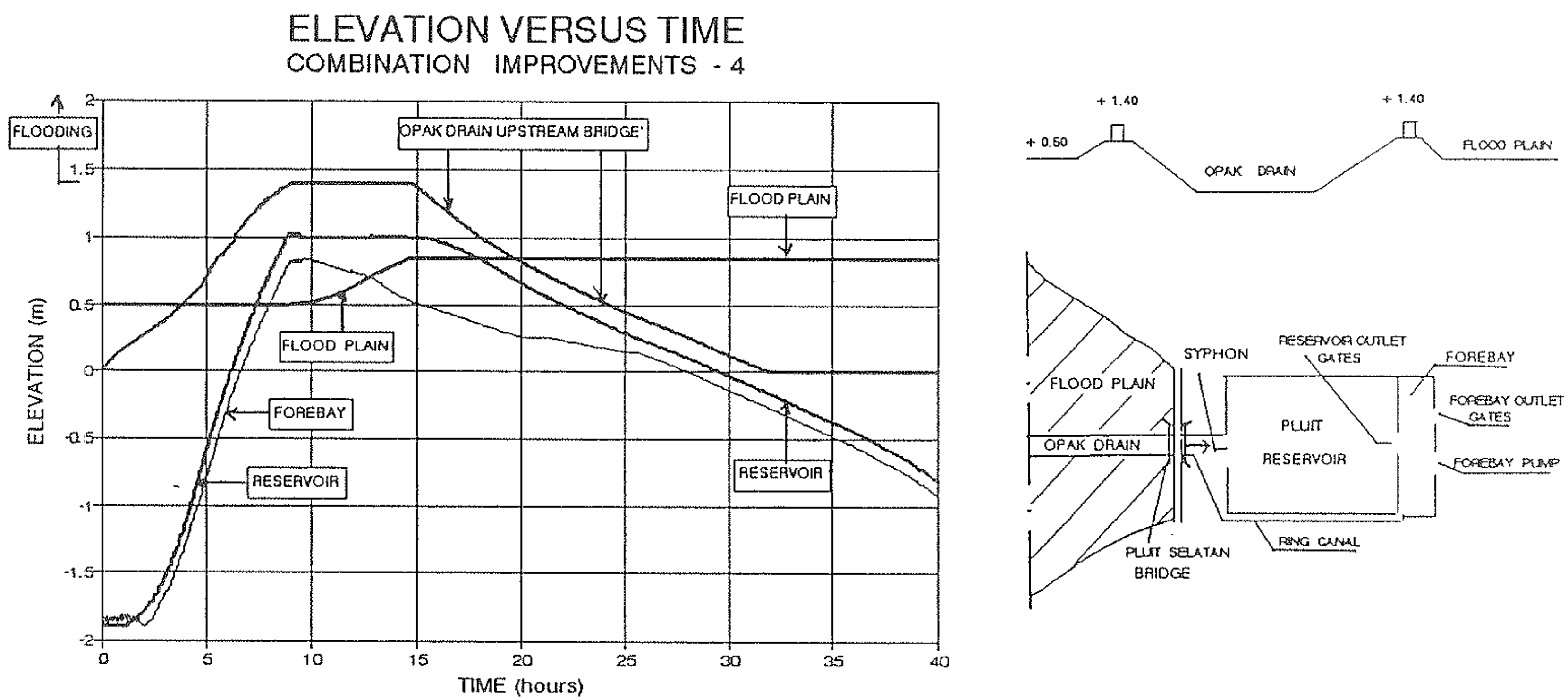


Figure 5.54 Elevation versus Time for Combination Improvement (Combination 4 using Inflow Hydrograph 1)



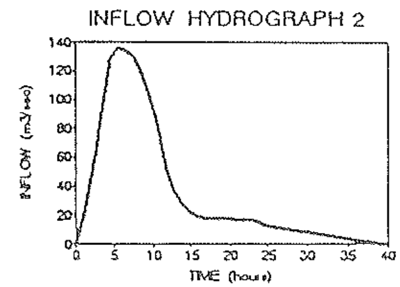
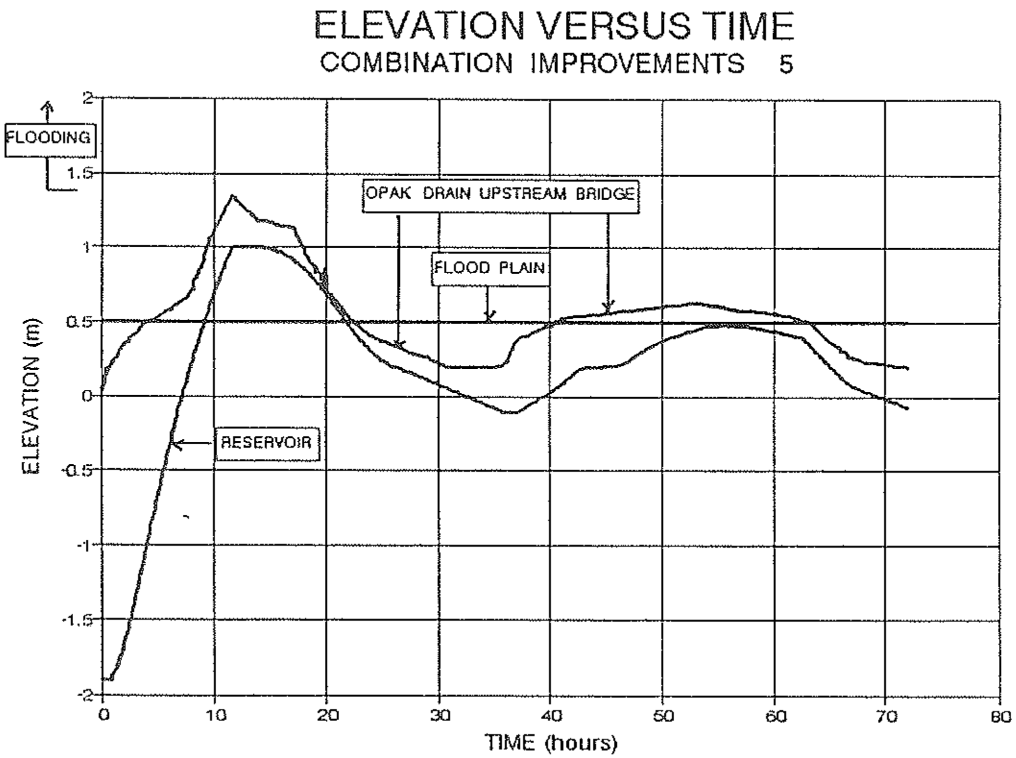
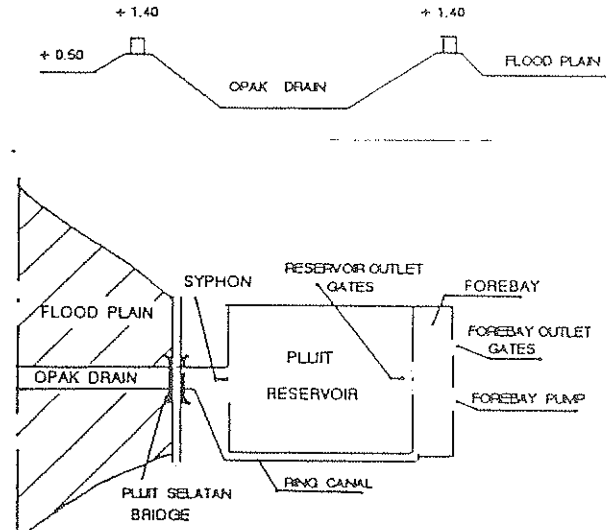


Figure 5.55 Elevation versus Time for Combination Improvement (Combination 4 using Inflow Hydrograph 2)





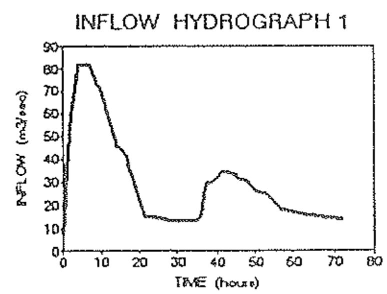
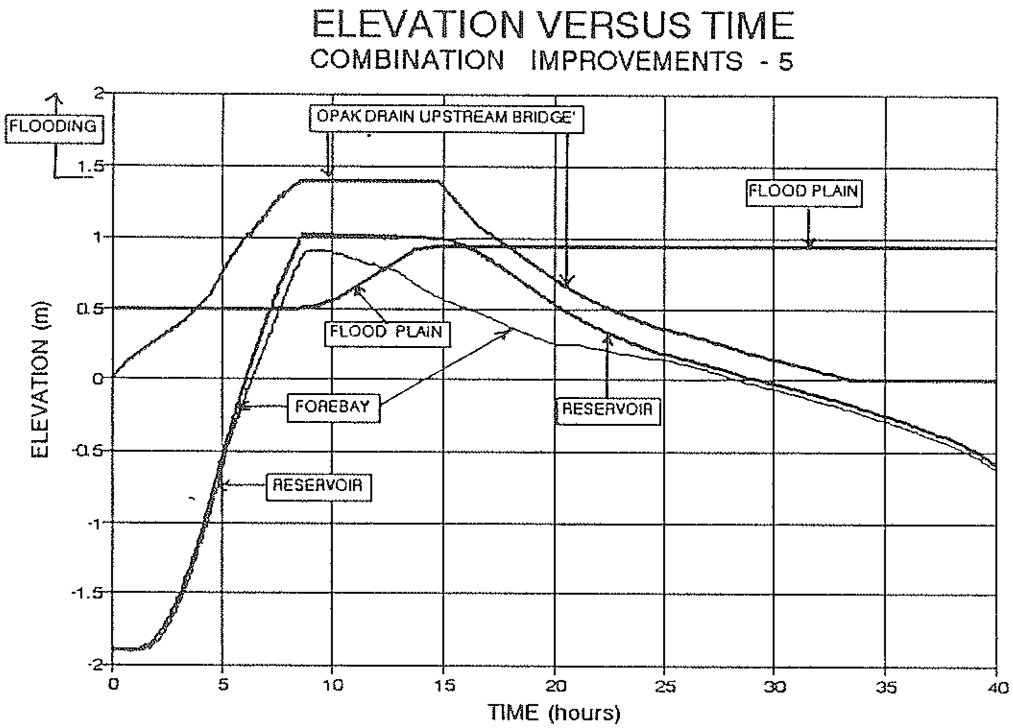
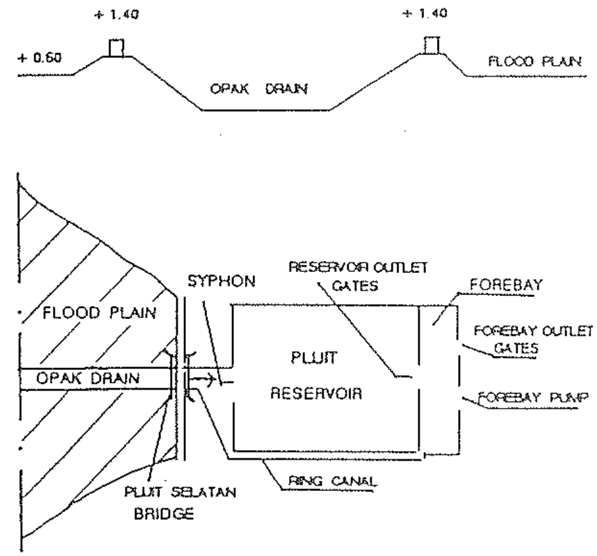


Figure 5.56 Elevation versus Time for Combination Improvement (Combination 5 using Inflow Hydrograph 1)





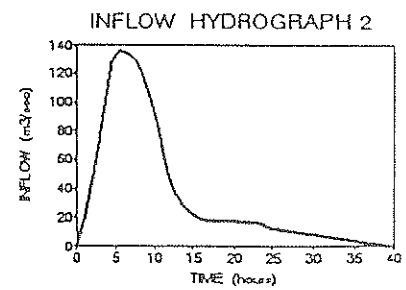
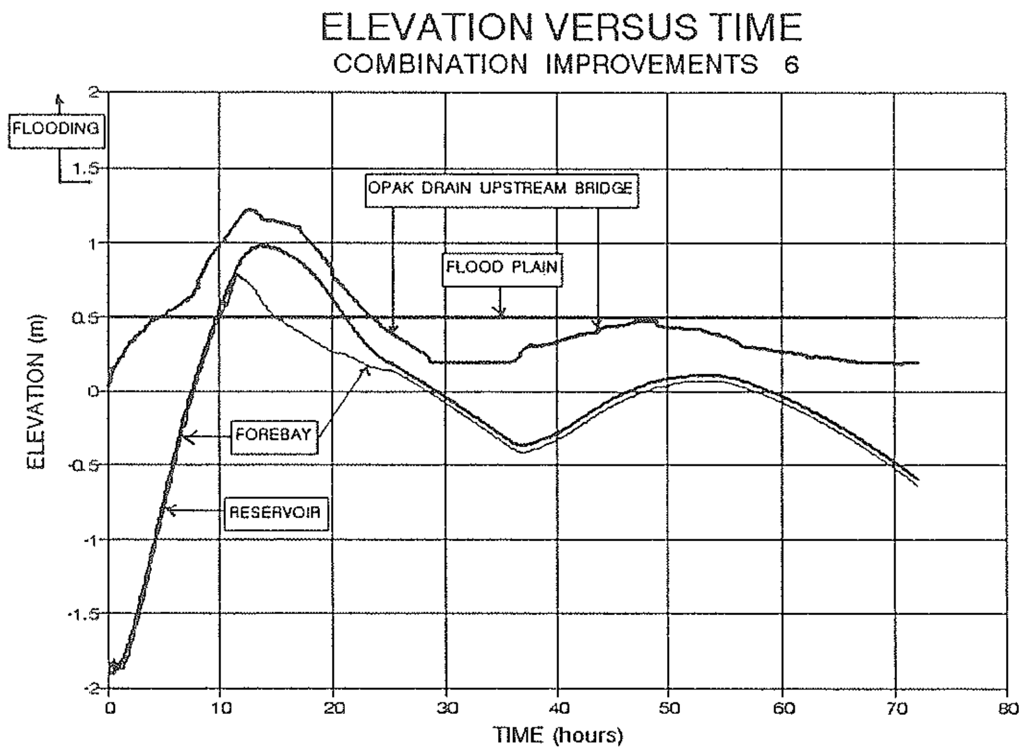
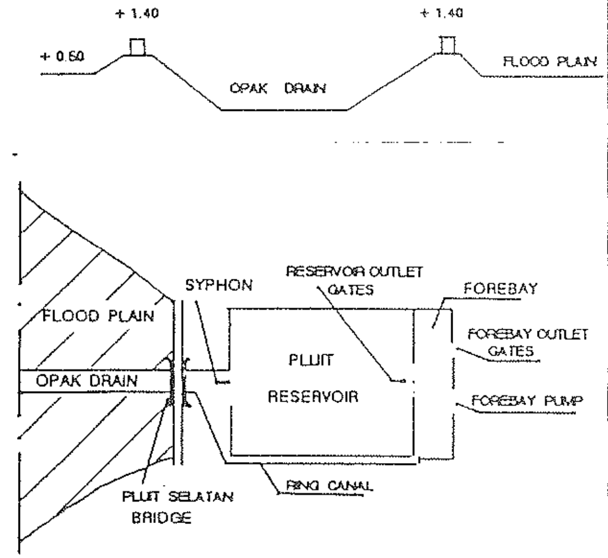


Figure 5.57 Elevation versus Time for Combination Improvement (Combination 5 using Inflow Hydrograph 2)





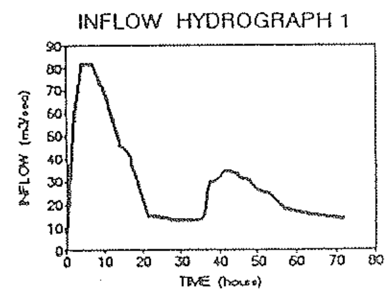
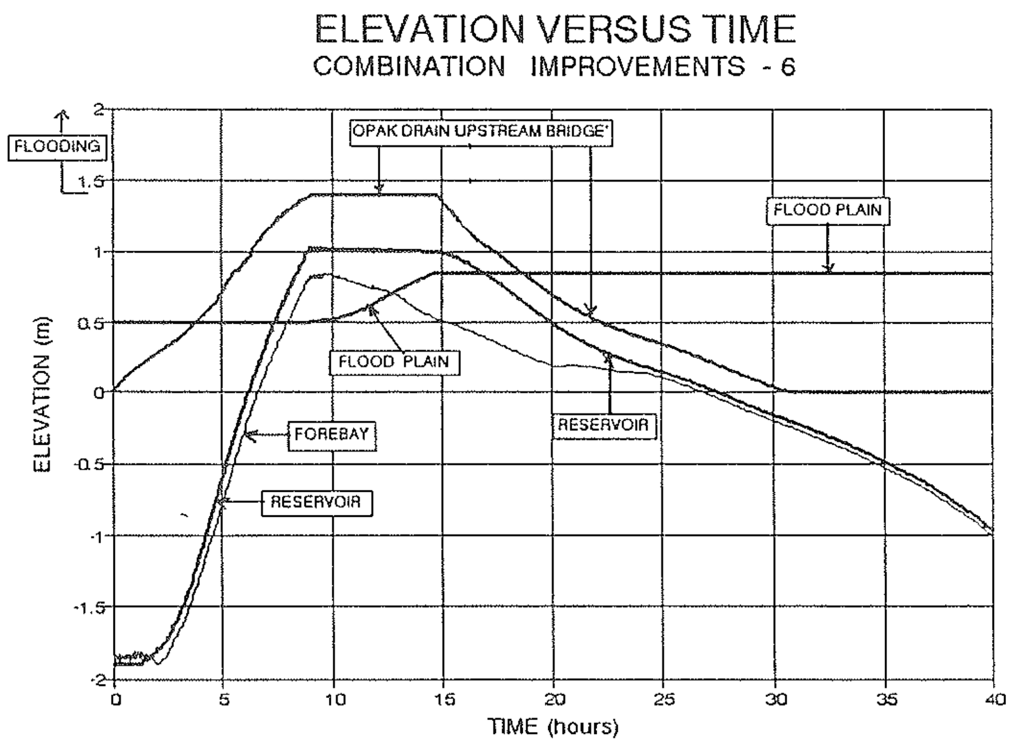
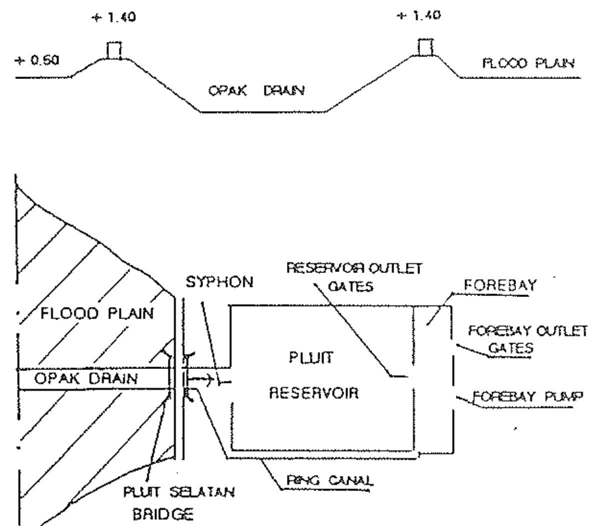


Figure 5.58 Elevation versus Time for Combination Improvement (Combination 6 using Inflow Hydrograph 1)





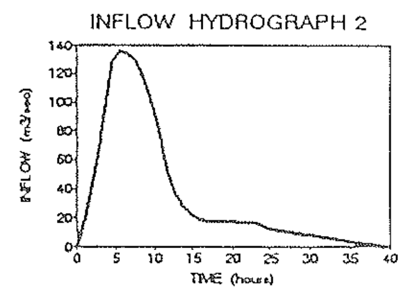
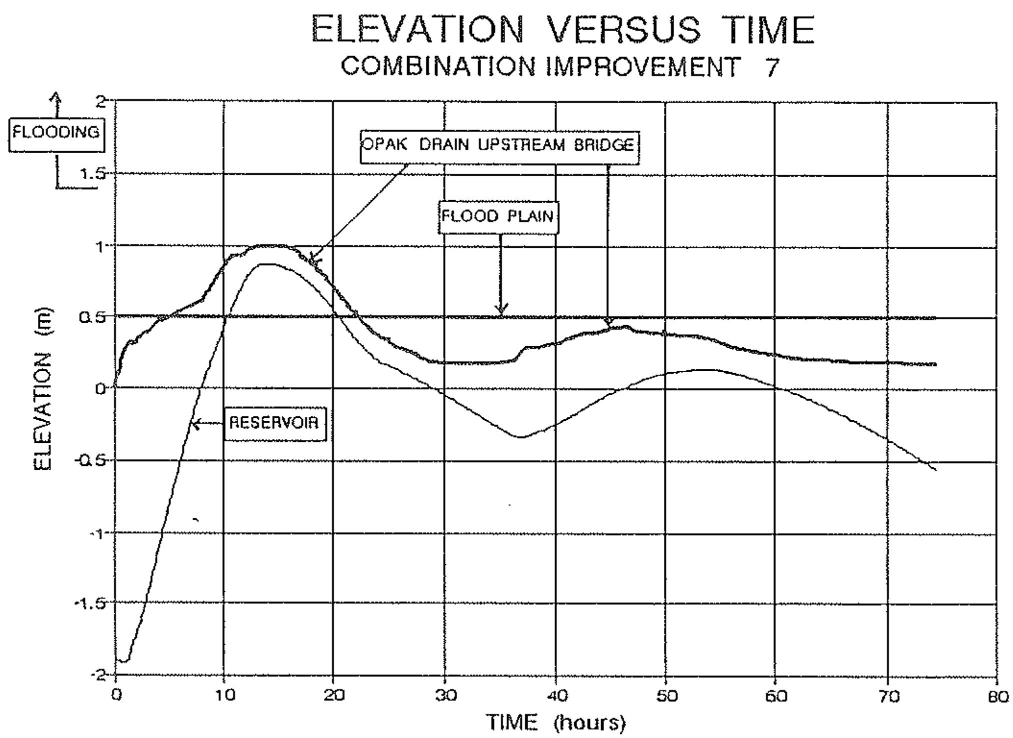
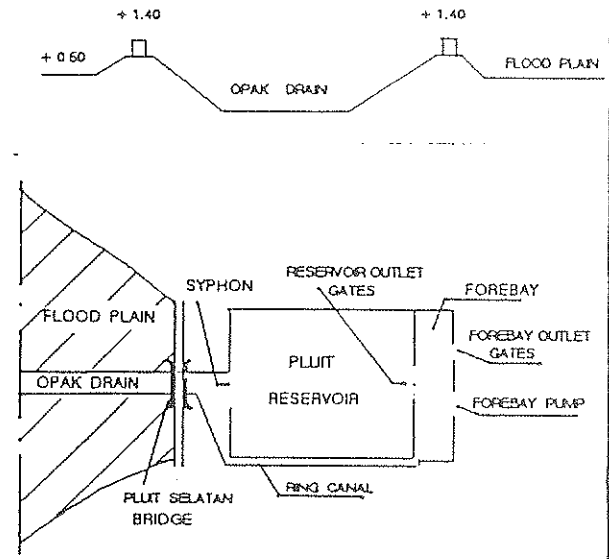


Figure 5.59 Elevation versus Time for Combination Improvement (Combination 6 using Inflow Hydrograph 2)





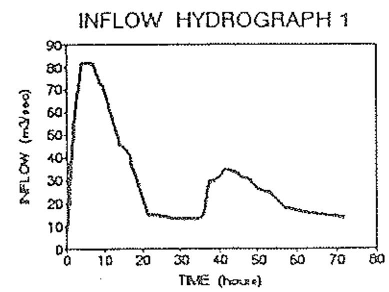
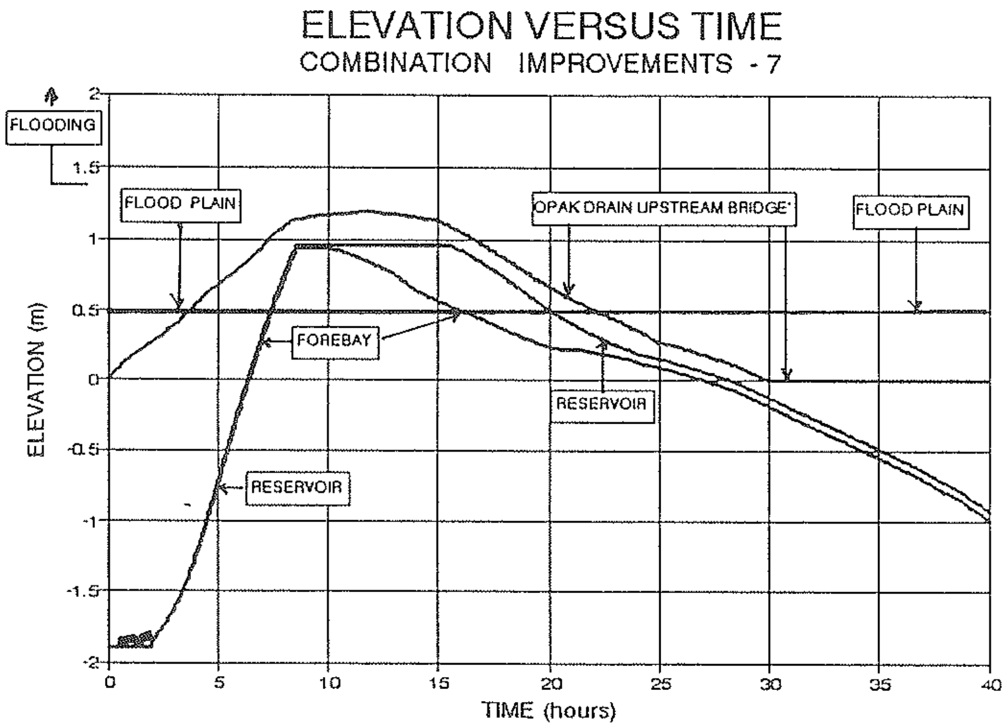
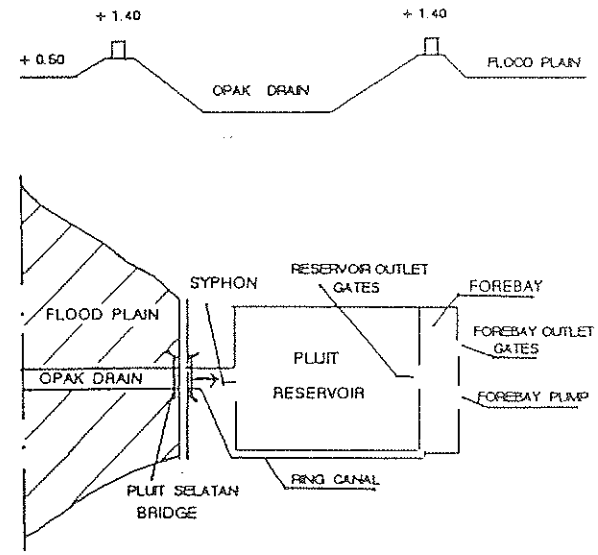


Figure 5.60 Elevation versus Time for Combination Improvement (Combination 7 using Inflow Hydrograph 1)





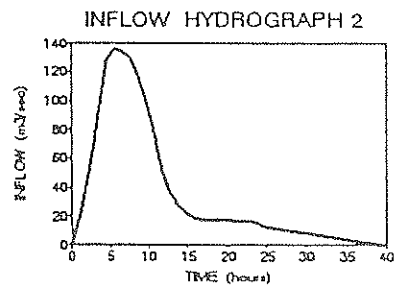


Figure 5.61 Elevation versus Time for Combination Improvement (Combination 7 using Inflow Hydrograph 2)

elevation which in turn results in an increase in Opak Drain water elevation and flooding in the flood plain. Therefore additional outlet improvements are needed to reduce flooding in the reservoir elevation more quickly.

Combination 2: Increasing the Opak Drain Capacity, Increasing the Forebay Pump Capacity

The result in Figure 5.50 shows that using Combination 2, and considering Inflow Hydrograph 1 would overcome the flooding problem in the flood plain. The increase in water level in the Opak Drain due to Inflow Hydrograph 1, and increasing the Opak bank elevation, increases the water level in the Reservoir. Since the forebay pump capacity also increased, more water from the reservoir could be diverted to the forebay and pumped to the sea. Therefore the increase in the water level in the reservoir could be reduced. The reservoir elevation at the end of the first storm is low enough that sufficient storage capacity remains for the flows resulting from the second storm. Using Inflow Hydrograph 2 (Figure 5.51), flooding would still occur. The volume of flooding in this improvement is 488,000 m^3 compared to 1,179,000 m^3 (current condition). Reduction in reservoir elevation also occurs at the end of the second flood, which means a larger available reservoir volume is available if another storm occurs.

Combination 3: Increase the Opak Drain Capacity, increase the Forebay Pump Capacity and Introduce Automatic Forebay Outlet Gates

Introducing Automatic Forebay Outlet Gates of 2 @ 2 m in addition to increasing Opak Drain capacity and increasing forebay pump capacity to 20 m^3/s would result in a lower reservoir elevation.

Using Inflow Hydrograph 1 (Figure 5.52), this improvement would not change the condition during the peak from the first storm in comparison to Combination 1 because the tidal elevations are high during the peak flow from first storm. Therefore no flow through the forebay gates occur during the peak flows from the first storm. The outlet capacity only depends on the forebay pump capacity. It does, however, change the results for the second storm as more water could be diverted via the forebay outlet gates after the peak of the first storm. Using this combination, the flooding upstream could be overcome and the reservoir is ready in the event another storm should occur shortly after.

Using Inflow Hydrograph 2, flooding upstream is reduced to 507,000 m^3. With this improvement, additional excess water due to insufficient outlet capacity occurs. The water level in the Opak Drain increases and flooding occurs. The outlet capacity during the peak flow from this storm depends only on the forebay pump capacity, as the tidal elevations are higher than the water elevation in the forebay. As a result, no water flows through the forebay outlet gates. After the peak flood passes, the tidal elevation is lower than the water elevation in the forebay and flow through the automatic forebay gates occurs. Therefore more water from the reservoir could be diverted to the forebay and the reservoir elevation would be lowered.

Combination 4: Increase Opak Drain Capacity, Increase the Forebay Pump Capacity and Introduce Automatic Forebay Outlet Gates

Introducing automatic forebay outlet gates of 2 @ 2 m in addition to increasing Opak Drain capacity and increasing forebay pump capacity to 24 m^3/s would result in a lower reservoir elevation.

As in Combination 3, using Inflow Hydrograph 1, no additional outlet capacity due to introducing forebay outlet gates occurs in the first peak storm. The outlet capacity depends on the pump capacity. Since the pump capacity in this combination (24 m^3/s) is larger than in Combination 3 (20 m^3/s), the reduction in total volume of flood and the reduction in reservoir elevation is larger.

Using Inflow Hydrograph 2, flooding occurs during the peak storm. As discussed in Combination 2, this is because the tidal elevation is high during peak runoff from the storm. Therefore no flow through the forebay outlet gates occurs, and the outlet capacity depends only on the forebay pump capacity (24 m^3/s), which is insufficient for Inflow Hydrograph 2. However, the reduction in reservoir elevation using this Combination is larger than using Combination 2. This leads to a larger reservoir volume being available if another storm occurs.

Combination 5: Combination of Increased Opak Drain capacity, Increased Forebay Pump Capacity, Introducing Automatic Forebay Outlet Gates and Increased Reservoir Outlet Gates.

The results in Figures 5.56 and 5.57 show that Combination 5 gives better results compared to Combination 3 because more water could be diverted to the sea. The reservoir outlet gate capacity is larger,which increases the water level in the forebay, and increases the gross head at the forebay automatic gates. As a result, the forebay outlet gate capacity is increased. Using Inflow Hydrograph 1, the larger outlet capacity reduces both the water elevation in the reservoir and water elevation in the Opak Drain. Therefore more reservoir volume is available for the second storm. Using Inflow Hydrograph 2, the larger outlet capacity affects both the water elevation in the Opak Drain and the upstream flooding. The increase in the reservoir outlet gates results in a reduced reservoir elevation, which can increase flow through the syphon. The increased capacity of the syphon will reduce the total flooding upstream of the Pluit Selatan Bridge.

Combination 6: Combination of Increased Opak Drain capacity, Increased Forebay Pump Capacity, Introduce Automatic Forebay Outlet Gates and Increased Reservoir Outlet Gates.

The results in Figures 5.58 and 5.59 show that Combination 6 gives better results compared to Combination 5 because the forebay pump capacity (24 m^3/s) is larger than in Combination 5 (20 m^3/s), and therefore more water could be diverted to the sea. Using Inflow Hydrograph 1, no flooding occurs and, compared to Combination 5, the available reservoir volume is larger for the next successive storm. Using Inflow Hydrograph 2, results in a reduction in the total volume of flooding. The total volume of flooding is now 482,000 m^3. The available reservoir volume after the storm also increases in the event another storm occurs shortly after.

Combination 7: Increase Syphon Capacity by Adding 1 Additional Syphon of 4.0 m and Increase Ring Canal Capacity by Cleaning Regularly <u>in Addition</u> to Combination 6.

This combination <u>includes all</u> improvement alternatives of the outlet and inlet systems. The results from Combination 6 show that the difference in the elevation downstream of the Pluit Selatan Bridge (Point C, refer to Figure 2.2) and the reservoir is large. Therefore more flow could be diverted to the reservoir via the syphon. This increase in flow to the reservoir could be done by increasing the syphon capacity, in this case the addition of a syphon similar to that existing. Increasing the Ring Canal capacity by regularly cleaning would support additional flow from Opak Drain to the sea. Using this combination, a better result could be gained compared to Combinations 1 to 6. This combination is able to handle Inflow Hydrograph 2 or a combination of Inflow Hydrograph 2 and another storm occuring directly after. The Combination of Inflow Hydrograph 2 and another storm will be discussed in the Section 7, Sensitivity Analysis.

The results (Figures 5.60 and 5.61) show that Combination 7 could account for Inflow Hydrograph 1 as well as Inflow Hydrograph 2. Using Combination 7 for

Inflow Hydrograph 1 leads to a reduction in proposed dyke elevation along the Opak Drain. The required change in dyke height in the Opak Drain could be reduced to EL.+1.20 m rather than + El. 1.40 m which includes some freeboard, which is necessary to account for wave action, settlement etc. Using Inflow Hydrograph 2, the water elevations in the Opak Drain are still high although no overflow occurs. However, an increase in Opak Drain to +1.40 m should be considered for the peak storm of Inflow Hydrograph 2. For both inflow hydrographs, the reservoir elevation at the end of the storm is low, which gives a large available reservoir capacity.

CHAPTER 6

EVALUATION OF THE ALTERNATIVES

6.1 GENERAL

For the following discussion:

- the "flood" refers to flooding in the Flood Plain upstream the Pluit Selatan Bridge. Refering to Figure 5.5, there are two conditions of flooding in the Flood Plain:

  Flood Condition I: Flooding in the Flood Plain below the Opak bank level.

  The maximum volume of Flood I (if flooding in the flood plain reachesan elevation of +1.00 m) is 577,000 m^3. The flood plain will return back to "dry" condition by normal drainage paths.

  Flood Condition II: Flooding in the flood plain above the Opak bank level (above El. +1.00 m).

  The volume of flooding in the flood plain, rather than flood stage, has been chosen to evaluate effectiveness of improvement alternatives. This choice was made because no topographical data for the Flood Plain area are available. In this study, the flood plain area is assumed as a flat area. Using the assumption that the flood plain area is flat, the flood stage for each modification alternative is shown in Figures 5.23 to 5.61. Once topographical data are available, the flood stage together with volume of flooding in the flood plain can be used to assess the extent of flooding.

- The reservoir elevation refers to the elevation at the end of the second storm (T = 72 hours) using Inflow Hydrograph 1, or at the end of the single storm (T= 37.9 hours) using Inflow Hydrograph 2. In this study calculations were not taken beyond a time = 72 hours.

As a basis for comparison for flood reduction due to modifications, the following are the current flood conditions (without improvements) in the flood plain upstream of the Pluit Selatan Bridge:

- Using Inflow Hydrograph 1, and referring to Figure 6.1, flooding in the flood plain begins at T = 8.50 hours and increases to 390,000 m^3 at T = 20.0 hours. This volume of flooding in the flood plain is caused by the first storm. Since the volume of flooding is lower than 577,000 m^3, the condition of flooding is Flood Condition I (Figure 5.5). Between T = 20.0 hours until T = 38.50 hours the volume of flooding remains constant. This condition occurs between the first and second storm. At T = 38.50 hours, flooding increases and reaches the maximum volume of flooding 798,000 m^3 at T = 57.0

hours. Flooding still exists at the end of the second storm (T = 72 hours). The volume of flooding at T = 72 hours is 772,000 m^3. The reservoir elevation at the end of the second storm is +0.90 m (Figure 5.13 at T = 72 hours), which means that the Pluit Reservoir is full. No further investigation was included in this study after the end of the storm(s).

- Using Inflow Hydrograph 2, and referring to Figure 6.3, flooding in the flood plain begins at T = 6.00 hours and reaches the maximum flood volume of 1,179,000 m^3 at T = 13 hours. The condition of flooding in the flood plain is Flood II (Figure 5.5), which is flooding above the Opak Drain bank level. From T = 13 hours to T = 22 hours, flooding above the Opak Drain bank reduces since the inflow also reduces (Inflow Hydrograph 2). From T = 22 hours until T = 27 hours, flooding remains constant. This is the condition where constant inflows occurs. As the inflow reduces, the flood also reduces after T = 27 hours. At T = 34.5 hours, volume of flooding in the flood plain remains constant at 577,000 m^3. This flooding occurs in the flood plain, below the Opak Drain bank level (Flood condition I) and will be reduced by normal drainage paths. The reservoir elevation at T = 37.9

hours (the end of Inflow Hydrograph 2) is + 0.60 (Figure 5.15). From the investigation of possible improvements to the outlet System and inlet System and the combination of the various improvement alternatives, the following evaluations were derived:

6.2 IMPROVEMENT ALTERNATIVES TO THE OUTLET SYSTEM (FIGURE 6.1 TO 6.4)

The results of the simulation of the improvements are shown in Figures 5.23 to 5.42. The following are the evaluations of each alternative modification:

Outlet Alternative 1: Increase Forebay Pump Capacity

Increasing the forebay pumping capacity by itself results in a reduction in flood volume in the flood plain. The amount of reduction depends on the amount of increase in pump capacity. In this study, two alternatives for increasing pump capacity were considered, pump capacity of 20 m^3/s and pump capacity of 24 m^3/s. The resulting volume of flood plain flooding using these alternatives is shown in Figure 6.1, and summarized in Table 6.1.

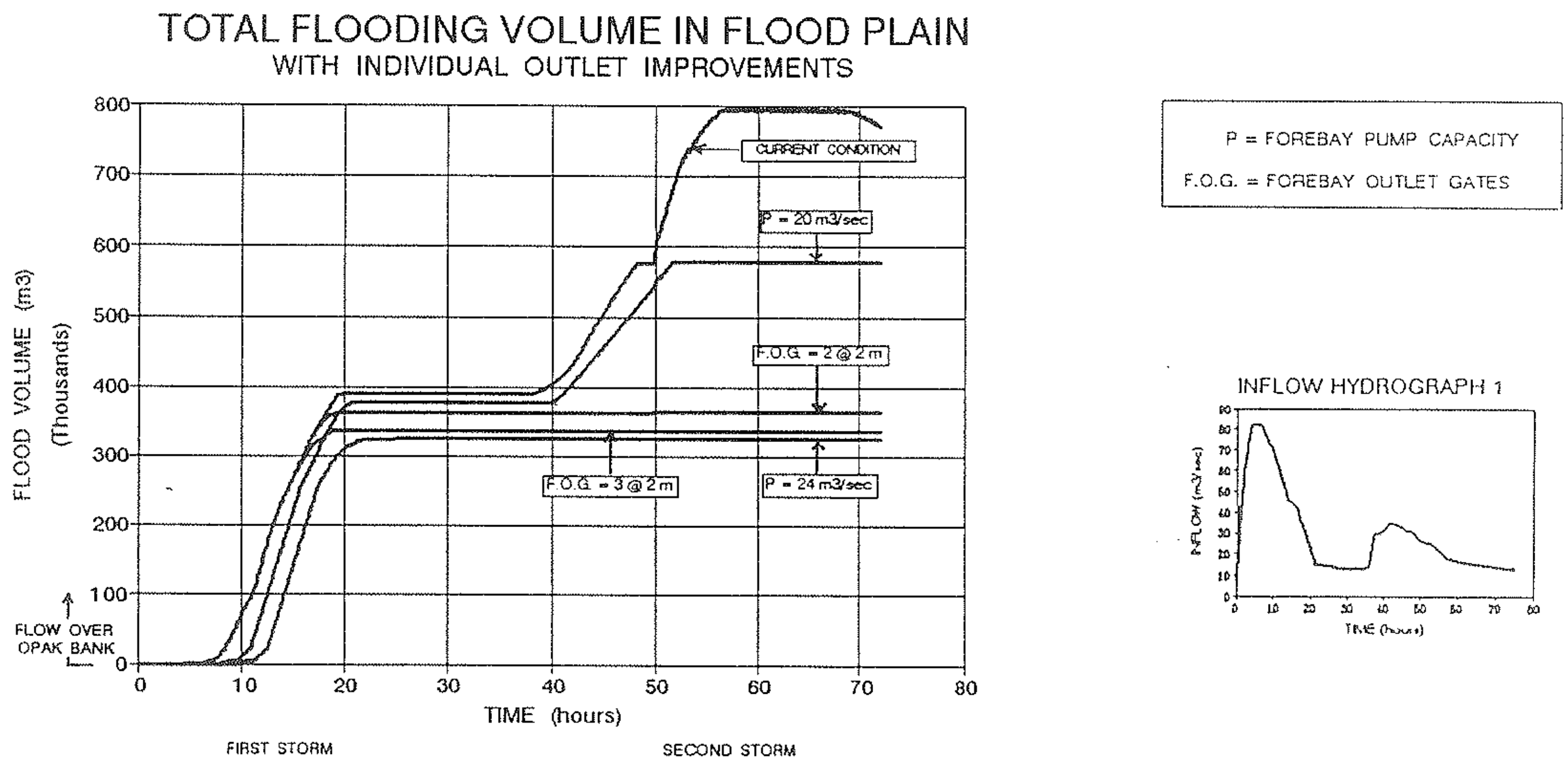


Figure 6.1. Individual Outlet Improvements using Inflow Hydrograph 1

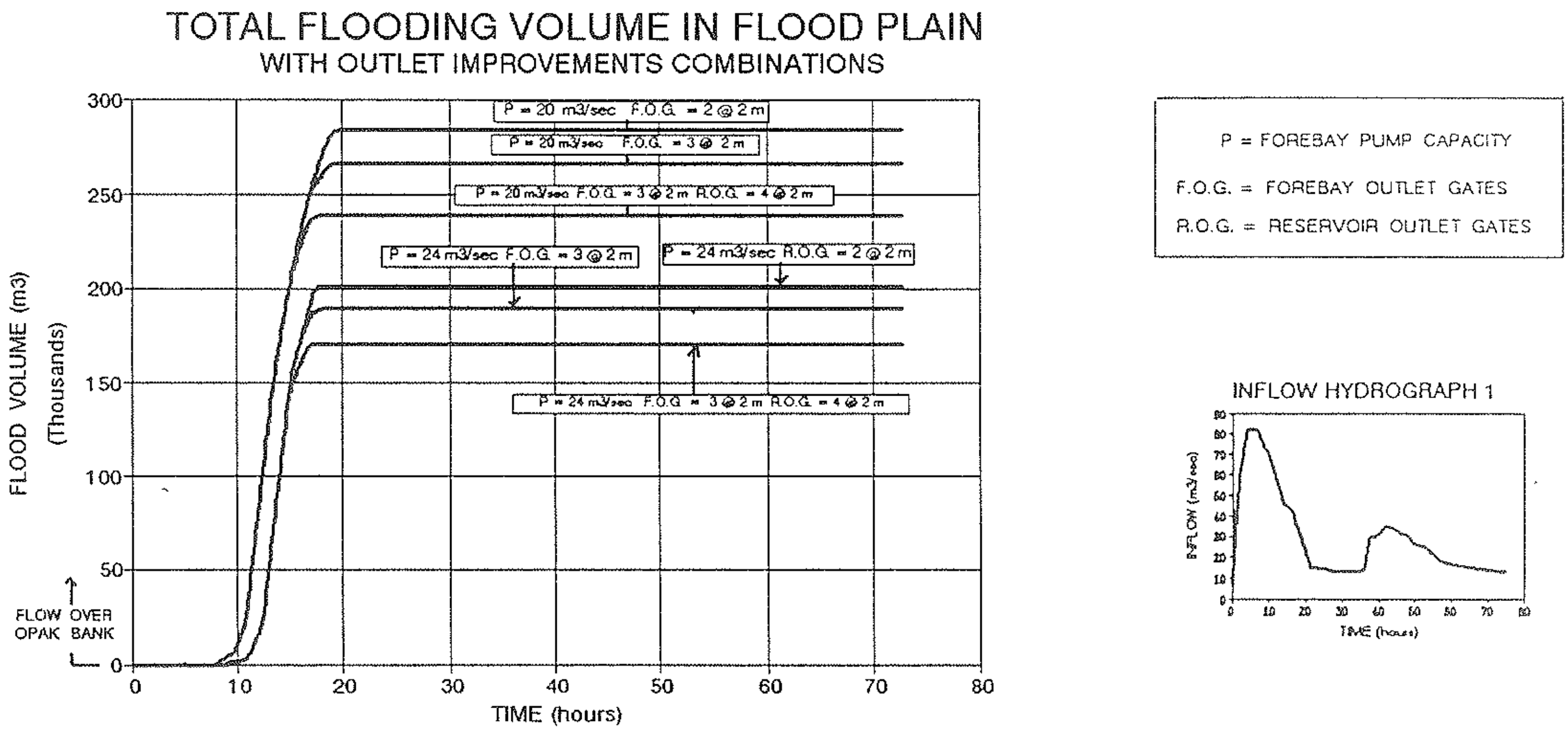


Figure 6.2. Combinations Outlet Improvements using Inflow Hydrograph 1

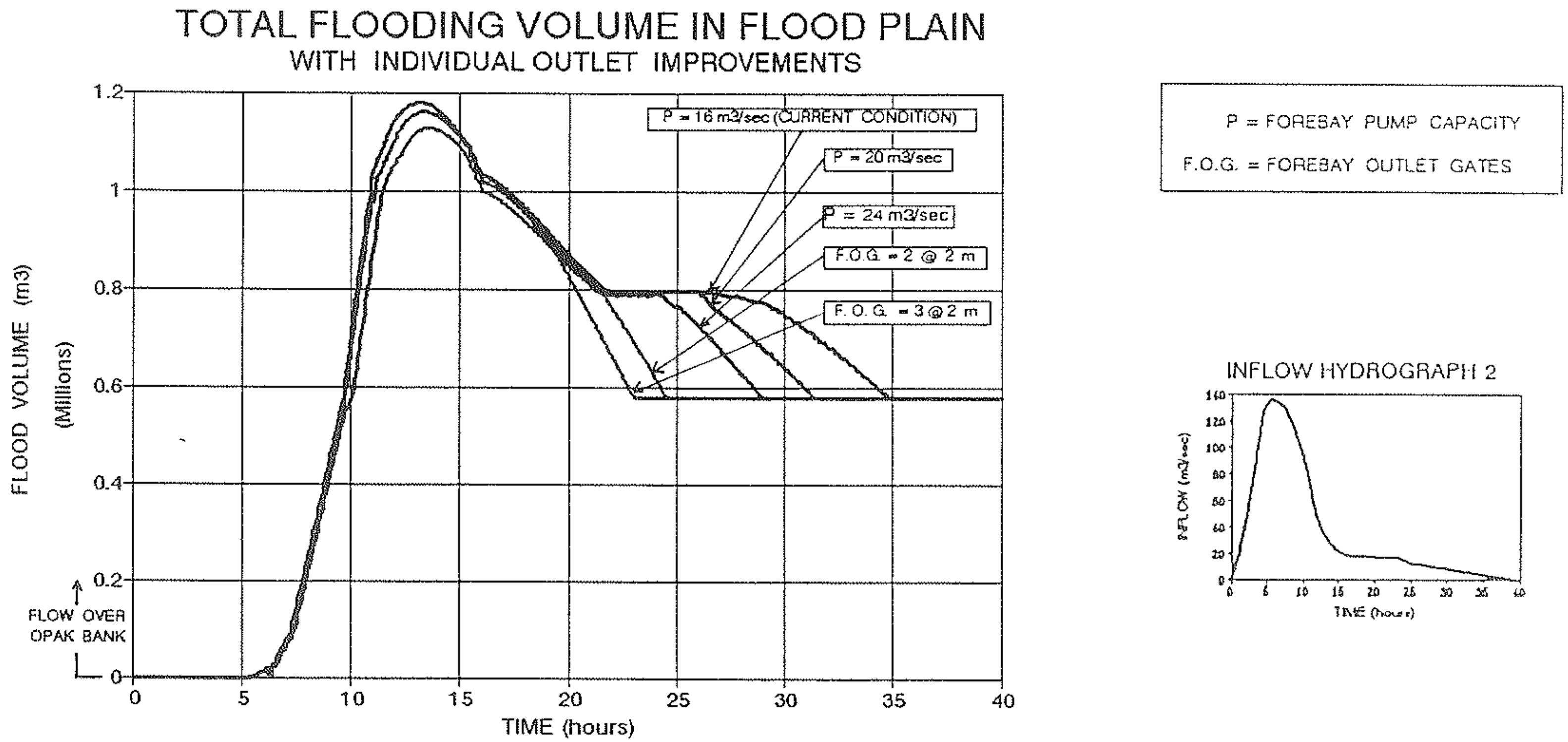


Figure 6.3. Individual Outlet Improvements using
Inflow Hydrograph 2

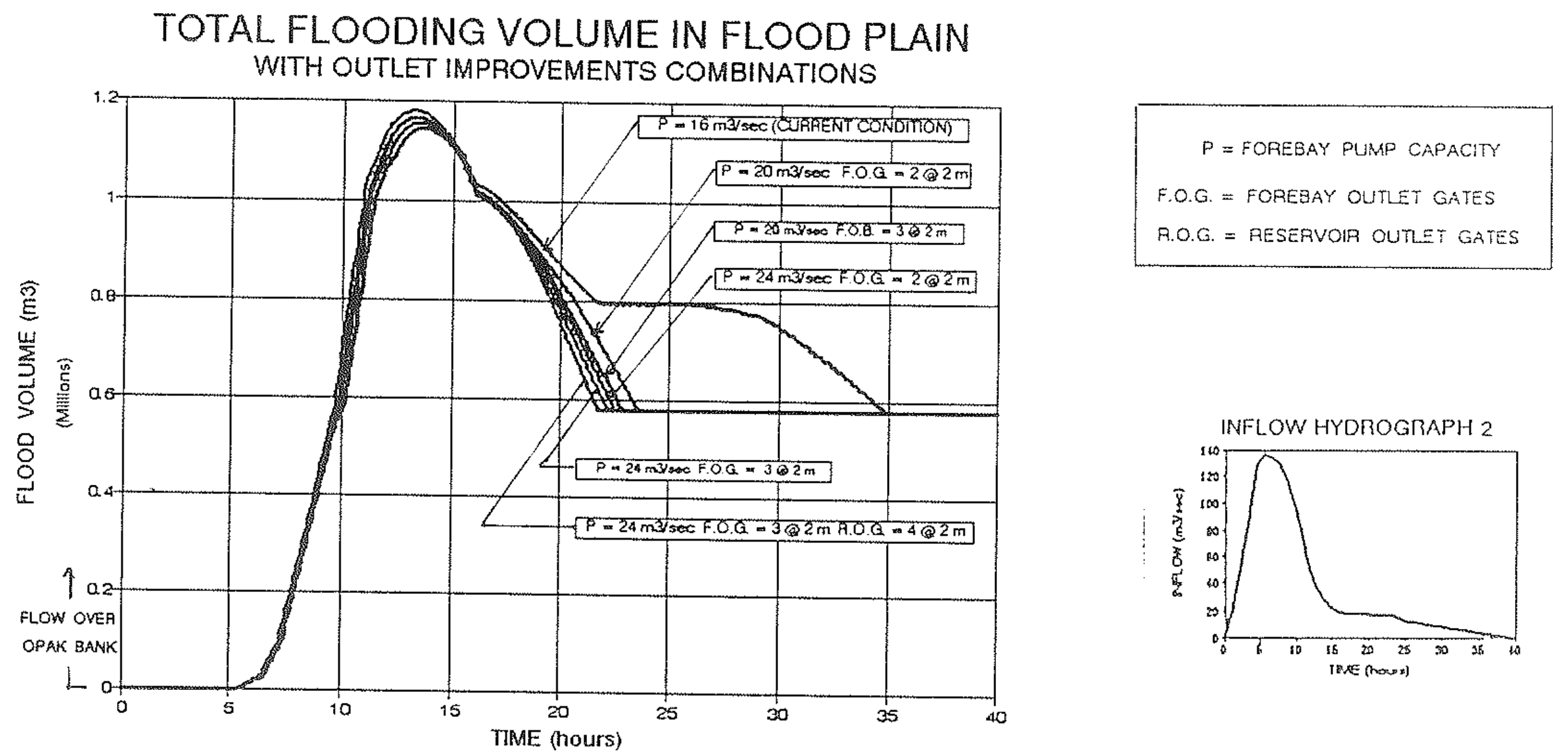


Figure 6.4. Combinations Outlet Improvements using Inflow Hydrograph 2

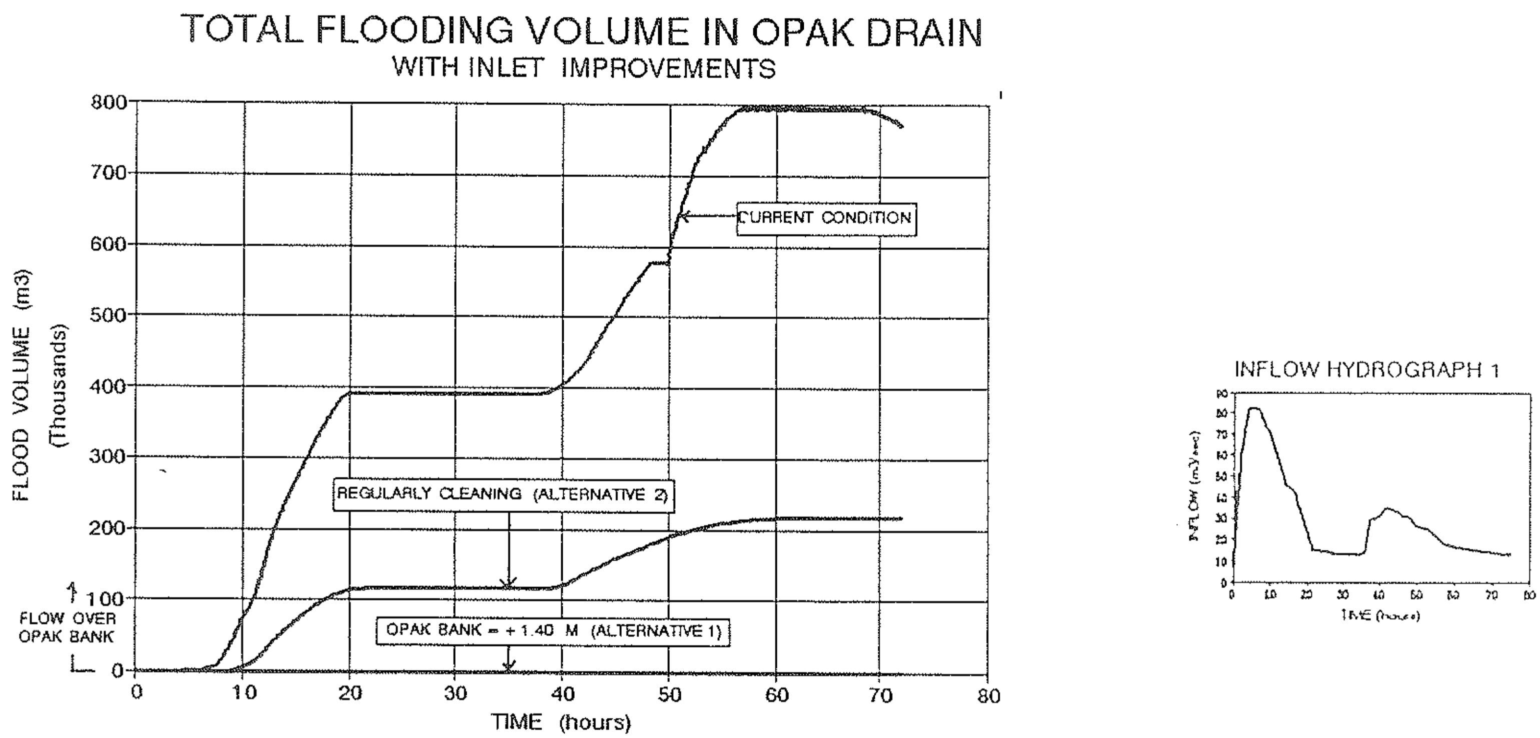
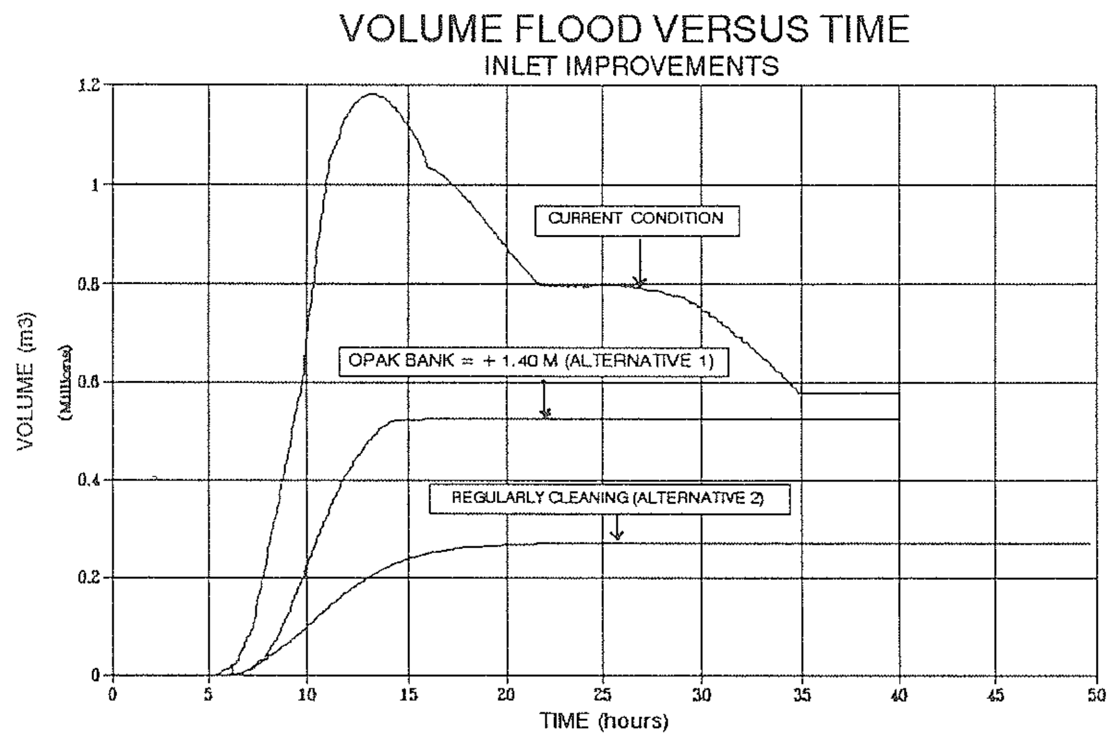


Figure 6.5. Inlet Improvements using Inflow Hydrograph 1



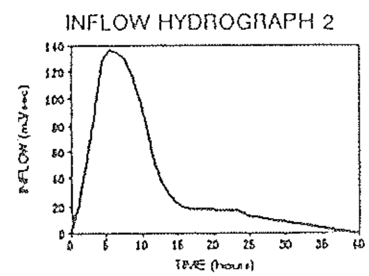
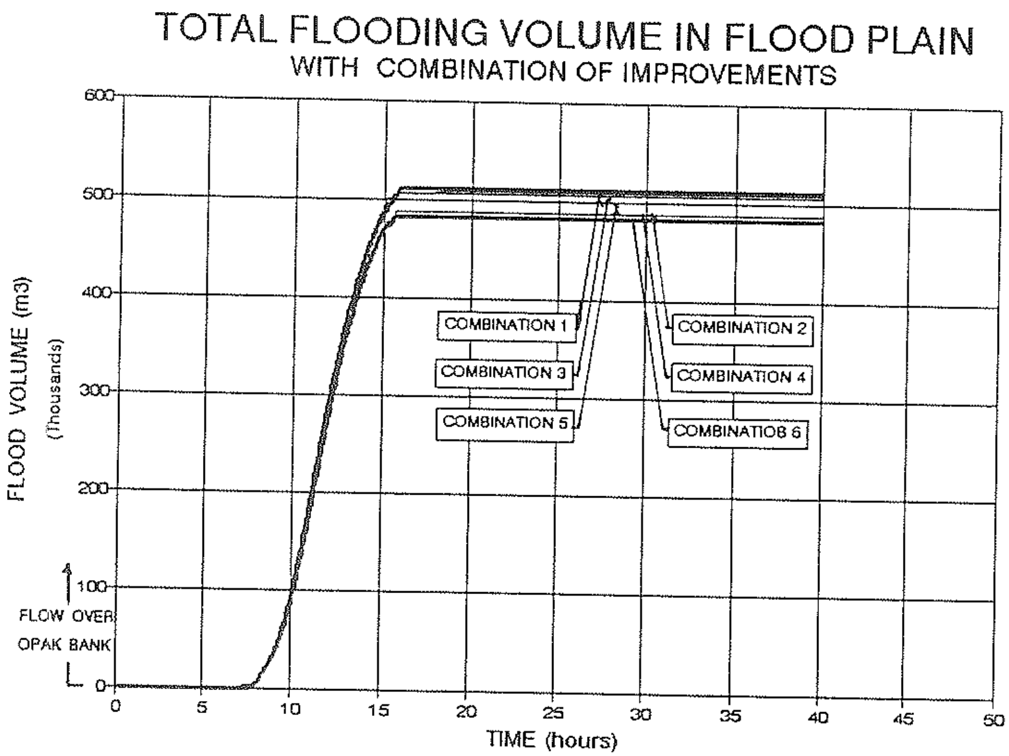


Figure 6.6. Inlet Improvements using Inflow Hydrograph 2



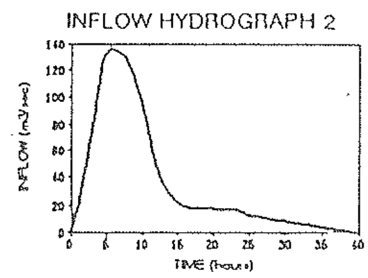


Figure 6.7. Combinations Improvements using
Inflow Hydrograph 2

TABLE 6.1. Volume of Flooding in the Flood Plain using Outlet Improvement Alternative 1

Figure	Flood Hydrograph	Forebay Pump Capacity m^3/s	Volume of flooding in the Flood Plain m^3
6.2	1	16*)	798,000
6.2	1	20	577,000
6.2	1	24	325,000
6.3	2	16*)	1,179,000
6.3	2	20	1,160,000
6.3	2	24	1,141,000

*) Current forebay pump capacity

The increase in pump capacity also reduces the Reservoir elevation, or increase in Reservoir volume available after the storm as listed below:

TABLE 6.2. Pluit Reservoir volume available after the storm(s) using Outlet Improvement Alternative 1

Figure	Flood Hydrograph	Forebay Pump Capacity m^3/s	Reservoir Volume after Storm(s) m^3
5.13	1	16*)	48,000
5.23	1	20	338,000
5.25	1	24	894,000
5.15	2	16*)	297,000
5.24	2	20	420,000
5.26	2	24	661,000

*) Current forebay pump capacity = 16 m3/s

Referring to Figure 6.1, using Flood Hydrograph 1 and considering a forebay pump capacity of 20 m^3/s, the flood in the Flood Plain begins at T = 9.0 hours, and increases to a volume of 377000 m^3 at T = 20 hours. From T = 20 hours until

T = 40 hours, flooding remains constant. This is flooding in the flood plain below the Opak Drain bank (Flood Condition I, Figure 5.5) which occurs between the first storm and the second storm. At T= 40 hours, additional flooding due to the second storm occurs and reaches the maximum volume of flood = 577,000 m^3 at T = 52 hours. Using the same Inflow Hydrograph and a forebay pump capacity 24 m^3/s, flooding in the Flood Plain begins at T = 10 hours and reaches the maximum value of 325,000 m^3 at T = 22 hours. From T = 22 hours no additional flooding occurs because the greater additional pump capacity can handle the second flood.

Referring to Figure 6.3, using Inflow Hydrograph 2, and considering a forebay pump capacity of 20 m^3/s, flooding begins at T = 6.0 hours and increases until T = 13.5 hours when the maximum flooding volume of 1,160,000 m^3 occurs. From this time, the flooding decreases to 798,000 m^3 at T = 22 hours and remains constant until T = 26 hours. This situation occurs because of a constant inflow in the hydrograph. As the Inflow decreases at T = 26 hours, the volume of flooding in the flood plain also decreases until it reaches Flood Condition I (Figure 5.5) at T = 31 hours, with a volume of flooding of 577,000 m^3. After T = 31 hours the reduction of flood in the flood plain is by normal drainage paths. Using pump capacity of 24 m^3/s, the maximum flooding volume occurs at T = 13.7 hours. Decrease in flooding is faster, and flooding above the Opak bank (Flood II, Figure 5.5) ends at T = 28.5 hours.

Comparison of the flooding resulting from Outlet Improvement Alternative 1 is shown in Figures 6.1 and 6.3.

Outlet Alternative 2: Introducing Automatic Forebay Outlet Gates

Referring to Figure 6.1, considering Inflow Hydrograph 1, and introducing automatic forebay outlet gates, flood plain flooding still occurs from the first storm, but the volume is reduced. As discussed in Section 5.5, the reduction depends on the number of forebay outlet gates and the difference between the forebay and tidal elevations. The results (Figures 5.27 - 5.30 and Figures 6.1 and 6.3) show that flooding occurs during first storm. The reason is that tidal elevation is high, and the gates are ineffective, and therefore no flow to the sea occurs through the Forebay Gates. As a result, the outlet capacity depends only on the current Forebay Pump capacity. However, the duration of flood is smaller than the current condition since in this case, the tidal elevation becomes lower at T = 11 hours. Using Inflow Hydrograph 1 and forebay outlet gates of 2 @ 2 m, flooding occurs from T = 8.5 hours until T = 19 hours. No additional flooding occurs as a result of the second storm. Using forebay outlet gates of 3 @ 2 m, flooding begins at T = 8.5 hours, increases until it reaches a maximum at T = 18.3 hours. After the first storm, the lowering of reservoir elevation using forebay outlet gates is larger compared to pump capacity improvement (Outlet Alternative 1). Therefore more water could be diverted to the reservoir from the syphon. As a result, the water elevation at Opak Drain and flood plain areas remain constant (Flood Condition I, Figure 5.5). Referring to Figure 6.3 and considering Inflow Hydrograph 2, and using forebay outlet gates 2 @ 2 m, flooding begins at T = 6 hours, and increases until T = 13.0 hours when the peak flood occurs. The flooding above the Opak Drain bank (Flood Condition II, Figure 5.5) ends at T = 24 hours, which is about 11 hours sooner than the current condition. Using Forebay Gates of 3 @ 2 m, the peak flood occurs at T = 13.0 hours and the flood above the Opak Drain bank elevation (Flood Condition II) ends at T = 22.5 hours, which is 13.5 hours sooner than the current condition.

Comparison of the volume of flooding in the Flood Plain using each improvement of Outlet Alternative 2 is shown in Figures 6.1 and 6.3. The volumes of flood plain flooding and the reservoir volume available after the storm(s) for Alternative 2 are summarized on Table 6.3.

Table 6.3. Volume of Flooding and Reservoir Volume Available After Storm(s) for Outlet Improvement Alternative 2

Refer to Figures	Flood Hydro-graph	Number of Forebay Gates 2 meters Width	Volume of Flooding in Flood Plain m^3	Reservoir Volume after Storm(s) m^3
6.1 & 5.13	1	-*)	798,000	48,000
6.1 & 5.27	1	2 @ 2.00 m	364,000	660,000
6.1 & 5.28	1	3 @ 2.00 m	337,000	700,000
6.3 & 5.15	2	-*)	1,179,000	297,000
6.3 & 5.29	2	2 @ 2.00 m	1,178,000	724,000
6.3 & 5.30	2	3 @ 2.00 m	1,177,000	803,000

*) current condition

Outlet Alternative 3: Increasing the Forebay Pump Capacity and Introducing Automatic Forebay Outlet Gates

The results (flood stages in the flood plain and reservoir elevation) using this improvement are shown in Figures 5.31 - 5.38. The resulting flood volumes using different combinations of pump capacity and forebay outlet gates are shown in Figures 6.2 and 6.4 and summarized on Table 6.4.

Table 6.4 Volume of Flooding and Reservoir Volume Available After Storm(s) using Outlet Improvement Alternative 3

Figures	Inflow Hydro-graph	Forebay Pump Capacity m^3/s	Number of Forebay Gates of 2 m Width	Volume of Flooding in Flood Plain m^3	Reservoir Volume After Storm(s) m^3
6.1 & 5.13	1	16 *)	-	798,000	48,000
6.2 & 5.31	1	20	2	285,000	780,000
6.2 & 5.33	1	24	2	201,000	1,119,000
6.2 & 5.32	1	20	3	266,000	781,000
6.2 & 5.34	1	24	3	171,000	1,156,000
6.4 & 5.15	2	16 *)	-	1,179,000	297,000
6.4 & 5.35	2	20	2	1,155,000	931,000
6.4 & 5.37	2	24	2	1,137,000	1,043,000
6.4 & 5.36	2	20	3	1,151,000	976,000
6.4 & 5.38	2	24	3	1,133,000	1,113,000

*)current conditions

Considering Inflow Hydrograph 1, the resulting flood volume for each combination is shown in Figure 6.2 (Comparison of Flood Volume). Based on these results, it can be concluded that the beginning of flooding and the time of maximum flooding occurs is approximately the same for all combinations in Outlet Improvement Alternative 3. This flooding is caused by the first storm. Flooding occurs in the Flood Plain area which is lower than the Opak Drain bank elevation (Flood Condition I, Figure 5.5). Therefore the volume of flood remains constant after the peak flood occurs (After T = 18 hours). No additional flooding occurs as a result of the second storm, because the additional outlet capacities (outlet improvements) can handle the second flood. The reduction of flooding in the flood plain after T= 18 hours is by normal drainage paths. Using Inflow Hydrograph 2, the result in Figure 6.4 shows that flooding in the flood plain from T = 6.0 hours until T = 13 hours using the various outlet improvements combinations are approximately the same. This situation occurs because the Opak Drain capacity is not sufficient to carry the Inflow, so that overflow of the Opak Drain banks occurs. Therefore improvements of the outlet system do not influence the amount of flooding very much. However, as shown in Figure 6.4, compared to the current condition, the duration of flood above the Opak Drain bank (Flood Condition II, Figure 5.5) is much smaller for all of these alternative improvements. The end of Flood Condition II (flooding above the Opak Drain bank, Figure 5.5) is between T = 21.5 hours to T = 23.5 hours, compared to the current condition of T = 34.5 hours. Compared to alternatives 1 and 2, this alternative improvement also gives a larger available reservoir storage in the event that another storm should occur shortly after.

In general, improvement of the outlet system alone decreases the water elevation in the forebay, thereby decreasing the elevation in the Pluit Reservoir. The lower the reservoir elevation, the larger the syphon capacity, hence, the larger the flows that can be carried from the Opak Drain. These outlet improvements alone would not solve the flooding problems in the flood plain upstream of the Pluit Selatan Bridge because the current capacity of Opak Drain is not sufficient to handle the incoming flow from upstream. The capacity of the Opak Drain is also reduced due to garbage. For this reason, improvement of the Opak Drain should be carried out.

6.3 IMPROVEMENT ALTERNATIVES TO THE INLET SYSTEM

Two Alternatives of improvements to the Inlet System have been considered:

- Alternative 1: Increasing the Opak Drain Capacity by Increasing the Bank Elevation.
- Alternative 2: Increasing Opak Drain Capacity by cleaning regularly.

The results of the simulation of the inlet system improvement alternatives using Inflow Hydrograph 1 and Inflow Hydrograph 2 are shown in Figures 5.44 - 5.47 and discussed below:

Inlet Alternative 1: Increasing the Opak Drain Capacity by Increasing the Bank Elevation.

Larger flows could be diverted through Opak Drain by this improvement. As a result, flood plain flooding could be reduced. The resulting flood volumes in the flood plain using individual inlet improvements are summarized on Table 6.5

Table 6.5 Volume of Flooding in Flood Plain and Reservoir Volume Available at the End of the Storm(s) using Inlet Improvement Alternative 1

Figures	Flood Hydro-graph	Opak Bank Elevation m	Volume of Flooding in Flood Plain m^3	Reservoir Volume After Storm(s) m^3
6.1 and 5.13	1	+1.00 *)	798,000	48,000
6.5 and 5.49	1	+1.40	0.0	48,000
6.3 and 5.15	2	+1.00 *)	1,179,000	297,000
6.6 and 5.45	2	+1.40	770,000	420,000

*) current conditions

The results in table 6.5 show that although this improvement would eliminate flooding caused by a hydrograph resembling Hydrograph 1, this Inlet improvement alone would not solve the flooding problem entirely caused by flood similar to Inflow Hydrograph 2. The outlet capacity is still limited, because an increase in the Opak Drain elevation would result in an increase in the reservoir elevation and forebay elevation. Once the reservoir elevation is high, the syphon capacity is reduced and excess water occurs upstream of the syphon, and flooding occurs. As explained in Section 5.6 this improvement should be combined with outlet improvements so that sufficient reservoir capacity is available. Comparison of flooding using inlet improvements and current condition is shown in Figure 6.5 and 6.6.

Inlet Alternative 2: Increasing Opak Drain Capacity by Regular Cleaning

In this modification, the discharge capacity is increased because the roughness coefficient is reduced. For the same water level, using this modification, more discharge can be passed. The resulting flood volumes in the flood plain are shown in Figures 5.46 and 5.47, and summarized on Table 6.6. The results on flood volume are shown in Figures 6.5 and 6.6.

Table 6.6. Volume of Flooding in Flood Plain and Reservoir Volume Available at the End of the Storm(s) using Inlet Improvement Alternative 2

Figures	Flood Hydro-graph	Drain Roughness Coefficient	Volume of Flooding in Flood Plain m^3	Reservoir Volume After Storm(s) m^3
6.5 & 5.13	1	0.028 *	798,000	48,000
6.5 & 5.46	1	0.023	217,000	189,000
6.6 & 5.15	2	0.028 *	1,179,000	297,000
6.6 & 5.47	2	0.023	320,000	420,000

* Without Cleaning the Drain

Referring to Figure 6.5, using Inflow Hydrograph 1 and considering inlet improvement Alternative 2, the flooding in the flood plain occurs in the first storm and the second storm. This situation occurs because the current outlet capacity, which is the forebay pump capacity is still limited. As discussed before, this limited pump capacity results in high forebay and reservoir elevation, and therefore, reduced syphon capacity and increased water level in the Opak Drain.

Flooding in the flood plain begins at T = 9 hours, increases to a volume of 116000 m^3 at T= 20 hours. From T= 20 hours until T = 39 hours which is the lag time between the first storm and the second storm, no additional flooding occurs in the flood plain. From T= 39 hours until T= 60 hours, flooding increases as a result of the second storm. The maximum volume of flooding is 217,000 m^3, which is Flood Condition I (Figure 5.5). The reduction of flooding in the flood plain after T = 60 hours is by normal drainage paths.

Referring to Figure 6.6, using Inflow Hydrograph 2, and considering Inlet Improvement Alternative 2, this modification gives a larger flood reduction and lower Reservoir elevation at the end of the storm compare to Inlet Improvement Alternative 1. The risk of this modification is that it depends on the degree of cleaning and the speed of regrowth of the vegetation due the nutrient loading from garbage. If cleaning is regular, and provided regular field inspections are performed, such cleaning would be an inexpensive and effective alternative improvement.

In general, improvement of the Opak Drain only, without improvement of the outlet system (forebay outlet), <u>does not</u> solve the flood problem in the flood plain upstream. The reason is that the small outlet capacity leads to higher forebay levels. High Forebay elevation causes water to backup in the reservoir and in the ring canal which in turn leads to a low syphon capacity and ring canal capacity. Hence, excess water occurs in the upstream of the syphon and Pluit Selatan Bridge because the incoming flow is larger than the syphon and Ring Canal capacity. This excess water increases the water level in the Opak Drain creating the flood.

Inlet Alternative 3: Combinations of Improvement Alternatives to the Outlet System and Inlet system

From the investigation of improvement alternatives combinations, the following evaluations were derived:

Combination 1 and 2: Increase the Opak Drain Capacity and Increase the Forebay Pump Capacity

Combination 1:
- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation + 1.40 m
- Increase the forebay pump capacity from 16 m^3/s (4 @ 4 m^3/s) to 20 m^3/s (5 @ 4.00 m^3/s).

Combination 2:
- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation + 1.40 m
- Increasing the Forebay Pump capacity from 16 m^3/s (4 @ 4 m^3/s) to 24 m^3/s (6 @ 4.00 m^3/s).

The results of the simulation model using Combinations 1 and 2 are shown in Figures 5.48 to 5.51 and Figure 6.7. The resulting flood volumes using Combination Improvements 1 and 2 are summarized on Table 6.7.

Table 6.7 Volume of Flooding in Flood Plain and Reservoir Volume Available at the End of the Storm(s) using Combination Improvements 1 and 2

Figures	Flood Hydro-graph	Forebay Pump Capacity m^3/s	Volume of Flooding in Flood Plain m^3	Reservoir Volume After Storm(s) m^3
6.5 & 5.13	1	16 *)	798,000	48,000
6.5 & 5.48	1	20	-	222,000
6.5 & 5.50	1	24	-	700,000
6.6 & 5.15	2	16 *)	1,179,000	297,000
6.7 & 5.49	2	20	512,000	640,000
6.7 & 5.51	2	24	488,000	912,000

*) current conditions

Note:

- The Opak Drain Elevation for current condition = +1.00 m
- The Opak Drain Elevation for combination improvements = + 1.40 m

No flooding occurs in the event of Inflow Hydrograph 1 as the capacity of the improved Opak Drain and the outlet is sufficient for the Inflow Hydrograph 1. However, using Inflow Hydrograph 2, flooding would still occur. The capacity of Opak Drain would be sufficient if no excess water occurred. Excess water occurs because the capacity of the outlet system is too small, so that forebay elevation is high.

As a result:

- High water levels occur in the reservoir, causing low syphon capacity, until finally, the reservoir is full and no water can flow to the reservoir anymore.
- Water backs up in the ring canal, reducing the flow through the Ring canal. This results in an increased water level in the Opak Drain and causes flooding. Therefore additional improvement at the Outlet should be made to reduce flooding caused by Inflow Hydrograph 2.

Combinations 3 and 4: Increasing Opak Drain Capacity by Increasing the Bank Elevation, Increasing the Forebay Pump Capacity, and Introducing Automatic Forebay Outlet Gates

Combination 3:
- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation + 1.40 m
- Increase the forebay pump capacity to 20 m^3/s
- Introduce automatic forebay outlet gates of 2 @ 2.00 m

Combination 4:
- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation + 1.40 m
- Increase the forebay pump capacity to 24 m^3/s

- Introduce automatic forebay outlet gates of 2 @ 2.00 m

The results of the simulation model using combinations 3 and 4 are shown in Figures 5.52 to 5.55 and Figure 6.7. The resulting flood volumes using Combination Improvements 3 and 4 are summarized on Table 6.8.

Table 6.8 Volume of Flooding in Flood Plain and Reservoir Volume Available at the End of the Storm(s) using Improvement Combinations 3 and 4

Combination improvement	Flood Hydrograph	Number of Forebay Automatic Gates 2 m width	Forebay Pump Capacity m^3/s	Volume of Flooding in Flood Plain m^3	Reservoir Volume After Storm(s) (m^3)
*)	1	2	16 *)	798,000	48,000
3	1	2	20	-	780,000
4	1	2	24	-	1,062,000
*)	2	3	16 *)	1,179,000	297,000
3	2	2	20	507,000	1,042,000
4	2	2	24	483,000	1,229,000

*) current conditions

Note:
- The Opak Drain Elevation for current condition = +1.00 m
- The Opak Drain Elevation for Combination Improvements = + 1.40 m.

Referring to Figures 5.52 and 5.54, using Inflow Hydrograph 1 and considering these Combinations of improvements results in a lower reservoir and forebay elevation. The reservoir and forebay elevations decrease faster after the first storm when the tidal elevation is low. The lowering of reservoir elevation increases the syphon capacity. Therefore no excess water occurs. The increase in the Opak Drain capacity and outlet improvements could overcome Inflow Hydrograph 1, so that no flooding would occur.

Using Inflow Hydrograph 2, the outlet capacity is still not sufficient as the reservoir elevation is high when the peak storm occurs. Therefore very little or no flow from upstream could be diverted through the syphon. The water from upstream is diverted through the ring canal whose capacity is limited. These condition cause an increase in the Opak Drain elevation and flooding in flood plain. Therefore, additional outlet capacity is still needed to overcome the flood using Inflow Hydrograph 2. There is not much difference in the volume of flooding occurs using Combinations 3 and 4. The reason is that flooding occurs when the reservoir elevation is full, is that the additional pump capacity of 4 m^3/s (Combination 3) or 8 m^3/s (Combination 4) is too small compared to the incoming inflow from upstream.

Since the difference between reservoir and forebay elevations is large, more water can be diverted to the forebay if the reservoir outlet gates capacity is larger. Therefore an increase in the reservoir outlet gates is needed as an addition to Combinations 3 and 4.

Combination 5 and Combination 6: Increase Reservoir Outlet Gates in Addition to Increase in Forebay Pump Capacity and Forebay Outlet Gates.

Combination 5:
- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation + 1.40 m
- Increase the forebay pump capacity to 20 m^3/s
- Introducing automatic forebay outlet gates of 3 @ 2.00 m
- Increase the reservoir outlet gates from 3 @ 2 m to 4 @ 2 m

Combination 6:
- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation + 1.40 m
- Increase the forebay pump capacity to 24 m^3/s
- Introducing automatic forebay outlet gates of 3 @ 2.00 m
- Increase the reservoir outlet gates from 3 @ 2 m to 4 @ 2 m

The results of modifications to outlet system Alternative 3 shows that the combination of increasing pump capacity and introducing automatic forebay outlet gates to 3 @ 2.00 m leads to a larger difference in water level in reservoir and forebay. This means more discharge could be diverted to the forebay if the reservoir outlet gates size is increased. In Combination 5 and Combination 6, an increase in reservoir outlet gates is included. The results of the simulation are in Figures 5.56-5.59 and Figure 6.7 and summarized in Table 6.9.

Table 6.9 Volume of Flooding in Flood Plain and Reservoir Volume Available at the End of the Storm(s) using Combination Improvement 5 and 6

Combination improvement	Flood Hydrograph	Forebay Pump Capacity m^3/s	Volume of Flooding in Flood Plain m^3	Reservoir Volume After Storm(s) m^3
*)	1	16 *)	798,000	48,000
5	1	20	-	893,000
6	1	24	-	1,229,000
*)	2	16 *)	1,179,000	297,000
5	2	20	506,000	1,090,000
6	2	24	482,000	1,322,000

*) current conditions

Note: Opak Bank Elevation = +1.40 m
Forebay Outlet Gates = 3 @ 2.00 m
Reservoir Outlet Gates = 4 @ 2.0 m

For current condition: Opak bank elevation = +1.0 m
Reservoir Outlet Gates = 3 @ 2.0 m

Referring to Figures 5.56 and 5.58, using Inflow Hydrograph 1 and considering Combination Improvements 5 and 6, the results show that by adding one reservoir outlet gate of 2.00 m width, the available reservoir volume at the end of the storm is increased. The increase in reservoir volume is greater by using pump capacity of 24 m 3/s compared to using pump capacity of 20 m^3/sec because the forebay elevation is lower. This situation in turn results in a larger differential head between the reservoir and forebay resulting in higher reservoir outlet capacity and lower reservoir elevation. Using Combination Improvement 5, the reservoir elevation at the end of the storms (T = 72 hours) is -0.15 m, which corresponds to an available reservoir volume of 893,000 m^3. Using Combination Improvement 6, the reservoir elevation at the end of the storms (T = 72 hours) is -0.60 m, which corresponds to an available reservoir volume of 1,229,000 m^3. No flooding occurs using this Combination of Improvements. Since the difference in the Opak Drain elevation and reservoir elevation is large, a larger syphon capacity or ring canal capacity could be introduced to divert more water from Opak Drain. This possible improvement is discussed in Combination 7 below.

Using Inflow Hydrograph 2, and pump capacity of 24 m^3/s, the reservoir elevation at the end of the storms (T = 72 hours) is -0.73 or corresponding to an available reservoir volume of 1,322,000 m^3. Flooding still occurs, and larger Opak Drain capacity is still needed. This can be done by cleaning the Opak Drain regularly.

Comparison of volume of flooding using Inflow Hydrograph 2 and all combinations above (Combination 1 until 6) is shown in Figure 6.7. The results show that there is not much difference in volume of flood which occurs using Combination

Improvements 1 to 7. The main difference is that there are an increase in forebay pump capacity because the Opak Drain Capacity is still insufficient, and at the beginning of storm, the tidal elevation is high. Therefore introduction of the forebay outlet gates would not give much reduction in flooding.

Combination Alternative 7: Increase Syphon Capacity and Increase Ring Canal Capacity and Opak Drain capacity by regular cleaning in addition to Combination 6

The components of Combination Alternative 7 are listed as follows:

- Increase the Opak Drain capacity by using dykes to increase the bank elevation to elevation + 1.40 m
- Increase the Opak Drain capacity by cleaning regularly
- Increase the forebay pump capacity to 24 m^3/s
- Introduce automatic forebay outlet gates of 3 @ 2.00 m
- Increase the reservoir outlet gates from 3 @ 2 m to 4 @ 2 m
- Increase the syphon capacity by adding 1 syphon of 4.00 m length (from 13 @ 4 m to 14 @ 4 m)
- Increase the ring canal capacity by cleaning regularly

This combination includes all improvements to the inlet and outlet systems. It is expected that this improvement would lead higher costs, but it should result in significantly higher benefits. Therefore this alternative is included in the study. This combination is also considered in the sensitivity analysis discussed in Section 7. The results using this combination of improvements are shown in Figures 5.60 - 5.61 and summarized in Table 6.10.

Table 6.10 Volume of Flooding in Flood Plain and Reservoir Volume Available the End of the Storm(s) using Combination Improvement 7

Combination improvement	Flood Hydrograph	Forebay Pump Capacity m^3/s	Volume of Flooding in Flood Plain m^3	Reservoir Volume After Storm(s) m^3
*)	1	16 *)	798,000	48,000
7	1	24	-	1,157,000
*)	2	16 *)	1,179,000	297,000
7	2	24	-	1,322,000

*) current conditions

Using this combination no flooding occurs for either Inflow Hydrograph 1 or 2. Referring to Figure 5.60, the reservoir elevation is higher than using Combination Improvement 6 (Figure 5.58), but the elevation in the Opak drain is lower. This is because in Combination 7, the syphon capacity is larger, and therefore more water can be diverted to the reservoir. The elevation in the Opak Drain is also lower because of the increase in the Ring Canal capacity by regular cleaning. The capacity of the Opak Drain using

Combination Improvement 7 is sufficient and if regular cleaning could be relied upon, the increase in the Opak Drain elevation could be reduced to +1.20 m, an increase of 0.20 m from the current condition.

From the evaluation of the aforementioned improvement alternatives, the following observations can be made:

- Improvement of the outlet system decreases the water elevation in the forebay and therefore decreases the elevation of the Pluit Reservoir. The lower the reservoir elevation, the larger the syphon capacity and, hence, the larger the flows that can be carried from the Opak Drain. Improvements to the outlet system only would solve the flood problem in the flood plain upstream of the Pluit Selatan Bridge, if the Opak Drain capacity is high enough to carry the inflow. However, the current capacity of the Opak Drain is reduced due to the dumping of garbage, therefore the capacity of the Opak Drain is smaller than the incoming flow from upstream. For this reason, improvement of the Opak Drain should also be carried out.
- Improvement of the Opak Drain by itself without improvements to the outlet system (forebay outlet) would not solve the flooding problem in the flood plain upstream because the forebay elevation is high due to the inadequate forebay pump capacity. As a result the reservoir elevation is also high. This condition leads to a low syphon capacity and excess water upstream of the

syphon and Pluit Selatan Bridge. The excess water occurs because the incoming flow is larger than the syphon and ring canal capacity. This excess water increases the water level in the Opak Drain and in the Flood Plain area.

- Based on the results of the improvement of the inlet system (Opak Drain) and outlet system (pumps, automatic forebay outlet gates and reservoir outlet gates), a combination of improvements (inlet and outlet improvements) is required to solve flooding problems in Pluit Polder System.
- Figure 6.1 shows the importance of the reduction in reservoir elevation in readiness for the next successive storm. The previous study did not consider successive storm events. The results of this study show that because the reservoir elevation is still high after the first storm, flooding increases with the occurrence of the second storm event, even for storms with a lower peak intensity. The duration of the first storm and the time lag between the successive storm is important in determining the available detention storage. Therefore, it is very important to consider successive storm events in the design of any Polder system.
- Topographical data in the flood plain area is required to determine the flood stage. In this study the flood plain area is assumed to be flat plain. Once topographical data

is available, the flood stage can be used with flood volume to determine extent of flooding.

- The computer model developed in this study can be used to determine the resulting flooding in the flood plain and the resulting stages in the system as a result of other inflow hydrographs and other variations in improvements.
- This model can also be used for the design of other similar Polder systems.
- The choice of improvements should be made depending on the cost of the improvements and the benefit of the reduction in flood damage. This study did not include the optimization of the combination of alternative improvements. Other combinations of improvements can be evaluated using this model. Further study should be done to optimize alternatives or combinations improvements. The hydrological data to support the study on the probabilities of multiple storms are important in optimization study of the Alternatives or Combinations Improvements.
- The environment and social impact caused by flooding must also be considered in cost calculations. It is not included in this study since no data are available. The benefits can be calculated based on the impact of volume of flood reduction to the corresponding damage, risk and flood probabilities.

CHAPTER 7

SENSITIVITY ANALYSIS

This section discusses the sensitivity of the Pluit Polder System to the occurrence of two types of events. These are:

- Multiple Storm Events
- Forebay Pump Clogging Due to Garbage

7.1 MULTIPLE STORM EVENTS

From the discussion in Chapter 5 it is apparent that the reservoir elevation at the beginning of a storm event is critical in determining the ability of the Pluit system to prevent flooding caused by the next successive storm. The previous studies (refer to Section 2.2) do not consider multiple storm events. The results in Chapter 5 show that if the reservoir has not been emptied after a storm, flooding may occur due to the occurrence of another storm. The duration of the storm and the time lag between successive storms are important in determining the available reservoir storage. Therefore, it is very important to consider multiple storms in analyzing the Polder system. For this sensitivity analysis, four combinations of inflow hydrographs were considered.

Flow Scenario 1: Combination of Inflow Hydrograph 2 followed by the second storm of Inflow Hydrograph 1 (Figure 7.1). The time lag between the storms is taken to be the same as in the three days of actual streamflow measurement shown in Inflow Hydrograph 1 (refer to Figures 3.5 and 4.5).

Flow Scenario 2: Flow Scenario 1 with a different time lag between the stormss. Three time lags are simulated:

Flow Scenario 2 A:

Time lag between the storms is 6 hours longer than that used in Flow Scenario 1 (Figure 7.2)

Flow Scenario 2 B:

Time lag between the storms is 6 hours shorter than that used in Flow Scenario 1 (Figure 7.3).

Flow Scenario 2 C:

Time lag between the storms is 12 hours longer than that used in Flow Scenario 1 (Figure 7.4)

Flow Scenario 3: Combination of the second storm of Inflow Hydrograph 1 followed by the Inflow Hydrograph 2 (Figure 7.5).

Flow Scenario 4: Combination of the second storm of Inflow Hydrograph 1 followed by the first storm of the same inflow hydrograph (Figure 7.6).

The computer model was used to simulate each of the above flow scenarios Inflow hydrographs. Two conditions of the Pluit System were considered in each combination:

- Current condition
- Improved Conditions: Combination 7 was selected in the investigation since this combination of improvements could overcome the flooding problem for Inflow Hydrograph 1 and Inflow Hydrograph 2 (Section 5.7).

The current condition and the improved conditions are shown in Table 7.1:

Table 7.1 The Current Condition and Improved Conditions

	Current Condition	Improved Condition
Elevation Opak Drain bank	+1.00	+1.40
Cleaning Opak Drain	----	regularly
Syphon Length	52.00 m	56.00 m
Pump capacity	16 m^3/s	24 m^3/s
Forebay outlet gates	----	3 @ 2.00 m
Reservoir Outlet Gates	3 @ 2.00 m	4 @ 2.00 m
Cleaning Ring Canal	----	regularly

The results are shown in Figures 7.7-7.18 and are discussed below:

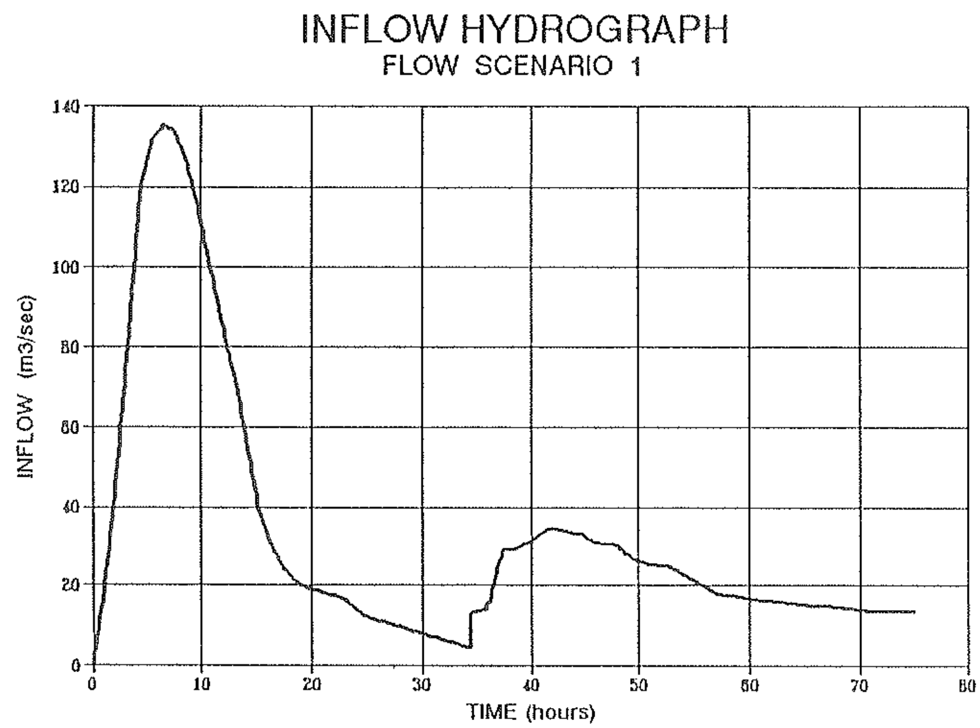


Figure 7.1 Inflow Hydrograph Flow Scenario 1

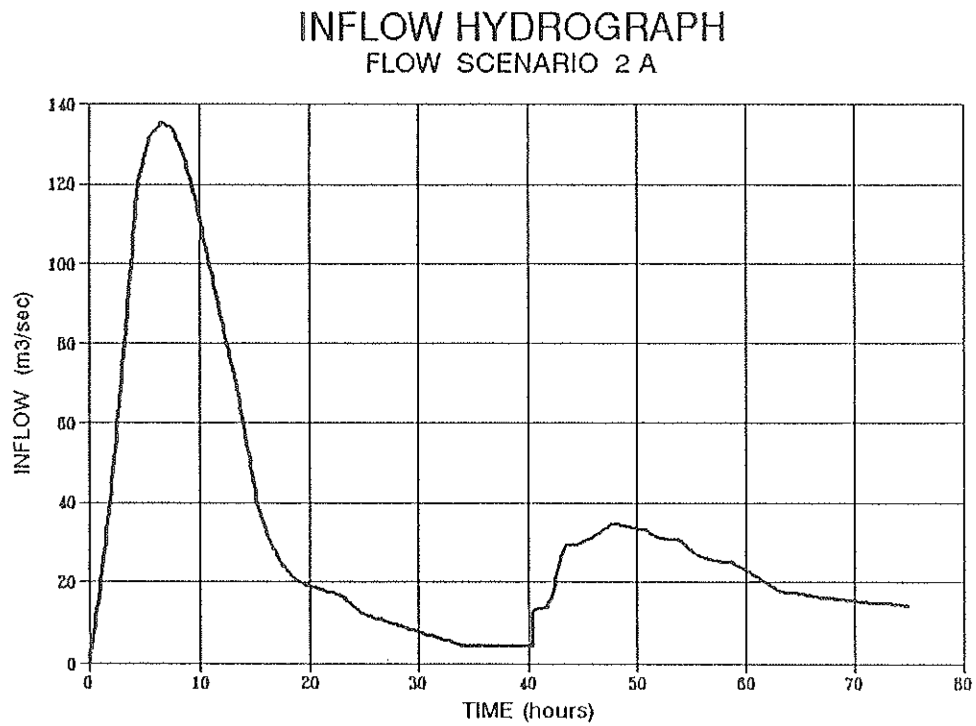


Figure 7.2 Inflow Hydrograph Flow Scenario 2 A

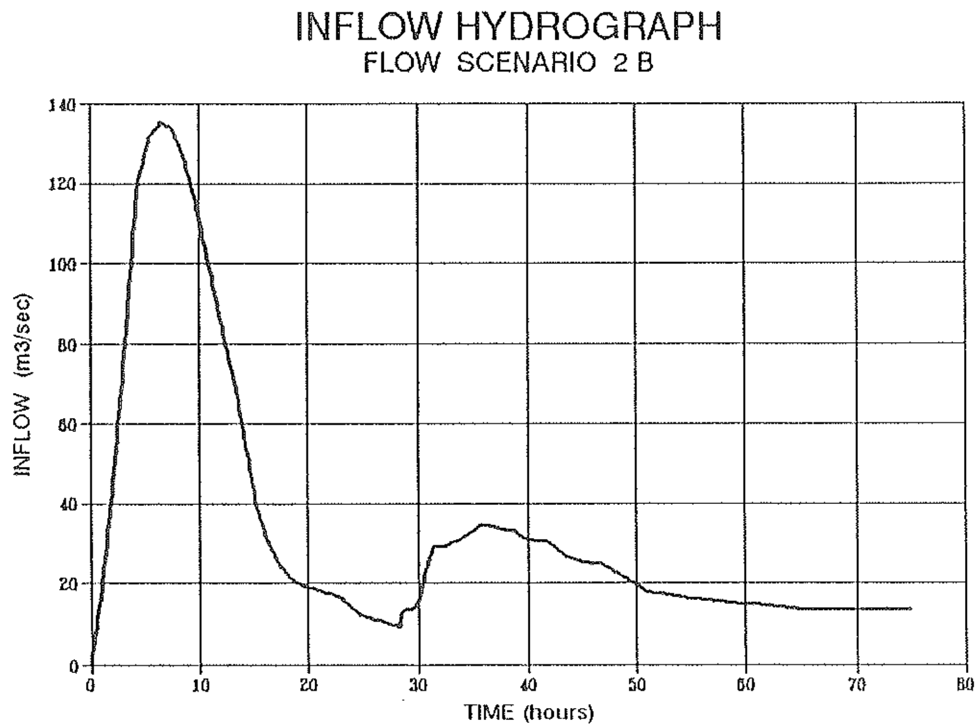


Figure 7.3 Inflow Hydrograph Flow Scenario 2 B

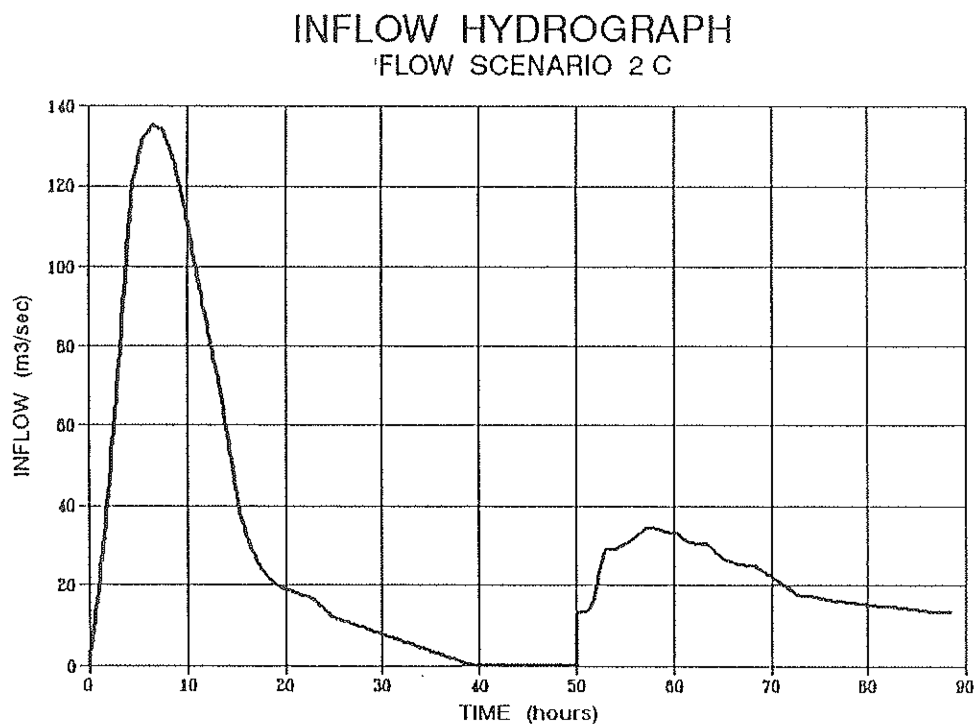


Figure 7.4 Inflow Hydrograph Flow Scenario 2 C

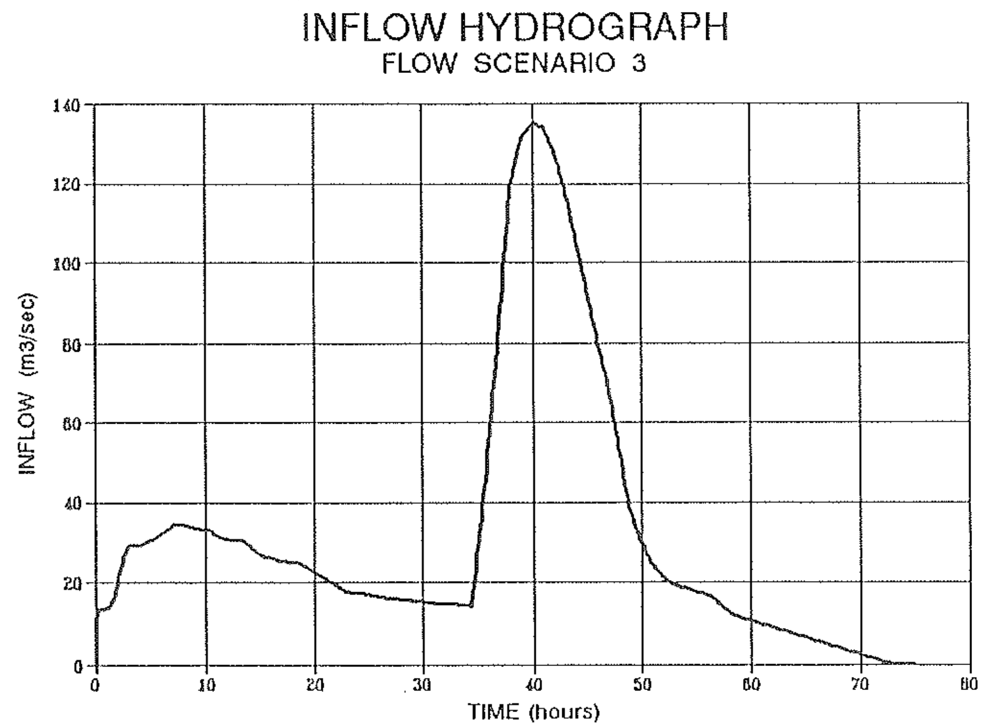


Figure 7.5 Inflow Hydrograph Flow Scenario 3

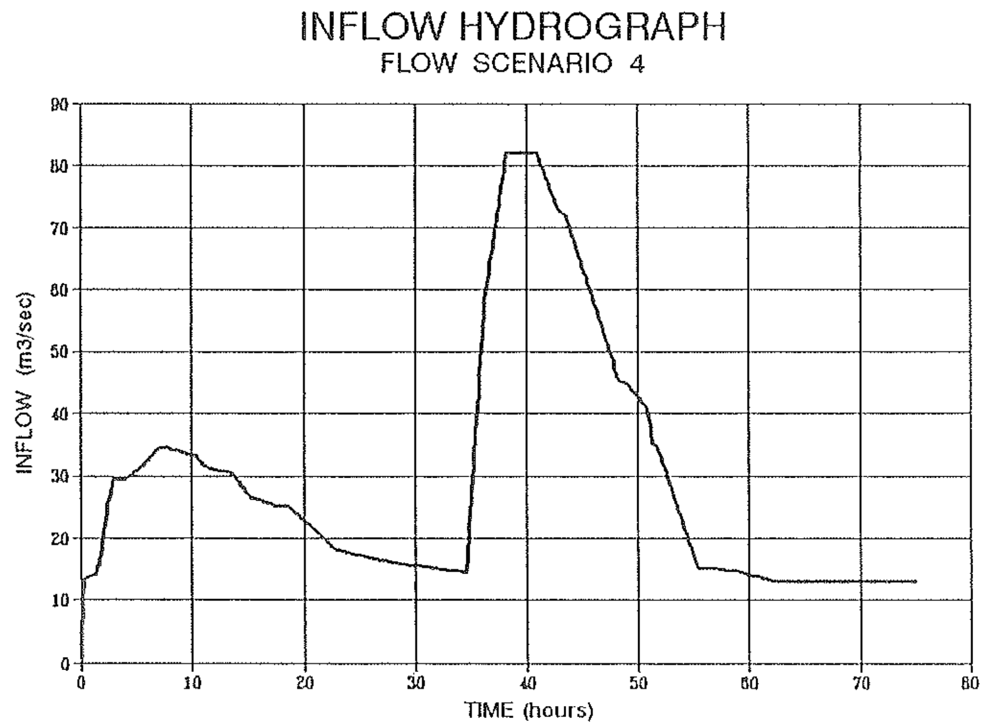
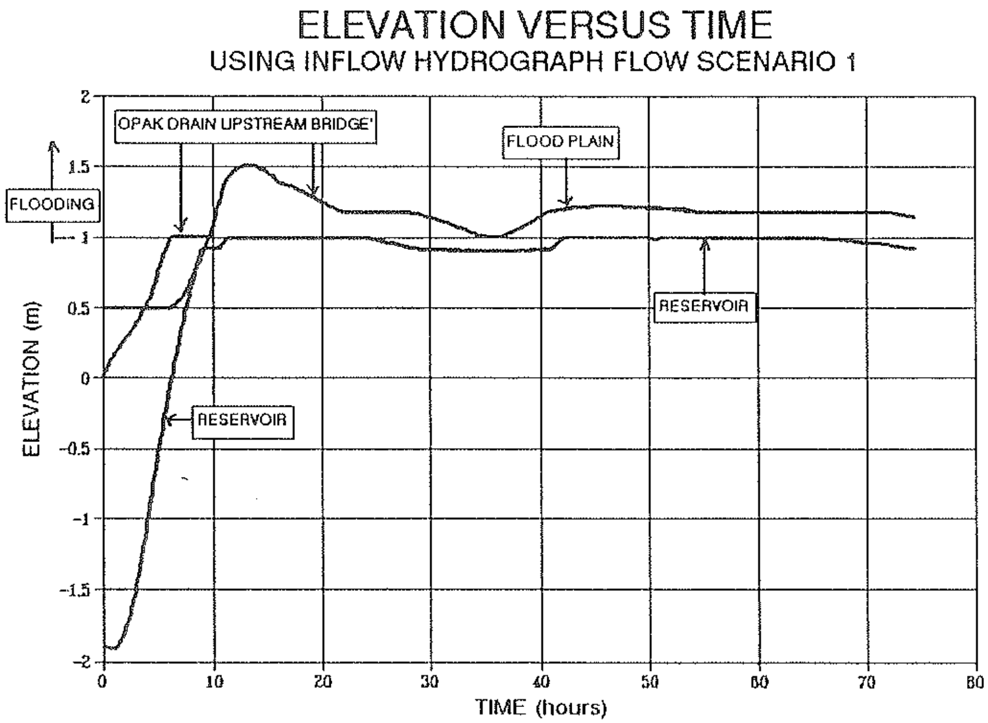
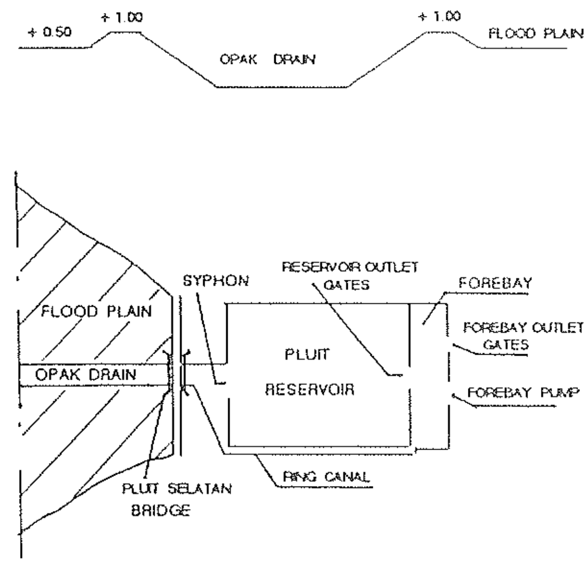


Figure 7.6 Inflow Hydrograph Flow Scenario 4





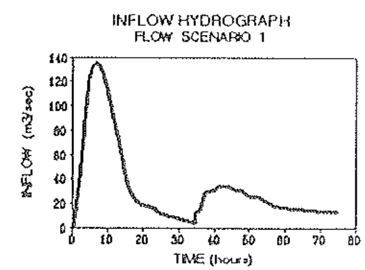
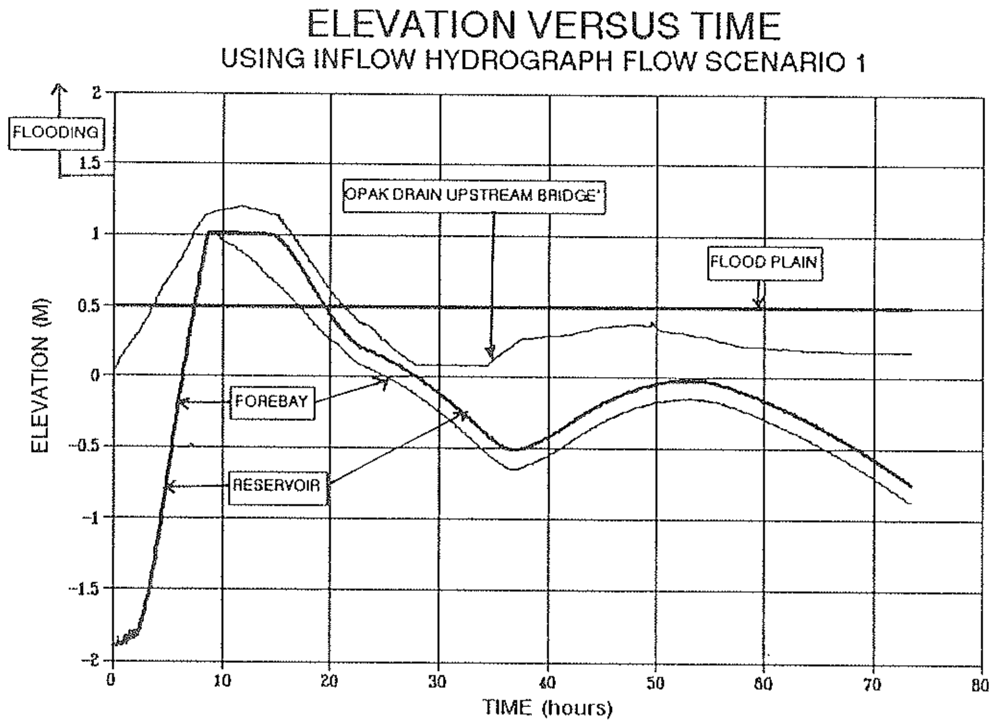
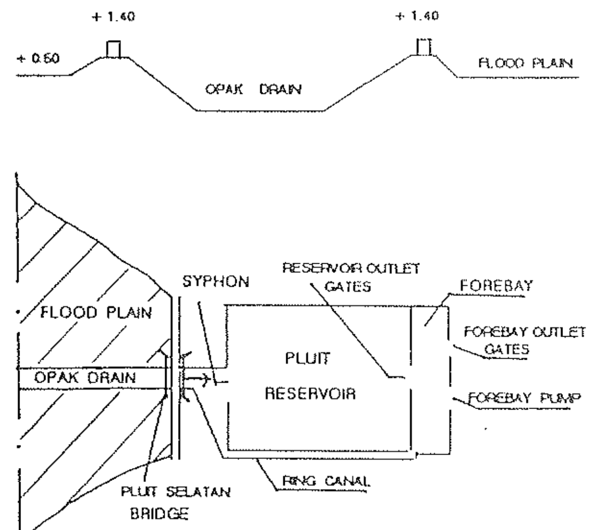


Figure 7.7 Elevation versus Time using Inflow Hydrograph Flow Scenario 1 (Current Condition)





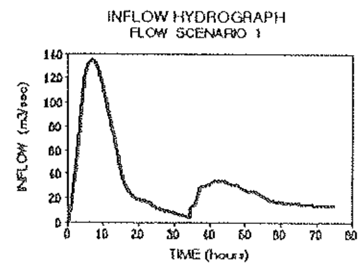
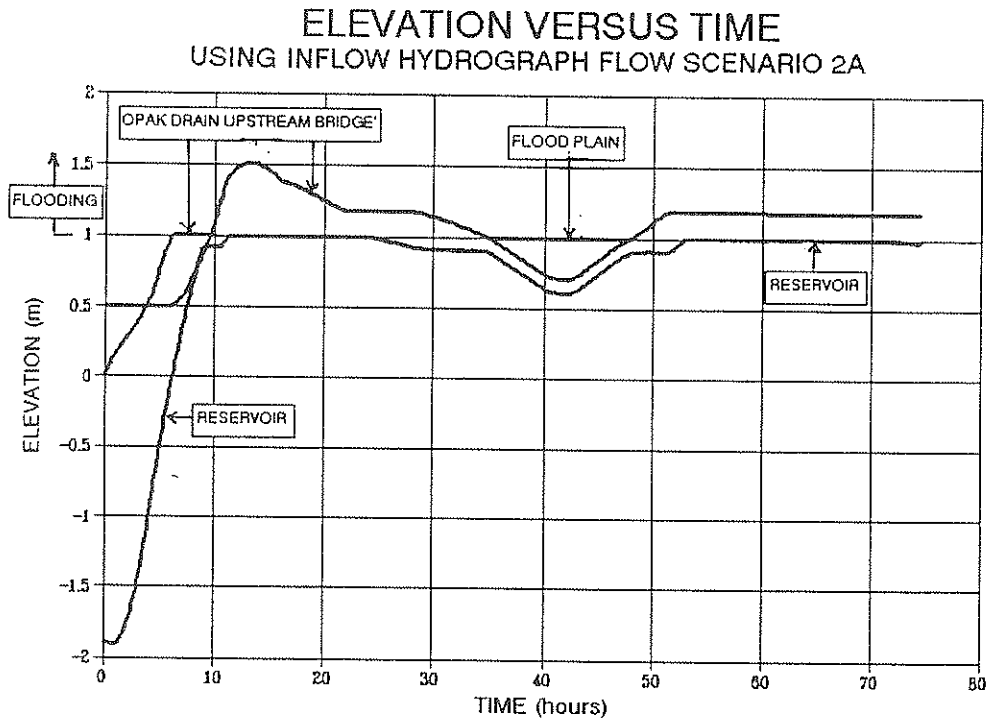


Figure 7.8 Elevation versus Time using Inflow Hydrograph Flow Scenario 1

Improved Conditions: Elevation Opak Drain Bank: + 1.40 m
Regularly Cleaning Opak Drain
Pump capacity: 24 m3/sec
Forebay Outlet Gates: 3 @ 2.0 m
Reservoir Outlet Gates: 4 @ 2.0 m
Syphon Length: 56.00 m
Regularly Cleaning Ring Canal



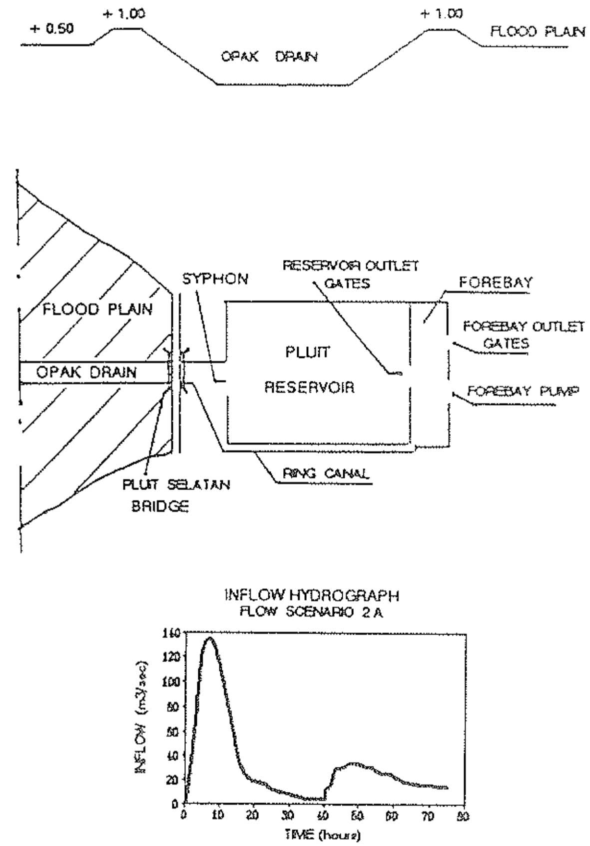
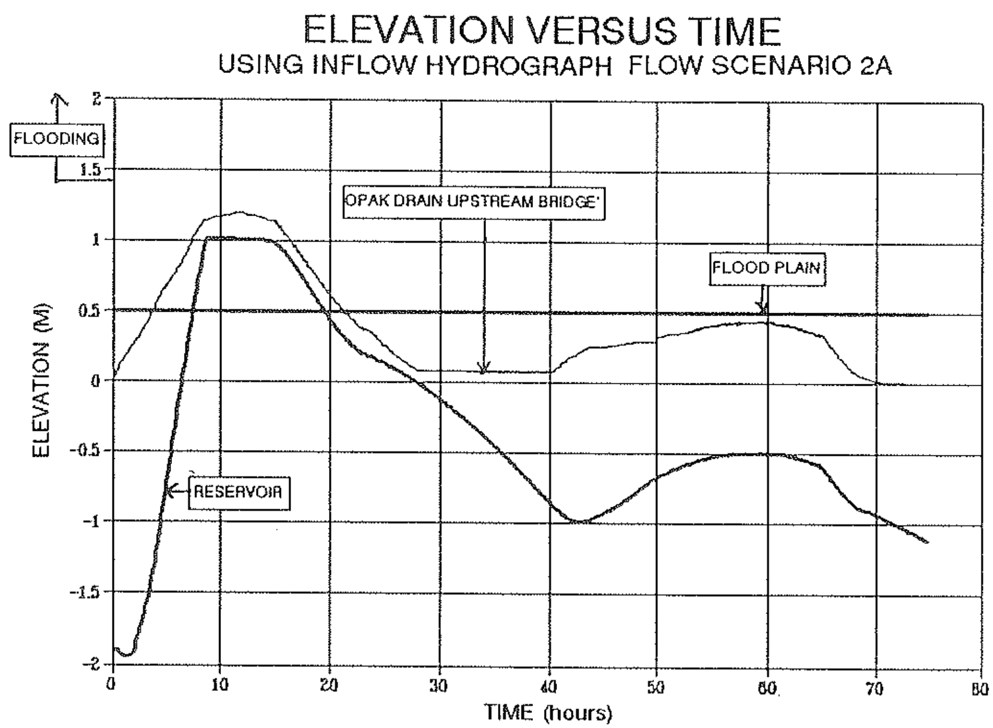
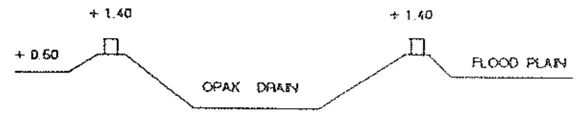
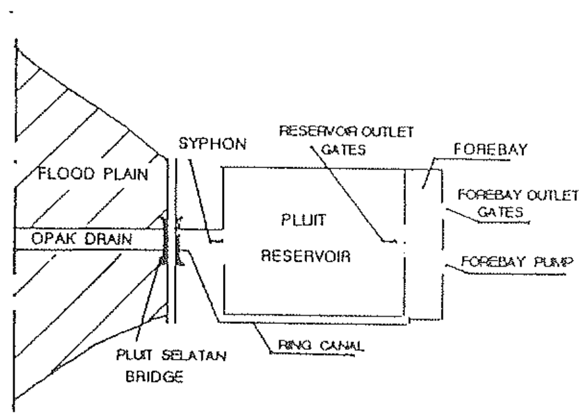


Figure 7.9 Elevation versus Time using Inflow Hydrograph Flow Scenario 2 A (Current Condition)







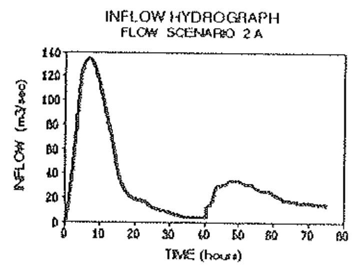
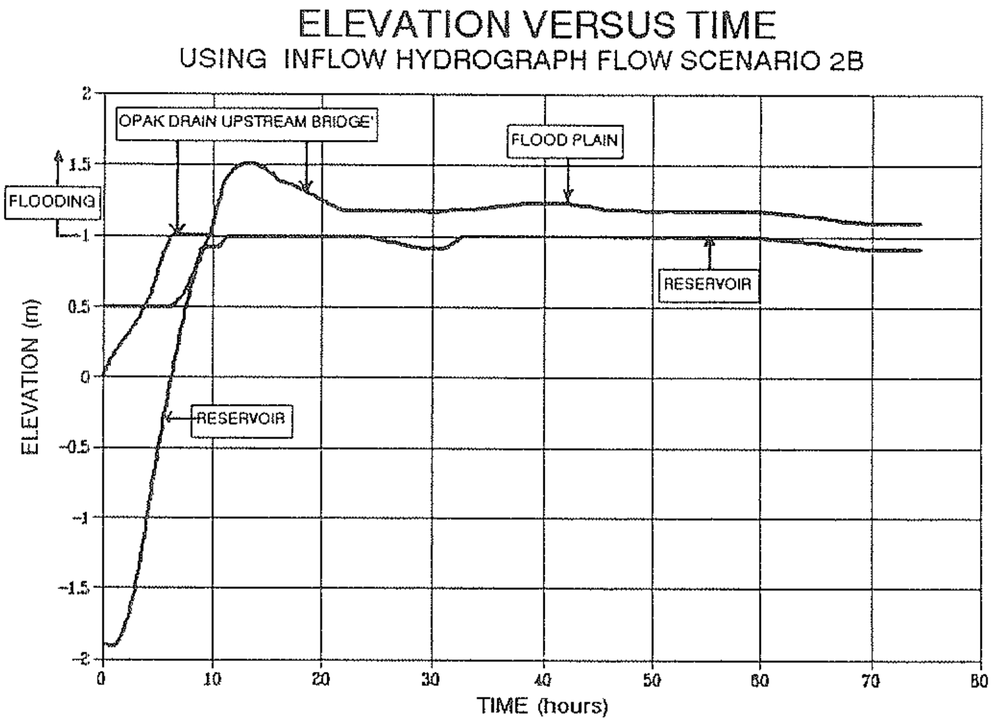


Figure 7.10 Elevation versus Time using Inflow Hydrograph Flow Scenario 2 A

Improved Conditions: Elevation Opak Drain Bank: + 1.40 m
Regularly Cleaning Opak Drain
Pump capacity: 24 m3/sec
Forebay Outlet Gates: 3 @ 2.0 m
Reservoir Outlet Gates: 4 @ 2.0 m
Syphon Length: 56.00 m
Regularly Cleaning Ring Canal



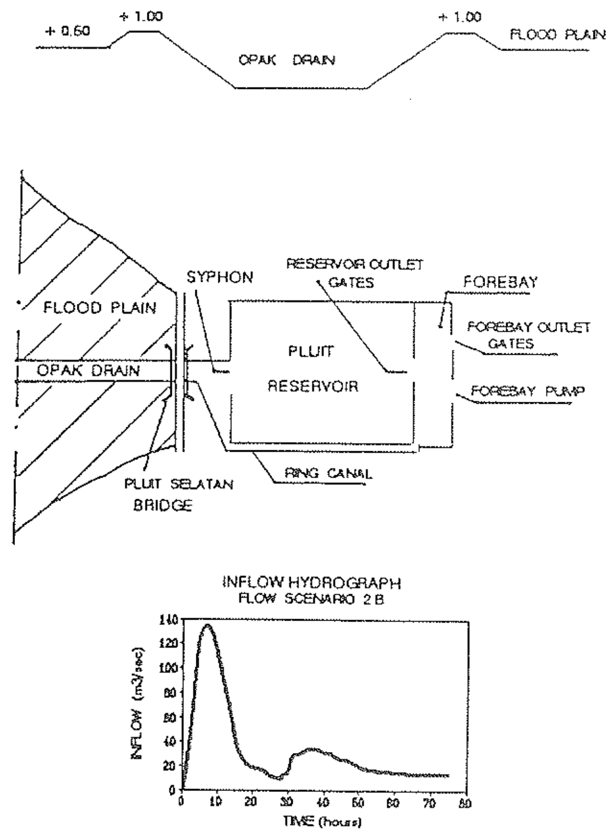
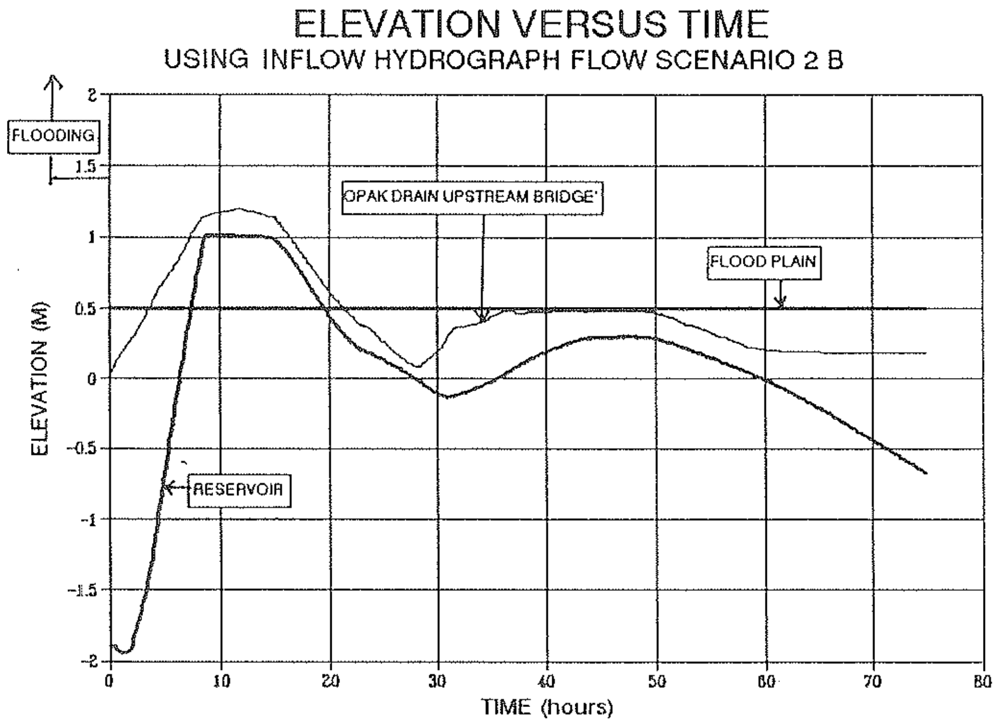


Figure 7.11 Elevation versus Time using Inflow Hydrograph Flow Scenario 2 B (Current Condition)



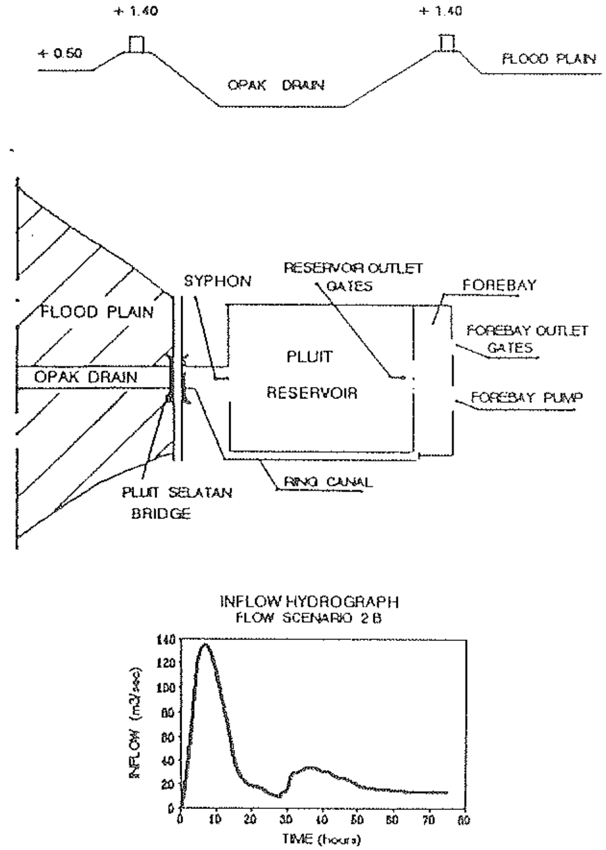
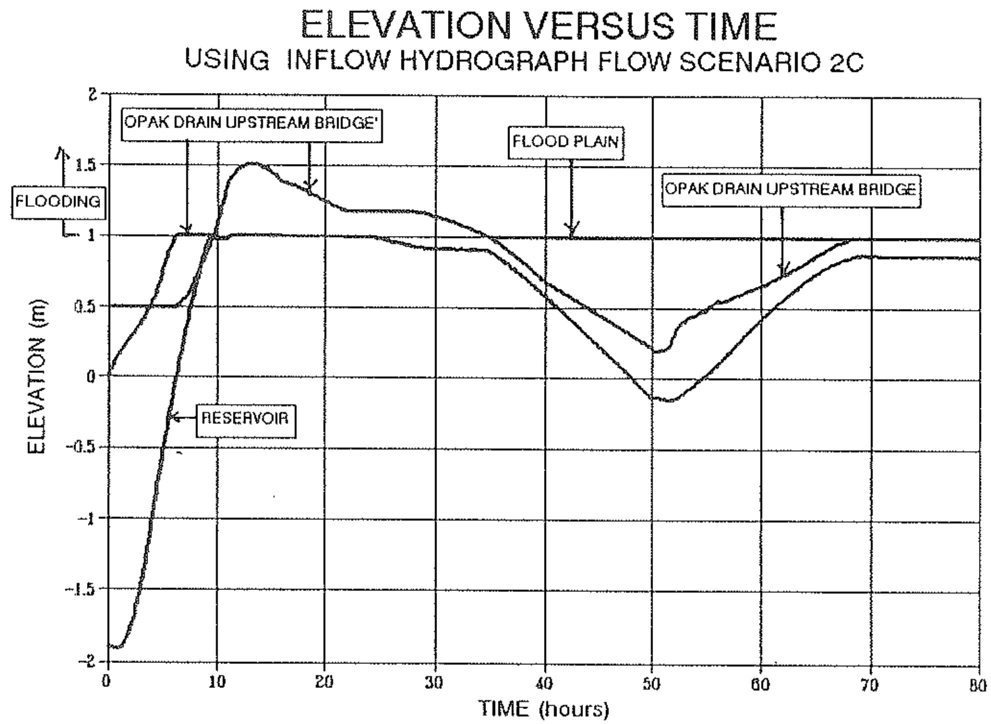
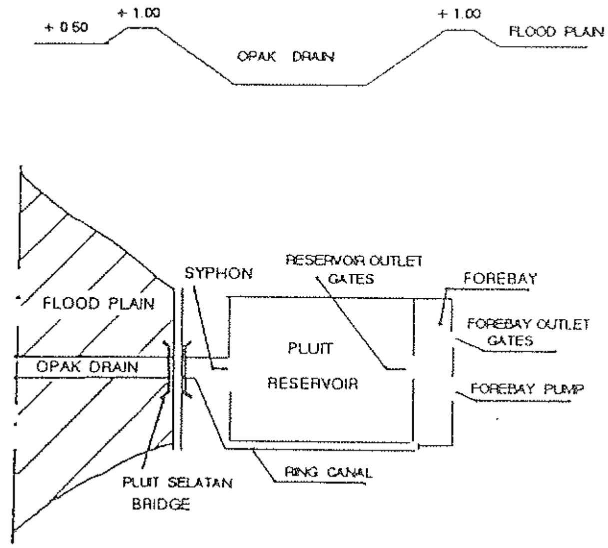


Figure 7.12 Elevation versus Time using Inflow Hydrograph Flow Scenario 2 B

Improved Conditions:
- Elevation Opak Drain Bank: + 1.40 m
- Regularly Cleaning Opak Drain
- Pump capacity: 24 m3/sec
- Forebay Outlet Gates: 3 @ 2.0 m
- Reservoir Outlet Gates: 4 @ 2.0 m
- Syphon Length: 56.00 m
- Regularly Cleaning Ring Canal





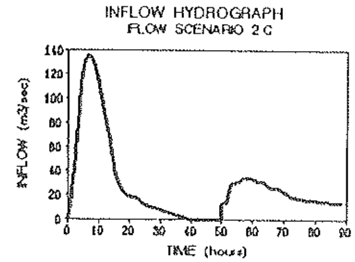
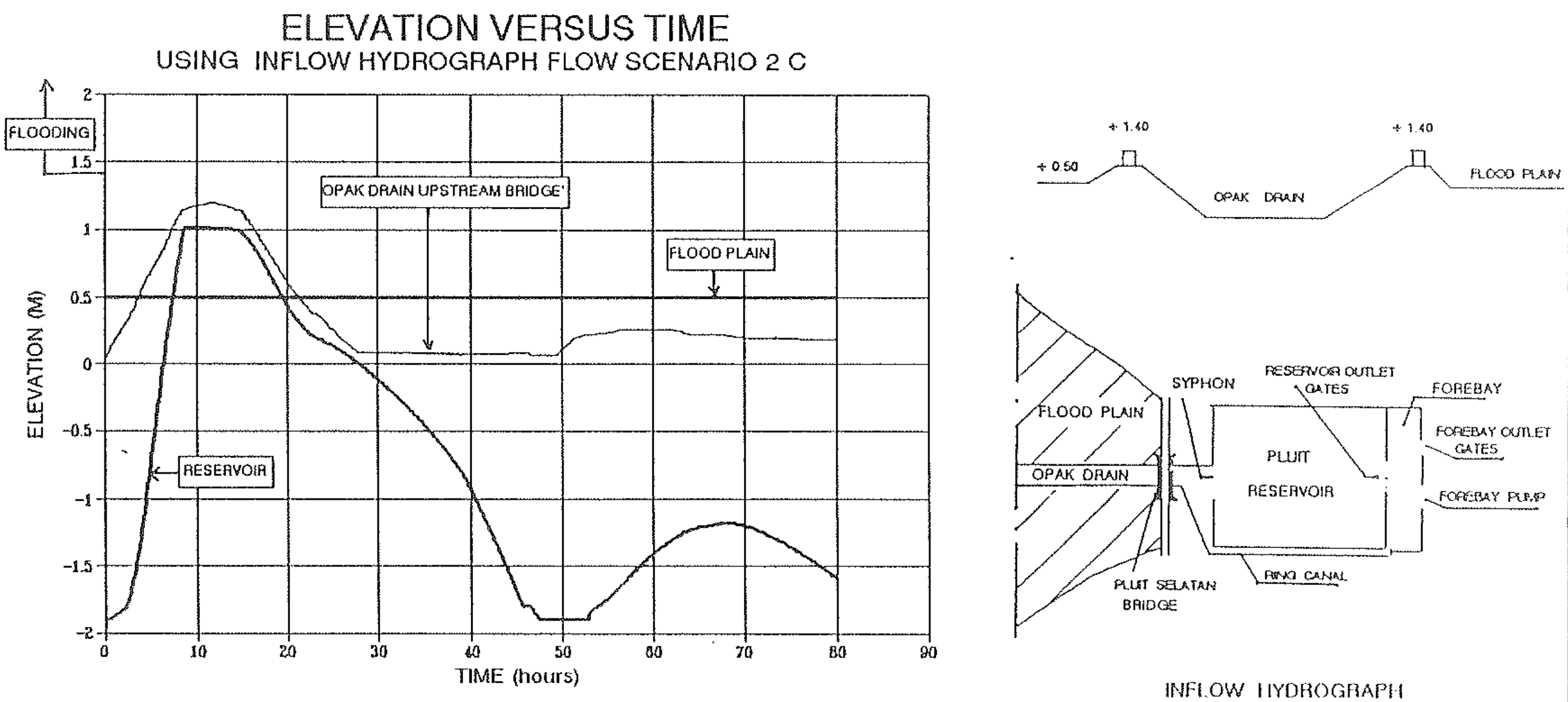


Figure 7.13   Elevation versus Time using Inflow Hydrograph Flow Scenario 2 C (Current Condition)



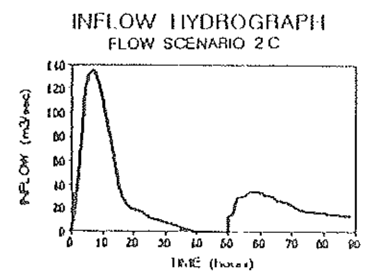
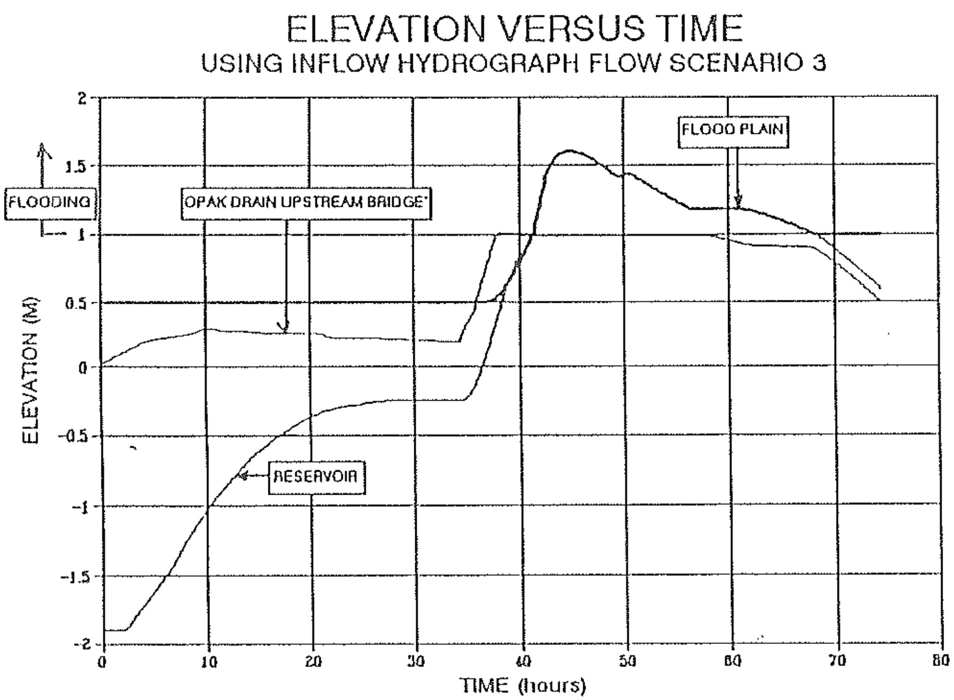
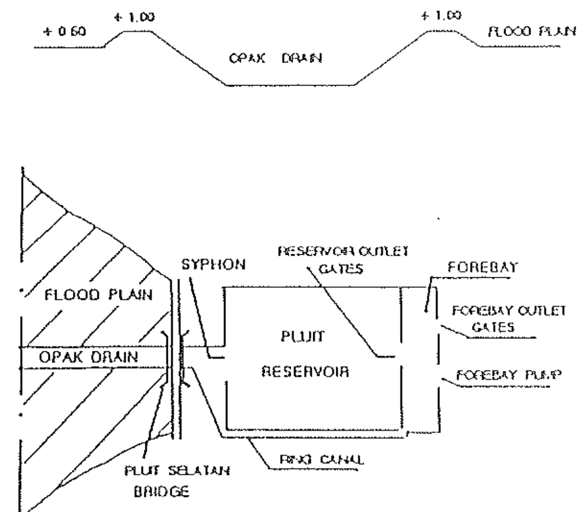


Figure 7.14 Elevation versus Time using Inflow Hydrograph Flow Scenario 2 C

Improved Conditions: Elevation Opak Drain Bank: + 1.40 m
Regularly Cleaning Opak Drain
Pump capacity: 24 m3/sec
Forebay Outlet Gates: 3 @ 2.0 m
Reservoir Outlet Gates: 4 @ 2.0 m
Syphon Length: 56.00 m
Regularly Cleaning Ring Canal





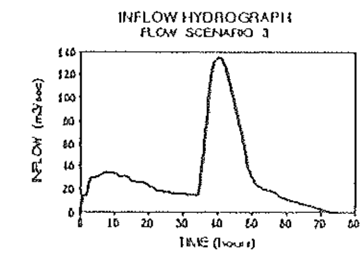
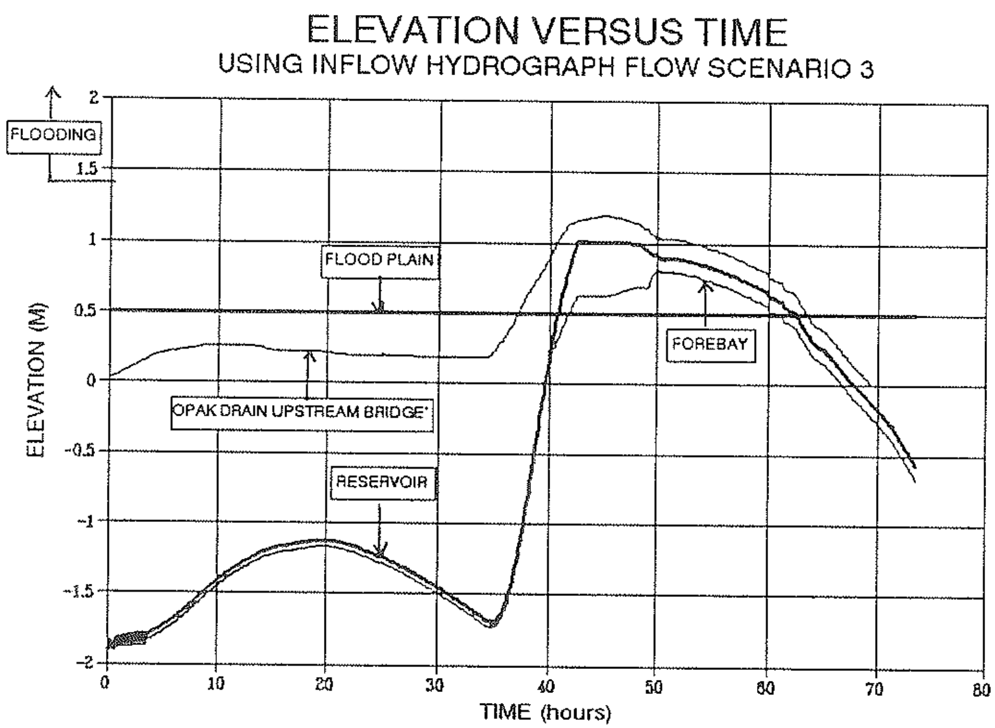


Figure 7.15 Elevation versus Time using Inflow Hydrograph Flow Scenario 3 (Current Condition)



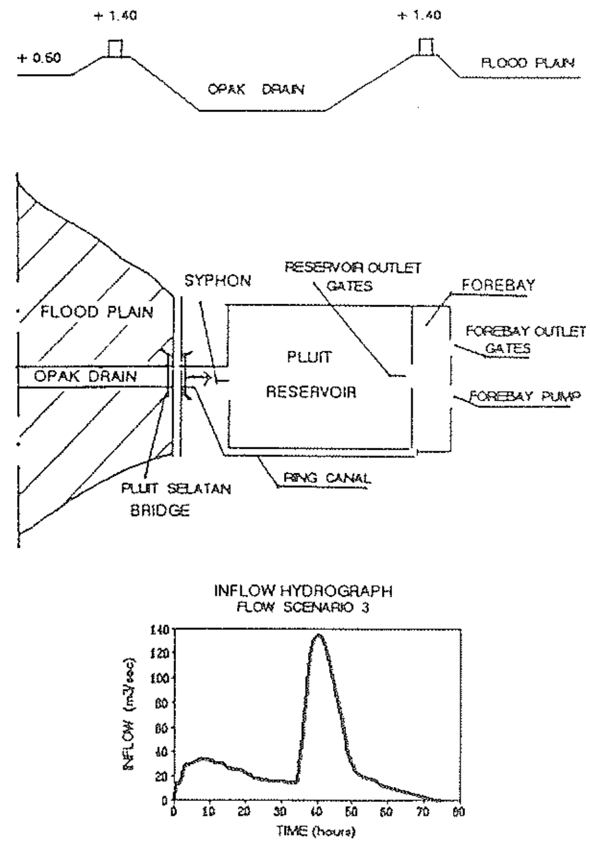
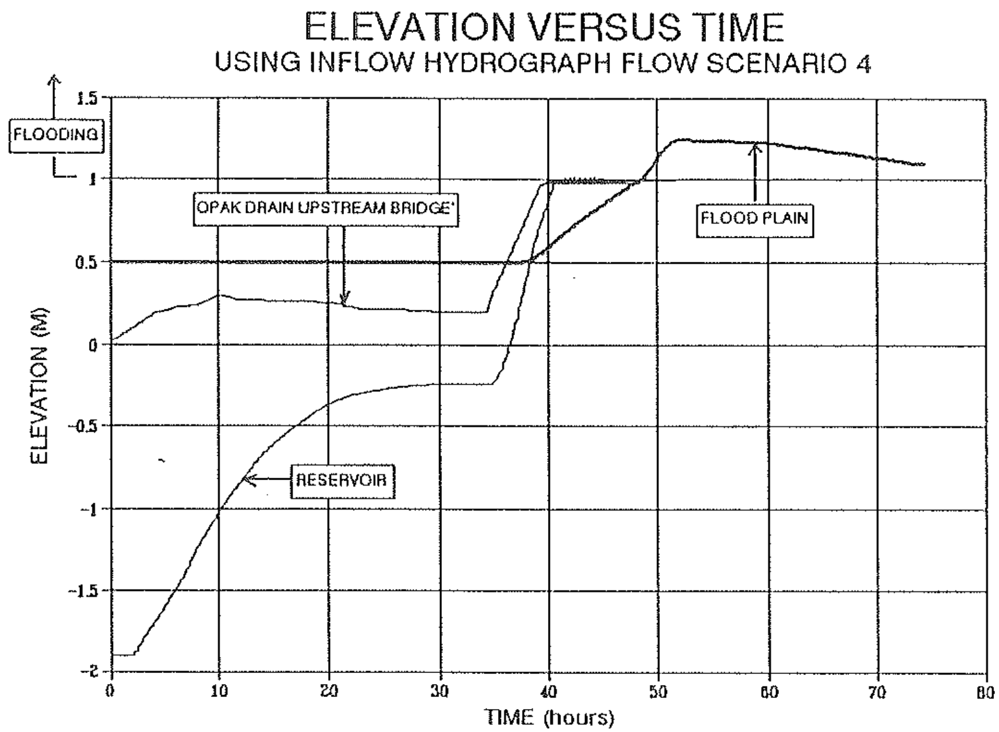
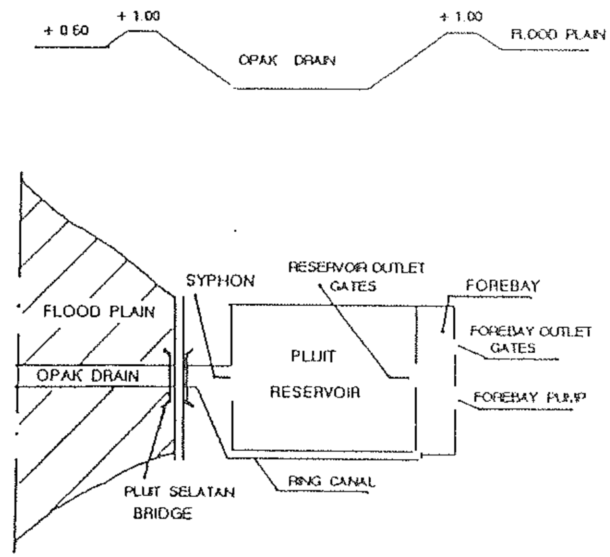


Figure 7.16 Elevation versus Time using Inflow Hydrograph Flow Scenario 3

Improved Conditions: Elevation Opak Drain Bank: + 1.40 m
Regularly Cleaning Opak Drain
Pump capacity: 24 m3/sec
Forebay Outlet Gates: 3 @ 2.0 m
Reservoir Outlet Gates: 4 @ 2.0 m
Syphon Length: 56.00 m
Regularly Cleaning Ring Canal





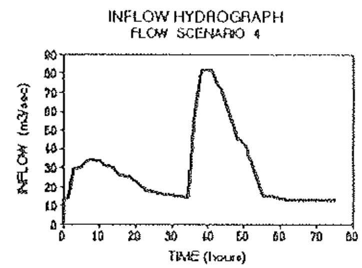
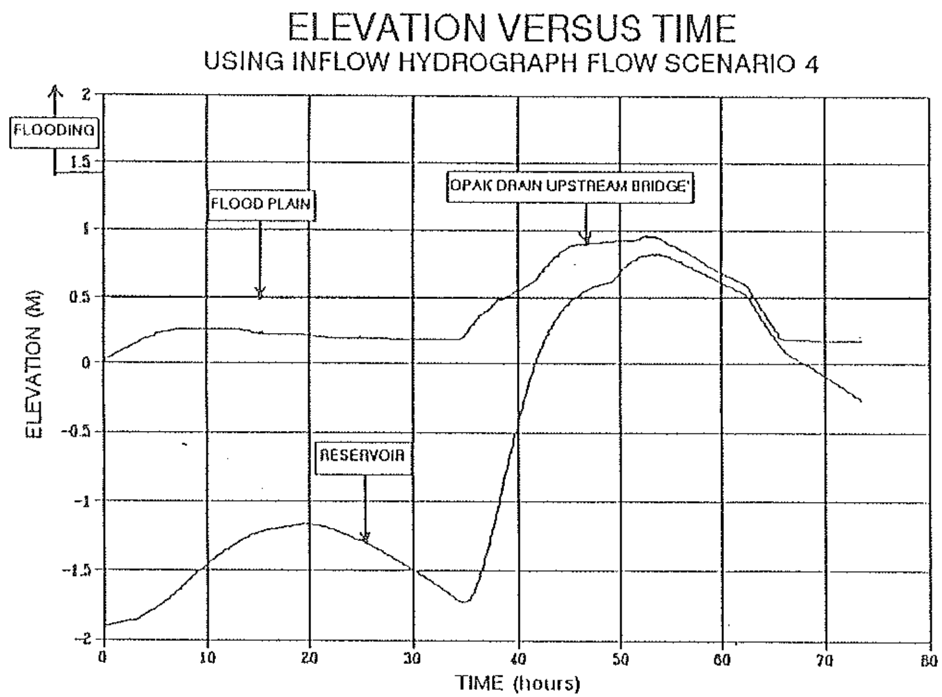
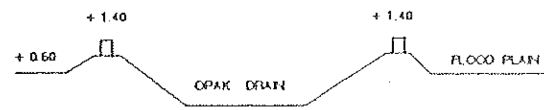
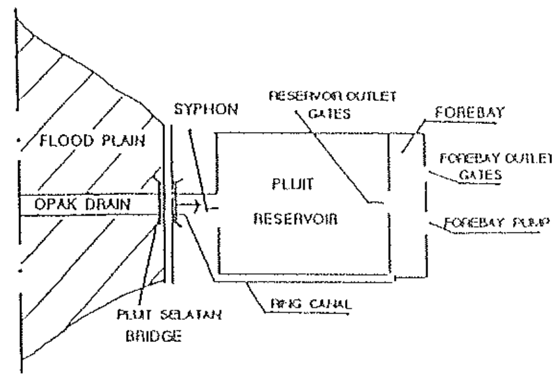


Figure 7.17 Elevation versus Time using Inflow Hydrograph Flow Scenario 4 (Current Condition)







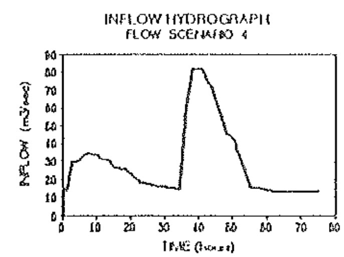



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Figure 7.18  Elevation versus Time using Inflow Hydrograph
             Flow Scenario  4
Improved Conditions:  Elevation Opak Drain Bank: + 1.40 m
                      Regularly Cleaning Opak Drain
                      Pump capacity: 24 m3/sec
                      Forebay Outlet Gates: 3 @ 2.0 m
                      Reservoir Outlet Gates: 4 @ 2.0 m
                      Syphon Length:  56.00 m
                      Regularly Cleaning Ring Canal
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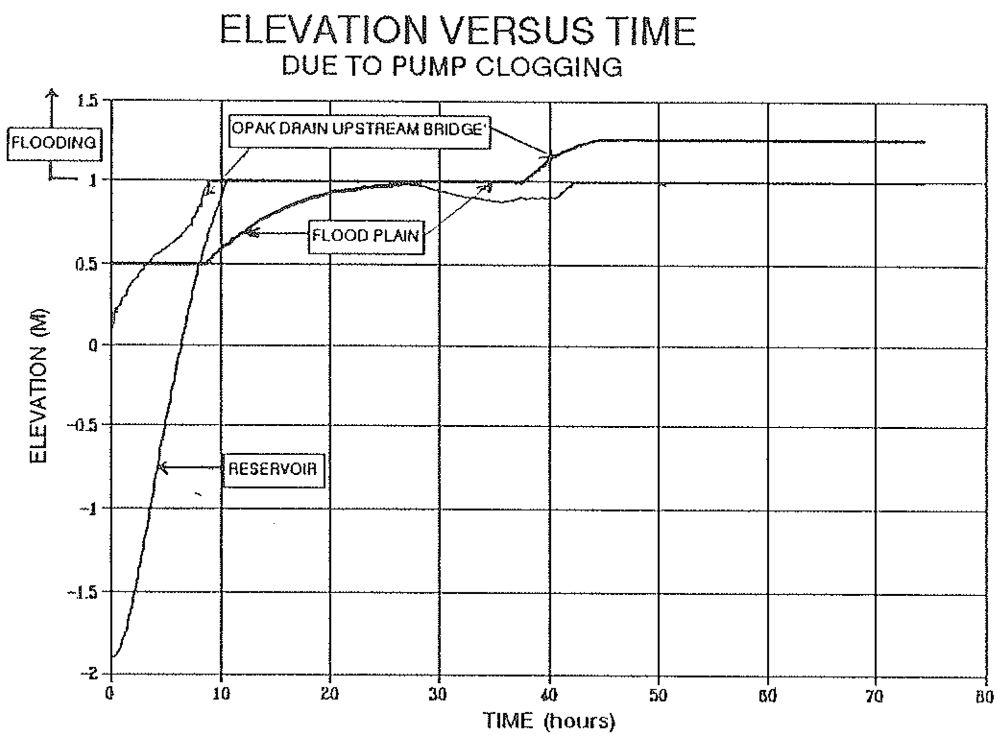
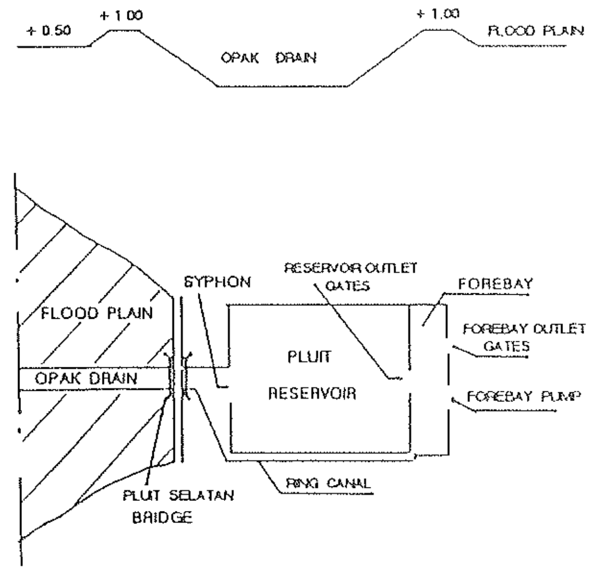




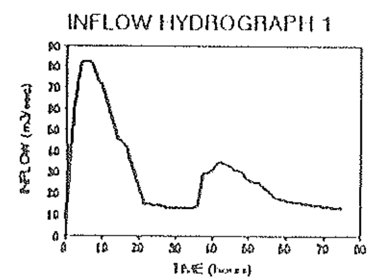
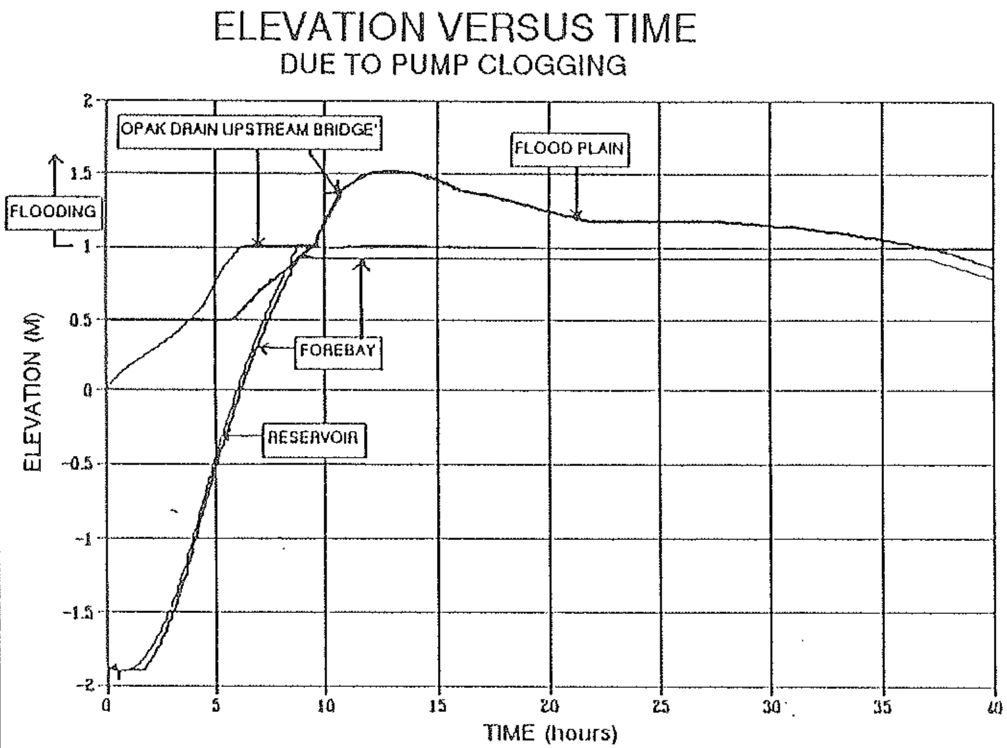
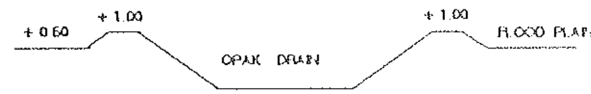
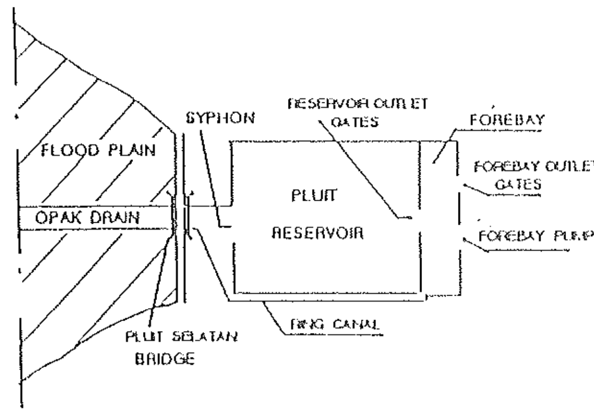


Figure 7.19 Elevation versus Time
Condition: Pump capacity: 12 m3/sec (due to Pump clogging)
Inflow Hydrograph 1







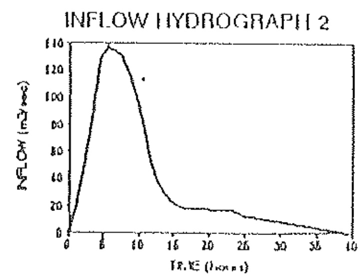


Figure 7.20 Elevation versus Time
Condition: Pump capacity: 12 m3/sec (due to Pump clogging)
             Inflow Hydrograph 2

Flow Scenario 1: Using the derived hydrograph shown in Figure 7.1 and the current condition of Pluit Polder system, the results are shown in Figure 7.7. The elevation of flooding in the flood plain caused by the first storm is + 1.50 m, which corresponds to the volume of flooding in the flood plain of 1,179,000 m^3. The peak of the flood in the flood plain occurs at T=13 hours and reduces until elevation +1.02 or volume of flooding 583,000 m^3 at T= 34 hours. From T=34 hours until T= 36 hours the condition of flooding in the flood plain remains the same. This is the time between the first storm and the second storm. The reservoir elevation at the beginning of the second flood is still high, which means very little reservoir volume is available for the second storm. This high Reservoir elevation results in a small syphon capacity, large excess of water in Opak Drain, and hence flooding occurs as a result of the second storm. The peak elevation of flooding in the Flood Plain caused by the second storm is +1.23 m, which corresponds to a volume of flooding in the Flood Plain of 846,000 m^3. Using improved conditions (Figure 7.8), no flooding occurs as a result of the first and the second storm. The reservoir elevation after the first storm reduces to - 0.51 m, which corresponds to an available Reservoir volume of 1,160,000 m^3. This available reservoir volume and the outlet capacity is sufficient to overcome the second storm. The reservoir elevation at the end of the second storm is -0.68 m (T= 72 hours).

Flow Scenario 2: The effect of different lag times between the storms on flooding volume in the Flood Plain is shown in Table 7.2.

Table 7.2 Volume of Flooding in the Flood Plain for Different Time Lags caused by the Second Storm

Flow Scenario	Starting Time of Second Flood hours	Volume of Flooding in the Flood Plain m^3
1	34	846,000
2 A	40	805,000
2 B	28	878,000
2 C	50	578,000

The above results show the role of time lag in flooding in the flood plain. Time lag would not influence flooding due to the second storm if the normal reservoir elevation (-1.90 m) had been achieved after the first storm. Using Inflow Hydrograph Flow Scenario 2 A, the reservoir elevation at the beginning of the second storm is +0.6 m, or in other words, the second storm begins with an available Reservoir storage Volume of 297,000 m^3 (Figure 7.9). Because of the limited Reservoir storage available, and small syphon capacity, flooding occurs in the Flood Plain. Using Improved conditions (Figure 7.10), no flooding occurs due to either storm events. The Reservoir elevation at the beginning of the second storm is -0.99 m, which is sufficient to overcome the second storm.

Using Inflow Hydrograph 2 B, there is almost no reservoir volume available for the second storm (Figure 7.11). Therefore excess water will exist in Opak Drain, which results in flooding upstream the Pluit Selatan Bridge. The Reservoir elevation is still high at the end of the second storm (at T=72 hours). Applying the improved conditions in the system to handle the Flow Scenario Hydrograph 2 B, results in no flooding (Figure 7.12). This means that the Improved Conditions are sufficient to handle Inflow Hydrograph Flow Scenario 2 B. The reservoir elevation at the beginning of the second storm is -0.12 m and at the T = 72 hours is -0.53 m.

Using Inflow Hydrograph 2 C, a longer time is available to reduce the water in the reservoir after the first storm. This results in a lower reservoir elevation for the second storm (Figure 7.13). The reservoir elevation at the end of the first storm is -0.14 m or the volume available for the second storm is 893,000 m^3, and in other words therefore more water could be diverted to the Reservoir. As a result, no additional flooding occurs in the flood plain due to the second storm. The reservoir volume available at the end of the second storm (T=72 hours) is 48,000 m^3. Using the improved conditions, no flooding occurs for both storms (Figure 7.14). The reservoir reaches the normal level (-1.90 m) at the beginning of the second storm. Therefore a large volume of storage is available during the second storm.

Flow Scenario 3: The results of using the Inflow Hydrograph shown in Figure 7.5, and the current Pluit System, the result is shown in Figure 7.15. The peak elevation of flooding in the Flood Plain is +1.6 m, which corresponds to volume of flooding about 1,288,000 m^3. Although the first storm does not result in any flooding, the resulting volume of run off is stored in the Reservoir. Hence, the Reservoir elevation at the beginning of the second storm is -0.25 m, which is higher than the normal level (-1.90 m). The corresponding available reservoir volume at the beginning of the second storm is, thus, reduced from normal levels and the reservoir elevation is still high when the second storm starts. The resulting loss of head across the syphon reduces flow through the syphon and causes an increase in the water level in the Opak Drain. Hence, flooding upstream increases. Using improved conditions, no flooding occurs as a result of either storm (Figure 7.16) because the Reservoir elevation after the first storm is low (-1.73 m), or almost reaching the normal level of -1.90 m. The Reservoir elevation at the end of the second storm, T=72 hours, is -0.38 m, which corresponds to a reservoir volume of 101,000 m^3.

Flow Scenario 4: The result of this combination (Figure 7.17) shows that the increase in flooding in the flood plain occurs because the reservoir volume is reduced by the occurrence of the first storm. When the second storm occurs, with a higher peak run off, the water elevation in the reservoir is high. The syphon capacity is reduced, and flooding occurs upstream of the Pluit Selatan Bridge. Using improved conditions (Figure 7.18), the reservoir elevation at the beginning of the second storm is - 1.7 m, which corresponds to an available reservoir volume of 1,960,000 m^3. This available reservoir volume and the outlet capacity is sufficient to overcome the second storm. The reservoir volume available at the end of the second storm is 931,000 m^3 and is available if another storm occurs.

7.2 CLOGGING OF FOREBAY PUMPS

The possibility of pump clogging due to garbage is investigated in this section. This possibility is simulated by assuming that one pump is clogged. Under this assumption Inflow Hydrograph 1 was routed through the system. The results, given in Figure 7.19, show that the flooding in the Flood Plain due to pump clogging increased to elevation + 1.25 with a the volume of flood in the flood plain of 940,000 m^3

(or an increase of 142,000 m^3 compared to current conditions). The reservoir is full at the end of the storm.

Using Inflow Hydrograph 2, the flooding in the Flood Plain increases to 1,192,000 m^3 and the available Reservoir elevation at the end of the storm (T = 37.9 hours) is + 0.96 m (Figure 7.20). This means that if one pump clogged, the flooding at the Flood Plain upstream Pluit Selatan bridge would increase by about 13,000 m^3 and no reservoir volume would be available at the end of storm.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

1. The previous studies (NEDECO, 1973 and Physical Model Investigation, Institute of Hydraulic Engineering, 1983) investigating the operation of the Pluit Polder System used a single storm to represent the design conditions. The results obtained for the Pluit Polder System simulation study demonstrate that the design should be based on a multiple flood hydrograph, which is a critical issue in a tropical climate characterized by a pronounced rainy season. Since a key element of the Polder system is the use of detention storage, not only the peak inflow, but also volume of flow influences the entire system. The three days of actual streamflow data show that the occurrence of successive storms actually occurs in this area.

2. Flooding in Flood Plain upstream of Pluit Selatan Bridge occurs because:
   - the Opak Drain is not large enough. The Opak Drain capacity is not sufficient to carry the inflow based on a calculated flood hydrograph for a 25

year return period. Additional reduction in channel capacity occurs due to the presence of garbage.

- The current outflow capacity determined by the forebay pump capacity and the Pluit Reservoir capacity is too small to prevent flooding. The current pump capacity is 16 m^3/s and the Pluit reservoir capacity is 2.6 million m^3.

3. The presence of garbage in the Opak Drain creates a major problem. The growth of aquatic plants is promoted by the presence of garbage in the channel. The aquatic plants increase the roughness of the channel and, therefore, decrease its capacity. The garbage can also reduces the capacity of the hydraulic structures and the capacity of the pumps, severely enough that the garbage may plug the syphon and the pumps, possibly actually plugging the pumps. In this study the reduction of the capacities due to the presence of garbage in Opak Drain is simulated by reducing capacity of the channel by 10% and 25%.

4. Concurrent improvement of the inlet and outlet systems should be undertaken to overcome flooding problems in this area. Improvements to only one part of the system gives limited benefits:

- Improvements to the inlet System only, would overcome the insufficient capacity to carry inflow from upstream. However, the limited outlet capacity would result in excess water in the Opak Drain. As a result, flooding in the flood plain occurs.
- Improvements to the outlet system only, would still result in flooding in the flood plain upstream, since the capacity of the Opak Drain is not sufficient to carry the 25 year return period flood.

Improvement Alternatives of the Inlet System include:

- Increase the capacity of the Opak Drain by increasing the bank elevation to 1.40 m by dyking.
- Regular cleaning of the Opak Drain.
- Increase the syphon capacity by increasing syphon length from 13 @ 4.00m (52.00 m) to 14 @ 4.00 m.

The Alternative improvements of the Outlet system include:

- improve the forebay pump capacity from 16 m^3/s to 20 m^3/s or 24 m^3/s.
- Introduce automatic forebay outlet Gates. 2 gates, each with 2 m width and 3 gates, each with 2.0 m width are considered in this study.

- improve the reservoir outlet gates from 3 gates, each with 2.0 m width to 4 gates of 2.0 m width.

5. The choice of which alternative improvements are the most appropriate depends on the results of a cost-benefit study. A cost benefit study was not carried out in this study. The cost of improvements could be calculated based on the results of this study on the Alternative Improvements to the Pluit Polder System structures. The environmental and social impact caused by flooding should also be considered in the cost calculation. The benefits could be calculated based on the impact of the volume of flood reduction (or stages of flood reduction) in the flood plain on flood damage.

6. The introduction of a waste collector screen at the Pluit Selatan Bridge, as recommended by the physical model test, to reduce the garbage entering the Pluit system would only be useful, <u>if</u> the screen is cleaned regularly. Otherwise the waste screen would restrict flow in the Opak Drain channel at Pluit Selatan Bridge and increase flooding upstream in the Flood Plain.

7. The tidal elevation plays an important role if forebay outlet gates are considered as one of the Improvements to the outlet structure. Further investigations are needed

to obtain more data on tidal elevations for longer periods of record.

8.2 RECOMMENDATIONS

1. Further investigations of two or more successive storms are essential. Multiple storms, occurring in a short periods of time, can greatly increase the resulting flood durations and volumes. The flood volume is critical because once the polder has been filled, this system has very little capacity left to pass another flood.

2. Further investigations (data collection) of the amount of garbage in the Opak Drain should be carried out to confirm the assumptions made in this study. The occurrence of clogging in the hydraulic structures and the pumps due to garbage should also be investigated.

3. Further investigations (data collection) of tidal elevations are essential. This is important in determining the forebay outlet gates capacity, which depends on the gross head between forebay elevation and tidal elevation.

4. Sediment was not considered in this study, since it is assumed that the source of flow is mostly surface runoff from residential and industrial areas where a significant

volume of sediment is not expected to be picked up. Therefore, the amount of sediment was assumed to be insignificant. However, sedimentation should be considered in the further investigations to confirm this assumption. Data collection on the sediment size, volume of sediment and the sediment characteristics should be undertaken.

5. Regular cleaning in Opak Drain and Ring Canal is essential to increase the capacity of the channel and therefore the efficiency of Pluit Polder System. The presence of garbage in the Opak Drain reduces the capacity of the channels. Aquatic plants growth promoted by developed from the garbage increases the roughness of the channel and therefore decreases the capacity of the channel. The garbage also reduces the capacity of the hydraulic structures to the point where garbage may even plug in the structures.

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APPENDIX A

FLOOD HYDROGRAPH ANALYSIS USING SCS METHOD

The data used for calculation of flood hydrograph in Pluit Polder is as follows:

Catchment Area	-	25.45 km^2
H	-	3.96 m
L	-	8 km

Rainfall distribution

The rainfall distribution and total rainfall of 24 hour rainfall for 25 year return period (Nedeco, 1973):

Time (hours)	Rainfall distribution (%)	Total Rainfall (mm)
0.0 - 1.0	4.67	91
1.0 - 2.0	47.20	101
2.0 - 3.0	59.35	127
3.0 - 4.0	67.76	145
4.0 - 5.0	73.83	158
5.0 - 6.0	77.10	165
6.0 - 12.0	87.85	188
12.0 - 24.0	100.00	214

The calculation of direct runoff and plotting data for return period 2 year and 25 year are tabulated in Tables A.2 and A.3. The calculation follows the following procedures:

1. Estimation of the Time of Concentration

$$Tc = (0.869\ L^3/\ H)^{0.385} \qquad \text{(A-1)}$$

$$= 6.16 \text{ hours}$$

2. Determining the Runoff Curve Number (Refer to table 1): The runoff Curve Number depends on soil condition and land use. In this calculation, it is assumed that the soil condition is type C and the land used 75% of residential areas and 25% industrial district. From table 1, the runoff Curve Number for such condition is 90.

3. Determining runoff data from rainfall data (Refer to Figure 4.4):

$$Q = (P - 0.2\ S)^2/(P + 0.8\ S) \qquad \text{(A-2)}$$

$$S = 2540/CN - 25.4 \qquad \text{(A-3)}$$

where:

Q = runoff in cm

P = rainfall in cm

4. Calculation the Time to Peak and Time Base

$$Tp = D/2 + \text{Lag Time} \qquad \text{(A-4)}$$

$$\text{Lag Time} = 0.6\ Tc \qquad \text{(A-5)}$$

$$Tb = 2.67\ Tp \qquad \text{(A-6)}$$

where:

Tp = Time to hydrograph peak in hours

D = Time interval

5. Calculation of unit Peak Discharge

$$qp = 2.081\ CA/Tp \qquad \text{(A-7)}$$

where:

qp = unit peak discharge in m^3/s(/mm effective rainfall)

CA = catchment area in km^2

Tp = Time to hydrograph peak in hours

TABLE A.1

Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Land Use (Antecedent moisture condition II; $I_a = 0.2S$)

LAND USE DESCRIPTION		HYDROLOGIC SOIL GROUP A	B	C	D
Cultivated land[1]					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow					
Good condition		30	58	71	78
Wood or forest land					
Thin stand, poor cover, no mulch		45	66	77	83
Good cover[2]		25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.					
Good condition: grass cover on 75% or more of the area		39	61	74	80
Fair condition: grass cover on 50–75% of the area		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential[3]					
Average lot size	Average % Impervious[4]				
1/8 ac or less	65	77	85	90	92
1/4 ac	38	61	75	83	87
1/3 ac	30	57	72	81	86
1/2 ac	25	54	70	80	85
1 ac	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers[5]		98	98	98	98
Gravel		76	85	89	91
Dirt		72	82	87	89

1. For a more detailed description of agricultural land use curve numbers, refer to *National Engineering Handbook*, Section 4, "Hydrology," Chapter 9, Aug. 1972.

2. Good cover is protected from grazing and litter and brush cover soil.

3. Curve numbers are computed assuming that the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltration could occur.

4. The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

5. In some warmer climates of the country a curve number of 95 may be used.

SOURCE : PHILIP B. BEDIENT and WAYNE C. HUBER, "HYDROLOGY AND FLOODPLAIN ANALYSIS"

TABLE A.2

COMPUTATION OF DIRECT RUNOFF AND PLOTTING DATA
FOR RETURN PERIOD 2 YEAR

TIME	RAINFALL DISTRI - BUTION	TOTAL RAIN- FALL	INCRE- MENTAL RAINFALL	RUNOFF ACCUMU- LATIVE	RUNOFF INCRE MENT	INCRE- MENTAL LOSS	qp for 1 mm	qp for INCREMENTAL RUNOFF	INCREMENTAL HYDROGRAPH Begin	INCREMENTAL HYDROGRAPH Peak	INCREMENTAL HYDROGRAPH End
hours	%	mm	mm	mm	mm	mm		m3/sec			
0.0-1.0	54.33	69	69	4.384	4.384	2.516	12.640	55.415	0	4.20	11.20
1.0-2.0	65.35	83	14	5.669	1.285	0.115	12.640	16.239	1	5.20	12.20
2.0-3.0	71.65	91	8	6.416	0.747	0.053	12.640	9.441	2	6.20	13.20
3.0-4.0	75.59	96	5	6.886	0.470	0.030	12.640	5.947	3	7.20	14.20
4.0-5.0	79.53	101	5	7.359	0.473	0.027	12.640	5.977	4	8.20	15.20
5.0-6.0	82.68	105	4	7.739	0.380	0.020	12.640	4.801	5	9.20	16.20
6.0-12.0	90.55	115	10	8.693	0.955	0.045	7.900	7.542	6	12.70	23.90
12.0-24.0	100.00	127	12	9.847	1.154	0.046	5.310	6.126	12	22.00	37.90

TABLE A.3

COMPUTATION OF DIRECT RUNOFF AND PLOTTING DATA
FOR RETURN PERIOD 25 YEAR

TIME hours	RAINFALL DISTRI - BUTION %	TOTAL RAIN-FALL mm	INCRE-MENTAL RAINFALL mm	RUNOFF ACCUMU-LATIVE mm	RUNOFF INCRE MENT mm	INCRE-MENTAL LOSS mm	qp for 1 mm	qp for INCREMENTAL RUNOFF m3/sec	INCREMENTAL HYDROGRAPH Begin	INCREMENTAL HYDROGRAPH Peak	INCREMENTAL HYDROGRAPH End
0.0-1.0	ˉ4.67	91	91	6.416	6.416	2.684	12.640	81.095	0	4.20	11.20
1.0-2.0	47.20	101	26	8.885	2.469	0.131	12.640	31.214	1	5.20	12.20
2.0-3.0	59.35	127	18	10.620	1.735	0.065	12.640	21.929	2	6.20	13.20
3.0-4.0	67.76	145	13	11.882	1.262	0.038	12.640	15.945	3	7.20	14.20
4.0-5.0	73.83	158	10	12.856	0.974	0.026	12.640	12.313	4	8.20	15.20
5.0-6.0	77.10	165	7	13.539	0.684	0.016	12.640	8.640	5	9.20	16.20
6.0-12.0	87.85	188	23	15.793	2.254	0.046	7.900	17.804	6	12.70	23.90
12.0-24.0	100.00	214	26	18.351	2.558	0.042	5.310	13.585	12	22.00	37.90

APPENDIX B

COMPUTER LISTING

```
C        LIST OF VARIABLE WHICH IS USED IN THIS PROGRAM

C        PROGRAM : PLUIT SYSTEM

C        MAIN CHANNEL ( OPAK DRAIN)

C        DEBUG       : INDICATES DEBUGGING LEVEL

C        MAIN CHANNEL UPSTREAM PLUIT BRIDGE

C        AU(J)       : CHANNEL CROSS SECTION AREA AT TIME (J)
C        ELU(J)      : WATER ELEVATION IN THE CHANNEL AT TIME (J)
C        ELUF(J)     : FLOOD ELEVATION AT TIME (J)
C        ELUO(J)     : FLOOD ELEVATION AT TIME (J) DUE TO OVERFLOW
C        ELUMAX      : MAXIMUM WATER ELEVATION IN THE CHANNEL
C        ELOMAX      : MAXIMUM WATER ELEVATION OF OVERFLOW
C        HEU(J)      : ENERGY LEVEL AT TIME (J)
C        HU(J)       : WATER DEPTH IN THE CHANNEL AT TIME (J)
C        QRD(J)      : TOTAL INLET CAPACITY (SYPHON AND RING CANAL)
C                      AT TIME (J)
C        QU(J)       : CHANNEL DISCHARGE AT TIME (J)
C        VU(J)       : VELOCITY IN THE CHANNEL AT TIME (J)
C        VFUF(J)     : VOLUME OF FLOOD AT TIME (J)
C        VFUO(J)     : VOLUME OF FLOOD AT TIME (J) DUE TO OVERFLOW
C        VFUOM       : MAXIMUM FLOOD ELEVATION DUE TO OVERFLOW
C        ZU          : CHANNEL BED ELEVATION
C        ZF          : FLOOD ELEVATION AT THE BEGINNING OF FLOOD
C

C        MAIN CHANNEL AT THE LOCATION OF THE PLUIT BRIDGE

C        A11(J)      : CROSS SECTIONAL AREA AT TIME (J)
C        ALP1        : COEFFICIENT INCLUDE LOSSES DUE TO CONTRACTION
C        DHE1(J)     : DIFFERENCE IN ENERGY HEAD AT TIME (J)
C                      AT UPSTREAM AND AT THE BRIDGE (HEU(J)-HE1(J))
C        DHE2(J)     : DIFFERENCE IN ENERGY HEAD AT TIME (J)
C                      AT UPSTREAM AND AT THE BRIDGE (HE1(J)-HEU(J))
C        EL1(J)      : WATER ELEVATION IN THE CHANNEL AT TIME (J)
C        H1(J)       : WATER DEPTH IN THE CHANNEL AT TIME (J)
C        H1C(J)      : CRITICAL WATER DEPTH
C        HE1         : ENERGY LEVEL IN THE CHANNEL AT TIME (J)
C        Q1B         : DISCHARGE PER M WIDTH
C        V1(J)       : VELOCITY AT TIME (J)
C        W1          : CHANNEL WIDTH
C        Z1          : BED ELEVATION

C        MAIN CHANNEL AFTER BRIDGE

C        AC(J)       : CHANNEL CROSS SECTION AREA AT TIME(J)
C        AE          : INUNDATED FLOOD PLAIN AREA DUE TO OVERFLOW
C        AF          : INUNDATED FLOOD PLAIN AREA BETWEEN RESERVOIR AND
C                      BRIDGE
C        ALPHA1      : COEFFICIENT : 1/NC*(S^0.5)
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C        ALP2         : COEFFICIENT INCLUDE LOSSES DUE TO ENLARGEMENT
C        DHX          : DIFFERENCE IN THE WATER DEPTH BETWEEN
C                       ASSUMPTION AND AFTER CALCULATED AT CRITICAL
C                       CONDITION UPSTREAM
C        DHE3(J)      : DIFFERENCE IN ENERGY LEVEL AT TIME (J) AT
C                       THE BRIDGE AND AT THE CHANNEL (C)
C        DQC(J)       : DIFFERENCES BETWEEN QCMAX AND QC AT TIME (J)
C        DT           : INTERVAL TIME
C        ELC(J)       : WATER ELEVATION IN THE CHANNEL AT TIME(J)
C        ELCMAX       : MAXIMUM WATER LEVEL IN THE CHANNEL
C        EM           : SIDE SLOPE OF THE CHANNEL
C        ELO(J)       : ELEVATION OF FLOOD DUE TO OVERFLOW AT TIME (J)
C        ELF(J)       : ELEVATION OF FLOOD AT TIME (J)
C        HC(J)        : WATER DEPTH IN THE CHANNEL AT TIME(J)
C        HCX(J)       : WATER DEPTH IN THE CHANNEL AT TIME (J)
C                       WHEN CRITICAL FLOW AT UPSTREAM OCCUR
C        HEC1(J)      : ENERGY HEAD AT TIME (J)
C        LC           : LENGTH OF MAIN CHANNEL
C        M            : COEFFICIENT (4/3)
C        NC           : MANNING COEFFICIENT
C        NT           : NUMBER OF TIME
C        QC(J)        : CHANNEL DISCHARGE AT TIME(J)
C        QCMAX        : MAXIMUM DISCHARGE IN THE CHANNEL
C        PC(J)        : WETTED PERIMETER AT TIME (J)
C        SC           : CHANNEL SLOPE
C        T            : TOTAL TIME
C        VC(J)        : VELOCITY IN THE CHANNEL AT TIME (J)
C        VFC(J)       : VOLUME OF WATER IN THE CHANNEL AT TIME (J)
C        VFF(J)       : VOLUME OF FLOOD AT TIME (J)
C        WC           : CHANNEL WIDTH
C        ZC           : CHANNEL BED ELEVATION

C        SYPHON

C        AS(J)        : CROSS SECTIONAL AREA OF THE THROAT AT TIME (J)
C        C            : DISCHARGE COEFFICIENT IF SYPHON ACT AS WEIR
C        DH1(J)       : DIFFERENCES IN WATER LEVEL UPSTREAM AND
C                       DOWNSTREAM THE SYPHON  AT TIME(J)
C        DH1MIN       : MINIMUM DIFFERENCES IN WATER LEVEL UPSTREAM
C                       AND DOWNSTREAM THE SYPHON
C        ELSMAX       : UPPER BEND ELEVATION OF SYPHON
C        ELSMIN       : CREST ELEVATION OF SYPHON
C        ELSU         : UPSTREAM TOP OF SYPHON ENTRANCE
C        HS(J)        : DEPTH OF WATER IN THE THROAT AT TIME (J)
C        U1           : SYPHON COEFFICIENT DUE TO LOSSES
C        LS           : SYPHON LENGTH
C        QS(J)        : DISCHARGE THROUH SYPHON AT TIME(J)
C        QSMAX        : MAXIMUM DISCHARGE THROUGH SYPHON
C        ZS           : BED ELEVATION UPSTREAM THE SYPHON


C        RING CANAL

C        NOTE: ALL VARIABLES THAT HAVE ARGUMENTS (I,J) HAVE BEEN SET  C
               UP AS:
C              - PREVIOUS TIME INTERVAL - J=1
C              - CURRENT TIME INTERVAL  - J=2
C              - NEXT TIME INTERVAL     - J=3
C        I            : LOCATION OF EACH PARTIAL LENGTH IN THE CANAL
C                       FROM UPSTREAM
C        ARG(J)       : OPENING GATE AREA
C        AR(I,J)      : RING CANAL CROSS SECTION AREA AT LOCATION (I)
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C        ARS(I)       : RING CANAL CROSS SECTION AREA FOR FIRST TIME  C
                        INTERVAL
C        ALPHA        : COEFFICIENT : 1/NR*(S^0.5)
C        DH2(I,J)     : DIFFERENCES IN WATER LEVEL UPSTREAM AND
C                       DOWNSTREAM THE GATES AT TIME(J)
C        DHF(I,J)     : DIFFERENCES IN ENERGY SLOPE (BACKWATER CALC)
C        ELR(I,J)     : ELEVATION OF WATER INTHE RING CANAL AT
C                       LOCATION(I) AT TIME(J)
C        ELRS(I)      : ELEVATION AT FIRST TIME INTERVAL AT
C                       LOCATION I
C        ELRSF        : ELEVATION AT FIRST TIME INTERVAL AT
C                       LOCATION 1
C        HF(I,J)      : ELEVATION AT TOTAL HEAD (EL+V^2/2g)
C        HF1(I,J)     : ELEVATION AT TOTAL HEAD BASE ON SFAV
C                       (BACK WATER CALC)
C        HR(I,J)      : WATER DEPTH IN RING CANAL
C        HRS(I)       : WATER DEPTH IN RING CANAL IN FIRST TIME
C                       INTERVAL
C        LR           : LENGTH OF RING CANAL
C        MR           : SIDE SLOPE OF THE CANAL
C        NR           : MANNING COEFFICIENT
C        PR(I,J)      : PERIMETER
C        QR(I,J)      : DISCHARGE IN THE RING CANAL AT LOCATION(I),
C                       AT TIME(J)
C        QRSP(I)      : DISCHARGE AT TIME INTERVAL 1
C        QRMAX        : MAXIMUM RING CANAL CAPACITY
C        RR(I,J)      : RADIUS HYDRAULIC
C        SF(I,J)      : ENERGY SLOPE
C        SFAV(I,J)    : AVERAGE ENERGY SLOPE (BACK WATER CALC)
C        SR           : RING CANAL SLOPE
C        U2           : DISCHARGE COEFFICIENT THROUGH RING CANAL GATE
C        VR(I,J)      : VELOCITY AT LOCATION (I) , TIME (J)
C        WR           : BOTTOM WIDTH OF RING CANAL
C        ZR           : BOTTOM ELEVATION OF RING CANAL

C        RESERVOIR

C        A2           : RESERVOIR AREA
C        AG           : AREA OF RESERVOIR OUTLET GATES OPENING
C        DHG(J)       : DIFFERENCE IN WATER LEVEL UPSTREAM AND
C                       DOWNSTREAM OULET GATES AT TIME (J)
C        VOL(J)       : VOLUME OF FLOOD IF  EL2 ABOVE THE MAXIMUM
C                       RESERVOIR ELEVATION
C        EL2(J)       : ELEVATION OF WATER IN THE RESERVOIR AT TIME(J)
C        EL2MAX       : MAXIMUM ALLOWABLE ELEVATION IN THE RESERVOIR
C        EL2TOP       : TOP ELEVATION OF THE RESERVOIR
C        H2(J)        : WATER DEPTH IN THE RESERVOIR AT TIME (J)
C        QG(J)        : DISCHARGE THROUGH GATES DOWNSTREAM RESERVOIR C
                        AT TIME(J)
C        UG           : DISCHARGE COEFFICIENT THROUGH THE OUTLET GATES
C        VOL(J)       : VOLUME OF FLOOD AT TIME (J) IF EL(J) IS ABOVE
C                       THE MAXIMUM RESERVOIR ELEVATION
C        WG           : WIDTH OF GATE
C        Z2           : BOTTOM ELEVATION OF RESERVOIR

C        FOREBAY AND PUMPING STATION

C        A3           : FOREBAY AREA
C        EL3(J)       : WATER ELEVATION AT FOREBAY UPSTREAM PUMPING
C                       STATION
C        H3(J)        : WATER DEPTH AT FOREBAY
C        EL3MIN       : MINIMUM WATER LEVEL AT FOREBAY
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C         QP(J)          : DISCHARGE THROUGH THE PUMP AT TIME(J)
C         Z3             : BOTTOM ELEVATION AT FOREBAY

        DIMENSION A11(600),AC(600),AGO(600), AR(20,3),ARG(600),
     $  ARS(20)
        REAL AS(600),AT(600),AT1(600),AU(600), A1,A2,A3,AG,AUO,ALP1,
     $  ALP2,ALPHA,ALPHA1,B,BETA,C,DAL(600),DH1(600),
     $  DH1MIN,DH2(600),DH2MIN,DHE1(600),DHE2(600),DHE3(600),
     $  DHE4(600),DHF(20,3),DHG(600),DHGO(600),DHX(600),DHX1(600),
     $  DQ1O(600),DQ1OA(600),DT,DXR, EL1(600),EL2(600),EL3(600),
     $  EL31(600),ELC(600),ELF(600),ELK(600),ELO(600),ELOMAX
        REAL ELR(20,3),ELU(600),ELU1W(600),ELUF(600),ELUO(600),
     $  ELT(600),ELX(600),EL2MAX,EL2MIN,EL2TOP,EL3MIN,ELCMAX,
     $  ELRS(20),ELRSF,ELSMIN,ELSMAX,ELSU,EM,EN,H1(600),H1C(600),
     $  HC(600),HCX(600),HE1(600),HEC1(600),HEU(600),HF(20,3),
     $  HF1(20,3),HR(20,3),HRS(20),HS(600),HU(600),LC,LR,LS,M,MR,NR,
     $  PC(600),PR(600),PU(600),QC(600)
        REAL QCB(600),QC(600),QCMAX,QG(600),QGO(600),QO1O(600),
     $  QP(600),QR(20,3),QRMAX,QRD(600),QRSP(20),QS(600),QT(600),
     $  QU(600),QUB(600),QUMAX,QU1W(600),Q1B(600),RR(20,3),
     $  SF(20,3),SFAV(20,3),T,TT(600),U1,U2,UG,V1(600),
     $  VC(600),VFC(600),VFF(600)
        REAL VFO(600),VFU(600),VFUF(600),VFUO(600),VFUT(600),
     $  VR(20,3),VTT(600),VU(600),WG,WGO,WR,YG(600),ZR(600),Z1,Z2,
     $  Z3,ZC,ZF,ZS


        INTEGER NXR,NT,DEBUG,J

        OPEN(UNIT=2,FILE='DATA.PRN')
        OPEN(UNIT=12,FILE='ANS.PRN',ACCESS='APPEND')
        OPEN(UNIT=14,FILE='VO.PRN',ACCESS='APPEND')
        OPEN(UNIT=15,FILE='HAS.PRN',ACCESS='APPEND')
C       INPUT FIXED DATA
        DEBUG    =  5

        A3       =   12795
        AE       = 1184000
        AF       =  178000
        AG       =      12.00
        ALP1     =       1.1
        ALP2     =       1.15
        ALPHA    =       0.67
        ALPHA1   =       0.4169
        AO       =  120000
        AUO      = 1155200
        BETA     =       0.40
        C        =       1.00
        DH1MIN   =       0.10
        DH2MIN   =       0.30
        ELCMAX   =       1.00
        ELCMIN   =      -0.10
        ELUMAX   =       1.00
        ELUOM    =       1.00
        ELOMAX   =       1.00
        EL2MAX   =       1.10
        EL2MIN   =      -1.90
        EL2TOP   =       1.00
        EL3MIN   =      -1.90
        ELF(2)   =       1.00
        ELR(1,2)=       -0.30
        ELR(1,1) =      -0.30
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```
      ELRS(1)  =      -0.30
      ELSMAX   =       1.00
      ELSMIN   =       0.10
      ELSU     =       0.25
      EM       =       1.50
      EN       =       2.00
      LC       =     500.00
      LS       =      52.00
      M        =       1.333
      MR       =       1
      NC       =       0.0275
      NR       =       0.024
      QCMAX    =     101.3
      QPMAX    =      16.00
      QUMAX    =     101.3
      QSMAX    =     115.00
      QRMAX    =      16.00
      SC       =       0.00072
      SR       =       0.00094
      U1       =       0.70
      U2       =       0.94
      UG       =       0.80
      VFCMAX   =   36000.00
      VFOMAX   =   60000.00
      VFUOM    =  577600.00
      WC       =      15.00
      WU       =      15.00
      W1       =      15.00
      WG       =       7.50
      WR       =       2.50
      ZC       =     - 2.00
      ZU       =     - 2.00
      ZO       =       0.50
      ZR(1)    =     - 2.30
      ZS       =     - 2.00
      Z1       =     - 2.00
      Z2       =     - 4.40
      Z3       =     - 4.00
      ZF       =     + 1.00

C       INPUT DATA
C       WRITE (*,*)'INPUT INITIAL EL2'
C       READ(5,*)EL2(1)
        EL2(1)= -1.90
        EL2(2) = EL2(1)
C       WRITE(*,*)'INPUT INITIAL EL3'
C       READ(5,*)EL3(1)
        EL3(1) = -1.90
        EL3(2) = EL3(1)

C       INPUT TIMING PARAMETERS
C10     WRITE(*,*)'TOTAL TIME FOR SIMULATION'
C       READ(5,*)T
         T=89700

C       WRITE(*,*)'ENTER TIME INCREMENT (SECOND)'
C       READ(5,*)DT
           DT = 150

           NT = INT(T/DT)
```

```
      DO 14 I = 1,20
         QRSP(I) = -1
 14   CONTINUE

C     INPUT DISTANCE PARAMETERS
C     WRITE(*,*)'ENTER TOTAL LENGTH OF RING CANAL'
C     READ(5,*)LR
C     WRITE(*,*)'ENTER DISTANCE INCREMENT'
C     READ(5,*)DXR
      LR=1700.0
      DXR=100.0
      NXR=INT(LR/DXR)
      ZR(NXR+1) = -3.90

C     INPUT UPSTREAM DATA
      NK=NT+2
C     WRITE(*,*)NK
      DO 20 J =2,NK
C     WRITE(*,*)'ENTER DISCHARGE AND TIDAL FOR PERIOD J'
      READ(2,*)QT(J-1),DAL(J-1),QU (J-1)

      CALL EL110 (J,AU,QU,ALPHA1,HU,WU,ELU,ZU,PU,QO1O,DQ1O,
     $ DQ1OA,HEU,EN)

 20   CONTINUE
      NL = NT+1
C     WRITE(*,*)'NL = ',NL

      DO 500 J =2,NL

C     INITIAL CONDITION

       VFUO(2)= 0.00
       ELUO(2)= 0.50
       VFUF(2)= 0.00
       ELUF(2)= 1.00
       VFF(2)  = 0.00
       VFO(2)  = 0.00
       ELU(1)  = 0.00
       ELC(1)  = ELU(1)
       QCB(1)  = 0.00
       QR(1,1)= 0.0
       QS(1)   = 0.0
       QU1W(J)=QU(J)
       ELU1W(J)=ELU(J)
       QRD(1)= 0.0

C     WRITE(*,*)'TIME  VFUF BEGIN',J,VFUF(J)

      IF(VFUF(J).GT.0.001) THEN
          ELU(J)=ELUF(J)+(((QU(J)-QRD(J-1))*DT)/AE)
          ELU(J)=(ELU(J)+ELU(J-1))/2
C         WRITE(*,*)'ELU AFTER ADJUST',ELU(J)

          CALL EL1UA (J,HU,ELU,ZU,AU,WU,PU,QU,EN)

          ELK(J)=ELU(J)
          VFUF(J+1)=VFUF(J)+(QU1W(J)*DT)
          ELUF(J+1)=ZF+(VFUF(J+1)/AE)
          VFUO(J+1)=VFUOM
          ELUO(J+1)=ELUOM
```

```
      ELSE
         ELU(1)= 0.00
         ELU(J)=ELU(J-1)+(((QU(J)-QRD(J-1))*DT)/(LC*25))
         ELU(J)=(ELU(J)+ELU(J-1))/2
         ELK(J)=ELU(J)

       CALL EL1UA (J,HU,ELU,ZU,AU,WU,PU,QU,EN)

        IF(QU(J).GT.QUMAX) THEN
           VFUO(J+1)=VFUO(J)+((QU(J)-QUMAX)*DT)
           ELUO(J+1)=0.50+(VFUO(J+1)/AUO)
           ELU(J)=ELUMAX
           QU(J)=QUMAX

           IF(VFUO(J+1).GT.VFUOM ) THEN
C             WRITE(*,*)'ELUO . GT . ELUO MAX FLOOD'
              VFUF(J+1)=VFUO(J+1)-VFUOM
              ELUF(J+1)=ZF + (VFUF(J+1)/AE)
              VFUO(J+1)=VFUOM
              ELUO(J+1)=ELOMAX
              ELU(J)=ELUF(J+1)

           CALL EL1UA (J,HU,ELU,ZU,AU,WU,PU,QU,EN)

           ELSE
C             WRITE(*,*)' ELUO . LT. ELOMAX    OVERBANK'
              VFUO(J+1)=VFUO(J+1)
              ELUO(J+1)=ZO + VFUO(J+1)/AUO
              VFUF(J+1)=0.00
              ELUF(J+1)=ZF
              QU(J)=QUMAX
              ELU(J)=ELUMAX
C             WRITE(*,*)'VFO.LT.VFOMAX  QU1W  QU   VFUO   VFUF'
           END IF
        ELSE
            VFUO(J+1)=VFUO(J)
            ELUO(J+1)=ZO + VFUO(J+1)/AUO
            VFUF(J+1)=0.00
            ELUF(J+1) = ZF
            QU(J)=QU(J)
            ELU(J)=ELU(J)
        END IF
      END IF

C     WRITE (*,*)'ELU AFTER ALL ADJUST '

C      WRITE(*,*)'EXECUTE UPSTREAM QU  HU  ',QU(J),HU(J)

      CALL UPSTR (J,VU,QU,AU,HEU,HU,H1,EL1,Z1,A11,W1,QUB,H1C,
     $ ALP1,V1,DHE1,DHE2,HC,AC,VC,HEC1,ALP2,HE1,HCX,DHX,DHX1,DHE3,
     $ DHE4,ZC,QC,ELC,WC)

       TT(J)=QT(J)/3600

       ELR(1,2) = -0.30
       ELR(1,1) = -0.30
       ELRSF    = -0.30
       QR(1,1) = QRMAX
       NL=NT+1
```

```
      IF(EL3(J).GT.EL3MIN) THEN
        QP(J) = QPMAX
      ELSE
        QP(J) = 0.00
      END IF

C     CALCULATION OF FOREBAY OUTLET

       WGO = 6.00
       ELG = -0.50
      IF(EL3(J).GT.DAL(J)) THEN
        YG(J) = DAL(J)-ELG
        DHGO(J)=EL3(J)-DAL(J)
        AGO(J) = WGO*YG(J)
        QGO(J) = U2*AGO(J)*(SQRT(2*9.81*DHGO(J)))
        QP(J) = QP(J)+QGO(J)
      ELSE
        QGO(J) = 0.00
        QP(J)  = QP(J)
      END IF

      IF(EL2(J).LT.0.00) THEN
         A2= 474339+((EL2(J)+2.00)*145082)
      ELSE
         A2= 597963.5+(77350*(EL2(J)+2.0))
      END IF

       IF(EL2(J).GT.EL3(J)) THEN
         DHG(J)= EL2(J)-EL3(J)
         QG(J) = UG*AG*SQRT(2*9.81*DHG(J))
C        WRITE(*.*) 'OPEN OUTLET GATES'
       ELSE
         QG(J) = 0.00
         WRITE (*.*) 'CLOSED OUTLET GATES '
       END IF

       VFC(2)=HC(2)*(WC+(EM*HC(2)))*LC

         IF(VFF(J).GT.0.001) THEN
           ELF(J)= ZF + VFF(J)/AF
           VFF(J+1)=VFF(J)+(QU1W(J)*DT)
           ELF(J+1)=ZF+(VFF(J+1)/AF)
           VFO(J+1)=VFOMAX
           VFC(J+1)=VFCMAX
           ELC(J)=ELC(J)
           CALL ELEV1A (HC,ELC,J,ZC,AC,PC,QC,WC,EM,EN)

      ELSE
        IF(QC(J).GT.QCMAX) THEN
            VFO(J+1)=VFO(J)+(QC(J)-QCMAX)*DT
            ELO(J+1)=ZO+(VFO(J+1)/AO)
            ELC(J)=ELCMAX
            QC(J)=QCMAX

            IF(VFO(J+1).GT.VFOMAX) THEN
              VFF(J+1)=VFO(J+1)-VFOMAX
              ELF(J+1)=ZF + (VFF(J+1)/AF)
              VFO(J+1)=VFOMAX
              ELO(J+1)=ELOMAX
              ELC(J)=ELF(J+1)
```

```
               CALL ELEV1A (HC,ELC,J,ZC,AC,PC,QC,WC,EM,EN)

               ELSE
C                WRITE(*,*)'ELO . LT. ELOMAX    OVERBANK'
                    VFO(J+1)=VFO(J+1)
                    ELO(J+1)=VFO(J+1)/AO
                    VFF(J+1)=0.00
                    ELF(J+1)=ZF
                    QC(J)=QCMAX
                    ELC(J)=ELCMAX
C                   WRITE(*,*)'VFO.LT.VFOMAX    QC   VFO   VFF '

               END IF
           ELSE
                 VFO(J+1)=VFO(J)
                 VFF(J+1)=0.00
                 ELF(J+1)=ZF
                 QC(J)=QC(J)
                 ELC(J)=ELC(J)

           END IF
         END IF

C        WRITE (*,*)'ELC AFTER ALL ADJUST '

C        COMPUTE CANAL ELEVATION

         IF(ELC(J).GT.ELSMIN) THEN
           IF (VFF(J).GT.0.01) THEN
              IF (ELC(J).GT.EL3(J)) THEN
                QR(1,2) = QRMAX
                ELR(1,2)= ELF(J)
              ELSE
                QR(1,2)=0.00
                ELR(1,2)=EL3(J)
              END IF

C              ROUTE FLOWS THROUGH SYPHON

         CALL SYPHON (J,EL2,ELC,QS,HS,AS,U1,C,LS,DH1,
      $              EL2MIN,ELCMAX,DH1MIN,ELSMIN,ELSU,ASMAX)

                QG(1)=0.0
                EL2(J+1)=EL2(J)+(QS(J)-QG(J))*DT/A2

                IF(EL2(J).GT.EL2MAX) THEN
                   WRITE(*.*)'   FAILURE  '
                   STOP
                ELSE
                   EL2(J)=EL2(J)
                END IF

           ELSE


             IF(ELC(J).GT.EL3(J))THEN
                 ELRMAX = -0.30
                 DH2(J) = ELC(J) - ELRMAX
               IF(DH2(J).GT.DH2MIN) THEN
                 HR(1,2) = ELC(J)-ZR(1)
                 ARG(J)= WR*HR(1,2)
                 QR(1,2) = U2*ARG(J)*(SQRT(2*9.81*DH2(J)))
```

```
                  IF (QR(1,2).GT.QRMAX) THEN
                    QR(1,2) = QRMAX
                  ELSE
                     QR(1,2) = QR(1,2)
                  END IF
             ELSE
                  QR(1,2) = 0.00
             END IF
            ELSE
                  QR(1,2)=0.0
            END IF

C    ROUTE FLOWS THROUGH SYPHON

               CALL SYPHON (J,EL2,ELC,QS,HS,AS,U1,C,LS,DH1,
     $                  EL2MIN,ELCMAX,DH1MIN,ELSMIN,ELSU,ASMAX)

               EL2(J+1) = EL2(J)+(QS(J)-QG(J))*DT/A2
               WRITE(*,*)'EL2 AFTER - QS -QG  EL2 , QS  QG '
               WRITE(*,*) EL2(J+1),QS(J),QG(J)
          END IF
        ELSE
               QS(J)=0.00
               WRITE(*.*)'NO FLOW THROUGH SYPHON'
         IF(ELC(J).GT.ELCMIN) THEN
           IF(ELC(J).GT.EL3(J))THEN
             ELRMAX=-0.30
             DH2(J) = ELC(J) - ELRMAX
             IF(DH2(J).GT.DH2MIN) THEN
                HR(1,2) = ELC(J)-ZR(1)
                ARG(J)= WR*HR(1,2)
                QR(1,2) = U2*ARG(J)*(SQRT(2*9.81*DH2(J)))
                WRITE(*,100)
 100            FORMAT(/,'0.00 < ELC < 0.10 AND DH2 > DH2 MIN')
                IF(QR(1,2).GT.QRMAX) THEN
                   QR(1,2)=QRMAX
                ELSE
                   QR(1,2)=QR(1,2)
                END IF
             ELSE
                QR(1,2) = 0.00
                 WRITE(*,110)
 110            FORMAT(/,'0.00 < ELC < 0.10 AND DH2 < DH2 MIN ')
             END IF
           ELSE
             QR(1,2) = 0.00
           END IF
         ELSE
             QR(1,2) = 0.00
             IF(EL3(J).LT.EL3MIN)THEN
                EL3(J)=EL3MIN
             ELSE
                EL3(J)=EL3(J)
             END IF

             WRITE(*,*)'CLOSED RING CANAL'
         END IF
        END IF

        IF (EL2(J+1).GT.EL2MAX) THEN
            WRITE (*,*)'FAILURE'
             STOP
```

```
      ELSE
           EL2(J+1)=EL2(J+1)
      END IF

C     CALCULATION OF RING CANAL

C     SET PREVIOUS TIME DISCHARGE FOR FIRST TIME INTERVAL TO
C     QR(2,1)
      IF (J.EQ.2) THEN
        CALL SETQR (QR)
      ENDIF

C     CHECK THE WATER LEVEL IN THE RING CANAL

      CALL RING (J,ML,NXR,AR,QR,ALPHA,M,EM,ELR,ZR,HR,WR,
     +  DXR,SR,ELRS,ELRSF,EL3,LR,HRS,ARS,MR,QRSP,QRMAX,DT,PR,
     +  TOTAL,CBAR,QG,QP,A3,VR,HF,HF1,SF,SFAV,RR,DHF)

        EL3(J+1)=EL3(J)+(QR(NXR+1,2)+QG(J)-QP(J))*DT/A3

        IF(EL3(J+1).GT.EL3MIN) THEN
          QP(J) = QPMAX
       ELSE
          QP(J) = 0.00
       END IF
       WGO = 6.00
       ELG = -0.50
       IF(EL3(J).GT.DAL(J)) THEN
         YG(J) = DAL(J)-ELG
         DHGO(J)=EL3(J)-DAL(J)
         AGO(J) = WGO*YG(J)
         QGO(J) = U2*AGO(J)*(SQRT(2*9.81*DHGO(J)))
         QP(J) = QP(J)+QGO(J)
      ELSE
         QGO(J) = 0.00
         QP(J)  = QP(J)
      END IF
      EL2(J+1)=EL2(J)+(QS(J)-QG(J))*DT/A2
      IF(EL2(J+1).LT.0.00) THEN
         A2= 474339+((EL2(J+1)+2.00)*145082)
      ELSE
         A2= 597963.5+(77350*(EL2(J+1)+2.0))
      END IF
      IF(EL2(J+1).GT.EL3(J+1)) THEN
        DHG(J+1)= EL2(J+1)-EL3(J+1)
        QG(J+1) = UG*AG*SQRT(2*9.81*DHG(J+1))
        QG(J)=(QG(J+1)+QG(J))/2
        WRITE(*,*)'OPEN OUTLET GATES')
      ELSE
        QG(J) = 0.00
        WRITE(*,*)'CLOSED OUTLET GATES'
      END IF
       VFC(2)=HC(2)*(WC+(EM*HC(2)))*500
          EL2(J+1)=EL2(J)+(QS(J)-QG(J))*DT/A2
          EL3(J+1)=EL3(J)+(QG(J)+QR(NXR+1,2)-QP(J))*DT/A3
C         WRITE(*,*)'EL3 AFTER',EL3(J+1)
          IF(EL3(J+1).LT.EL3MIN)EL3(J+1)=EL3MIN
          IF(VFF(J+1).GT.0.01)THEN
             VFF(J+1) = VFF(J+1)-((QS(J)+QR(1,2))*DT)
             ELF(J+1)=VFF(J+1)/AF
C             WRITE(*,*)'VFF ADJUST QS , AR  VFF QS QR'
C             WRITE(*,*)VFF(J+1),QS(J),QR(1,2)
```

```
            IF(VFF(J+1).GT.0.001)THEN
               VFF(J+1)=VFF(J+1)
            ELSE
               VFF(J+1)=0.00
            END IF
          ELSE
             VFF(J+1)=0.00
          END IF

       ELF(J+1)=(VFF(J+1)/AF) + ZF
       ELO(J+1)=(VFO(J+1)/AO) +ZO
       VTT(J+1)=VFF(J+1)+VFO(J+1)
       AT(J)=(DT*(J-1))/3600
C      WRITE(*,*)'VFUF BEFORE - QS',VFUF(J+1)
       IF(VFUF(J+1).GT.0.01) THEN
         VFUF(J+1) = VFUF(J+1)-((QS(J)+QR(1,2))*DT)
         ELUF(J+1) = VFUF(J+1)/AE
C        WRITE(*,*)'VFUF AFTER ADJUST',VFUF(J+1)
       ELSE
         VFUF(J+1)=0.00
         ELUF(J+1)=ZF
       END IF
         ELUF(J+1) = (VFUF(J+1)/AE) +ZF
         ELUO(J+1)=(VFUO(J+1)/AUO)+ZO
C         AT(J)=(DT*(J-1))/3600
         AT1(J)=QT(J)/3600
       WRITE(14,502)AT1(J),QU1W(J),QU(J),QG(J),QS(J),QR(1,2),QP(J)
  502  FORMAT(F6.2,6F7.2)
       WRITE(15,504)DAL(J),ELU1W(J)
  504  FORMAT(2F6.2)
       WRITE(*,*)'VFUF,VFUO ELK ELU  EL2  EL3  QS  QR '
       WRITE(*,*)VFUF(J+1),VFUO(J+1),ELK(J),ELU(J),EL2(J+1),EL3(J+1),
     $ QS(J),QR(1,2)
       ELK(1)=ELU(1)
       ELT(J+1)= 0.5+(VFUO(J+1)/AUO)+(VFUF(J+1)/AE)
       ELX(J)=(ELK(J)+ELK(J-1))/2
       EL31(2)=EL3(1)
       EL31(J+1)=(EL3(J+1)+EL31(J))/2
       EL2(J)=(EL2(J)+EL2(j+1))/2
       VFUT(J+1)=VFUO(J+1)+VFUF(J+1)
       WRITE(12,505)AT1(J),VFUT(J+1),ELT(J+1),
     $ ELX(J),EL2(J+1),EL2(J),EL31(J+1)
  505  FORMAT(F6.2,F11.1,5F7.3 )
       QRD(J)=QS(J)+QR(1,2)
       QCB(J)=QU(J)-QS(J)-QR(1,2)

C    COPY CURRENT TIME INTERVAL RING CANAL VARIABLES TO CURRENT
         ELR(1,2) = ELR(1,3)
       DO 520 I = 1,ML
         AR(I,1) = AR(I,2)
         QR(I,1) = QR(I,2)
         ELR(I,1) = ELR(I,2)
 520   CONTINUE
 500   CONTINUE
       STOP
       END
```

```
      SUBROUTINE UPSTR(J,VU,QU,AU,HEU,HU,H1,EL1,Z1,A11,W1,
     $ Q1B,H1C,ALP1,V1,DHE1,DHE2,HC,AC,VC,HEC1,ALP2,HE1,
     $ HCX,DHX,DHX1,DHE3,DHE4,ZC,QC,ELC,WC)
      DIMENSION VU(600),QU(600),AU(600),HEU(600),HU(600),H1(600)
      REAL EL1(600),Z1,A11(600),W1,Q1B(600),H1C(600),ALP1,
     $ HE1(600),V1(600),DHE1(600),DHE2(600),HC(600),AC(600),
     $ VC(600),HEC1(600),ALP2,HCX(600),DHX(600),DHX1(600),
     $ DHE3(600),DHE4(600),ZC,QC(600),WC,ELC(600)

      VU(J)=QU(J)/AU(J)
      HEU(J)= HU(J)+((VU(J)**2)/(2*9.81))
      H1(J)=HU(J)-0.2
      EL1(J)=Z1+H1(J)
 313  A11(J)=W1*H1(J)
      V1(J)=QU(J)/A11(J)
      Q1B(J)=QU(J)/W1
      H1C(J)=(((alp1/9.81)*(Q1B(J)**2))**(.34))
      HE1(J)=H1(J)+(ALP1*((V1(J)**2)/(2*9.81)))
      DHE1(J)=HEU(J)-HE1(J)
      IF (H1(J).LE.H1C(J))THEN
         H1(J)=H1C(J)
         V1(J)=QU(J)/A11(J)
         HE1(J)=H1(J)+(ALP1*((V1(J)**2)/(2*9.81)))
         WRITE(*,*)'CRITICAL OCCUR',H1(J),HE1(J)
      ELSE
        IF(DHE1(J).GT.0.01)THEN
          H1(J)=H1(J)+0.003
          EL1(J)=Z1+H1(J)
C         WRITE(*,*)'RE-CALCULATE H1   NEW H1 ',H1(J)
          GO TO 313
        ELSE
          DHE2(J)=HE1(J)-HEU(J)
          IF(DHE2(J).GT.0.01) THEN
            H1(J)=H1(J)-0.003
            EL1(J)=Z1+H1(J)
C           WRITE(*,*)'RECALCULATE H1 HE1.GT.HEU',H1(J)
            GO TO 313
          ELSE
            H1(J)=H1(J)
            EL1(J)=Z1+H1(J)

C           WRITE(*,*)'SATISFIED   H1   EL1 ',H1(J),EL1(J)
          END IF
        END IF
      END IF

         HC(J)=H1(J)+0.2
 314     AC(J)=(WC+2.0*HC(J))*HC(J)
         VC(J)=QU(J)/AC(J)
         HEC1(J)=HC(J)+ALP2*((VC(J)**2)/(2*9.81))
      IF (H1(J).LE.H1C(J))THEN
         HE1(J)=H1C(J)+(V1(J)**2/(2*9.81))
         HEC1(J)=HE1(J)-(ALP2-1)*((VC(J)**2)/(2*9.81))
         HCX(J)=HE1(J)-(ALP2*(VC(J)**2)/(2*9.81))
         DHX(J)=HC(J)-HCX(J)
         DHX1(J)=HCX(J)-HC(J)
         IF(DHX(J).GT.0.01)THEN
           HC(J)=HC(J)-0.003
           GO TO 314
         ELSE
```

```
              IF(DHX1(J).GT.0.01) THEN
                HC(J)=HC(J)+0.003
              GO TO 314
              ELSE
                HC(J)=HCX(J)
             END IF
           END IF

         ELSE
           DHE3(J)=HE1(J)-HEC1(J)
           IF(DHE3(J).GT.0.01)THEN
             HC(J)=HC(J)+0.003
C            WRITE(*,*)'RE-CALCULATE HC    ',HC(J)
             GO TO 314
           ELSE
             DHE4(J)=HEC1(J)-HE1(J)
             IF(DHE4(J).GT.0.01) THEN
               HC(J)=HC(J)-0.003
C              WRITE(*,*)'RE-CALCULATE HC  ',HC(J)
               GO TO 314
             ELSE
               HC(J)=HC(J)
               ELC(J)=ZC+HC(J)
               HEC1(J)=HEC1(J)
C              WRITE(*,*)'SATISFIED    QU  HC  AC  VC  HEC1'
C              WRITE(*,*) QU(J),HC(J),AC(J),VC(J),HEC1(J)
             END IF
           END IF
         END IF

          QC(J)=AC(J)*VC(J)

          IF (QC(J).GT.QU(J)) THEN
              QC(J)=QU(J)
          ELSE
              QC(J)=QC(J)
          END IF

          ELC(J)=ZC+HC(J)
          WRITE(*,*)'RESULT FLOW QU  HU  H1  HC  '
          WRITE(*,*)QU(J),HU(J),H1(J),HC(J),QC(J)
        RETURN
        END


      SUBROUTINE EL110 (J,AU,QU,ALPHA1,HU,WU,ELU,ZU,PU,
     $ QO1O,DQ1O,DQ1OA,HEU,EN)
      DIMENSION AU(600),QU(600)
      REAL  ALPHA1,HU(600),WU,EM,ELU(600),ZU,PU(600),QO1O(600),
     $ DQ1O(600),DQ1OA(600),VU(600),HEU(600),EN

      AU(J-1) = (QU(J-1)/ALPHA1)**(0.75)
      HU(J-1) = (-WU+SQRT((WU**2)+(4*EN*(AU(J-1)))))/(3.0)
 712  ELU(J-1) = ZU+HU(J-1)
C     WRITE(*,*)'EL110 FIRST EST HU ELU'
      PU(J-1 )= (WU+2*(SQRT((HU(J-1)**2)+(EN*HU(J-1))**2)))
      AU(J-1) = (WU+ EN*HU(J-1))*HU(J-1)
      QO1O(J-1)=0.975   *AU(J-1)*((AU(J-1)/PU(J-1))**(2.0/3.0))

C     WRITE(*,*)'CHECK   QO1O PU AU HU',QO1O(J-1),PU(J-1),
C     AU(J-1),HU(J-1)
```

```
      DQ1O(J-1)=QU(J-1)-QO1O(J-1)
      IF(DQ1O(J-1).GT.0.10) THEN
         HU(J-1)=HU(J-1)+0.003
         GO TO 712
      ELSE
         DQ1OA(J-1)=QO1O(J-1)-QU(J-1)
         IF(DQ1OA(J-1).GT.0.10) THEN
           HU(J-1)=HU(J-1)-0.002
           GO TO 712
         ELSE
           HU(J-1)=HU(J-1)
           ELU(J-1)=ELU(J-1)
         END IF
     END IF
        VU(J-1)=QU(J-1)/AU(J-1)
        HEU(J-1)=HU(J-1)+((VU(J-1)**2)/(2*9.81))
      RETURN
      END


      SUBROUTINE EL1UA (J,HU,ELU,ZU,AU,WU,PU,QU,EN)
      DIMENSION HU(600),ELU(600)
      REAL ZU,AU(600),WU,PU(600),QU(600),EN
      HU(J) = ELU(J)-ZU
      AU(J) = (WU+(EN*HU(J)))*HU(J)
      PU(J) = (WU+2*(SQRT((HU(J)**2)+(EN*HU(J))**2)))
      QU(J) = 0.975    * AU(J)* ((AU(J)/PU(J))**(0.6666))
      WRITE(*,*)'EL1UA HU,AU,PU,QU',HU(J),AU(J),PU(J),QU(J)
      RETURN
      END


     SUBROUTINE SYPHON (J,EL2,ELC,QS,HS,AS,U1,C,LS,DH1,
  $                 EL2MIN,ELCMAX,DH1MIN,ELSMIN,ELSU,ASMAX)
     DIMENSION EL2(600),ELC(600),QS(600)
     REAL HS(600),AS(600),U1,C,LS,DH1(600),EL2MIN,ELCMAX,
  $  DH1MIN,ELSMIN,ASMAX

     WRITE(*,*)'ENTERING  SYPHON'
       DH1(J)=ELC(J) - EL2(J)
         IF(DH1(J).GT.DH1MIN) THEN
         IF(EL2(J).GT.EL2MIN) THEN
           IF(ELC(J).GT.ELSU) THEN
            AS(J) = LS*(ELC(J)-ELSMIN)
            QS(J) = U1*AS(J)*SQRT(2*9.81*DH1(J))
            WRITE(*,*)'SYPHON FLOW'
           ELSE
            HS(J) = ELC(J)-ELSMIN
            QS(J) = C*LS*(HS(J)**1.5)
            WRITE(*,*)'FLOW AS WEIR'
           END IF
         ELSE
           HS(J) = ELC(J) - ELSMIN
           QS(J) = C*LS*(HS(J)**1.5)
           WRITE(*,*)'FLOW AS WEIR'
         END IF
      ELSE
           QS(J)=0.0
      END IF
C          WRITE(*,*)'RESULT SYPHON   QS  EL2   ELC'
    RETURN
    END
```

```
      SUBROUTINE RING (J,ML,NXR,AR,QR,ALPHA,M,EM,ELR,ZR,HR,
     $    WR,DXR,SR,ELRS,ELRSF,EL3,LR,HRS,ARS,MR,QRSP,QRMAX,DT,PR,
     $    TOTAL,CBAR,QG,QP,A3,VR,HF,HF1,SF,SFAV,RR,DHF)
      DIMENSION EL3(600),ELR(20,3),QP(600),QG(600),QR(20,3)
      REAL HR(20,3),MR,ZR(600),A3,AR(20,3),QRMAX,DT,WR,LR,DXR,
     $ ALPHA,M,EM
      REAL PR(20,3),R(20,3),RR(20,3),DHF(20,3),HF(20,3),
     $ HF1(20,3),SFAV(20,3),SF(20,3),VR(20,3)
      REAL ELRS(20),HRS(20),ARS(20),QRSP(20),ELRSF
      INTEGER NXR
C     WRITE(6,137)
C137       FORMAT(' DISTANCE   TIME     HR         AR        QR')
C          WRITE (*,*) 'ENTERING RING'
           ML = NXR+1
C          WRITE(*,*) ML
           DO 150 I=2,ML
               AR(1,2)=(QR(1,2)/ALPHA)**(1.0/M)
               HR(1,2)=(-WR+SQRT(WR**2+4*EM*AR(1,2)))/2*EM

               ELR(1,2)=ZR(1)+HR(1,2)
               ZR(I) = ZR(I-1)-(DXR*SR)
C              WRITE(*,*)'ZR = ',ZR(I)
               ELRS(I) = ELRS(I-1)-((I-1)*(ELRSF)-EL3(2))*(DXR/LR)
               HRS(I) = ELRS(I) - ZR(I)
               ARS(I) = (WR+MR*HRS(I))*HRS(I)
               AR(I-1,2) = (QR(I-1,2)/ALPHA)**(1.0/M)
               QRSP(I-1) = QRMAX
               ARS(I-1) = (QR(I-1,1)/ALPHA)**(1.0/M)
               AR(I-1,1)=(QR(I-1,1)/ALPHA)**(1.0/M)
               QR(I,2) = QR(I-1,2)-(DXR/DT)*(AR(I-1,2)-AR(I-1,1))
               AR(I,2) = (QR(I,2)/ALPHA)**(1.0/M)
               HR(I,2) = (-WR+SQRT(WR**2+4*EM*AR(I,2)))/2*EM
               ELR(I,2)=ZR(I)+HR(I,2)

            IF (ELR(I,2).GT.EL3(J)) THEN
               ELR(I,2)=ELR(I,2)
               QR(I,2)=QR(I,2)
           ELSE
              QR(I,2)=QR(I,2)-(DXR/DT)*(AR(I,2)-ARS(I))

              IF (QRSP(I).EQ.(-1)) QRSP(I) = QR(I,2)


              CALL BACKW (ELR,NXR,EL3,J,QR,ZR,AR,WR,MR,VR,HF,QRSP,
     $                    I,RR,SF,HF1,SFAV,DHF,HR,DXR)
               ELR(I,2)=EL3(2)
               HR(I,2)=ELR(I,2)-ZR(I)
               AR(I,2)=(WR+MR*HR(I,2))*HR(I,2)
               PR(I,2)=WR+SQRT(HR(I,2)**2+(MR*HR(I,2))**2)
               R(I,2)=AR(I,2)/PR(I,2)
               QR(I,2)=QR(1,2)
C              WRITE(*,*)QR(I,2)
            END IF

 150        CONTINUE
C           COMPUTE CELERITY
            TOTAL = 0.00
            ML=NXR+1
            DO 160 K = 2,ML
               TOTAL=TOTAL+AR(K,2)
 160           CONTINUE
               CBAR = ALPHA*M*((TOTAL/NXR)**(M-1.0))
```

```
                IF (CBAR.GT.DXR/DT) THEN
C                  WRITE(*,*) 'CBAR DXR/DT'
C                  WRITE(*,*) CBAR,DXR/DT
                   AR(1,2)=(QR(1,2)/ALPHA)**(1.0/M)
                   HR(1,2)=(-WR+SQRT(WR**2+4*EM*AR(1,2)))/2*EM
                   ELR(1,2)=ZR(1)+HR(1,2)
                   ELR(1,3)=ELR(1,2)
                   EL3(J+1)=EL3(J)+(QG(J)+QR(NXR+1,2)-QP(J))*DT/A3

                ELSE
C                  WRITE(*,*) ' RE DO TIME STEP,CBAR<DXR/DT'
C                  GO TO 10
                END IF
                RETURN
                END


      SUBROUTINE BACKW(ELR,NXR,EL3,J,QR,ZR,AR,WR,MR,VR,
     $            HF,QRSP,I,RR,SF,HF1,SFAV,DHF,HR,DXR)
      DIMENSION EL3(600),ELR(20,3),QR(20,3)
      REAL HR(20,3),ZR(600),AR(20,3),WR
      REAL VR(20,3),HF(20,3),HF1(20,3),SF(20,3),SFAV(20,3),
     $ RR(20,3)
      REAL DHF(20,3),MR,QRSP(20)
      INTEGER NXR
C        WRITE (*,*) 'ENTERING BACKW'
         ELR(NXR+1,2)=EL3(J)
         QR(I,2)=QRSP(I)
C        WRITE(*,*)'BACKWATER'
         HR(NXR+1,2)=EL3(J)-ZR(NXR+1)
         AR(NXR+1,2)=(WR+(MR*HR(NXR+1,2)))*HR(NXR+1,2)
         RR(NXR+1,2)=AR(NXR+1,2)/((WR+
     $   2*(SQRT((MR*HR(NXR+1,2))**2+HR(NXR+1,2)**2))))
         VR(NXR+1,2)=QRSP(1)/AR(NXR+1,2)
         SF(NXR+1,2)=VR(NXR+1,2)**2*(NXR**2)/(RR(NXR+1,2)**4/3)
         HF(NXR+1,2)=ELR(NXR+1,2)+(VR(NXR+1,2))**2/(2*9.81)
         DO 210 I1=NXR,2,-1
C            I=NXR
C            WHILE(I.GE.2) DO
          ZR(17) = -3.90
          ZR(I1-1)=ZR(I1)+DXR*0.00094
C         WRITE(*,*)'ZR , I1',ZR(I1),I1
 204      HR(I1,2)=EL3(J)-ZR(I1)
          AR(I1,2)=(WR+(MR*HR(I1,2)))*HR(I1,2)
          RR(I1,2)=AR(I1,2)/((WR+2*(SQRT((MR*HR(I1,2)**2+
     $    HR(I1,2)**2)))))
          ELR(I1,2)=ZR(I1)+HR(I1,2)
          QR(I1,2) = QR(1,2)
          VR(I1,2)=QR(I1,2)/AR(I1,2)
          HF1(I1,2)=ELR(I1,2)+(VR(I1,2))**2/(2*9.81)
          SF(I1,2)=VR(I1,2)**2*(I1**2)/(RR(I1,2)**4/3)
          SFAV(I1,2)=(SF(I1+1,2)+SF(I1,2))/2
          HF(I1,2)=HF(I1+1,2)+SFAV(I1,2)*DXR
          DHF(I1,2)=HF1(I1,2)-HF(I1,2)
          IF(DHF(I1,2).GT.0.01)THEN
             HR(I1,2)=HR(I1,2)-0.008
C             WRITE (*,*)'FINISH BACKWATER'
C             GO TO 204
          ELSE
             HR(I1,2)=HR(I1,2)
             ELR(I1,2)=ZR(I1)+HR(I1,2)
C            WRITE(*,*)'LOOP BACK WATER'
```

```
              END IF
C               I=I-1
C             END WHILE
  210     CONTINUE
             RETURN
             END


          SUBROUTINE ELEVC (J,AC,QC,ALPHA1,M,HC,WC,EM,ELC,ZC,
     $    PC,DQ11,QC11,EN)
          REAL AC(600),QC(600),HC(600),WC,EM,ELC(600),ZC,PC(600)
          REAL DQ11(600),QC11(600),ALPHA1,M,EN
          WRITE(*,*) 'ENTERING ELEVC QC',QC(J)
          AC(J) = (QC(J)/ALPHA1)**(1/M)
          HC(J) = (-WC+SQRT((WC**2)+(4*EN*(AC(J)))))/(3.0)
 111      ELC(J) = ZC+HC(J)
          PC(J) = (WC+2*(SQRT((HC(J)**2)+ (EN*HC(J))**2)))
          QC11(J)=0.975  *AC(J)*((AC(J)/PC(J))**(2.0/3.0))
          DQ11(J)=QC11(J)-QC(J)
          IF(DQ11(J).GT.0.20) THEN
             HC(J)=HC(J)+0.02
             GO TO 111
          ELSE
             HC(J)=HC(J)
             ELC(J)=ELC(J)
          END IF
             WRITE(*,*)'CALC ELEVATION  QC    AC   HC   ELC '
             WRITE(*,*) QC(J),AC(J),HC(J),ELC(J),ALPHA1
          RETURN
          END


        SUBROUTINE ELEV11 (J,AC,QC,ALPHA1,M,HC,WC,EM,ELC,ZC,PC,QC11,
     $                    DQ11,M,EN)
         REAL ELC(600),QC(600),QC11(600),HC(600),DQ11(600),AC(600)
         REAL ZC,M,ALPHA1,EM,WC,PC(600),EN
         WRITE (*,*) 'ENTERING ELEV11 QC',QC(J)
          AC(J-1) = (QC(J-1)/ALPHA1)**(1/M)
          HC(J-1) = (-WC+SQRT((WC**2)+(4*EN*(AC(J-1)))))/(3.0)
 112      ELC(J-1)= ZC+HC(J-1)
          PC(J-1) = (WC+2*(SQRT((HC(J-1)**2)+(EN*HC(J-1))**2)))
          QC11(J-1)=0.975  *AC(J-1)*((AC(J-1)/PC(J-1))**(2.0/3.0))
          DQ11(J-1)=QC11(J-1)-QC(J-1)
          IF(DQ11(J-1).GT.0.20) THEN
             HC(J-1)=HC(J-1)+0.02
             GO TO 112
          ELSE
             HC(J-1) =HC(J-1)
             ELC(J-1)=ELC(J-1)
          END IF
          WRITE(*,*)'CALC ELEVATION  QC    AC   HC   ELC '
          WRITE(*,*) QC(J-1),AC(J-1),HC(J-1),ELC(J-1),ALPHA1
          RETURN
          END
```

```
      SUBROUTINE ELEV1A (HC,ELC,J,ZC,AC,PC,QC,WC,EM,EN)
      REAL HC(600),ELC(600),ZC,AC(600),PC(600),QC(600),WC,EM,EN
      WRITE(*,*) 'ENTERING ELEV1A'
      HC(J) = ELC(J)-ZC
      AC(J) = (WC+(EN*HC(J)))*HC(J)
      PC(J) = (WC+2*(SQRT((HC(J)**2)+(EN*HC(J))**2)))
      QC(J) = 0.975  * AC(J)* ((AC(J)/PC(J))**(0.6666))
      RETURN
      END

C     SET PREVIOUS DISCHARGE VALUES TO CURRENT VALUE AT HEAD OF
C        RING CANAL

      SUBROUTINE SETQR(QR)
      REAL QR(20,3)

      DO 600 I=2,20
        QR(I,1) = QR(1,2)
 600  CONTINUE
      RETURN
      END
```

APPENDIX C

C.1 OPAK DRAIN

The calculation of the capacity of Opak Drain using the Manning formula:

$$Q = 1/n\ A\ R^{2/3}\ S^{1/2} \qquad \text{(C-1)}$$

where:

Q	=	discharge
n	=	roughness coefficient
A	=	area
R	=	hydraulic radius
S	=	energy slope

Assuming a uniform flow, therefore the energy slope is equal to the bed slope.

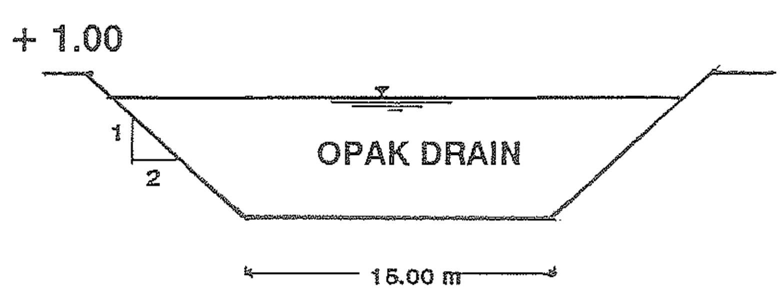

Figure C.1 Opak Drain

The bed slope for Opak Drain (S) = 0.00072

The roughness coefficient (n) = 0.02

Maximum height (h) = 2.90 m

Opak Drain width (W) = 15 m

Hydraulic Radius (R) = A/P

$$A = 0.5 \times (W + (W + 2 \times h)) \times (h) \quad \text{(C-2)}$$

$$= 0.5 \times (15 + (15 + 2 \times 2 \times 2.9)) \times (2.9)$$

$$= 60.32 \text{ m}^2$$

$$P = W + 2\sqrt{((2 \times h)^2 + h^2)} \quad \text{(C-3)}$$

$$P = 15 + 2\sqrt{((2 \times 2.9)^2 + 2.9^2)}$$

$$= 27.97 \text{ m}$$

$$R = A/P \quad \text{(C-4)}$$

$$= 60.32/27.97 = 2.16 \text{ m}$$

$$Q = 1/n \times A \times R^{2/3} \times S^{1/2} \quad \text{(C-5)}$$

$$Q = 135.08 \text{ m}^3/\text{s}$$

With these values, the maximum capacity of the Opak Drain is 135.08 m^3/s.

The above calculation is based on undisturbed flow in the channel. In fact, waste/garbage is always a problem in Opak Drain. The waste that is mostly household garbage, daily garbage disposal, increases as the population in this area increases. The presence of garbage in the channel reduces the capacity of the channel. In addition, aquatic plants whose growth is promoted by the garbage increase the roughness of the channel and, therefore, reduce the capacity of the Opak Drain. In this study, two cases are considered. The first case is the reduction of the channel capacity due to waste by 25%. The second case is the reduction of channel capacity by 10%.

For a reduction of 25%, the capacity of the Opak Drain is 101.3 m^3/s, and the corresponding roughness coefficient is 0.028. For a reduction of 10%, the capacity of the Opak Drain becomes 121.5 m^3/s, and the corresponding roughness coefficient is 0.023 (using the Manning's equation as above).

The Raya Pluit Selatan bridge upstream of the reservoir (refer to Figure 2.2) is a control section in the system. At this location, there is a constriction in the channel and as a result changes in the flow. These changes to the flow required local routing of flows. The calculation is based on the Energy Equation. First the Energy Equation is applied to section U and 1, then from 1 to C as follows:

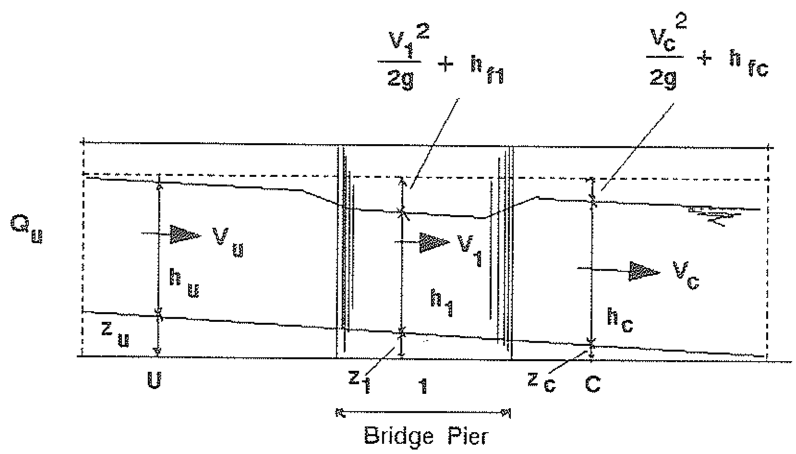

Figure C.2 Opak Drain at Pluit Selatan Bridge Location

$$H_u = H_1 \quad \text{(C-5)}$$

$$z_u + h_u + V_u^2/2g = z_1 + h_1 + V_1^2/2g + h_{f1} \quad \text{(C-6)}$$

$$h_{f1} = \alpha_1 V_1^2/2g \quad \text{(C-7)}$$

$$H_1 = H_c \quad \text{(C-8)}$$

$$z_1 + h_1 + V1^2/2g = z_c + h_c + V_c^2/2g + h_{fc} \quad \text{(C-9)}$$

$$h_{fc} = \alpha_c V_c^2/2g \quad \text{(C-10)}$$

where:

z_u = bed elevation at location u

h_u = water depth at location u

V_u = velocity at location u

z_1 = bed elevation at location 1

h_1 = water depth at location 1

V_1 = velocity at location 1

h_{f1} = losses due to contraction

α_1 = coefficient of losses due to contraction

z_c = bed elevation at location C

h_c = water depth at location C

V_c = velocity at location C

h_{fc} = losses due to expansion

α_c = coefficient of losses due to expansion

The procedure of the calculation is as follows:

1. Based on the inflow from upstream, the Manning's equation (C-1) is applied to calculate the depth and velocity at point U.

2. Use the depth and velocity at point U to calculate the total energy head at point U.

3. Assume a value of water depth at point 1 and calculate the velocity and total energy head at that point. The losses due to contraction are considered as a function of V_1^2. In this system the coefficient of losses due to contraction is taken as 0.05 (α_1).

4. Compare H_1 and H_u (results from 2 and 3). If the energy level at point 1 lower than the energy head at point U, try a higher value of water depth at point 1.

5. Repeat the above calculations (3 and 4) beginning from the assumption of water level at point 1 until the values agree to within a certain limit. In this study the limit is taken 0.01 m.

The same procedure is applied for the calculation of the water level at point C:

- The energy head at point 1: $H_1 = H_u - V_1^2/2g$
- The energy head at point 1 and point C is the same.
- The energy losses between 1 and C due to expansion should be taken into account. The energy loss due to expansion is $\alpha_c\ V_c^2/2g$ and the value αc in this system it is taken to be 0.15.
- The water depth at C can be calculated by the same process used to calculate the water depth at 1.

C.2 SYPHON

The calculation of the syphon capacity is based on the water levels upstream and downstream of the syphon. As the flow increases and the water elevation becomes higher than the syphon crest, the following are four steps of syphon operation starting from low flows upstream.

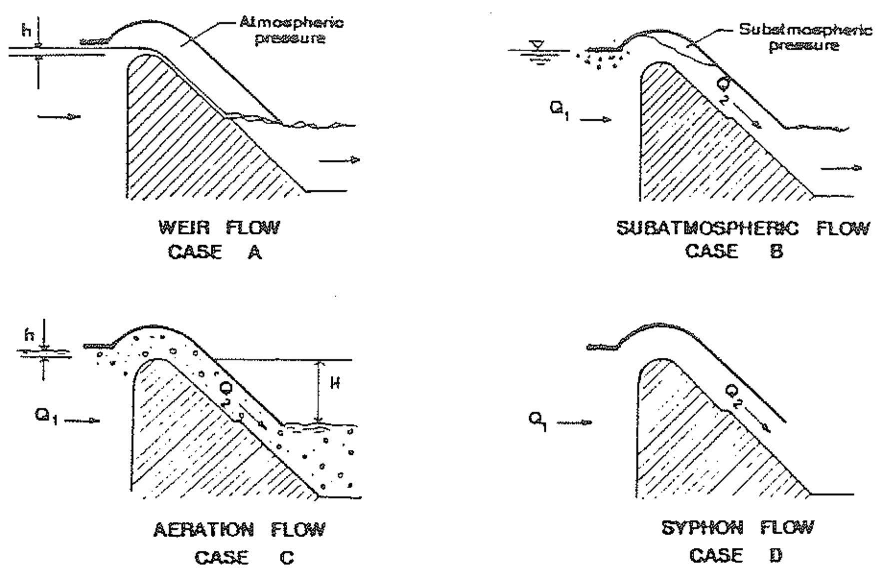

Figure C.3 Syphon Flow Condition

The flow phases are:

1. **Weir Flow (Case A).**

   Elevation of water upstream is higher than the syphon crest but below the top of syphon entrance. At this condition, normal atmospheric pressure occurs in the syphon (at the top of the syphon throat, therefore the weir flow occurs and syphon is working as a weir). The discharge capacity only depends on the upstream water level. The calculation at this condition is as follows:

$$Q = C\ Le\ H^{3/2} \tag{C-12}$$

where:

Q = discharge through the syphon m^3/s

C = weir coefficient $m^{1/2}/s$

Le = effective crest length (m)

H = total head on crest (m)

The value of C depends on the crest shape/radius and the coefficient can be obtained from "Design of Small Dams, U.S. Department of Interior" or "Irrigation Engineering," Leliavsky. The weir coefficient for the Pluit syphon is 1.0. In this study the effective crest length is taken as 90 % of the total crest length. The effect of contraction due to piers is included in this 10% reduction crest length.

2. <u>**Subatmospheric Weir Flow (Case B).**</u>

This condition appears as the water level increases upstream and downstream of the syphon, preventing air from entering the syphon from both ends. This condition is unstable:

- If the outflow(Q_2) is larger than the inflow(Q_1), the upstream water level will decrease and hence air will enter the syphon.
- If the outflow(Q_2) is smaller than the inflow(Q_1), the upstream water level increases.

Both condition occurs in this subatmospheric weir flow one after the other. Since this condition is a temporary adjustment to the syphon condition it is not included in the calculation of Pluit Polder System.

3. **Aeration Flow (Case C)**

This condition is nearly the same as subatmospheric condition but the upstream flow is higher. As a result the syphon is filled with a mixture of air and water. This is also a temporary condition, and therefore it is not included in the calculation of Pluit Polder System.

4. **Syphon Flow (Case D).**

If the pressure condition at the throat is zero (all the air has been removed), then the syphon works fully as a syphon. This condition can be reached if the water level upstream is higher than the mouth level of the syphon and the downstream water level is higher than the leg of syphon (preventing air entering the syphon). In Pluit Polder System, the water level in the leg of the syphon is the same as water level in the Pluit reservoir. The water level upstream is the water level at point C. (Refer to Figure 2.2 and 3.2). The calculation of the capacity for this condition is as follows:

$$Q = \mu A \sqrt{(2g H)} \qquad \text{(C-13)}$$

where:

Q = discharge through the syphon

μ = coefficient due to losses

A = cross sectional area of the throat

g = acceleration due to gravity

H = head available

The determination of the coefficient due to losses, μ, is based on " Design of Small Dam ",1977 and "Irrigation Engineering", Leliavsky:

- Loss at the entrance to the syphon

$$he = 0.2\ V2/2g \quad \text{(C-14)}$$

- Loss due to friction

$$he = 0.25\ V^2/2g \quad \text{(C-15)}$$

- Loss in the upper bend of the syphon (depend on centre line radius)

$$he = \alpha_1\ V^2/2g \quad \text{(C-16)}$$

- Loss at the outlet

$$he = (a_1/a_o)^2 \quad \text{(C-17)}$$

where:

a_1 = area at the throat

a_o = area at the outlet

For practical use the range of variation of the value of μ (the coefficient due to losses) is between 0.55 - 0.80 {source: "Irrigation Engineering," Leliavsky}. In this Pluit Syphon, the coefficient due to losses is taken to be 0.70.

C.3 CALCULATION OF BACKWATER CURVE IN THE RING CANAL

When the Forebay water level is above that at the Ring canal Intake structure(Figures 2.2 and 3.3), a back water calculation is necessary to calculate changes in water level along the length of the ring canal. The change in water level in the ring canal is gradual. It can be included as a gradually varied flow.

Since the channel slopes in the ring canal are mild and the flow is subcritical, the control point is at the Forebay, which is the downstream section of the ring canal. The control point for elevations in the ring canal is the elevation in the Forebay. Using the control point as the starting point, the calculation proceeds upstream.

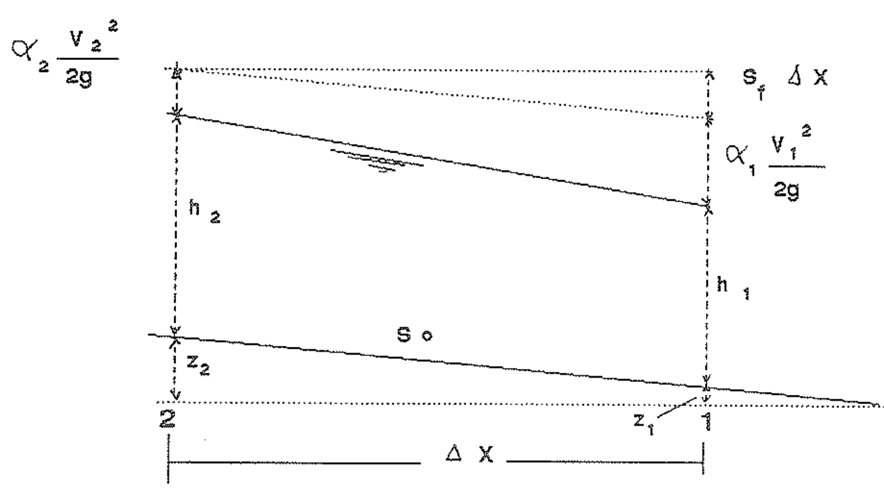

Figure C.4 Back Water

The basic equations in this calculation are the Energy Equation and Manning's Equation:

The Energy equation:

$$H_2 = H_1 + S_{fav} (x_2 - x_1) \qquad \text{(C-18)}$$

where:

H_2 = energy head at point 2

H_1 = energy head at point 1

S_{fav} = average friction slope between point 1 and 2

x_2 and x_1 = distance

The average friction slope of the water surface can be calculated by taking the average of the friction slopes at point 1 and point 2. The friction slope at each point can be calculated using the Manning's equation:

$$S_f = n^2 V^2/R^{4/3} \qquad \text{(C-19)}$$

where:

S_f = friction slope

n = Manning's roughness coefficient

V = velocity

R = hydraulic radius

The numerical calculation for this system uses the standard step method. The computation, which proceeds by trial and error methods, is as follows:

1. Point 1 (at ring canal forebay outlet) is considered as the control point, therefore it becomes the starting point. The energy level and friction slope at this point are as follows:

$$H_1 = z_1 + h_1 + \alpha_1 V_1^2/2g \quad \text{(C-20)}$$

$$S_{f1} = n^2 V_1^2/R1^{4/3} \quad \text{(C-21)}$$

where:

H_1 = total energy head at point 1

h_1 = water depth at point 1

g = acceleration due to gravity

n = Manning's 's roughness coefficient

R_1 = hydraulic radius at point 1

S_{f1} = energy slope at point 1

V_1 = velocity at point 1

z_1 = bed level at point 1

2. At point 2 assumed a value for the water level, then calculate velocity and energy head:

$$H_2 = z_2 + h_2 + \alpha_c V_2^2/2g \quad \text{(C-22)}$$

$$S_{f2} = n^2 V_2^2/R_2^{4/3} \quad \text{(C-23)}$$

3. The average energy slope can then be calculated as:

$$S_{fav} = (S_{f1} + S_{f2})/ 2 \quad \text{(C-24)}$$

$$H_2' = (S_{fav})*(x_2-x_1) \quad \text{(C-25)}$$

where:

H_2' = Total energy head at point 2 using equation (C-25)

4. Compare H_2' and H_2 (2 and 3). If the value of H_2' is higher than H_2, take a higher value h2 but smaller than $(H_2'-H_2)$. If the value of H_2' is lower than H_2, take a lower value h_2 but smaller the $(H_2'- h_2)$

5. Using point 2 as the starting point to calculate the water depth at point 3, iterate until the values agree to within a certain limit (calculation 2-4). In this study the limit is taken 0.01 m. These calculations are included in the computer model as subroutine Backw. After a satisfactory value of h_2 has been determined for point 3 repeat the calculation using the newly calculated point (in this case 3) as a starting point. Proceed until backwater affects are negligible.

C.4 AUTOMATIC TIDAL GATES

The automatic tidal gates divert water directly to the sea during low tide. The width of each gate is taken as 2.00 m. The equation for the flow through the gates is:

$$Q = \mu A \sqrt{(2g H)} \qquad \text{(C-26)}$$

where:

Q = discharge through the gates

μ = coefficient due to losses

A = cross sectional area of the gates opening

g = acceleration due to gravity

H = gross head available between the Forebay and the sea.

The calculation of the flow through the automatic gates are included in the computer program as one of the possibilities for improvement of the outlet system. In this study the coefficient due to losses at the gates is taken to be 0.80. Further field investigation on the relation between the water level upstream and downstream of the gates, the gates opening and the flow is essential. In this study the gate opening is assumed to be fully opened. The head losses along the channel from the Forebay to the location of the gates (about 100 m) are neglected in this calculation. If due to geological conditions the gates must be located at a distance from the Forebay, the head losses along the channel should be included.

C.5 CALCULATION OF OPAK DRAIN AND RING CANAL IMPROVEMENTS

The conditions were considered in the improvement of Opak Drain and ring canal:

- Increase the channel Bank to +1.40 (No cleaning, so Manning's n increased to 25%)
- Regular cleaning
- Combination of increase the channel bank to +1.40 and regular cleaning

By increasing the Channel bank elevation, the cross sectional area of the channel increases, using the Manning's equation, the following results can be achieved:

$$Q = 1/n \; A \; R^{2/3} \; S^{1/2} \qquad \text{(C-27)}$$

where:

The bed slope for Opak drain (S) = 0.00072

The roughness coefficient (n) = 0.028 (25% garbage)

Maximum height = 3.40 m

Opak Drain width (W) = 15 m

m (side slope) = 2

$$A = 0.5x(W+(W+2x\ m\ x\ h))x(h) \qquad (C\text{-}28)$$

$$= 0.5x(15+(15+2x2x3.4))x(3.4)$$

$$= 74.12\ m^2$$

$$P = W + 2\sqrt{((m\ h)^2 + h^2)} \qquad (C\text{-}29)$$

$$P = 15 + 2\sqrt{((2\ x\ 3.4)^2 + 3.4^2}$$

$$= 29.86\ m$$

$$R = 74.12/29.86$$

$$= 2.48\ m$$

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

$$Q = 130.2\ m^3/s$$

Considering regular cleaning of the canal, leads to a decrease in Manning's n and therefore an increase in the channel capacity. The increase in capacity depends on the effectiveness of the channel cleaning procedure. The effectiveness of these procedures needs field investigation. In this study, it is assumed that the design conditions correspond to a roughness value of 0.023 (assuming 10% reduction in capacity due to the presence of garbage). The discharge then becomes:

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

$$Q = 121.5 \ m^3/s$$

Using a combination of increasing the bank elevation to +1.40 and cleaning the channel regularly gives the following discharge in the Opak Drain:

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

$$Q = 1/0.023 \times 74.12 \times (2.48)^{2/3} \times (0.00072)^{1/2}$$

$$= 158.43 \ m^3/s$$