CONTRATO

IICA / INDRHI / CSU

INSTITUTO NACIONAL DE RECURSOS HIDRAULICOS (INDRHI)

UNIVERSIDAD DEL ESTADO DE COLORADO (CSU)

ESTUDIOS SOBRE LA OPERACION Y SEGURIDAD DEL SISTEMA DE EMBALSES DE VALDESIA

FINAL REPORT

VOLUME III

FLOOD OPERATION STUDIES 1/

DOCUMENTO No. 38
11/08/86

Serie Publicaciones Misceláneas
A1/DO-86-004
ISSN-0534-5391

FINAL REPORT

VOLUME III

FLOOD OPERATION STUDIES 1/

38

31/08/86

1/ This document was prepared by J. Obeysekera, K.Hiew, G. Tabios III, J.D. Salas and H.W. Shen, Colorado State University, Fort Collins, Colorado.

11CA
PM-A1/DO
86-004

BV 004201

00001498

PRESENTACION

Los estudios de Operación y Seguridad del Sistema de Embalses de Valdesia fueron ejecutados conjuntamente por el Instituto Nacional de Recursos Hidráulicos (INDRHI) de la República Dominicana, la Universidad del Estado de Colorado (CSU) y el Instituto Interamericano de Cooperación para la Agricultura (IICA), a través del Contrato IICA/INDRHI/CSU firmado el 6 de abril de 1984. Los estudios se iniciaron el 6 de agosto de 1984 y finalizaron el 31 de agosto de 1986.

Los estudios fueron financiados por el INDRHI a través del préstamo 1655-DO del Banco Mundial.

La ejecución de los estudios se desarrolló en seis áreas:

a) Estudios Hidrológicos
b) Operación Normal
c) Operación de Emergencia
d) Inspección, Mantenimiento y Seguridad de Presas
e) Organización para la Operación del Sistema de Embalses
f) Entrenamiento y Transferencia de Tecnología

En este documento se incluye parte del material técnico del Informe Final, el cual consta de los siguientes volúmenes:

- Resumen
- Estudios Hidrológicos
- Operación Normal
- Estudios de Operación de Crecidas
- Estudios de Inspección, Mantenimiento y Seguridad de Presas
- Organización y Funciones para la Operación del Sistema de Embalses de Valdesia.

-Transferencia de Tecnología y Capacitación.

-Plan de Operación de Emergencia para el ~~Sistema~~ de Embalses de Valdesia.

-Plan de Operación Normal para el Sistema de Embalses de Valdesia: (1) Riego y Energía, (2) Control de Crecidas.

-Manuales de Operación de Modelos Computarizados para la Operación Normal del Sistema de Embalses.

-Manual de Usuario de Modelos de Sistemas Hidrológicos.

Santo Domingo, República Dominicana
31 de agosto de 1986

DR. JOSE D. SALAS
Coordinador por CSU

DR. AGUSTIN A. MILLAR
Coordinador General
Estudios Embalse Valdesia
(IICA)

ING. JULIO M. LLINAS
Coordinador por INDRHI

VOLUME III
FLOOD OPERATION STUDIES

TABLE OF CONTENTS

VOLUME III. FLOOD OPERATION STUDIES

3.1 INTRODUCTION

The Valdesia reservoir system, located on the Nizao River in the Dominican Republic, was designed to provide irrigation water to the Nizao project areas and hydroelectric energy to the national electrical network system. The reservoir system consists of a main reservoir, dam and spillway, a power plant and outflow regulating works, together with an afterbay, a diversion and spillway system a short distance downstream.

The study on the operational management of the Valdesia system reported in a series of volumes including this one, involved several interrelated areas. This volume reports in detail the work involved in the development of operating rules for Valdesia system under flood emergency conditions. Nizao basin experiences floods due to two distinct types of storms: (a) small storms due to local weather phenomena or weak tropical storms which result in relatively small floods; and (b) large scale tropical cyclones in general, or hurricanes in particular, which result in large floods. Clearly the emergency conditions which may arise due to these two types of floods require different operating procedures. Specifically, the hurricane related floods require the forecasts of tracks and precipitation of hurricanes so that necessary steps may be taken in advance to eliminate or minimize catastrophic damages.

3.2 METHODS OF INVESTIGATION

The work required in the development of flood operating rules may be listed under four topics:

1. Development of the flood routing model for Valdesia system
2. Flood routing studies
3. Development of emergency operating rules
4. Hurricane forecasting

For studies of flood routing through Valdesia-Las Barias system, a computer model to simulate the hydraulic characteristics of spillways and various outflow regulating works of the Valdesia-Las Barias system is developed. Three modes of operation for flood routing is included in the model: (a) operation by induced surcharge method; (b) existing operating procedure; and (c) hurricane operation in which all gates are left fully opened from the beginning of the flood. The model can be used in real-time for developing gate regulation schedules during floods.

The flood routing model is used to investigate the capacity of the outlet works to allow safe passage of hypothetical floods without overtopping the dam or if this is not the case to what extent these outlets can be used to prevent overtopping. The reconstructed hydrograph of hurricane DAVID is also used in routing studies. The influence of upstream reservoirs in controlling the design floods is also investigated. The routing of design flood hydrographs due to both hurricane and non-hurricane conditions are investigated. The results of these routing studies become the primary base for developing operating rules for hurricane and non-hurricane emergency conditions.

The possibility of forecasting track and precipitation from hurricanes is investigated. A review of existing models for track forecasting permitted the selection of CLIPER regression model for forecasting tracks of tropical cyclones up to 72-hours lead time. These

track forecasts are extremely useful for operating the system under hurricane emergency conditions. Unfortunately, a review of a state of the art of precipitation reveals that portable models for precipitation forecasting of hurricanes, which can be readily installed in the Dominican Republic are not available. An approximate procedure based on past precipitation patterns of Hurricane David is employed for predicting precipitation potential of a hurricane.

3.3 SUMMARY OF CONCLUSIONS

The following is a summary of conclusions based on the flood routing studies reported in this section.

1. A computerized reservoir operation model can be a useful and effective tool in real time emergency operation. A workable model has been developed based on the best available data and technology. The model can be and should be refined at a later stage as more data and better operating experience are gained.
2. The temporary flood control storage above the normal pool level of 145 m is small, being of the order of 40 million m^3. Advance drawdown of the reservoir to levels below 145.0 m is time consuming because of capacity constraints in the outlet works and hence is unlikely to be practical.
3. Within the practical limits of operation, the safe peak flood inflow is of the order of 11,000 m^3/sec corresponding to an initial reservoir level of 145.0 m in Valdesia (based on simulated hydrograph of Hurricane David).
4. For small floods, the induced surcharge method is more effective in suppressing the peak release, particularly if the initial reservoir level is low. For large floods, the induced

surcharge method can result in higher peak outflows and reservoir elevations than the other modes of operation and is therefore not recommended.

5. For medium floods to large floods, the induced surcharge method and the outflow equals inflow method will give very similar results. Both methods can be used but the latter is computationally simpler.

6. Where a clear-cut choice of mode of operation is not possible, a real time simulation using the two different modes of operation should be carried out and the final decision should be based on the analysis of the resulting outflow and stage hydrographs.

7. For large floods (hurricane flood), the hurricane mode of operation (Mode 2) is advantageous. However, caution must be exercised to control the initial outflow surge as the gates are suddenly opened to the full, particularly when the initial reservoir level is high. After the peak of the inflow hydrograph has passed, it is also advisable to revert back to Mode 0 or 1 so that the reservoir storage is not unnecessarily depleted.

8. The spillways at Valdesia do not have sufficient capacity to pass the SPF (AMC II and III conditions) and the PMF (all AMC conditions). If an upstream reservoir is constructed at Jiguey with some flood control allocation (about 56 million m^3 flood storage), the peaks of SPF for all antecedent moisture conditions can be effectively controlled. The proposed Jiguey Reservoir, however, will not be able to provide sufficient

control on the PMF. The safety of the dam under PMF inflow should be investigated urgently as a separate study.

9. The "official" track forecasts produced by the National Hurricane Center in Miami give best forecasts and they should be obtained whenever possible. However, the much simpler CLIPER model produce forecasts in the region below 24.5° latitude, which are as nearly accurate as forecasts computed by the other available models.

10. The models for forecasting precipitation from severe storms such as those due to hurricanes are still being developed. The current procedures used in U.S. combine the subjective methods based on forecasters' past experience with some objective analyses and therefore they cannot be readily transported to the Dominican Republic. This is an area which requires much attention in the future since the precipitation potential of a hurricane determines the nature of flood potential to a very large extent.

3.4 ORGANIZATION OF THE VOLUME

The work involved in the development of operating procedures for flood conditions is reported in four sections. The development of flood routing model for Valdesia - Las Barias system for the three modes of operation is discussed in Section 3.5. The routing of hypothetical floods and the reconstructed hydrographs of Hurricane David is presented in Section 3.6. The development of the operating rules for both hurricane and non-hurricane conditions is discussed in Section 3.7. The track forecasting of hurricanes and the state-of-the-art review of hurricane precipitation forecasting is included in Section 3.8.

3.5 FLOOD ROUTING MODEL

3.5.1 General Considerations

Flood reservoir operation refers to the operation of a reservoir system during short-term events such as floods where the primary concern is safety of the dam and downstream flood damages. The damage to and failure of gates in Valdesia Dam during Hurricane David (August 30, 1979) served to illustrate the importance of sound and proven emergency operation rules. The primary purpose of this section is to develop such rules or guidelines based on dam safety requirements. Downstream flood damage consideration is not incorporated at this stage since there is little quantitative data to permit any meaningful analysis.

An ideal real time flood operation model is one which accepts forecasts of flood inflows, simulates likely consequences under different assumptions of operation, and identifies the best course of action which meets operation objectives and system constraints. It is not a substitute for good judgement by the operator, but rather it is a tool to assist him in better decision making. Within reasonable limits, established rules, proven past experience and at times, operators' preference can be incorporated into a mathematical model to form what is known as a computerized decision supporting system. Given the fact that advanced time for decision making during emergency operation is generally short, which is particularly true in the case of a steep gradient river like Rio Nizao, the advantage of a computerized operation model is readily apparent.

The routing of flood inflows through a reservoir is a relatively straight forward exercise once the physical and hydraulic

characteristics of the reservoir systems are given. A significant part of the effort in developing the emergency operation model is therefore in collating the necessary data such as reservoir stage storage curves, stage discharge relationships of spillways, gates, sluices and hydropower plants from various sources. Due to the scarcity of data, the hydraulic characteristics of most outlet structures have to be derived using empirical equations and charts developed in the United States and published in the design manuals of the Bureau of Reclamation and the Army Corps of Engineers. In many cases, the quality of available data leaves much to be desired, but every effort has been exercised to make the best uses of the limited information to arrive at a workable model for simulating the response of the existing system. There is room for refinement of the model findings as more accurate data and better operations experience become available in the future.

3.5.2 Reservoir Characteristics

The Valdesia - Las Barias System is made up of two reservoirs namely the Valdesia Storage Reservoir and the Las Barias Regulative Storage Reservoir. Valdesia Dam is a 76 m high concrete buttress dam built on the Nizao River. It commands a total storage of 153.008 million m^3 at the normal full pool level of 150.0 m. The crest level of the dam is at 156.0 m and maximum flood surcharge level has been set at 154.0 m. The stage-storage curve for Valdesia Reservoir has been developed using the latest aerial survey of the reservoir basin carried out in May, 1981. The result is as given in Figure 3.5.1.

The Las Barias Dam is located about 5 km downstream of Valdesia Dam. It is a concrete gravity dam with a height of 22.6 m. It has a rather small storage ($3.0 \times 10^6 m^3$ at the normal full pool level of 77.0

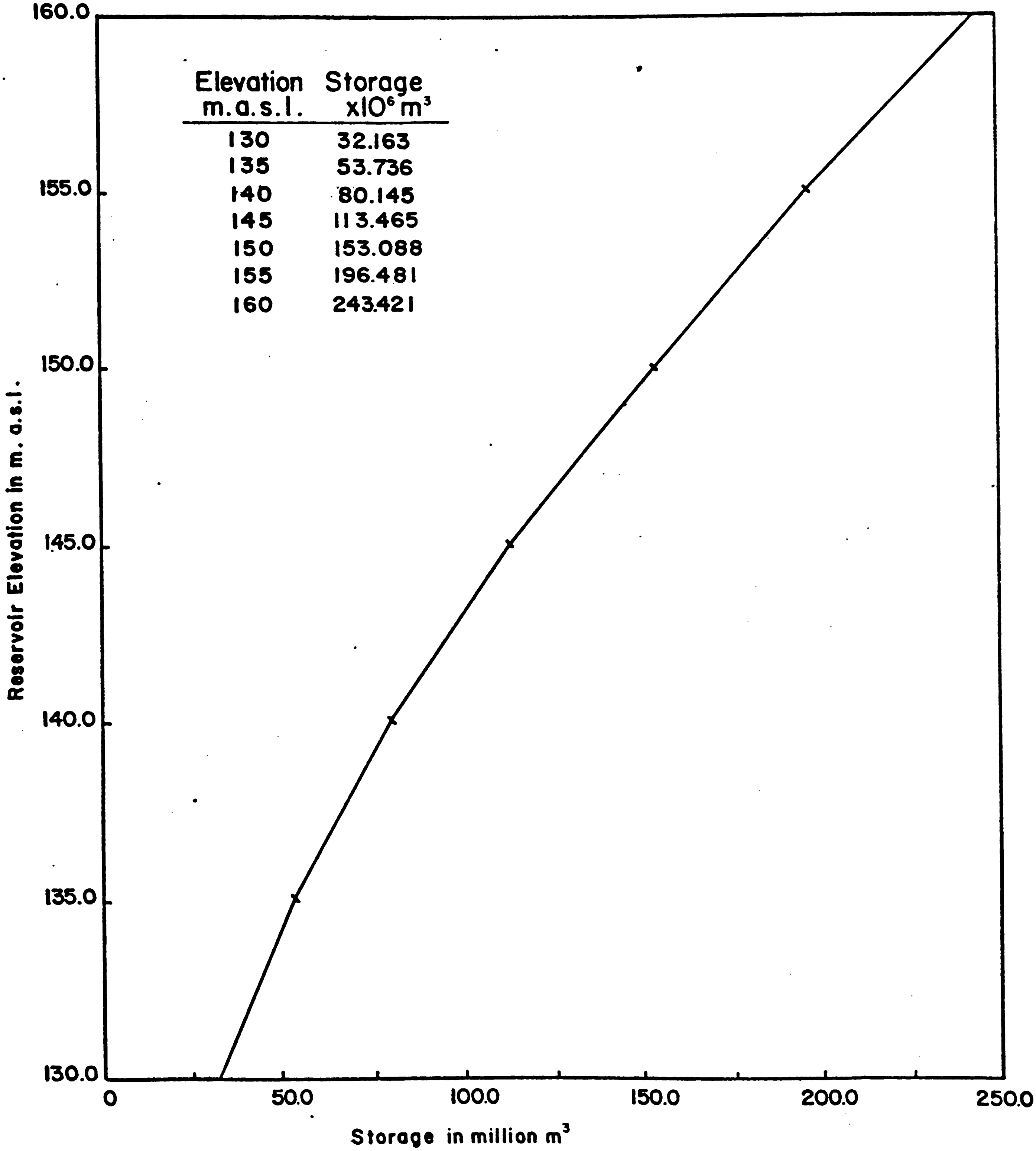


Figure 3.5.1. Stage-storage curve for Valdesia Reservoir.

m) since its primary purpose is to regulate the releases of Valdesia Dam and to develop the head required for flow through the irrigation diversion structure. The stage storage relationship is as given in Figure 3.5.2.

For both the reservoirs, the stage-storage relationships are coarsely defined. In the case of Valdesia, it is given at 5 m intervals which is certainly too crude for accurate flood routing studies. Linear interpolation is used to determine storages at other intermediate levels. A more accurate survey of the reservoir basin above 150 m, say at 1 m intervals would be useful for more refined flood routing investigations.

3.5.3 Hydraulic Characteristics of Outlets

Both the Valdesia and Las Barias dams are equiped with multi-gated spillways. The releases from the reservoirs are therefore dependent on how the gates are operated and the hydraulic characteristics of the gates. The hydraulics of release however falls into two types:

(i) Controlled release which refers to the flow regime when the regulation by gates is in effect.

(ii) Uncontrolled release when all the gates are fully opened and the spillway is behaving like the free overflow type.

Under most circumstances, the operator will control the release by judicious operation of the gates to achieve the best compromise between conservation requirements, downstream flood damage and reservoir safety. The gates offer much greater flexibility in operation, but demand greater expertise on the part of the operator. The hydraulics of gate release is also more complex.

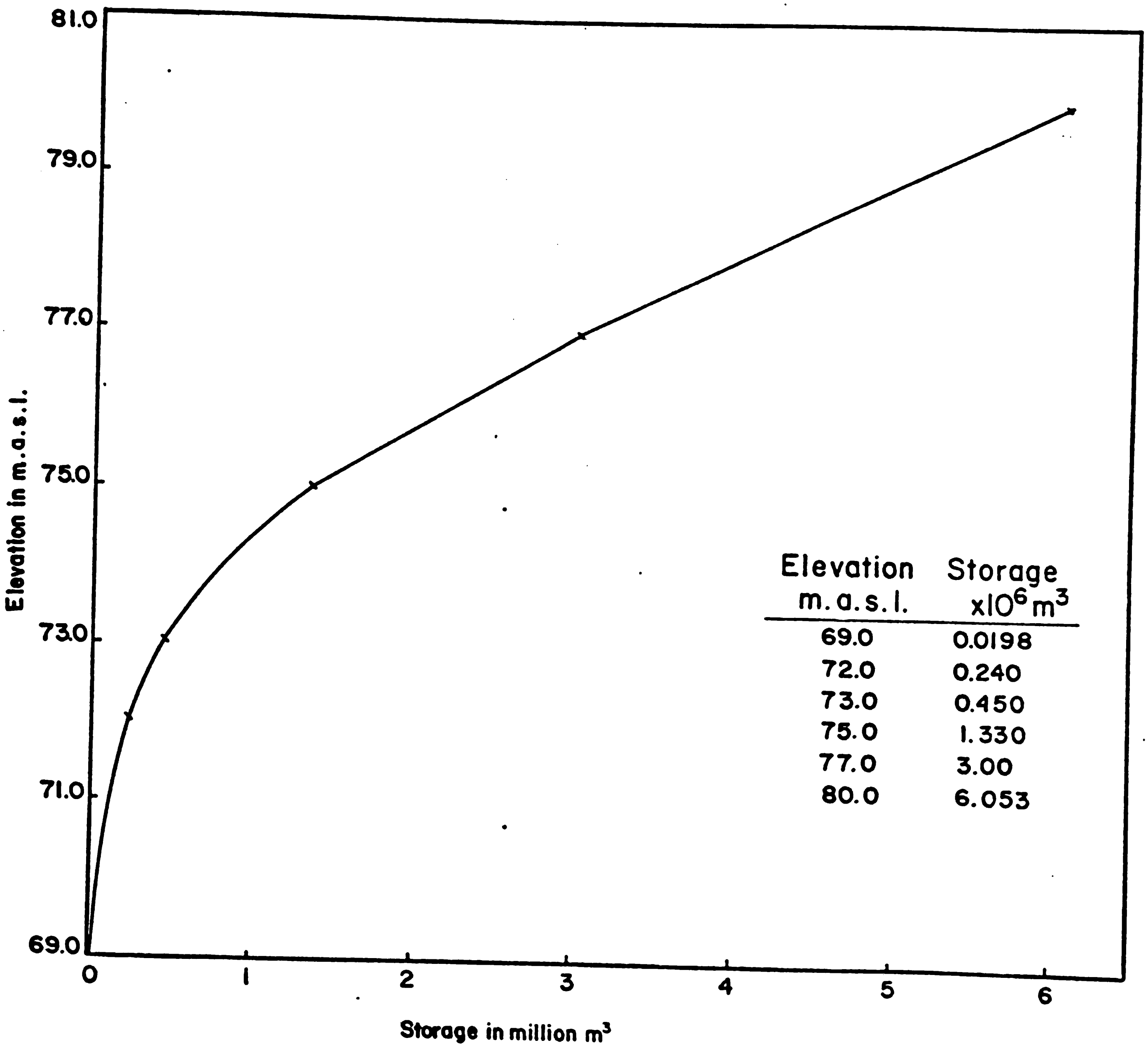


Figure 3.5.2. Stage-storage curve for Las Barias Reservoir.

Uncontrolled release will be discussed first since it is a lot simpler as far as the hydraulics of flows is concerned. The equation for a free overflow spillway as given in Design of Small Dams (Bureau of Reclamation, 1976) is

$$Q = C_q C_o C_s (W - 0.2\, H_e) H_e^{1.5}$$

where C_q = coefficient of discharge

C_o = correction for coefficient of dicharge for other than design head

C_s = corrections for tailwater submergence

W = width of spillway

H_e = head of spillway measured from the crest level

The values of C_q, C_o, W, H_e are directly dependent on the geometric dimensions of the spillways. The values of C_q, W for Valdesia and Las Barias are given below:

Parameter	Valdesia	Las Barias
C_q	2.16	1.91
W	120.0	118.0

The values of C_o can be read off directly from Figure 250 of the above mentioned reference. Alternately, it can be computed from the following polynomial regression equation:

$$C_o = 0.782896 + 0.40209\, Hr - 0.3473\, Hr^2 + 0.2210\, Hr^3 - 0.05941\, Hr^4$$

where $Hr = \frac{H_e}{H_o}$

H_o is the designed head which has a value of 9.0 in the case of Valdesia and 10.5 m in the case of Las Barias. There is no tailwater submergence effect at Valdesia and hence, C_s acquires a value of 1.00. In the case of Las Barias, the tailwater effect is significant and has to be treated separately.

Using the equations just described, it is possible to develop a stage discharge relationship for uncontrolled release over the Valdesia spillway. The result is as given in Figure 3.5.3. It can be seen that at the maximum flood surcharge level of 154 m, the spillway has a maximum capacity of about 6890 m^3/sec.

The stage-discharge relationship for Las Barias spillway for uncontrolled release is slightly more complex because of the tailwater submergence effect. The tailwater levels at Las Barias spillway for a range of spillway outflows were first computed using the standard HEC-2 program. The analysis was carried out starting at about 10 km downstream of Las Barias Dam using river cross section data from 1 : 10,000 aerial photomosaic maps with 2 m contour overlay. The results of analysis are given in Figure 3.5.4. Once the tailwater level for a particular discharge is known, the submergence coefficient can be read off from Figure 3.5.5 which is a direct reproduction of Figure 254 in Design of Small Dams. The determination of stage discharge relationship for Las Barias spillway, however, requires an iterative approach because of the submergence correction factor. Starting with an assumed submergence correction C_s, the outflow, Q is computed. An improved estimate, C_s is then obtained and the procedure repeated until convergence. The procedure can be easily implemented on a computer and

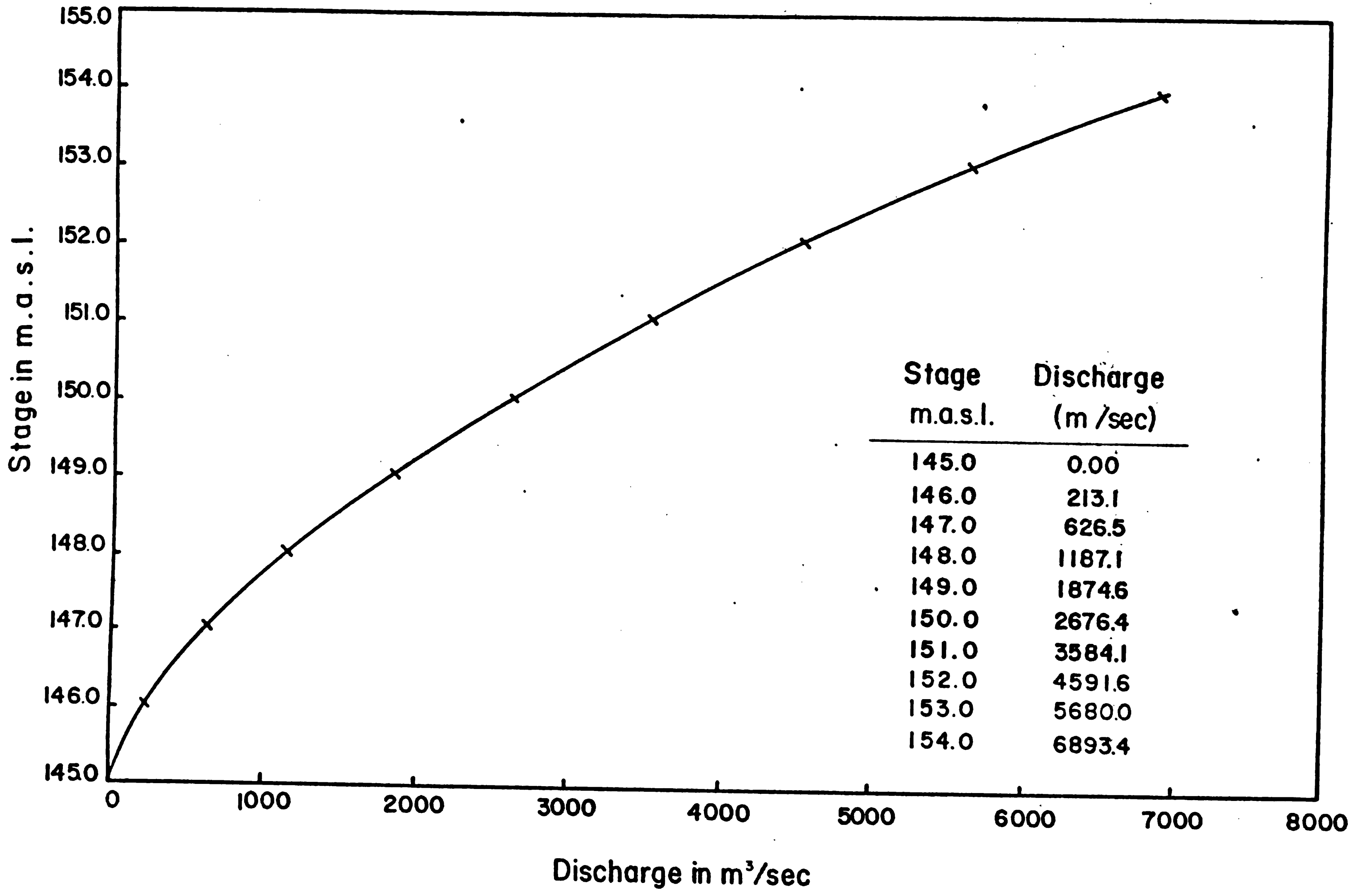


Figure 3.5.3. Stage-discharge curve for uncontrolled flow over Valdesia spillway.

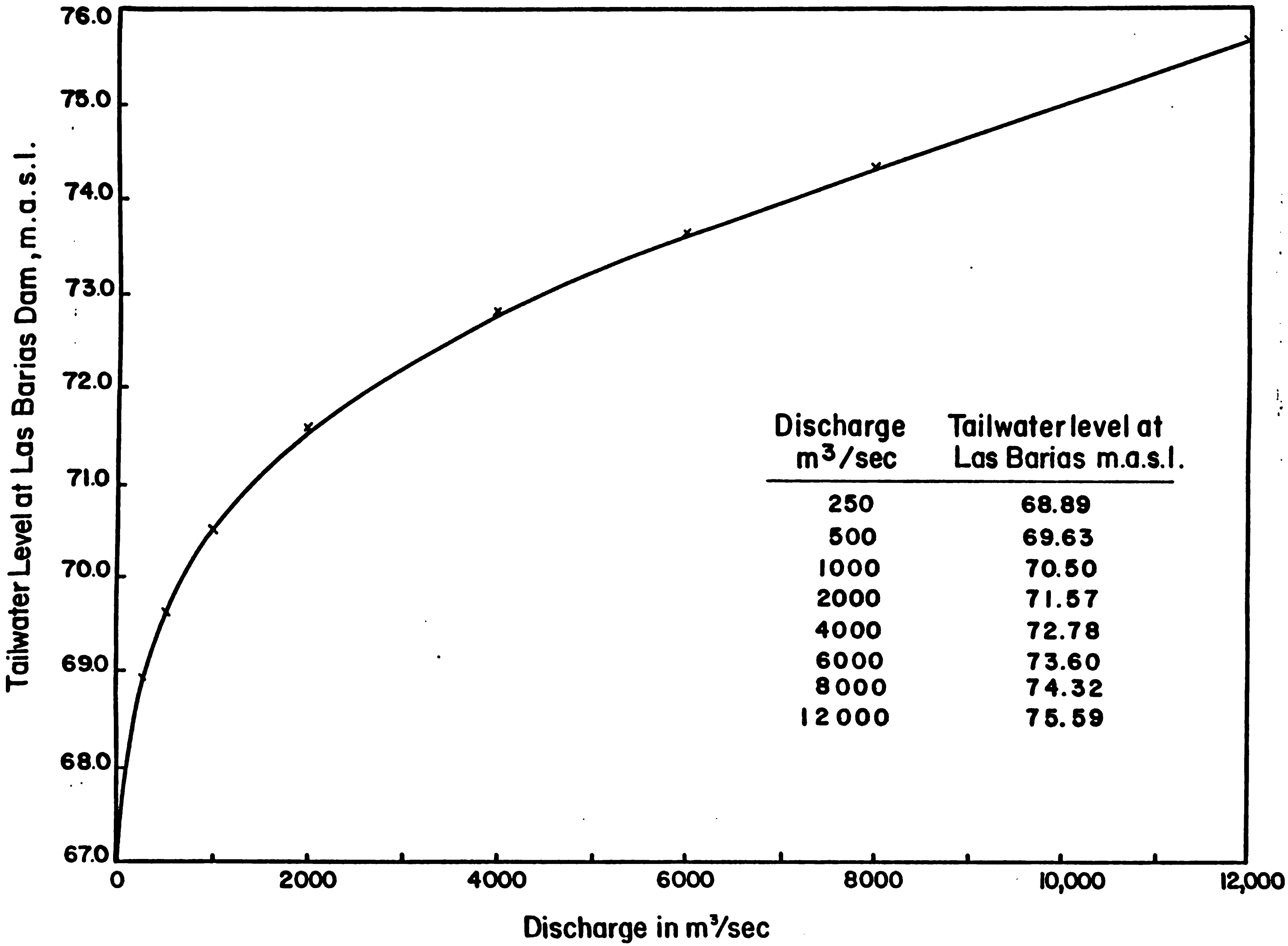


Figure 3.5.4. Tailwater level at Las Barias dam based on HEC-2 analysis.

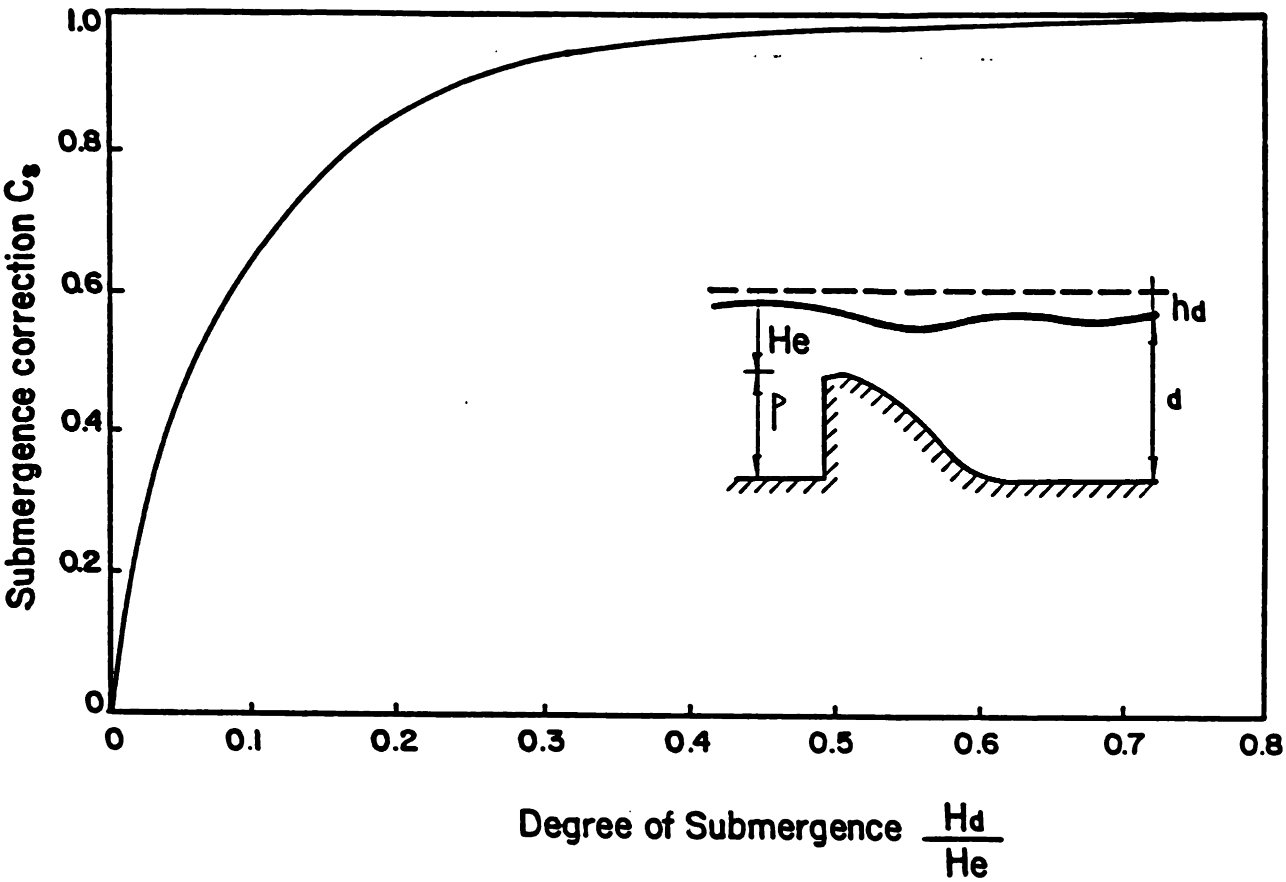


Figure 3.5.5. Submergence coefficient (adopted from Fig. 254 'Design of Small Dams', USBR, 1976).

the results are as given in Figure 3.5.6. It can be seen that at the maximum flood surcharge level of 79.5 m the Las Barias spillway has a maximum outflow capacity of 7350 m^3/sec.

The next step was to develop elevation-discharge curves for different openings of the gates on spillway. The original curves provided had probably been computed by using a coefficient of discharge given by Figure 256 of the Design of Small Dams manual (p. 386). This curve, however, does not provide the coefficients below 0.645 and therefore its application to the entire range of gate openings and water surface elevations is questionable. A better technique given by U.S. Army Corps of Engineers (Hydraulic Design Criteria, charts 311-1 and 311-2) which is applicable specifically to tainter gates on spillway crests was used to obtain the elevation-discharge relationships for different gate openings.

In this method, the discharge is given by

$$Q(m^3/s) = C_v \times G_o \times B \times \sqrt{2gH}$$

where G_o = net gate opening

C_v = coefficient of discharge

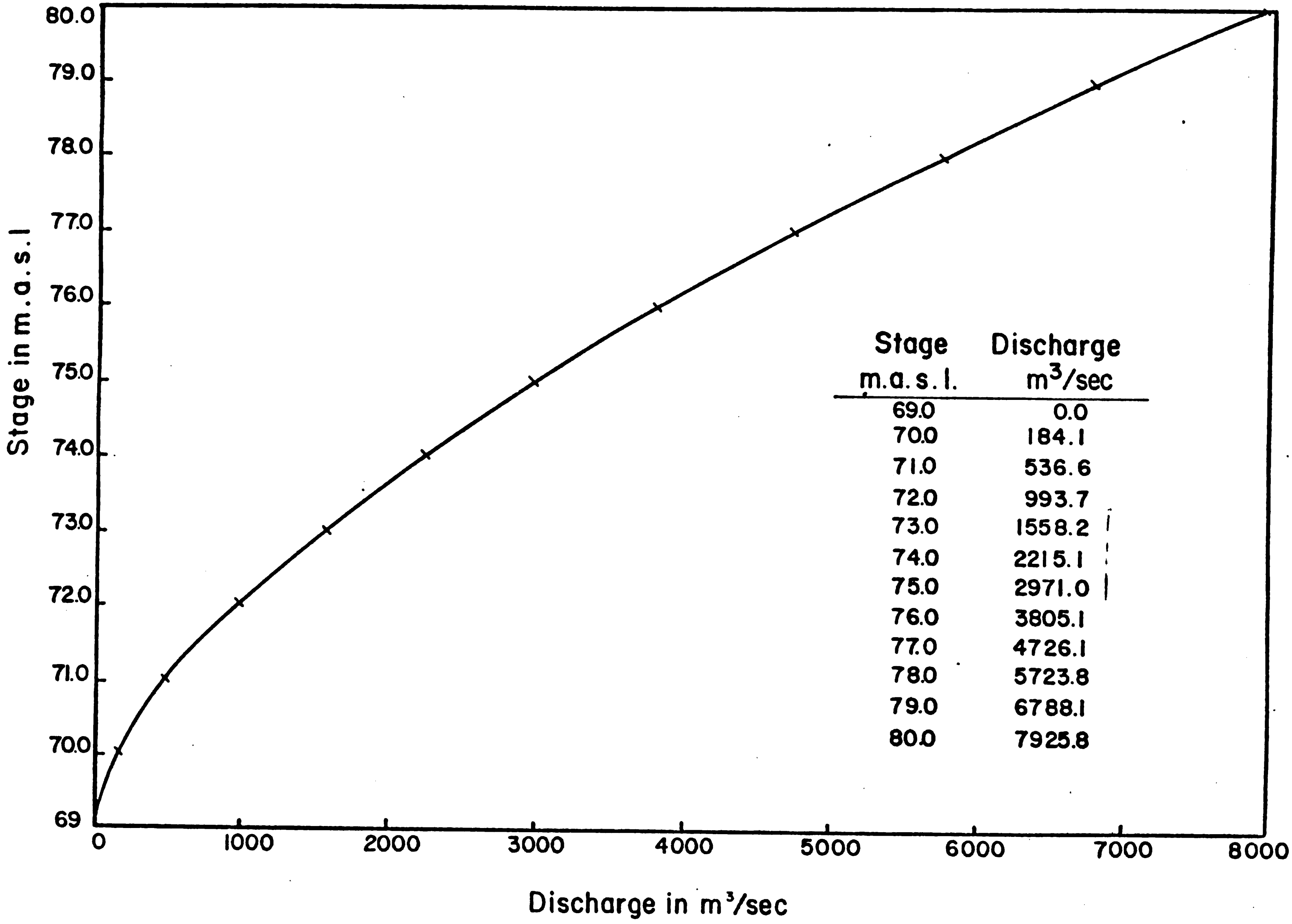
B = gate width

H = head to center of gate openings

The net gate opening G_o is given by

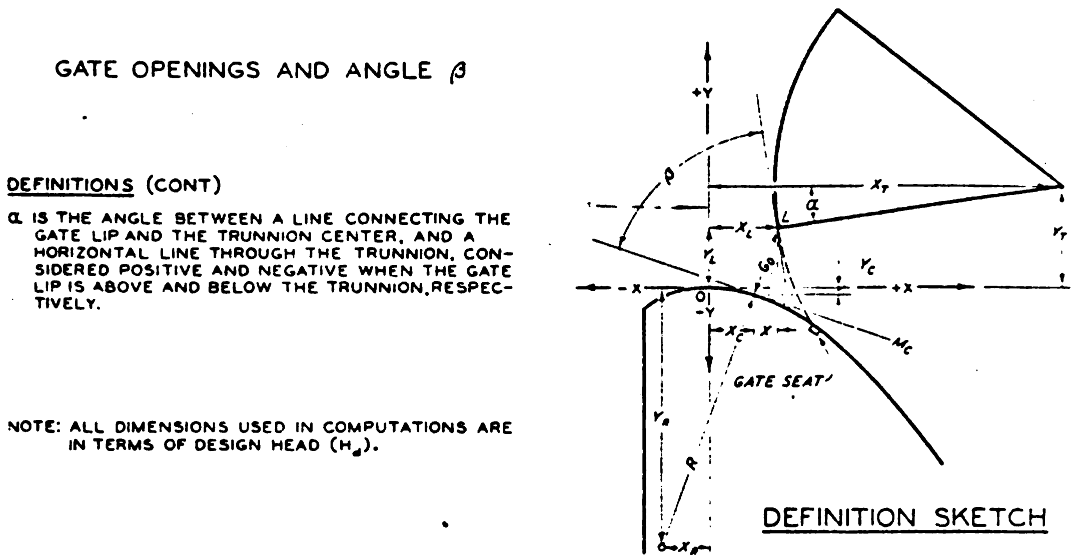
$$G_o = \sqrt{(X_L - X_C)^2 + (Y_L - Y_C)^2}$$

where X_L, Y_L, X_C, and Y_C are defined in the following sketch.



Stage m.a.s.l.	Discharge m^3/sec
69.0	0.0
70.0	184.1
71.0	536.6
72.0	993.7
73.0	1558.2
74.0	2215.1
75.0	2971.0
76.0	3805.1
77.0	4726.1
78.0	5723.8
79.0	6788.1
80.0	7925.8

Figure 3.5.6. Stage-discharge curve for uncontrolled flow over Las Barias spillway.



Definition sketch for computation of G_o and β.

Since the X_C coordinate is not readily avaliable it was computed by filling a polynomial of the form $y = kx^b$ to the spillway crest. The Newton-Raphson method was used to solve for X_C for given coordinates of X_L and Y_L in order to avoid a trial and error procedure.

The coefficient of discharge was computed from Chart 311-1 of the Hydraulic Design Criteria (Figure 2.90) given by the U.S. Army Corps of Engineers. The suggested design curve for $X/H_o = 0.1$ to 0.3 was used. This requires the computation of angle β shown in Figure 3.5.7.

The computation of controlled release using the method by the U.S. Army Corps of Engineers is more complicated, but can be implemented on a computer without much difficulty. For multi-bay gates, the total release is obtained by summing the discharge through gates which are opened. For realistic application, it is necessary to consider the stage-discharge relationships for various combination of gate openings.

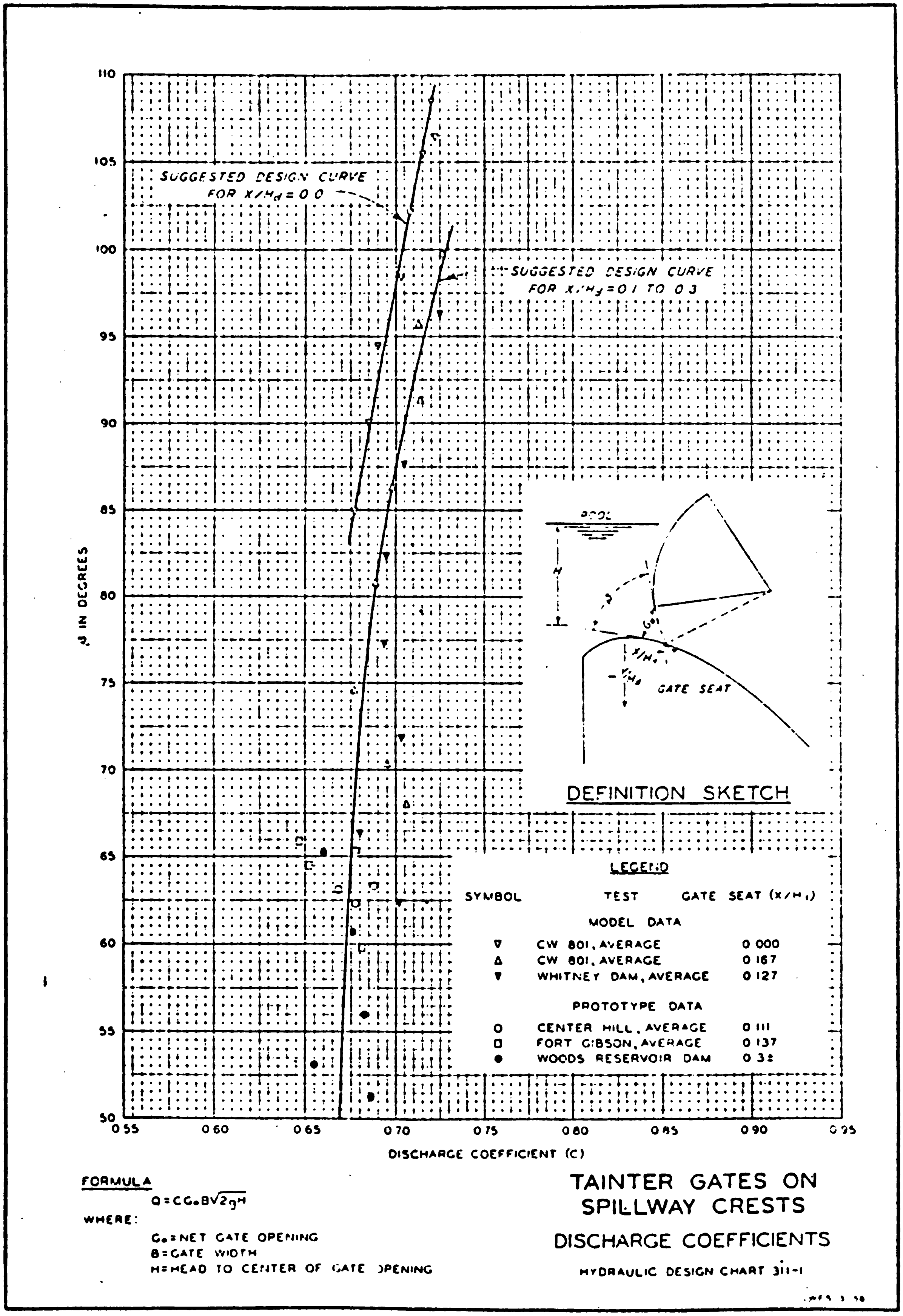


Figure 3.5.7. Discharge coefficients for tainer gates on spillway crests (U.S. Army Corps of Engineers).

These are referred to as gate operation schedules. There are three gate operation schedules following the existing operation rule for Valdesia Dam, namely

Schedule I - The centermost gate (No. 3) in operation

Schedule II - Gate No. 3 fully opened and two adjacent gates (No. 2 and No. 4) in operation

Schedule III - Gate No. 2, 3, and 4 fully opened and the last two gates (No. 1 and No. 5) in operation

The completion of Schedule III corresponds to full opening of all gates which is the case of uncontrolled release described earlier. The stage discharge curves for controlled flows through Valdesia Gates for the different operation schedules are as given in Figure 3.5.8. Similar curves have been developed for the Las Barias gates (see Figure 3.5.9). Four gate operation schedules have been formulated for Las Barias and the details are as described in the legend of Figure 3.5.9.

In addition to the gated spillways, there are 2 other release outlets which are of smaller capacity in the Valdesia Dam. They are the sluice valves whose primary purpose is to release irrigation flows, and the hydropower intake for supply to the hydropower units. The sluice valves outlets (2 units) are located at elevation 96.5 m with a combined maximum capacity of 49.5 m^3/sec at the normal full pool level of 150 m. The discharge through the sluice valve can be computed by the following equation:

$$Q = \left(\frac{34.5662 C_D^2 (H - 96.5)}{1 + 1.635 C_D^2}\right)^{1/2} \quad \text{(Note: for 1 valve)}$$

where C_D is the coefficient of discharge as given in Figure 3.5.10 and H is the water elevation in reservoir.

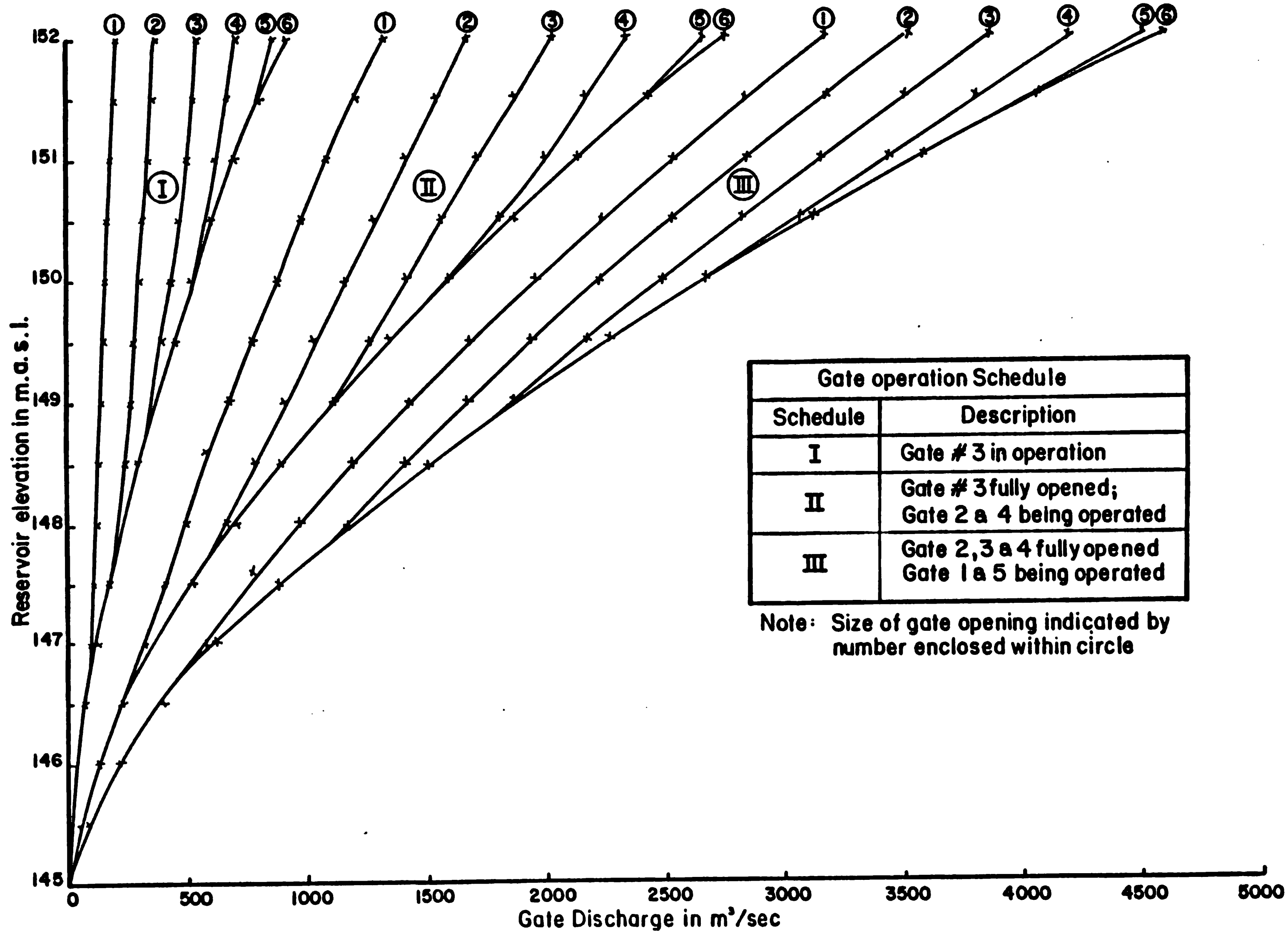
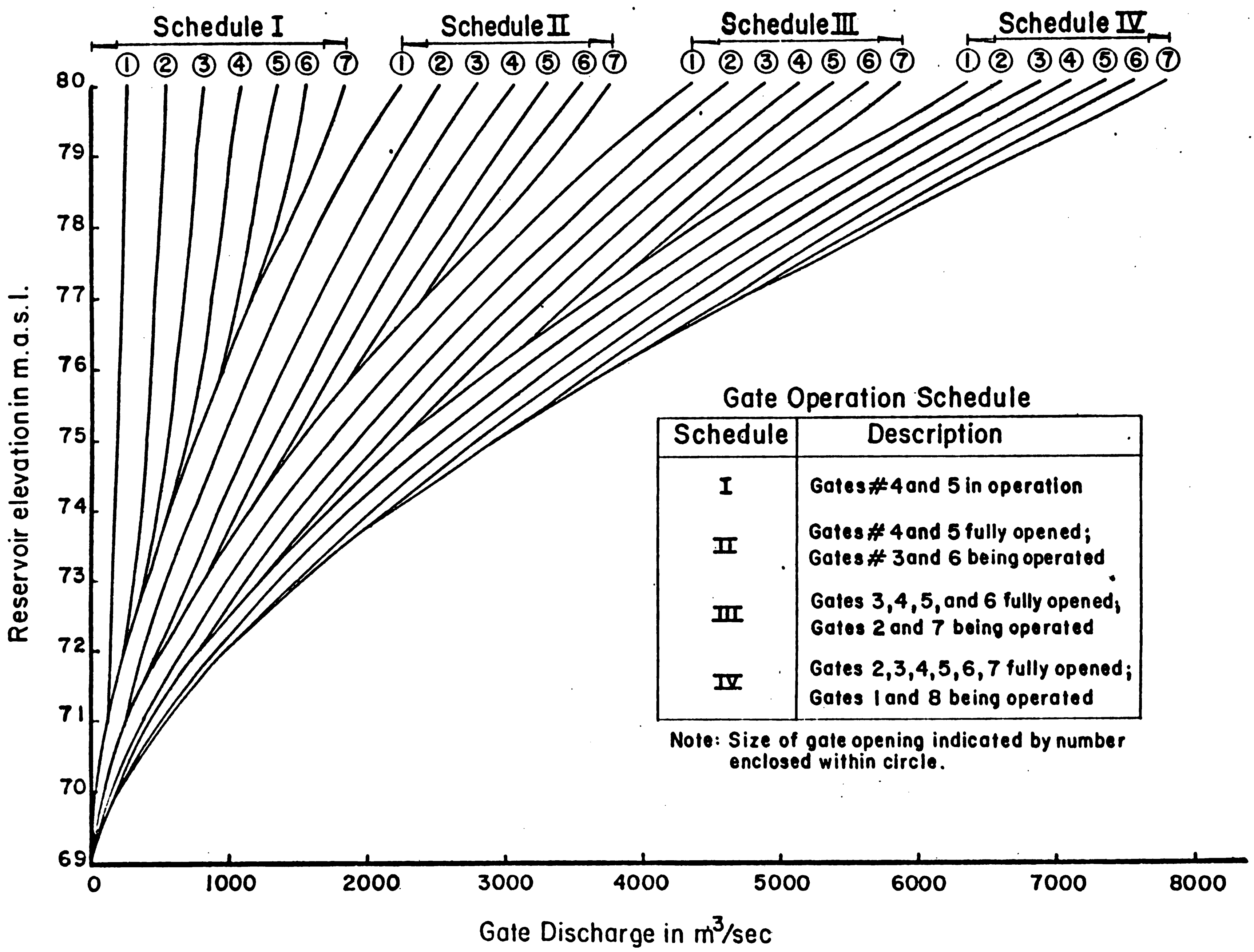


Figure 3.5.8. Stage-discharge curve for controlled flow through Valdesia gates.



Gate Operation Schedule

Schedule	Description
I	Gates # 4 and 5 in operation
II	Gates # 4 and 5 fully opened; Gates # 3 and 6 being operated
III	Gates 3,4,5, and 6 fully opened; Gates 2 and 7 being operated
IV	Gates 2,3,4,5,6,7 fully opened; Gates 1 and 8 being operated

Note: Size of gate opening indicated by number enclosed within circle.

Figure 3.5.9. Stage-discharge curve for controlled flows through Valdesia reservoir.

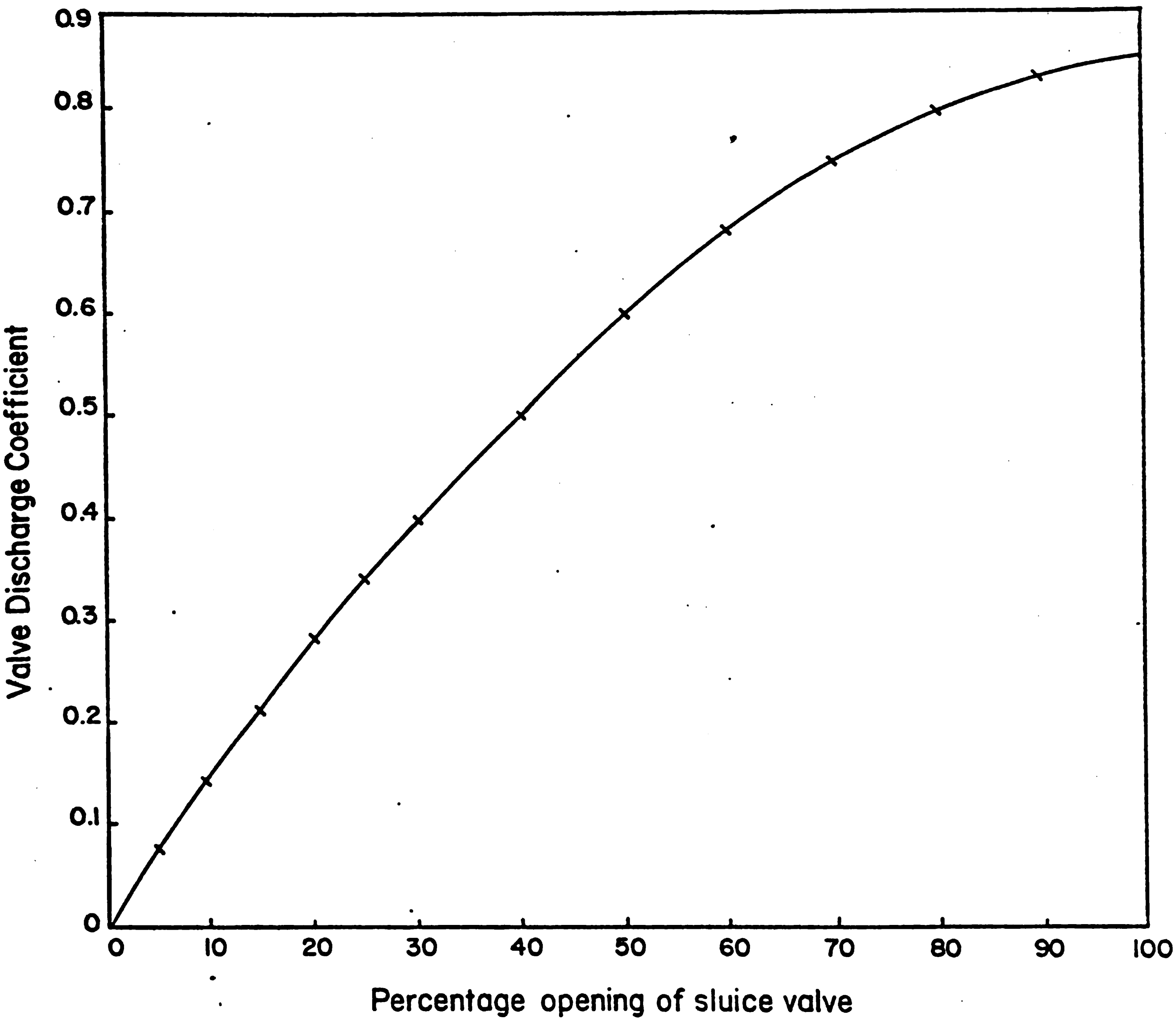


Figure 3.5.10. Discharge coefficients for sluice valve.

The release through hydropower units is dependent on the reservoir elevation and the power to be generated. The hydropower intake is located at elevation 130.75 m and it has a maximum capacity of 45 m^3/sec. Figure 3.5.11, which is developed from information and design files provided by counterparts gives the required hydropower release as a function of power generation and reservoir elevation.

Drawdown of a reservoir in anticipation of a flood is commonly used by reservoir operators for creating a larger flood control pool. With the knowledge of the stage-storage characteristics and the hydraulic relationships developed in this section, it is possible to develop a time of drawdown curve for Valdesia Reservoir. Figure 3.5.12 gives the time required for the drawdown of the reservoir from the initial normal pool level of 150 m, assuming that all the gates are fully opened and the sluices and hydropower units are operated at maximum capacity. However, no inflow into the reservoir is considered. The study has shown that the reservoir can be drawn down to the crest level of spillway (145.0 m) in about 24 hours, but further lowering of the reservoir elevation requires a much long time since it is restricted by the capacity constraints in the sluice and hydropower units.

3.5.4 Induced Surcharge Curves

The operation of gated spillway has received much research and attention in the United States and a comprehensive treatment of this subject is given in an Army Corps of Engineers Publication entitled "Engineering and Design, Reservoir Regulation" (May 1959). The following operational requirements have been suggested by the above manual (page 12) for gated spillways:

(i) Peak release from the reservoir should not exceed peak rates

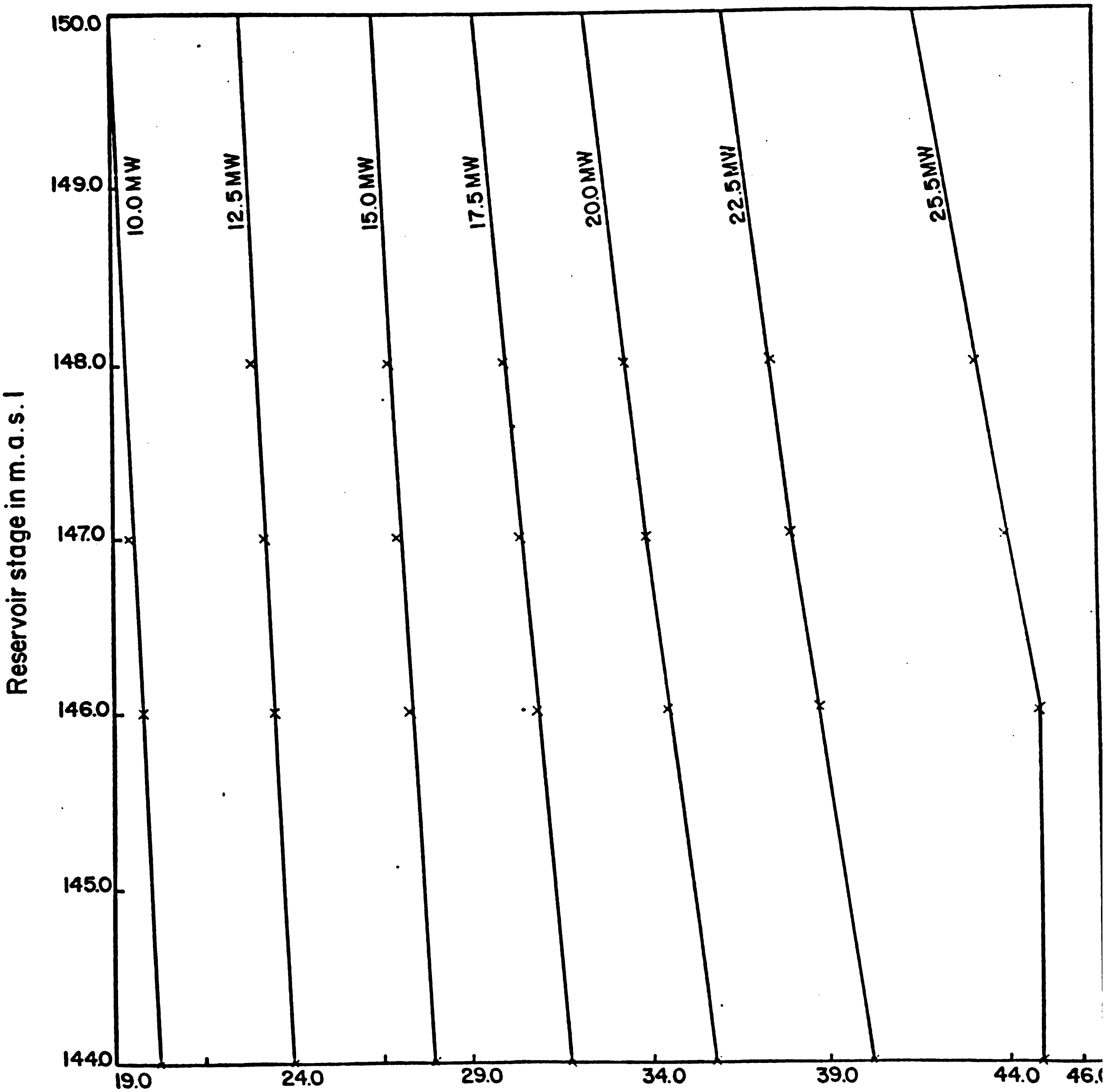
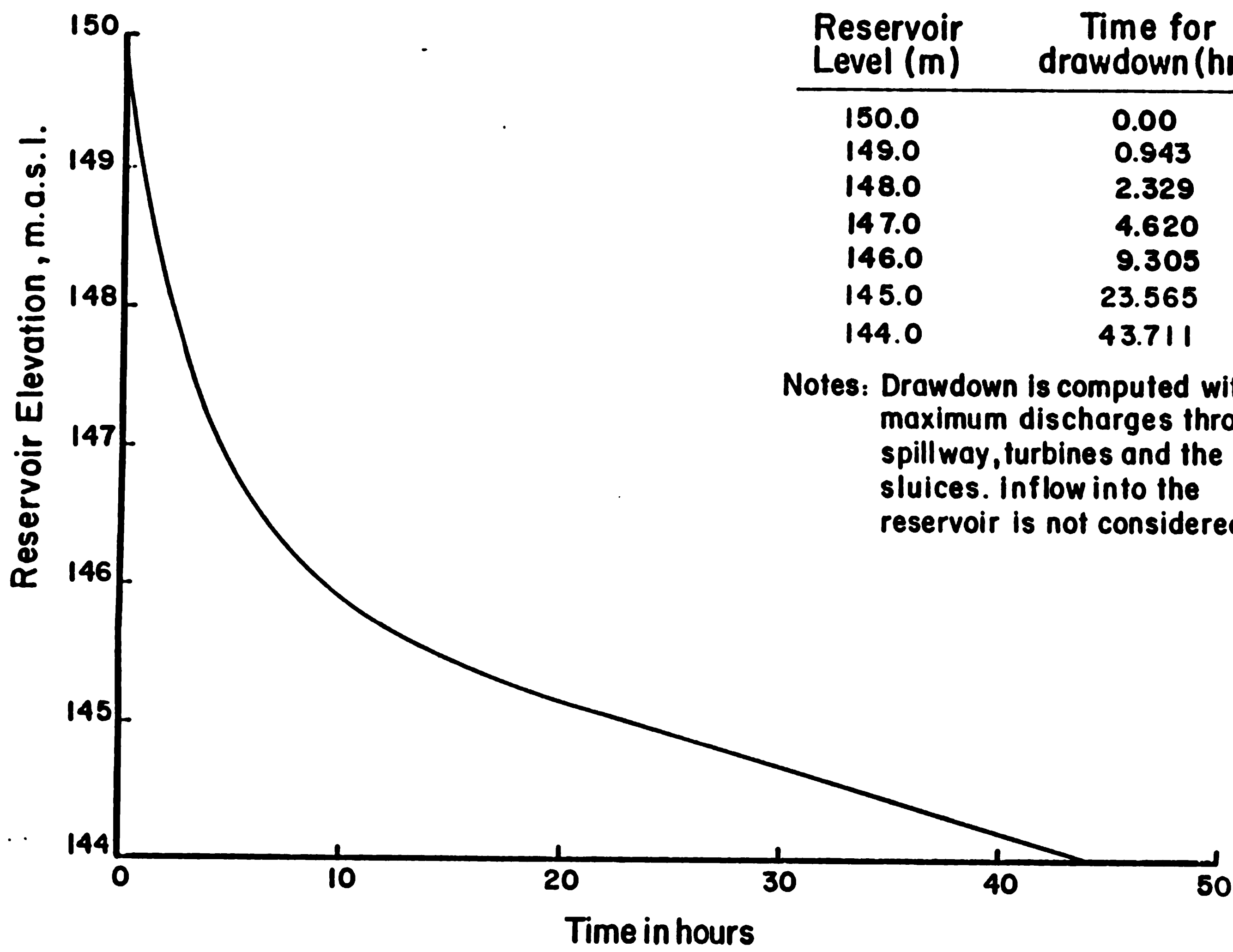


Figure 3.5.11. Hydro-release required for specified level of power output.



Reservoir Level (m)	Time for drawdown (hrs)
150.0	0.00
149.0	0.943
148.0	2.329
147.0	4.620
146.0	9.305
145.0	23.565
144.0	43.711

Notes: Drawdown is computed with maximum discharges through spillway, turbines and the sluices. Inflow into the reservoir is not considered.

Figure 3.5.12. Drawdown time for Valdesia reservoir.

of corresponding floods that would have occurred under runoff conditions prevailing before construction of the reservoir.

(ii) The rate of increase in reservoir releases during a significant amount of time should be limited to values that would not constitute a major hazard to downstream users.

To meet the above objectives it is suggested that the induced surcharge storage above the static full pool level be utilized to exercise partial control over the outflow rates after the reservoir has filled to the static full pool level. Depending on the design of the gate structures and the extent of flood damage to upstream properties, the tolerable induced surcharges may range from 1 to 2.5 m. Once the induced surcharge envelop curve is defined, it is possible to develop a set of operation curves to guide the operator on the magnitude of reservoir release given the flood inflows and the current reservoir elevation.

The above mentioned manual also gives a detailed derivation of the storage routing equation using the induced surcharge method. The basic concept of this technique (page 14 in the manual) is to set a level of outflow, generally smaller than the inflow, such that the balance of inflow can be contained within the given induced surcharge storage limits. Using some simplifying assumption, the following equation has been derived

$$S_A = 2T_s(Q_1 - Q_2(1 + \log_e \frac{Q_1}{Q_2})$$

where S_A = the remaining storage (i.e. storage to reach the induced surcharge limit)

Q_1 = inflow

Q_2 - outflow

T_s - a time constant equal to the time for the hydrograph to recess to 30.73% (1/217) of its original value

The direct solution of the above equation for outflow (Q_2) is not possible and hence an iterative approach is adopted. With a known reservoir elevation and given the induced surcharge envelope, S_A is first computed. For any known value of inflow (Q_1), it is possible to solve iteratively for a value of outflow (Q_2) which satisfies the above relationship. For practical real-time application, operation curves are more useful. However, the values of T_s may differ from one storm to another and hence a series of curves are required. For Valdesia catchment, a study of past flood-producing storms showed that T_s values range from 6 to 15 hours with the shorter time corresponding to the more intense hurricane type of storm. Two sets of operation curves have been developed, one for T_s = 6 hours (see Figure 3.5.13) and the other for T_s = 12 hours (see Figure 3.5.14). The operator has to make an estimate for T_s depending on the type and severity of storm prior to using these curves. Lower values of T_s lead to higher values of outflow and hence more conservatism in operation.

The induced surcharge method as described in the earlier section requires knowledge of reservoir inflow, the estimation of which may not be easy. Since it is possible to compute indirectly the reservoir inflow if the rate of rise (or fall) of the reservoir level is known, it is more convenient to work with the latter parameter. For this purpose, a second set of curves have been devloped in which the required release can be read off directly once the rate of rise of reservoir level is

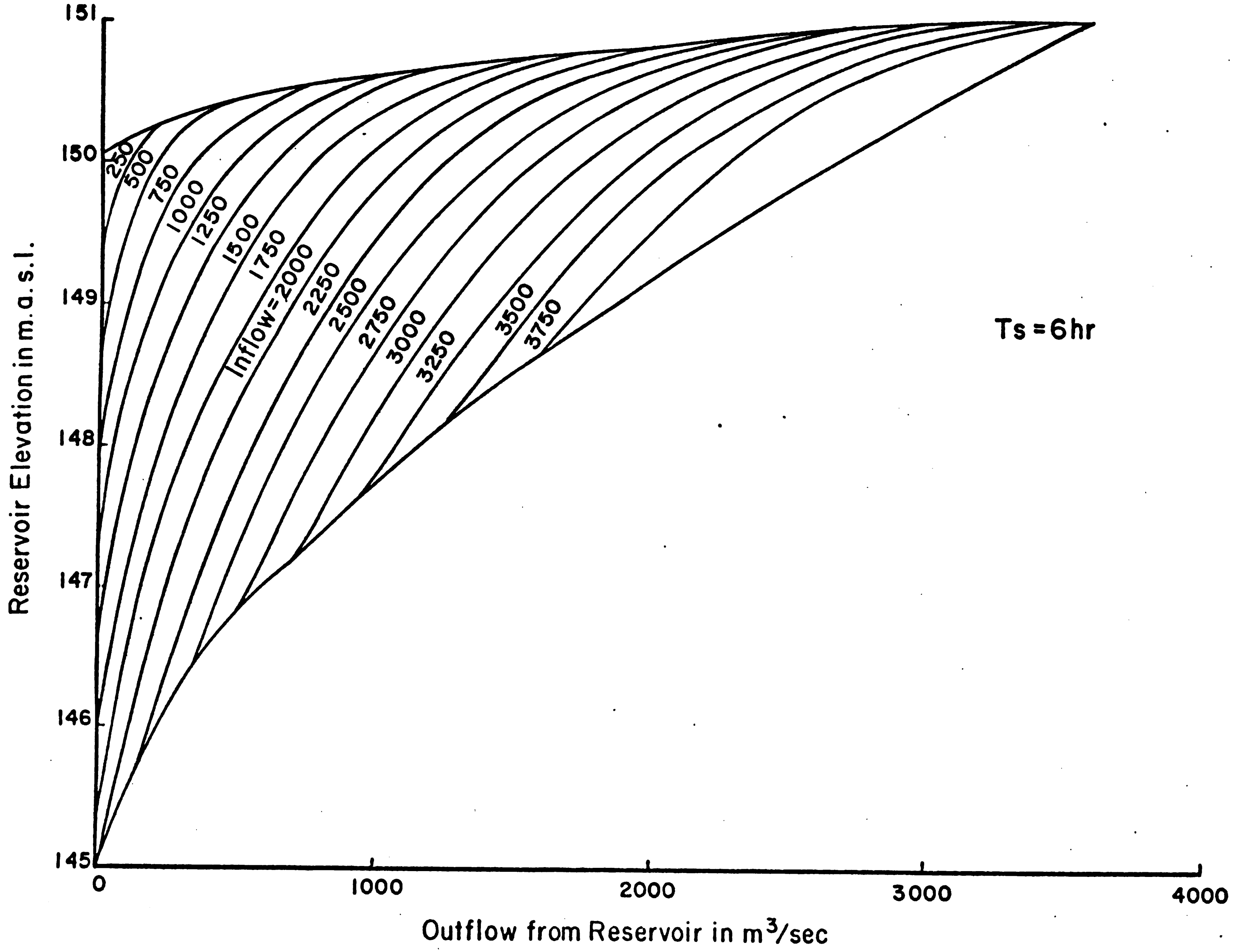


Figure 3.5.13. Induced surcharge computations for known inflow, T_s=6 hours.

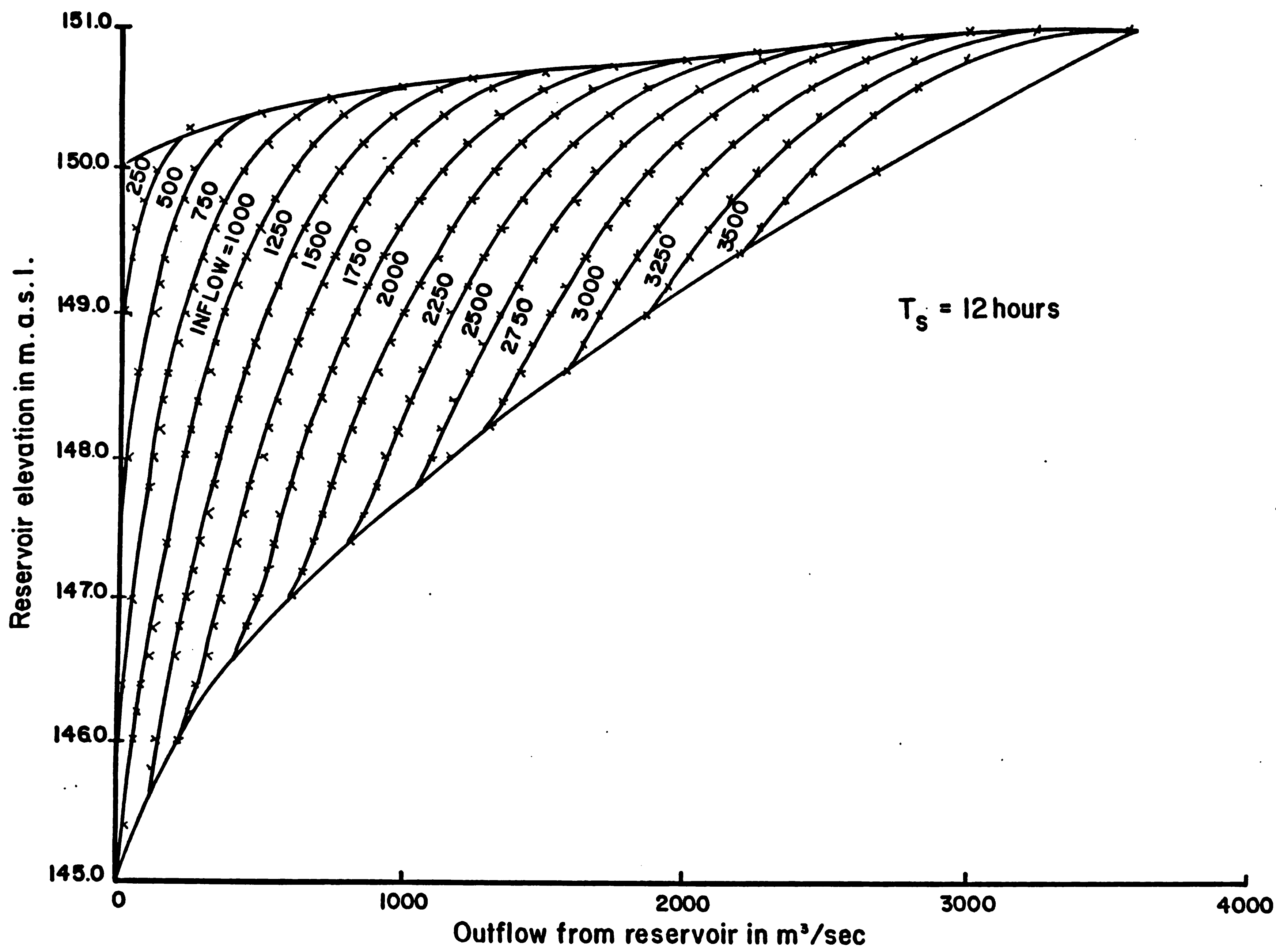


Figure 3.5.14. Induced surcharge computations for known inflow, T_s=12 hours.

known. These curves are given in Figure 3.5.15 (for T_s = 6 hours) and in Figure 3.5.16 (for T_s = 12 hours).

In the context of operation of the Valdesia Reservoir, the concept of induced surcharge may not be directly relevant since the primary concern is on dam safety and downstream flood damage is not considered. However, the use of this concept could be advantageous during the smaller floods and also when the reservoir level is low. In this way, some of the flood inflows can be saved in the reservoir for use at a later date while at the same time, the operator is confident that the outflows are still large enough to ensure that the safety of the dam is not compromised. For this reason, the induced surcharge operation is given together with two other modes of operation to provide more flexibility in the emergency operation of the Valdesia - Las Barias system.

3.5.5 Gate Regulation Schedule

In a reservoir system involving gated spillways, operation staff will be most concerned with how the gates are to be operated. Currently, the operation rule for the Valdesia Gates is as follows:

(i) When reservoir level reaches 145.0 m, the sluice valves will be opened.

(ii) When the reservoir level reaches 147.5 m, the center-most gate (No. 3) will be opened in a progressive manner such that the reservior outflow is equal to the mean inflow to the reservoir.

(iii) After gate No. 3 has fully opened and as the inflow increases, two adjacent gates (No. 2 and No. 4) will be

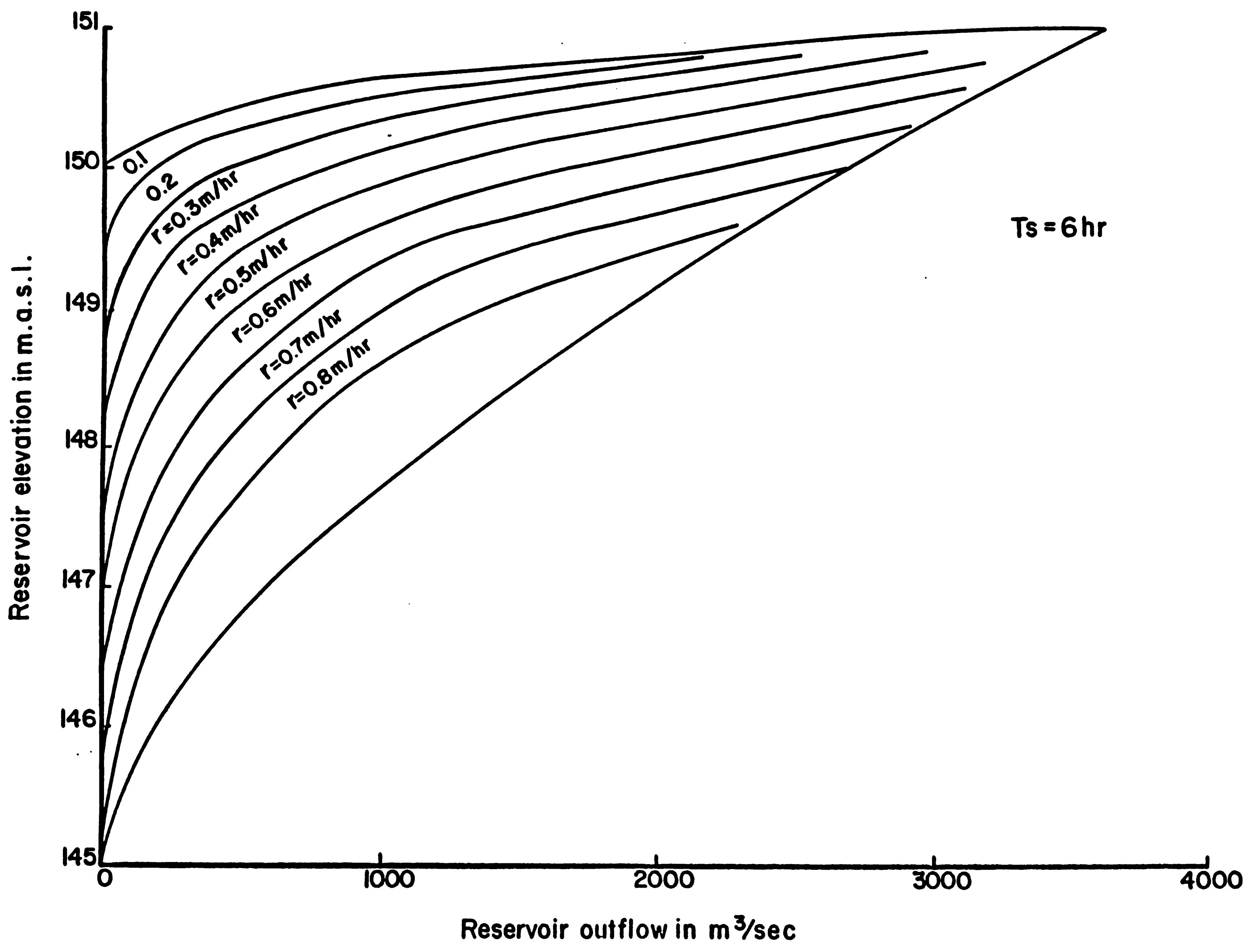


Figure 3.5.15. Induced surcharge computations based on observed rate of rise of water level in reservoir, T_s=6 hours.

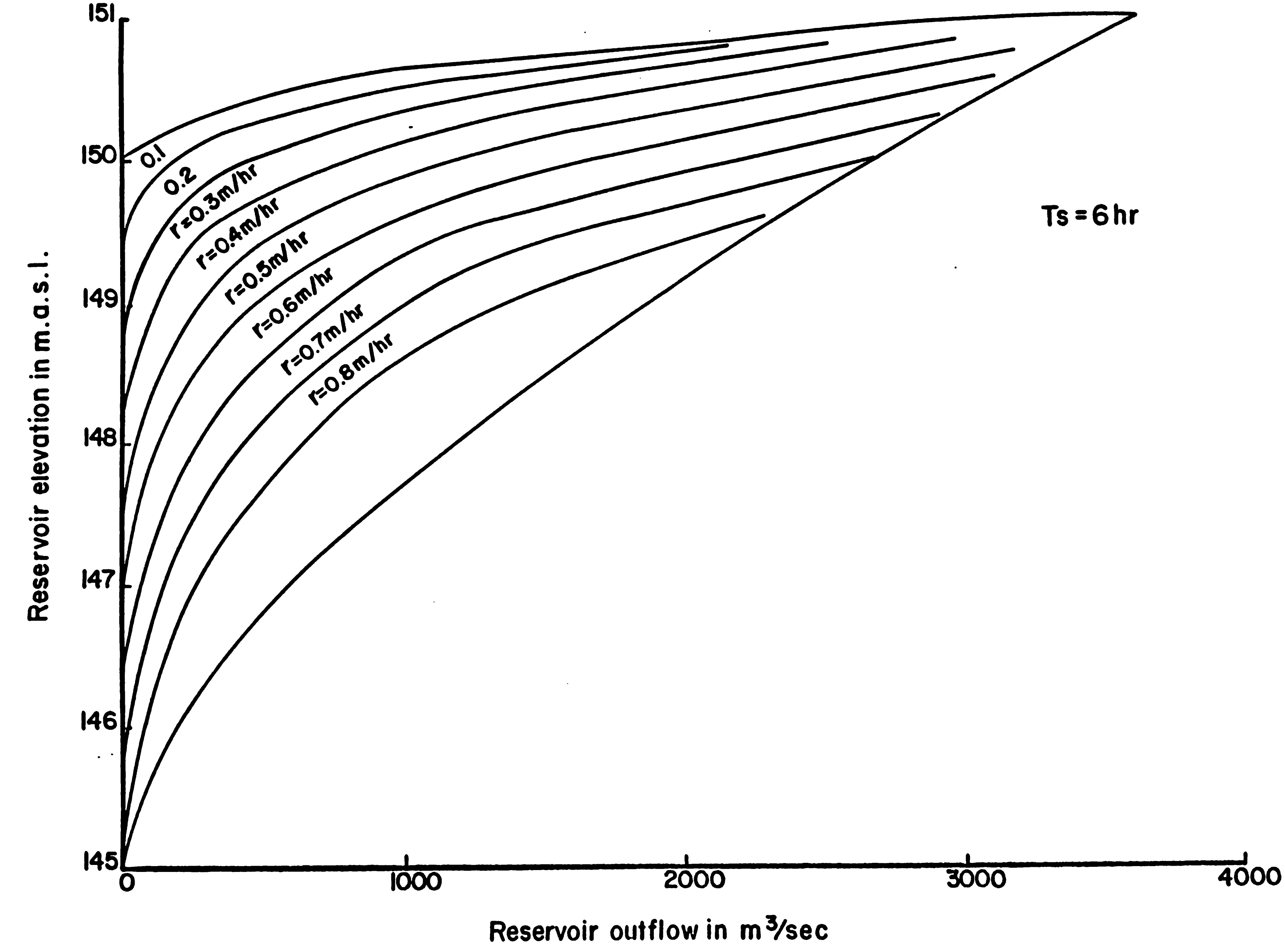


Figure 3.5.15. Induced surcharge computations based on observed rate of rise of water level in reservoir, T_s=6 hours.

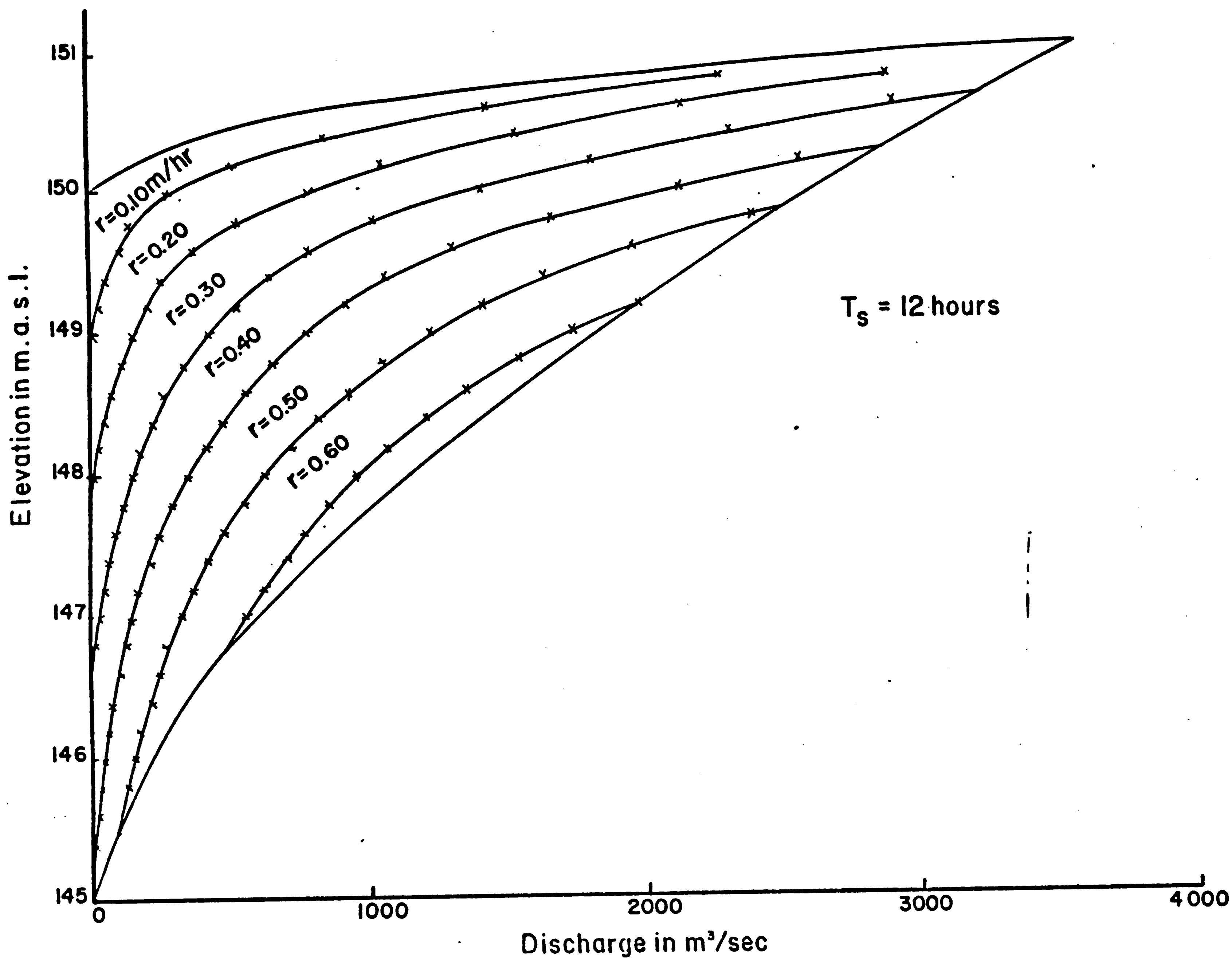


Figure 3.5.16. Induced surcharge computations based on observed rate of rise of water level in reservoir, T_s=12 hours.

opened. The process continues until the last pair of gates (No. 1 and No. 5) are fully opened.

(iv) After all the gates have been opened, the spillway enters the uncontrolled phase of operation during which the operator has no control on the outflow from the reservoir. The maximum reservoir elevation is determined by the spillway characteristics, stage-storage relationship and the inflow hydrograph. For adequate safety against overtopping, it is required that the maximum water level in Valdesia Reservoir should not exceed 154.0 m.

There is no documentation on the present operation rule of gates in the Las Barias system. Given that there are just as many gates, and the flood inflows to Las Barias are the direct outflows of Valdesia, one would expect a similar mode of operation to be practiced in Las Barias. The maximum tolerable flood surcharge level in Las Barias is 79.5 m. Reservoir levels in Las Barias, however, can be expected to vary much more rapidly because of the limited storage in this reservoir.

The existing operation rule for the gates represents a sound tradeoff between storage conservation and dam safety. Operating the gates in sequences involves more work on the part of the operator, but it helps to lessen the risk of depleting useful storage resulting from a mis-judgement of the flood inflows. For this reason, it is proposed to retain the existing method of operation but with a slight modification and concentrate on developing useful operation guide and a computerized model for carrying out such operation. The details of such a computer operation model will be described in Section 3.5.7.

As explained earlier, the gate regulation schedule for Valdesia Dam has been modelled on the existing operation rule. For the computer model, this is represented by three schedules as follow:

Schedule I - Center most gate (No. 3) will be opened

Schedule II - Gate No. 3 fully opened and two adjacent gates (No. 2 and No. 4) will be operated

Schedule III - Gates 2, 3, and 4 fully opened and the last pair of gates (No. 1 and No. 5) will be operated

Once a required gate release is specified, it is possible to determine the schedule of gate operation and the gate opening. In the computer model, this is carried out automatically. However, one could arrive at the same result using the stage discharge curve for controlled release (Figure 3.5.8). There is, however, a need for proper judgement on the number of gates to be opened when the reservior level is low. This is to avoid opening too many gates to pass a rather small outflow when a better policy will be to force some of these flows into storage and thereby build up the hydraulic head to enable a larger discharge through the gates. A suitable method to achieve the above is to implement a parallel operation scheme which limits the number of gates that are allowed open. For Valdesia Dam, the following rule is suggested

Reservoir Level	Max. no. of gates allowed open
less than 147.0 m	0
147.0 m to 148.0 m	1
148.0 m to 149.0 m	3
greater than 149.0 m	no restriction

In the case of Las Barias, four gate regulation schedules have been conceived. They are as follow:

Schedule I - The center most pair of gates (No. 4 and No. 5) are operated

Schedule II - Gates 4 and 5 are fully opened, gates No. 3 and No. 6 in operation

Schedule III - Gates 3, 4, 5, and 6 fully opened, gates No. 2 and No. 7 in operation

Schedule IV - Gates 2, 3, 4, 5, 6, and 7 fully opened and gates No. 1 and No. 8 in operation

Similarly the gate operation schedule for Las Barias can be read off directly from Figure 3.5.9 once the required release has been determined. For reasons explained earlier in the case of Valdesia, it is also necessary to have a parallel operation scheme to limit the number of gates that are allowed opened when the reservoir levels are low. The proposed scheme is as follows:

Reservoir level in Las Barias	Maximum No. of gates allowed open
Less then 71.0	0
71.0 to 73.0	2
73.0 to 74.5	4
Greater than 74.5	No restriction

The above restrictions on gate opening specify the modifications made to the current operating procedure which appears in documents made available by the counterparts.

The gate regulation schedules as described earlier are primarily concerned with the treatment of the rising limb of the inflow hydrograph. However, the operator must also be provided with a guide on

how to manage the releases when the hydrograph has peaked and is recessing. There is no standard approach in dealing with such a situation and hence the following strategy is used:

(i) The reservoir level should return to its original level (the normal full pool level) as quickly as possible so that it is ready to cope with a subsequent flood event.

(ii) The release from the reservoir should be controlled such that it will approach or be maintained at the normal full pool level.

A suitable operation strategy that will achieve the above is to hold the gates at their position settings, just prior to the recession of flood inflows, for a sufficient period of time until the reservoir has fallen to the normal full pool level and thereafter maintain a gate opening that equates outflow to inflow.

The gate operation schedules proposed in this section have been tested extensively by reservoir operation studies using the computer model (to be described in section 3.5.7) and have been found to give satisfactory results under most circumstances. These rules represent the best judgement at this stage of development. They are by no means final and may be refined from time to time as better experiences are gained in the operation of the system.

3.5.6 Modes of Operation

In Section 3.5.4, the induced surcharge method of determining reservoir release has been described. It was also shown that the use of this method for reservoir operation is advantageous if it is intended to restrict the magnitude and rate of reservoir release at the beginning of

the flood inflow period. This is more likely to be the case when the reservoir level in Valdesia is low. In this section, two other modes of operation will be described.

The existing operation rule has been in use by the operation staff and it is basically as case of equating outflow to mean inflow during the controllable phase of the spillway operation. This mode of operation is easy to implement. The gate operators will adjust the gate openings progressively as the inflow increases, attempting to maintain a constant reservoir pool for as long as is feasible. This mode is included with a slight modification to incorporate the restrictions on gate opening specified above.

The third mode of operation to be introduced is, strictly speaking, a prior drawdown approach. It will be referred to as the Hurricane mode of operation. When large inflows to the reservoir are anticipated, it is often advantageous to draw down the reservoir as much as possible to maximize the flood control storage space. Since the normal pool level is 150.0 m, drawing it down to 145.0 m, the crest level of the spillway will result in an additional 40.4 million m^3 of flood storage space which could help in absorbing a medium size flood. As demonstrated in section 3.5.3, such a draw down operation can be achieved in about 24 hours if all the gates and other outlet facilities are operated at maximum capacity. If the advance warning time of flood inflow is longer, further drawdown of the reservoir storage is possible although the rate of drawdown will be much slower because of capacity limitation in the outlet structures. For very large floods, the Hurricane mode of operation is advised, but it has to be cautioned that

(i) The additional flood storage space thus created may not have a significant impact in reducing the outflow peak of very large floods.

(ii) The initial surge of outflow (about 2700 m^3/sec) as the gates are opened suddenly may have damaging effects on downstream areas. Operators have, therefore, to exercise appropriate judgement in the initial operation of the gates to reduce the effect of such an outflow surge.

The Hurricane mode of operation is particularly useful in planning studies because it represents a limiting condition (in this case, a lower limit) on the outflow and reservoir level build up as the inflow hydrograph is routed through the reservoir. One could safely conclude that the results thus obtained are the maximum achievable limits under the capacity constraints of the system.

The three modes of operation have been developed to allow greater flexibility in operation of the reservoir system. For best results, the operator has to make appropriate on-site decisions based on the real-time information such as reservoir level and flood forecast which are available to him. In this respect, there is no substitute for best judgement and experience. The computer model can be used to provide predictions of results of different assumptions of operation modes, but the ultimate decision still rests with the operator.

3.5.7 Computer Model

Routing a flood inflow hydrograph through a reservoir system and determining the gate regulation schedules during the controllable phase of spillway operation is a tedious excercise. The graphs and charts developed in the earlier sections can be of tremendous help in real-time

operation, but they have to updated from time to time with the accumulation of experience or as new or better quality data becomes available. Considerable effort has therefore been expended to develop a comprehensive computer model for carrying out the flood routing operation described in this chapter.

The computer model is basically a set of interacting mathematical algorithms that attempt to simulate the working and interaction of the component works that constitue the physical system for known sets of inputs, boundary conditions and system constraints. It is basically a simulation model which provides a 'what if' type of results/response for use by the operator in decision making. When used in real-time, the model can also advise on the gate regulation schedule for direct implementation.

The computer model is included in the software package "Colorado State University Hydrologic Modeling System" which combines all the software developed for Hydrologic Studies and Emergency Operation. A complete description with examples is provided in the users manual (CSU-HMS, 1986).

3.6 FLOOD ROUTING STUDIES

The computer model developed in Section 3.5 was used to study the routing of various historic and design floods through the Valdesia-Las Barias system and the results of these studies are reported in this chapter.

3.6.1 Historic Flood

The flood of August 1972 is the largest historic flood where measurements of flood flows were available. This flood reached a peak discharge of 1760 m^3/sec at Valdesia and lasted for about 70 hours. It is a small flood when compared to Hurricane David or any of the design floods such as SPF and PMF.

The August 1972 flood was routed through the Valdesia-Las Barias system under 3 different modes of operation:

Mode 0 - Induced Surcharge Method

Mode 1 - Outflow Equals Inflow

Mode 2 - Hurricane Operation Mode

Two sets of initial reservoir levels were assumed in this study.

Case 1 - Valdesia Reservoir at 150 m (normal pool level) and
Las Barias Reservoir at 77 m (normal pool level).

Case 2 - Valdesia Reservoir at 145 m (spillway crest level)
and Las Barias Reservoir at 70 m.

The outflow and stage hydrograph are given in 8 plots.

Case 1 - Figure 3.6.1 to 3.6.4.

Case 2 - Figure 3.6.5 to 3.6.8.

The results of routing under different modes of operation and initial reservoir levels are summarized in Table 3.6.1 and 3.6.2.

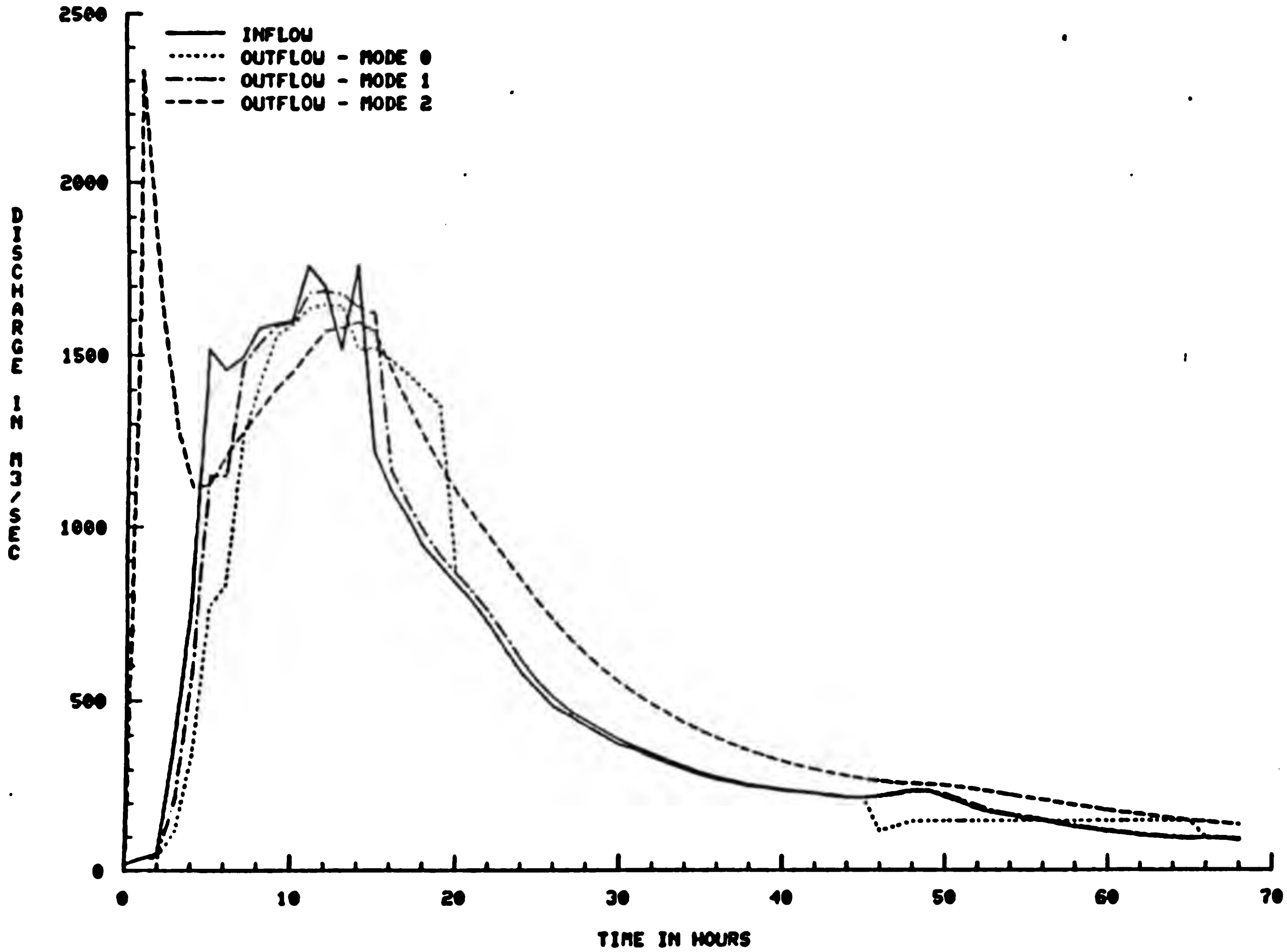


Figure 3.6.1. Inflow and simulated outflow hydrographs of May 1972 flood at Valdesia reservoir - Initial water level of 150 m.

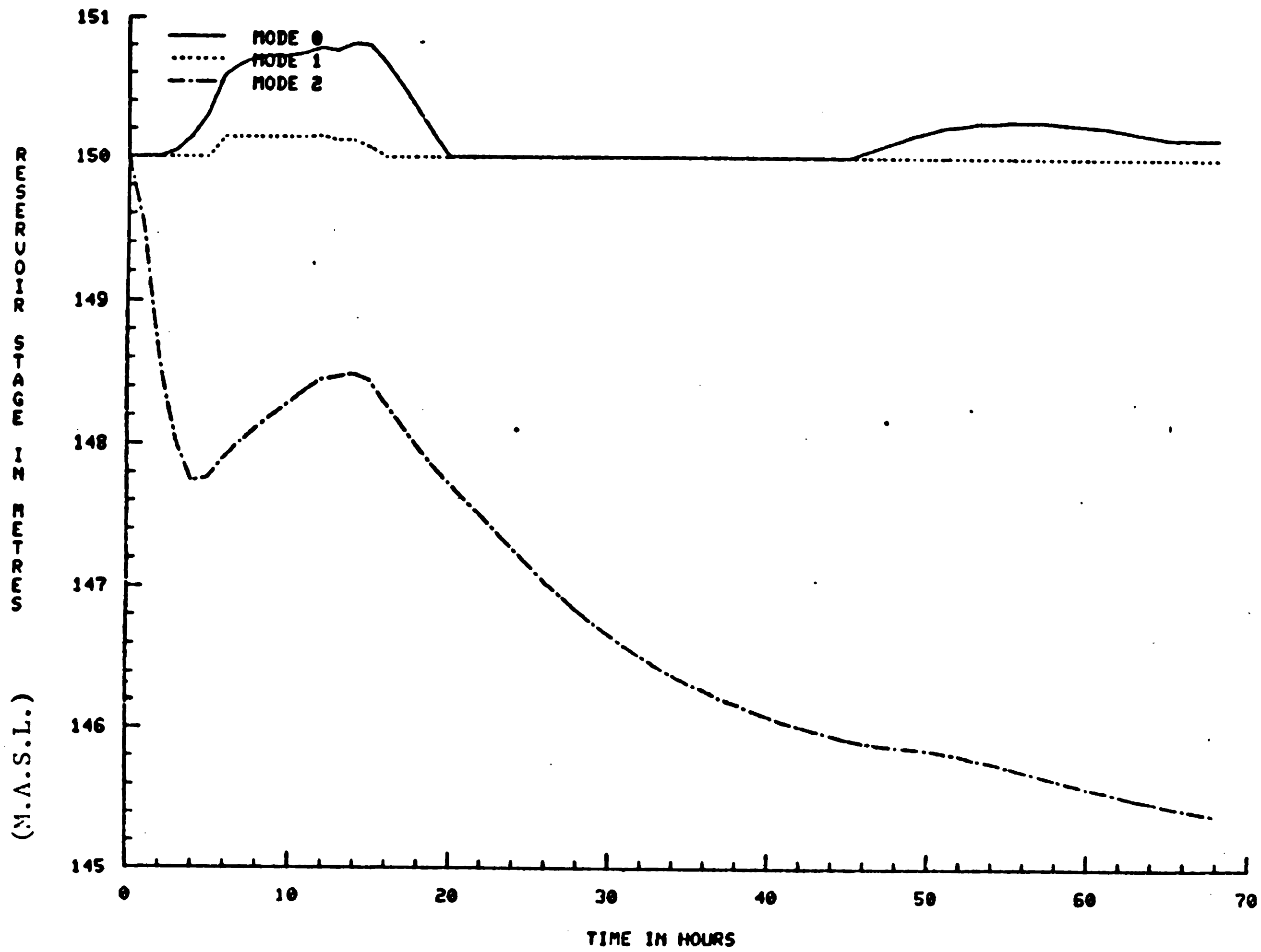


Figure 3.6.2. Simulated stage hydrograph of May 1972 flood at Valdesia reservoir - Initial water level of 150 m.

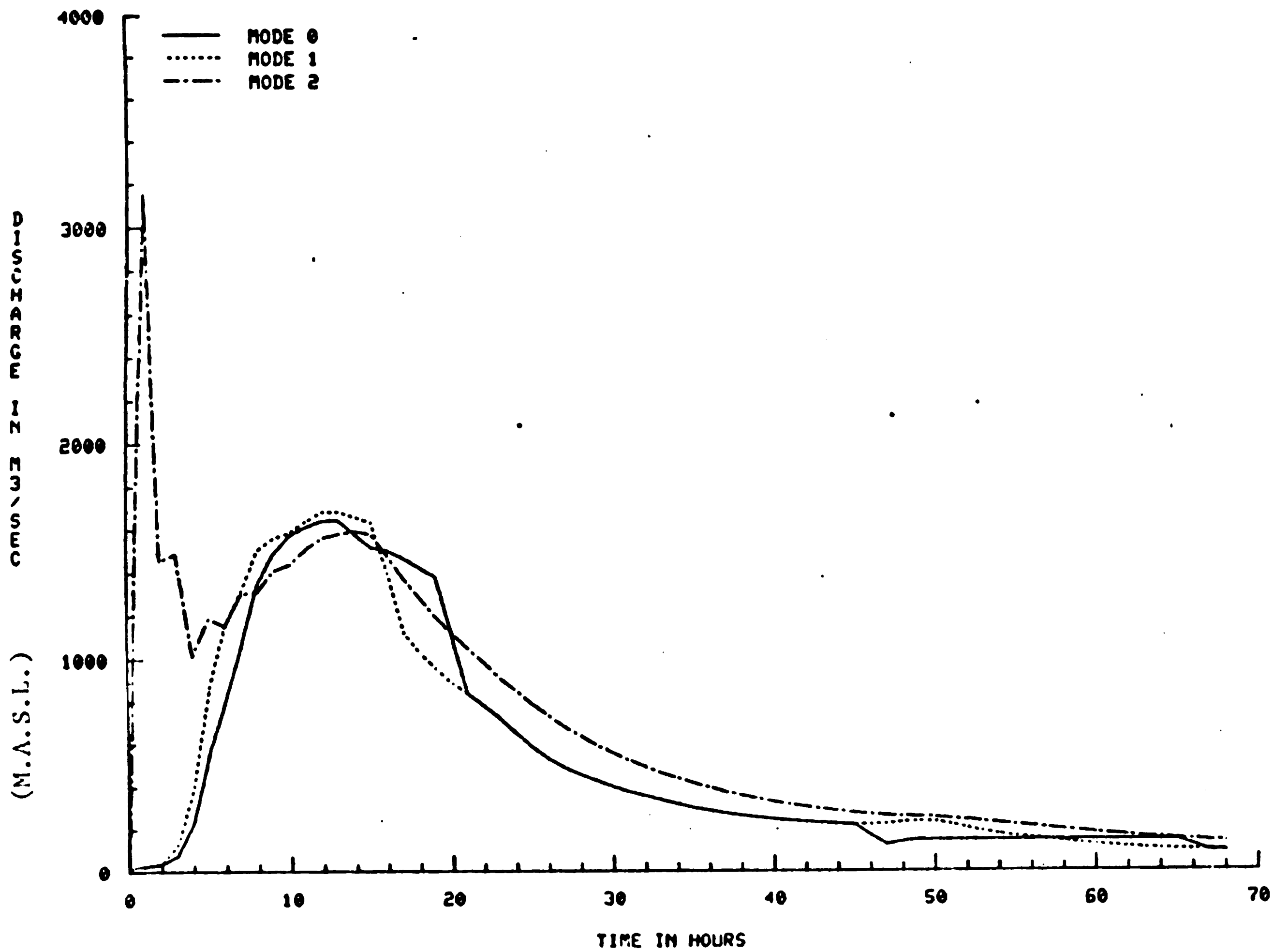


Figure 3.6.3. Simulated outflow hydrograph of May 1972 flood at Las Barias reservoir - Initial water level of 77 m.

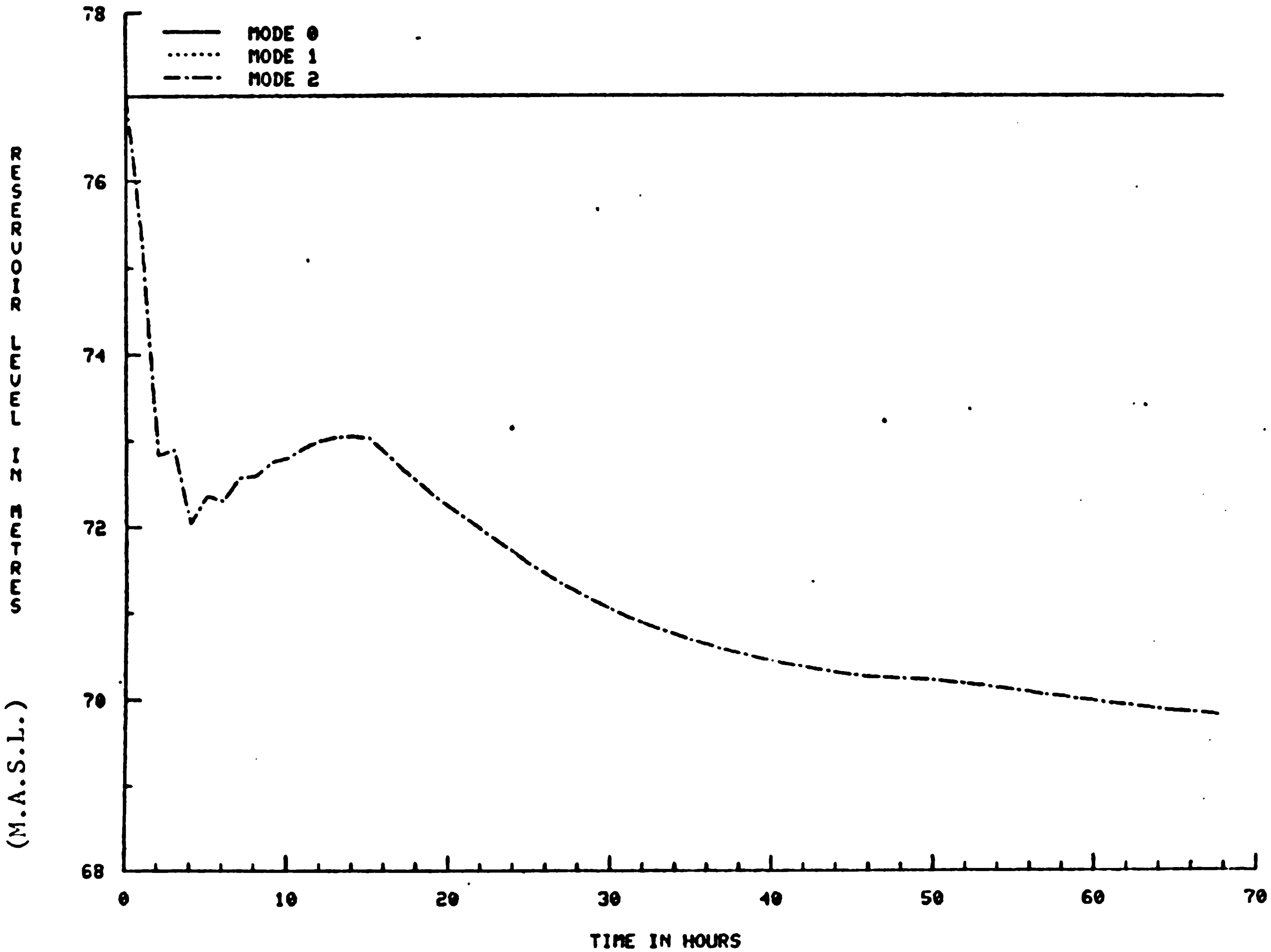


Figure 3.6.4. Simulated stage hydrograph of May 1972 flood at Las Barias reservoir - Initial water level of 77.0 m.

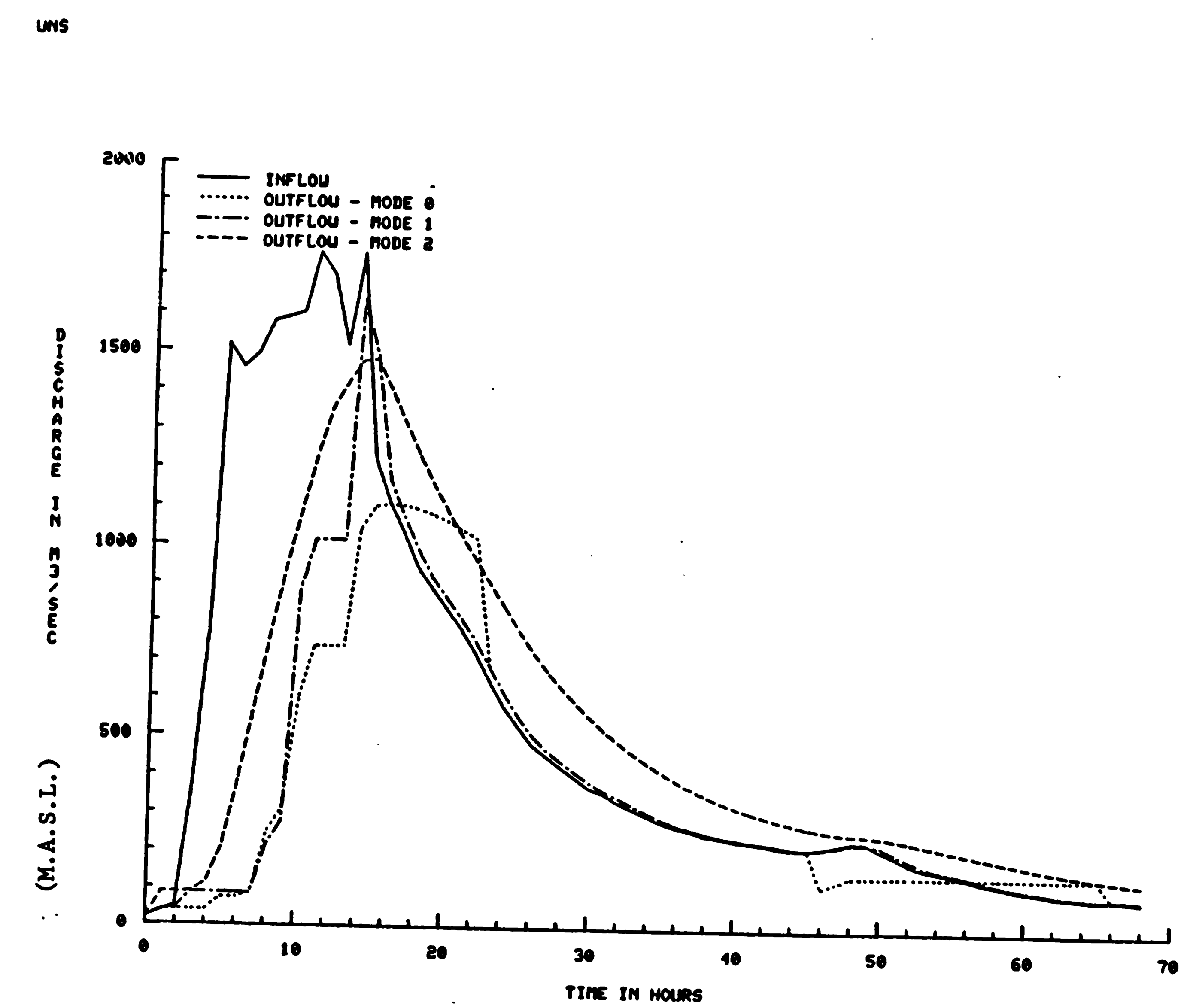


Figure 3.6.5. Inflow and simulated outflow hydrographs of May 1972 flood at Valdesia reservoir - Initial water level of 145 m.

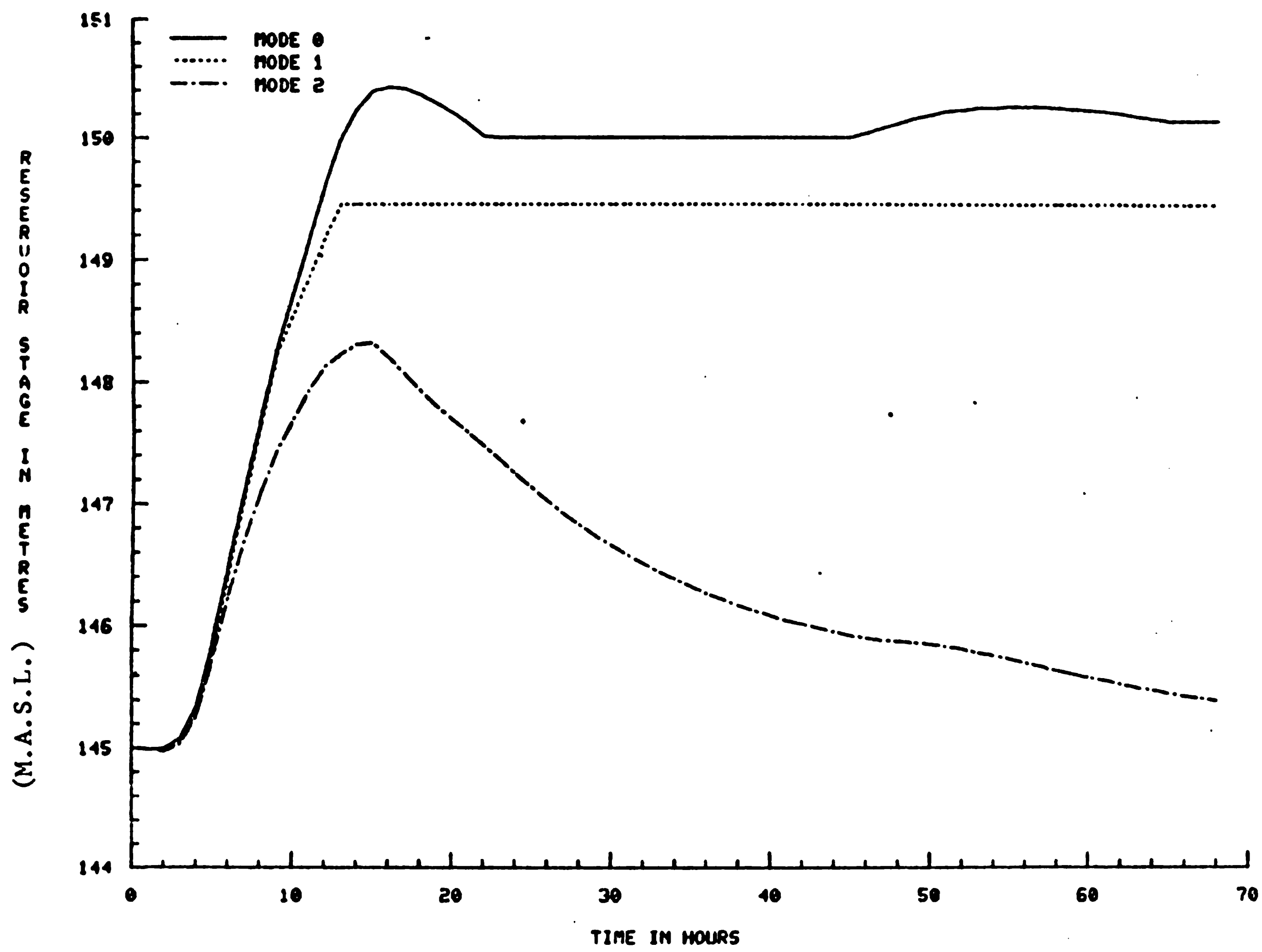


Figure 3.6.6. Simulated stage hydrograph of May 1972 flood at Valdesia reservoir - Initial water level of 145 m.

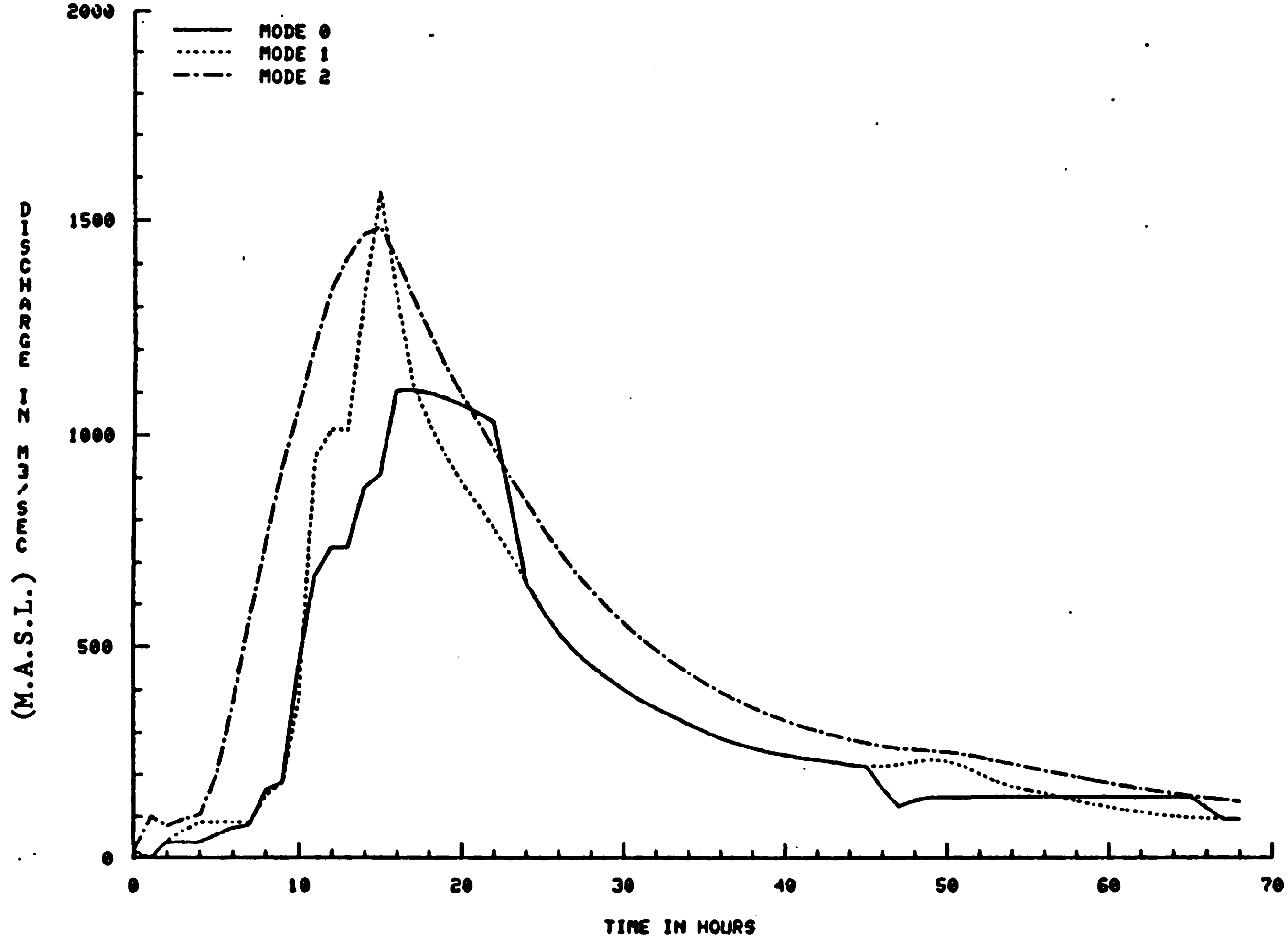


Figure 3.6.7. Simulated outflow hydrographs of May 1972 flood at Las Barias reservoir - Initial water level of 77.0 m.

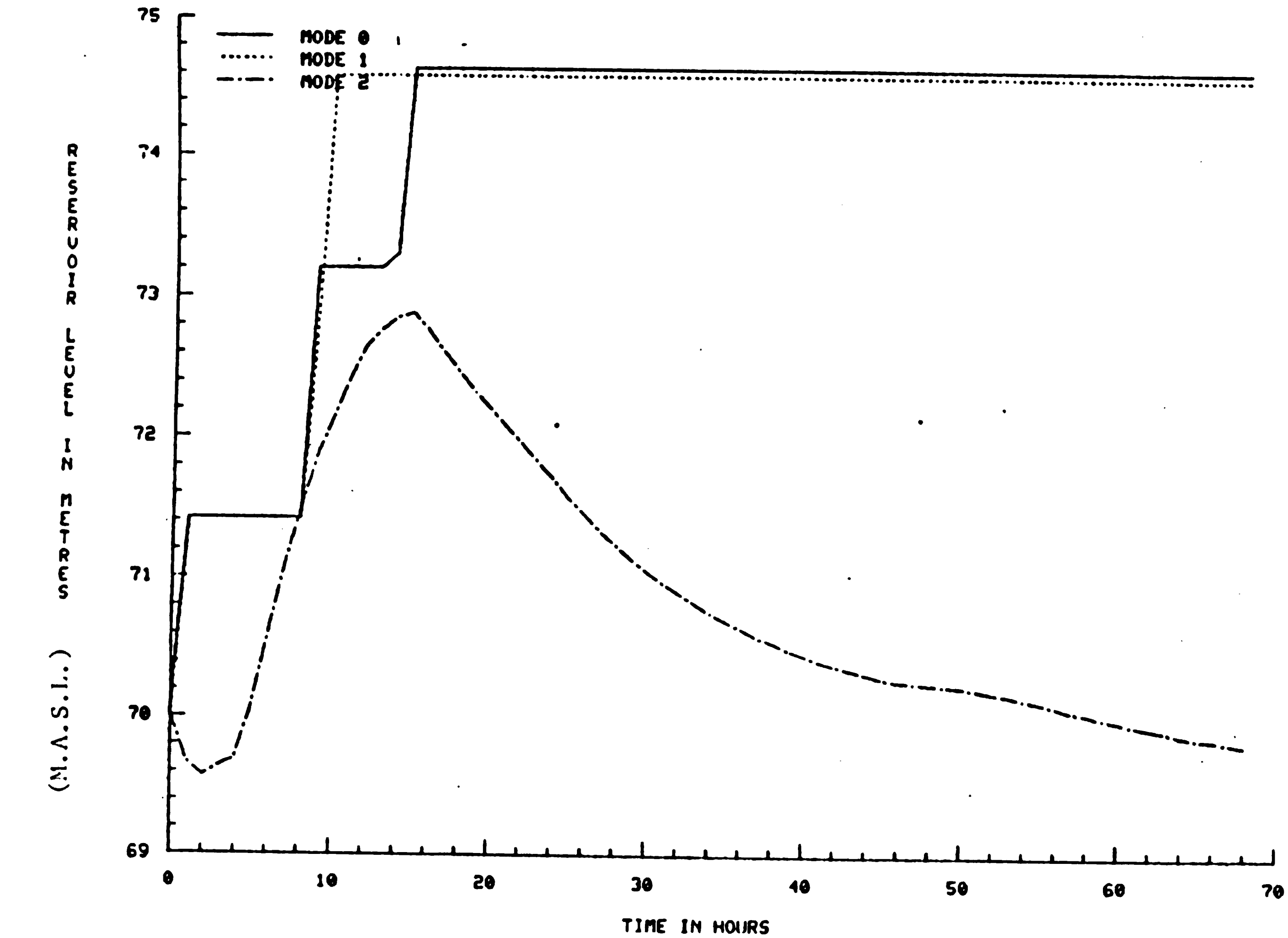


Figure 3.6.8. Simulated stage hydrographs of May 1972 flood at Las Barias reservoir - Initial water level of 77.0 m.

Table 3.6.1. Routing May 1972 Flood with Initial Reservoir Levels of 150 m at Valdesia and 77.0 m at Las Barias

DESCRIPTION	MODE 0	MODE 1	MODE 2
<u>Valdesia Reservoir</u>			
Inflow Peak (m^3/sec)	1760	1760	1760
Outflow Peak (m^3/sec)	1647	1686	1595*
Max. Reservoir Elevation (m)	150.81	150.15	150.00
<u>Las Barias</u>			
Outflow Peak	1645	1684	1590
Max. Reservoir Elevation (m)	77.0	77.0	77.0

* Ignoring the initial outflow surge

Table 3.6.2. Routing May 1972 Flood with Initial Reservoir Levels of 145.0 m at Valdesia and 70.0 m at Las Barias

DESCRIPTION	MODE 0	MODE 1	MODE 2
<u>Valdesia Reservoir</u>			
Inflow Peak (m^3/sec)	1760	1760	1760
Outflow Peak (m^3/sec)	1107	1638	1479
Max. Reservoir Elevation (m)	150.42	149.45	148.32
<u>Las Barias</u>			
Outflow Peak (m^3/sec)	1106	1565	1484
Max. Reservoir Elevation (m)	74.63	74.58	72.88

It can be seen from the above routing studies that the hurricane mode of operation results in the lowest reservoir water elevations whilst the induced surcharge mode of operation is more effective in suppressing the outflow peaks particularly when the initial reservoir levels are lower. For a flood of this magnitude, the hurricane mode of operation is not advised for the following reasons:

(1) When the initial reservoir levels are high (for example, at the normal pool level), an initial surge of outflow will occur when the gates are suddenly opened to the full.

(2) The reservoir will be unnecessarily drawn down after the flood since gates are kept opened for the entire duration of the flood.

3.6.2 Reconstructed Hydrograph of Hurricane David

The flood resulting from Hurricane David (August 30, 1979) has been reconstructed using HEC-1 model under different assumptions of antecedent moisture conditions (i.e., AMC I, AMC II and AMC III). The reconstructed hydrographs will be used as input to the routing program.

It was reported that the flood of Hurricane David overtopped the gates at Valdesia. The initial reservoir level in Valdesia was 143.0 m and a maximum level of about 156.0 m was reached (personal communication, operator of Valdesia Reservoir). The gates at Valdesia were not opened during the entire duration of the flood. To simulate the situation of non-opening of the gates, HEC-1 program was used. It was assumed that there is no outflow from the reservoir until the reservoir elevation has built up to 150 m, after which water overflows

the gates which behave like a sharp-crested weir with a total width of 120 m. A series of hypothetical inflow hydrographs were generated by applying a scaling factor on the reconstructed David hydrograph under AMC-II condition. The results of the study are summarized in Table 3.6.3.

Table 3.6.3. Routing Reconstructed David Hydrograph (AMC-II) through Valdesia with All Gates Closed

Scaling Factor	Peak Discharge (m^3/sec) Inflow	Outflow	Peak Reservoir Elevation (m)
0.40	3491	366	151.25
0.60	5237	1155	152.70
0.80	6983	2182	154.13
0.90	7856	2763	154.84
1.00	8729	3350	155.50
1.10	9602	3971	156.16
1.20	10474	4580	156.78
1.30	11347	5229	157.40

From the above study, it can be seen that a flood with a peak of about 9000 m^3/sec would result in a peak water level of about 156.0 m in Valdesia Reservoir if the gates were kept closed. This is close to the peak of the reconstructed Hurricane David hydrograph under AMC-II condition. This finding provides a support to the validity of the HEC-1 model for rainfall-runoff simulation in the Nizao Basin.

Having obtained the reconstructed Hurricane David hydrograph under different antecedent moisture conditions, the next step of the study is to route these hydrographs through the Valdesia-Las Barias Reservoir system using the routing model that has been developed. This was carried out and the results of this study are summarized in Table 3.6.4..

Table 3.6.4. Routing Reconstructed David Hydrograph under Different Modes of Operation

AMC	DESCRIPTION	MODE 0	MODE 1	MODE 2
AMC-I	<u>Valdesia</u>			
	Peak Inflow (m^3/sec)	5332	5332	5332
	Peak Outflow (m^3/sec)	4254	3935	3226
	Peak Reservoir Level (m)	151.59	151.27	150.53
	<u>Las Barias</u>			
	Peak Inflow (m^3/sec)	4227	3931	3212
	Peak Reservoir Level (m)	77.0	77.0	77.0
AMC-II	<u>Valdesia</u>			
	Peak Inflow (m^3/sec)	8729	8729	8729
	Peak Outflow (m^3/sec)	6274	5940	5614
	Peak Reservoir Level (m)	153.42	153.14	152.85
	<u>Las Barias</u>			
	Peak Outflow (m^3/sec)	6234	5926	5552
	Peak Reservoir Level (m)	78.49	78.20	77.83
AMC-III	<u>Valdesia</u>			
	Peak Inflow (m^3/sec)	10358	10358	10358
	Peak Outflow (m^3/sec)	7529	7335	7217
	Peak Reservoir Level (m)	154.44	154.29	154.19
	<u>Las Barias</u>			
	Peak Outflow (m^3/sec)	7430	7259	7154
	Peak Reservoir Level (m)	79.57	79.42	79.33

The results of the routing studies for reconstructed hydrographs of Hurricane David are plotted in a series of outflow and stage hydrographs as in Figure 3.6.9 to 3.6.12 for AMC-I condition, Figure 3.6.13 to 3.6.16 for AMC-II condition, and Figure 3.6.17 to 3.6.20 for AMC-III condition.

For the David inflow hydrographs, routing by the induced surcharge mode results in slightly higher peak outflow (and reservoir elevation) than the other two modes of operation. The reason for this is that by restricting the initial release from the reservoir, the reservoir builds up to a higher level and enters into the uncontrolled phase of spillway operation slightly ahead of the other two modes of operation. This results in a higher peak reservoir elevation and thereby, a larger peak release.

3.6.3 Standard Project Flood

The magnitude of Standard Project Foods (SPF) depends on whether it is the non-hurricane or the hurricane type of storm. The non-hurricane SPF has an average basin precipitation of 260 mm whilst the hurricane SPF has a corresponding average of 493 mm. Routing studies will, therefore, by carried out for both types of SPF hydrographs.

The resulting routing of the non-hurricane SPF are summarized in the Table 3.6.5.

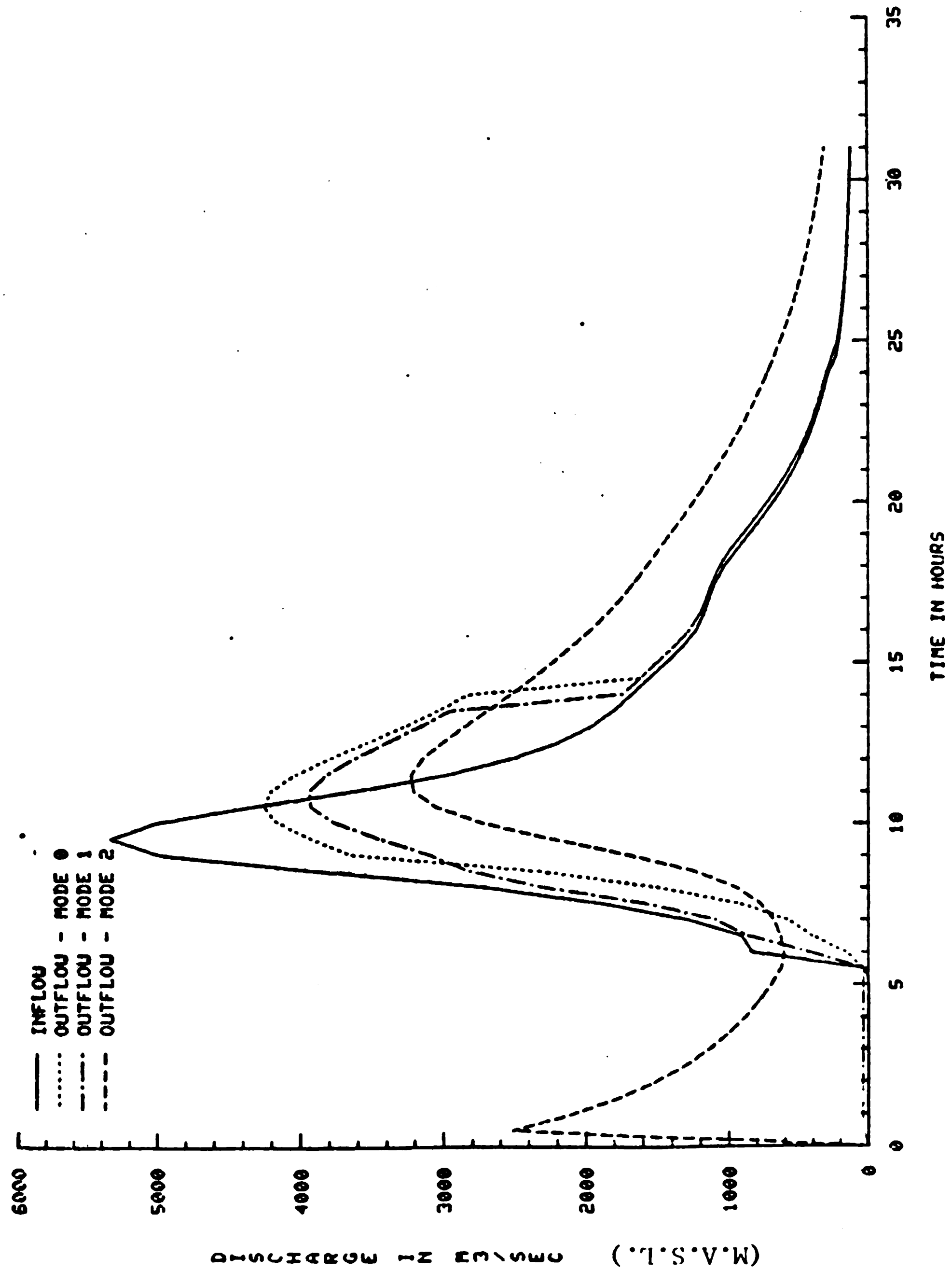


Figure 3.6.9. Reconstructed inflow and simulated outflow hydrographs of hurricane David at Valdesia reservoir - AMC I.

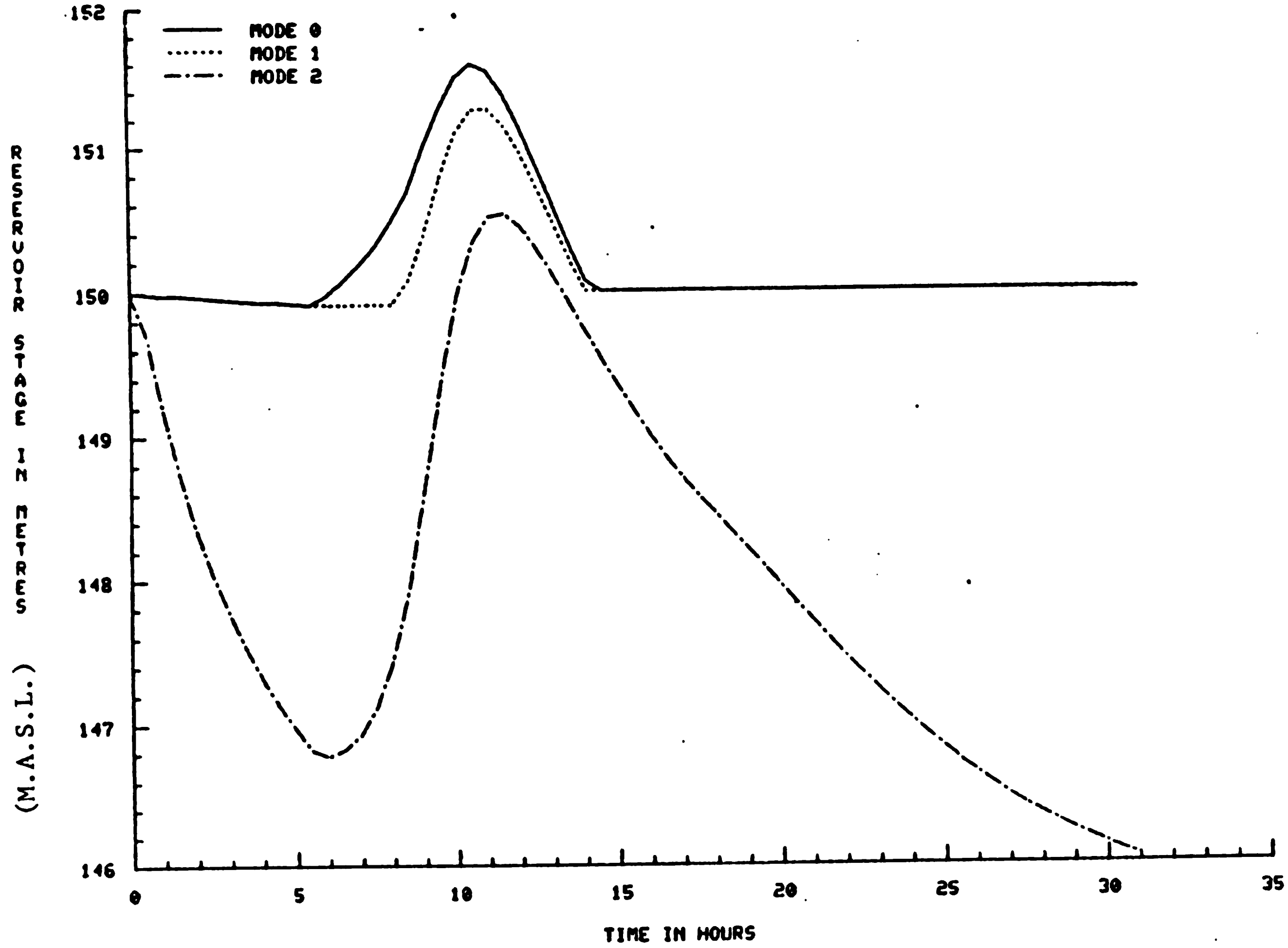


Figure 3.6.10. Simulated stage hydrographs of hurricane David at Valdesia reservoir - AMC I.

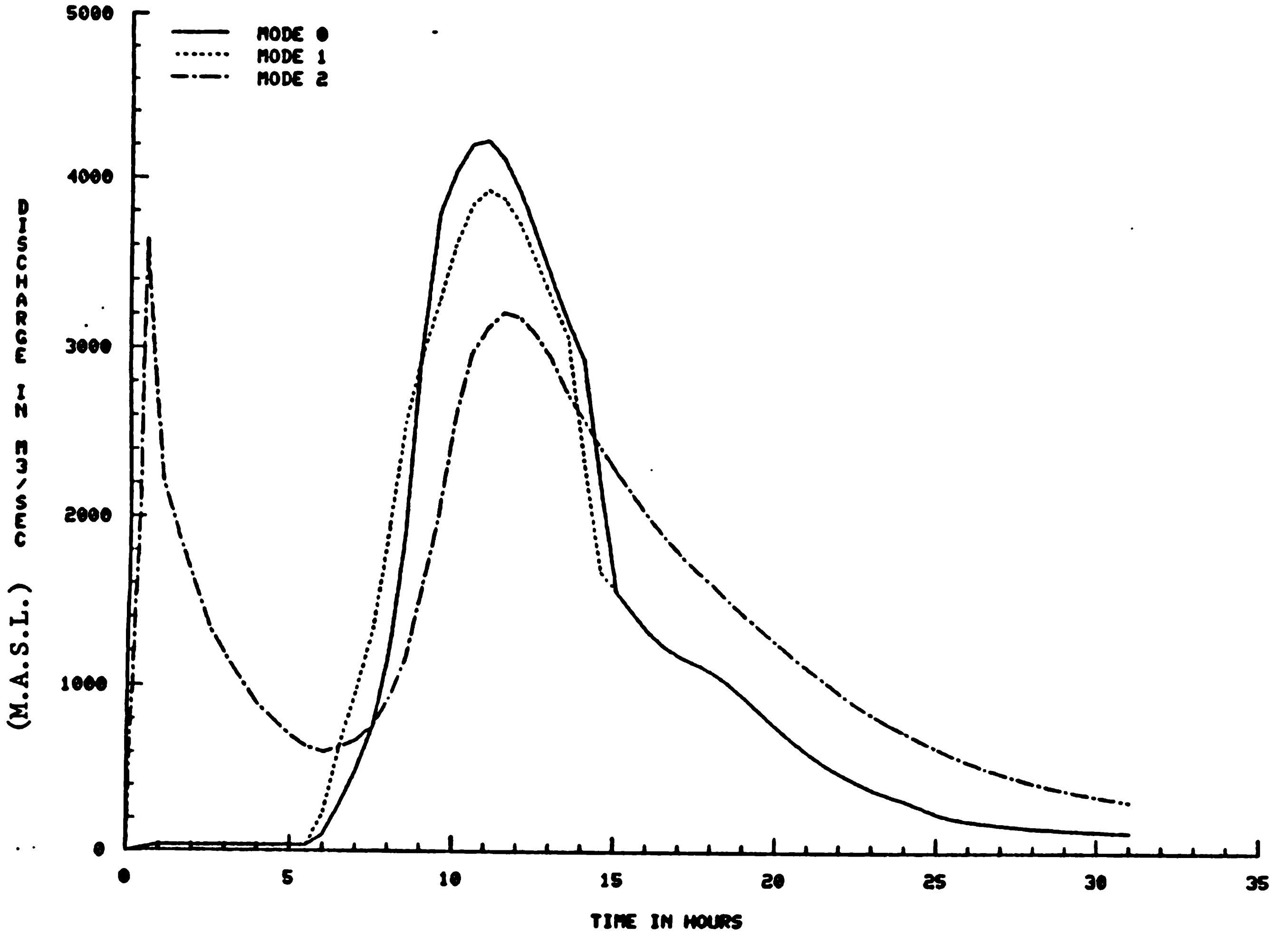


Figure 3.6.11. Simulated outflow hydrographs of hurricane David at Las Barias reservoir - AMC I.

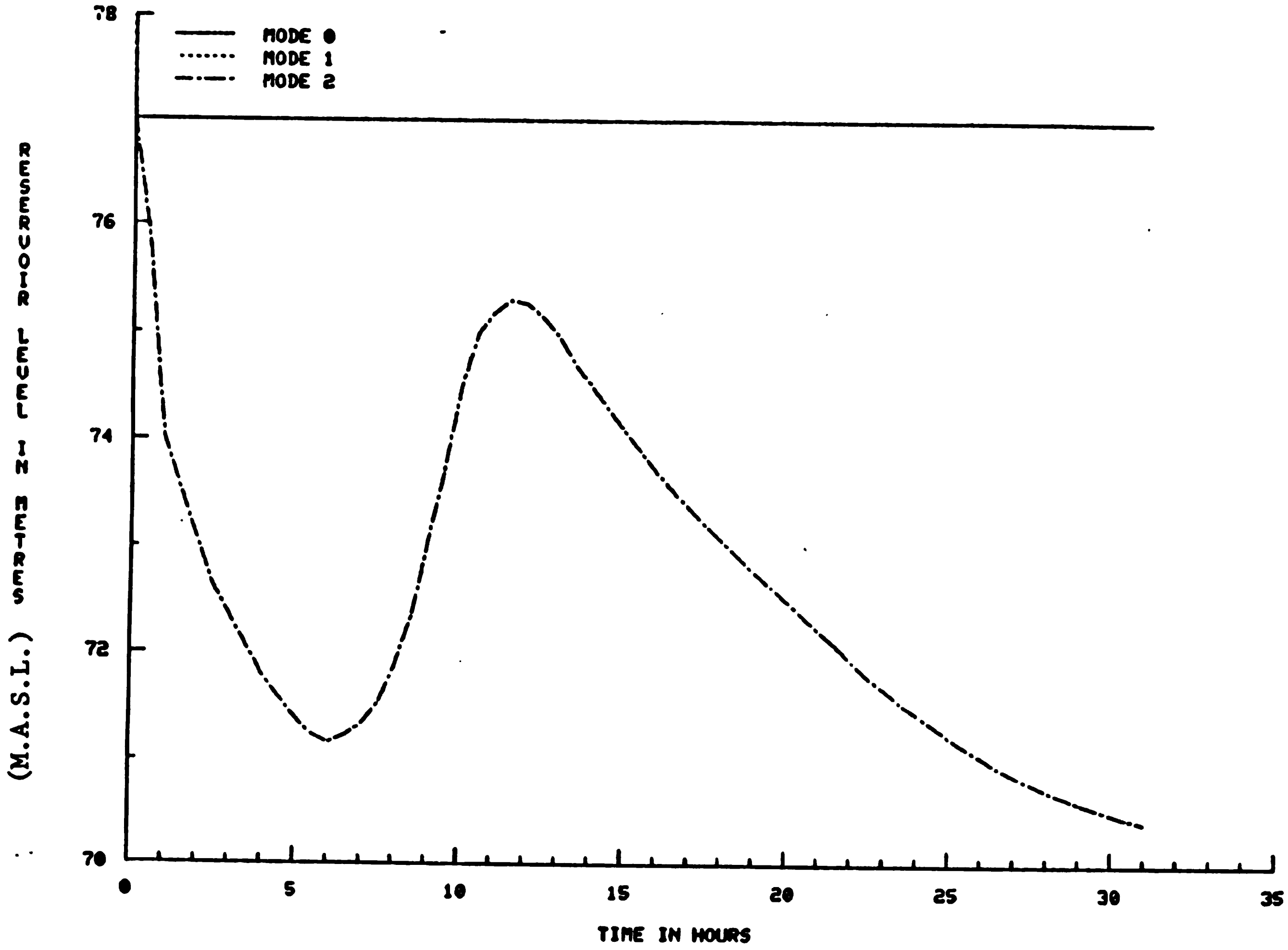


Figure 3.6.12. Simulated stage hydrographs of hurricane David at Las Barias reservoir - AMC I.

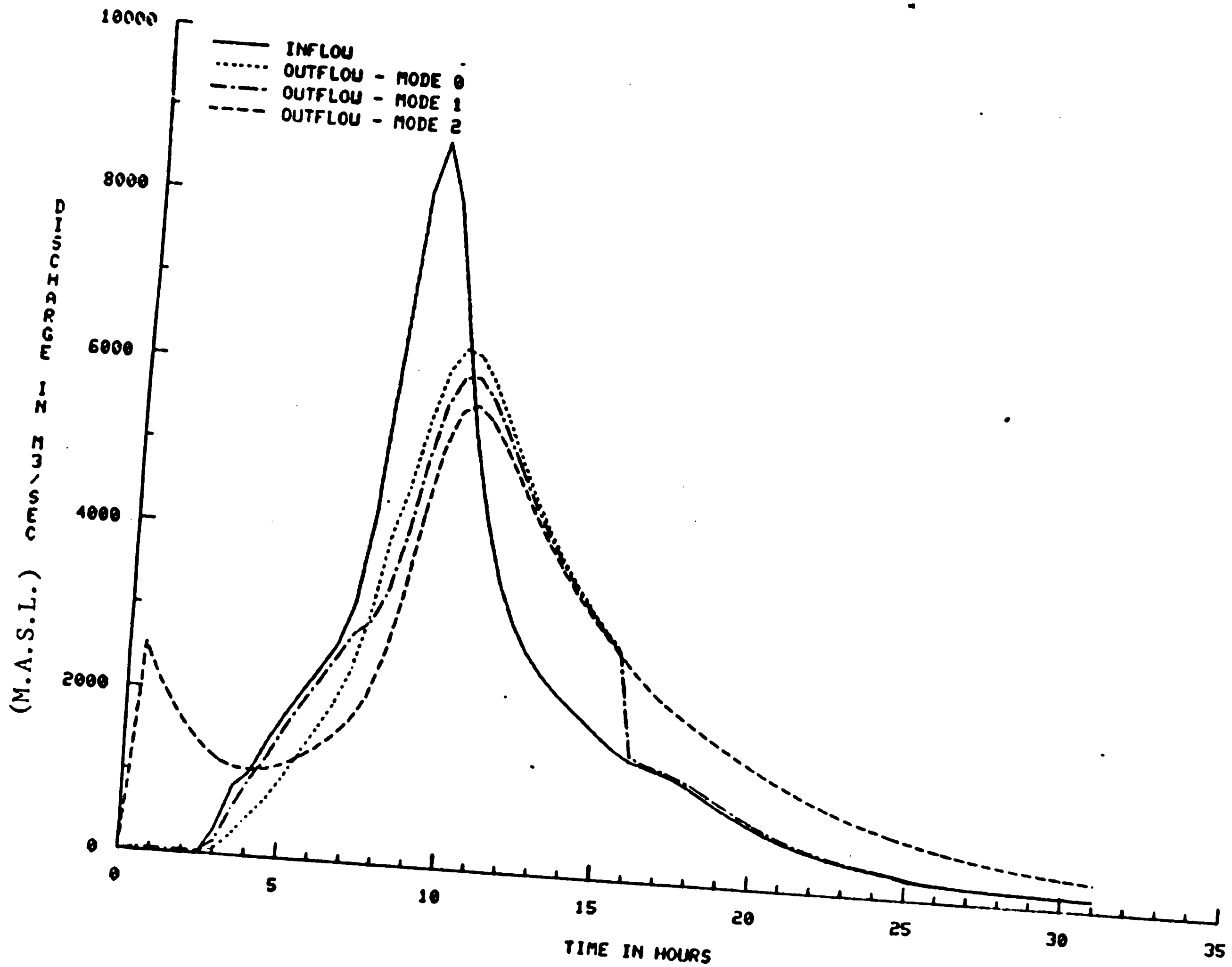


Figure 3.6.13. Reconstructed inflow and simulated outflow hydrographs of hurricane David at Valdesia reservoir - AMC II.

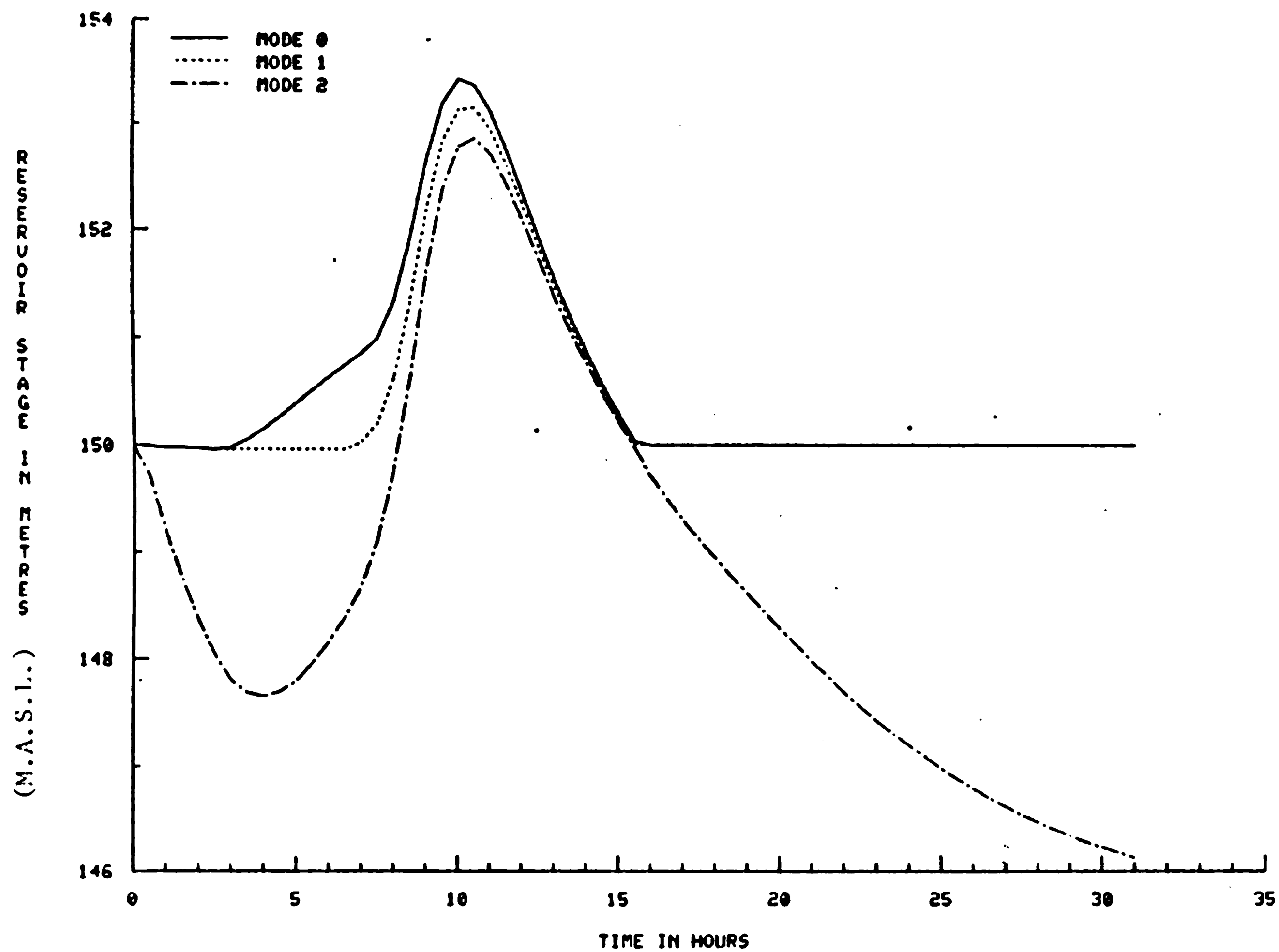


Figure 3.6.14. Simulated stage hydrographs of hurricane David at Valdesia reservoir - AMC II.

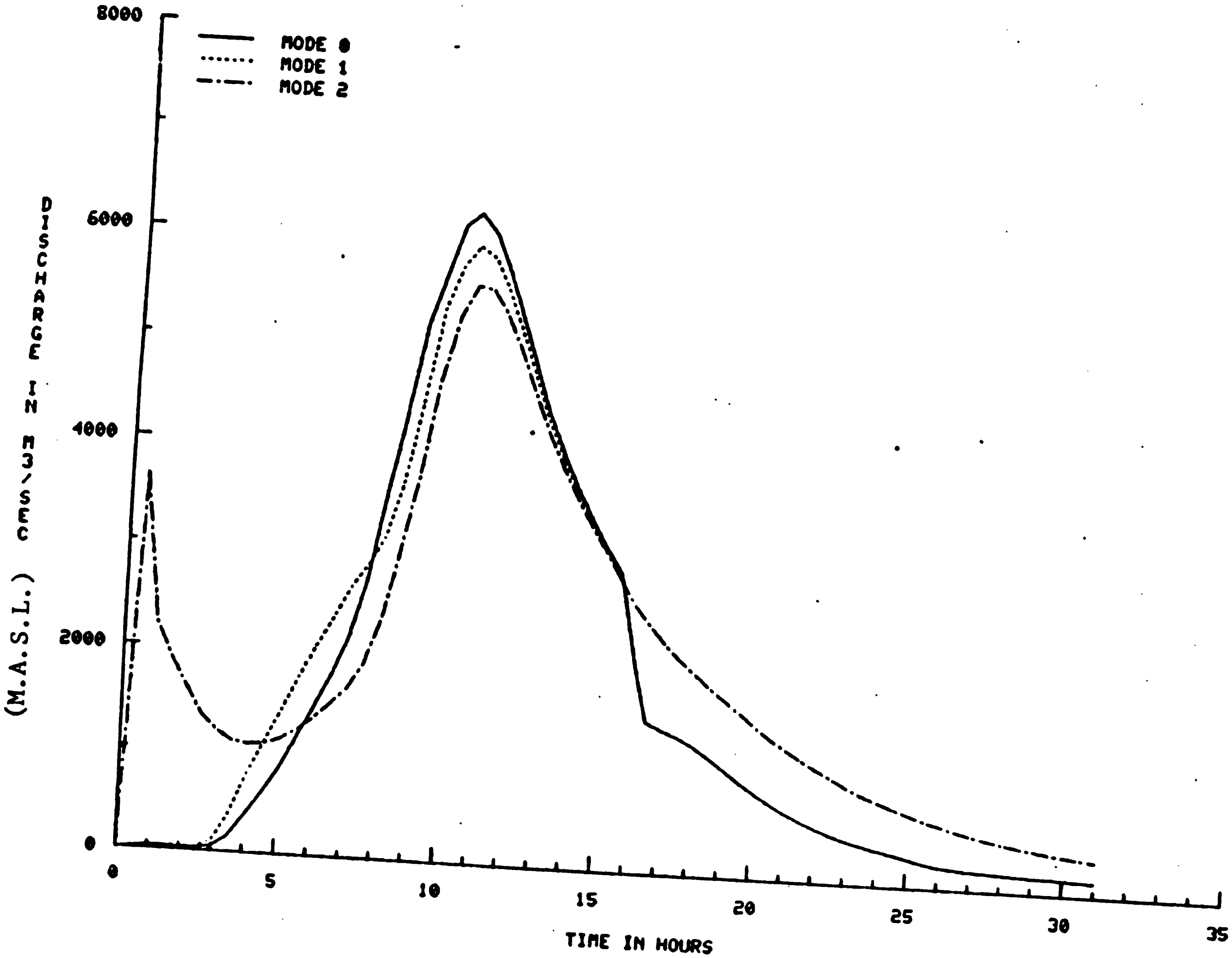


Figure 3.6.15. Simulated outflow hydrographs of hurricane David at las Barias reservoir - AMC II.

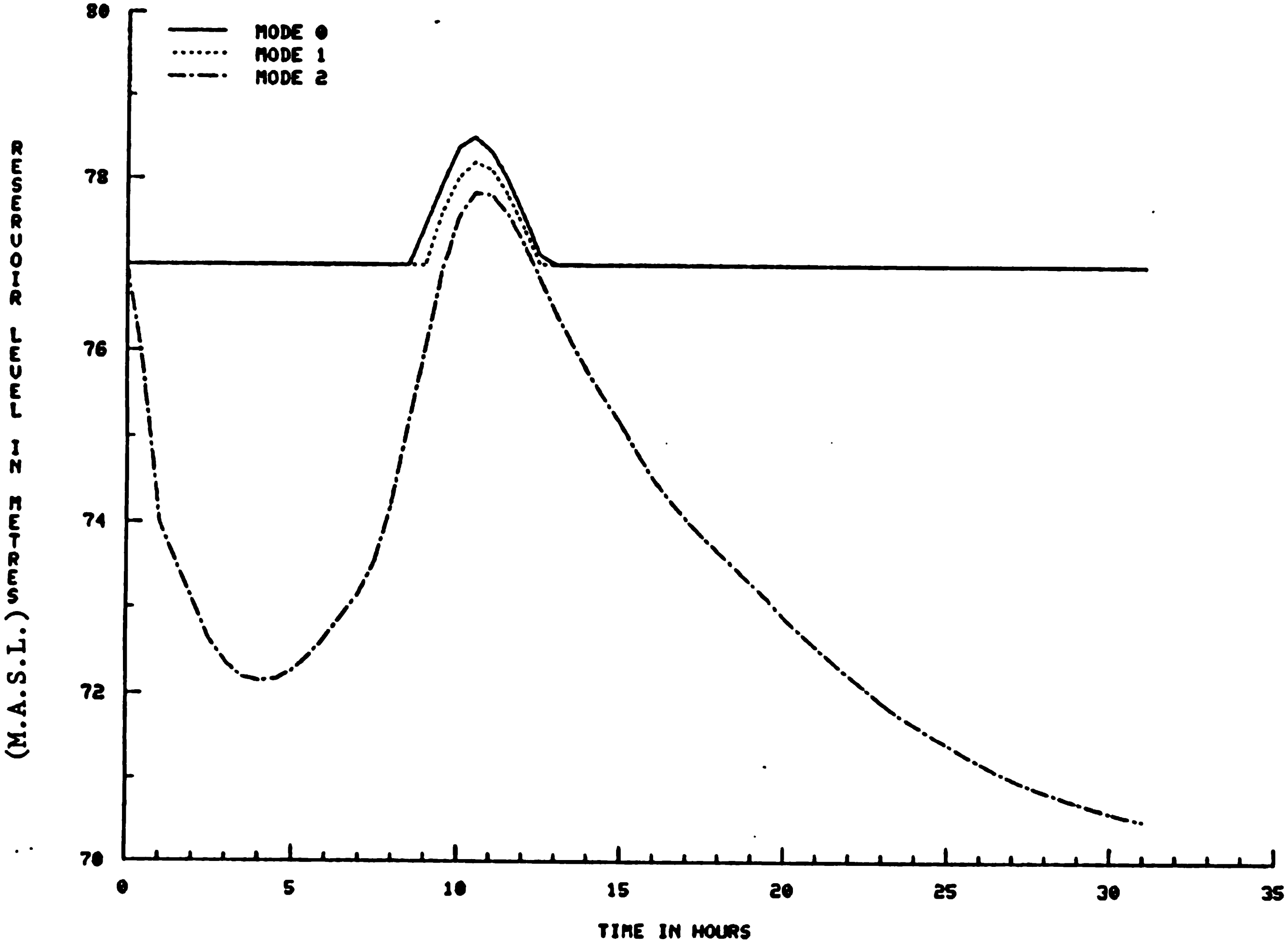


Figure 3.6.16. Simulated stage hydrographs of hurricane David at Las Barias reservoir - AMC II.

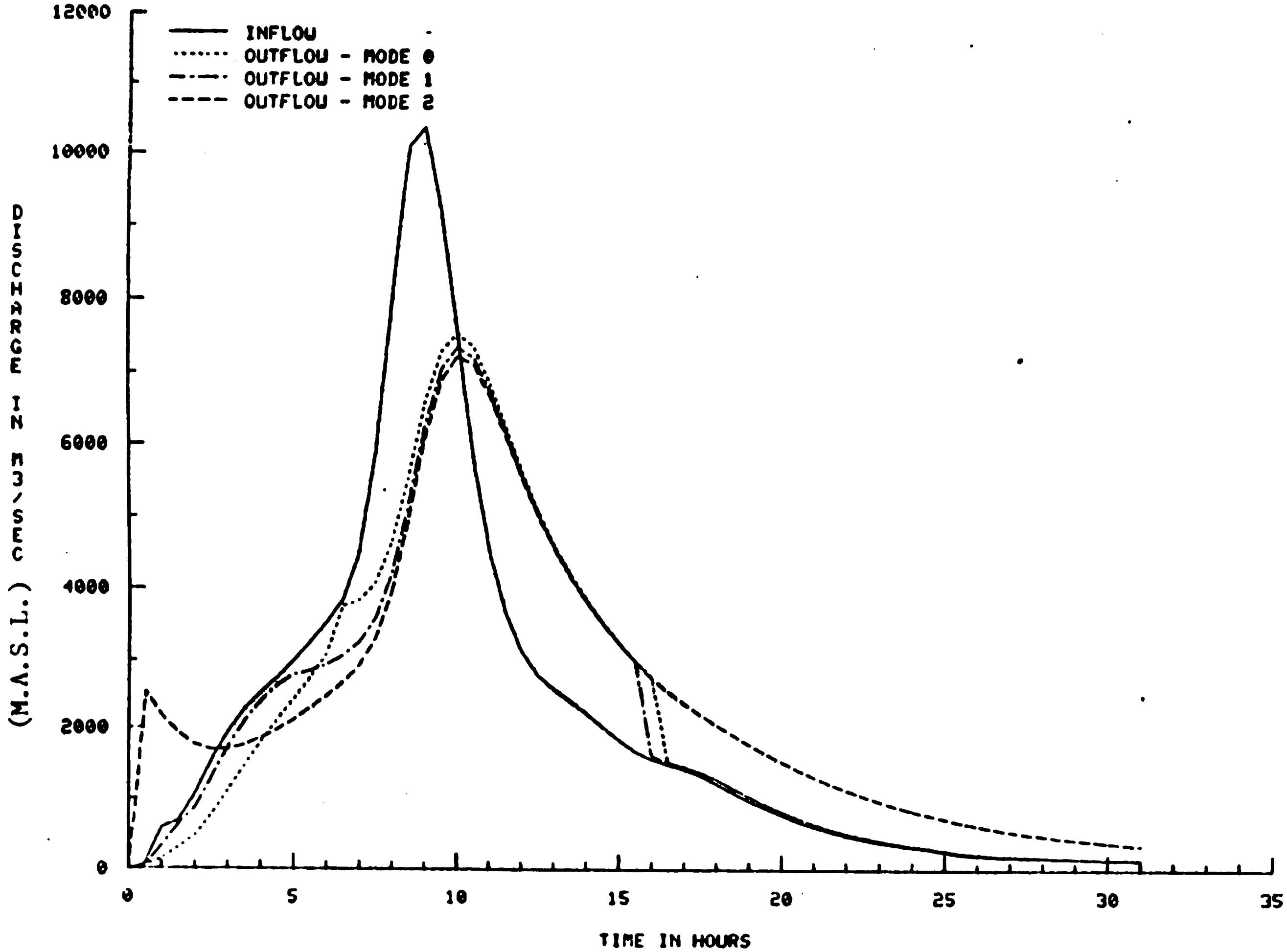


Figure 3.6.17. Reconstructed inflow and simulated outflow hydrographs of hurricane David at Valdesia reservoir - AMC III.

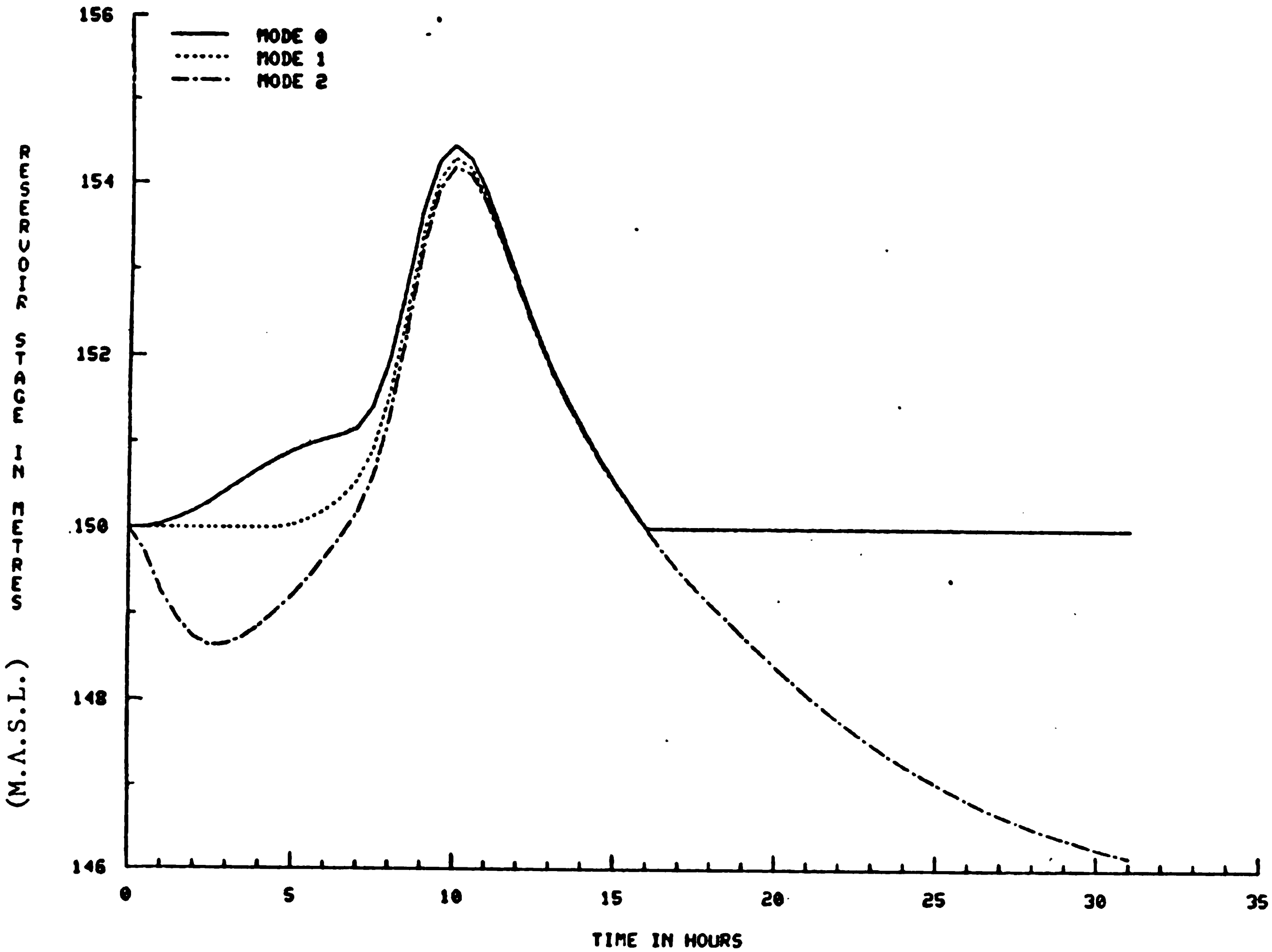


Figure 3.6.18. Simulated stage hydrographs of hurricane David at Valdesia reservoir - AMC III.

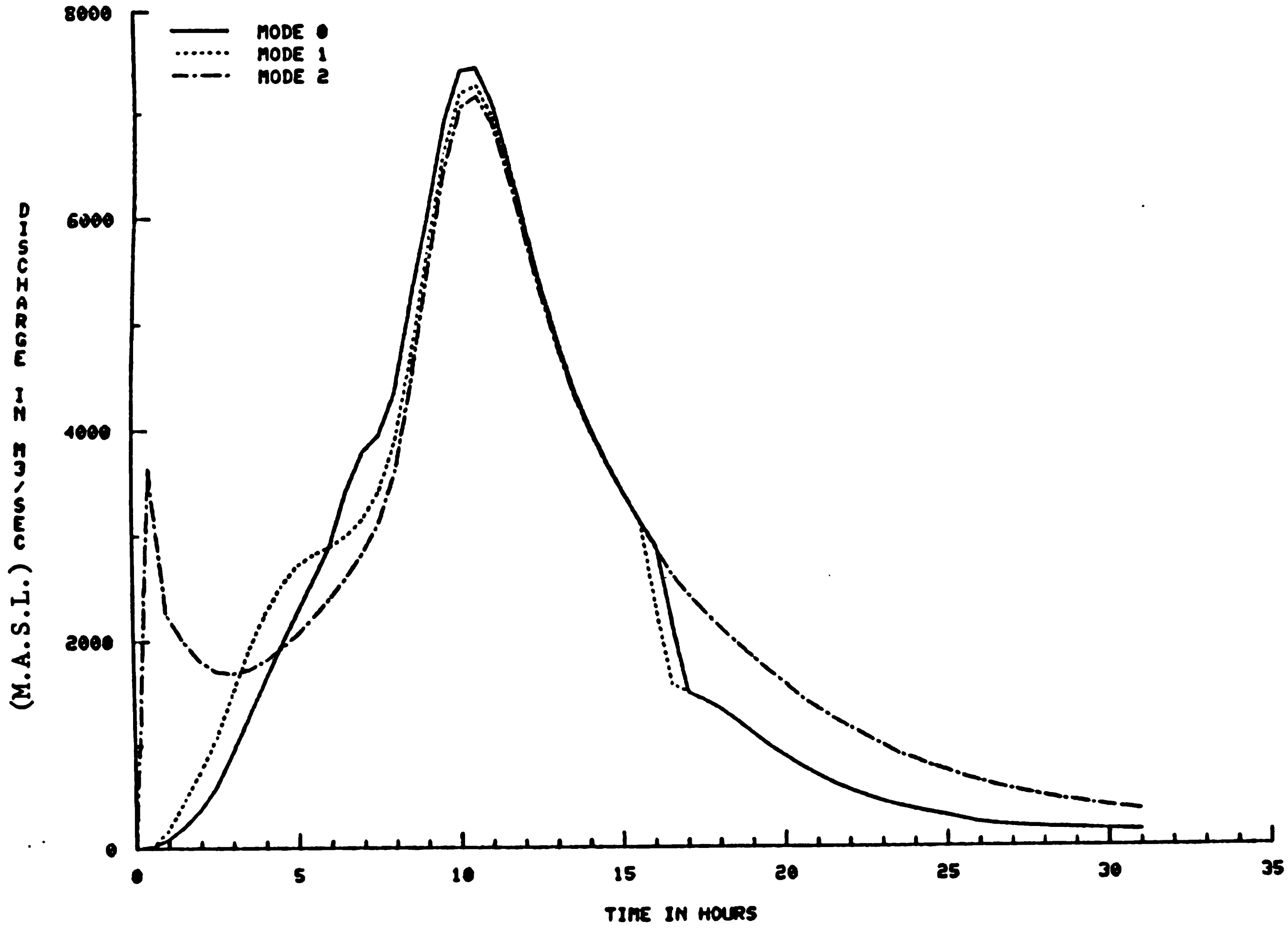


Figure 3.6.19. Simulated outflow hydrographs of hurricane David at Las Barias reservoir - AMC III.

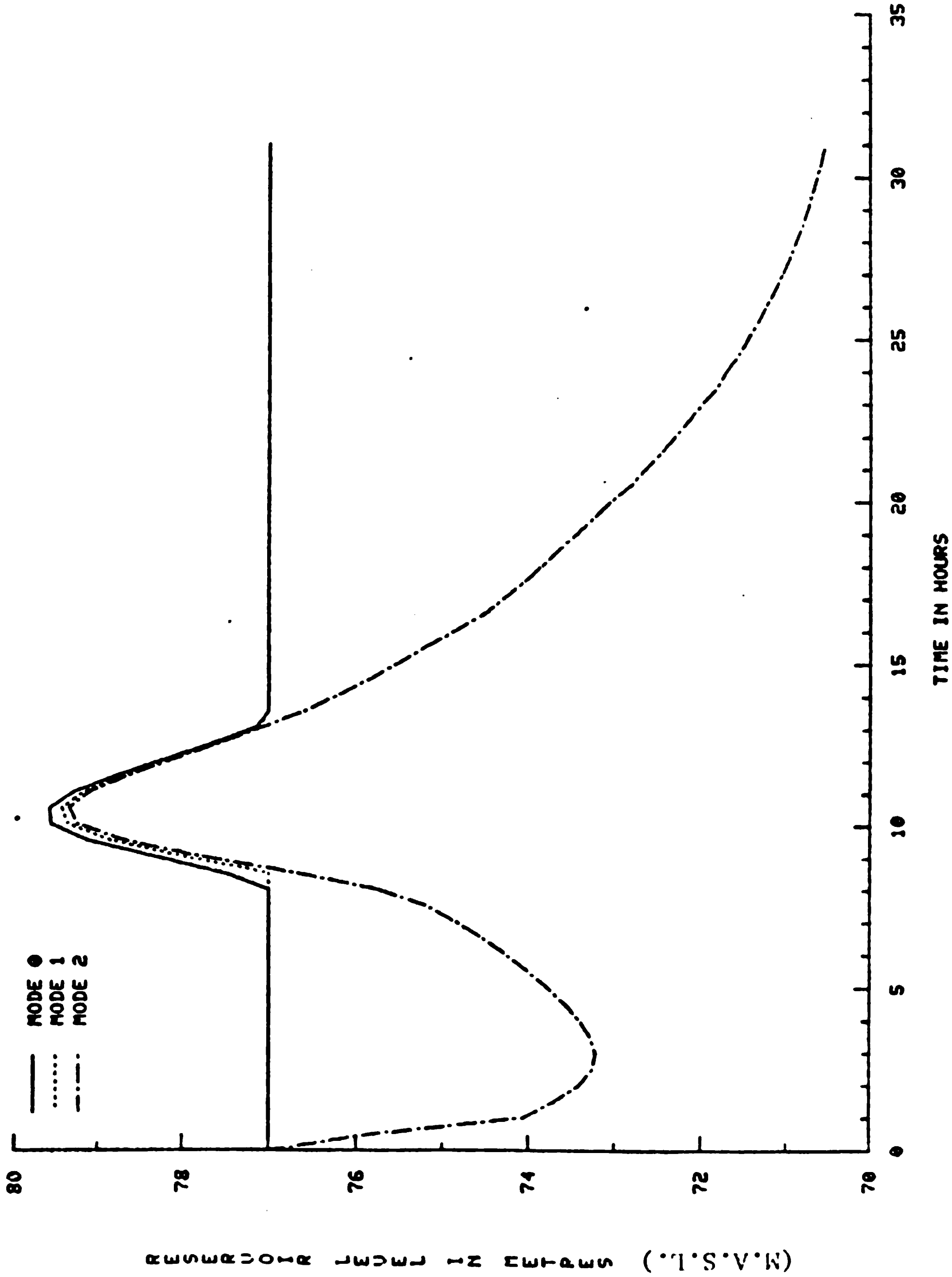


Figure 3.6.20. Simulated stage hydrographs of hurricane David at Las Barias reservoir - AMC III.

Table 3.6.5. Routing Non-Hurricane SPF under Different Modes of Operation

AMC	DESCRIPTION	MODE 0	MODE 1	MODE 2
AMC-I	<u>Valdesia</u>			
	Peak Inflow (m^3/sec)	2469	2469	2469
	Peak Outflow (m^3/sec)	1877	2450	1685
	Peak Reservoir Level (m)	150.86	150.0	150.0
	<u>Las Barias</u>			
	Peak Outflow (m^3/sec)	1874	2422	1684
	Peak Reservoir Level (m)	77.0	77.0	77.0
AMC-II	<u>Valdesia</u>			
	Peak Inflow (m^3/sec)	5380	5380	5380
	Peak Outflow (m^3/sec)	4304	4142	3647
	Peak Reservoir Level (m)	151.64	151.48	150.98
	<u>Las Barias</u>			
	Peak Outflow (m^3/sec)	4302	4125	3631
	Peak Reservoir Level (m)	77.0	77.0	77.0
AMC-III	<u>Valdesia</u>			
	Peak Inflow (m^3/sec)	7544	7544	7544
	Peak Outflow (m^3/sec)	5817	5669	5404
	Peak Reservoir Level (m)	153.03	152.90	152.67
	<u>Las Barias</u>			
	Peak Outflow (m^3/sec)	5804	5651	5357
	Peak Reservoir Level (m)	78.08	77.93	77.64

The outflow and stage hydrographs of routing the non-hurricane SPF were displayed in a series of plots shown in Figure 3.6.21 to 3.6.32.

Similarly, the hurricane SPF was routed through the Valdesia-Las Barias Reservoir system and the results are summarized in Table 3.6.6.

The outflow and stage hydrographs of routing the hurricane SPF were displayed in a series of plots shown in Figure 3.6.33 to 3.6.44. The studies have shown that the maximum pool level in both the Valdesia and Las Barias Dams will exceed the design level for hurricane SPF under AMC-II and AMC-III conditions, if the initial reservoir levels were at normal full pool levels.

If the reservoir level is lowered in advance of the arrival of the SPF flood, additional flood control stage thus made available can be effectively used to cope with larger flood peaks. This was investigated through a series of routing studies with different initial reservoir levels. The results are as shown in Figure 3.6.45 which is a plot of initial reservoir elevation against the maximum peak inflow that can safely pass through the Valdesia Reservoir (without exceeding the maximum surcharge elevation of 154.0 m). The results indicate that if the initial reservoir level is at 130 m, an SPF flood peak of about 15,000 m^3/sec can be accommodated. It is of relevance, however, to mention that an advance time of about 11 days is required for the reservoir level to be drawn down from 150 m to 130 m.

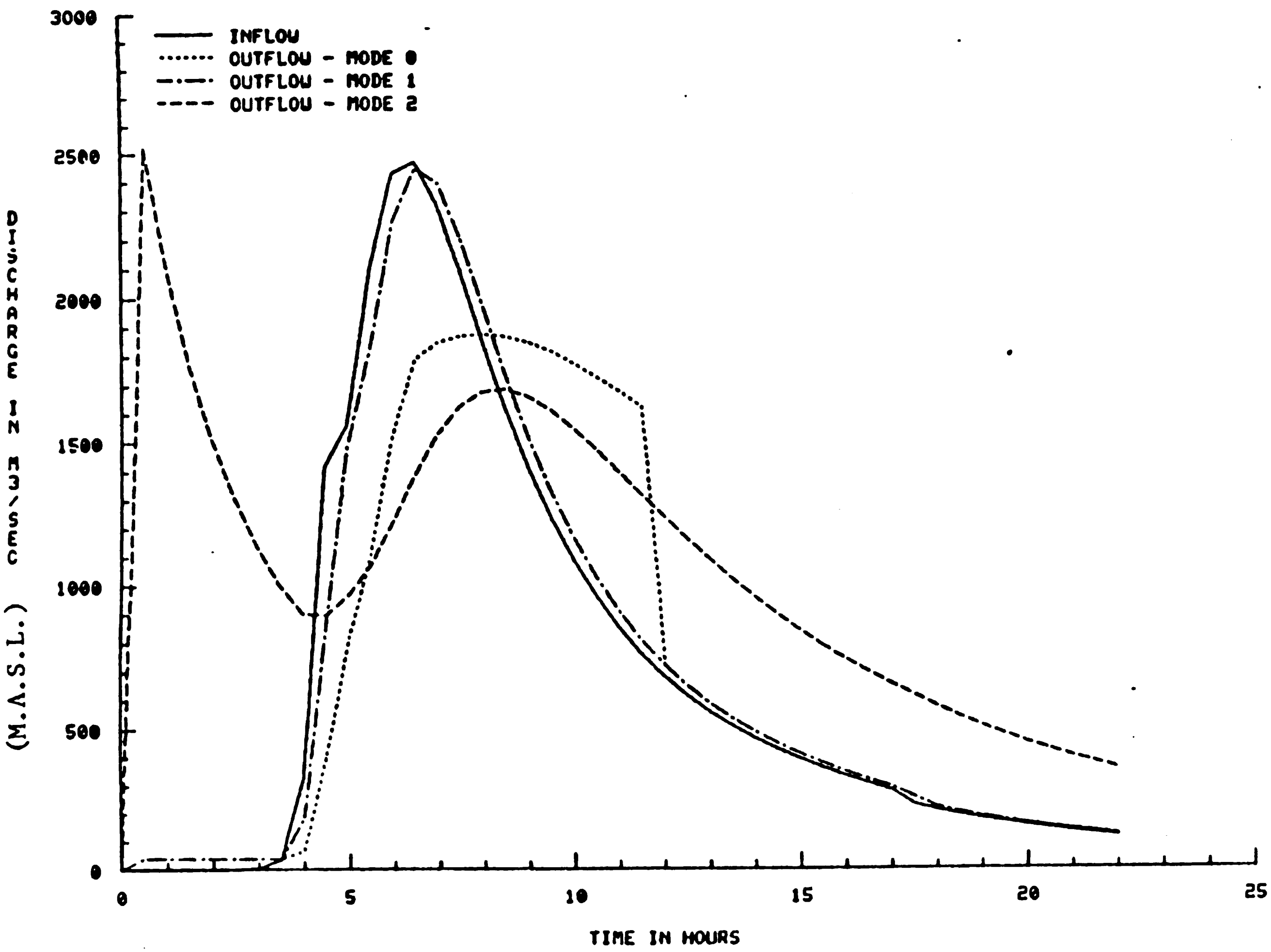


Figure 3.6.21. Inflow and simulated outflow hydrographs of non-hurricane SPF at Valdesia reservoir - AMC I.

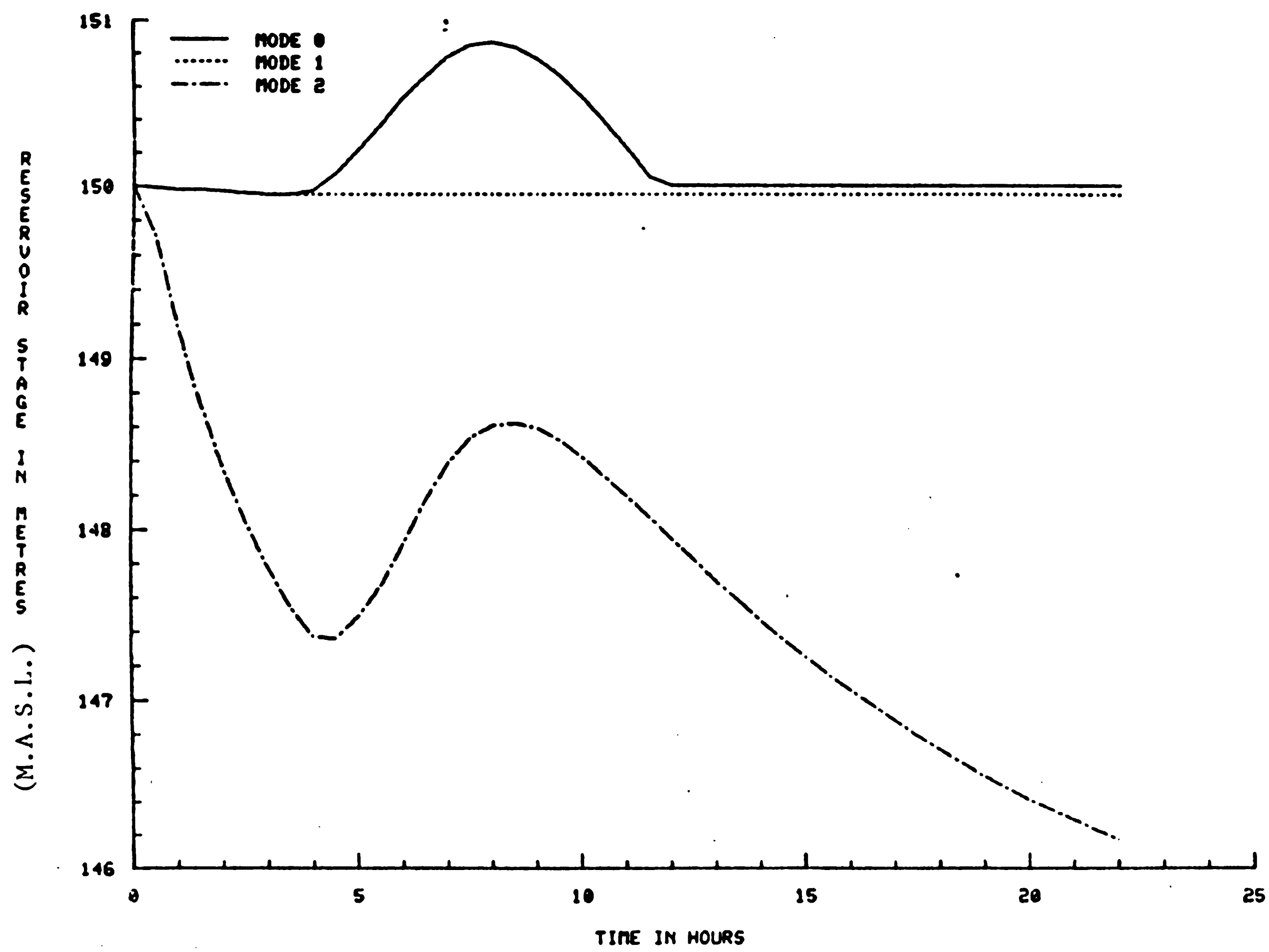


Figure 3.6.22. Simulated stage hydrographs of non-hurricane SPF at Valdesia reservoir - AMC I.

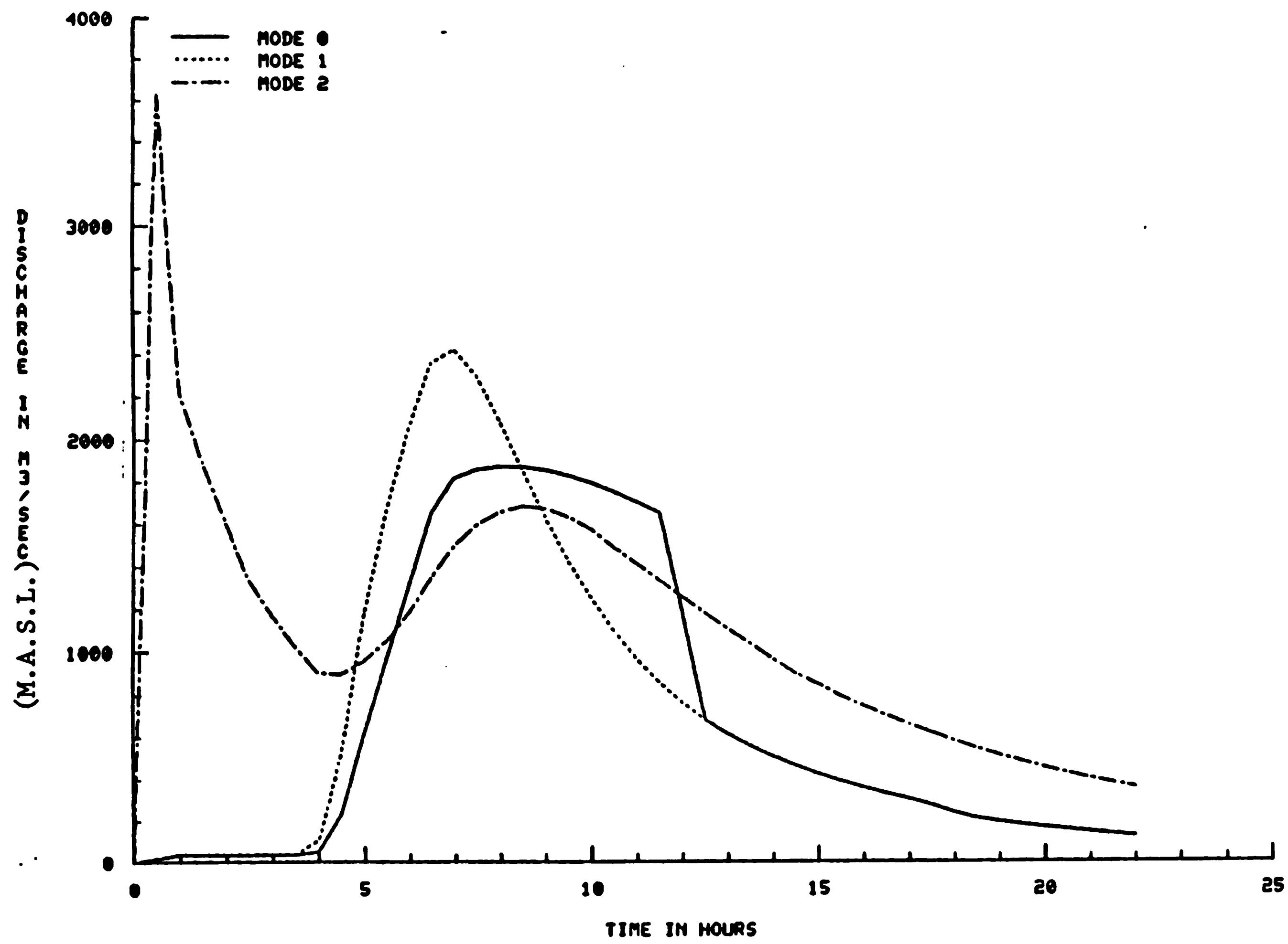


Figure 3.6.23. Simulated outflow hydrographs of non-hurricane SPF at Las Barias reservoir - AMC I.

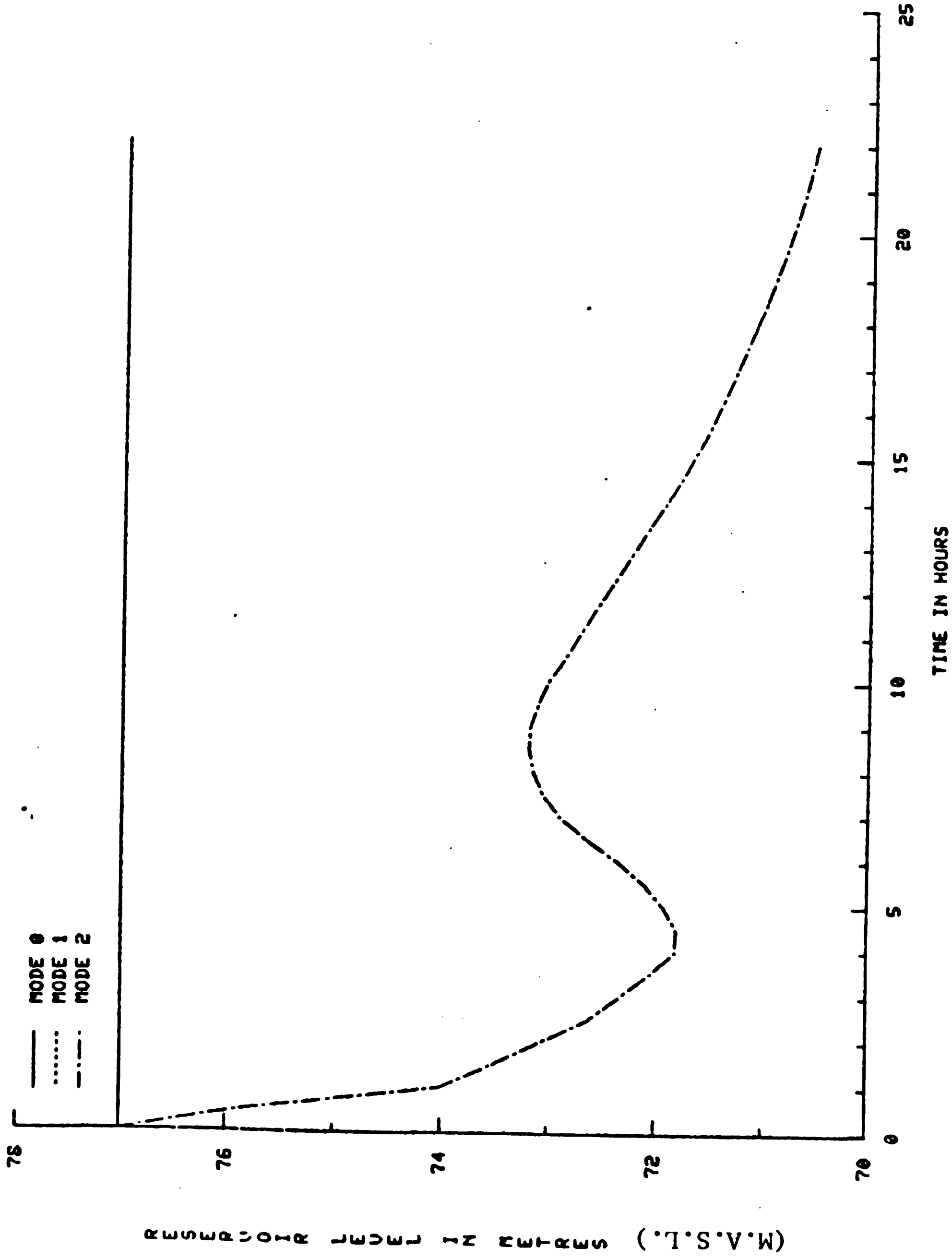


Figure 3.6.24. Simulated stage hydrographs of non-hurricane SPF at Las Barias reservoir - AMC I.

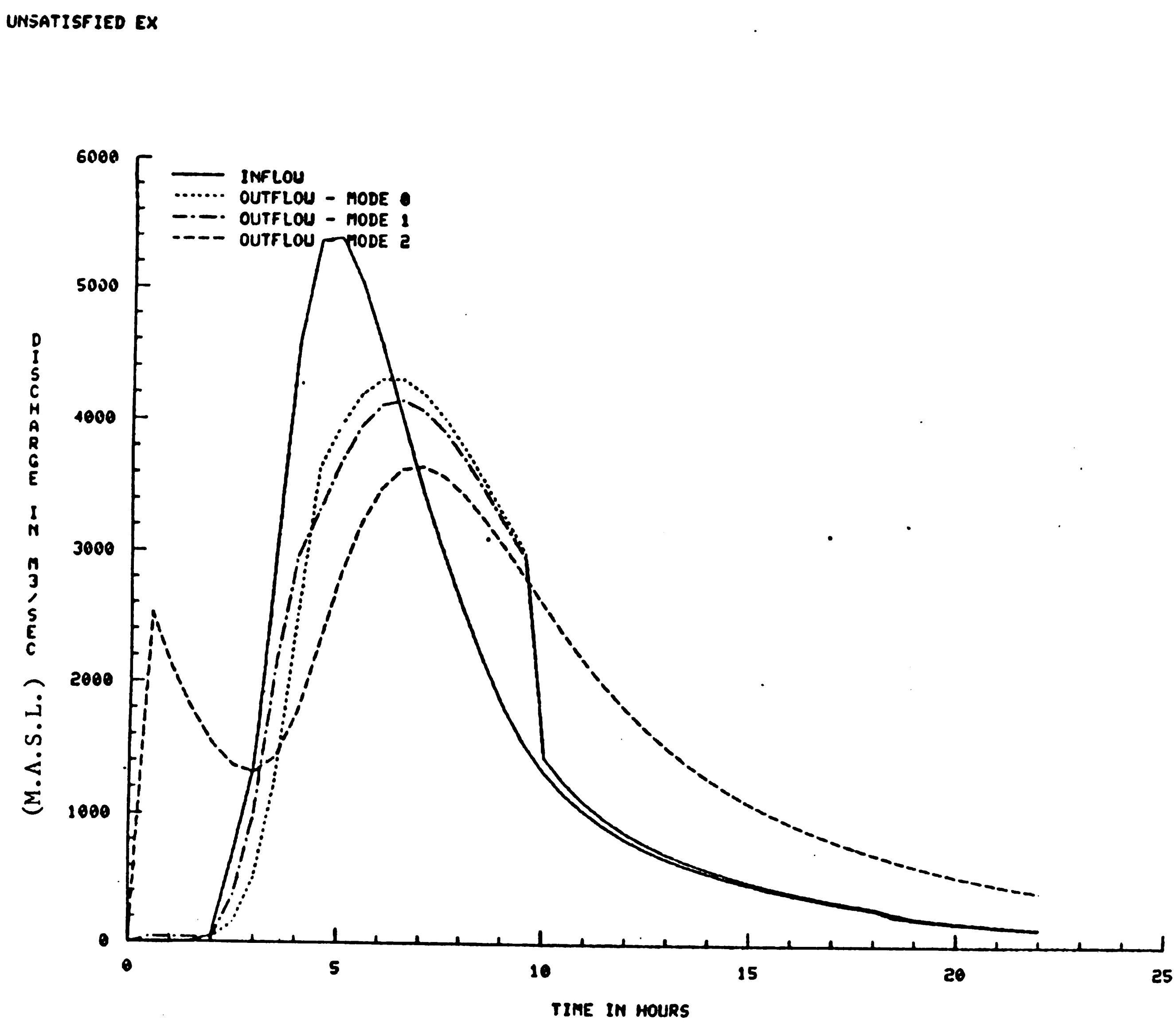


Figure 3.6.25. Inflow and simulated outflow hydrographs of non-hurricane SPF at Valdesia reservoir - AMC II.

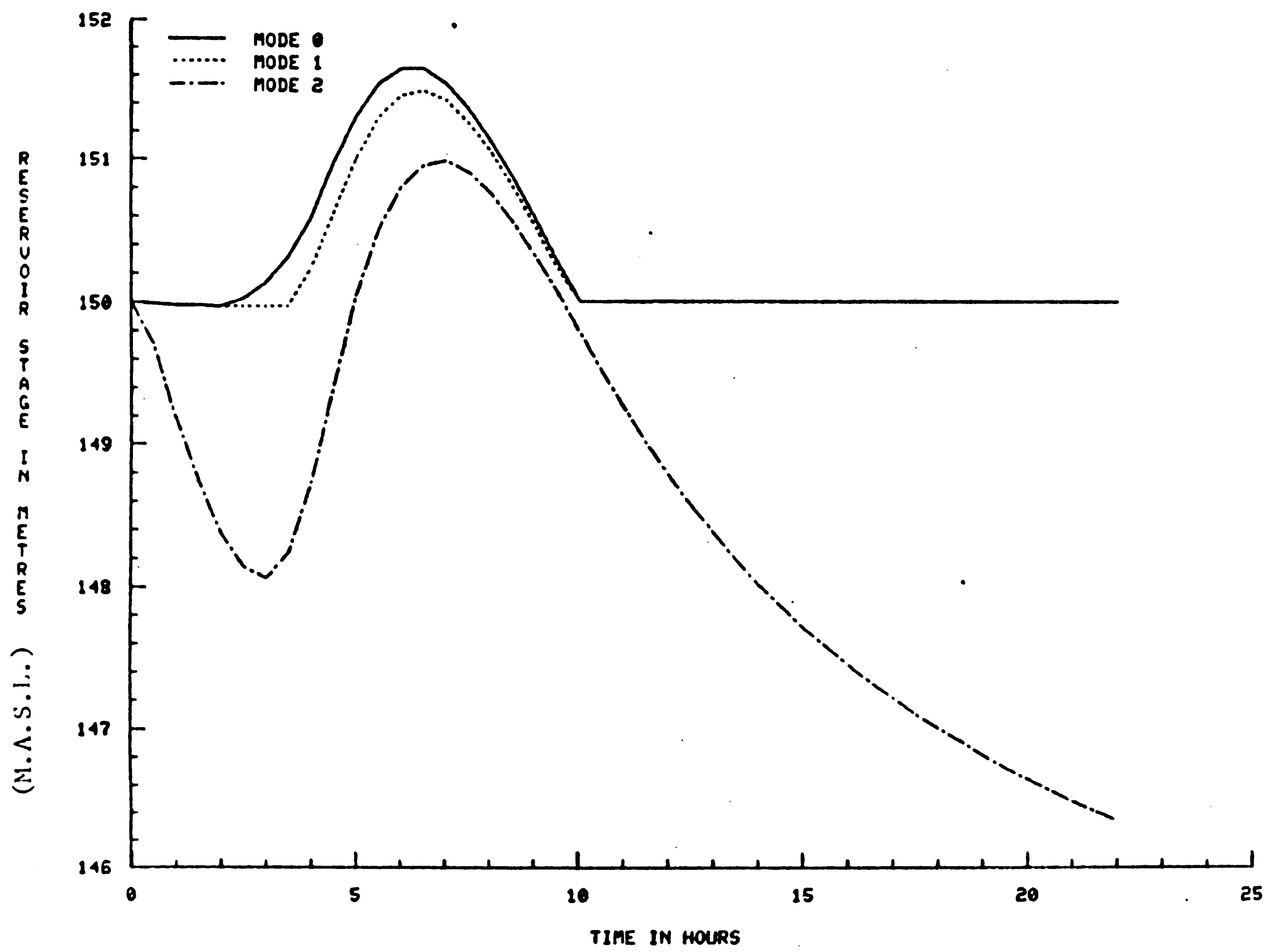


Figure 3.6.26. Simulated stage hydrographs of non-hurricane SPF at Valdesia reservoir - AMC II.

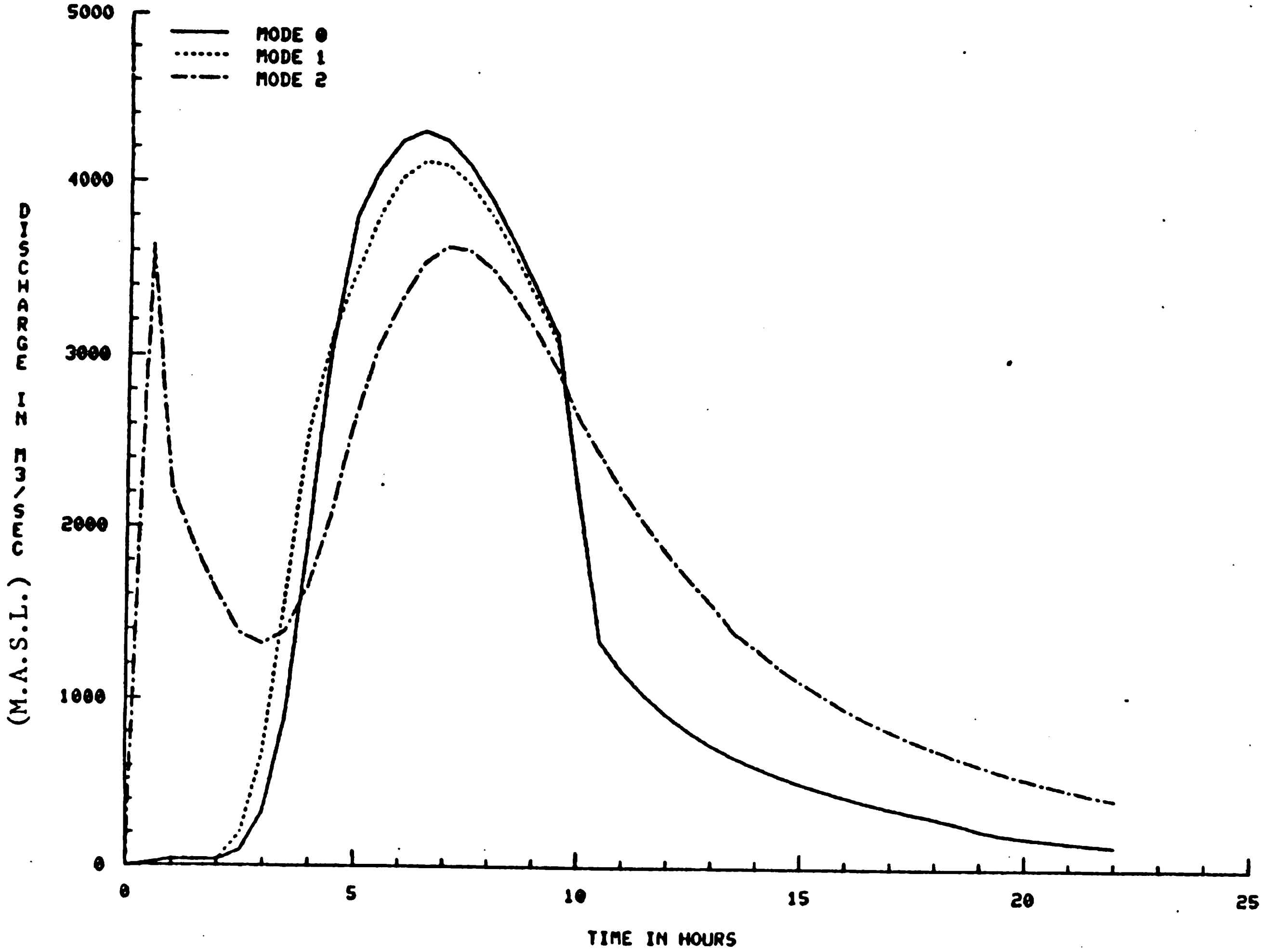


Figure 3.6.27. Simulated outflow hydrographs of non-hurricane SPF at Las Barias reservoir - AMC II.

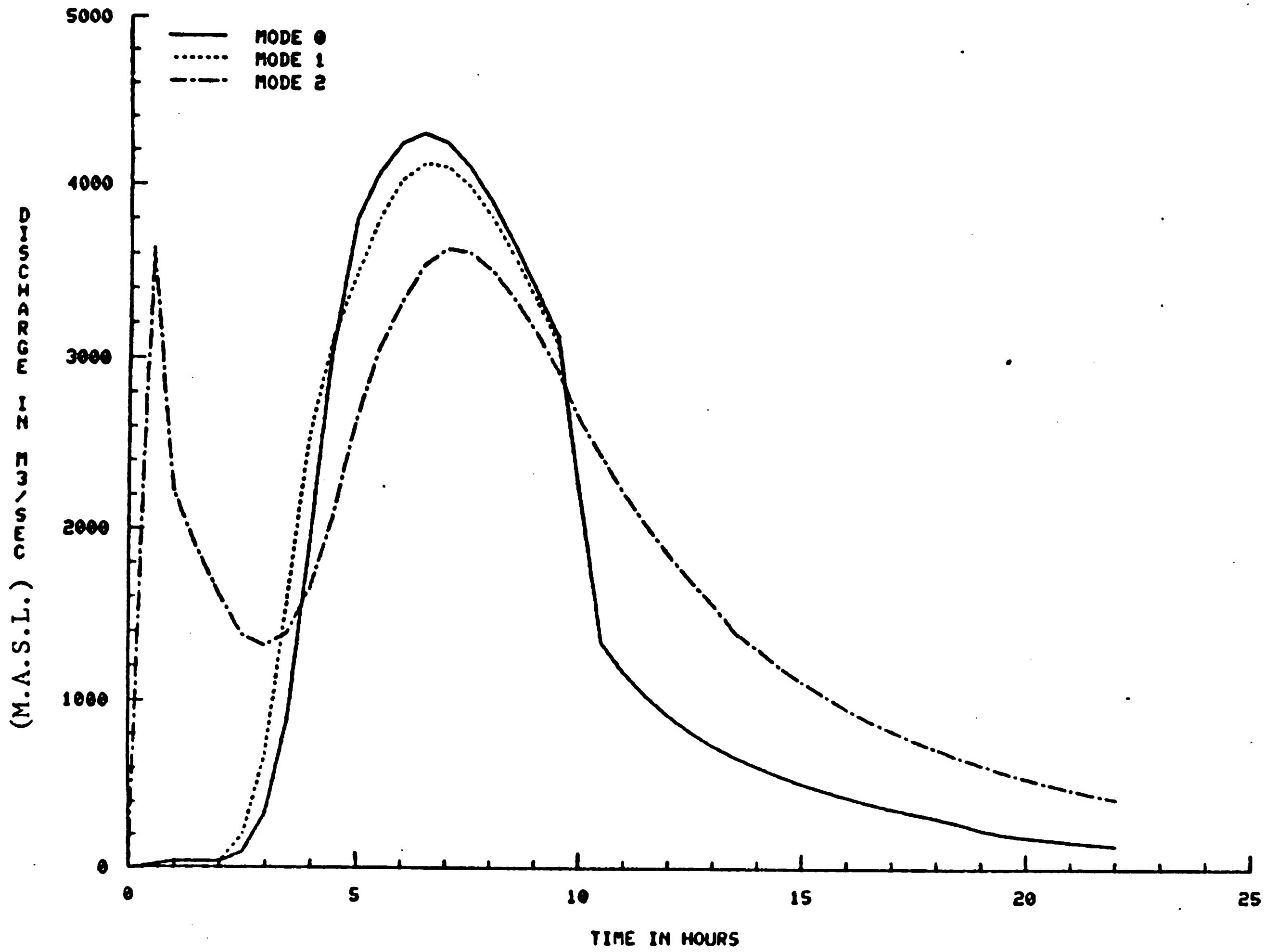


Figure 3.6.27. Simulated outflow hydrographs of non-hurricane SPF at Las Barias reservoir - AMC II.

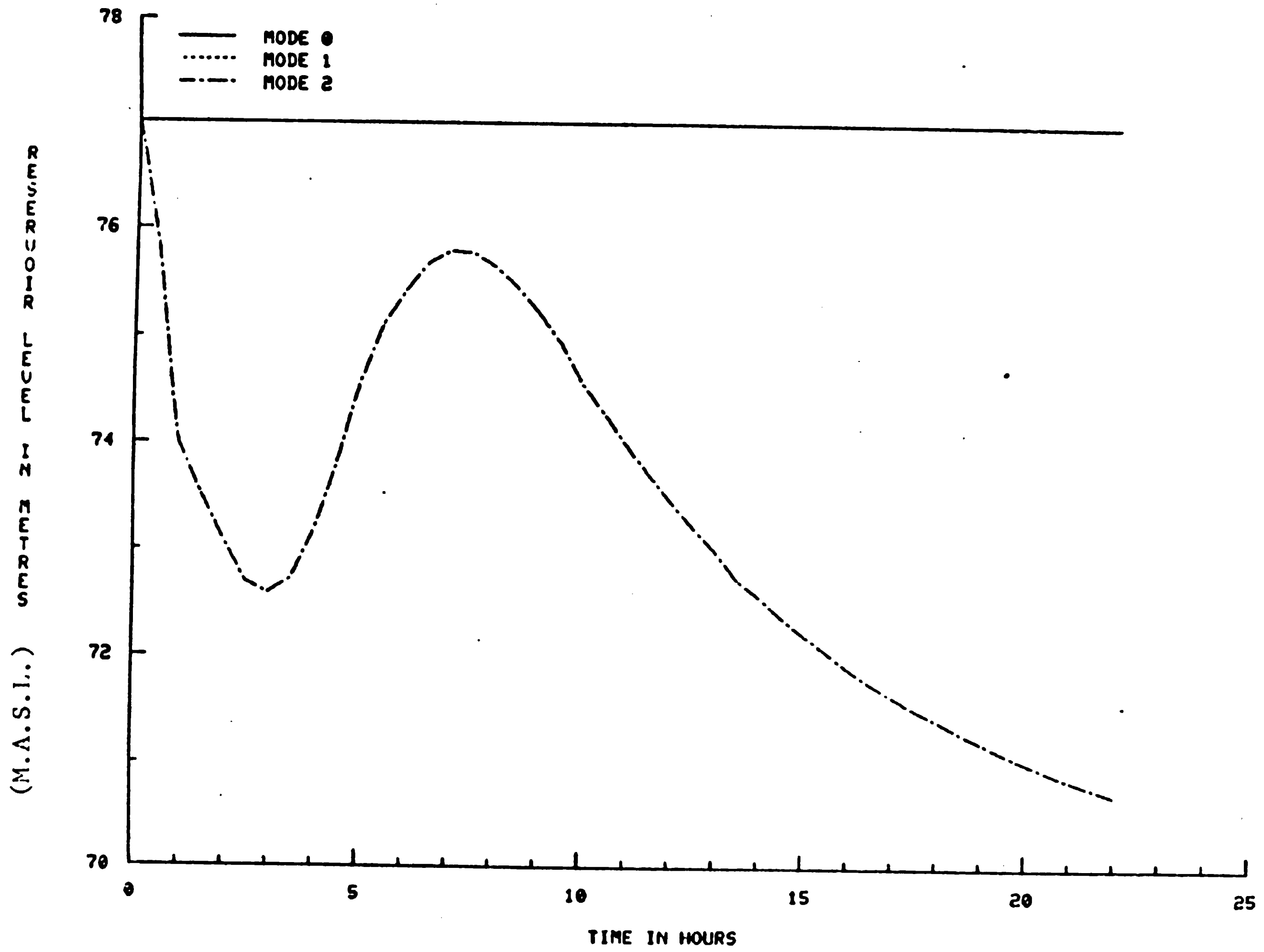


Figure 3.6.28. Simulated stage hydrographs of non-hurricane SPF at Las Barias reservoir - MAC II.

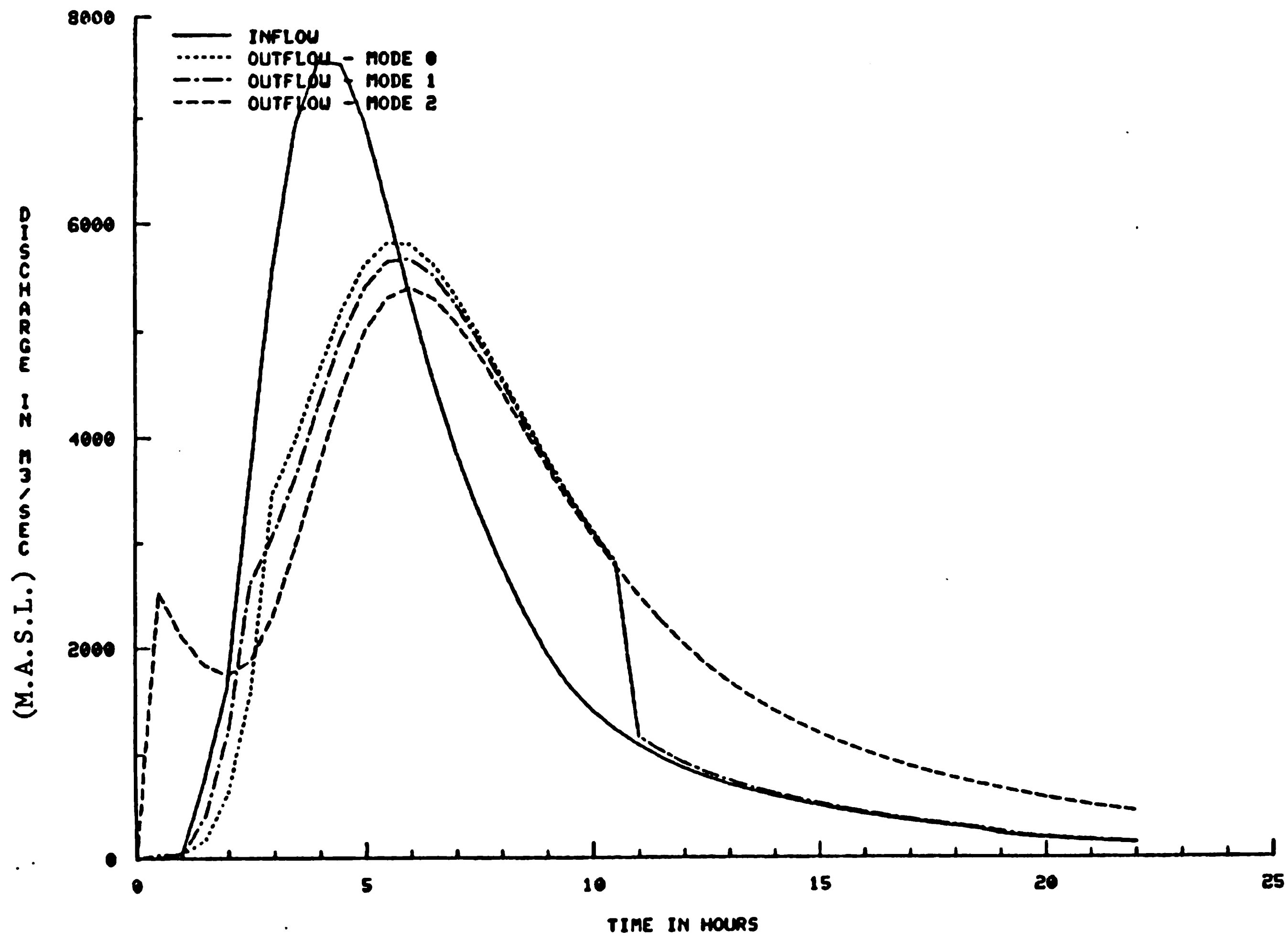


Figure 3.6.29. Inflow and simulated outflow hydrographs of non-hurricane SPF at Valdesia reservoir - AMC III.

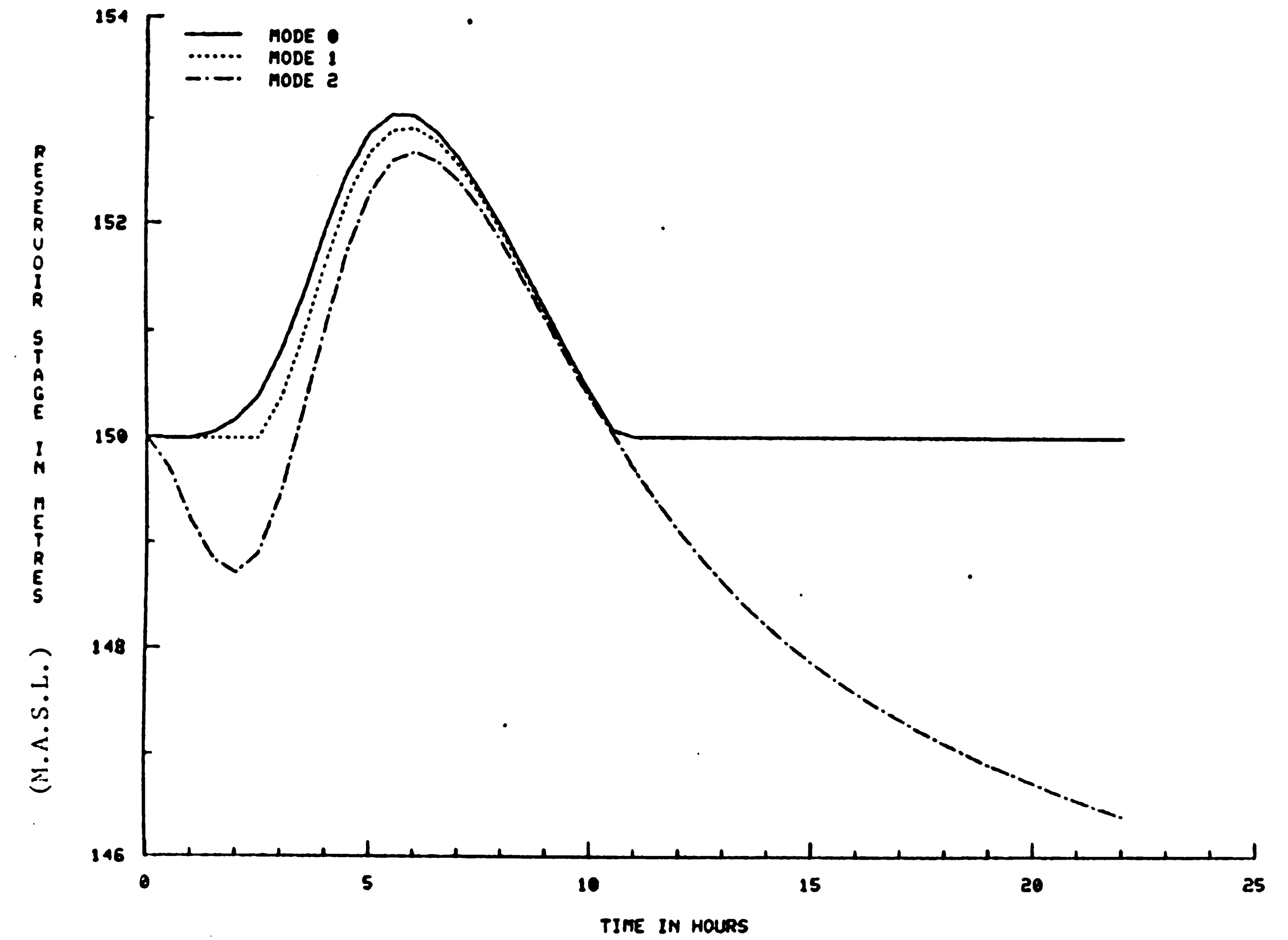


Figure 3.6.30. Simulated stage hydrographs of non-hurricane SPF at Valdesia reservoir - AMC III.

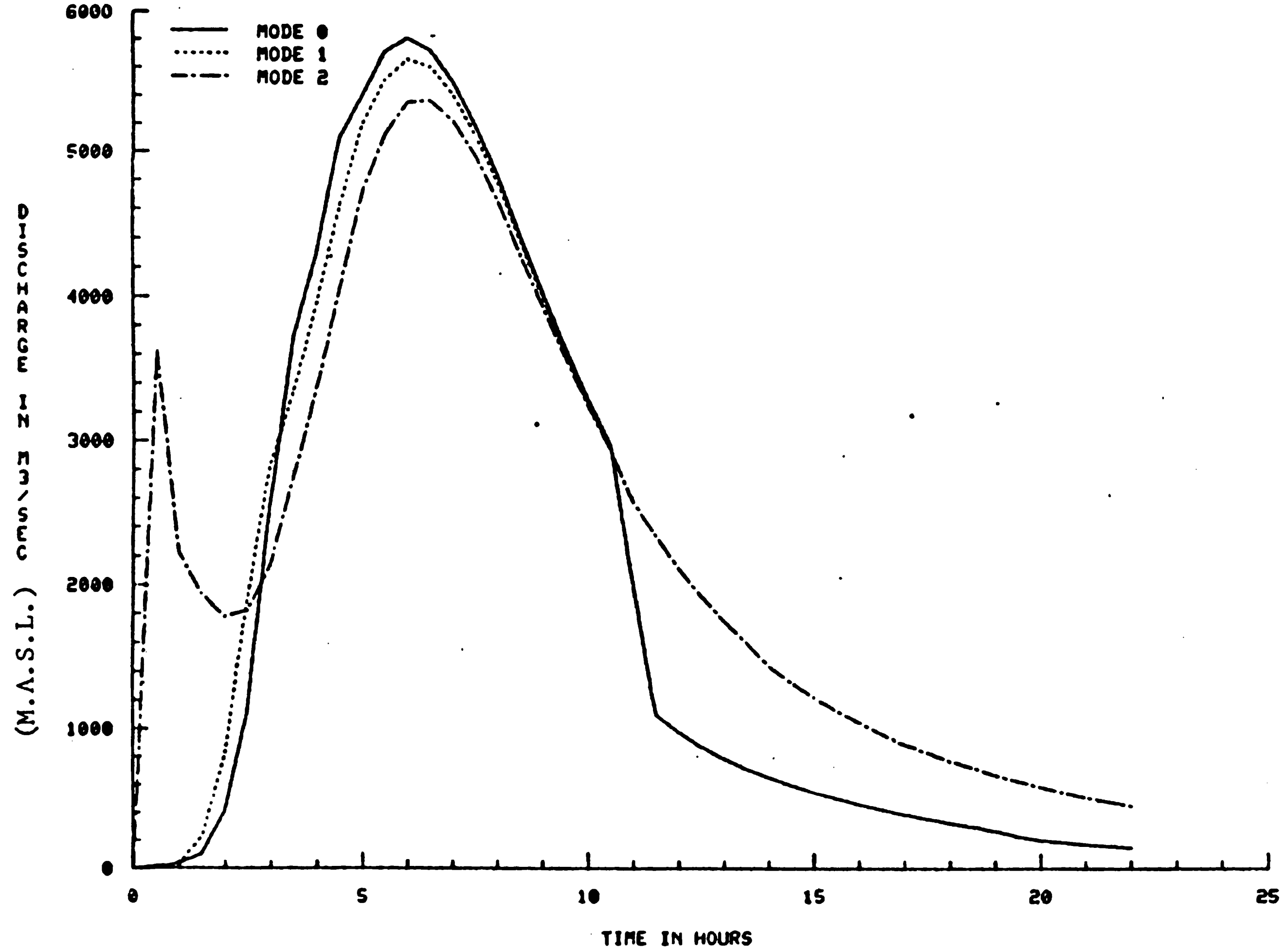


Figure 3.6.31. Simulated outflow hydrographs of non-hurricane SPF at Las Barias reservoir - AMC III.

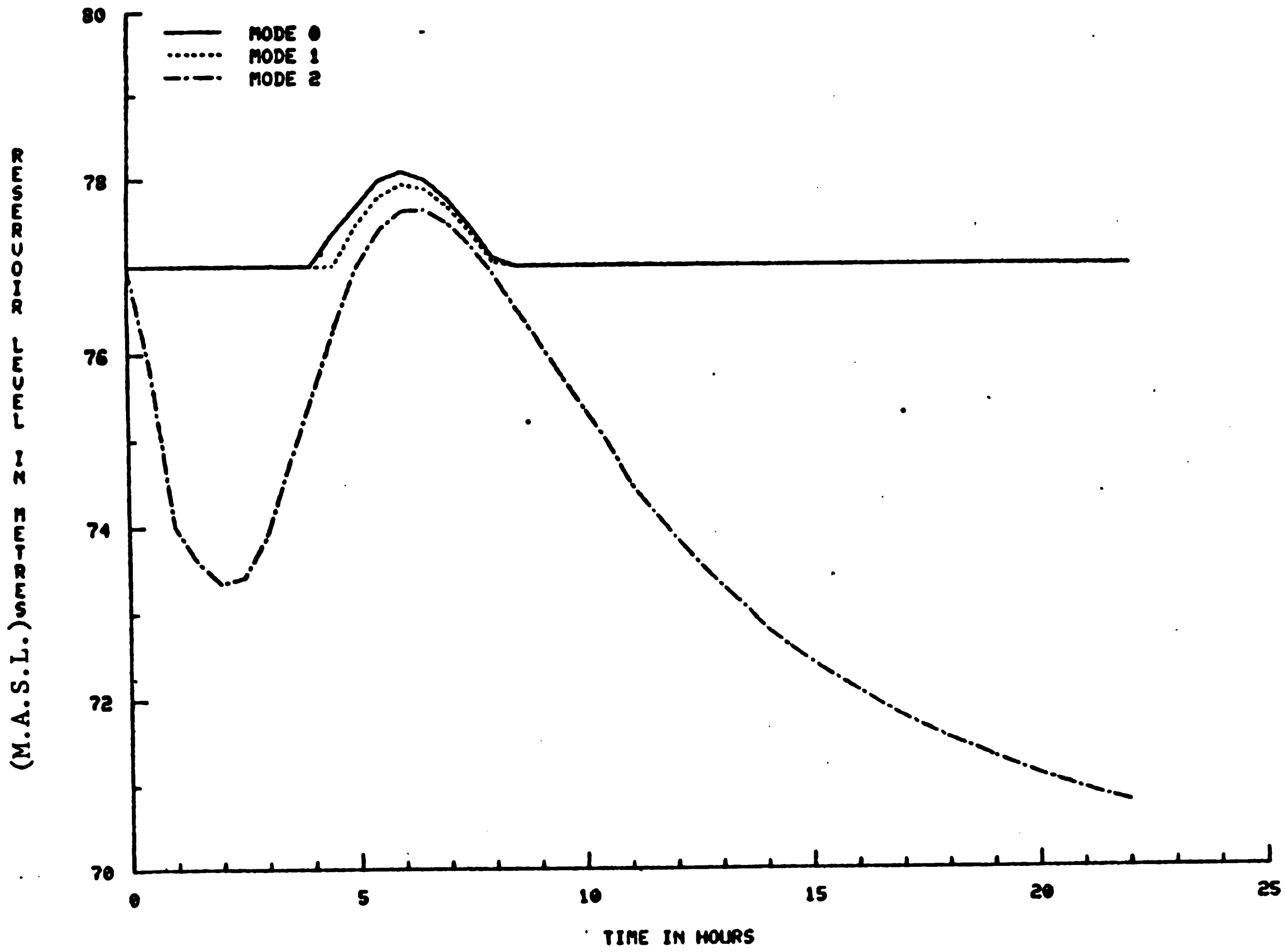


Figure 3.6.32. Simulated stage hydrographs of non-hurricane SPF at Las Barias reservoir - AMC III.

Table 3.6.6. Routing Hurricanes SPF under Different Modes of Operation

AMC	DESCRIPTION	MODE 0	MODE 1	MODE 2
AMC-I	Valdesia			
	Peak Inflow (m^3/sec)	10185	10185	10185
	Peak Outflow (m^3/sec)	7332	7249	6632
	Peak Reservoir Level (m)	154.29	154.22	153.72
		(Above max. design pool level)		
	Las Barias			
	Peak Outflow (m^3/sec)	7252	7178	6628
	Peak Reservoir Level (m)	79.43	79.35	78.85
AMC-II	Valdesia			
	Peak Inflow (m^3/sec)	14663	14663	14663
	Peak Outflow (m^3/sec)	10697	10661	10302
	Peak Reservoir Level (m)	Above max. design pool level (> 154.0 m)		
	Las Barias			
	Peak Outflow (m^3/sec), Peak Reservoir Level (m)	Above max. design pool level (> 79.5 m)		
AMC-III	Valdesia			
	Peak Inflow (m^3/sec)	16547	16547	16547
	Peak Outflow (m^3/sec), Peak Reservoir Level (m)	Above max. design pool level (> 154.0 m)		
	Las Barias			
	Peak Outflow (m^3/sec), Peak Reservoir Level (m)	Above max. design pool level (> 79.5 m)		

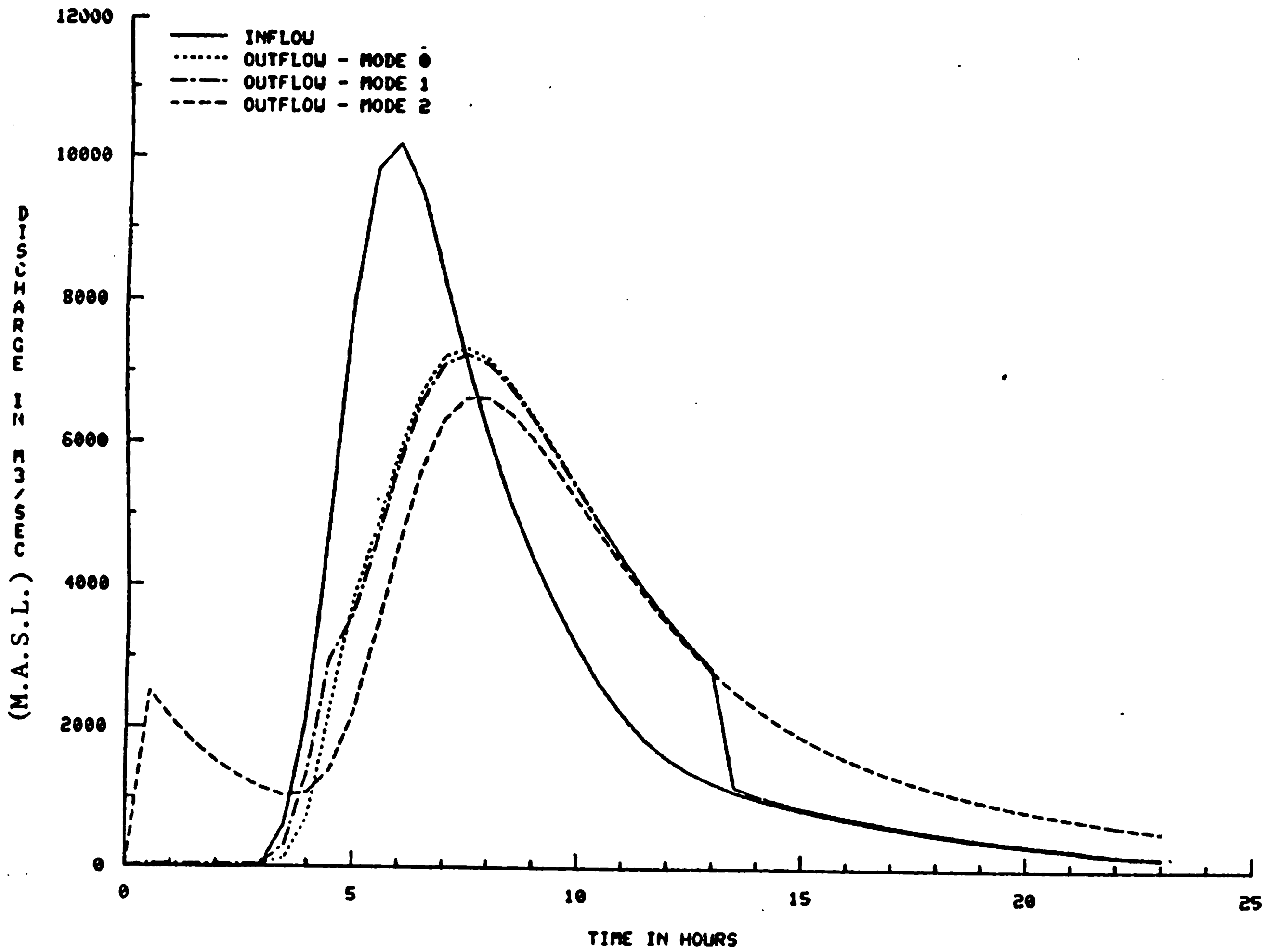


Figure 3.6.33. Inflow and simulated outflow hydrographs of hurricane SPF at Valdesia reservoir - AMC I.

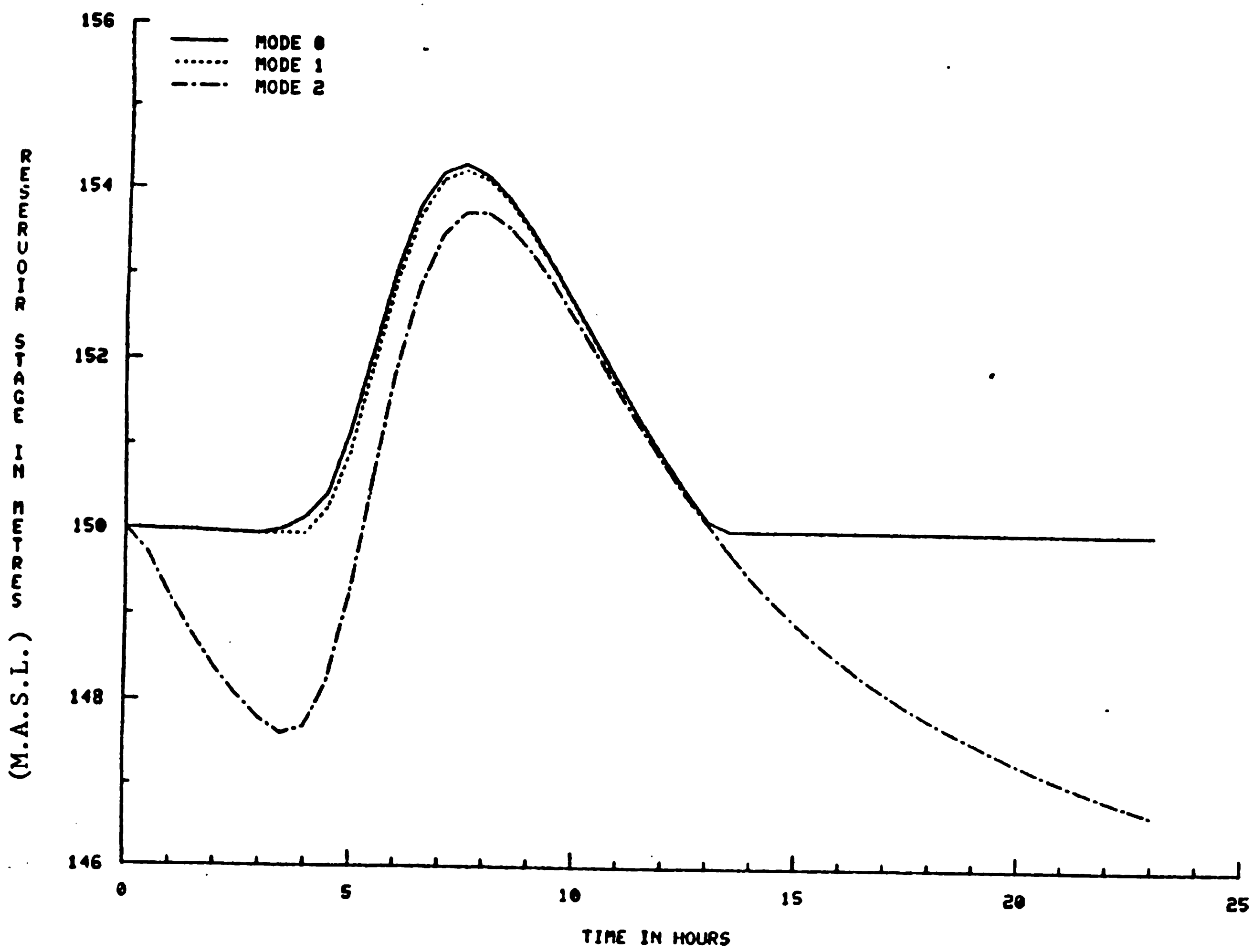


Figure 3.6.34. Simulated stage hydrographs of hurricane SPF at Valdesia reservoir - AMC I.

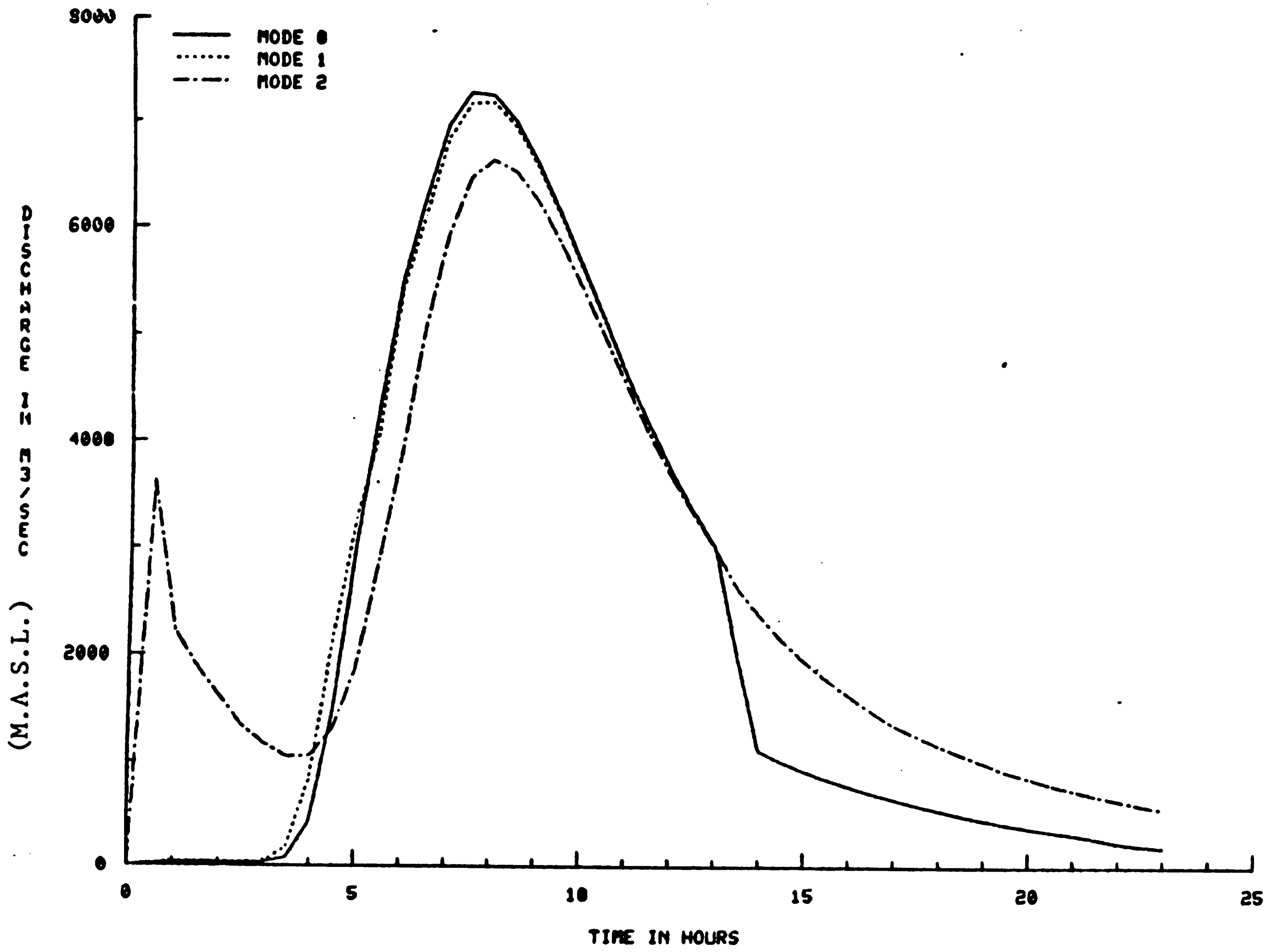


Figure 3.6.35. Simulated outflow hydrographs of hurricane SPF at Las Barias reservoir - AMC I.

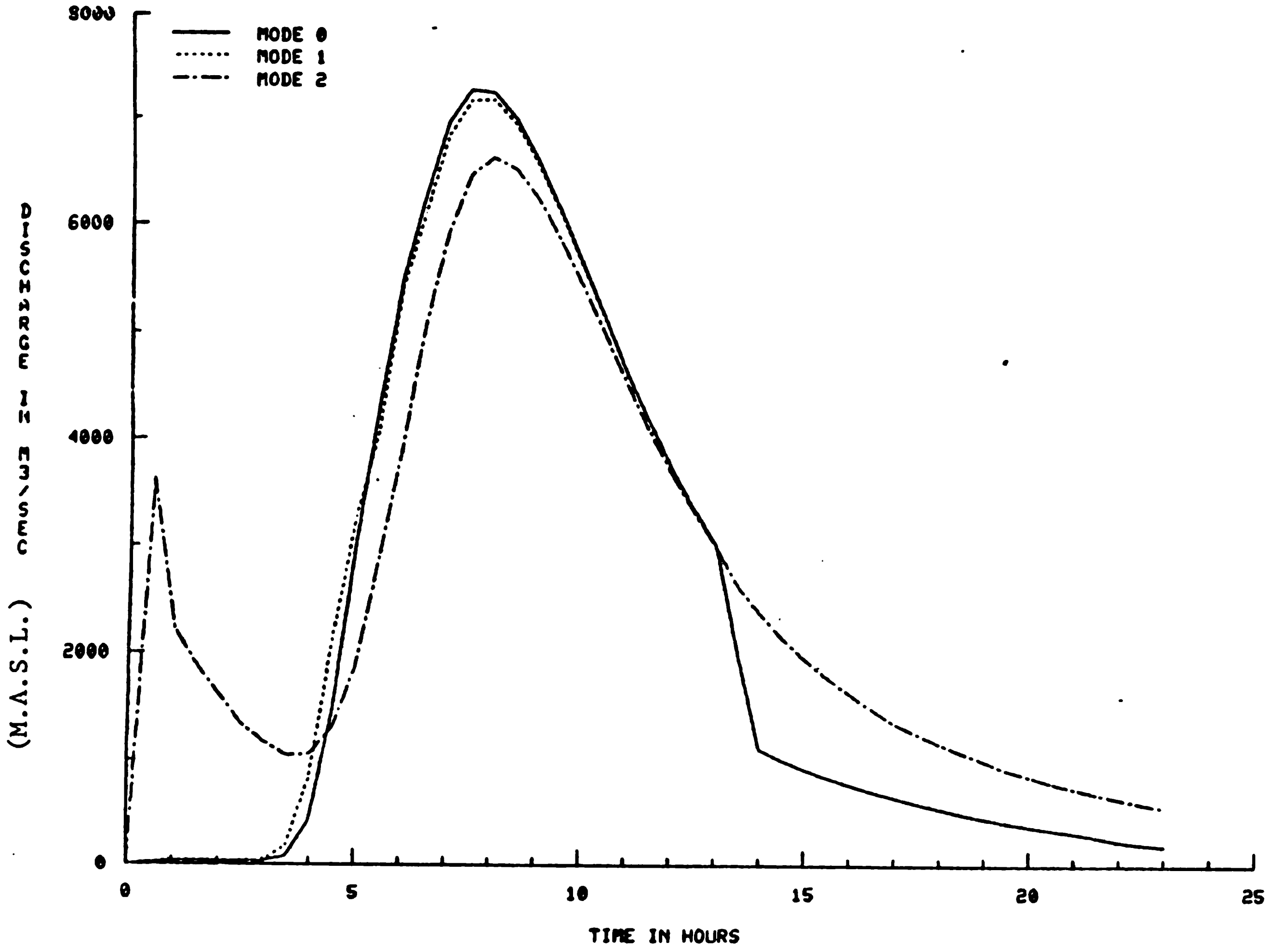


Figure 3.6.35. Simulated outflow hydrographs of hurricane SPF at Las Barias reservoir - AMC I.

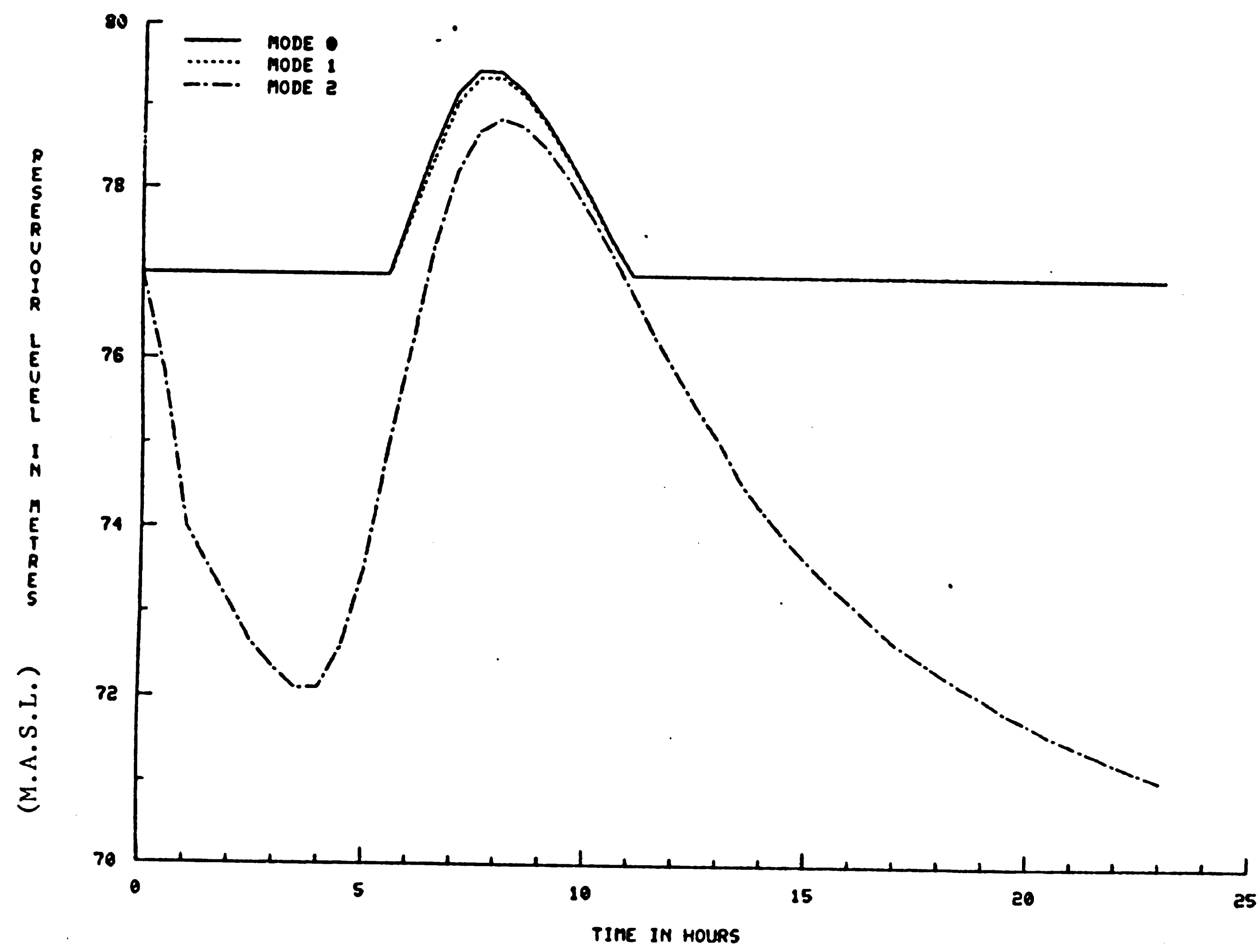


Figure 3.6.36. Simulated stage hydrographs of hurricane SPF at Las Barias reservoir - AMC 1.

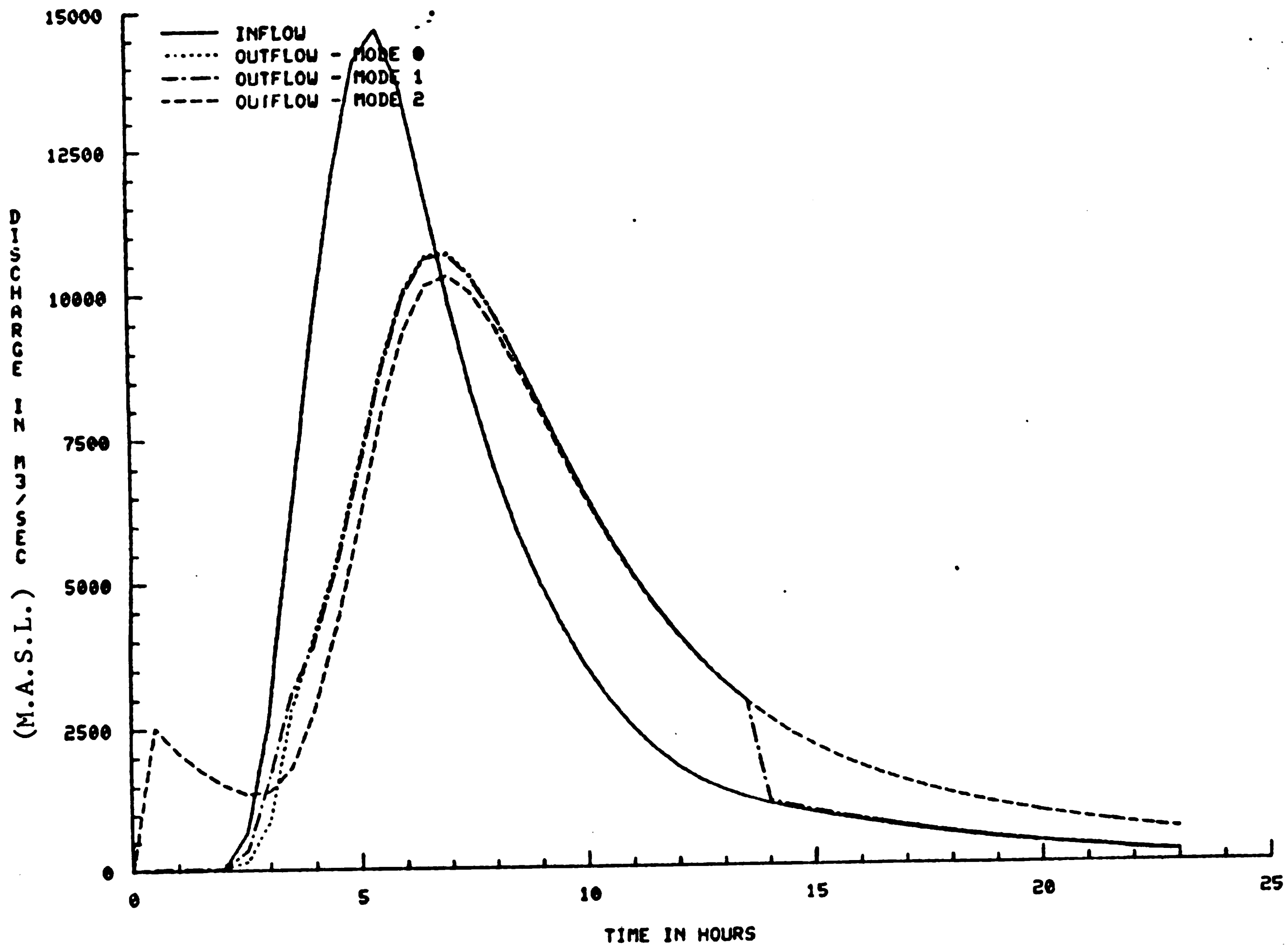


Figure 3.6.37. Inflow and simulated outflow hydrographs of hurricane SPF at Valdesia reservoir - AMC II.

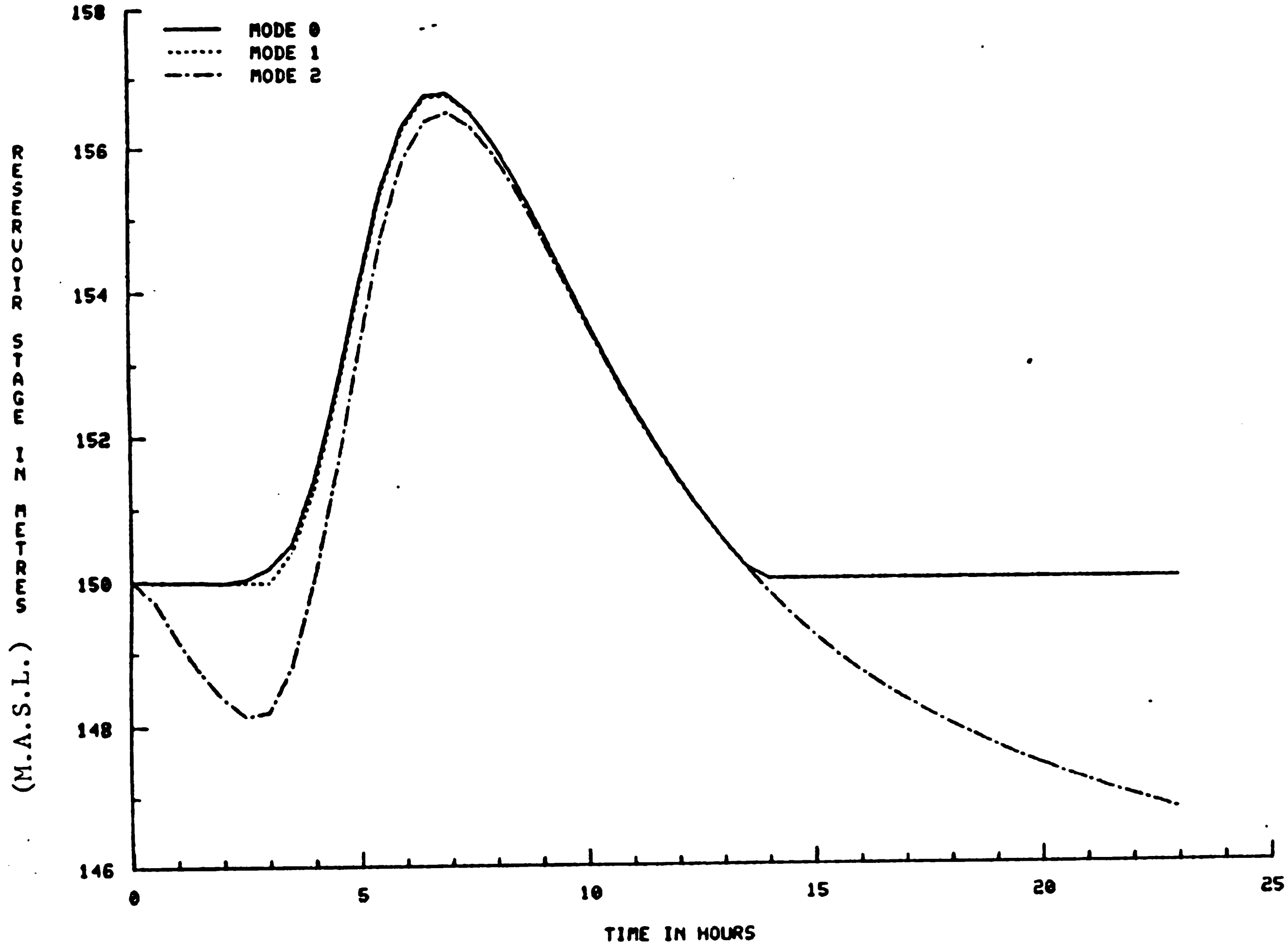


Figure 3.6.38. Simulated stage hydrographs of hurricane SPF at Valdesia reservoir - AMC II.

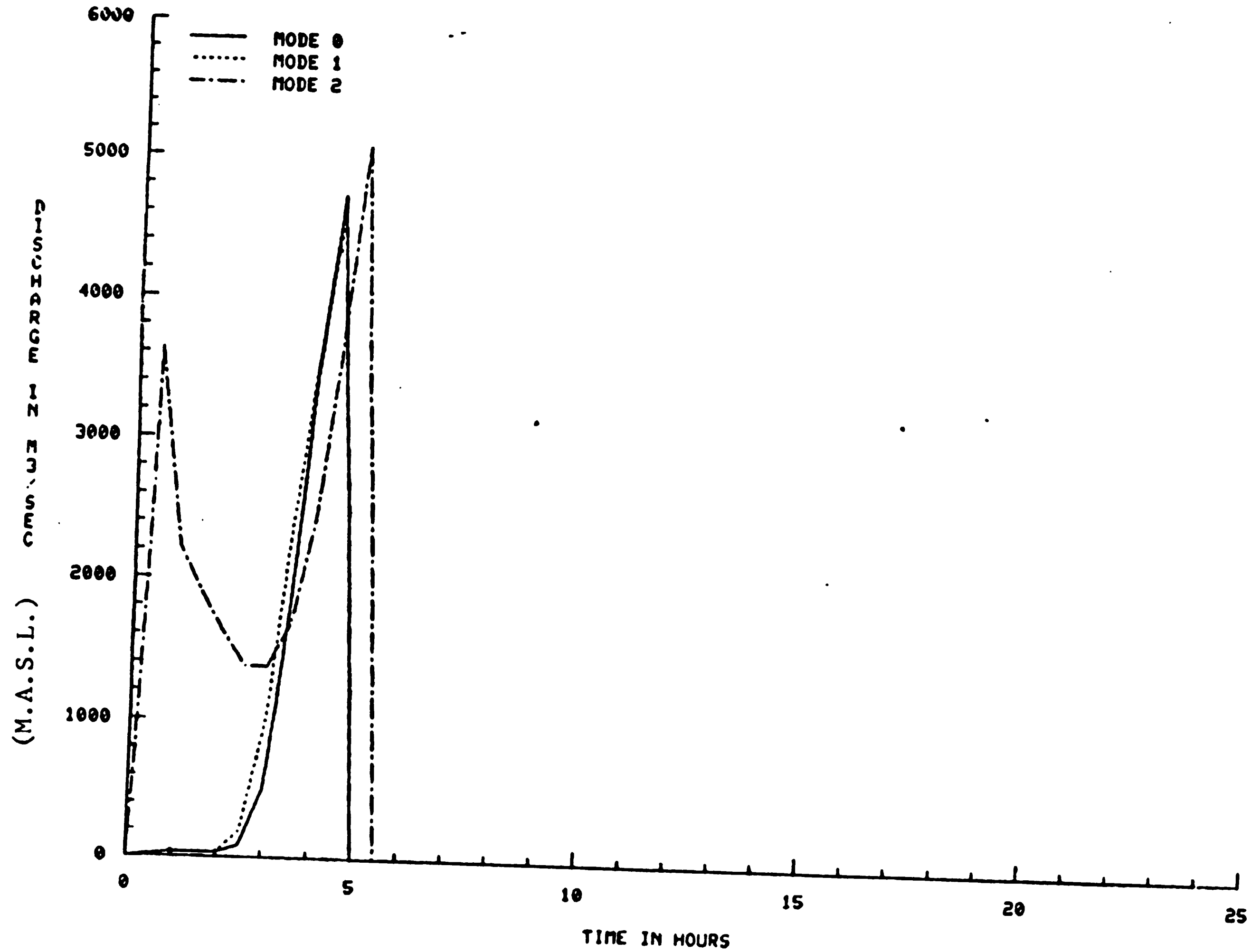


Figure 3.6.39. Simulated outflow hydrographs of hurricane SPF at Las Barias reservoir - AMC II.

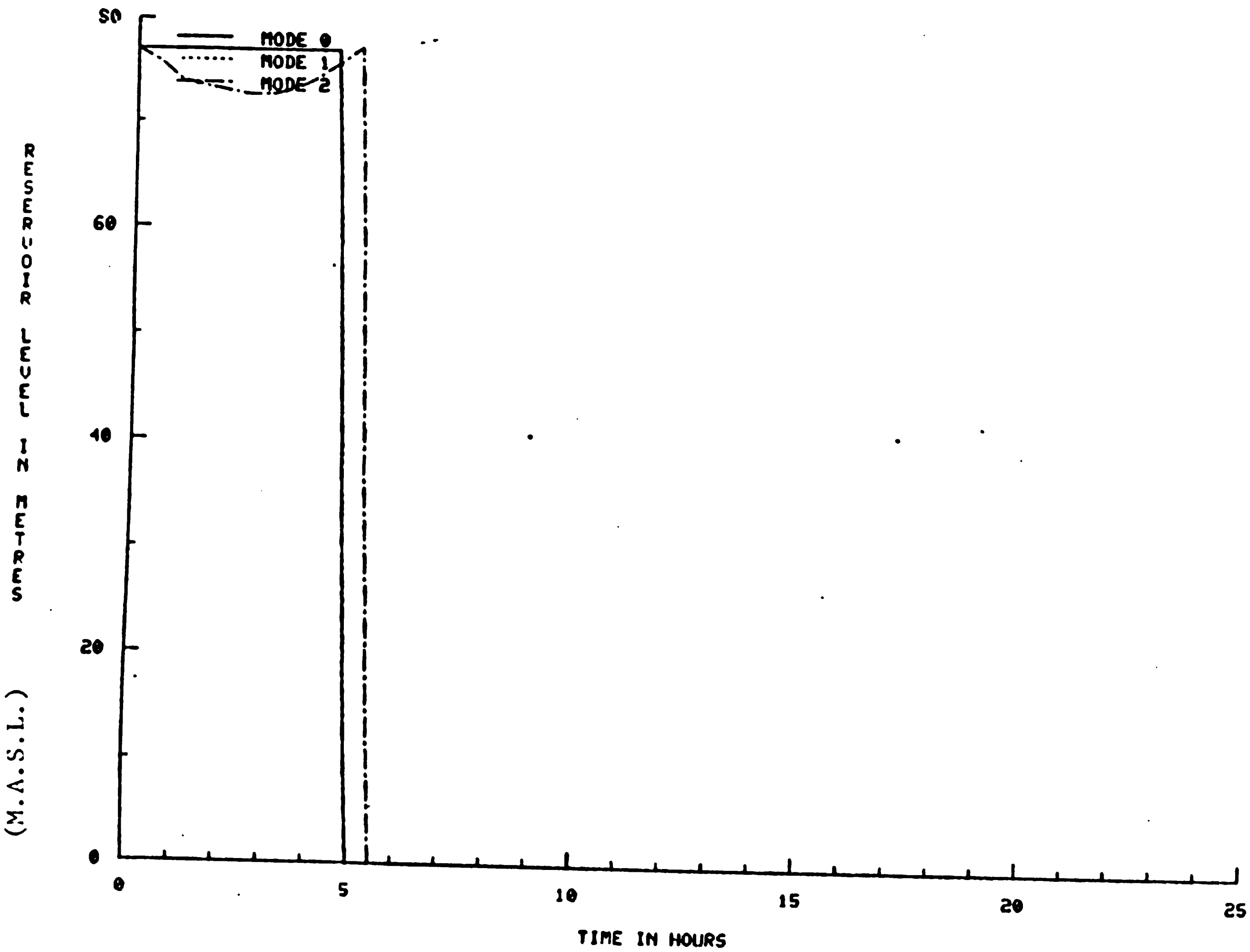


Figure 3.6.40. Simulated stage hydrographs of hurricane SPF at Las Barias reservoir - AMC II.

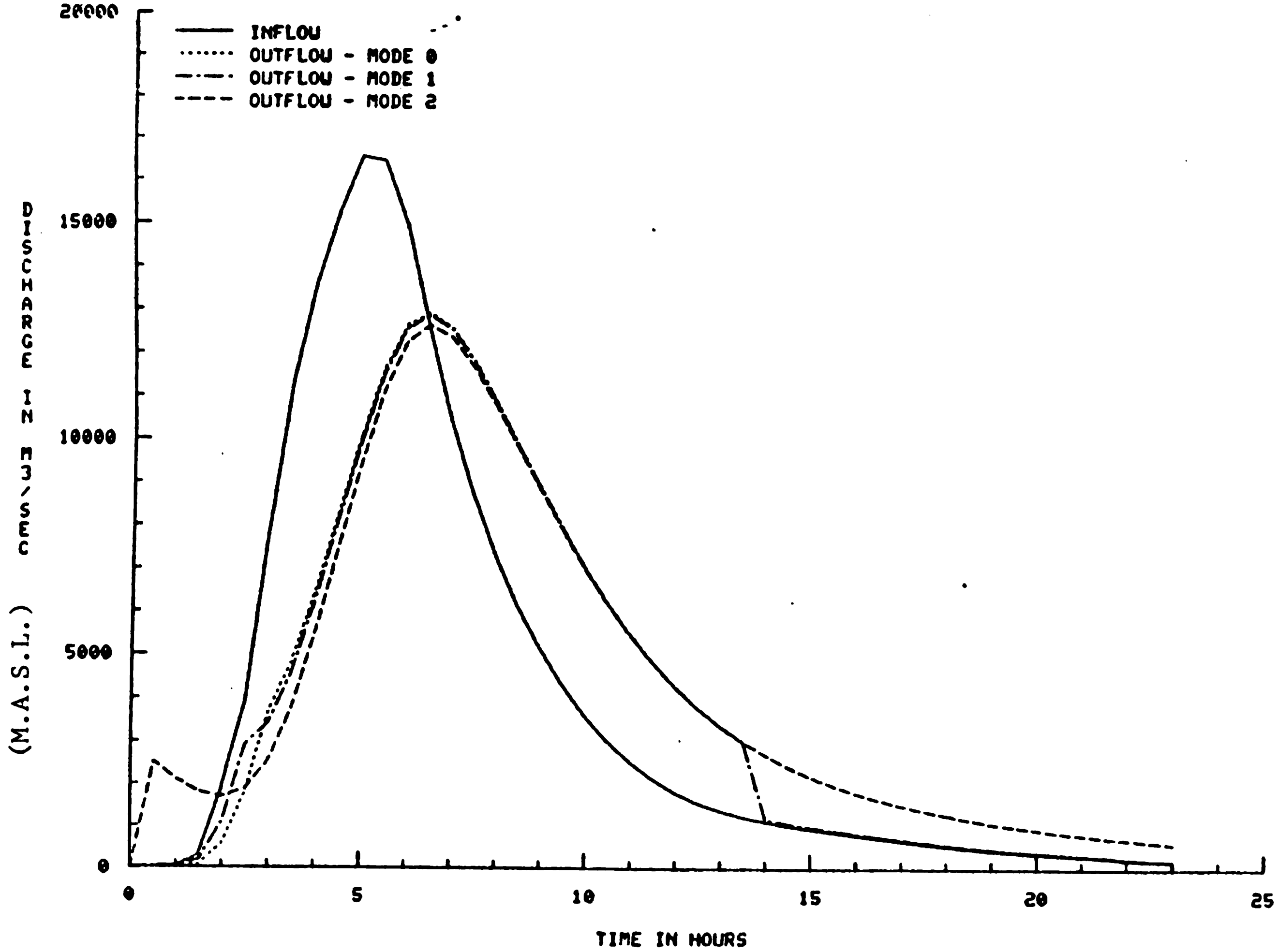


Figure 3.6.41. Inflow and simulated outflow hydrographs of hurricane SPF at Valdesia reservoir - AMC III.

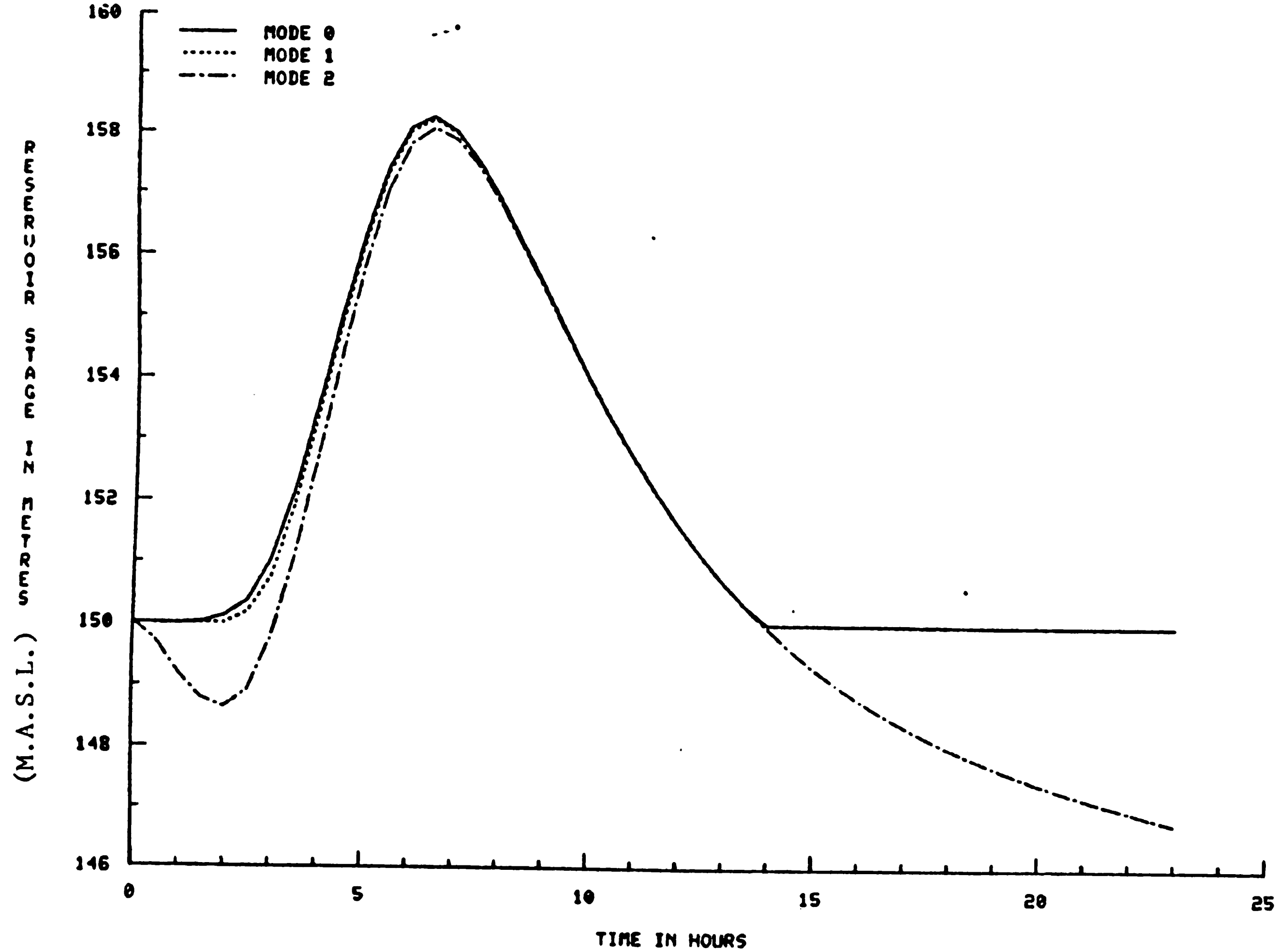


Figure 3.6.42. Simulated stage hydrographs of hurricane SPF at Valdesia reservoir - AMC III.

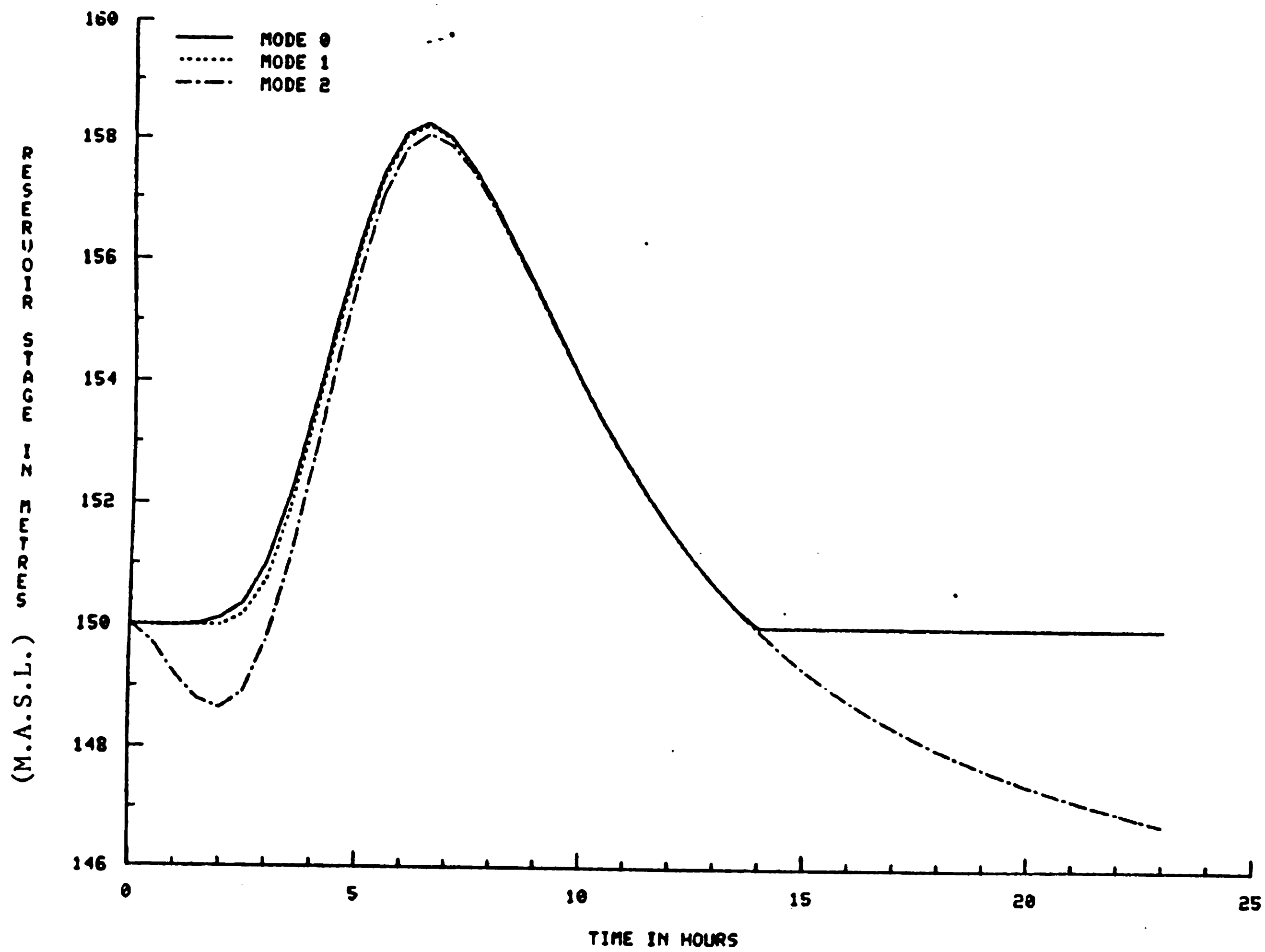


Figure 3.6.42. Simulated stage hydrographs of hurricane SPF at Valdesia reservoir - AMC III.

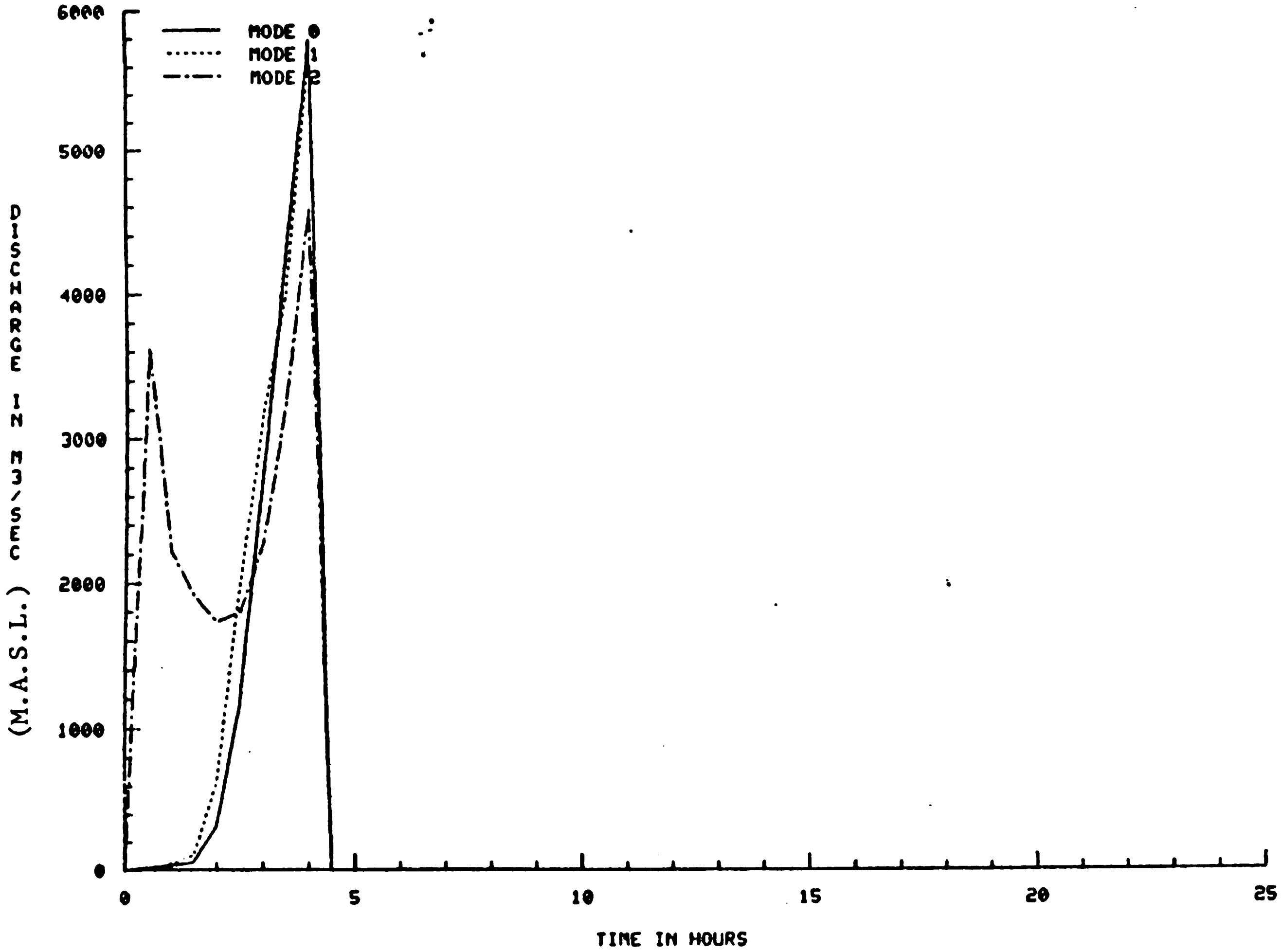


Figure 3.6.43. Simulated outflow hydrographs of hurricane SPF at Las Barias reservoir - AMC III.

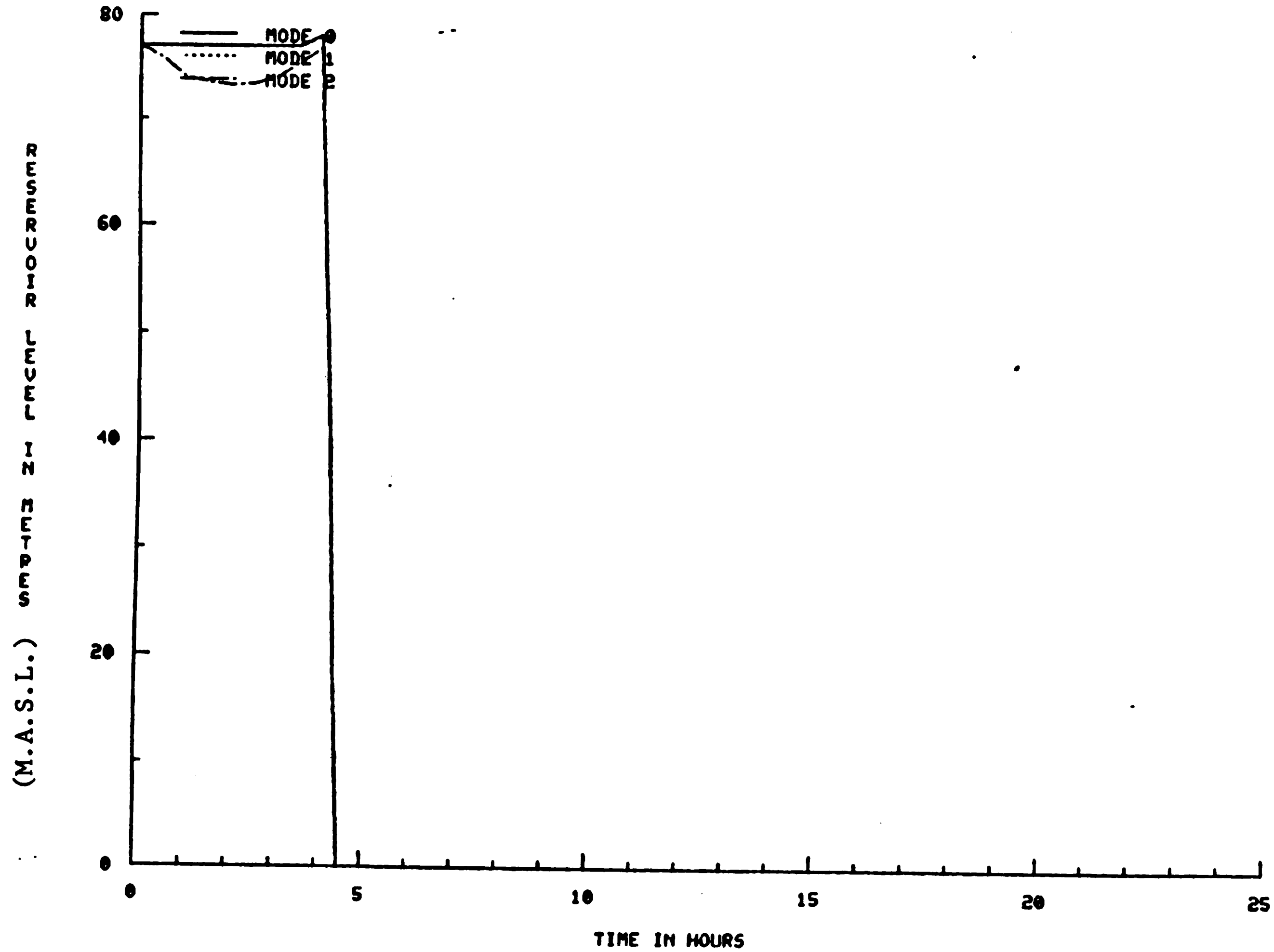


Figure 3.6.44. Simulated stage hydrographs of hurricane SPF at Las Barias reservoir - AMC III.

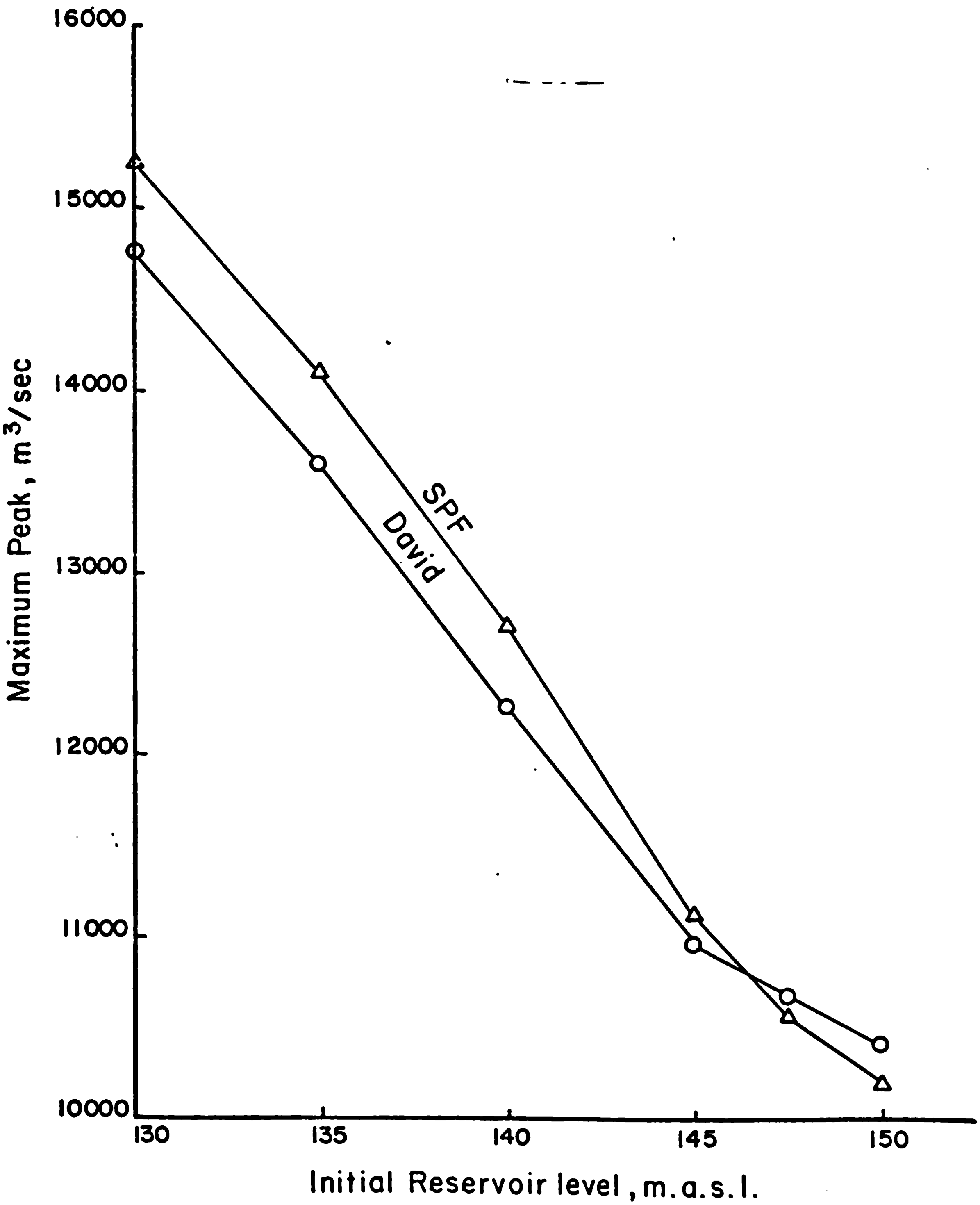


Figure 3.6.45 Maximum peak inflow of the hydrographs patterned after those of hurricane David and SPF which can safely pass through Valdesia reservoir without overtopping 154 m for different initial pool levels

3.6.4 Probable Maximum Flood

The Probable Maximum Flood (PMF) has a magnitude of 20,285 m^3/sec to 23,053 m^3/sec, depending on the antecedent moisture conditions. The runoff volume is also very large. The following table compares the rainfall and runoff volumes of Hurricane David, Hurricane SPF and PMF.

	Rainfall/Runoff		
	David	SPF	PMF
Average Basin Precipitation (mm)	372	493	1338
Runoff/AMC-I ($X10^6 m^3$)	125	194	708
Runoff/AMC-II ($X10^6 m^3$)	194	280	833
Runoff/AMC-III ($X10^6 m^3$)	245	338	902

From the table, it can be seen that the PMF under all antecedent moisture conditions are significantly higher than Hurricane David and SPF floods. The spillways of Valdesia and Las Barias do not have sufficient capacity to cope with floods of such magnitude. The PMF will certainly overtop the dams under all conceivable modes of operation and for this reason, no meaningful routing study can be conducted.

The counterpart personnel in CDE have developed a PMF using the same PMP that was used here but in a different rainfall-runoff model. Specifically, the model used was the CDE-version of the Stanford Watershed Model but with parameters calibrated for a nearby basin. It is noted that this model uses a storage routing technique (Clark's method) in contrast to the kinematic wave routing technique in HEC-1. The hydrograph computed by CDE is of the order of 16,000 m^3/s even with no infiltration. The differences may be due to one or both of the following reasons:

1. Storage routing technique, depending on the parameter values can significantly attenuate the hydrography whereas the kinematic routing technique has no built in facility to attenuate the hydrograph. However, it was shown in Section 1.7 that, when the Clark method, calibrated for the larest recorded flood is used with PMF, the magnitude of PMF is comparable to that computed by the kinematic wave method;
2. The parameters in the Stanford Watershed Model used by CDE were not calibrated for Nizao basin. These parameters apparently do not reproduce the fast rise observed in the actual recorder hyrographs.

In any case, the routing of PMF computed by CDE was found to exceed the maximum water level of 154 m.a.s.l. even with the initial level of 132 m.a.s.l. prior to the arrival of PMF.

3.6.5 Influence of Upstream Reservoirs

The routing studies carried out thus far have shown that the spillway at Valdesia (also true of Las Barias) is undersized to pass safety the SPF and PMF. Based on reconstructed Hurricane David flood and SPF, the limiting peak inflows that can be allowed (i.e., peak reservoir elevation not exceeding 154.0 m) have been estimated as follows:

Initial Reservoir Elevation at Valdesia	Limiting Peak Inflow
150.0 m	10,200 m^3/sec
145.0 m	11,000 m^3/sec

It has also been shown in Section 3.5.3 that drawdown of the reservoir to levels below 145.0 m is time consuming because of capacity

constraints in the release facilities. All the above point to the need for special provisions to handle large floods to ensure safety of the dam against overtopping. In this respect, there are two possible remedies:

(1) to increase the spillway capacity, and

(2) to retain flood flows in upstream storage.

For an existing dam, increasing the spillway capacity may not be easy, both from the technical feasibility and economic standpoints. The second alternative is less complicated if suitable storage sites can be found. In the water resources development masterplan of Nizao River Basin, a number of suitable storage sites for hydropower development have been identified. Of these sites, the one at Jiguey seems to be most promising in as far as flood control relief to Valdesia is concerned.

The proposed Jiguey Reservoir is located at the confluence of Mahona River with Nizao River, about 36 km upstream of Valdesia. The topography of the site permits the construction of a large storage reservoir at a modest cost. It was originally proposed as a hydropower facility with a normal pool level of 540 m. An initial assessment on the basis of topographical layout plans, suggests that there may not be any major technical difficulties in constructing a higher dam to provide for additional flood retention storage. Using HEC-1 program, the effects of Jiguey Reservoir on the resulting flood hydrograph at Valdesia have been investigated under different assumptions of flood control storage (at proposed Jiguey Reservoir). For simplicity, a simple overflow type spillway of a 120 m width was assumed. The results of the study are summarized below.

CASE 1 - JIGUEY SPILLWAY CREST AT 550 M

(Flood control storage of 56 million m^3)

Flood*	Peak Flows at Valdesia (m^3/sec) Original	With Jiguey Reservoir
David	8729	3193
SPF	14663	6587
PMF	22401	19176

CASE 2 - JIGUEY SPILLWAY CREST AT 560 M

(Flood control storage of 123.4 million m^3)

Flood*	Peak Flows at Valdesia (m^3/sec) Original	With Jiguey Reservoir
David	8729	2613
SPF	14663	5022
PMF	22401	17413

* Flood under AMC-II conditions.

From the above preliminary study, it is evident that Jiguey Reservoir can be highly effective in reducing the peak inflows to Valdesia for floods such as Hurricane David and SPF. In the case of SPF, the original flood peak of 14,663 m^3/sec is reduced to 6587 m^3/sec with the provision of about 56 million m^3 flood storage at the proposed Jiguey Reservoir. The attentuated SPF flood (6587 m^3/sec peak discharge) can be safely accommodated by the Valdesia-Las Barias Reservoir system. The above conclusion can be expected to hold for SPF under all AMC conditions.

In the case of PMF, the results are not encouraging. Due to the magnitude of the runoff volume, even a large flood control storage (120 million m^3) at Jiguey Reservoir will not reduce the peak discharge to a level which is low enough for safe passage through Valdesia Reservoir. If the reservoirs were to be safe against PMF, some major works have to be implemented, involving perhaps a combination of the two possible remedial measures outlined earlier. This would constitute a major engineering study which should include, among other things, a review of the hydrological estimation of PMF.

3.6.6 Final Remarks and Recommendations

Extensive studies have been carried out to study the behavior of the Valdesia-Las Barias Reservoir system subject to different types of flood inflow and under different assumptions of modes of operation. From these studies, the following recommendations are developed as guidelines to future operation/planning:

(1) The Valdesia spillway can safely cope with floods with peak flows of the order of 10,000 to 11,000 m^3/sec, corresponding

to initial reservoir level of 150 m and 145 m, respectively. Although higher peaks (up to 15,000 m^3/sec) can be accommodated if the reservoir level is drawn down to about 130 m, such a practice may not be practical because of the very long advance time (up to 260 hours) required for such an operation. It is noted however, that this conclusion is based on the hydrograph shapes of Hurricane David and SPF while these numbers may change for other hyrograph patterns.

(2) For small and medium floods (up to 5,000 m^3/sec), the hurricane mode of operation is not advised because it can result in a sudden outflow surge which can be damaging to downstream areas. If hurricane mode of operation is used during a large flood, it should be discontinued after the peak inflow has passed by reverting back to mode 0 or 1, so that the reservoir storage will not be unnecessarily depleted, unless a final reservoir elevation of 145.0 m is desired.

(3) For smaller floods (less than 2500 m^3/sec), the induced surcharge mode may be more effective in suppressing the outflow peak, particularly if the initial reservoir level is low and the reservoir pool level does not exceed the highest level corresponding to the induced surcharge envelope curve (i.e., 151 m.a.s.l.). If the flood is large enough such that reservoir outflow enters into uncontrolled phase (corresponding to spillway rating curve) then induced surcharge mode and inflow equals outflow modes may give similar results. For large floods, such as SPF and Hurricane David, the induced surcharge mode is likely to result in higher peak outflow

rates and reservoir elevation than the other modes of operation and is, therefore, not recommended.

(4) For medium and large floods, the induced surcharge method and "outflow equals inflow" modes of operation will result in quite similar outcomes. There is no distinct advantage between one method over the other, although computation-wise, the induced surcharge method is more complex and requires the estimation of a routing time constant (Ts). However, possible variations can be expected depending on the duration and peakiness of the inflow hydrograph. For best results, it is suggested that on-line simulation of the two modes of operation be carried out so that the operator can decide on the better mode of operation based on the results of the simulation.

(5) The Valdesia spillway will not be able to cope with the SPF (under AMC-II and -III conditions) and the PMF (underall AMC conditions). If the proposed Jiguey Reservoir is built with some provision for flood retention storage, the peaks of SPF can be effectively controlled for safe passage through the Valdesia-Las Barias system. In the case of PMF, Jiguey Reservoir alone will not be able to provide the required level of control .

3.7 DEVELOPMENT OF OPERATING RULES

3.7.1 Introduction

In Section 3.6, various historic and design floods were routed through the Valdesia-Las Barias Reservoir system under different operating assumptions to determine the resulting outflow and stage hydrographs. The results of the study have shown that

(1) Within practical limits of advance drawdown of reservoir storage in Valdesia, the existing system can cope with a peak flood of the order 11,000 m^3/sec.

(2) For small floods and under low initial reservoir levels, the induced surcharge method may be more effective in reducing the outflow peak.

(3) For medium floods and large floods, there is little difference between the results obtained by induced surcharge routing (Mode 0) and 'outflow equals inflow' method (Mode 1). Both methods can be used although the latter is computationally simpler.

(4) For large floods, the hurricane mode of operation (Mode 2) may be advantageous because additional flood control space (up to about 40 million m^3) can be created to help to absorb the incoming flood.

The above results will form the basis for formulation of flood operation rules for the operation of the Valdesia-Las Barias system.

In real-time operation, the forecast of inflows to a reservoir requires advance knowledge of precipitation and its time distribution. There are many flood forecasting models that operate on deterministic input of precipitation. In the case of Valdesia, given the short time

of flood response, such an approach may be of little use. There is, therefore, a need for precipitation forecasting, a technique which is still at the infancy of research/development. In Section 3.8, a state-of-art review of precipitation forecasting is provided. In view of the lack of suitable models, a simplified precipitation forecast model is used. There is, therefore, potential for refinement at the later stage as better understanding and new techniques are developed in this field.

3.7.2 Estimation of Average Nizao Basin Precipitation Resulting from a Hurricane

Assuming that it is possible to forecast the precipitation potential of an oncoming hurricane, then the next step of study is to estimate the average Nizao Basin precipitation using an acceptable areal interpolation/extrapolation technique. For this purpose, Program PCMAP described in Volume I is ideally suited. By incorporating a feature for translation and rotation of the position vector of rainfall stations used by PCMAP, it is possible to simulate the effects of a shift in storm centre (or the 'eye' of a hurricane) resulting from different assumptions of tracks and distance of an incoming hurricane. The detailed procedures are as follows:

(1) Identify the track of the hurricane by the angle of intersection that the track makes with the longitudinal centroidal axis of the Nizao Basin (a positive angle is in clockwise direction).

(2) Determine the shortest perpendicular distance from the centroid of Nizao Basin to the track of the hurricane.

(3) Position the 'eye' of the hurricane at different locations along the track.

(4) Determine the resulting average Nizao Basin precipitation for each 'eye' location and pick up the maximum value along the assumed track.

The following table summarizes the results of this study.

Angle θ	Average Nizao Basin Precipitation (mm) for hurricane distance of 0 km	25 km	50 km	75 km	100 km
0°	452.6	360.0	300.0	313.7	310.0
15°	443.4	362.6	295.2	313.4	310.1
30°	436.0	360.1	289.6	312.8	310.6
45°	430.2	354.1	291.7	312.6	310.9
60°	426.3	351.1	291.1	313.5	311.1
-15°	461.5	360.1	302.2	313.6	309.2
-30°	464.7	362.3	303.1	313.4	309.2
-45°	458.1	361.1	302.7	313.4	309.4
-60°	445.6	354.2	301.4	313.5	310.0

In deriving the above results, the observed storm precipitation of Hurricane David was used as the base reference. From the study, it can be seen that:

(1) Track angle is not a sensitive parameter as compared to the perpendicular distance of the hurricane from the centroid of Nizao Basin.

(2) If Hurricane David is exactly on track (in Nizao Basin), a potential precipitation of about 465 mm can be expected (as compared to the 372 mm predicted for the actual track).

(3) If the hurricane is more than 50 miles away, the distance parameter seems to lose its effect on the resulting Nizao Basin precipitation. This conclusion is in conflict with intuitive reasoning and could have been caused by errors introduced in extrapolation (used by Program PCMAP).

In the lack of better defined information, the above study has been used in deriving a relationship for precipitation potential against hurricane distance for use in developing hurricane operation rules. The relationship is certainly crude and approximate and should be refined as more data or better techniques become available at a later stage.

3.7.3 General Plan of Operation

Operation of the Valdesia-Las Barias system during flood conditions resulting from large storms arising from tropical cyclones (hereafter referred to as "hurricane conditions") or small to moderate storms arising from local weather phenomena or weak tropical storms (hereafter referred to as non-hurricane conditions) is primarily in the interest of the safety of the dam. The downstream flooding under various discharge conditions is not explicitly considered in the development of operating plans although this has received attention in the development of different modes of operation. A future study must be undertaken to identify downstream damage areas and the associated extents of inundation which can be used to modify the operating plans, if necessary.

3.7.4 Operational Objectives for Flood Control

It is clear that the small capacity (only about 40 million m^3) of the flood control space and the particular spillway and outlet characteristics of the Valdesia Reservoir limit the effectiveness of operation by way of advance emptying of the reservoir for controlling major floods such as those due to hurricanes. In view of severely limited capacity of outlet works for water levels below 145 m.a.s.l. and

the consequent large time to drawdown, the advance operation for flood control is only practical for water levels between 145 m.a.s.l. to 150 m.a.s.l.

During non-hurricane floods, the releases should be sufficient to limit the pool rise to elevation 150 m.a.s.l. If it is predicted that the storage of inflow is such that the pool elevation would rise above 150 m.a.s.l., release flow sufficient to limit the pool rise to elevation corresponding to induced surcharge curve. In any case, consideration must be given to the maximum permissible downstream discharge to control flood damages.

When the pool level is falling, it is maintained at elevation 150 m.a.s.l after it is reached by making subsequent releases equal to inflow, unless the forecasted meteorological conditions are unfavorable.

During hurricanes or severe tropical storms which are expected to result in large floods at Valdesia dam site, the safety of the dam is the primary consideration. If the forecast of track and precipitation of the storm indicates a potential for overtopping, then releases must be made at the maximum rate possible without endangering the safety of the downstream communication if this is applicable. Proper warning to downstream damage centers must be made in advance so that communities may be evacuated on time. Once again, when the reservoir stage is falling after the passage of the peak; it is maintained at 150 m.a.s.l. by making subsequent releases equal to the inflow, unless forecasted meteorological conditions indicate otherwise.

[illegible]

3.7.5 Operation Procedures for Non-Hurricane Flood Emergency Conditions

During storm events leading to emergency conditions as specified by the executive committee for emergency operation, all regulation instructions shall be furnished by the emergency task force working with the committee. The conditions leading to the initiation of action by the committee are fully described in the accompanying manuals specially prepared for actual operation of Valdesia-Las Barias System. The task force will receive and analyze the hydrologic data and issue instructions for gate changes to the damtender by radio.

The operation procedures for emergency conditions arising due to non-hurricane storms are summarized in the flowchart presented in Figure 3 7.1. When the communications cannot be established between the damtender and the emergency task force working in the central offices of INDRHI and/or CDE, and the reservoir pool is rising at a critical rate, the damtender shall operate the reservoir according to the standing instructions provided in Appendix 3.7.A. The following section describes the procedures that must be followed once a non-hurricane emergency condition is declared.

3.7.5.1 Basic Hydrologic Data

The precipitation reporting network of Nizao watershed and the surrounding basins can be used to obtain real-time precipitation measurements from a storm. Since the response of Nizao watershed is rapid, measurement of precipitation after the storm has occurred may be of little use. However, for moving storms, the precipitation measured in other locations in the Dominican Republic may indicate the potential precipitation that may occur over Nizao. In any case, the real-time measurements obtained via Data Collection Platforms (DCP) would be

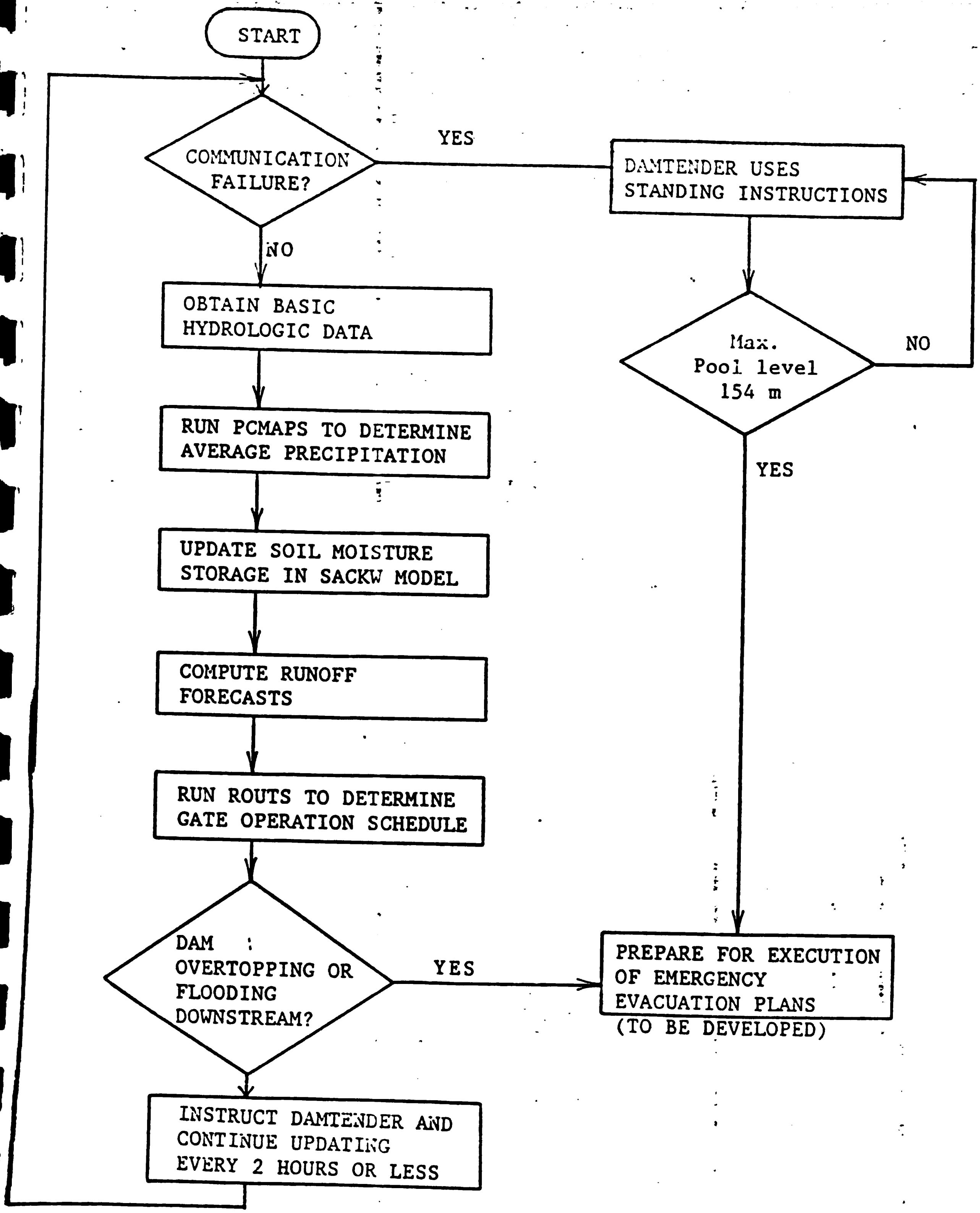


Figure 3.7.1. Flow chart of the nonhurricane flood operation procedure to be followed by the Committee for emergency operation.

extremely useful. Every attempt must be made to obtain precipitation forecasts from the National Meteorological Office. If no such arrangement exists, steps must be taken to obtain the expertise of this office to provide such precipitation forecasts.

In addition to precipitation, the river stages at gage locations where DCP's will be installed should be obtained. The damtender must act as an observer for reporting of precipitation at the dam site, reservoir elevation at both Valdesia and Las Barias.

3.7.5.2 Runoff Forecasts

The following procedures must be followed in computing runoff forecasts.

1. Use the PCMAPS program to compute the weights for which the precipitation data are available. This can be done prior to the emergency situation and stored for subsequent use. An interface program to be developed once all DCP's are installed will convert the incoming precipitation data into a form suitable for SACKW forecast model. The weights and the converted precipitation data will serve as a part of the input to runoff forecast model.

2. Update the soil moisture storages of the SACKW model. If it has not been operating for the last five days, use only the precipitation and streamflow data observed for last five days for updating soil moisture storages.

3. Compute runoff forecasts from the forecasted or measured precipitation and the updated soil moisture storages by using the real-time forecast facility of the SACKW model. The length of the forecast period may be determined on the basis of availability of precipitation data and its lead time.

4. Run the reservoir routing model ROUTS with the forecasted runoff and current pool levels at Valdesia and Las Barias. For medium floods, there is little difference between Mode 0 (induced surcharge method) and Mode 1 (inflow equals outflow after level has reached 147 m.a.s.l.) operations and the much simpler Mode 1 may be used. In doubtful cases, particularly for small to medium floods, route the inflow hydrograph for both models 0 and 1 and then choose the one which results in the most favorable conditions as far as downstream flooding is concerned. In any case, the output of the program specifies the gate operation schedules for both Valdesia and Las Barias.

5. If the routing results indicate the potential overtopping and/or downstream flooding which can cause loss of lines or heavy property damage, the steps must be taken immediately to execute the emergency evacuation plans. These plans are to be developed after a study on the downstream flooding for various magnitudes of floods. These plans must include three stages leading to evacuation: (a) alert of a possible evacuation (watch); (b) warning for preparation; (c) immediate evacuation. The lead times necessary for each stage must also be determined from the aforementioned study.

6. Instruct the damtender by radio as to what gate regulation should be until it is updated at a future time.

7. As new observations or forecasts of precipitation become available, update the gate operation schedules by repeating the procedures described above.

8. Once the passage of the flood is completed and the reservoir level has fallen to 150 m.a.s.l. if applicable, return to normal operation procedures unless the meteorological forecasts or observations

indicate a potential for another emergency condition due to a possible flood.

9. If for some reason, the above models are not operational, the standing instructions provided to the damtender (see Appendix 3.7.A) may serve as a guide for making decisions.

3.7.6 Operation Procedures for Hurricane Flood Emergency Conditions

A summary of operating procedures to be followed by the emergency task force during a hurricane emergency flood condition is given in Figure 3.7.2. Once again, the specifications for initiating the action by the emergency committee under hurricane flood emergency conditions are described in the accompanying manuals specially prepared for operation of Valdesia-La Barias system. Details of the procedures are given below:

1. The potential rainfall obtained from forecast services would be used as input to the runoff forecast model if the hurricane strikes the Nizao Basin. However, when the hurricane strikes at a point distance from Nizao, the amount of precipitation expected on Nizao would be less than the precipitation potential of the hurricane. Current technology of precipitation forecasting does not permit the accurate prediction of precipitation amount in such situations. However, based on the isohyetal pattern of precipitation of Hurricane David, an approximate procedure has been devised to compute the precipitation forecast on Nizao knowing the distance of the forecast track and the precipitation potential of the hurricane. Figure 3.7.3 presents a plot of the ratio as a function of distance, by which the potential precipitation may be multiplied to obtain precipitation forecast in

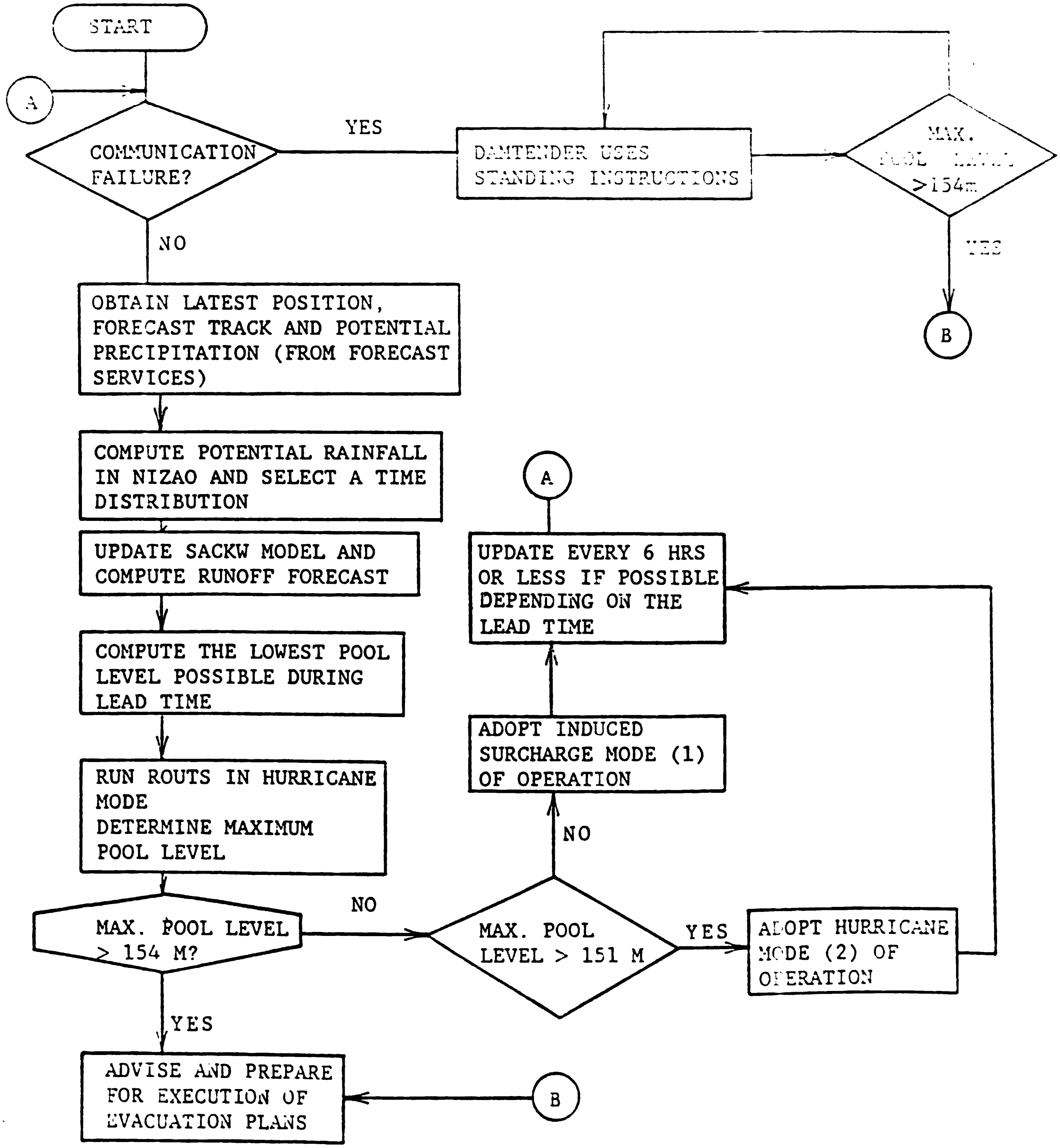


Figure 3.7.2. A flow chart of the hurricane flood operation procedures to be followed by the committee for emergency operation.

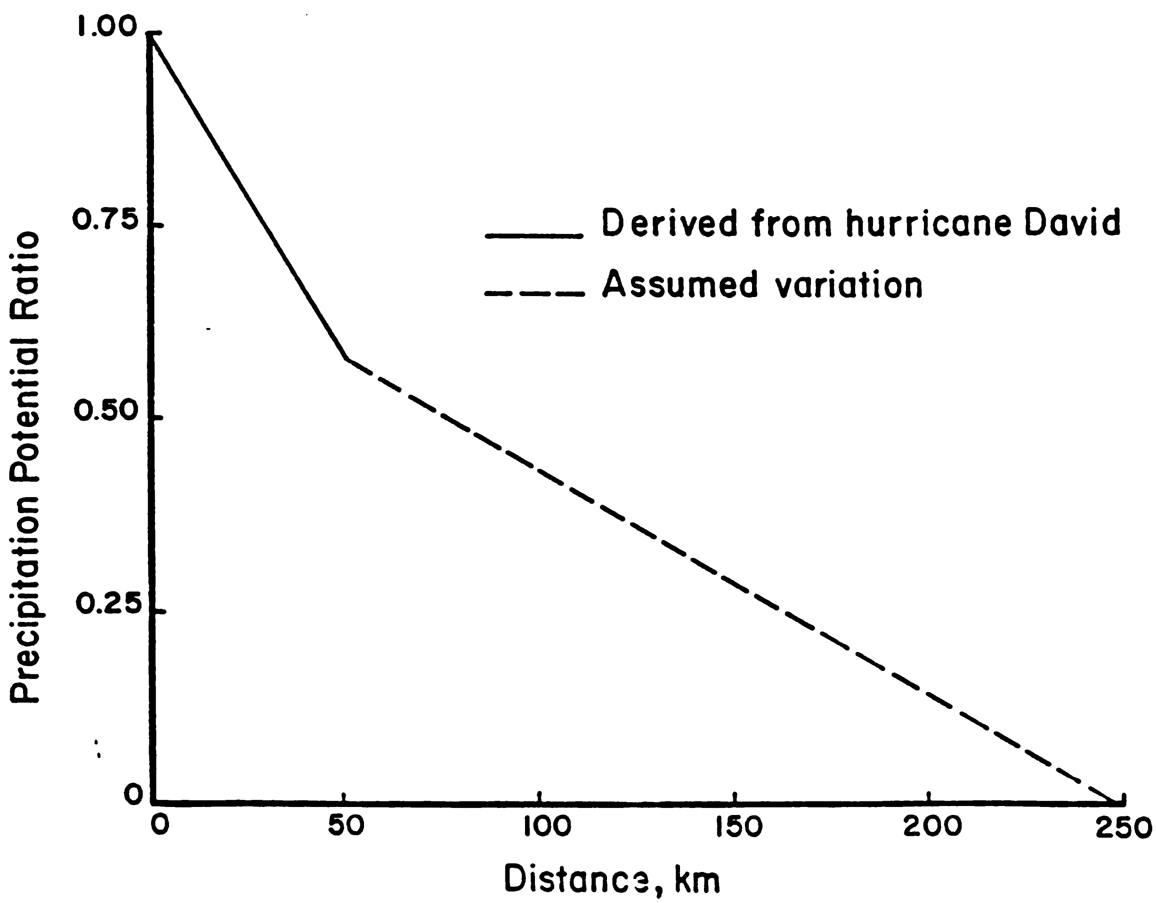


Figure 3.7.3. Proposed variation of precipitation potential ratio versus distance to the tack from the basin

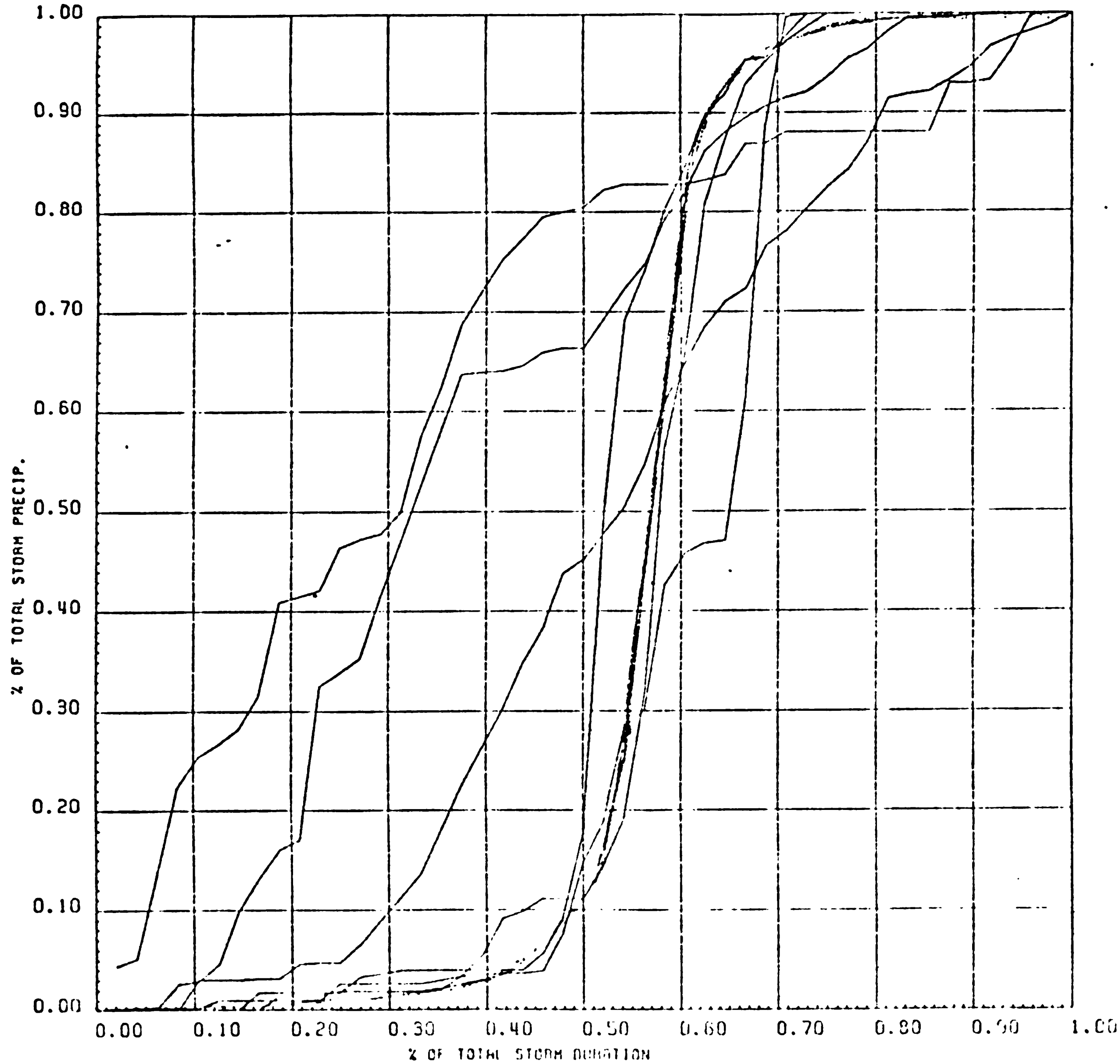


Figure 3.7.4. Historic rainfall patterns and selected design temporal distirubtions.

Nizao. It is expected that this procedure will be revised soon as new methods for precipitation forecasts become available.

2. Distribute the precipitation forecast in time by using the temporal distribution provided in Figure 3.7.4. This distribution is the same as that used for deriving DAD curves developed in Section 1.5. in the Volume I: Hydrologic Studies.

3. Update the soil moisture storages of the SACKW model. Use last 5 days of observed precipitation and streamflow for updating the storages. Once storages are updated, compute the runoff forecasts using the precipitation forecasts as inputs to the SACKW model.

4. Knowing the current pool level and the lead time of forecasts to determine the lowest pool level that to which the reservoir can be drawn down during the period of lead time.

5. Run the reservoir routing model ROUTS in hurricane mode of operation (i.e., Mode 2) to determine the forecast of maximum pool level. If the forecast level is greater than 154 m.a.s.l., advise the civil defense personnel to prepare for evacuation if the need arises at a later time, as a consequence of the hurricane strike. If the distance usind in computing precipitation forecast is the actual distance to the hurricane, and the precipitation mesurements obtained via DCP's and other communication facilities indicate heavy precipitation which would result in a catastrophic flood at the Valdesia dam, then steps must be taken immediately to evacuate the downstream communities which would be affected by large relesaes. The actual evacuation plans are to be developed after a study on downstream flooding under floods of various magnitudes. In the case of potential overtopping releases must be made according to the hyrricane mode of operation.

6. If the routing studies indicate no potential for overtopping but the maximum predicted pool level is above 151 m, the releases must still be made according to the hurricane mode of operation. However, if the pool level is predicted to be below 151 m, then the induced surcharge mode of operation may be followed. These decisions may change at a later time when more accurate forecasts are available.

7. Continue to update forecasts of tracks, precipitation, hydrographs and gate regulation schedules as new information regarding the motion of the tropical cyclone become available. Every effort must be made to update tracks at lead times shorter than 6 hrs particularly when the torm is predicted to be within 250 km.

8. When the reservoir stage is falling after the passage of the flood, the operation must be switched to Mode 1 (inflow equals outflow) once the pool level decreases to 150 m. Unless the forecasted meteorologic conditions indicate potential for another damaging flood.

9. In case of difficulties in using the models, the standing instructions provided to the damtender (see Appendix 3.7.A) may be used as a guide in making decisions.

APPENDIX 3.7.A

STANDING INSTRUCTIONS TO DAMTENDER

A.1 Flood Control Operation

During storm events leading to flood emergency conditions at the Valdesia dam site, all operation instructions shall be furnished by the emergency operation group housed in the central office in Santo Domingo. This group will receive and analyze the hydrologic data and issue instructions for gate operation by radio.

A.2 Flood Emergency Operation

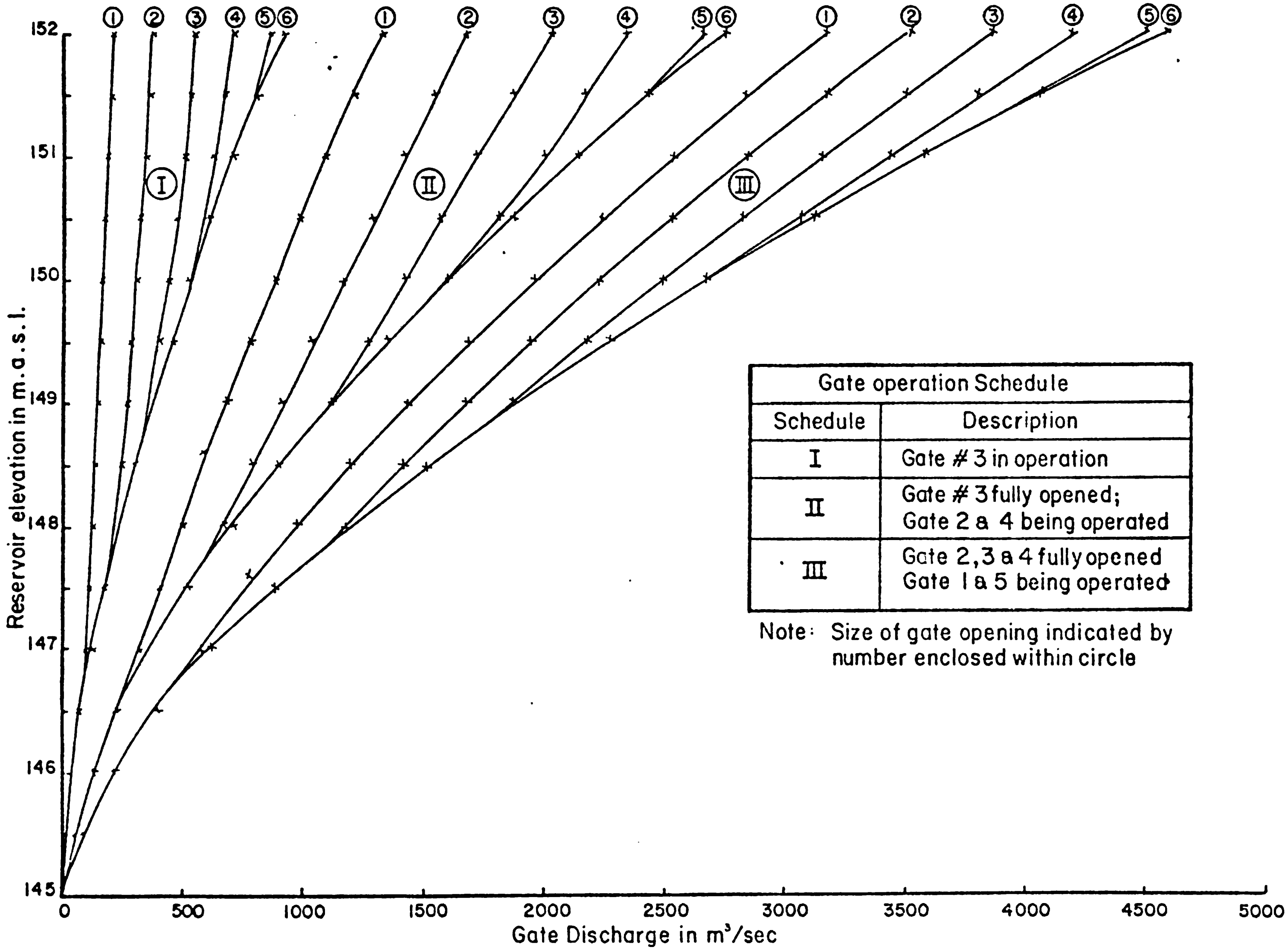
An "emergency" exists for the damtender when communications cannot be established between the damtender and the central office, and the arrival of a potentially damaging flood is imminent. Under these conditions, the damtender shall operate the reservoir according to the instructions given in Section A.3 (for non-hurricane conditions) or Section A.4 (for hurricane conditions) depending on the nature of the flood condition.

A.3 Non-hurricane Flood Emergency Operation

1. For moderate to large floods, follow the Mode 1 operation. That is, when the pool is rising and after it has reached 147 m.a.s.l., maintain the pool level by making releases equal to mean inflow.

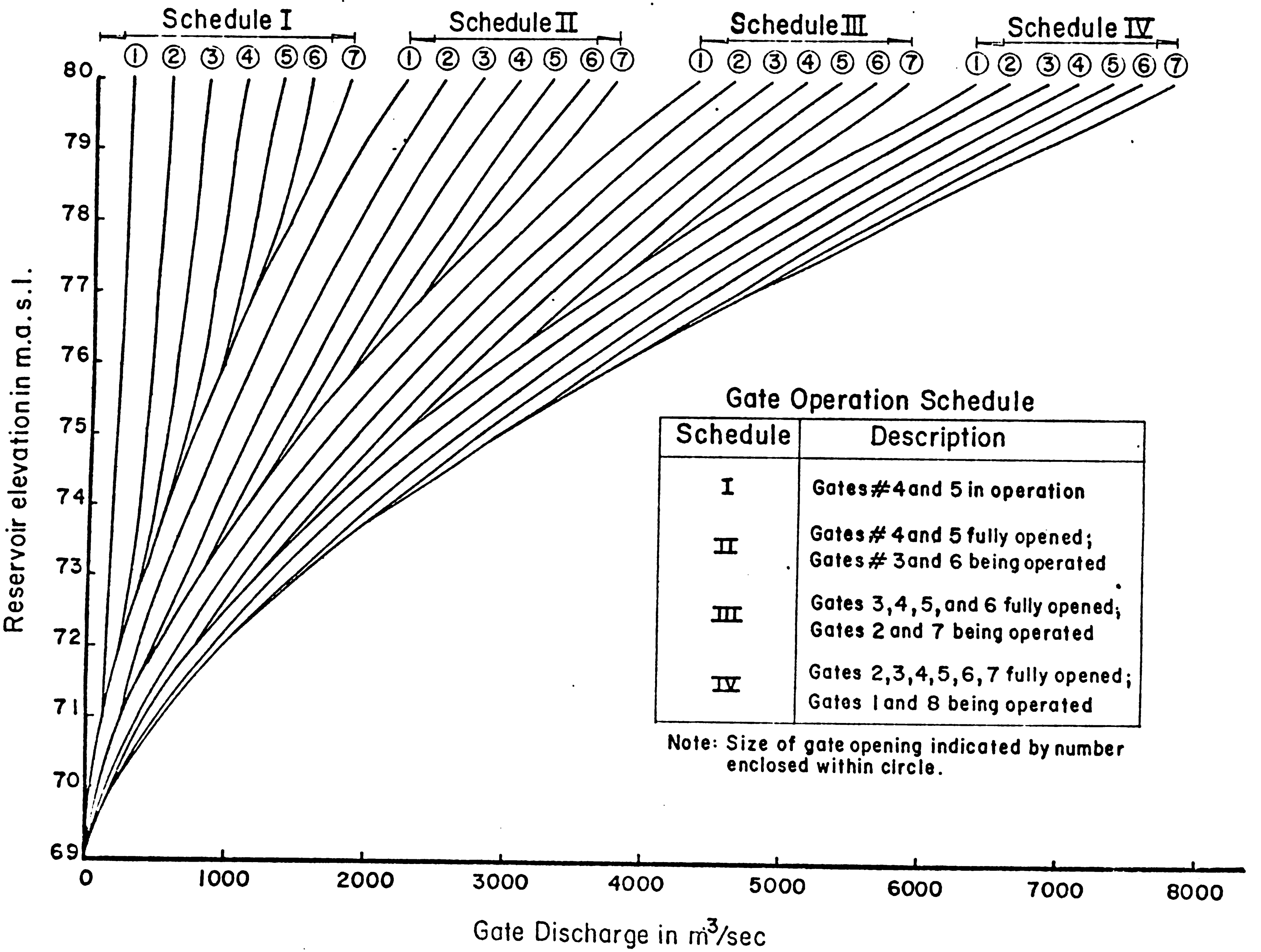
2. Consult Plate 1 for determining gate operation schedule at Valdesia. For Las Barias, consult Plate 2. Schedules must be followed in the order shown.

3. If the downstream flooding conditions indicate that the early holding back of water is desirable, use the induce surcharge method



Gate operation Schedule	
Schedule	Description
I	Gate # 3 in operation
II	Gate # 3 fully opened; Gate 2 a 4 being operated
III	Gate 2,3 a 4 fully opened Gate 1 a 5 being operated

STAGE DISCHARGE CURVE FOR CONTROLLED FLOW THROUGH VALDESIA GATES



Gate Operation Schedule

Schedule	Description
I	Gates # 4 and 5 in operation
II	Gates # 4 and 5 fully opened; Gates # 3 and 6 being operated
III	Gates 3, 4, 5, and 6 fully opened; Gates 2 and 7 being operated
IV	Gates 2, 3, 4, 5, 6, 7 fully opened; Gates 1 and 8 being operated

Note: Size of gate opening indicated by number enclosed within circle.

STAGE DISCHARGE CURVES FOR CONTROLLED FLOW THROUGH LAS BARIAS GATES

(Mode 0) of operation for Valdesia. In this method, if the inflows are known, consult Plate 3 or Plate 4 depending on the estimated value of Ts. If the flood is expected to recede fast, use the smaller Ts = 6 hours.

4. If the inflows are unknown, determine the average rate of rise of reservoir headwater pool in m/hr over the preceding 2-3 hour period.

5. Consult curves provided in Plate 5 or Plate 6 to determine releases. At the intersection of the appropriate rate of rise curve with current reservoir headwater pool elevation, read required amount of reservoir release under this schedule.

6. Consult Plate 1 to determine the gate operation schedule.

7. When the pool level is falling, maintain releases until it reaches 150 m.a.s.l. after which this level is maintained by making releases equal to mean inflow.

8. Continue to make every attempt possible to contact central office personnel.

9. Make every attempt possible to inform the local civil defense personnel regarding the increased releases and potential adverse effects downstream.

A.4 Hurricane Flood Emergency Operation

1. If it is possible to obtain forecast or actual track information in the form of distance to the track, and the precipitation potential from the hurricane, use Plate 7 to compute the forecast precipitation in Nizao. If only the precipitation measurements in the basin or near dam site are available, they may be used as estimates of hurricane precipitation.

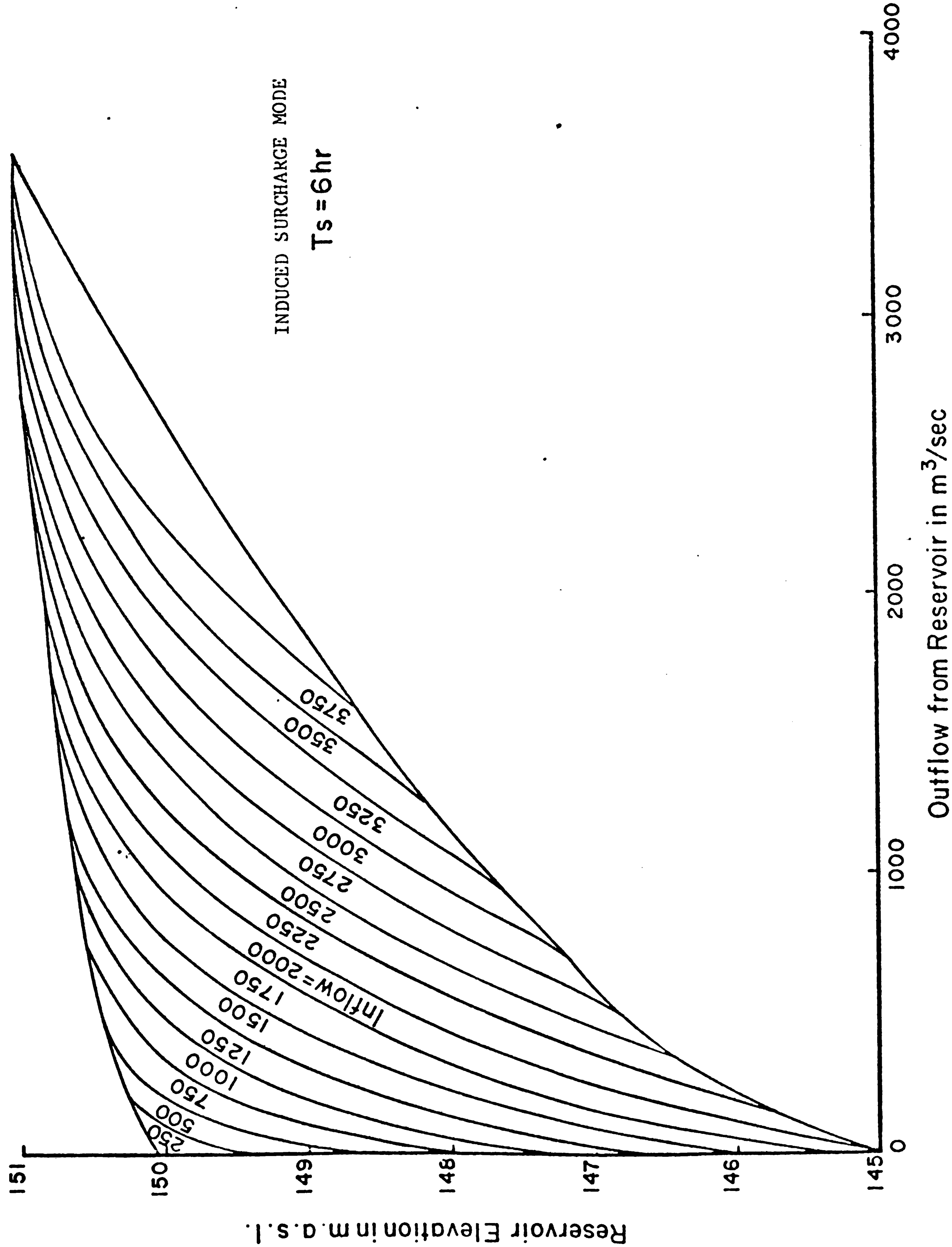

INDUCED SURCHARGE MODE
Ts = 6hr
Reservoir Elevation in m.a.s.l.
151
150
149
148
147
146
145
Outflow from Reservoir in m³/sec
0
1000
2000
3000
4000
250
500
750
1000
1250
1500
1750
Inflow=2000
2250
2500
2750
3000
3250
3500
3750

Plate 3

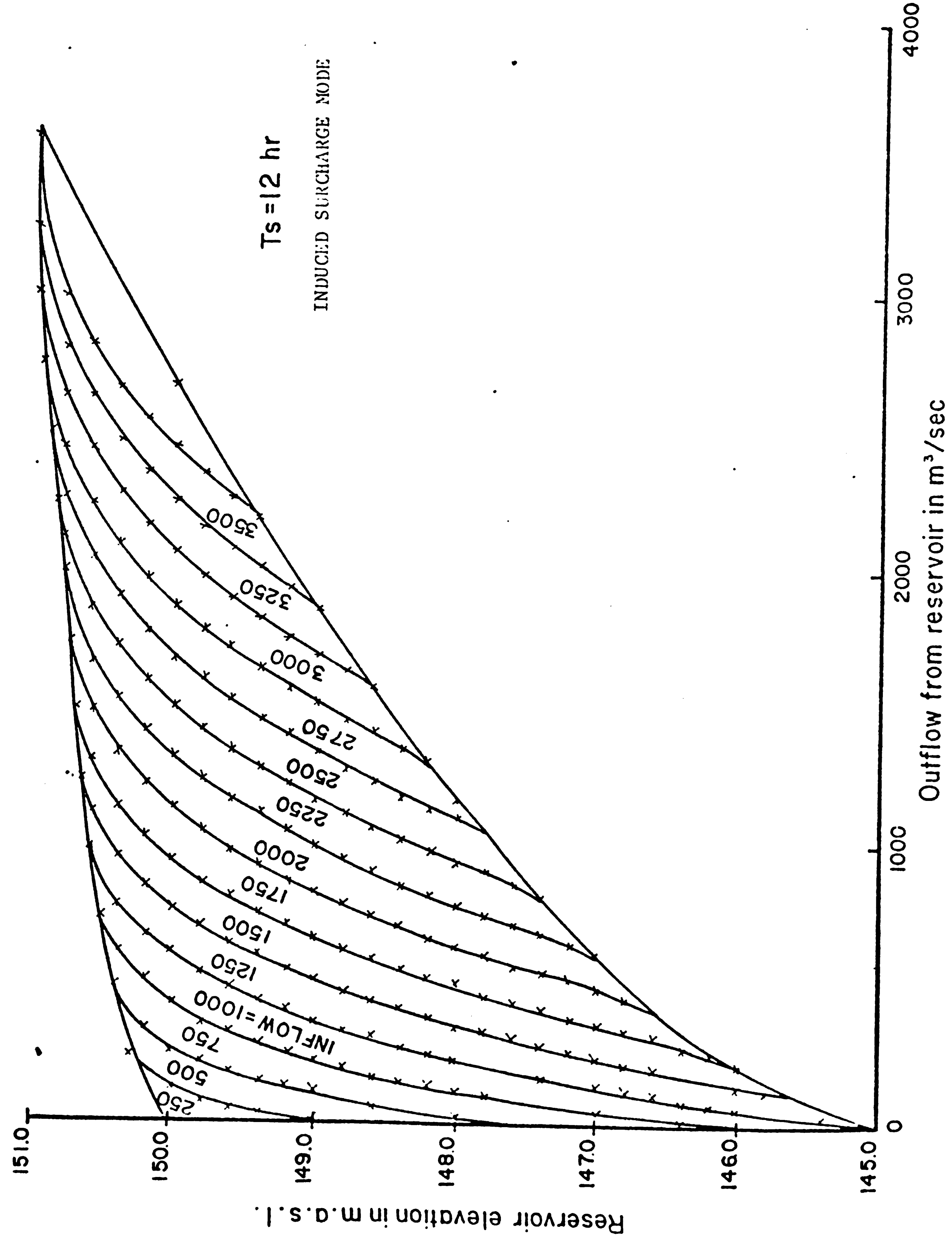

Ts = 12 hr
INDUCED SURCHARGE MODE
Reservoir elevation in m.a.s.l.
Outflow from reservoir in m³/sec
151.0
150.0
149.0
148.0
147.0
146.0
145.0
0
1000
2000
3000
4000
250
500
750
INFLOW=1000
1250
1500
1750
2000
2250
2500
2750
3000
3250
3500

Plate 4

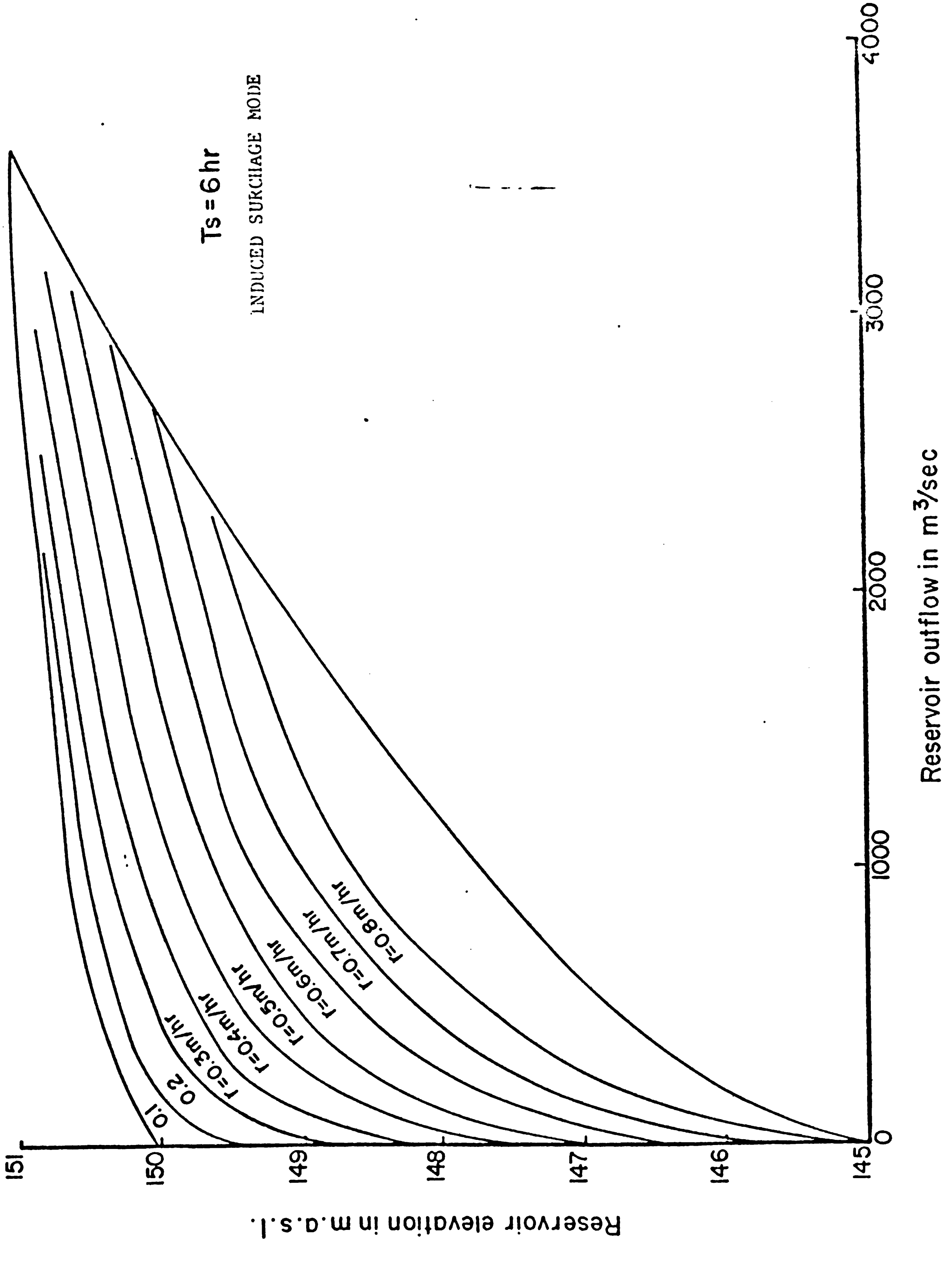
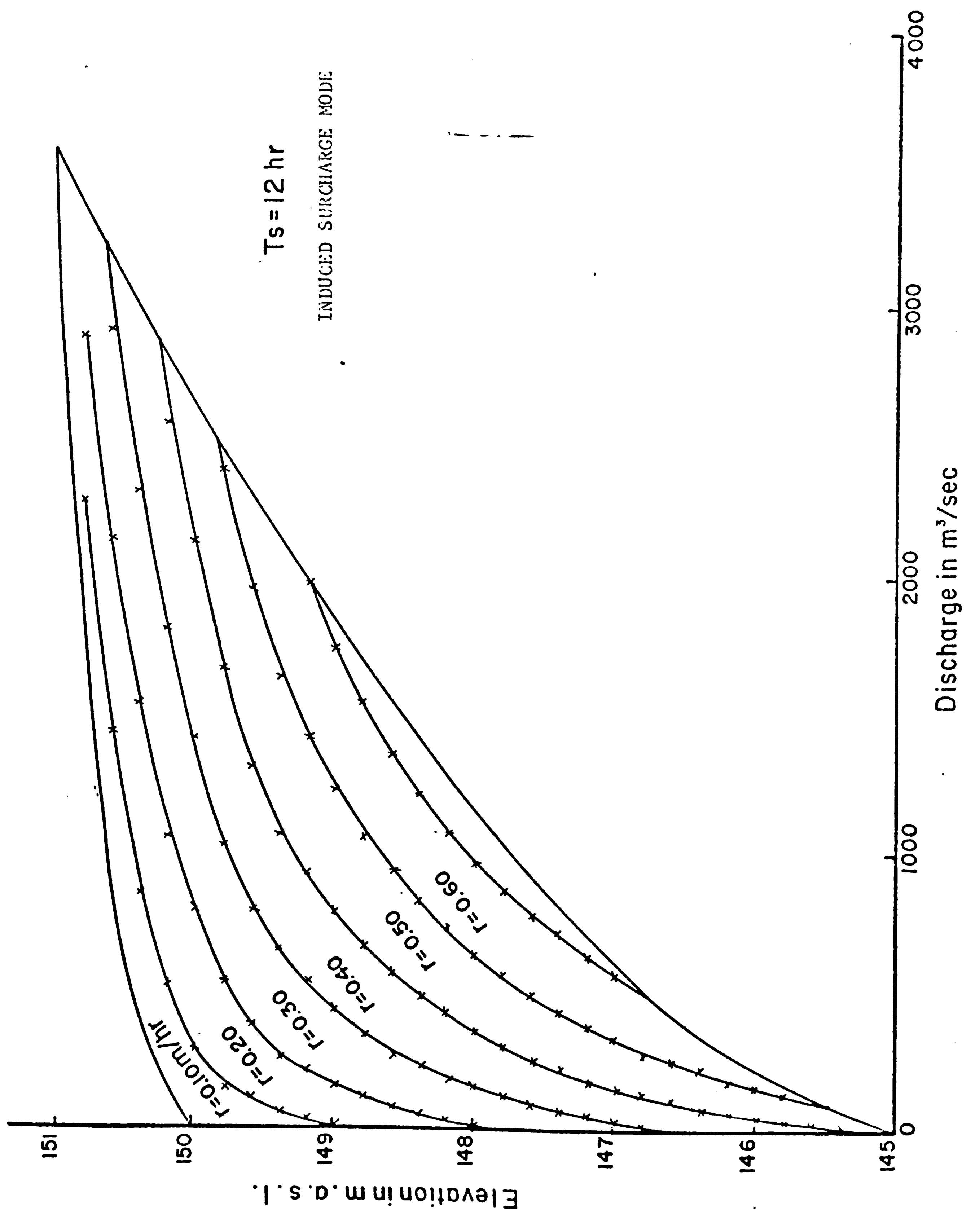
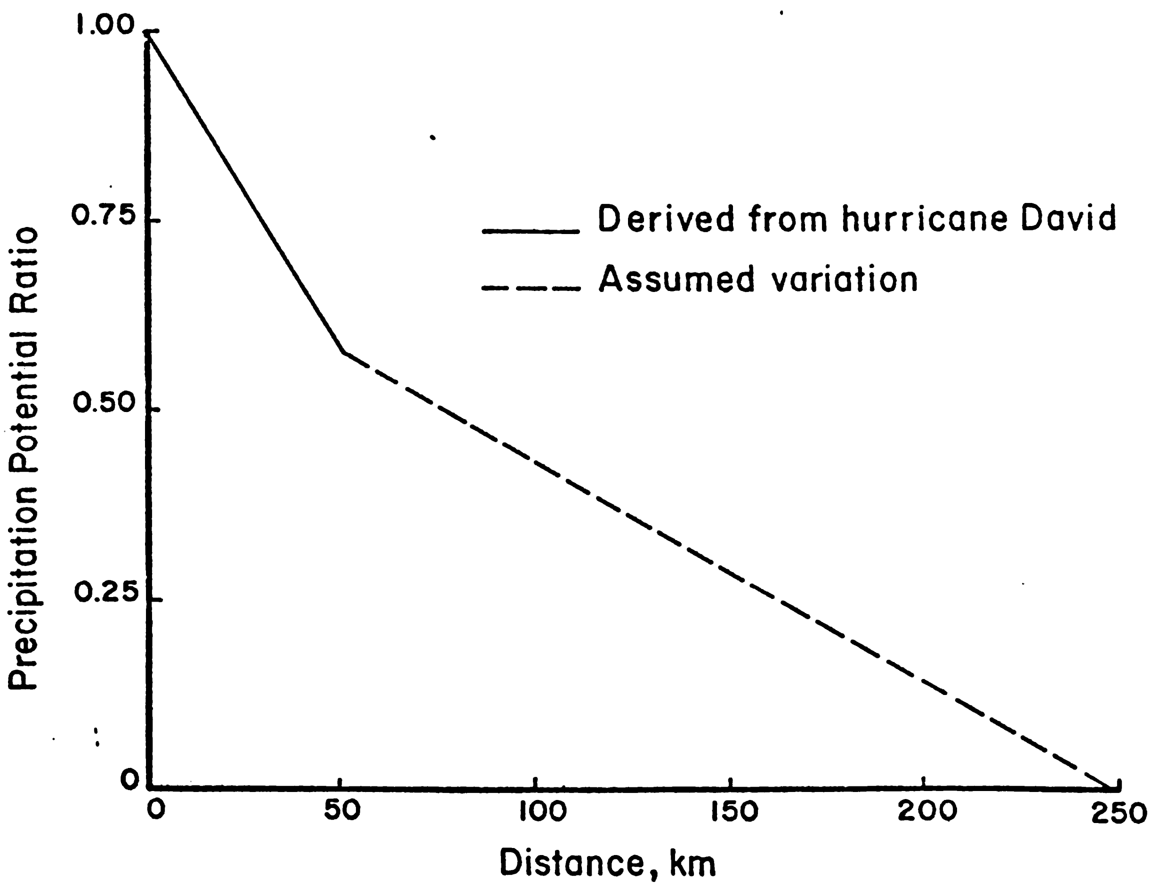

Ts = 6hr
INDUCED SURCHAGE MODE
Reservoir outflow in m³/sec
Reservoir elevation in m.a.s.l.
0
1000
2000
3000
4000
145
146
147
148
149
150
151
0.1
0.2
r=0.3m/hr
r=0.4m/hr
r=0.5m/hr
r=0.6m/hr
r=0.7m/hr
r=0.8m/hr

Plate 5



Ts = 12 hr
INDUCED SURCHARGE MODE
r=0.10m/hr
r=0.20
r=0.30
r=0.40
r=0.50
r=0.60
Discharge in m³/sec
0
1000
2000
3000
4000
Elevation in m.a.s.l.
145
146
147
148
149
150
151

Plate 6



PROPOSED VARIATION OF PRECIPITAION POTENTIAL VERSUS DISTANCE
FOR COMPUTING PRECIPITATION FORECASTS

Plate 7

2. Consult Plate 8 to estimate the peak of the hurricane flood hydrograph.

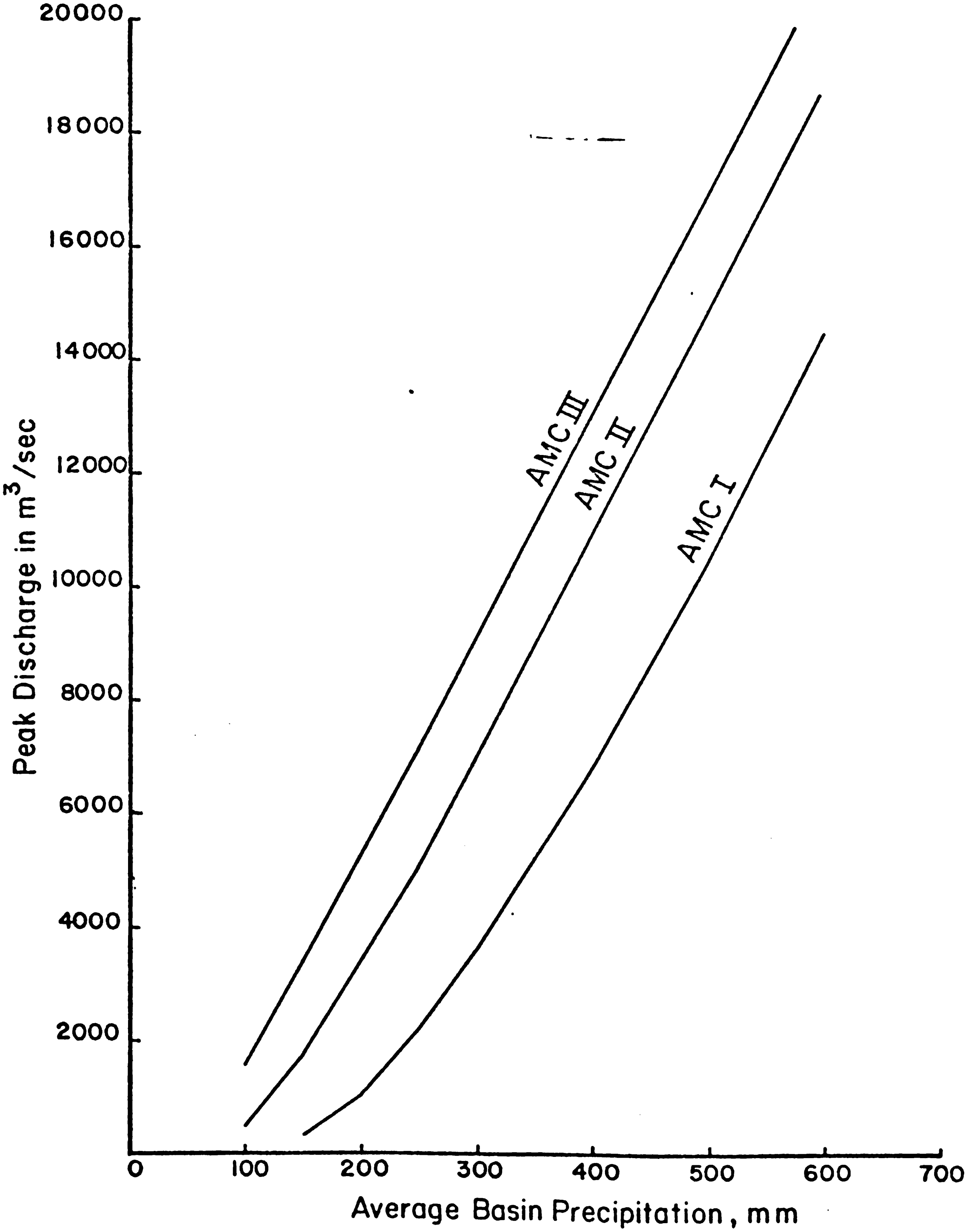
3. Consult Plate 9 to determine the initial level necessary to avoid overtopping the dam.

4. Consult Plate 10 to determine the time required to drawdown the reservoir to the required initial level and compare it with the lead time of flood forecast if available. If the lead time is adequate, carry out necessary operations to drawdown the reservoir.

5. If the lead time is inadequate, use the hurricane operation model (all gates open) to make releases so as to minimize the maximum pool level due to the hurricane flood.

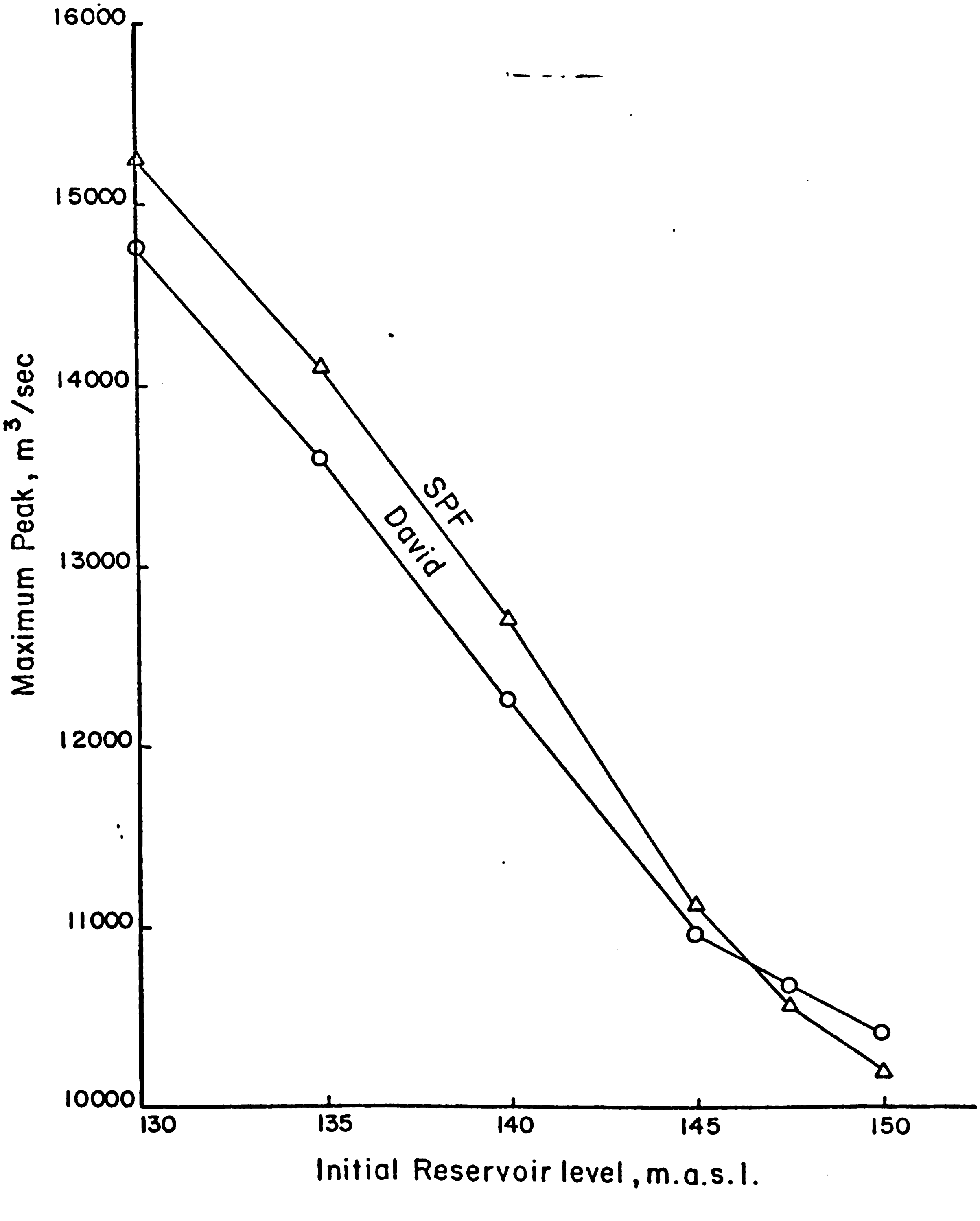
6. If the overtopping is imminent, inform the local civil defense personnel of the potential danger, and depending on the estimated lead time advise evacuation, if necessary.

7. Continue to make every attempt possible to contact central office personnel.

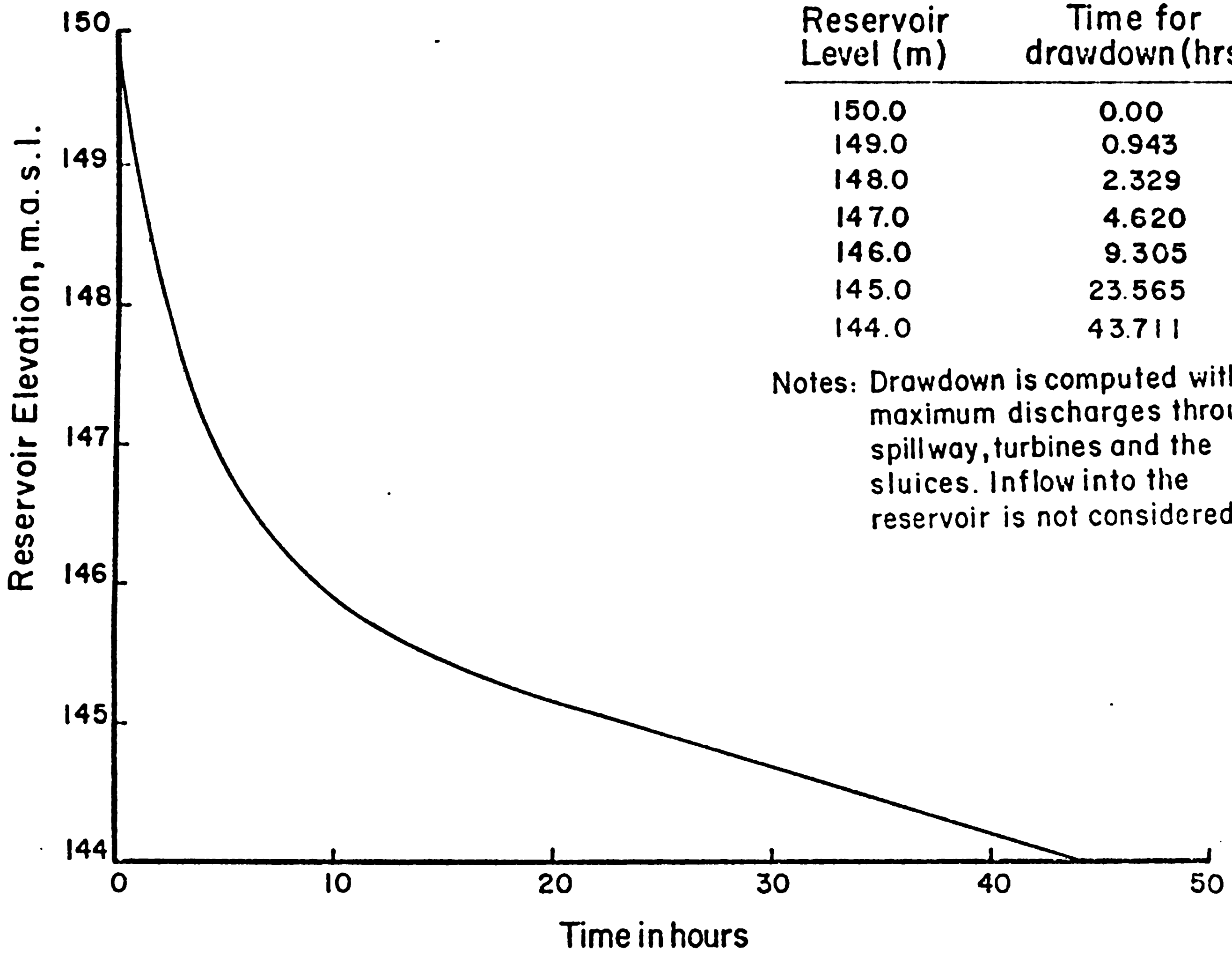


PEAK DISCHARGE ESTIMATES VERSUS AVERAGE BASIN PRECIPITATION FOR

ANTECEDENT CONDITIONS AMC I, AMC II, AND AMC III



REQUIRED INITIAL LEVEL AS A FUNCTION OF THE PEAK DISCHARGE TO TARGET OVERTOPPING POOL LEVEL ABOVE 154 METERS



Reservoir Level (m)	Time for drawdown (hrs)
150.0	0.00
149.0	0.943
148.0	2.329
147.0	4.620
146.0	9.305
145.0	23.565
144.0	43.711

Notes: Drawdown is computed with maximum discharges through spillway, turbines and the sluices. Inflow into the reservoir is not considered.

TIME REQUIRED TO DRAWDOWN THE VALDESIA RESERVOIR FROM 150 METER ELEVATION TO THE INDICATED LEVEL

3.8 HURRICANE FORECASTING

For emergency operation of reservoirs, forecasting of both track and precipitation amount (both temporal and spatial distribution) of hurricanes is important. After intense investigation into the state of art of these two areas, it was found that in the United States, the models and techniques currently in use are not readily transferable. Often, subjective methods which depend on the experience of the forecaster are employed. Development of new models for forecasting of precipitation and track of hurricanes constitute a major undertaking not covered in resources and effort allotted to the project. Every attempt was made to locate simple models which could be readily implemented in the Dominican Republic.

3.8.1 Past Hurricanes

A record of the tracks of all Atlantic tropical cyclones occurred since 1871 is available with the National Oceanic and Atmospheric Administration of the U.S. Department of Commerce. The general orientation of these tracks can be seen in Figure 3.8.1 which depicts the tracks of the 223 named Atlantic tropical cyclones for the period 1954-80. Clearly, the forecasting of the erratic behavior of individual storms is a formidable task. The tracks of 23 tropical cyclones that have crossed the geographical area of the Dominican Republic is shown for the period 1930-1979 in Figure 3.8.2. A list of some major storms occurred during the more recent period of 1955-1980 is shown in Table 3.8.1. The general orientation of these tracks is in the direction of SE-NW; however as seen from Figure 3.8.2, they can take other orientations as well. During the 50 year period 1930-1979, the island

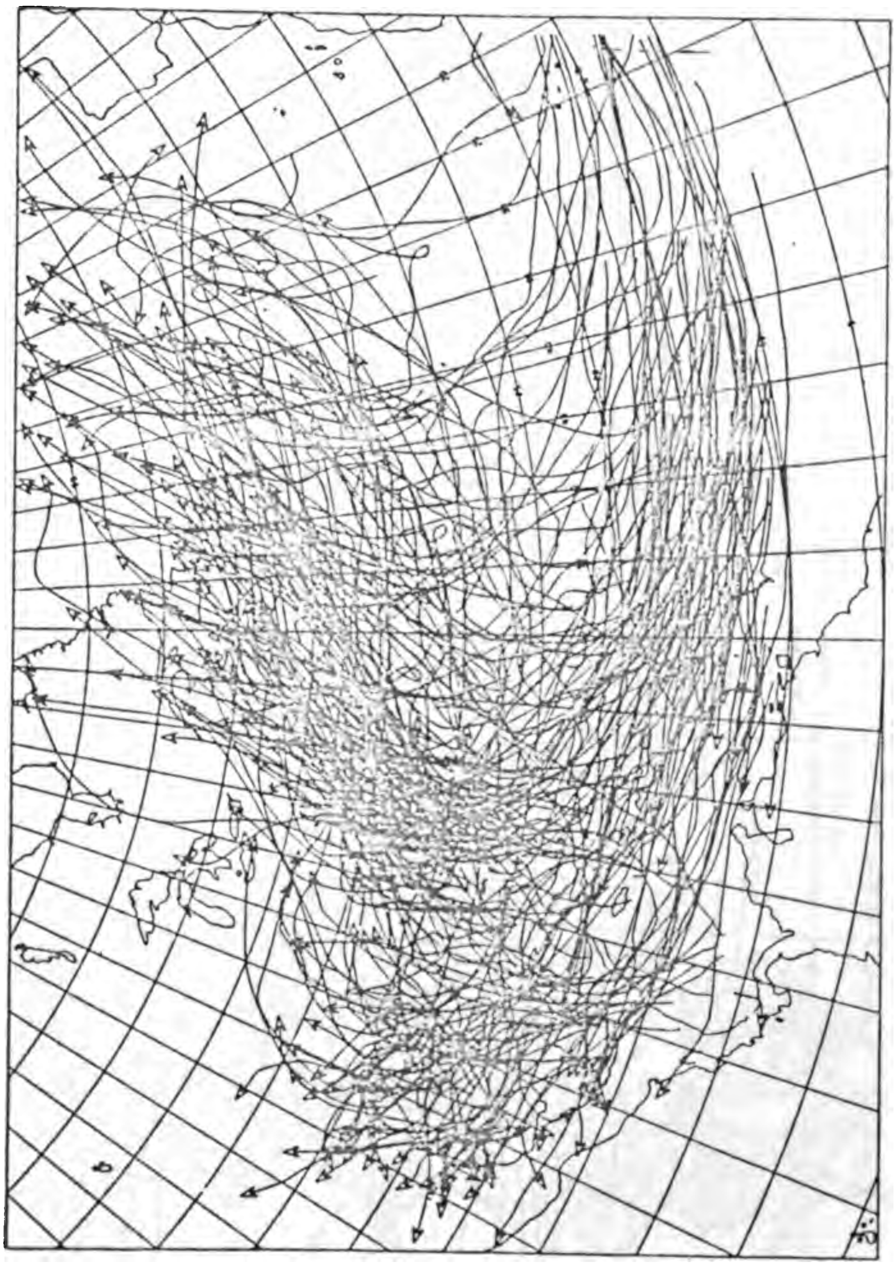


Figure 3.8.1. Tracks of the 223 named Atlantic tropical cyclones (1954-1980). After Neumann (1981).

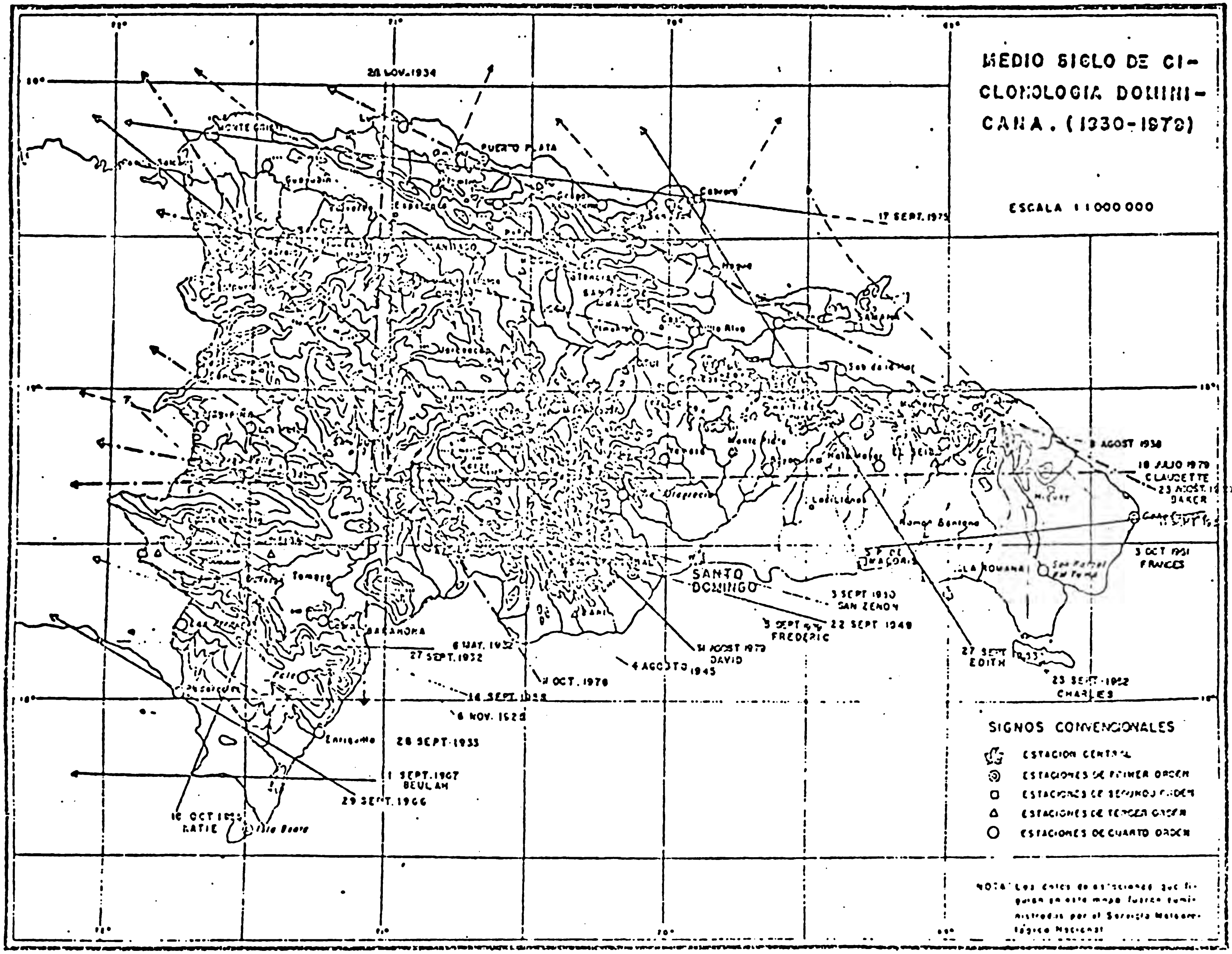


Figure 3.8.2. North Atlantic tropical cyclones which crossed the Dominican Republic during the period 1930-1979.

TABLE 3.8.1 MAJOR TROPICAL CYCLONES WHICH AFFECTED DOMINICAN REPUBLIC DURING THE PERIOD 1955-80

DATE	TYPE AND NAME	MAXIMUM WIND SPEED m/hr
Oct. 17, 1955	Tropical Storm Kattie	35/50
Aug. 31, 1958	Hurricane Ella	74
Sep. 14, 1958	Tropical Storm Gerda	39
Sep. 6, 1960	Hurricane Donna	50
Oct. 3, 1961	Tropical Storm Frances	40/50
Sep. 23, 1963	Tropical Storm Edith	40/50
Oct. 3, 1963	Hurricane Flora	90
Aug. 24, 1964	Hurricane Cleo	74/80
Aug. 27, 1966	Hurricane Faith	40/50
Sep. 29, 1966	Hurricane Inez	134/150
Sep. 10-11, 1967	Hurricane Beulah	35/90
Sep. 16-17, 1975	Hurricane Eloisa	
Aug. 31, 1979	Hurricane David	100/130
Sep. 6, 1979	Tropical Storm Frederick	35/40
Aug. 11, 1980	Hurricane Allen	144

has not experienced any hurricane which has westerly direction (Placido, 1982).

The seasonal occurrence of tropical cyclones is also important for operation of reservoirs. The frequency of occurrence of tropical storms and hurricanes during May 1 through December 31, 1886-1980 in the North Atlantic region is shown in Figure 3.8.3. It is seen the North Atlantic hurricane season is June through November although majority of tropical storms or hurricanes occur during the months of August through October. The peak hurricane season occurs during the first three weeks of September.

3.8.2 Hurricane Track Forecasting

The U.S. National Hurricane Center (NHC) in Miami routinely produces track forecasts for tropical cyclones up to 72 hours lead time. The NHC maintains several models that are activated whenever an Atlantic tropical cyclone is in progress. A summary of these operational models

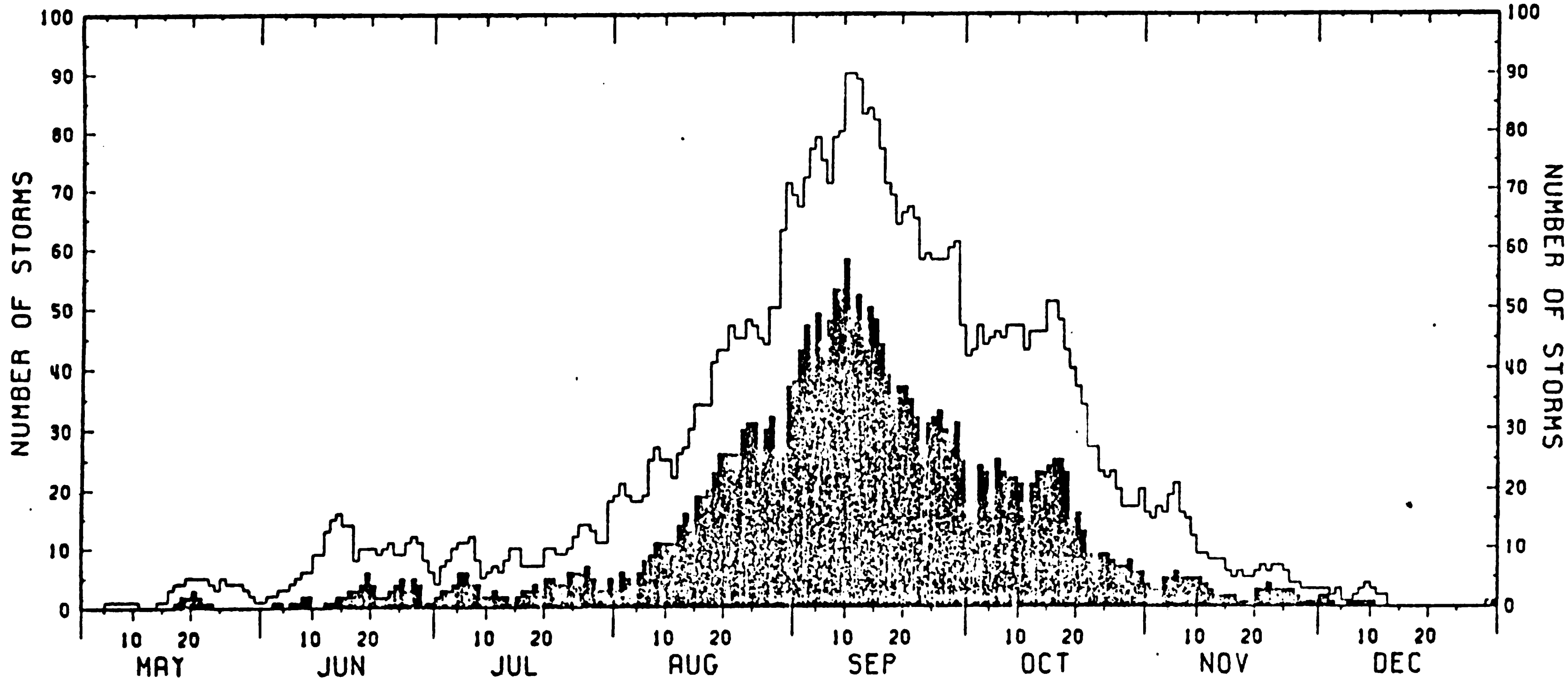


Figure 3.8.3. Number of tropical storms (openbar) and hurricanes (solid bar) observed on each day, May,1 - December 31, 1886 through 1980. After Neumann et al (1981).

appear in Table 3.8.2. The "official" forecasts are released by the NHC after consideration of available guidance from the results of the models as well as forecasters' past experience. The objective of this phase of the project was to locate a suitable track forecast model which can be installed and made operational in the computing facilities at INDRHI. Consequently, the dynamic models such as MFM in Table 3.8.2 which demands a substantial computational effort and also data were not considered in the present search.

Past studies on Atlantic tropical cyclone forecast errors have shown that the errors are principally correlated with the initial latitude of the storm. Consequently, storms are stratified into two groups: a) those initially at, or south of, 24.5^{o} N (referred to as south-zone storms); and b) those north of 24.5^{o} N (referred to as north-zone storms). This subdivision is illustrated in Figure 3.8.4. This particular latitude was chosen by Charles Neumann and J. Pelissier of NHC to assess the error characteristics of the prediction models in use at the NHC. Latitude 24.5^{o}N essentially divides storms that remain in the easterlies, or which are just beginning to recurve, from storms that have already recurved into the westerlies, or which are well into recurvature. It is noted that the Dominican Republic is located in the South zone.

<u>Model Selection:</u> Neumann and Pelissier (1981) reported a comparison of all operational models including official forecasts with respect to the forecast errors of storms in north and south zones. The CLIPER statistical model which is based only on predictors derived from climatology and persistence, was taken as the frame of reference. Their

TABLE 3.8.2 OPERATIONAL MODELS FOR THE PREDICTION OF TROPICAL CYCLONE MOTION OVER THE NORTH ATLANTIC OCEAN

	Model	Type	Description
1.	HURRAN	Statistical	Analog model based on tracks of all Atlantic tropical cyclones since 1886.
2.	CLIPER	Statistical	Regression equation model utilizing predictors derived from climatology and persistence.
3.	NHC67	Statistical-synoptic	Regression equation model utilizing predictors derived from climatology, persistence and observed geopotential height data.
4.	NHC72	Statistical-synoptic	Regression equation model utilizing predictors derived from output of CLIPER model and observed geopotential height data.
5.	NHC73	Statistical-dynamical	Regression equation model utilizing predictors derived from output of CLIPER model, observed and numerically forecast geopotential height data.
6.	SANBAR	Dynamical	Barotropic model based on pressure-weighted wind field averaged through troposphere and represented on a 154 km spaced grid.
7.	MFM	Dynamical	Movable Fine Mesh (MFM) baroclinic model having 10 levels in the vertical and 60 km grid spacing in the horizontal

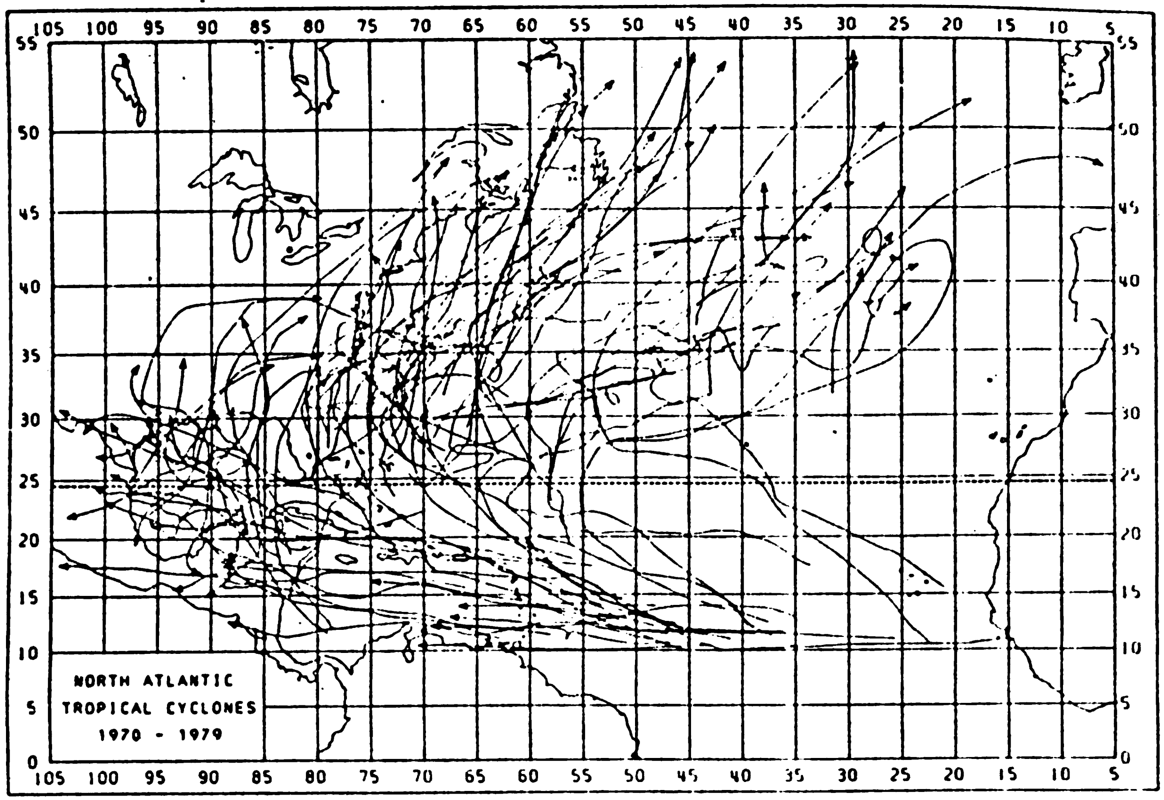


Figure 3.8.4. Tracks of the 82 Atlantic tropical cyclones, 1970-1979. Horizontal dashed line along 24.5° N delimits north-zone from south-zone: storms that were initially at 24.5° N are assigned to south-zone. After Neumann and Pellissier (1981)

results for the south zone may be summarized by the plots presented in Figure 3.8.5. Following observations have been made:

1. The statistical-synoptic models NHC67 and NHC72, and the statistical-dynamical model NHC73 are clearly inferior to climatology and persistence.
2. HURRAN and CLIPER give similar performances over the tropics.
3. Results indicated a little difference, on the average between CLIPER forecasts and official forecasts.
4. Of the models, CLIPER itself probably provides the best overall guidance on southern storms.

Based on the above results for storms in south storms which affect the Dominican Republic CLIPER model was chosen for tropical cyclone track forecasting. It is a simple regression model, based on predictors derived from climatology and persistence, which can be easily implemented on a personal computer.

3.8.3 The CLIPER Model

CLIPER stands for CLImatology and PERsistence. It is a system of regression equations fitted to several predictors available from past observations on the motions of tropical cyclones in the North Atlantic. A total of 286 storms over a 40-year period (1931-1970) have been utilized in its development. Eight basic predictors are used and these are listed in Table 3.8.3. Twelve dependent variables or predictands used are listed in Table 3.8.4. In the development of the CLIPER a total of 164 possible combinations involving the products or cross-products of the original 8 predictors were analyzed. Using the regression analysis, 9 predictors from the original 164 were selected

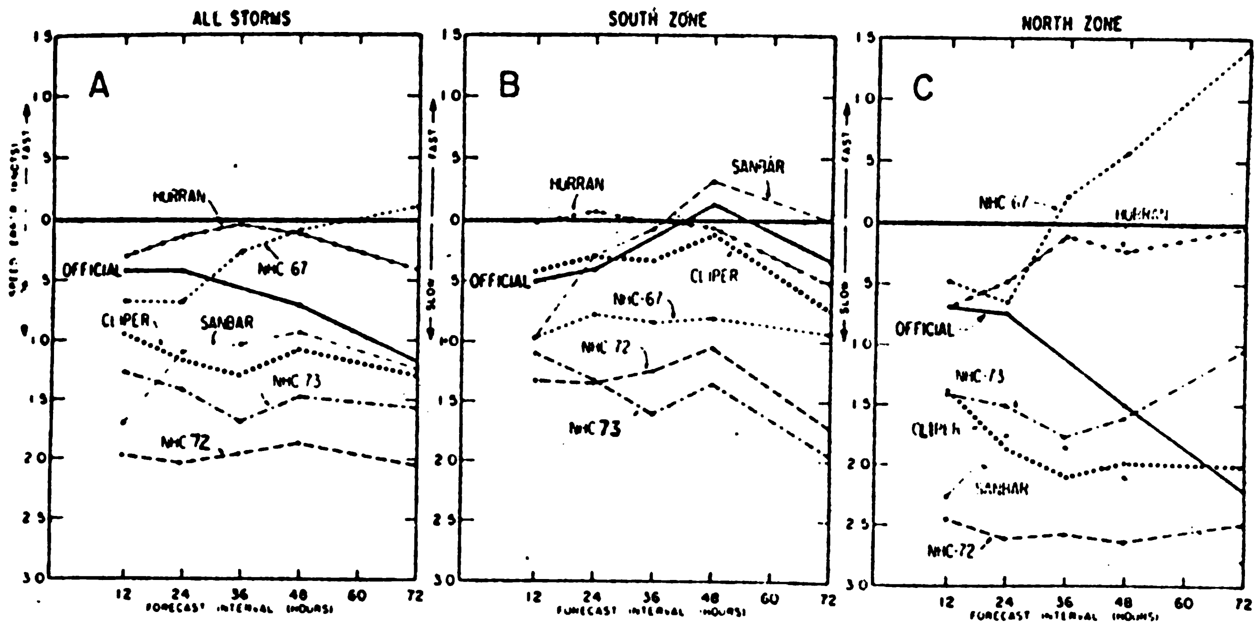


Figure 3.8.5. Performance of specified models relative to performance of CLIPER model on (A) entire sample of storms, (B) south zone storms, and (C) north zone storms. After Neumann and Pellissier (1981).

TABLE 3.8.3 BASIC PREDICTORS USED IN CLIPER MODEL

PREDICTOR	SYMBOL
Initial longitude	X_o
Initial latitude	Y_o
Initial E-W component of speed	U_o
E-W component 12 hours ago	U_{-12}
Initial S-N component of speed	V_o
S-N component 12 hours ago	V_{-12}
Maximum wind	W
Day number	D

TABLE 3.8.4 PREDICTANDS USED IN CLIPER MODEL
(all units in Nautical Miles)

PREDICTAND	SYMBOL
12-hr N/S displacement	DY_{12}
24-hr N/S displacement	DY_{24}
36-hr N/S displacement	DY_{36}
48-hr N/S displacement	DY_{48}
60-hr N/S displacement	DY_{60}
72-hr N/S displacement	DY_{72}
12-hr E/W displacement	DX_{12}
24-hr E/W displacement	DX_{24}
36-hr E/W displacement	DX_{36}
48-hr E/W displacement	DX_{48}
60-hr E/W displacement	DX_{60}
72-hr E/W displacement	DX_{72}

for inclusion in the multiple regression equation. For zonal motion six final prediction equation of the form:

$$DX_{12i} = C_{i,1} \sum_{i=1}^{6} \sum_{j=2}^{8} C_{i,j} P_j \qquad i = 1,2,\ldots 6 \qquad (3.8.1)$$

were proposed. The constants $C_{i,j}$ and the predictors P_j are given in Table 3.8.5. For meridional motion, the six displacement equation are

$$DY_{12i} = C_{i,1} \sum_{i=1}^{6} \sum_{j=2}^{14} C_{i,j} P_j \qquad i = 1,2,\ldots 6 \qquad (3.8.2)$$

where the predictors P_j and constants $C_{i,j}$ are given Table 3.8.6. Both DX_{12i} and D_{12i} are given in units of nautical miles with westward and southward motion considered negative.

As a final comment, it has been found that considerably more variance is explained by CLIPER for zonal motion than for meridional motion. Since synoptic parameters which influence storm motion are not included, the forecaster must be prepared to make subjective corrections to the displacements whenever warranted by anomalous pressure-patterns (Neumann, 1972). In order to obtain accurate forecasts, every effort should be made to define the input parameters as precisely as possible. It must be recognized that, as with any other model, forecasts of CLIPER are not perfect and ideally one should compute probabilistic measures of forecast errors to complement the forecasts themselves. Unfortunately, such a scheme is not available for CLIPER and it was found considerable work is needed to develop one whenever possible. The CLIPER should be

Table 3.8.5. Values of constants C(I,J) for zonal motion. After Neumann (1972)

J	P(J)	C(I,J) I=1	I=2	I=3	I=4	I=5	I=6
1	--- (Intercept)	-3.52591	-13.12388	-28.48156	-44.13759	-55.80913	-60.23074
2	U_0	13.69309	23.30256	32.37355	39.93667	43.27097	45.26022
3	U_{-12}	-2.63735	-3.21553	-5.34286	-6.81978	-7.86100	-8.80390
4	T_0-24	0.81513	3.55451	8.07389	14.10797	21.27143	29.11625
5	V_0	0.68678	3.94936	9.32126	16.35476	24.07252	32.91178
6	$V_0{}^2U_{-12}$	-0.00217	-0.00784	-0.01318	-0.01967	-0.02254	-0.02182
7	$(T_0-24)V_0U_{-12}$	-0.00060	-0.00676	-0.02061	-0.03853	-0.05992	-0.08556
8	X_9-68	0.12473	0.51356	1.04462	1.69802	2.47757	3.29118

Table 3.8.6. Values of constants C(I,J) for meridional motion. After Neumann (1972)

J	P(J)	C(I,J) I=1	I=2	I=3	I=4	I=5	I=6
1	--- (Intercept)	7.60553	30.30846	67.69326	120.27143	186.02612	263.15653
2	V_0	13.59909	22.91538	31.94291	38.94701	44.48386	48.41731
3	V_{-12}	-2.37513	-2.48460	-3.69760	-4.38088	-4.72498	-4.45466
4	$V_0(V_{-12})^2$	-0.00019	0.00497	0.00967	0.01323	0.01074	0.01127
5	$(W-71)V_{-12}$	0.00460	0.00930	0.00954	0.02293	0.03200	0.04297
6	$V_0(W-71)$	0.00226	0.07511	0.06322	0.09532	0.13383	0.15962
7	$V_0{}^2V_{-12}$	-0.00149	-0.00754	-0.01332	-0.01664	-0.01607	-0.01748
8	$(T-24)^2V_0$	-0.00027	-0.00598	-0.01611	-0.03201	-0.04866	-0.06486
9	$(D-248)^2V_{-12}$	-0.00007	-0.00035	-0.00073	-0.00122	-0.00172	-0.00222
10	$V_0(D-248)^2$	0.00004	0.00016	0.00023	0.00032	0.00036	0.00036
11	$(T_0-24)^2(D-248)$	-0.00020	-0.00100	-0.00281	-0.00545	-0.00977	-0.01268
12	$(W-71)(D-248)V_{-12}$	0.00008	0.00048	0.00115	0.00187	0.00271	0.00369
13	U_0	0.14306	0.38795	0.89408	1.66666	2.76818	4.12125
14	$(D-249)^2$	-0.00008	-0.00067	-0.00218	-0.00435	-0.00733	-0.01102

supplemented by "official" forecasts produced by NHC, which also supplies associate strike probabilities.

<u>Computer Program:</u> Using a routine provided by NHC for CLIPER system, an interactive computer program has been written for convenient use of the CLIPER model. The program with graphic display options is running in an IBM-personal computer. A complte description of this software with examples is included in the users manual of the Colorado State University Hydrologic Modeling System (CSU-HMS, 1986) which is the software package developed for hydrologic and emergency operation studies.

3.8.4 Precipitation Forecasting

Ideally, one must have forecasts of both temporal and spatial distribution of rainfall from the storms. These forecasts must be available with sufficient lead time so that the runoff forecast models may be executed and necessary emergency operating procedures activated to avoid catastrophic damages. After many inquiries from practitioners in U.S. locations which are affected by tropical storms it was found that objective methods for forecasting precipitation are rarely used in practice. Often prediction method is based on a subjective approach of experienced forecasters who apply their skills and experience to "guess" the precipitation amounts. The contacts with the National Weather Service of the U.S. Department of Commerce has revealed that the precipitation forecasts are reliably made only after the storms hit shore. The forecasts are made by combining current synoptic data, past precipitation isohyetal maps from similar storms and the experience and skills of forecasters. Although precipitation forecasts are produced in the form of isohyetal maps, their use in a case like Nizao is not very

fruitful in view of the long lead time required for emergency operation. Moreover, transferring such a procedure which has been developed with many years of experience in U.S. to Dominican Republic will require considerable length of time.

Most models developed for forecasting precipitation from severe storms in general are continually evolving. Currently a number of prediction methods are available. These methods can be generally grouped into three categories: 1) use of numerical-meteorological models; 2) use of statistical regression models; and 3) use of "nowcasting" procedure which employs satellite cloud imagery and weather radar data. A complete review of these methods was carried out and a summary has been included as Appendix 3.8.A.

For emergency operation, the use of TP42 which is the hurricane model developed by the U.S. Weather Bureau to forecast precipitation is another possibility. It has been modified and used in the Dominican Republic to compute probable maximum precipitation. Its application to a given area usually requires a substantial amount of work since many types of data including that of local orography is required as input. It is recommended that a future study be undertaken to streamline the application of the hurricane model to various parts of the Dominican Republic so that knowing the forecast track up to 72 hours in advance one can apply the hurricane model to predict potential rainfall around the track. Of course, it must be recognized that these are not real-time forecasts of precipitation but only estimates of rainfall potential for that forecast track. Nevertheless, they should be useful for making decisions as to how the reservoirs should operated for minimum potential damage.

APPENDIX 3.8.A

REVIEW OF PRECIPITATION FORECASTING FOR HURRICANE

3.8.A.1 Introduction

This report presents a review of modeling and forecasting researches and practices of rainfall prediction with emphasis on severe storms and hurricane-related events. There are currently a number of rainfall prediction methods available in the literature. These methods can be generally grouped into three categories: 1) the use of numerical meteorological models which simulates atmospheric dynamics based on dominant aspects of the meteorological and physical behavior of the atmosphere, 2) the use of statistical regression models to correlate rainfall on a station or areal basis with hydrometeorological, climatological and orographic variables; and/or output variables of the large-scale numerical weather models which are computed on large spatial grids, and 3) the employment of "nowcasting" procedures which rely heavily on remote sensing observations such as satellite cloud imagery and weather radar data for mesoscale rainfall prediction. The ensuing sections of this report examine various approaches of rainfall prediction according to the three general categories given above. Specifically, methods that are potentially useful for prediction of rainfall events associated with severe storms or hurricane are given emphasis.

Aside from automated or model-oriented methods above, it must be mentioned that a different category to rainfall prediction methods is the human-experiential approach wherein meteorologists or forecasters apply their own skills and experiences to diagnose weather features from

current hydrometeorological observations, climatology, knowledge of local influences such as orography, and present or past weather patterns. Based on this diagnosis, the forecaster prepares his/her interpretive skills to arrive at the best forecasts feasible. This type of forecasting has been utilized for many years and probably will continue with further enhancements in the future such as combining state-of-the-art models and techniques, and one's interpretive skills (Georgakakos and Hudlow, 1984). This latter rainfall prediction approach will not be dealt with in the review below.

3.8.A.2 Numerical Meteorological Prediction Methods

Currently, there are a number of numerical meteorological models available either in practice or under research and development. These models are physically and dynamically based in the sense that they are designed to describe and simulate the behavior and dynamics of atmospheric processes. All of these models can provide quantitative rainfall predictions but only a few are capable of predicting rainfall quantities associated with severe storms or hurricanes. In terms of spatial coverage, models currently available range from large ones such as the Limited-Area Fine Mesh (LFM) model developed for numerical weather prediction (NWP) by the National Meteorological Center (NMC), which covers areas in the United States with a grid mesh of approximately 150 km; to very small ones consisting of one-dimensional microphysical cloud-physics model such as the precipitation model developed by Georgakakos and Bras (1982). The time resolution of forecasts ranges from 15 minutes to 6 hours time intervals and maximum forecast lead times of 6 hours to 48 hours, for large-scale to

small-scale models, respectively. We now examine some numerical meteorological models currently available.

The Limited-Area Fine Mesh (LFM) model is currently in operational use by the U.S. National Weather Service (NWS). The model is documented in Gerrity (1977) and it is based on a hydrodynamic system of partial differential equations (primitive equation). Accordingly, the primitive equations are usually sufficient to describe the predominant aspects of the meteorological behavior of the atmosphere for phenomena with horizontal scales greater than 1,000 km and time periods of a few days (Gerrity, 1977). A modified version of the model, referred to as LFM-II was done by Newell and Deaven (1981) by incorporating changes in the vertical structure of its computational grids, and improving the convective precipitation and pressure interpolation components of the model. The grid size of the current version LFM-II model is 127 km at 60°N latitude, and the North American continent comprises the land domain of the model.

The initialization and operation of the LFM model require a huge amount of data and interpolation metholodologies are applied to assign observations from various data sensors, taken at irregular spacings in the atmosphere, to the grid points of a regular grid. Output variables of the model include forecasts of precipitation, temperature, pressure and humidity for various levels in the atmosphere and for each grid point. Forecasts can be made available every 12 hours, and forecast lead times extend up to 48 hours with both 6-hours and 12-hours resolutions. The model provides various forecast output including forecasts of precipitation, temperature, pressure and humidity for different levels in the atmosphere and for each grid point. With regard

to forecasting precipitation the LFM model accounts for: 1) large-scale precipitation process, and 2) a subgrid convective precipitation process. The large-scale precipitation is computed internally in the model every six minutes based on whether the value of relative humidity at each grid point is larger than an empirically reduced saturation value of the relative humidity, determined through the solution of the hydrodynamic equations. The convective precipitation is computed every hour which is solely related to the prediction of the existence of a conditionally unstable air mass with vapor content exceeding an empirical threshold value.

A model specifically designed for severe storms or hurricane-related rainfall forecasting is the so-called Movable Finé Mesh (MFM) model (National Weather Service, Meteorological Services Division, 1979). The physics of the model is generally the same as the hydrodynamic equations (primitive equation) used in the LFM model except for some minor charges in these equations. However, one of its unique characteristic as opposed to the LFM model is that its computational grids follow storms as they move during a forecast. Another major difference between MFM and LFM is that the MFM is of finer resolution both in the horizontal and vertical scales. With current operation and model initialization considerations, a grid spacing of 60 km is used, which makes a total areal coverage of about 3,000 km by 3,000 km (much smaller than the LFM model). The time resolution of the MFM model in 6 hours with a maximum lead time of 48 hours.

In practice, the model is only run when an operational requirement exists, mainly for hurricane forecasting or when a flood or precipitation threat is suspected. Due to this and also from

observational and computational constraints, to initialize the model with detailed structure of the actual hurricane vortex, a model storm, derived from an axisymmetrical vortex that is qualitatively similar to the hurricane is used instead (Neumann and Pelissier, 1981). This two-dimensional analog has been empirically formulated so that when it is added to the initial steering current, a balanced, stable initial field is produced and this forms the initial conditions for the numerical integration.

A one-dimensional microphysical cloud-physics model in its experimental stage of development has been presented by Georgakakos and Bras (1982). Based on simplified cloud microphysics, the amount of moisture in terms of liquid water equivalent flowing into and out in a unit area cloud column is computed. Model inputs are the air temperature, dew point temperature and pressure at the ground surface. Precipitation rate at the ground surface is the model output. The model physical parameters are vertically averaged updraft velocity, terminal pressure level and average hydrometeor (water equivalent) particle diameter at the cloud base. Equations of the precipitation model have been expressed in terms of parameters that are storm invariant so that robust parameter estimates can be obtained.

For operational forecast applications the non-linear precipitation model was put into state-space formulation where the equations are linearized based on Taylor's series expansions. The state variable of the model is the liquid water content of the storm clouds at a certain time at the station location. Model verification was done by employing the precipitation model to the state-space stochastic soil moisture accounting model of the NWSRFS (Kitanidis and Bras, 1980) with a

statistically linearized non-linear channel model. In this case, forecasts of both precipitation and basin discharge can be obtained to assess the model performance.

A much more involved microphysical cloud-physics model has been developed by Nickerson and Richard (1981) in three dimension. The model is particularly useful for terrain-induced mesoscale systems such as orographic precipitation processes since it explicitly accounts for the terrain boundary layer. The areal domain of the model is 230x230 square km and it has 15 vertical computational levels with horizontal grid of about 10 km. The computational time step is 15 seconds. Experiments with the model so far show close agreement between observed and forecast values. As with other models of this type, however, the disadvantages are the long computational time and the large amount of good quality data required for initialization. Because of current inadequacy of real-time atmospheric observations, models of this type are not at the present operationally useful for real-time forecasting applications. But one can possibly run the model for several typical synoptic situations at particular drainage basins and archived the results for future guidance in forecasting.

3.8.A.3 Statistical-Based Prediction Models

The use of statistical modeling in precipitation prediction has been motivated primarily by the lack of data required in physically-based models and the inadequacy of large-scale models to account fore meso-scale or localized precipitation. The approach involves development of multiple regression or time series models where the precipitation forecast (predictand) is a linear function of causal variable (predictor) such as indicators of the potential moist

instability of the atmosphere computed from most recent observations, the output of large-scale dynamical models, or locally observed past/present weather variables. An important aspect of the statistical-based methodology is their capability to supply probabilistic forecasts. These forecasts are particularly useful when consideration is made to maximizing expected gains or minimizing expected loss. The use of these models is not straight forward however, due to difficulty in the identification of all the relevant meteorological variables that will be used as explanatory variables and usually the absence of high linear correlation in the station precipitation records. Additionally, no guarantee is provided regarding the invariance of the model parameters for different rainfall events, due to the absence of explicit physical process-mechanism in the model (Georgakakos and Bras, 1984).

The failure of large-scale atmospheric models to adequately consider the meso-scale aspects of precipitation led to the development of Model Output Statistics (MOS). The MOS method presented in Glahn and Lowry (1972) and more recently Lowry and Glahn (1976) provides forecasts of variables that are not computed explicitly in large-scale models such as localized rainfall rates or subgrid scales. In addition, the MOS partially accounts for biases in the numerical models since numerically forecast values are used rather than observations as the independent variables when the equations are developed.

For precipitation amount forecasting, the procedure used in MOS is to divide accumulated precipitation for a forecast interval into categories, and then to derive forecast equations for each category using multiple linear regression. The equations are derived

simultaneously, so that the equations for each category use the same predictions but with different regression coefficients. There are several output variables from large scale models with which the MOS regression model uses as predictor variables. These include forecast of accumulated precipitation amount during a preceeding time interval and large-scale model grid domain, relative humidity from ground surface to 490 mb, vertical velocities at different pressure levels, wind speed shear and wind directional shear at different pressure levels, 1,000 to 500 mb thickness, 850 mb upslope wind and orographic lifting, and mean monthly precipitation at pertinent ground stations. In practice, only some of the above variables are used as predictors on the basis of statistical significance, physical interpretation and predictive value. An objective approach in screening the desirable variables is by stepwise regression. Another approach is using a one-parameter multiplicative model which provides a means of evaluating the relative importance of additive and multiplicative influences of predictor variable on a response variable (Aritt and Frank, 1986). This latter approach, however, is only used as a guide rather than a determining factor,. so that predictor variables to be included in the multiple regression model would be finally selected along with other criteria such as physical significance.

Currently, the forecast domain of MOS is comprised of the conterminous United States. Time resolution of forecasts is either 6 or 12 hours with maximum forecast lead times of 36 hour and 48 hours, respectively. The large-scale model base of MOS is usually the Limited-Fine Mesh (LFM) numerical model of the US-NWS. Testing and evaluation of the performance of MOS have been reported by several

investigators. Bermowitz and Zurndorfer (1979) suggested that the inclusion of LFM precipitation forecast amount would probably be a linear variable, while Aritt and Frank (1985) suggested instead a multiplicative relationship. Zurndorfer (1980) demonstrated that substantially lower precipitation amount forecasting skill is experienced in warm season than in the cool season due to greater frequency of mesoscale and convective scale events in summer; since such events are not well resolved by the LFM's grid mesh as the synoptic scale systems which tend to prevail in winter. This is further substantiated by Aritt and Frank (1985) by saying that the reason for this could be that the synoptic phenomena which tend to dominate the production of precipitation in the cool season are more readily integrated into the LFM precipitation parameterization while the meso-scale and convective scale processes important in the warm season are not well simulated.

An approach based on time series models has been presented by Johnson and Bras (1980) in the development of a method for very short-term forecasting of precipitation with forecast lead times ranging from five minutes to one hour. The model is based on the assumption that the precipitation rate at each point in a precipitation field in the sum of a time-and-space dependent mean and a lag-1 Markovian residual with zero mean. The spatial and temporal variation of the precipitation-field mean and standard deviation is represented by an assumed time-and-space dependent covariance function. The model is presented in state-space form and tested with a Kalman filter in real time applications using telemetered raingage data.

Results of the above model indicate skills in capturing the fine structure of the observed rainfall for forecast lead times ranging from five to ten minutes. On the other hand, the model performed poorly for one-hour forecast lead times. Johnson and Bras concluded that for a raingage network of reasonable size, the model is computationally feasible and it explains part of the variance in the observed rainfall records. For real-time applications, a rather dense raingage network is required for good model performance since the model calibration and operation depends solely on raingage data.

3.8.A.4 Nowcasting Procedures

Nowcasting is an observation-intensive approach to weather forecasting. This approach relies heavily on the timely use of current data in which remote sensing observations such as weather radar data and satellite cloud imagery play a dominant role. The use of weather radar data currently comprise the major observation procedure in nowcasting of precipitation because of the ability of individual radars to detect and track severe weather and precipitation conditions that have life cycles of less than six hours. Satellite cloud imagery stations provide the bridge from larger-scale systems associated with forecast periods of several hours to meso-scale systems associated with forecast periods of several hours to meso-scale or convective storm systems associated with warning times of a few minutes. Experiences so far show that effective precipitation nowcasting are obtained from the combination of radar and satellite data. Current nowcasting systems are capable only for forecast lead times of up to one or two hours. Because of the very short forecast lead times, the resulting forecasts have to be disseminated quickly to the users. Thus, depending on the magnitude and

type of event, a nowcasting system requires a developed communication system both in volume and speed of transmission. Several countries, including the United States have been increasingly involved in the development of nowcasting systems. A number of operational systems in the U.S., Japan, Canada and other countries are described in Browning (1982).

In a series of papers, Scofield and Oliver (1977), Scofield (1984), Spayd and Scofield (1984a, 1984b), describe procedures for estimating precipitation generated from convective systems, tropical storms and tropical cyclones (hurricanes) using satellite imagery. Generally, the procedures involve identifying cloud forms and analyzing ongoing changes in cloud patterns taken from geostationary satellite pictures. Then based on some criteria on the process physics and subjective information (expert opinion), the potential amount and areal extent of precipitation producing cloud systems are estimated. Basically, the working hypothesis in these procedures is that, the higher the top of the cloud (cumulonimbus), the heavier the rain. This qualitative hypothesis is put into a quantitative relationship in the form of a decision tree or flow diagram. The decision tree basically involves three steps: i) identify rainfall producing cloud systems, ii) estimate rainfall rates based on cloud features, and, iii) compare rainfall estimates to ground observations. Improvements of the above procedures have been presented by Scofield and Spayd (1983) using a combination of satellite, radar and conventional data as well as the work of Waters, et al (1985) by developing an automation of the technique.

3.9 REFERENCES

Aritt, R.W. and W.M. Frank, 1985, Experiments in Probability of Precipitation Amount Forecasting using Model Output Statistics. Monthly Weather Review, Vol. 113, November, pp. 1837-1851.

Bermowitz, R.J. and E.A. Zurndorfer 1979, Automated Guidance for Predicting Quantitative Precipitation, Monthly Weather Review, Vol. 107, pp. 122-128.

Browning, K.A., (ed), 1982, Nowcasting, Academic Press, New York.

CSU-HMS, 1986, Colorado State University Hydrologic Modeling System - User's Manual, developed by J.T.B. Obeysekera, G.Q. Tabios III, J.D. Salas, and H.W. Shen, Dept. of Civil Engineering, Colorado State University, Fort Collins, Colorado 80523.

Gerrity, J.F., 1977, The LFM Model - 1976: A Documentation, NOAA Tech. Memo. NWS NMC GO, Washington, D.C., December.

Georgakakos, K.P. and R.L. Bras, 1982, A Precipitation Model and Its Use in Real-Time River Flow Forecasting, MIT Report No. 286, Dept. of Civil Engrg., MIT, Cambridge, Mass., July.

Georgakakos, K.P. and M.D. Hudlow, 1984, Quantitative Precipitation Forecast Techniques for Use in Hydrologic Forecasting, Bull. Amer. Meteor. Soc., Vol. 65, No. 11, November, pp. 1186-1200.

Glahn, H.R. and D.A. Lowry, 1972, The Use of Model Output Statistics (MOS) in Objective Weather Forecasting, J. of Appl. Meteor., Vol. 11, pp. 1203-1211.

Johnson, E.R. and R.L. Bras, 1980, Multivariate Short-Term Rainfall Prediction, Water Resour. Res., Vol. 16, No. 1, pp. 173-185.

Kitanidis, P.K. and R.L. Bras, 1980, Real-Time Forecasting with a Conceptual Hydrologic Model: 2 Applications and Results, Water Resour. Res., Vol. 16, No. 6, pp. 1034-1044.

Lowry, D.A. and H.R. Glahn, 1976, An Operational Model for Forecasting Probability of Precipitation-PEATMOS PoP, Monthly Weather Review, Vol. 104, pp. 221-232.

National Weather Service, Meteorological Services Division, 1979, The Movable Fine Mesh (MFM) - an Operational Forecast Model, Tech. Proc. Bull. Series No. 259, Meteor. Services Div., NWS, NOAA, Silver Spring, Md.

Neumann, C. J., 1972, An alternate to the HURRAN tropical cyclone forecast system. NOAA Tech. Memo NWS SR-62, 24 pp.

Neumann, c. J., 1981, Trends in Forecasting the Tracks of Atlantic Tropical Cyclones, Bulletin of the Meteorological Society, Vol. 62, No. 10.

Neumann, C.J. and J.M. Pellissier, 1981, Models for Prediction of Tropical Cyclone Motion over the North Atlantic: An Operational Evaluation, Monthly Weather Review, Vol. 109, pp. 522-538.

Neumann, C. J., G. W. Cry, E. L. Caso, B. R. Jarvinen, 1981, Tropical Cyclones of the North Atlantic Ocean, 1871-1980, National Hurricane Center, Miami.

Newell, J.E. and D.G. Deaven, 1981, The LFM-II Model--1980, NOAA Tech. Memo, NWS NMC 66, Washington D.C., September.

Nickerson, E.C. and E. Richard, 1981, On the Distribution and Evolution of Clouds and Rain over the Vosges and Black Forest Mountains: A Three-Dimensional Mesoscale Simulation with Parameterized Microphysics, Fifth Conference on Numerical Weather Prediction, pp. 223-227.

Scofield, R.A. and V.J. Oliver, 1977, A Scheme for Estimating Convective Rainfall from Satellite Imagery, NOAA Tech. Memo., NESS 86, Washington, D.C., April.

Scofield, R.A. and L.E. Spayd, 1983, A Technique that uses Satellite, Radar and Conventional Data for Analyzing Precipitation from Extratropical Cyclones, Fifth Conference on Hydrometeorology, Tulsa, Oklahoma, Oct. 17-19, Amer. Meteor. Soc., pp. 259-268.

Scofield, R.A., 1984, The NESDIS Operational Convective Precipitation Estimation Technique, 10th Conference on Weather Forecasting and Analysis, Clearwater Beach, Fla., June 25-29, Amer. Meteor. Soc., pp. 171-180.

Spayd, L.E. and R.A. Scofield, 1984a, An Experimental Satellite-Derived Heavy Convective Rainfall Short-Range Forecasting Technique, 10th Conference on Weather Forecasting and Analysis, Clearwater Beach, Fla., June 25-29, Amer. Meteor. Soc., pp. 400-408.

Spayd, L.E. and R.A. Scofield, 1984b, A Typical Cyclone Precipitation Estimation Technique using Geostationary Satellite Data, NOAA, Tech. Memo., NESDIS 5, Washington, D.C., July.

U.S. Army Corps of Engineers, 1956, Hydraulic Design Chart 311-1 to 311-5 Hydraulic Design Criteria, Volume 2. Waterways, Experiment Station.

U.S. Army Corps of Engineers, 1959, Reservoir Regulation, Engineering and Design, EM 1110-2-3600, U.S. Govt. Printing Office, Washington, D.C.

U.S. Army Corps of Engineers, 1981, HEC-1 Flood Hydrograph Users Manual, Hydrologic Engineering Center, Davis, California.

U.S. Army Corps of Engineers, 1982, HEC-2 Water Surface Profiles, Users Manual, Hydrologic Engineering Center, Davis, California.

U.S. Bureau of Reclamation, 1976, Design of Small Dams, Denver Federal Center, Denver, Colorado 80225.

Waters, M.P., C.G. Griffith and W.L. Woodley, 1985, Use of Digital Geostationary Satellite Imagery for Real-Time Estimation of Hurricane Rain Potential in Land Filling(?) Storms, NESS, NOAA, preprint.

Zurndorfer, E.A., 1980, The Use of Model Output Statistics for Predicting the Probability of Precipitation Amount and Precipitation Amount Categories, Tech. Proc. Bull., Series 283, National Weather Service, NOAA, Silver Spring, M.D.

FECHA DE DEVOLUCION

FECHA DE DEVOLUCION

Autor

Final Report Volume III

Título

flood operation Studies

Fecha Devolución	Nombre del solicitante