The problem with manual spillways lies in the fact that no unique elevation - discharge curve exists. An elevation - discharge curve for the non-manual spillway components and an elevation - storage curve for the reservoir can however still be drawn up.

Flow from the reservoir Q1, at the start of a time step, which includes flow through the manual components, is known and

$$BAL = (S1 + \frac{\Delta tQ1}{2}) - Q1\ \Delta t + \frac{\Delta t}{2}(I1 + I2) \text{ can be evaluated.} \tag{11}$$

Using the water level in the reservoir at the start of the time step as an estimate of the water level at the end of the time step an estimate of flow through the manual components at the end of the time step can be made, say QE. The balance required then becomes

$$BAL = (S2 + \frac{\Delta tQ2}{2}) + \frac{\Delta t}{2} QE \tag{12}$$

to yield $S2 + \frac{\Delta tQ2}{2}$ and ultimately Q2 and the reservoir water level.

If the reservoir water level calculated correlates with the initial estimate, QE becomes the flow through the manual components and Q2 the flow through the non-manual components at the end of the time step.

If the calculated water level does not agree with the previous estimate the calculated water level becomes the new estimate, QE is recalculated and finally $S2 + \frac{\Delta tQ2}{2}$ to yield a new reservoir water level and Q2.

This process is repeated until the final water level is in agreement with the previous estimate. This water level is then accepted as the water level at the end of the time step, which is also the water level at the beginning of the new time step, and the next time step can be calculated.

2.3 DESCRIPTION OF ITERATIVE METHOD OF FLOOD ROUTING

When the selected spillway configuration contains only manual components the storage indication method becomes unworkable. Under these conditions an iterative search technique is more appropriate. The continuity of flow through a river or reservoir reach applies i.e. what enters the reach must either emerge or temporarily move into storage

$$I = Q + \frac{ds}{dt} \tag{8}$$

Where I = inflow to the reach

Q = discharge from the reach

$\frac{ds}{dt}$ = rate of change in reach storage with respect to time

Equation (8) is approximated, for a time interval Δt, by:

$$1/2\ (I1 + I2)\ \Delta t + S1 = 1/2(Q1 + Q2)\ \Delta t + S2 \tag{13}$$

The left hand side of equation (13) is known and presents the volume of storage in the reach at the start of the time step plus the total volume of inflow during the time step i.e. The total volume of water available to the reach during the time step. The right hand side presents the volume of storage in the reach at the end of the time step plus the total volume of outflow during the time step i.e. The total volume of water accounted for by the reach during the time step. However, S2 and Q2 are unknown and due to the spillway configuration no unique relationship between S2 and Q2 exist. In practice this equation is solved by iteration as follows.

At the beginning of any time step the reservoir level, outflow, storage and inflow is known. Inflow at the end of the time step is also known. From the inflow values the volume of inflow into the reservoir during the time step can be calculated. This volume is added to the storage volume at the beginning of the time step to yield a total volume of water available to the reservoir during the time step.

i.e. $$\text{INFLOW} = \frac{\Delta t}{2}(I1 + I2) + S1 \qquad (14)$$

where INFLOW = volume of water available to reservoir

I1 & I2 = inflows at beginning and end of time step

S1 = storage at beginning of time step

The reservoir level at the beginning of the time step is now compared with the level at the beginning of the previous time step. Depending on whether the level is rising or falling an increment is added to, or subtracted from the reservoir level at the beginning of the time step to estimate the level at the end of the time step. From this estimated level a new storage and outflow at the end of the time step are calculated.

From the outflow values at the beginning and end of the time step, the volume of outflow from the reservoir during the time step can be calculated. This volume is added to the storage estimate calculated at the end of the time step to yield a total volume of water accounted for by the reservoir during the time step.

i.e. $$\text{OUTFLOW} = \frac{\Delta t}{2}(Q1 + Q2) + S2 \qquad (15)$$

where OUTFLOW = volume of water required by reservoir

Q1 & Q2 = outflows at beginning and end of time step

S2 = storage at end of time step

The volume of water available is now compared with the volume of water accounted for. i.e. INFLOW < = > OUTFLOW. If they are the same the estimated water level at the end of the time step is accepted as the level in the reservoir at the end of the time step. This level also corresponds to the water level at the beginning of the next time step and the next time step can be calculated. If they are not the same a decreased increment is added to or subtracted from the estimated water level as required. The iteration is repeated until a balance is obtained. When a balance is obtained the estimate for water level at the end of the time step is accepted as the water level at the end of the time step, and thus also as the water level at the beginning of the next time step and the next time step can be calculated.

2.4 DESCRIPTION OF THE MANUAL GATE OPERATION OPTIMIZATION ROUTINE

One of the most important components of a spillway is the control device, since it regulates and controls the outflows from the reservoir. This control limits or prevents outflow below fixed reservoir level, and it also regulates releases when the reservoir rises above that level. The control structure may consist of a sill, weir, orifice, tube or pipe. The discharge - head relationship may be fixed as in the case of a simple overflow crest or unregulated port, or it may be variable as with a gated crest or a valve-controlled pipe. Spillways with a fixed discharge - head relationship are often referred to as uncontrolled spillways whereas spillways with a variable discharge - head relationship are known as controlled spillways.

Controlled spillways require operation and the person, or persons, responsible for this operation require rules, guidelines or a means of calculating how and when to operate the spillway. In practice this decision depends on a large number of unrelated factors which include the size and shape of the incomming flood hydrograph, the pre-flood warning time, the reaction time required, the delay time between successive gate operations, the water level in the reservoir, the water level required in the reservoir after the flood, the maximum allowable level in the dam during the flood, the minimum drawdown level possible, the maximum allowable rate of increase of flow and the maximum allowable flow rate downstream from the reservoir.

Another equally important consideration is the end use of the reservoir. For example, if a reservoir is used primarily for water storage the objective would be to have a full reservoir after the flood has passed. If the same reservoir were to be used for flood control the objective would be to absorb the flood temporarily and discharge it at a rate as low as possible over as long a period as practicable. Ultimately the reservoir would be drawn down as low as possible in preparation for the next flood.

As so many factors influence the gate operation rules required for a particular dam it is very difficult and often impossible to define an optimum operation sequence for a particular flood. The optimization routine presented here would calculate a gate operation sequence, to yield an outflow hydrograph with as low and flat a peak as possible, subject to the constraints applicable to the particular dam, for a given inflow hydrograph. It is therefore important to note that the resulting gate operating rule can only be as good as the flood prediction method used. If the predicted inflow hydrograph does not represent the actual flood the calculated operation rule would not be correct.

When using the optimization technique the user should be aware of another possible danger. When the "optimum" outflow hydrograph cuts through the receding limb of the inflow hydrograph the dam will be full to the maximum level allowed. If the hydrograph used during the simulation does not contain the flood peak the user may later face the situation of having to cope with a larger flood with a full reservoir. It is therefore suggested that the inflow hydrograph prediction method used be such that it would produce a possible maximum and minimum flood hydrograph. The optimization routine should then be run with both hydrographs and the results should be interpreted by a person with a good working knowledge of the catchment under consideration.

If experience indicates that a larger flood may follow the optimization technique could be run allowing a reduced water level in the dam. This would provide additional storage for the expected flood. Again the decisions would be subjective and require a good hydrological background.

The optimization routine should therefore not be seen as definitive but only as a guide to rationalize decision making by the experienced user.

Continuity of flow through the reservoir reach applies

i.e. $$I = Q + \frac{ds}{dt} \qquad (8)$$

where I = inflow to the reach

Q = discharge from the reach

$\frac{ds}{dt}$ = rate of change in reach storage with respect to time

rearranging:

$$Q = I - \frac{ds}{dt}$$

This equation can be approximated, for a time interval Δt, by:

$$\bar{Q} = \bar{I} - \frac{S2-S1}{\Delta t} \tag{16}$$

Where $\bar{Q}$ = average outflow rate from the reach over Δt

$\bar{I}$ = average inflow rate to the reach over Δt

S1, S2 = storage in the reservoir at the start and end of a time step respectively

As the average inflow rate $\bar{I}$ is fixed for each time interval during a particular flood by the inflow hydrograph, it is clear that the only way in which the average outflow rate $\bar{Q}$ can be decreased is by increasing the value of S2 - S1 i.e. The more storage you use, the smaller the ouflow would be.

If this logic is repeated for each time step until $\bar{Q}$ exceeds $\bar{I}$, i.e. when the outflow hydrograph cuts through the receding limb of the inflow hydrograph, it is clear that the outflow volume is minimized when the storage volume is maximised. The outflow hydrograph peak can therefore be minimized by ensuring that all available storage in the reservoir is used when the outflow hydrograph cuts through the receding limb of the inflow hydrograph i.e. when the reservoir level starts dropping again. This principle is demonstrated in Figure 5. This 'optimum' outflow hydrograph is calculated as follows.

Initially the volume under the inflow hydrograph, INVOL, the permanent storage capacity in the reservoir, PERMSTOR (if available), and the temporary storage capacity in the reservoir, TEMPSTOR, can be calculated. If the permanent storage capacity exceeds the inflow volume no gate operation is required and no further calculations need be done.

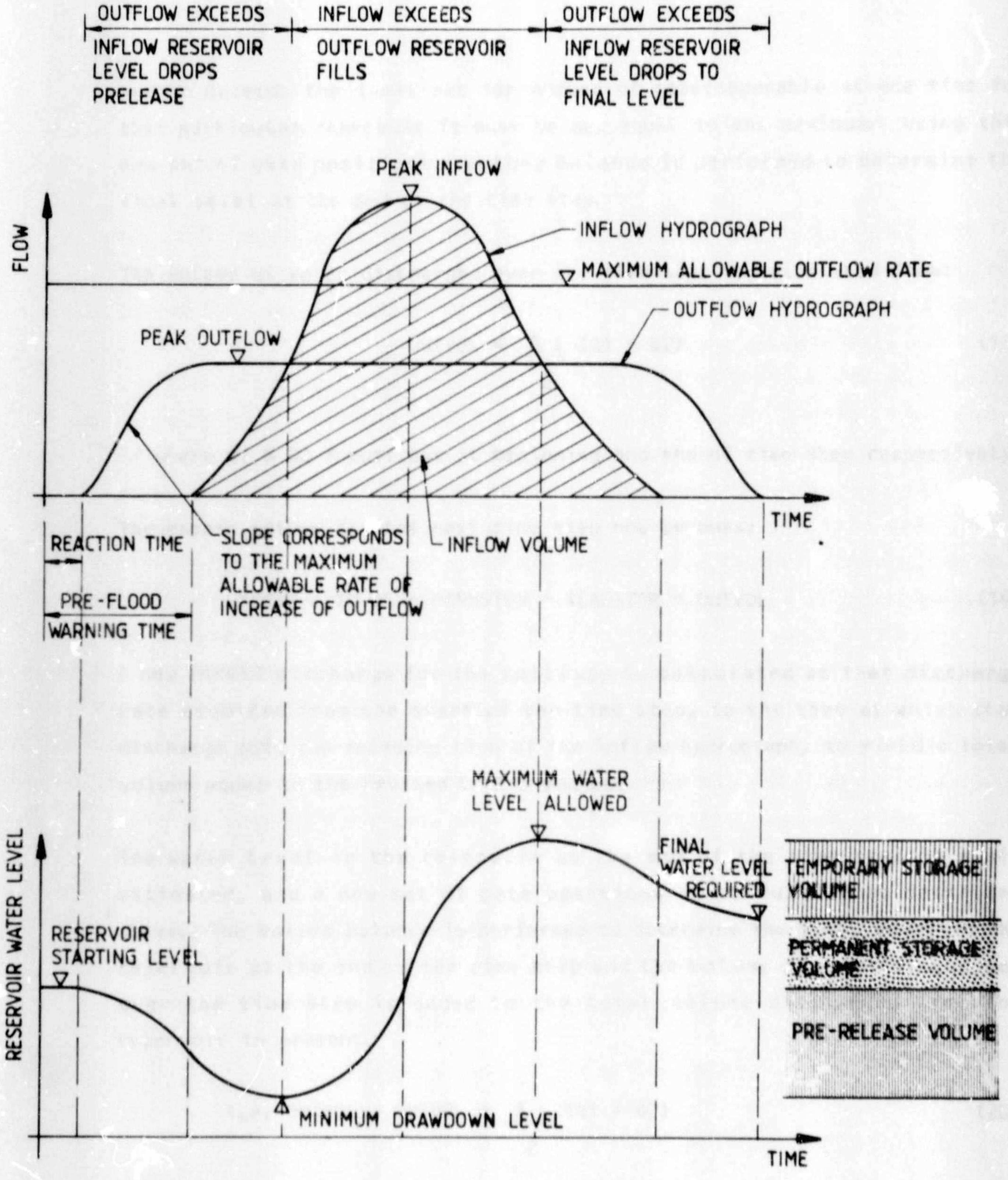
If we now define EXCESS as an excess volume of water so that:

$$EXCESS = INVOL - PERMSTOR - TEMPSTOR \tag{17}$$

it follows that this volume of water must be discharged before the peak reservoir level allowed is reached i.e. the temporary and permanent storage capacity of the reservoir must be fully utilized only at the point where the outflow from the reservoir starts to exceed the inflow into the reservoir.

A TARGET discharge for the spillway is calculated as that discharge rate required from the earliest reaction time, to the time at which that discharge cuts the receding limb of the inflow hydrograph, to yield a total flow volume equal to EXCESS. The calculated TARGET discharge is now checked against the inflow hydrograph peak. If TARGET exceeds the inflow hydrograph peak the gate contraints are too stringent and should be revised.

The water level in the reservoir at the end of the time step is estimated by adding or subtracting, an increment equal to the change in level in the reservoir over the previous time step as the case may be. Using this estimated level an expected outflow is calculated at the end of the time step keeping the gates in their present positions. This expected outflow is compared with TARGET. If they are equal no gate operation is required for that time step and a volume balance is performed to determine the final level at the end of the time step. If they are not equal the number of gates to be opened or closed to make them equal is calculated. If this



OPTIMIZED OUTFLOW HYDROGRAPH AS CALCULATED
BY THE OPTIMIZATION ROUTINE

FIGURE 5

number exceeds the limit set for number of gates operable at one time for that particular reservoir it must be set equal to the maximum. Using this new set of gate positions a volume balance is performed to determine the final level at the end of the time step.

The volume of water discharged over the time step is calculated from:

$$\text{OUTVOL} = \frac{\Delta t}{2} (Q1 + Q2) \qquad (18)$$

where Q1 & Q2 = outflows at beginning and end of time step respectively

The excess volume for the next time step now becomes:

$$\text{EXCESS} = \text{INVOL} - \text{PERMSTOR} - \text{TEMPSTOR} - \text{OUTVOL} \qquad (19)$$

A new TARGET discharge for the spillway is calculated as that discharge rate required from the start of the time step, to the time at which that discharge cuts the receding limb of the inflow hydrograph, to yield a total volume equal to the revised EXCESS volume.

The water level in the reservoir at the end of the time step is again estimated, and a new set of gate positions is calculated as described above. The volume balance is performed to determine the final level in the reservoir at the end of the time step and the volume of water discharged over the time step is added to the total volume discharged from the reservoir to present.

$$\text{i.e. OUTVOL} = \text{OUTVOL} + \frac{\Delta t}{2} (Q1 + Q2) \qquad (20)$$

where Q1 & Q2 = outflows at the beginning and end of the time step under consideration

This procedure, demonstrated in Figure 6, is repeated for each time step until the outflow hydrograph cuts through the receding limb of the inflow hydrograph.

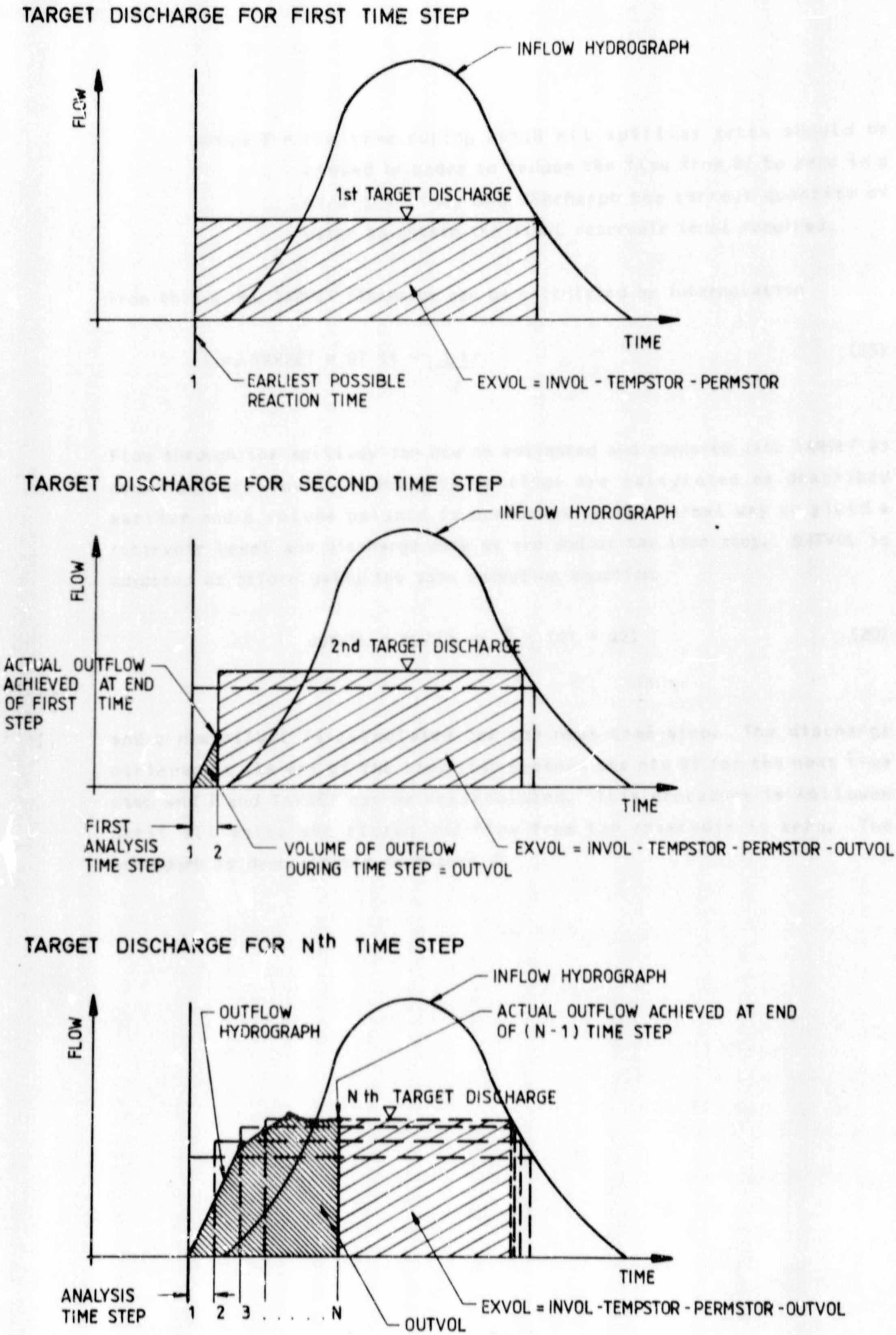
At this point, where the outflow hydrograph cuts the receding limb of the inflow hydrograph, the reservoir is full to its maximum allowed level, and as the inflow hydrograph is now smaller than the outflow hydrograph the water level in the reservoir will start to drop. In principle the flood has now been passed and attenuated as much as possible and all that remains to be done is for the reservoir level to be dropped to the final level required. This closing of the gates is best done gradually and the proposed method aims to do this in such a way that discharge from the reservoir falls from the peak outflow rate to zero in a straight line on the outflow hydrograph. This method has the advantage of avoiding the need to operate gates to their allowed maximum rate as far as possible. If gates are left open longer initially they will have to be closed in a shorter time later to avoid emptying the dam below the final level required. If the gates are closed more quickly initially the tail end of the storm will be retained in the reservoir longer which increases the time over which the gates have to be operated. As the outflow at this stage will be less than the peak outflow achieved, this increased operation time serves no purpose.

If we define DISVOL as the volume of water to be discharged from the point where the outflow hydrograph cuts through the inflow hydrograph, in order to return the reservoir to its final required level it follows that:

$$DISVOL = INVOL - OUTVOL + TEMPSTOR \tag{21}$$

If Qf denotes the discharge rate from the reservoir at the end of the previous time step it follows that:

$$T = \frac{2*DISVOL}{Qf} \tag{22}$$



CALCULATION OF TARGET DISCHARGE, RISING LIMB FIGURE 6 where T = the time during which all spillway gates should be closed in order to reduce the flow from Qf to zero in a straight line, and discharge the correct quantity of water to obtain the final reservoir level required.

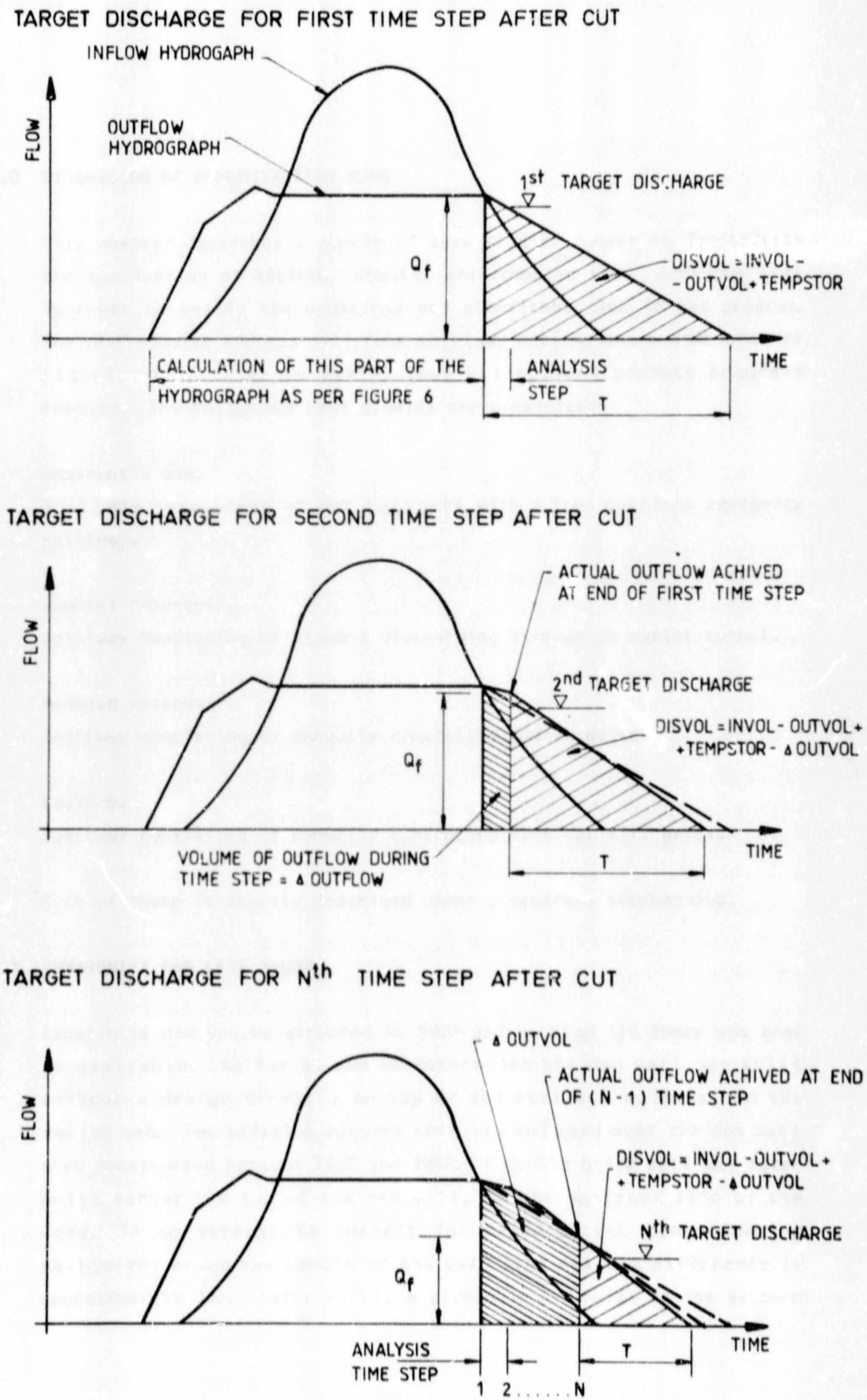
From this a new TARGET discharge can be calculated by interpolation

$$\text{i.e. TARGET} = Qf\left(1 - \frac{\Delta t}{T}\right) \tag{23}$$

Flow through the spillway can now be estimated and compared with TARGET as described previously. New gate settings are calculated as described earlier and a volume balance is performed in the normal way to yield a reservoir level and discharge rate at the end of the time step. OUTVOL is adjusted as before using the same summation equation.

$$\text{OUTVOL} = \text{OUTVOL} + \frac{\Delta t}{2}(Q1 + Q2) \tag{20}$$

and a new DISVOL is calculated for the next time step. The discharge achieved at the end of the time step becomes the new Qf for the next time step and T and TARGET can be recalculated. This procedure is followed until all gates are closed and flow from the reservoir is zero. The procedure is demonstrated in Figure 7.



CALCULATION OF TARGET DISCHARGE, RECEDING LIMB

FIGURE 7

3.0 DISCUSSION OF DEMONSTRATION RUNS

This chapter describes a number of case studies chosen to demonstrate the application of RESIMO. Results are compared with previous work in order to verify the equations and algorithms used in the program. The case studies address different spillway options and volume balance algorithms in order to verify that all options produce accurate results. The following case studies are presented:-

Emmarentia dam.
Spillway consisting of box culverts with a free overflow emergency spillway.

Jumbles reservoir.
Spillway consisting of siphons discharging through an outlet tunnel.

Mvumase reservoir.
Spillway consisting of manually controlled radial gates.

Vaaldam.
Spillway consisting of manually controlled vertical lift gates.

Each of these is shortly described under a separate sub-heading.

3.1 EMMARENTIA DAM CASE STUDY

Emmarentia dam was constructed in 1902 and no plan (if there was one) is available. As far as can be determined the dam wall was built without a design directly on top of the residual material in the valley bed. The existing culvert spillway and road over the dam wall were constructed between 1947 and 1949. A double brick wall was later built across the top of the dam wall, on the upstream side of the road, in an attempt to correct for differential consolidation settlement along the length of the wall (the maximum difference is approximately 1m). Later still, a clubhouse was built on the eastern bank of the dam only 4,5m upstream from the culvert spillway inlet, partially blocking three of the original six spillway bays. A number of large trees have been planted around the clubhouse further obstructing flow to the spillway.

These factors, together with the very steep downstream slope of the wall lead to the safety of the dam being questioned.

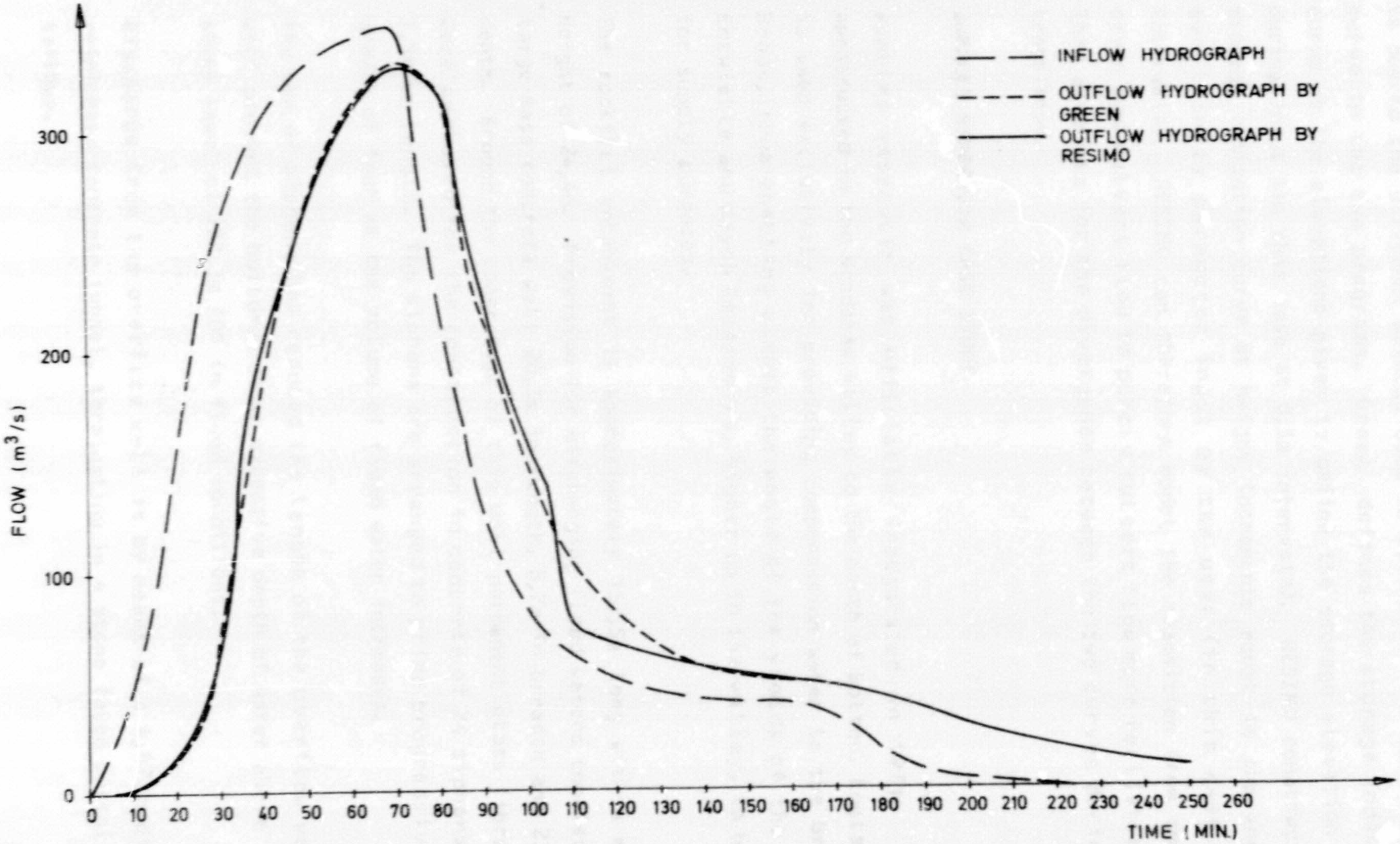
In 1983 the Johannesburg City Council initiated an analysis of the existing spillway capacity.

In the final report, Furstenburg (10), showed that the culvert spillway could pass a 100 year recurrence interval flood, provided that the boathouse and trees in the spillway were removed. Due to development downstream of the dam wall the Department of Water Affairs required the spillway to pass a 400 year recurrence interval flood without endangering the dam wall.

A number of feasible ways of enhancing the spillway capacity were analysed. The analysis was done using a reservoir simulation model by Green (11). This model uses the storage-indication method of volume balance and allows box culverts and ogee crests as spillway options.

Results of analysis of one of these solutions, consisting of the six existing culverts, with the boathouse and trees removed, and a sixty metre wide free overflow emergency spillway over the top of the dam wall, are presented in Appendix F on page F-2. This solution was reanalysed using RESIMO and the results are presented in Appendix F on page F-6.

Figure 8 shows a plot of the inflow hydrograph as well as the two outflow hydrographs. Close correlation between the two hydrographs verify the storage-indication method of volume balance used in RESIMO as well as the application of the basic equations used to describe box culverts and ogee crests.



INFLOW AND OUTFLOW HYDROGRAPHS FOR EMMARANTIA DAM CASE STUDY

FIGURE 8

The divergence of the outflow hydrographs between 100 and 150 minutes is due to the different methods used to define the storage indication curve in the two programs. Green defines the storage indication curve at the elevations given to define the storage elevation curve during input (in this case at 0,5m intervals). RESIMO constructs the storage indication curve at height increments equal to the analysis sensitivity parameter input by the user (in this case 0,01m intervals). RESIMO can therefore model the transition from combined crest and culvert flow to purely culvert flow more realistically. This accounts for the divergence between the two curves during the transition.

3.2 JUMBLES RESERVOIR CASE STUDY

Jumbles reservoir was officially inaugurated in 1971. It is constructed in the Bradshaw Valley to the north of Bolton, England and is used exclusively for providing compensation water to the Bradshaw Brook, thus enabling almost the whole of the yields of the older Entwistle and Wayoh Reservoirs, higher up in the valley, to be used for supply purposes.

The rockfill embankment is approximately 152,5m long with a maximum height of 24,4m. Provision for discharging flood waters consists of a large mass concrete well 20,3m in length, 8,7m in breadth and 23,5m in depth. Around the upper edge of this well permanent glass fibre forms were employed for the construction in concrete of 24 siphons, each 1,5m in length. The siphons are arranged to prime progressively in groups of four as the volume of flood water increases.

The use of siphons has reduced the length of the overflow weir and will prevent the build-up of an excessive depth of water above the top water level of 132,3m AOD in flood conditions.

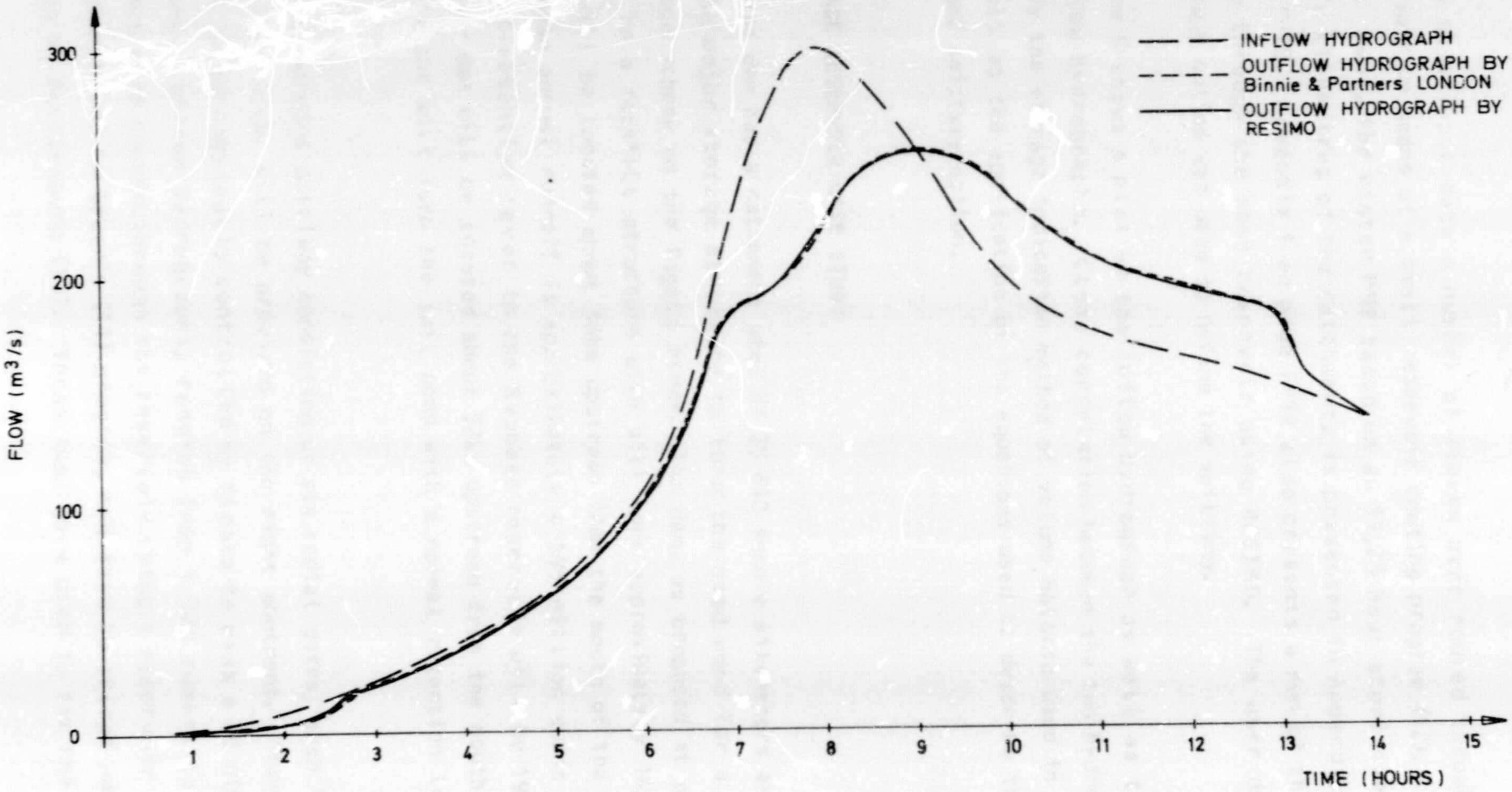
Discharge from the overflow well is by means of a 4,876m diameter reinforced concrete tunnel, terminating in a stone faced portal in the tailbay.

Using TWL as datum the following storage-elevation curve was drawn up for the Jumbles Reservoir.

Elevation (m)	Storage volume (m^3)
0.0	0
[illegible]	256 000
2.0	530 000
3.0	822 000
4.0	1 132 000
5.0	1 460 000

A model study of the siphon spillway system yielded the following elevation-discharge curve:-

Elevation (m above TWL)	Discharge (m^3/s)
0.00	0
0.06	3
0.12	7
0.24	84
0.37	159
0.43	181
0.49	184
1.19	201
1.50	219
1.70	233
1.85	246
2.2[illegible]	262
2.78	279
3.22	296
4.10	330
4.98	364



INFLOW AND OUTFLOW HYDROGRAPHS FOR JUMBLES RESERVOIR CASE STUDY

FIGURE 9

Using this input data a number of storms were routed through the reservoir by means of a small reservoir routing program (12). One of these runs, the winter PMF based on an 11,25 hour storm over 34,3 square kilometres of the catchment, is presented in Appendix F, on page F-10. Appendix F on page F-12 also presents a run of the same storm through the same reservoir using RESIMO. The user defined spillway option was used to define the spillway.

Figure 9 shows a plot of the inflow hydrograph as well as the two outflow hydrographs. Close correlation between the two hydrographs verify the storage indication method of volume balance used in RESIMO as well as the application of the equations used to describe the user defined spillway option.

3.3 MVUMASE RESERVOIR CASE STUDY

Mvumase dam has a catchment area of 28 640 square kilometers and will be the major storage structure to form the head pond for a pumped storage scheme on the Tugela river. The dam, as proposed at present, will be a rockfill structure with silt core approximately 145m high and will be located about 56km upstream from the mouth of the Tugela. The mean annual run-off is approximately 4 595 million cubic metres. Normal retention level in the Mvumase reservoir will be 193m AOD. Sunbury dam will be located about 32km upstream from the mouth of the Tugela and will form the tail pond with a normal retention level of 76m.

A gated service spillway consisting of six radial gates, each 17m wide and 14,5m high, will be provided on the right abutment. These gates will be automatically controlled by floats to pass a 20 000 cumec flood. Various hydrographs, ranging from 1 000 cumecs to 20 000 cumecs were routed through the reservoir using a reservoir routing program based on Green's (11) program, modified to accept automatic gates by Furstenburg (13). These runs were used to set the levels

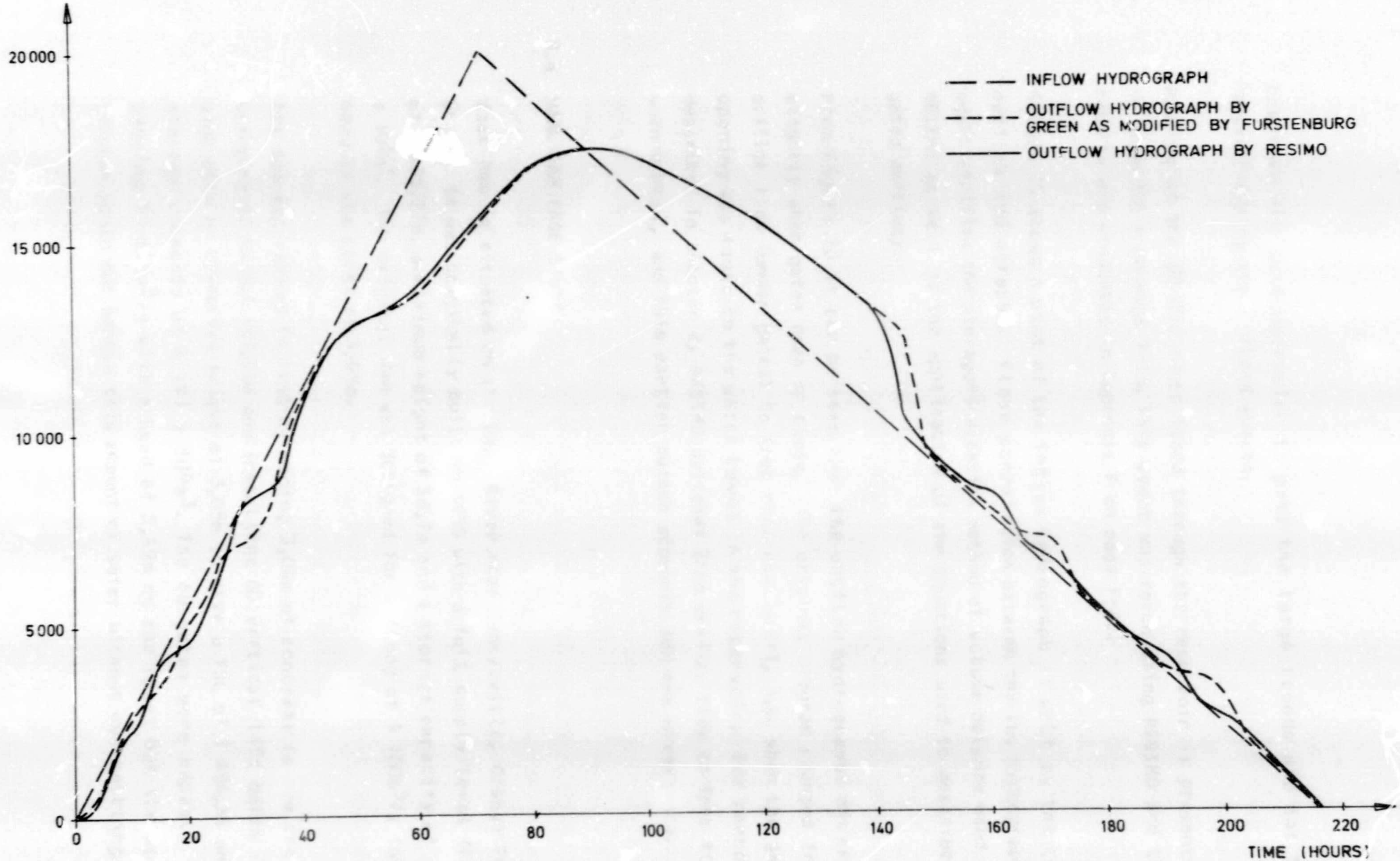


FIGURE 10

INFLOW AND OUTFLOW HYDROGRAPHS FOR MVUMASE DAM CASE STUDY required for gate operation to pass the large floods and not to "search" during small flood events.

Routing of the 20 000 cumec flood through the reservoir is presented in Appendix F on page F-16. This event was rerun using RESIMO and the results are presented in Appendix F on page F-22.

Figure 10 shows a plot of the inflow hydrograph as well as the two outflow hydrographs. Close correlation between the two hydrographs again verifies the storage-indication method of volume balance used in RESIMO as well as the application of the equations used to describe a gated spillway.

From Figure 10 it can be seen that the outflow hydrographs diverge slightly when gates open or close. The original program changed from orifice flow (under gates) to free flow when d/H=1, i.e. when the gate opening was equal to the water level in the reservoir. For reasons described in chapter 2, RESIMO switches from orifice flow to free flow when d/H=0.7, and this earlier switch accounts for the divergence.

3.4 VAAL DAM CASE STUDY

Vaal Dam is situated on the Vaal River near Deneysville, Orange Free State. It was originally built in 1938 with a full supply level (FSL) of 1 478,27m, a maximum height of 56,7m and a storage capacity of 880 x $10^6 m^3$. The original dam was designed for a flood of 5 $100 m^3/s$ for a head on the crest of 3,05m.

The dam was raised in 1956 by adding 3,04m of concrete to the crest, bringing it to RL1 481,31m and installing 60 vertical lift gates 7,78m wide with an effective height of 3,05m to give a FSL of 1 484,36 and a storage capacity of 2 191 x $10^6 m^3$. The 60 gates were capable of passing 10 $800 m^3/s$ with a head of 5,49m on the crest but the apron troughs could not handle this amount of water without overspilling.

In the present betterments the dam is being strengthened by prestressing and thickening. The storage FSL is being raised by 1,12 m to 1 485,48 m and the new gates which are vertical lift gates 7,78 m wide by 6,35 m high, will allow storage to the servitude level of RL 1 487,41 m giving an extra flood absorption capacity above FSL of 651 x $10^6 m^3$. The apron troughs have been enlarged and the old Rand Water Board outlet pipes along the right bank of the river have been protected, and the embankments have been raised and provided with a wave wall. The spillway will be capable of passing the probable maximum flood (PMF) of 25 000 m^3/s without the dam being overtopped when the present works are completed in 1985.

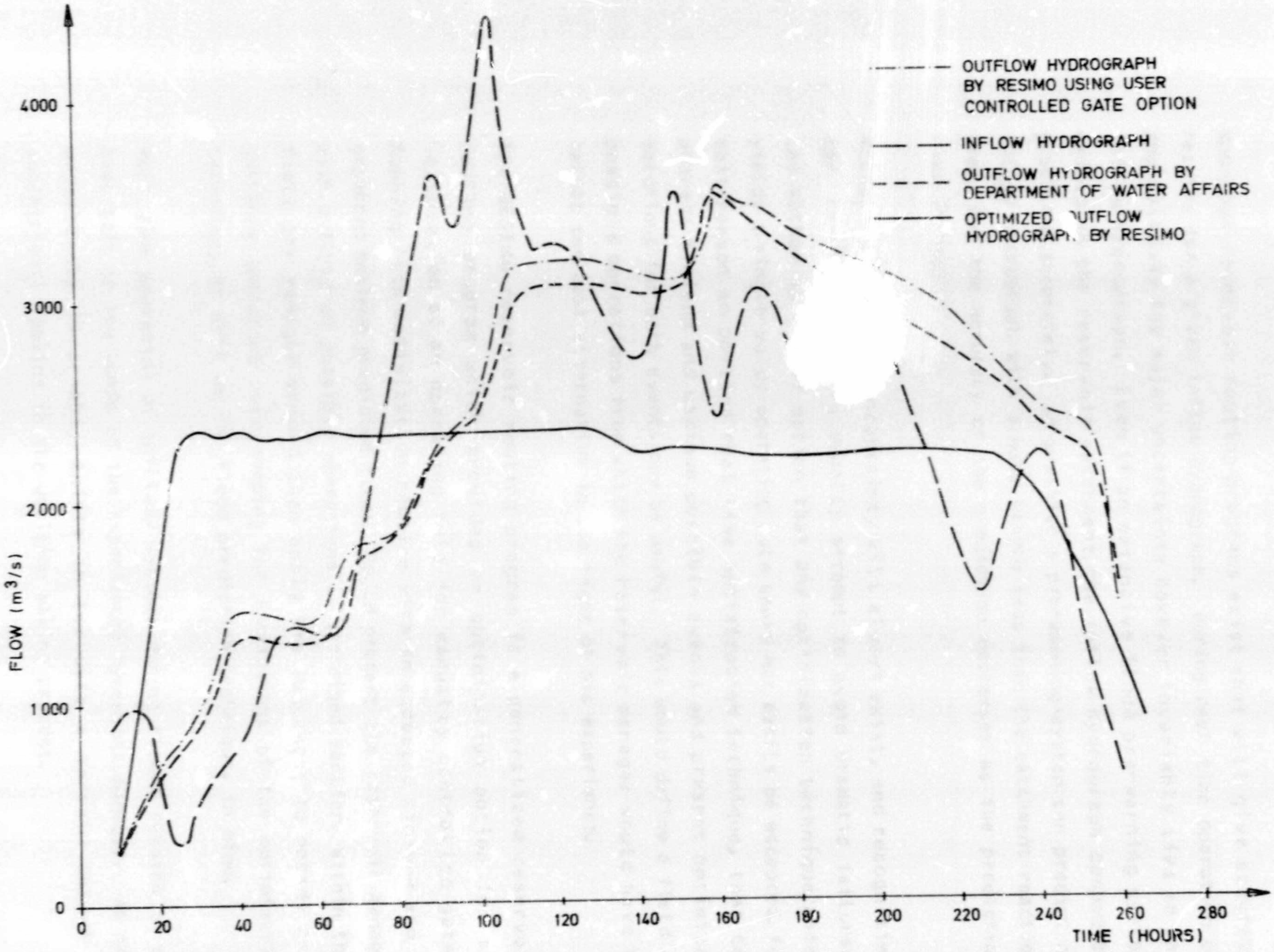
The 60 vertical lift gates on the spillway crest are manually operated and an operation sequence for the gates must be determined for each flood.

An extensive flood pre-warning system exists in the Vaal Dam catchment. Data obtained from this pre-warning system is routed through the river system using a calibrated Muskingum routing program, to yield an expected inflow hydrograph into the dam. A gate operation sequence is assumed and the flood routed through the dam using a Vaal Dam specific routing program. This operation sequence is revised, and the routing program rerun until a suitable gate operation rule is obtained. The actual operation of Vaal Dam during the 1975 flood, calculated using the above procedure is presented in Appendix F on page F-29.

This same flood, using the same gate operation sequence was rerun using the user operated manual gate option in RESIMO. The results are presented in Appendix F on page F-41. The same flood was then rerun using the program operated manual gate option in RESIMO and these results are presented in Appendix F on page F-51.

Figure 11 shows a plot of the inflow hydrograph as well as the three outflow hydrographs. Close correlation between the hydrograph calculated by the Department of Water Affairs, and the hydrograph calculated by the user operated manual gate option in RESIMO, verifies the iterative search technique of volume balance used in RESIMO, as well as the application of equations describing gate flow.

Figure 11 also clearly shows that the "optimized" outflow hydrograph calculated by RESIMO yields a much lower outflow peak than that achieved by the Department of Water Affairs during the 1975 flood. This is due to optimal use of prerelease and storage volumes in the dam. It snould however be stressed again that this calculation is only possible if the whole inflow hydrograph is known. It is therefore unlikely that this optimal operation could have been achieved totally in 1975 even with the aid of RESIMO. The user is referred to chapter 2.4 for a description of the use of optimization technique during real time operations.



INFLOW AND OUTFLOW HYDROGRAPHS FOR VAAL DAM CASE STUDY

FIGURE 11

4.0 DISCUSSION AND CONCLUSIONS

Numerous reservoir routing programs exist that will give accurate results for a given inflow hydrograph. During real time operation of any spillway the major uncertainty however invariably lies in the inflow hydrograph. Even if an extensive flood pre-warning system exists in the reservoir catchment the inflow hydrograph cannot be predicted accurately. In practice a pre-warning system can predict an inflow hydrograph with a base length less than the catchment reaction time, but the accuracy of the prediction decreases as the prediction time increases.

Accepting that this uncertainty will always exist, and recognizing that reservoir managers usually attempt to avoid dramatic failures, the author is of the option that any optimization technique which yields a single-value operating rule would not easily be accepted. For this reason an on-line real time optimization technique, that can address minimum and maximum possible events and present optimized solutions for both events may be useful. This would define a field of possible operations from which the reservoir manager would have to select the best alternative in the light of his experience.

The RESIMO reservoir routing program is a generalized reservoir routing program which provides an optimization option for the calculation of an operating rule for manually controlled gates. Running this optimization routine with an expected minimum and an expected maximum predicted flood, would present the reservoir manager with a field of possible operations as described earlier. Within this field the manager should then apply his judgement to decide on a suitable operating rule keeping the limitations of the optimization technique, as well as the flood prediction technique, in mind.

Real time operation of spillway systems during floods remains an art best left in the hands of the experienced reservoir manager. As some uncertainty invariably exists in the data a certain amount of subjectivity remains in the decision making process.

The most serious limitation of the RESIMO program must be the fact that it requires a full inflow hydrograph in order to run the optimization routine. The addition of a block to predict expected maximum and minimum floods during real time operations must be given serious consideration in the future.

Recognizing this shortcoming it is believed that RESIMO can be applied successfully in the design office to size and evaluate alternative spillway systems and to set operating limits for automatically controlled gates. It will also prove a useful on-line real time optimization tool for determining reservoir operation during floods.