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THE EFFICIENT OPERATION OF DAM SPILLWAY GATES

by

C. H. Munro, F. C. Bell and B. Watson

November, 1967

University of New South Wales

WATER RESEARCH LABORATORY

THE EFFICIENT OPERATION OF DAM SPILLWAY GATES

by

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Synopsis

Frequently in Australia one water conservation dam, and one only, is constructed with a gated spillway on a major river, and the gate operation procedures used take inadequate account of the possibility that the apparently conflicting objectives of water conservation and flood mitigation can often be largely reconciled by the use of flood forecasting techniques. In this report basic principles of gate operation are first reviewed. Details of a case study are then presented using three different methods of gate operation. It is demonstrated that gate operation based on flood forecasting can achieve appreciable flood mitigation without any reduction in water conserved and without any danger to the safety of the dam.

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THE EFFICIENT OPERATION OF DAM SPILLWAY GATES

by

C.H. Munro, F.C. Bell and B. Watson

1. THE BASIC PROBLEM AND PROPOSED SOLUTION

In recent years in Australia it has become almost standard practice to instal radial gates on spillways of large dams, in order to reduce the cost of the dam and the extent of inundation of land upstream of the reservoir during floods. As droughts are a greater menace than floods in Australia (the world's driest continent), these dams are usually single purpose dams for water conservation, and on most rivers there is only one such dam.

When major floods occur, the communities downstream of the dam often complain that it has aggravated flood damage. If the gates are not efficiently operated, these complaints may well be justified.

If all causes of such complaints are to be eliminated, the gate operation procedure should be such that during any given flood:-

(i) The peak rate of spillway outflow and peak flood levels downstream of the dam do not exceed those which would have occurred under natural conditions if the dam did not exist. (It is of course desirable that these peaks should in fact be reduced as much as possible below the natural peaks).

(ii) The maximum rate of increase and decrease of flood discharge over the dam spillway and the rate of increase and decrease in flood levels and the stream velocities downstream do not exceed those which would have occurred if the dam did not exist.

Except in special circumstances, ungated spillways ensure automatically that damage caused by a given flood will be less than the damage which would have been caused by that flood under natural conditions if the dam did not exist, but if gated spillways are inefficiently operated, the flood damage may exceed the damage which would have occurred under natural conditions. On the other hand, an efficient system of gate operation will result in less flood damage than would have occurred with an ungated dam of the samestorage capacity, and in damage very materially less than would have occurred under natural conditions.

It is important that gate operation should be highly efficient, and the intention of this paper is to discuss the methods of achieving the best results.

In regard to the conditions specified above, condition (i) must be observed at all costs. Condition (ii) is a severe constraint. Too rigid an observance of this condition on the rising flood may result in greater overall flood damage than would occur if rates of rise are permitted to be a little greater than would occur under natural conditions. However, appropriate permissible limits of such rates should be laid down which, on a given river, will be non-damaging. A sophisticated operation procedure might provide that such limits may be exceeded by a specified amount if it is clear that by so doing the overall flood damage will be reduced.

An important cause of damage on many rivers is that of river bank sloughing due to the "draw down" effect of sudden falls of river levels due to too rapid closing of spillway gates during the recession limb of the flood hydrograph. It is therefore essential that a limit should be placed on the maximum permissible rate of decrease of spillway discharge.

The main objective of this paper is to demonstrate that, to achieve the required efficiency, the operation of spillway gates should be based on a modern and complete system of flood hydrograph forecasting for the catchment.

It is often said that water conservation and flood damage mitigation are conflicting objectives which cannot be reconciled, because storage space reserved for flood mitigation can only be provided at the expense of conservation storage. The authors contend that these conflicting objectives can be reconciled to a material extent by basing gate operation on quantitative flood hydrograph forecasting. This is particularly important in Australia, because the extreme importance of obtaining the maximum possible regulated water yield renders the provision of empty flood mitigation storage an expensive luxury.

The authors also contend that if a water conservation authority constructs a dam with a gated spillway, thus saving some millions of dollars in the cost of the dam, it incurs a legal obligation to operate the gates in such a manner that flood damage is not greater than that which would have occurred under natural conditions, provided it can do so while ensuring that the dam is full when the flood is over. There is obviously also a moral obligation to reduce the flood damage as much as possible below that which would have occurred under natural conditions.

The saving in the capital cost of the dam due to the use of a gated instead of an uncontrolled spillway is very much greater than the cost of developing a modern flood forecasting system upon which to base the operation of the spillway gates.

If the water authority is not prepared to recognise this obligation by using an efficient system of gate operation, the inhabitants of the valley below the dam are justified in demanding that the spillway should be ungated and the height of the dam increased to obtain the same storage.

This paper therefore considers the case of a hypothetical dam, with storage-reservoir-water elevation curve and spillway and gate characteristics similar to those of Warragamba Dam in the Hawkesbury Valley in New South Wales, but located on a hypothetical river generally similar to the Macleay River in New South Wales, and analyses various methods of gate operation, assuming that:-

(a) There is only one dam on the river, with a town downstream on the river bank, and an uncontrolled tributary joins the river between the dam and the town. This is a situation which is common in Australia.

(b) The reservoir storage must be at "full supply level" after every flood in which the gates release water.

(c) The rates of increase and decrease in spillway discharge during every flood must not exceed those which would have occurred at the dam site for such flood under natural conditions if the dam did not exist.

(d) Any "pre-release" of water from the conservation storage during the early stages of a flood, to provide flood storage for later flood mitigation must be limited to non-damaging flows.

(e) The peak rate of spillway discharge must not exceed that which would have occurred at the dam site under natural conditions, and should in fact be reduced to the minimum possible while observing conditions (b), (c) and (d).

Reasonable relaxation of conditions (b), (c) and (d) would probably usually lead to a greater overall benefit to the community as a whole, but their observance, as in this study, forestalls any complaints by the two interests concerned, i.e.:-

(i) Beneficiaries of water stored for conservation use, and

(ii) Downstream communities which may sustain flood damage.

The stipulations in the conditions specified above do not prohibit the duration of flooding of low lying land exceeding the duration which would occur under natural conditions. Reduction of natural flood peaks by reservoir storage necessitates an increase in the duration of the lower flood discharges. This can be minimised by providing in the recession phase for the maximum rate of decrease of discharge permissible without aggravating bank sloughing. In any case, in most cases a downstream property owner would prefer to have his low lying land flooded for a longer period if this means that his higher land would not be flooded at all, or would be flooded to a lower level.

2. SOME RELEVANT ASPECTS OF DAM DESIGN

Figure I is a reproduction of a diagram from "Elements of Hydraulic Engineering" (McGraw-Hill) by Linsley and Franzini showing the zones of storage in a reservoir.

Graphs showing the relation of reservoir water surface elevation to volume of water in storage and to discharge over the spillway are essential pre-requisites to design. If the spillway is ungated, water must be stored above full supply level to cause a discharge over the spillway, and this "surcharge storage" automatically attenuates the reservoir inflow hydrograph and usually provides some flood control, so that the required capacity of an ungated spillway may be less than the peak design flood inflow. Gated spillways should have a discharge capacity, with all gates fully open, equal to the peak design flood inflow rate. The length of the spillway selected affects the elevation of the design flood surcharge level and the required height of gates if the spillway crest is controlled.

Figure 2 shows how the placing of gates on a spillway reduce the required height of a dam for a given storage.

When a dam is built in a river channel, the natural flow of the river is replaced by a long deep pool. The celerity of the flood wave through this pool is approximately equal to the square root of g x d, where "g" is the acceleration due to gravity and "d" is the depth of flow so that flood inflow at the upstream end affects the water elevation at the dam wall in a brief period of a few minutes to a few hours. Before the dam was built the natural flood wave passing down the river channel would take periods of a few hours to a few days to travel the same distance. Hence the "inflow to reservoir pool" hydrograph is steeper, higher and earlier than the natural hydrograph at the dam site for any given flood (see Fig. 3) which also demonstrates the manner in which the storage in a reservoir flattens and delays the hydrograph of flow entering the reservoir pool during a flood.

For gated dams, a common method of gate operation is to keep the gates closed during a flood until the water level reaches static full pool at the top of the gates or one foot below the top of the gate, and then keep on opening the gates to maintain the rate of outflow equal to the rate of inflow, so that the water level remains constant until all gates are fully opened. Thenceforward, the flow is uncontrolled, and surcharge storage builds up in the same manner as for an ungated dam.

Under this procedure, if the reservoir is at or near full supply level before the flood, the first section (and for small and medium floods the whole) of the rising limb of the outflow hydrograph will be identical with the "inflow to full pool" hydrograph, which is steeper, higher, and earlier than the natural hydrograph at the dam site. If the reservoir is only partially full before the flood, this procedure causes a very rapid rate of increase of discharge when the reservoir level reaches the top of the gates. (See Section 4).

These ill-effects can be avoided by using the "induced surcharge" method. This involves opening the gates gradually from the time the flood inflow causes the reservoir to rise above the spillway crest. As the top of the gates rise, additional storage space becomes available between the static full pool level and the top of the gates. This has the effect of forcing floodwater into this additional storage space (termed "induced surcharge storage") with resultant delay and attenuation of the first portion (and for small and medium floods the whole)of the rising limb of the outflow hydrograph, as discussed in Section 4 below.

At a certain level, to ensure the safety of the dam, all gates must be fully opened, and spillway discharge is thenceforward uncontrolled.

3. THE MAXIMUM PERMISSIBLE INDUCED SURCHARGE CURVE

If, using the induced surcharge method, the gates are held at a small opening for too long a period, and the inflow is progressively increasing, the water level would approach the top of the gates, and to avoid overtopping of the gates and the attainment of dangerous levels, the gates would have to jump to the full open position very rapidly, with consequent damaging rates of increase of outflow greater than would have occurred if no surcharge had been induced.

G.A. Hathaway 1950(1), put forward a procedure to safeguard against such ill effects. Figure 4, which is copied from Hathaway's paper, shows the geometry of the problem. It is necessary to place limits on how much surcharge storage may be induced, in order to remove any possibility of rapid opening of the gates to prevent over-topping. Hathaway does this by developing a "maximum permissible induced surcharge curve". The first step is to compute a set of spillway rating curves showing the discharge which would occur with all gates raised simultaneously by successive increments of one foot until fully opened. (See the graph on Fig. 5 which is copied from Hathaway). By projecting horizontal lines to this chart from the top of gate elevations of the gate diagram of this figure, line G can be drawn showing the elevation of the top of the gates for various gate openings. Obviously, induced surcharge should not be permitted to rise above this level, and some few feet of freeboard is usually allowed.

The next step is to decide the reservoir water level at which all gates should be fully opened, and the inducing of surcharge should cease. As will be seen from Section 10.2 of this paper, the benefits of induced surcharge decrease rapidly as gate openings increase, whereas cost of gates increase with increased induced surcharge. Hathaway states "In most cases it is practicable to design the gate opening machinery and facilities to operate satisfactorily when the partially controlled reservoir level is 4 to 8 ft. above the static-full-pool elevation without an appreciable increase in cost".

Having made this decision (4 ft. maximum surcharge by the time all gates are full open in the example given in Fig. 5), it is necessary to decide the manner in which the induced surcharge should reach this figure. This is explained fully in the later section 10.2 of this paper. A straight line from full pool level with all gates closed to the level for all gates full open would ensure the minimum rate of increase of discharge. Hathaway has selected line E of Fig. 5. This increases the rate of increase in discharge for discharges exceeding 20,000 cusecs, but has the advantage that for the more common minor floods a greater surcharge and lower release rates would be achieved, due to the curvature of the line at low outflows. The minor to medium floods are usually much more frequent than major floods, and often contribute the major portion of the "average annual flood damage" and hence mitigation of these medium and minor floods is usually the main contribution to "average annual flood mitigation benefits" over the economic life of the dam, and it is for this reason that Hathaway chooses this shape for the curve.

4. COMPARISON OF FLOOD CONTROL EFFECTIVENESS OF CONTROLLED AND UNCONTROLLED SPILLWAYS

Hathaway, 1950(1), considers the relative merits of two designs for a dam at a given dam site, as shown in Fig. 6(a) and Fig. 6(b).

Both dams have the same maximum surface water level 414 ft. during the passage of the design flood of 400,000 cusecs. However, the dam of Fig. 6(b) is gated (giving a static full pool level of 405 ft. compared with 395 ft. for the ungated dam), and the advantage of gates is shown by the fact that it can provide the same flood mitigation storage as the ungated dam, but can also provide considerable conservation storage, so that it can be a multi-purpose dam for hydro-power production and flood mitigation, whereas the ungated dam can only provide for flood mitigation. (Presumably also its capital cost would be less than that of the ungated dam.) Hathaway assumes that an average rate of outflow of 100,000 cusecs is permissible during periods when available flood control capacity is being filled, and that an outflow exceeding 30,000 cusecs would cause considerable downstream damage.

For gate operation, Hathaway considers two methods:-

"Plan A. Gates would be opened at the rate necessary to maintain an outflow rate equal to the reservoir inflow rate until all gates are fully opened, after which the reservoir pool elevation would rise until the head necessary to pass the rate of inflow is attained.

Plan B. The gates would be raised in unison by small increments, limiting the spillway discharge to rates represented by curve 3 in Fig.6(c) as the reservoir rises." (i.e. using induced surcharge).

Fig. 6(c) shows the relation between reservoir water level and spillway discharge for the ungated dam and for the gated dams under Plans A and B.

Hathaway then compares the performance of the two dams when subjected to the design flood, firstly when the dams are at static full pool level at start of flood, and secondly when 50% of the flood control storage is empty at start of the flood. The resultant hydrographs are shown in Fig. 7(a) and Fig. 7(c).

Referring to Fig. 7(a), it will be seen that under Plan A, the outflow hydrograph follows the inflow hydrograph up to a discharge of approximately 120,000 cusecs, and is thus steeper than would be the case for the flood.

under existing conditions if the dam did not exist (see Fig. 3), and the rate of discharge under Plan A exceeds that under Plan B for the gated dam until the inflow approximates 160,000 cusecs, although the peak rate of outflow under Plan B is slightly greater than that for Plan A. The ungated dam is superior in all respects, so far as flood mitigation is concerned, to the gated dam under either plan.

However, Fig. 7(a) is unrealistic. Firstly, the spillway design flood (which may often be the estimated maximum possible flood) is such a rare event that it has little effect on the average annual flood damage taken over the whole life of a dam. The more frequent medium and minor floods contribute most of the damage. Secondly, even for conservation dams the reservoir is rarely full when a flood occurs.

Therefore Hathaway in Fig. 7(c) presents a more typical case with a dam half full receiving a flood whose hydrograph ordinates are 50% of those of the design flood. In such a case, the gated dam under Plan A allows no release until static full pool level of 405 is reached, and the discharge suddenly jumps to 125,000 cusecs. This is most undesirable. On the other hand, the gated dam under Plan B gives a relatively low rate of rise (and also fall) and a much lower peak outflow discharge. The ungated dam gives a better flood mitigation in all respects.

Fig. 7(b) shows that for all floods of peak inflow rates of less than 180,000 cusecs, even if the dam is full when the flood arrives, the peak outflow rate is less under Plan B than under Plan A for the gated dam, while for all floods the peak outflow rate is least for the ungated dam.

5. U.S. CORPS OF ENGINEERING PROCEDURE

As a result of studies of this nature, Hathaway comments:-

"Experience in the operation of gated spillways has shown that significant flood damages may arise under Plan A because of the fact that reservoir releases may be larger during floods which occur when the reservoir is full, or near full, than would have been the case under natural conditions before the construction of the reservoir. Difficulties have also been encountered in the operation of gated spillways because of sudden increases in rate of outflow. For these reasons, the Corps of Engineers adopted a policy requiring that, insofar as practicable, reservoirs controlled by gated spillways be designed and operated to accomplish the following objectives during periods when the reservoir is filled or nearly filled.

(i) Peak rates of reservoir release during damaging floods should not exceed peak rates of the corresponding floods that would have occurred under runoff conditions prevailing before construction of the reservoir, and

(ii) The rate of increase in reservoir released during a significant increment of time should be limited to values that would not constitute a major hazard to downstream interests".

U.S. Corps of Engineers' practice 1959(2) combines the following procedures to achieve these aims:-

"(a) When predictions indicate that anticipated runoff from a storm will appreciably exceed the storage capacity remaining in the reservoir, the opening of spillway gates will be initiated before the reservoir has filled, and will be scheduled to limit the rate of increase in outflow to an acceptable value," and also

"(b) The use of induced surcharge."

An important feature of the Corps of Engineers' Manual is that it also provides for a primitive form of flood forecasting to be employed for (a) above on those occasions when damage to telephone and radio communications prevents the use of methods based on prediction of catchment runoff hydrographs, based on records of rainfall which has occurred. This involves the use of an "emergency release chart" showing the relation between outflow release and reservoir water level, with rate of rise of level as a parameter. This chart was proposed by Hathaway 1950(1). Its derivation and use is described in Section 10.5 of this paper.

6. THE BASIC PRINCIPLES OF GATE OPERATION DURING FLOODS

6.1. Introductory Comment

The discussion in this Section assumes that gate operation of a gated dam is based on quantitative flood forecasting and Plan B (see page 7) is followed.

It is convenient to divide a flood period into three phases, as shown in Fig. 8, viz.:- (i) the non-damaging phase, (ii) the damaging phase, (iii) the recession phase.

It is clear from the foregoing discussion that an efficient system of gate operation involves the use of induced surcharge. However, if by this technique the levels are allowed to become too high and the flood inflow continues, it may be difficult to release sufficient water at a later stage to prevent overtopping or, more commonly, excessive rates of increase of spillway outflow may become necessary. This is prevented by assuming that every flood may prove to be the "design flood" (which may in some cases be the "maximum possible flood") and insisting that the induced surcharge level for a given outflow release rate must never exceed that specified by a "maximum permissible induced surcharge curve", developed as described in Section 10.5 below. Also, the release of too much water in readiness for an expected flood may mean a loss of conservation storage, because the water level may be below static full pool level at the end of the flood.

For a dam which is nominally a single purpose dam for water conservation, as is the case in this discussion, good flood mitigation requires the release of as large a volume of water as possible at non-damaging rates without risking a less-than-full dam at the end of the flood. This is necessary in order to make available as much storage as possible during the damaging phase and thus reduce the damaging outflows. The effectiveness of this operation obviously depends on how accurately the flood hydrographs can be predicted in advance for various "forecast intervals".

The ideal to be aimed at is to ensure that the storage will not reach the maximum permissible induced surcharge level until after the peak inflow has passed and the conditions at this stage should represent the maximum flood level and also the maximum steady outflow as shown by line CDE in Figure 8. This ideal cannot be achieved for very large floods, as the surcharge level would probably rise above the point when all gates are fully opened and no further control can be exercised over the flood. The better methods of gate operation attempt to reproduce the ideal case of Figure 8, i.e. they aim to make the maximum flood level in the reservoir coincide with a maximum induced surcharge level determined as described in Section 10.2 below.

6.2. The Non-Damaging Phase

For a water conservation dam it is desirable to commence releasing water at non-damaging rates soon after it becomes apparent that the storm runoff will raise the reservoir to full supply level. At this stage there are no advantages in storing water over and above the assured inflows that will eventually fill the dam, and the more space that can be reserved for later flood control, the more effective will that control be.

The time of commencement of non-damaging releases therefore depends on the predicted future minimum runoff volume. In most cases this can be obtained in the early stage of a storm with modern forecasting methods but even in the absence of forecasts, a minimum inflow volume can be predicted well in advance from relatively simple hydrograph analyses. (See Section 10.5).

Downstream flows from other tributaries and local runoff should be considered in determining the maximum non-damaging flows under various circumstances. In many cases, it is economically advantageous, from the point of view of the community as a whole, to provide channel and other downstream improvements that will allow higher non-damaging releases. Sometimes minor damage may be justified in this phase if it reduces the risk of severe damage in later phases, but decisions of this type would usually be made before the gate operation method is scheduled, and preferably with the consent of representatives of the downstream community.

As mentioned in Section 1, damage is caused not only by high magnitudes of outflow but by high rates of change of outflow. Relatively rapid increases result in surge wages which are associated with localized high velocities and turbulence, and rapid rates of rise of flood level frustrate attempts to remove goods and stock to a place of safety. Rapid decrease in outflow are also undesirable because they tend to cause sloughing of banks. It is therefore necessary to specify maximum allowable rates of change of outflow in gate operation schedules.

Prior to the commencement of the non-damaging phase it is important to provide early warnings to facilitate the removal of cattle, securing of boats and so on. Such warnings should be based on rainfall (rather than flood) forecasts and are of the nature of an "alert" rather than a notice of definite intention. Definite notice of intention to commence gate releases would also be essential but could not be given much in advance of the actual first opening of the gates.

When the conservation of the maximum possible amount of floodwater is an essential condition of operation, as is assumed in this paper, then at all times when spillway releases are being made, the predicted future inflow volume should always equal or exceed the volume of water required to raise the reservoir to static full pool level. There should be no releases when the predicted inflow is less than the volume necessary to ensure static full pool level, because under these circumstances a full dam is not assured when the flood is over. In practice, if the two volumes approach each other during the non-damaging phase it is necessary to stop the outflow. In many minor floods, there would consequently be either no outflow or only a short period of non-damaging outflow, with no following damaging phase.

It is therefore clear that the time of commencement and the magnitude of non-damaging releases depend on:-

(a) The level of the reservoir and corresponding inflow volume required to bring it to full pool level.

(b) The predicted minimum storm runoff and whether this is sufficient to satisfy (a).

(c) Downstream flood conditions which determine the maximum non-damaging releases.

(d) The maximum allowable rates of change of outflow.

The estimates of the minimum storm runoff volume is subject to error. To be on the safe side in ensuring the maximum conservation of water, some water supply authorities may prefer to underestimate it. Unless conservation of water is abnormally important, the wisdom of such a policy is doubtful, from the point of view of the community as a whole. A better overall benefit-cost ratio over the life of the dam would be achieved if flood mitigation benefits are given their true weight, and the best estimate of future inflow is adopted, instead of the lowest estimates. Similarly, water authorities may be tempted to adopt low estimates of the maximum non-damaging release to avoid criticism, but such a policy may lead to greater overall flood damage than would a higher estimate.

6.3 The Damaging Phase

Figure 9 shows the predicted future hydrograph at time t_1, at which time the current gate outflow release rate is Q. In the early stages of a flood, the predicted total inflow volume of water (PTI) represented by this hydrograph between t_1 and t_3 will be less than that required to fill the reservoir, and Q would be zero. As time goes on, the predicted hydrograph becomes higher and longer, and when volume PTI of Fig. 9 exceeds the volume necessary to fill the reservoir, the gates would be opened to give a release Q such that the volume represented by the shaded area of Fig. 9 would just fill the reservoir. This value of Q might at first be less than the maximum non-damaging outflow, but in many floods the time would eventually be reached when a release rate greater than the maximum non-damaging rate would be necessary to make the predicted net inflow volume (PNI) equal to the volume necessary to fill the reservoir. At this point the spillway outflow would be increased to the maximum non-damaging rate as soon as possible. At this stage the reservoir level would be below static full pool level, but the predicted future net inflow would indicate that if this rate of outflow were continued indefinitely and the predicted inflow hydrograph did not change, the level would later rise above full pool. In other words, the later inflows would be forced into induced surcharge storage. At first the predicted surcharge level at this maximum non-damaging outflow rate may be lower than the maximum permissible for this rate of outflow, but in a major flood the time would be reached when the predicted net inflow volume would later raise the level above that permitted for this outflow, if this rate of outflow were maintained indefinitely. Then, in order to ensure a minimum peak outflow at later stages, it is necessary to commence damaging releases immediately.

However, such an immediate release may cause greater damage due to shortness of time for evacuation or other emergency measures. A compromise, which "plays safe" in regard to criticism in this respect is to delay the damaging releases until the estimated flood discharge under natural conditions (if the dam did not exist) slightly exceeds the maximum non-damaging flow. The first damaging outflow release is such that if the predicted inflow hydrograph does not change and this outflow rate is maintained throughout the flood, the peak reservoir level reached later on will be just equal to the maximum permissible induced surcharge level for this outflow, which may be called the "optimum flood control outflow". After an appropriate interval, another check would be made on the predicted net inflow, and in a major flood it would be found that this has increased, so that if the current outflow is maintained, the later peak reservoir level will exceed the maximum permissible surcharge level for this outflow, which is consequently no longer the optimum value for flood control. It is therefore necessary to increase the outflow to such a value that the predicted net inflow volume at this rate will result in a later peak reservoir level just equal to the maximum permissible for this new optimum outflow rate. In making this change to a greater outflow rate, the rate of increase in outflow should not exceed the maximum permissible rate of increase of outflow. This limitation means that in a major flood the predicted net inflow volume keeps on increasing so rapidly that even when the outflow is increased at the maximum permissible rate of increase, the equilibrium stage (where predicted peak reservoir level equals maximum induced surcharge level for the current rate of outflow) is never reached, even for a brief period. Under these conditions the time comes when the maximum induced surcharge level for the current outflow is actually reached, and thenceforward the gates are so operated that the reservoir level-outflow rate relation follows the maximum permissible induced surcharge curve until all gates are fully opened, and the outflow then becomes uncontrolled. In other words, in these circumstances the maximum permissible rate of increase of outflow ceases to be the governing factor in gate operation.

The principles involved in these operations during the damaging phase are illustrated by Figure 9. If at time t_1 the volume of the shaded area PNI is greater than VML, which is the volume necessary to raise the reservoir level to the maximum permissible induced surcharge level corresponding to the current value of Q, then the outflow rate must be increased to a new value of Q so that if this new outflow rate is maintained indefinitely and the predicted inflow hydrograph does not change, then the new net inflow volume equals the volume of water necessary to raise the storage to the maximum permissible induced storage level corresponding to the new value of Q. This new value of Q is the required optimum flood control outflow.

6.4 The Recession Phase

This phase commences when the reservoir commences to fall after reaching its highest level during the flood. The maximum rate of outflow for the flood has now been released and the usual objective at this stage is to bring the reservoir level back to near full supply as quickly as possible without causing further damage.

This is particularly important if more rain is likely on the wet catchment because further flooding may be difficult to control when the reservoir level is high.

In some cases the previous maximum outflow can be maintained without detriment as shown in Figure 8. It is reduced at the maximum allowable rate as the full supply level, or as some other appropriate level is approached (point F in Figure 8). In this regard, if further flooding is predicted a level lower than full supply level may be selected.

When the continuation of the previous maximum outflow is likely to cause additional damage a decision must be made as to whether a reduction in the outflow is worth the increased risk of later serious flood damage. This is a similar problem to those mentioned in previous sections and such decisions should be made in the design phase of the dam investigations from analyses of flood behaviour and the nature of the damage caused by various operation procedures.

7. GATE OPERATION BASED ON CURRENT RESERVOIR LEVELS ONLY

There are two types of procedures currently used in Australia which allow the gate operation to be governed by the reservoir water levels existing during the flood, as follows:-

(a) Those in which outflow does not commence until the reservoir level attains static full pool level, and thenceforward the outflow-storage level relationship follows the maximum permissible induced surcharge curve until the gates are fully opened, when the outflow becomes uncontrolled. Sometimes slightly different level-outflow relations are used for rising and falling stages.

(b) Those in which outflows do not commence until the reservoir level attains static full pool level (or sometimes a lower level) and thenceforward the level is kept constant for as long as possible, with outflows equal to the inflow, until the gates are fully opened and the outflow becomes uncontrolled.

Both of these methods are inefficient from the point of view of flood control, as they usually store the early non-damaging flows and thereby increase the later damaging flows and possess the other disadvantages pointed out by Hathaway. From the point of view of the initial cost of the dam, method (b) may have some advantages because it gives a lower design outflow than the induced surcharge methods, permitting the construction of a lower dam. However, the major requirements of gate operation procedures, as given in section 1, are violated by method (b).

The above methods are readily automated but this should not be used during floods, except for minor dams.

8. GATE OPERATION BASED ON RESERVOIR LEVELS AND PREDICTED RATE OF INFLOW

8.1 Gate operation Based on Reservoir Level and Prediction of Flood Inflow Volumes Based on Rate of Change of Level.

At any given instant during a flood inflow into a reservoir, a study of the preceding increase in reservoir level and of the storage - elevation curve enables the rising limb of the inflow to reservoir pool hydrograph to be plotted.

The line CB of Figure 10 is such a plot. If all rain ceases at time t the probable shape of the future hydrograph would be as shown dotted in the figure. Hathaway and the Corps of Engineers, presumably in order to be on the safe side from the point of view of ensuring that future inflow is not overestimated, make the conservative assumption that point B of Figure 10 is actually the start of the recession limb of the flood hydrograph. With this assumption, the shaded area of Figure 10 is a low estimate of the minimum future inflow, and this is the predicted future inflow volume referred to in Sections 6.2 and 6.3 above for the case (under emergency conditions when communications are disrupted) when predicted future inflow is based on rise in water levels only. During the damaging phase the required outflow rate at any time t is Q in Figure 10, which is calculated so that the area AA^1B represents a volume which will just fill the remaining storage up to the maximum permissible surcharge levels for outflow Q. To use this method of estimating future flood inflow for gate operation, an "emergency release chart" such as that shown in Figure 11 (from Hathaway's paper) is drawn up by making a series of computations with various assumed values of inflow rate and of amounts of storage available from various reservoir water levels to the maximum permissible induced surcharge curve. This figure shows the relation between the appropriate outflow release and reservoir level, with average rate of inflow for the previous three hours as a parameter. For semi-skilled dam operators it is more convenient to express rate of inflow in terms of the average rate of rise in feet in the preceding three hours, as in Figure 12 which is taken from the Manual of U.S. Corps of Engineers 1959 (2).

This method is clearly superior to methods based on reservoir levels only, as it provides, by a primitive form of flood forecasting, for pre-releases to be made to prepare for flood waters which will arrive at some future time.

It assumes that the operator is capable of simple arithmetical calculations, but has no access to a telephone to obtain instructions based on rainfall and runoff on the catchment.

8.2. Gate Operation Based on Flood Forecasts from Rainfall and Streamflow Data on the Catchment

This is the method advocated by the authors. Modern flood forecasting procedures based on catchment data provide a much greater "forecast interval" and more accurate estimates of future inflow than the emergency method described in Section 8.1 and with such a system it is possible to release water from storage much earlier while still ensuring a full dam at the end of the flood, and this results in lower peak outflows during the damaging phase of the flood. Whenever gates are installed on dam spillways, the authority concerned, as part of the dam design and construction, should instal and operate such a flood forecasting system. The cost is relatively minor when compared with the total cost of the Dam.

The primary purpose of this paper is to compare the relative merits of various gate operating procedures.

The Commonwealth Bureau of Meteorology has been carrying out flood forecasting in the Macleay Valley for some years for the purpose of predicting peak flood heights and times of peak at the town of Kempsey in the Lower Macleay Valley. These forecasts have been made progressively during past floods, and a typical medium and a typical major past flood, with their progressive forecasts, provide an excellent basis for comparing the various gate operation procedures for a hypothetical dam (generally similar to Warragamba Dam) assumed to exist on the Macleay River.

9. FLOOD FORECASTING IN THE LOWER MACLEAY VALLEY OF NEW SOUTH WALES

9.1 General

Streamflow forecasting or, in a more limited sense, flood forecasting, is of relatively recent origin in Australia. The basic concepts are, however, well established in overseas practice and involve the application of modern hydrologic techniques to determine the rate of runoff from storm rainfall and the magnitude of the flood wave as it moves through a river system. It involves reaching an understanding of the rainfall-runoff regime of a river basin within the limitations of available data and manpower resources. Such an understanding can provide a sound basis for the construction and operation of man-made structures without unduly upsetting the natural order existing within the river basin.

Many factors have to be taken into account and the complex problems which are encountered in each river basin require careful analysis before a sound flood forecasting system can be established.

Some of the factors influencing the type of flood forecasting system developed are:-

The quality and length of the rainfall and streamflow records available for the catchment.

The type of forecast and length of warning required.

The location of the flooded area and the physical nature of the catchment area.

The variability of the rainfall distribution.

The progress in basic research and understanding of the hydrologic process.

The value of a flood forecasting system depends on its accuracy and the amount of advance notice it provides.

The optimisation of these two factors is often difficult as the longer the warning periods the more inaccurate the forecasts are likely to be. The actual amount of warning which can be provided is a function of the catchment physiography and the rainfall duration.

No hard and fast rule can be laid down and each particular catchment has its own peculiarities. For example, the basin lag in the Macleay Valley (4,000 sq. miles) is about 30 hours, which is approximately the time between the centre of mass of the excess rainfall and the peak of the hydrograph. For uniform storms this time to peak remains fairly constant irrespective of the storm rainfall duration and in the longer storms the time between the last increment of excess rain and the peak may be only a matter of six hours. Smaller mountain catchments typical of conditions in N.S.W. may have lags varying from 4 to 24 hours for catchments varying from 200 to 3,000 square miles.

The basin lag gives some idea of the maximum amount of warning which could be provided if a unitgraph procedure is used. Flood routing procedures provided less warning but they eliminate errors in the excess rainfall estimation. It is often necessary to combine these two basic techniques because, in many instances, a unit hydrograph is required to predict the local inflow into the routing reach. This local inflow from relatively small catchments must be carefully considered as it plays an important role in many river basins on the coast of New South Wales.

9.2 Cost of Flood Forecasting System

No accurate figures can be provided on the cost of a flood forecasting system and the following figures can only provide a rough guide to the type of costs one would expect:-

Development of Unitgraph and Routing Procedure etc. for a complex catchment	$10,000
Operation of station manned by volunteer observer	$ 300 per station per year
Operation of telemeter station	$ 300 per station per year
Implementation of telemeter network	$ 5,000 per station

9.3 The Forecasting Method

The forecasting method used in the Macleay Valley is based on a study carried out in 1959-1961 based on limited rainfall and streamflow data, Watson 1962(3), Bureau of Meteorology, 1963(4). It involves the application of a three hour unitgraph to an excess rainfall pattern which is supplemented by a simple streamflow routing technique for adjusting early warnings if it becomes apparent that the runoff has been incorrectly estimated.

The Bureau procedure for deriving loss rates and the catchment unitgraph are closely integrated and each step in the analysis is controlled to achieve a high degree of standardisation.

Estimates of the average catchment rainfall are based on a fixed network of stations which are selected by correlating the rainfall measured at each station with the "true" rainfall calculated from isohyetal maps. At least 25 individual storms are analysed for this purpose.

The subsequent values of daily average rainfall, the daily antecedent precipitation index and the gross storm rainfall are calculated using the fixed network of stations.

Unitgraphs are then derived for as many floods as possible using the least squares procedure outlined by D.N. Body and A.F. Rainbird, 1963(5). This method provides values of "initial loss" and "continuing loss". The "initial loss" or the initial portion of the gross rainfall which occurs before runoff commences, is correlated with the antecedent precipitation index. The "continuing loss" or the portion of the gross rainfall which does not contribute to runoff after surface runoff has commenced, is assumed to be constant for forecasting purposes.

The simple initial loss relationship adopted for the Macleay Valley is illustrated in Figure 13.

To ensure the derived and operational techniques are similar the same group of rainfall stations used for the derivation of the loss relations and unitgraph are used for operational forecasting.

9.4. The Forecasting System

A number of publications describe the investigation and development of the flood forecasting system for the Lower Macleay Valley and it is not intended to enlarge on the procedures that have been adopted. However, it should be clearly appreciated that the system has been streamlined to achieve the maximum warning period with a minimum possible error in flood peak predictions.

9.41. Data Requirements

To forecast the peak flows at Kempsey, rainfall reports from eight rainfall stations and river reports from two streamflow stations are required.

Each morning the rainfall and river height reports are received by the Bureau in Sydney and during critical periods arrangements are made for additional reports at three hourly intervals.

9.42. Communications and Equipment

Five of the rainfall stations, and the streamflow stations report by phonogram through the normal PMG telegraphic channels which are used for all the incoming and outgoing traffic at the Forecasting Centre in Sydney.

Full use is made of the Tress and Telex facilities for rapid handling of data and forecasts, allowing use of standardised message and address tapes for despatching forecasts to all radio stations, T.V. stations, and local authorities.

Three of the rainfall stations have been fitted with VHF automatic radio reporting rain gauges (Langford 1965 (6)) which transmit the accumulative rainfall total continuously every three hours via a repeater to a permanent Meteorological Office at Coffs Harbour.

The only other special equipment installed at the present time are remote indicating rain gauges which allow observers to take the rainfall readings from a digital counter in the house. A radio-telemetering river height station is being installed at Bellbrook, 50 miles above Kempsey. This station will be linked into the rainfall telemeter network.

Experience has shown that it is possible for the Bureau to receive, analyse the data, prepare and despatch the forecasts, within 1½ hours from the time of observation. To date no serious dislocation of the service has occurred due to either a breakdown in communications or observer failure.

9.43. Procedure

Speed without sacrificing accuracy is the essence of good forecasting. The daily 9:00 a.m. rainfall and streamflow observations are entered on pre-prepared forms and the gross average rainfall computed by taking the arithmetic means of the reports. (In some catchments a multiple correlation analysis of isohyetal average rainfall and station totals has provided partial regression coefficients and the sum of the weighted rainfall observations is used).

The current API is calculated and the corresponding value of initial loss which applies for the next 24 hours is entered on a summary sheet. A preliminary check is made before noon, then at 3.00 p.m. each day the duty weather forecaster determines the likelihood of rainfall occurring in the next 24 hours. If this estimate exceeds the initial loss, a precautionary phase is entered. The flood forecasting staff are organised and as the situation develops, the rainfall and streamflow observers are put on a three hour reporting schedule.

All reports are entered on appropriate forms and average accumulated rainfall total calculated until the initial loss is satisfied. Following increments of gross rainfall are reduced by the constant loss (10 pts per three hours in the Macleay) and the excess rainfall values are multiplied by the average 3 hour unitgraph ordinates (listed in Appendix A) to define the peak of the surface runoff hydrograph. Adjustment is made for baseflow by adding the discharge prior to the rise.

The discharge is converted to stage and checked with the observed river behaviour and the prediction based on flood routing from the upstream station. A forecast is then issued to the public.

9.5 Weather Forecasts

The normal weather forecasting service operated by the Bureau plays a vital role in the flood forecasting operation. As a matter of routine the Bureau collects meteorological observations from a permanent network of stations throughout the Commonwealth every three hours except midnight, and as far as possible each flood forecasting system is arranged to fit in with the existing organisation.

The continuous watch kept on the conditions of the catchment area and the meteorological situation allows the meteorologist to set the forecasting system in operation in ample time. Quantitative rainfall trend forecasts provide a basis for assessing the longer period development in more general terms.

9.6 The Results of Operational Flood Forecasting

Many critics of forecasting systems object to results based on paper operation which may be influenced by a degree of hindsight.

To overcome these objections, as far as practicable the data and results achieved using the Macleay Flood Forecasting System are set out in some detail in Appendix B to this paper.

Perfection is not claimed for this procedure, nor is it claimed to be the best that can be achieved; it is merely a step in the right direction. In some of the early floods regular 3 hour forecasts were not issued to the public but the results from a routine application of the procedure are shown.

9.7. Accuracy of Predicted Volume of Direct Runoff

For the eight floods experienced since 1962, the observed and predicted volumes of direct runoff for the site of the town of Kempsey in the Lower Macleay Valley are given in Table 1.

TABLE 1.

Comparison of Predicted and Observed Total Volume of Direct Runoff Expressed in Inches

Storm No.	Direct Runoff - Inches		Percentage Error
	Observed	Predicted	
30	0.82	0.82	0
31	3.07	3.19	+ 4
32	2.08	1.88	- 10
33	0.80	1.63	+ 104
34	2.06	2.29	+ 11
35	6.46	6.41	0
36	0.64	0.86	+ 34
37	1.72	1.80	+ 5

The two major errors (storms 33 & 36) occurred in January 1963 and January 1964, from similar situations when short intense storms on the Tableland region produced nuisance floods at Kempsey approximately 30 hours after the rainfall ceased. This type of flood can be accurately predicted by flood routing from Bellbrook with about 12 hours warning. The failure in both cases can be attributed to the unsuitability of the method for determining losses when only a portion of the catchment contributes to runoff. The Tableland and Coastal segments of the catchment are not hydrologically homogenous and there is little doubt that future subdivision of the catchment could minimise this error.

There is also a tendency for the API - Initial Loss relationship to underpredict the loss from rainfall following a prolonged dry period, and although it is difficult to attribute the inadequacy to any one factor, there is some evidence to suggest the depletion in groundwater storage during dry periods may cause an increase in the amount of loss to this portion of the moisture store within the catchment. The API (which is primarily indicative of surface conditions) does not adequately define the sub-surface storage conditions. An attempt has been made to correct the API following such conditions using the base flow as a parameter of groundwater storage - Body 1963 (6).

From the point of view of gate operation of a dam, as distinct from predicting flood heights and times at Kempsey due to natural river flows, errors in estimating flood hydrographs caused by short intense storms in one zone only of the catchment, as in the case of storms 33 and 36, are relatively unimportant, as the total flood volume is so small.

9.8. Accuracy of Predicted Flood Discharge Hydrograph

The estimated volume of direct runoff (excess rainfall) is distributed in time and converted to discharge at Kempsey by using a single average three hour unitgraph. Although the final predictions of peak discharge (Table 2) are reasonable, both in magnitude and time of occurrence, the average unitgraph does not cater for the variations in the shape of the rising limb due to departures of the spatial rainfall distribution from the normal pattern.

A major error occurred in storm 34 when unusually heavy rain fell on the coastal segment of the catchment with very little contribution from the remainder of the catchment. No similar event had been recorded and the unitgraph and flood routing procedures failed, although in fact the warnings for all practical purposes were adequate at the time.

TABLE 2.

Comparison of Observed and Predicted Peak Discharges

Storm No.	Final Peak Discharges - cusecs Observed	Predicted	Error in Time of Peak Hours
30	77,000	64,000	6 hrs. early
31	149,000	146,000	3 " "
32	127,700	113,000	3 " late
33	75,900	89,000 (routing)	3 " early
34	109,000 (2nd peak)	121,000	15 " late
35	349,000	290,000	Correct
36	63,500	55,000	3 hrs. early
37	94,600	110,000	Correct

Again it is considered subdivision of the catchment will enable this type of error to be largely eliminated or at least minimised.

9.9. Effect of Storm Characteristics

9.91 Storm Duration

Whilst the rainfall continues, progressive forecasts can be issued at the end of each observational period, but a problem arises as the time lag between the end of excess rainfall and peak of the hydrograph diminishes.

In the case of a short intense storm (e.g. January 1962) up to 30 hrs. warning of the peak is possible compared with only nine hours in April 1962, when the peak occurred nine hours after the end of effective rainfall.

9.92. Effect of Rainfall Intensity and Storm Magnitude

The use of the three hour unitgraph should, to a large degree, allow for the variation in rainfall intensity and magnitude. In the initial analysis of the hydrologic data for the Macleay Catchment, a correction was made for the tendency for the average unitgraph to over predict small floods and under predict larger floods. A reanalysis of the data including recent events has led to the elimination of this step in the procedure.

9.93. Effect of Non Uniform Areal Rainfall Distribution

This factor has already been mentioned and is considered to be one of the main sources of error. This can be reduced by additional subdivision of the catchment and the instrumentation programme implemented by the Bureau will allow such a refinement in the future.

9.10. Applicability of Macleay Valley Flood Forecasting to Gate Operation of a Dam

The experience gained through flood forecasting for the Macleay River seems to indicate that the forecasts for the smaller floods are more liable to error than moderate or major events. This is to be expected as the minor floods are generally associated with heavy rainfall over portion of the catchment and are more liable to errors due to non uniform rainfall distribution. Major floods are associated with major storm mechanisms such as the tropical cyclone or the rain depression and the rainfall is not only widely distributed but follows well defined patterns.

If a dam existed in the Macleay Valley above Kempsey, there is little likelihood of flood damage arising from the minor floods such as Nos. 30, 33 and 36 of Table 1, and sophisticated forecasting with long forecast intervals would hardly be required for gate operation for such floods. In fact, much of such small floods would be trapped in a conservation dam, no matter what operation procedure was followed.

With this in mind, floods Nos. 32 and 35 have been selected for comparing various methods of gate operation in Section 10 below.

10. TESTING AND COMPARISON OF VARIOUS GATE OPERATION PROCEDURES

10.1 Details of Dam and River Systems

It is assumed that a dam with storage-elevation characteristics as shown in Figure 14 is located at the site of the "Big River" Dam shown in Figure 15. It will be noted that there is a major tributary between the dam and Smith City. This is a typical situation for many dams in Australian river basins. The catchment area is 4,000 sq. miles, and the design flood is shown in Figure 16 with a peak hydrograph discharge under natural conditions at the dam site of 500,000 cusecs. The corresponding "inflow to full pool" maximum hydrograph in this Figure has a peak flow of 540,000 cusecs.

10.2 Fixing the Maximum Permissible Induced Surcharge Curve

The basic essentials of this curve are:-

(i) It must allow the design flood to be passed safely over the spillway.

(ii) It must be compatible with the geometry of the gates as previously demonstrated in Figure 4, i.e. the storage must not spill over the top of the gates.

(iii) Relatively flat sections of the curve are undesirable, as these cause excessively rapid rates of increase of outflow during major floods.

The permissible surcharge level increases with increase in gate release, because more of the future inflow will be removed from surcharge storage at the higher release rate. If, for a given gate opening, the permissible surcharge level is reached, the outflow is increased and a higher maximum permissible surcharge level becomes applicable. This procedure is continued, keeping the level at or below the maximum permissible, until all gates are fully opened, after which the outflow is uncontrolled.

The curve is decided upon by routing the design flood through the dam, assuming that the dam is at static full pool level when the design flood occurs, that no early releases of water are made to provide space for predicted future inflow, and that the gates are operated so that the storage-outflow relationship follows the maximum permissible induced surcharge curve from the beginning of inflow until all gates are fully opened.

This is, of course, unrealistic but represents the worst possible conditions that could exist, and ensures that the curve selected will safeguard the dam.

Four alternative maximum induced surcharge curves are shown in Figure 17 together with the storage level- outflow curve when all gates are fully opened. Curve A represents the limiting surcharge as controlled by the geometry of the gates only. This curve is the height of the top of the gates for the gate opening which gives the corresponding outflow. If the water level rose any higher, flow would occur over the top of the gates. Curve B is similar up to R.L.406, but requires all gates open at R.L.407. Curves C and D are of relatively constant slope to avoid the flat sections of A and B.

Figure 18 shows that Curves A and B produce lower rates of outflow than C and D in the early part of the flood but this is offset by the higher rates in the later part of the flood and particularly by the higher peak outflows. A and B are also unfavourable from the aspect of rate of change of outflow, the very steep rises being due to the flat sections of the maximum induced surcharge curves.

In comparing the outflows from C and D it is seen that C gives better flood control up to 270 thousand cusecs but for major floods exceeding this, and particularly for the design flood, D is superior. A maximum induced surcharge curve lower than D would evidently result in an even lower design peak outflow but it would also cause some outflows to exceed those of the natural hydrograph and would therefore generally be undesirable.

Curve D has therefore been adopted in this example as the most suitable maximum induced surcharge curve to be used in the several possible methods of gate operation.

If method (b) of Section 7 above is used with no outflow until static full pool level is reached, this is the limiting case of the maximum induced surcharge curve, i.e. the use of no induced surcharge at all. In Fig. 17 this "inflow equals outflow" method would mean that reservoir level R.L.400 would be maintained until the "all gates open" stage is reached at 187,000 cusecs.

10.3 Floods Used for Comparison

The objective of this paper is to test and compare the relative merits of various gate operations procedures under "real life" conditions with a hypothetical dam in a river similar to the Macleay River, N.S.W. Storms numbered 30 to 37 in Appendix B are "real life" storms on the Macleay River for which actual flood forecasts are available. Storm No. 35 provides the greatest volume of run-off, and could be regarded as a typical "major flood". Minor storms, such as Numbers 30, 36 and 37 would cause little damage, and might even fail to bring the reservoir to static full pool level. Storms 31, 32 and 34 are typical "medium" floods, with No. 32 giving less than half the run-off of No. 35

For this paper, Storm No. 32 has been selected to test a typical medium flood, causing moderate damage, and No. 35 is used as an example of a typical major flood.

The effect of gate operation on flood mitigation depends upon the water level existing in the reservoir at the onset of the flood. For a conservation dam, it would be only on rare occasions that the reservoir would be at full pool level when the flood commences.

In this study it was assumed that prior to the major flood (No. 35) the level was 10 ft. below static full pool. For the medium flood (No. 32) with this level, it was found that so much of the flood volume would be retained in the reservoir that the results of different procedures could not be demonstrated, so that for this case it was assumed that the initial level was only 3 ft. below the top.

The Bureau of Meteorology supplied the authors with the predicted peak discharge at Kempsey and the corresponding time of peak as made by the Bureau every three hours during the actual storms. The observed discharges at the time of forecast were also supplied and it was known from previous hydrograph analyses that the steepest hourly recession constant was 0.85 (see Section 10.5 for further discussion of the recession constant). This enabled the authors to synthesize forecast flood hydrographs at 3 hourly intervals for Kempsey which was assumed to be at the hypothetical dam site. These forecast "natural hydrographs at dam site" were modified to give corresponding forecast "inflow hydrographs to full pool", which had peaks approximately 10% higher and 3 hours earlier, allowing for the storage and attenuation effects of the channel as discussed in section 2.

Figure 19 shows the observed natural flows at the dam site for storm No. 32, the resultant inflow to full pool hydrograph of the hypothetical dam, and at 3 hourly intervals the forecast inflow hydrographs calculated from the Bureau's forecasts as above. Figure 20 gives similar data for storm No. 35.

The forecast inflow hydrographs of figures 19 and 20 are essentially the same as those that could have been obtained directly from the predicted excess rainfall by applying an "inflow unitgraph" derived from past Kempsey hydrographs which had been modified to allow for storage and attenuation. The peak of this inflow unitgraph would be about 72,000 cusecs and its lag 27 hours, compared with a peak of 66,000 cusecs and a lag of 30 hours for the Kempsey unitgraph, as given in Appendix A.

10.4 Gate Operation Based on Reservoir Levels Only, Using Induced Surcharge

This method of gate operation is governed by a single storage level-outflow curve, and the best results are obtained by using the maximum induced surcharge curve. In deciding which curve to use, the considerations discussed in Section 10.2 and Figure 17 govern, and curve D of Figure 17 has been adopted in this example. Figure 21 shows the results with the two sample floods of Figures 19 and 20.

In Figure 21 it will be noted that outflows do not commence until the reservoir is at static full pool level and then the hydrograph appears as a series of small steps rather than a smooth curve. This is caused by the gates being opened at discrete hourly intervals rather than being opened gradually and continuously. The latter conditions are preferable but difficult to achieve in practice.

For the second (major) flood the hydrograph is smooth after 50 hours from the start of excess rainfall. At this point the reservoir level has reached the stage when all gates must be fully open and the subsequent flows at higher levels are virtually uncontrolled.

The calculations to determine the outflows of Figure 21 are shown in Tables 3 and 4 and are self explanatory.

TABLE 3

"Moderate Flood" Operations based on Reservoir Level Only Using Induced Surcharge

A Time	B Storage Level	C Change in Level	D Inflow (Volume)	E Out flow (from Curve D Fig. 17)	F Nett Inflow (D - E)
hours	ft.	ft.	th. cfs. -hrs	th. cfs. - hrs.	th. cfs. - hrs.
0	397.0				
		.25	50	0	50
10	397.25				
		2.45	490	0	490
20	399.70				
		.36	73	0	73
21	400.06				
		.31	74	10	64
22	400.37				
		.28	77	20	57
23	400.65				
		.22	84	40	44
24	400.87				
		.21	93	50	43
25	401.08				
		.22	105	60	45
26	401.30				
		.25	117	65	52
27	401.55				
		.28	127	70	57
28	401.83				
		.28	136	80	56
29	402.11				
		.24	138	90	48
30	402.35				
		.21	138	95	43
31	402.54				
		.16	137	105	32
32	402.70				
		.16	136	105	31
33	402.86				
		.08	132	115	17

TABLE 3 (Contd.)

A Time	B Storage Level	C Change in Level	D Inflow (Volume)	E Outflow (from Curve D Fig. 17)	F Nett Inflow (D - E)
hours	ft.	ft.	th. cfs. -hrs.	th. cfs. -hrs.	th. cfs. -hrs.
34	402.94				
		.06	128	115	13
35	403.10				
		.03	126	120	6
36	403.13				
		.01	122	120	2
37	403.14				
		-.01	118	120	-2
38	403.13				
		-.02	116	120	-4
39	403.11				
		-.03	113	120	-7
40	403.08				
		-.05	109	120	-11
41	403.03				
		-.05	104	115	-11
42	402.98				
		-.06	102	115	-13
43	402.92				
		-.08	98	115	-17
44	402.84				
		-.10	95	115	-20
45	402.74				
		-.09	92	110	-18
46	402.65				
		-.08	88	105	-17
47	402.57				
		-.10	85	105	-20
48	402.47				
		-.09	82	100	-18
49	402.38				
		-.08	79	95	-16
50	402.30				
		-.09	77	95	-18
51	402.21				
		-.08	74	90	-16
52	402.13				
		-.09	71	90	-19
53	402.04				
		-.08	69	85	-16
54	401.96				

TABLE 4

"Major Flood" Operations Based on Reservoir Level Only Using Induced Surcharge

A Time	B Storage Level	C Change in Level	D Inflow (Volume)	E Outflow (Volume) From Fig.17	F Nett Inflow
hours	ft.	ft.	th.cfs.-hrs	th.cfs.-hrs.	th.cfs.-hrs.
0	R.L.390.00				
		.29	60	0	60
10	390.29				
		1.96	400	0	400
20	392.25				
		4.65	950	0	950
30	396.90				
		2.50	505	0	505
34	399.40				
		0.67	137	0	137
35	400.07				
		0.60	142	20	122
36	400.67				
		0.54	150	40	110
37	401.21				
		0.47	155	60	95
38	401.68				
		0.78	340	180	160
40	402.46				
		0.83	390	220	170
42	403.29				
		0.88	460	280	180
44	404.17				
		0.98	540	340	200
46	405.15				
		1.08	620	400	220
48	406.23				
		1.22	710	460	250
*50	407.45				
		1.17	750	510	240

* Gates fully open

TABLE 4 (Contd.)

A Time	B Storage Level	C Change in Level	D Inflow Volume	E Outflow Volume From Fig.17	F Nett Inflow
hours	ft.	ft.	th. cfs-hrs.	th. cfs-hrs.	th. cfs-hrs.
52	408.62				
		1.12	740	510	230
54	409.74				
		0.69	700	560	140
56	410.43				
		0.29	660	600	60
58	410.72				
		0.07	615	606	11
60	410.79				
		-0 .10	580	600	-20
62	410.69				
		-0.20	540	580	-40
64	410.49				
		-0.39	500	580	-80
66	410.10				
		-0.29	480	540	-60
68	409.81				
		-0.39	450	530	-80
70	409.42				
		-0.49	430	530	-100
72	408.93				

10.5 Gate Operations Based on Reservoir Levels and Average Inflow Rate for the Preceding Time Interval

The principles of the method used have already been discussed in Section 8.1 above for the preparation of an emergency release chart for use when communications fail.

The Hathaway - Corps of Engineers method assumes that point B of Figure 10 is the commencement of the recession limb of the hydrograph, and recommends the use of the steepest recession curve, and in the computations for this paper this method has been followed, and an hourly recession constant of 0.85 has been adopted for the critical inflow hydrograph. This means that for the critical inflow hydrograph of Big River Dam:

$$q_{t+1} = 0.85\, q_t$$

where q_t = discharge on recession limb of hydrograph at t hours.

q_{t+1} = discharge on recession limb of hydrograph at t + 1 hours.

This is obtained by examining the recorded hydrographs and selecting the steepest recession limb observed during major floods. If hydrographs at the dam site are the only ones available it is necessary to estimate the corresponding inflow hydrographs from these by making suitable allowances for translation and storage in the natural valley. Some of the routing methods described in the standard textbooks are useful for this purpose (e.g. the Muskingum method).

The recession constant and maximum induced surcharge curve enable the preparation of storage level-inflow-outflow relationships described by U.S. Corps of Engineers 1959 (2). The chart for use by the dam operator computed by this method for Big River Dam is shown in Figure 22. In this figure, rise in level is plotted against current storage, with outflow as a parameter, whereas Hathaway plotted outflow against storage to provide the same information.

In some cases it may not be possible to express the inflow recession limb in terms of a single recession constant and the data for the preparation of schedules such as Figure 22 are then obtained by direct measurement from plots similar to Figure 10.

The non-damaging phase of Figure 22 is designed to release as much water as possible at non-damaging rates as soon as the inflow indicates that a full dam is assured. The schedule also provides for these releases to be curtailed when the guaranteed inflow volume is no longer in excess of the volume required to fill the dam. This requires consideration of the maximum permissible rate of change of outflow, which has been assumed to be 40,000 cusecs per hour in the case of the hypothetical Big River Dam.

To carry out the operation of the gates by this method consideration must be given to the maximum non-damaging outflow. Obviously this will vary from storm to storm, and during any given storm, depending on hydrologic conditions existing downstream. This method of gate operation is only justified if all communications are disrupted, and it is assumed that no downstream or upstream hydrologic data can be made available to the dam operator. Therefore it is assumed in this paper that a study has been made of past floods, which reveals that very rarely would released flows not exceeding 40,000 cusecs cause any damage downstream of the dam, so that a fixed limit of 40,000 cusecs for non-damaging releases has been assumed in the computations and in compiling Figure 22.

The recession phase of Figure 22 is in accordance with the principles outlined previously although there are limits to the possible outflows at some levels imposed by the outlet capacity with all gates open. These are indicated by the curve in the upper left-hand corner of the Figure which is designed to enable the outflow to be reduced at less than 40,000 cusecs per hour before the storage falls below full supply level.

The outflow hydrographs produced by this method of gate operations during the typical floods are given in Figure 23 and the corresponding calculations are set out in Tables 5 and 6.

The authors feel that the Corps of Engineers' procedure could be much improved. For example, the use of the steepest recession curve is too conservative in the non damaging phase and is quite wrong in the damaging phase, because then there is no doubt that the dam will be filled. The recession constant of 0.85 is too steep for most floods, as can be seen by comparing the forecast recessions of Figures 19 and 20 with the actual recessions for the same floods. The emergency release chart method in this study would have provided greater flood mitigation if the average recession constant (0.90) had been used instead of 0.85 for the set of curves in Figure 22 for the damaging phase. Such a procedure involves no increased risk at all if the procedure for the non damaging phase remains unchanged, because when the conditions require damaging releases there is no longer any doubt about ending up with a full dam. If this modification were made to the procedure the peak outflow of the major flood of Figure 23 would have been reduced by a further 5%.

TABLE 5

"Moderate Flood" Gate Operations Based on Reservoir Level and Inflows

A Time	B Storage Level	C Change in Level	D Inflow Volume	E Outflow Volume From Fig.22	F Nett Inflow (D - E)
hours	ft.	ft.	th.cfs-hrs.	th.cfs-hrs.	th.cfs-hrs.
0	RL 397.00				
		.25	50	0	50
10	397.25				
		.37	75	0	75
13	397.62				
		.17	39	0	39
14	397.79				
		.23	46	0	46
15	398.02				
		.16	52	20	32
16	398.18				
		.18	57	20	37
17	398.36				
		.21	63	20	43
18	398.57				
		.13	67	40	27
19	398.70				
		.25	70	20	50
20	398.95				
		.16	72	40	32
21	399.11				
		.17	74	40	34
22	399.28				
		.19	78	40	38
23	399.47				
		.22	84	40	44
24	399.69				
		.25	92	40	52
25	399.94				
		.30	102	40	62

TABLE 5 (Contd.)

A Time	B Storage Level	C Change in Level	D Inflow Volume	E Outflow Volume From Fig. 22	F Nett Inflow (D-E)
hours	ft.	ft.	th. cfs-hrs.	th. cfs-hrs.	th. cfs-hrs.
26	400.24				
		.32	124	60	64
27	400.56				
		.24	128	80	48
28	400.80				
		.27	135	80	55
29	401.07				
		.28	138	80	58
30	401.35				
		.28	138	80	58
31	401.63				
		.18	137	100	37
32	401.81				
		.17	134	100	34
33	401.98				
		.15	132	100	32
34	402.13				
		.13	127	100	27
35	402.26				
		.12	124	100	24
36	402.38				
		.10	120	100	20
37	402.48				
		.09	118	100	18
38	402.57				
		.05	115	105	10
39	402.62				
		.03	112	105	7
40	402.65				
		.01	108	105	3
41	402.66				
		0	105	105	0
42	402.66				
		-.01	102	105	-3

TABLE 5 (Contd.)

Time	Storage Level	Change in Level	Inflow Volume	Outflow Volume From Fig. 22	Nett Inflow (D-E)
hours	ft.	ft.	th. cfs-hrs.	th. cfs-hrs.	th. cfs-hrs.
43	402.65				
		-.03	98	105	- 7
44	402.62				
		-.05	94	105	-11
45	401.57				
		-.06	92	105	-13
46	401.51				
		-.08	88	105	-17
47	401.43				
		-.10	85	105	-20
48	401.33				
		-.11	82	105	-23
49	401.22				
		-.12	80	105	-25

TABLE 6

"Major Flood" Gate Operation Based on Reservoir Level and Inflow

A Time	B Storage Level	C Change in Level	D Inflow Volume	E Outflow Volume From Fig. 22	F Nett Inflow (D-E)
hours	ft.	ft.	th. cfs-hrs.	th. cfs-hrs.	th. cfs-hrs.
0	390.00				
		2.25	460	0	460
20	392.25				
		1.98	410	0	410
25	394.23				
		.47	97	0	97
26	394.70				
		.40	103	20	83
27	395.10				
		.43	107	20	87
28	395.53				
		.45	112	20	92
29	395.98				
		.28	117	40	57
30	396.26				
		.49	120	20	100
31	396.75				
		.40	122	40	82
32	397.15				
		.43	127	40	87
33	397.58				
		.45	132	40	92
34	398.03				
		.48	137	40	97
35	398.51				
		.50	142	40	102
36	399.01				
		.44	150	60	90
37	399.45				
		.46	155	60	95
38	399.91				
		.41	164	80	84

TABLE 6 (Contd.)

A Time	B Storage Level	C Change in Level	D Inflow Volume	E Outflow Volume From Fig. 22	F Nett Inflow (D-E)
hours	ft.	ft.	th. cfs. -hrs.	th. cfs. -hrs.	th. cfs.-hrs.
39	400.32				
		.47	177	80	97
40	400.79				
		.44	189	100	89
41	401.23				
		.50	203	100	103
42	401.73				
		.48	218	120	98
43	402.21				
		.47	236	140	96
44	402.68				
		.54	261	150	111
45	403.22				
		.54	281	170	111
46	403.76				
		.55	303	190	113
47	404.31				
		.56	325	210	115
*48	404.87				
		.57	342	225	117
49	405.44				
		.64	360	230	130
50	406.08				
		.65	372	238	134
51	406.73				
		.65	378	245	133
52	407.38				
		.61	375	250	125
53	407.99				
		.55	367	255	112
54	408.54				
		.47	357	260	97
55	409.01				
		.38	347	270	77

* Gates fully open, outflow uncontrolled

TABLE 6 (Contd.)

A Time	B Storage Level	C Change in Level	D Inflow Volume	E Outflow Volume From Fig. 22	F Nett Inflow (D-E)
hours	ft.	ft.	th. cfs. -hrs.	th. cfs. -hrs.	th. cfs. -hrs.
56	409.39				
		.29	335	275	60
57	409.68				
		.22	322	277	45
58	409.90				
		.16	312	280	32
59	410.06				
		.09	301	282	19
60	410.15				
		.04	291	282	9
61	410.19				
		0	281	282	-1
62	410.19				
		-.04	272	282	-10
63	410.15				
		-.09	263	282	-19
64	410.06				
		-.12	256	280	-24
65	409.94				
		-.15	247	277	-30
66	409.79				
		-.16	240	273	-33
67	409.63				
		-.18	235	272	-37
68	409.45				
		-.22	228	272	-44
69	409.23				
		-.23	222	270	-48
70	409.00				
		-.25	217	267	-50
71	408.75				
		-.26	210	263	-53
72	408.49				

10.6 Gate Operation Based on Flood Forecasting

It has already been argued that the gate operation cannot be carried out efficiently if the dam is viewed as an isolated entity within the river valley. As one moves to the more sophisticated procedures the dam becomes an integral part of the valley development and the dam operation must depend not only on forecasts of expected inflow to the storage, as shown in Figures 19 and 20, but also on forecasts of the flood hydrograph below the dam at key points of interest. Outflow from the dam should then be regulated as far as possible to fit in with the predicted downstream conditions.

In the typical examples of this paper, forecasts of downstream flood conditions are allowed for by Figure 24 which enables the estimation of the maximum non-damaging outflow discussed in section 6. As also mentioned in section 6, there are several other factors or constraints that may control gate operations based on flood forecasts and the estimation of these other factors or constraints is facilitated by Figures 25, 26 and 27. The complete calculations for determining the gate operations during the selected storms are shown in Tables 7 and 8 and the corresponding outflow hydrographs are plotted in Figure 29.

The objective of the calculations in Tables 7 and 8 is to determine the values of outflow that satisfy the constraints of columns (10) to (14) while corresponding as closely as possible to the maximum non-damaging outflows of column (9) (in the non-damaging phase) or the optimum flood control outflows of column (8) (in the damaging phase). It is advantageous to consider separately each item of columns (5) to (14).

Predicted Total Inflow Volume, column (5) (PTI), is obtained directly from the forecast either by converting the total excess rainfall to thousands of cusec hours and subtracting the previous inflow, or by measuring the area under the plotted forecast inflow hydrograph.

Predicted Net Inflow Volume, column (6), (PNI), may be obtained by measuring the area between the plotted forecast inflow hydrograph and a line representing the current outflow as shown by the shaded area in Figure 9.

Volume to Maximum Induced Surcharge Level, column (7), (VML) may be read from Figure 25 which was prepared from the Big River Dam storage-elevation curve (Fig. 14) and Maximum Induced Surcharge Curve (Fig.17).

Optimum Flood Control Outflow, column (8), (QO), is calculated from PNI (column 6) and VML (column 7) as explained in section 6.3 and Figure 9. It may be shown by geometrical means (see Fig. 32) that equation (1) gives a satisfactory approximation of the optimum flood control outflow in the case of the Big River Dam, i.e.

$$QO = Q + \frac{PNI - VML}{T + 6.7} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where QO = required optimum flood control outflow in cusecs.

Q = current outflow in cusecs

T = approximate time in hours during which the predicted inflow rate will exceed Q. This may be measured from the plotted forecast hydrograph.

PNI and VML are as defined above.

Maximum Non-Damaging Outflow, column (9), (QM), is read from Figure 24 which is a typical relationship derived from analyses of flood data on the Hawkesbury-Nepean river system. In the hypothetical example of Table 7 it is assumed that the gate operator is receiving hourly rainfall reports from Smith City and runoff data from Jones Crossing in order to read off the maximum non-damaging outflow from figure 24. These rainfall and runoff reports are not shown in Table 7 but the values selected by the authors are based on observed conditions in the Hawkesbury-Nepean Rivers and are quite realistic.

This constraint only applies during the non-damaging phase when QM exceeds both the natural flow at the dam site (QN) and the optimum flood control outflow (QO).

Maximum or Minimum Outflow by Rate of Change, column (10), (QU) or (QL). These maximum or minimum values are obtained from the current outflow by respectively adding or subtracting 40,000 cusecs which is the adopted maximum allowable rate of change of outflow, as discussed in section 1 and 6.2.

TABLE 7.

GATE OPERATIONS BASED ON FLOOD FORECASTING

MODERATE FLOOD

STORAGE BEHAVIOUR				COMPUTED OUTFLOWS					CONSTRAINTS ON COMPUTED OUTFLOWS						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
				PTI	PNI	VML	QO	QM	QU or QL	QF	QN	QR	QS	Q	
Time from Start of Excess Rain	Storage Level	Change in Storage Volume	Average Inflow for Period	Predicted Total Inflow Volume	Predicted Nett Inflow Volume	Volume to Max. Ind. Sur. Level	Optimum Flood Control Outflow	Maximum Non-Damaging Outflow	Max. or Min. Outflow by rate of Change	Maximum Outflow for Full Dam	Natural Flow at Damsite	Maximum Recession Outflow	Minimum Outflow for Dam Safety	Actual Outflow Released	Factors Controlling Outflow
	Observed	Fig. 14	(3)+(15)	Forecast	Forecast	Fig. 25	+ equation I	Fig. 24	(15) ∓ 40	Fig. 26	Calc. fm (4)	Fig. 27	Fig. 27	See Fig. 28	
hours	feet	thousand cusec hours	thousand cusecs	thousand cusec hours	thousand cusec hours	thousand cusec hours	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	
0	397·00														
6	397·05	+ 3	** 0·5	1540	1540	600	20	65	40	260	1		0	0	Forecasts Commence
7	396·90	− 32	8	1532*	30*	750	0	40	80	230	4		0	40	QU
8	396·80	− 20	20	1512*	30*	780	0	40	80	230	10		0	40	QM
9	396·70	− 19	21	2560	410	810	25	40	80	>280	15		0	40	QM New Forecast
10	396·61	− 18	22	2538*	410*	850	23	40	80	>280	20		0	40	QM
11	396·56	− 10	30	2508*	410*	860	23	40	80	>280	21		0	40	QM
12	396·55	− 3	37	3600	580	870	30	65	80	>280	22		0	40	QM
13	396·45	− 21	44	Not Required Because Full Dam is Assured	580*	1000	52	65	105	Not Required Because Full Dam is Assured	29		0	65	QM
14	396·37	− 15	50		580*	1020	52	65	105		30		0	65	QM
15	396·32	− 10	55		630	1040	50	50	105		30		0	65	QM
16	396·37	+ 11	61		630*	950	40	50	90		43		0	50	QM
17	396·45	+ 16	66		630*	920	40	50	90		50		0	50	QM
18	396·54	+ 19	69		1150	900	60	50	90		60		0	50	QM Damaging Phase Commences
19	396·60	+ 13	73		950*	950	60	45	100		60		0	60	QN
20	396·67	+ 14	74		950*	950	60	45	100		65		0	60	QN
21	396·75	+ 17	77		980	900	64	50	100		70		0	60	QO
22	396·85	+ 20	84		910*	910	64	50	104		75		0	64	QO
23	396·99	+ 31	93		880*	880	64	50	104		79		0	64	QO
24	397·19	+ 41	105		830*	830	64	55	104		83		0	64	QO
25	397·45	+ 53	117		No	Further	Assumed Constant until Max. Ind. Surch. Level Reached	50	104		87		0	64	QO
26	397·76	+ 63	127		Forecasts			45	104		94		0	64	OO
27	398·11	+ 72	136					45	104		105		0	64	QO
28	398·47	+ 74	138					45	104		111		0	64	QO
29	398·83	+ 74	138					45	104		116		0	64	QO
30	399·49	+ 73	137					50	104		121		0	64	QO
31	399·84	+ 72	136					50	104		126		0	64	QO
32	400·17	+ 68	132					50	104		127		16	64	QO
33	400·48	+ 64	128					50	104		128		20	64	QO
34	400·79	+ 62	126					55	104		128		35	64	QO
35	401·06	+ 58	122					55	104		127		50	64	QO
36	401·33	+ 54	118					55	104		126		60	64	QO
37	401·58	+ 52	116					55	104		124		70	64	QO Maximum Induced Surcharge Level Reached
38	401·79	+ 43	113				Not Required Because QS Controls Outflow	55	110		122		75	70	QS
39	401·96	+ 34	109					60	115		120		80	75	QS
40	402·08	+ 24	104					60	120		118		83	80	QS
41	402·17	+ 19	102					60	123		116		85	83	QS
42	402·23	+ 13	98					60	125		114		87	85	QS
43	402·26	+ 7	95					60	128		112		88	88	QS
44	402·28	+ 4	92					65	128		110		88	88	QS
45	402·28	0	88					65	48		109	210	88	88	QS Recession Phase Commences
46	402·26	− 4	84					65	48		108	210	88	88	QS

* Estimated from previous forecast and storage behaviour assuming previous forecast correct.

\+ Equation (I) is $QO = Q + \frac{PNI - VML}{T + 6{\cdot}7}$ (see Section 10·6 and Figure 32)

** First value of inflow is given by $\frac{(3)+(15)}{6}$ because the time increment is 6 hours. All other values of inflow are given by (3)+(15)

TABLE 8.

GATE OPERATIONS BASED ON FLOOD FORECASTING

MAJOR FLOOD

STORAGE BEHAVIOUR					COMPUTED OUTFLOWS				CONSTRAINTS ON COMPUTED OUTFLOWS							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	
				PTI	PNI	VML	QO	QM	QU or QL	QF	QN	QR	QS	Q		
Time from Start of Excess Rain	Storage Level	Change in Storage Volume	Average Inflow for Period	Predicted Total Inflow Volume	Predicted Nett Inflow Volume	Volume to Max. Ind. Sur. Level	Optimum Flood Control Outflow	Maximum Non-Damaging Outflow	Max. or Min. Outflow by rate of Change	Maximum Outflow for Full Dam	Natural Flow at Damsite	Maximum Recession Outflow	Minimum Outflow for Dam Safety	Actual Outflow Released	Factors Controlling Outflow	
	Observed	Fig. 14	(3)+(15)	Forecast	Forecast	Fig. 25	† equation 1	Fig. 24	(15) ∓ 40	Fig. 26	Calc. fm. (4)	Fig. 27	Fig. 27	See Fig. 28		
hours	feet	thousand cusec hours	thousand cusecs	thousand cusec hours	thousand cusec hours	thousand cusec hours	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs	thousand cusecs		
0	390·00	+179	** 13											0		
14	390·88	− 2	38	2150	2150	1820	6	55	40	120	13		0	40	QU	Forecasts Commence
15	390·87	− 2	43	2112*	160*	2000	-	45	80	110	30		0	45	QM	
16	390·86	+ 2	47	2069*	160*	2000	-	45	85	110	37		0	45	QM	
17	390·87	− 12	53	2720	1400	2000	-	65	85	250	40		0	65	QM	
18	390·81	− 4	61	2667*	600*	2120	-	65	105	240	45		0	65	QM	
19	390·79	0	65	2606*	600*	2120	-	65	105	220	49		0	65	QM	
20	390·79	+ 4	69	3600	1700	2120	-	65	105	>280	54		0	65	QM	
21	390·81	+ 10	75	3531*	1700*	2120	-	65	105	>280	59		0	65	QM	
22	390·86	+ 16	81	3456*	1690*	2120	-	65	105	>280	63		0	65	QM	
23	390·94	+ 20	85	Not Required Because Full Dam is Assured	1700	2110	-	65	105	Not Required Because Full Dam is Assured	67		0	65	QM	
24	391·04	+ 26	91		1680*	2100	-	65	105		72		0	65	QM	
25	391·17	+ 38	98		1654*	2060	-	60	105		79		0	60	QM	
26	391·36	+ 42	104		2050	1980	62	55	100		85		0	62	QO	Damaging Phase Commences
27	391·57	+ 42	104		1950*	1950	62	45	102		91		0	62	QO	
28	391·78	+ 48	110		1930*	1930	62	45	102		98		0	62	QO	
29	392·02	+42	116		2300	1870	74	50	102		101		0	74	QO	
30	392·23	+44	118		1920*	1920	74	55	114		104		0	74	QO	
31	392·45	+ 46	120		1900*	1900	74	60	114		108		0	74	QO	
32	392·68	+ 14	126		3600	1800	122	60	114		112		0	112	ON	
33	392·75	+ 16	132		2030*	2030	119	60	152		116		0	116	ON	
34	392·83	+20	139		2020*	2020	119	65	156		120		0	119	QO	
35	392·93	+ 18	142		2200	2030	124	65	159		125		0	124	QO	
36	393·02	+26	150		2020*	2020	124	65	164		132		0	124	QO	
37	393·15	+30	154		2010*	2010	124	65	164		137		0	124	QO	
38	393·30	+34	163		2150	1990	129	65	164		143		0	129	QO	
39	393·47	+46	175		2000*	2000	129	65	169		149		0	129	QO	
40	393·70	+58	187		1960*	1960	129	65	169		156		0	129	QO	
41	393·99	+58	201		2350	1890	143	65	169		163		0	143	QO	
42	394·28	+74	217		1950*	1950	143	65	183		177		0	143	QO	
43	394·65	+90	233		1880*	1880	143	70	183		195		0	143	QO	
44	395·10	+100	258		2200	1760	158	70	183		206		0	158	QO	
45	395·60	+120	278		1800*	1800	158	70	198		222		0	158	QO	
46	396·20	+142	300		1670*	1670	158	70	198		241		0	158	QO	
47	396·91	+134	323		2360	1480	189	70	198		260		0	189	QO	
48	397·58	+150	339		1600*	1600	189	70	229		278		0	189	QO	
49	398·33	+168	357		1350*	1350	189	70	229		295		0	189	QO	
50	399·17	+144	369		2070	1180	225	70	229		310		0	225	QO	
51	399·89	+150	375		No Further Forecasts		Assumed Constant until Max. Ind. Surch. Level Reached	70	265		321		0	225	QO	
52	400·64	+146	371					70	265		330		35	225	QO	
53	401·37	+140	365					72	265		340		61	225	QO	
54	402·07	+130	355					72	265		349		87	225	QO	
55	402·72	+120	345					72	265		348		104	225	QO	
56	403·32	+108	333					72	265		348		127	225	QO	
57	403·86	+96	321					72	265		344		140	225	QO	
58	404·34	+ 86	311					72	265		339		158	225	QO	
59	404·77	+ 74	299					72	265		330		170	225	QO	
60	405·14	+ 64	289					74	265		320		185	225	QO	
61	405·46	+ 54	279					74	265		310		195	225	QO	
62	405·73	+ 46	271					74	265		295		198	225	QO	
63	405·96	+ 38	263					74	265		285		205	225	QO	
64	406·15	+30	255					74	265		275		209	225	QO	
65	406·30	+22	247					74	265		268		216	225	QO	
66	406·41	+ 14	240					74	265		260		222	225	QO	
67	406·48	+ 6	234				QS Controls Outflow	74	265		252		227	227	QS	Maximum Induced Surcharge Level Reached
68	406·51	− 2	228					75	267		245		230	230	QS	
69	406·50	− 8	222					75	270		238		230	230	QS	
70	406·46	− 12	218		0	−ve	< 230	75	190		232	235	225	230	QS	Recession Phase Commences
71	406·40	− 16	214		0	−ve	< 230	75	190		226	230	222	230	QR	
72	406·24				0	−ve	< 230	75	190		220	225	213	225	QR	
73																

* Estimated from previous forecast and storage behaviour assuming previous forecast correct

† Equation (1) is $QO = Q + \frac{PNI - VML}{T + 67}$ (see Section 10·6 and Figure 32)

** First value of inflow is given by $\frac{(3)+(15)}{14}$ because the time increment is 14 hours. All other values of inflow are given by (3)+(15)

Maximum Outflow for Full Dam, column (11), (QF) may be read from Figure 26 which takes the following factors into account:

(1) The volume of water required to raise the reservoir to full supply level at any particular storage level (calculated from Figure 14).

(2) The volumesof water released for various values of outflow during the adopted hourly time increments.

(3) The maximum allowable rate of decrease in outflow (40,000 cusecs per hour) so that any necessary reductions to ensure a full dam are not too rapid.

In general this constraint only applies in the non-damaging phase because a full dam is usually guaranteed when damaging releases are commenced.

Natural Flow at Dam Site, column (12), (QN), may be obtained by routing the calculated inflow values (column (4)) through the reservoir by a standard routing procedure such as the Puls method.

Maximum Recession Outflow, column (13), (QR) is given by Figure 27 which shows essentially the same curve as the upper left hand section of Figure 22, as explained in section 10.5. This constraint only applies during the recession phase when the objective is to bring the reservoir level back to full supply level as soon as possible unless the latest flood forecasts are such that a new higher damaging phase is indicated.

Minimum Outflow for Dam Safety, column (14), (QS), is also obtained from Figure 27 which shows the maximum induced surcharge curve D of Figure 17.

Under some special circumstances it may be impossible to satisfy all of the above constraints because their values could become conflicting; for example if the maximum outflow for dam safety were to exceed the maximum outflow by rate of change. It is therefore necessary to allocate priorities to the constraints so that the consequences of their violation may be minimized. The order of priorities depends to some extent on the phase of the flood as shown in Table 9 which was adopted for the examples in this paper. Table 9 guarantees the safety of the dam at all times and it also guarantees a full dam whenever releases are made during a flood.

TABLE 9.

	Non-Damaging Phase	Damaging Phase	Recession Phase
1st Priority	Safety of Dam (QS)	Safety of Dam (QS)	Safety of Dam (QS)
2nd Priority	Full Dam (QF)	Natural Flow not exceeded (QN)	Maximum Recession Outflow not exceeded (QR)
3rd Priority	Maximum Rate of Change not exceeded (QU)	Maximum Rate of Change not exceeded (QU)	Maximum Rate of Change not exceeded (QL)

The sequence of computations in columns (5) to (16) of Tables 7 and 8 is shown diagrammatically by Figure 28 which makes due allowance for the selected order of priorities. In this procedure an initial computation of the required outflow is made (equal to either QO or QM in columns (8) and (9)) and its value is then compared with the value of each constraint (columns (10) to (14)), commencing with the lowest priority and ending with the highest priority. When it fails to satisfy a constraint the computed value of outflow (QC) is changed so that it does satisfy the constraint and the final value (in column (15) is adopted for the gate operations. Column (16) shows the item that determines the final value of outflow at each time increment during the flood.

Columns (2) to (4) are concerned with the relatively simple calculations of storage behaviour and should not require detailed explanation.

10.7 Comparison of Results

The outflow hydrograph produced by the three methods of gate operation are compared in Figures 30 and 31. The superiority of the method based on flood forecasts is demonstrated from a flood mitigation viewpoint, as well as the dam operation viewpoint when the condition of dam safety and a full dam are dominant. This superiority depends on the 'pre-release' of appreciable volumes of water at non-damaging rates after the forecasts have indicated a full dam is assured from future inflows.

The method of gate operation based on reservoir levels and average inflow rate for the preceding time interval may be regarded as a useful emergency procedure when forecasts are not available. To apply this method a simple schedule as shown in Figure 22 is all that is required together with regular readings of the storage level.

Methods of gate operation based entirely on the current storage level would be difficult to justify under most conditions, particularly when a major dam is concerned and the flood damage downstream from the dam is of some consequence.

10.8 Discussion

This paper has set out to compare various methods of gated dam operation, on the assumption that conditions (a) to (e) of Section 1 must be met. This implies that Hathaway's "Plan A" referred to in Section 4 must not be used, because it contravenes condition (c) of Section 1.

Plan A is still widely used in the design of dams in New South Wales in spite of the fact that the U.S. Corps of Engineers' standard procedures have discarded this method.

One possible reason for this non-acceptance of Hathaway's Plan B, involving induced surcharge, is that Plan A gives a lower peak outflow for the design flood than does Plan B, so that it leads to the design of a lower and cheaper dam. This is cold comfort to the community downstream of the dam. It might well reply that cheapening the annual capital costs of the dam and thus raising the average annual flood damage is unsound economics, from the point of view of the nation as a whole. The "design flood" is often the estimated "maximum possible flood", which some cynics may say is impossible anyway, and therefore the governing factor from the economic viewpoint should be the effect of the dams on the more common medium floods.

The authors would concede that in the design of dams the effects of Plan A as well as Plan B should be studied, provided that in both cases gate operation based on quantitative flood forecasting is taken for granted, and the assessment of average annual flood damage is also taken for granted as being an integral part of the design of the dam.

The main point is that if gated spillways are part of the design, a flood forecasting system must be provided by the constructing authority to control the operation of the gates. This applies whether Plan A or Plan B is decided upon.

If this is not done, the authority should design a dam with an ungated spillway.

10.9 Conclusions

In the past it has been the practice in Australia to leave to the constructing organisation the decision as to the method of gate operation to be used and such organisations tend to adopt the simplest possible method which ensures the safety of the structure. However, the decision made affects a wider sphere of the community than that represented by the construction authority. In fact the State Government has the ultimate responsibility in the matter. With the recent growth of specialised forecasting, improvement in the dependability of communication systems, and the advent of automation and computers, the time is now past when avoidable flood damage can be dismissed as an "act of God", when appropriate "acts of man" have been neglected. The lack of reliable forecasts in some valleys of Australia is not a valid reason for using primitive methods of gate operation. If an authority places gates on a spillway to reduce capital expenditure, it should devote some part of the saving to improving the forecasting system, particularly in the direction of extending the length of time in which effective action can be taken to release the maximum amount of water before the main flood arrives. In some situations the uncontrolled tributaries below the dam must become part of the forecasting and operational procedure, and the consequence of early releases on flashy local run-off must be studied.

Ideally the benefits and costs of downstream channel improvements and levee construction, permitting higher undamaging pre-releases, should be studied, although the water authority responsible for the dam might in some cases argue with justification that the cost of such work is a matter for the State Government rather than for the authority.

The degree of flood mitigation which can be achieved while still finishing with a full dam will vary with the forecast interval which can be achieved, and this depends on the topography and hydrology of the valley in question.

Clearly, however, an authority which builds a dam with a gated spillway has the following obligations:-

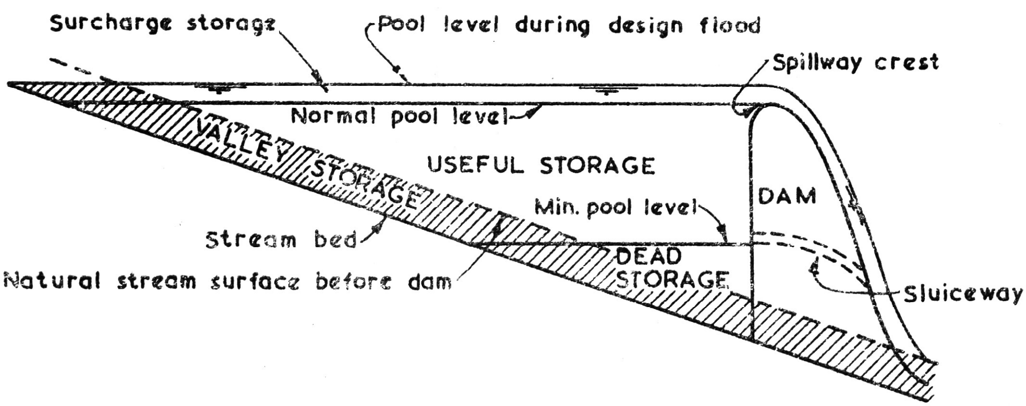
(i) To instal and develop a flood forecasting system.

(ii) To carry out a number of "dry runs" to test various methods of gate operation and choose the most efficient, with particular attention to the maximum degree of flood mitigation which can be achieved while still ensuring a full dam when the flood is over.

(iii) If the system is not fully developed by the time the dam is ready for service, to use, at least as a temporary measure, the technique of forecasting based on rate of rise of reservoir level.

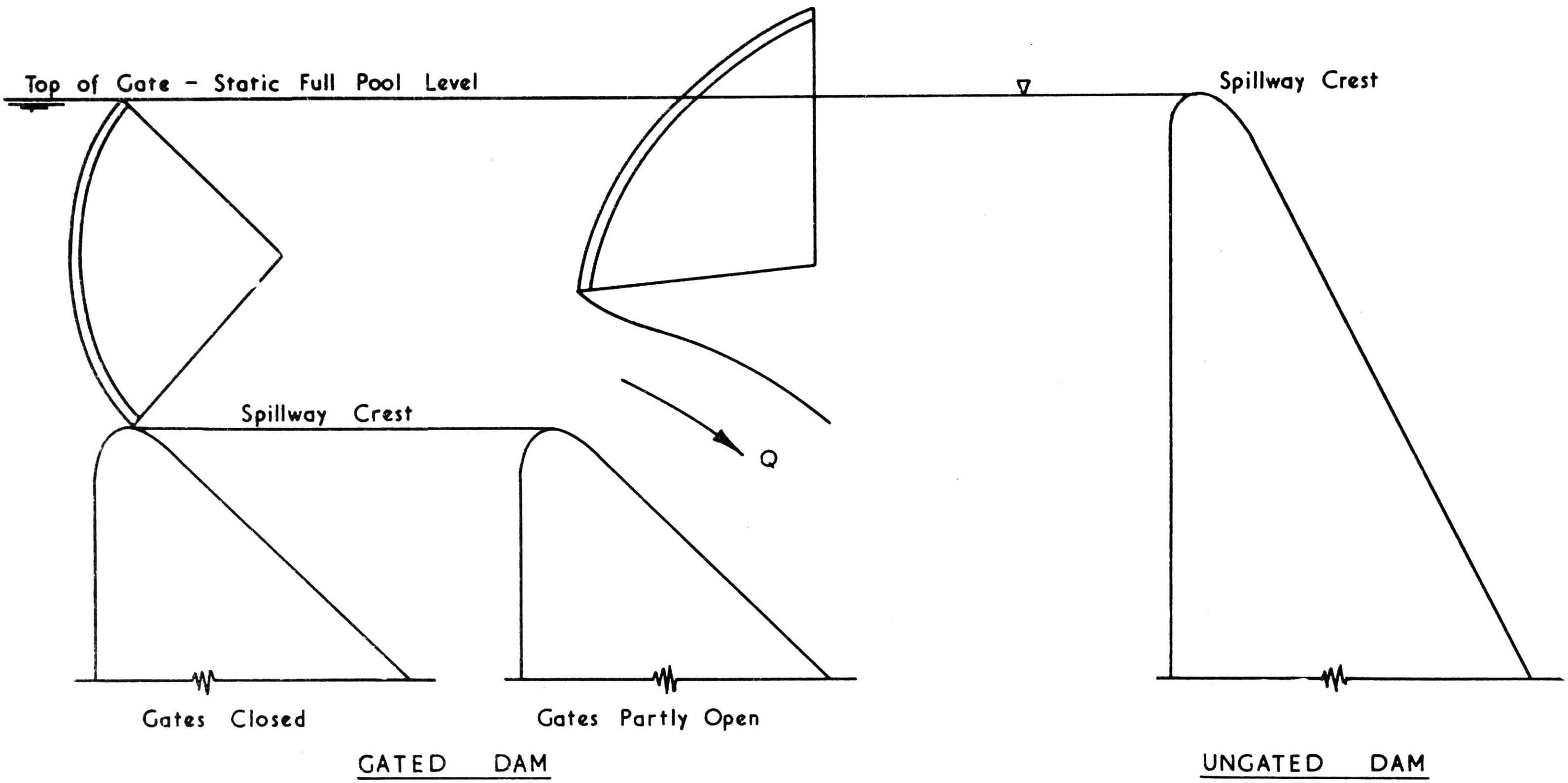
(iv) To draw the attention of the appropriate authorities to the beneficial effects on spillway gate operation which could result from channel improvement and levees downstream of the dam.

In regard to (i) above, the Hydrometeorological Branch of the Commonwealth Meteorological Service can be of considerable assistance.



(FROM "ELEMENTS OF HYDRAULIC ENGINEERING")
by Linsley and Franzini

FIGURE 1.



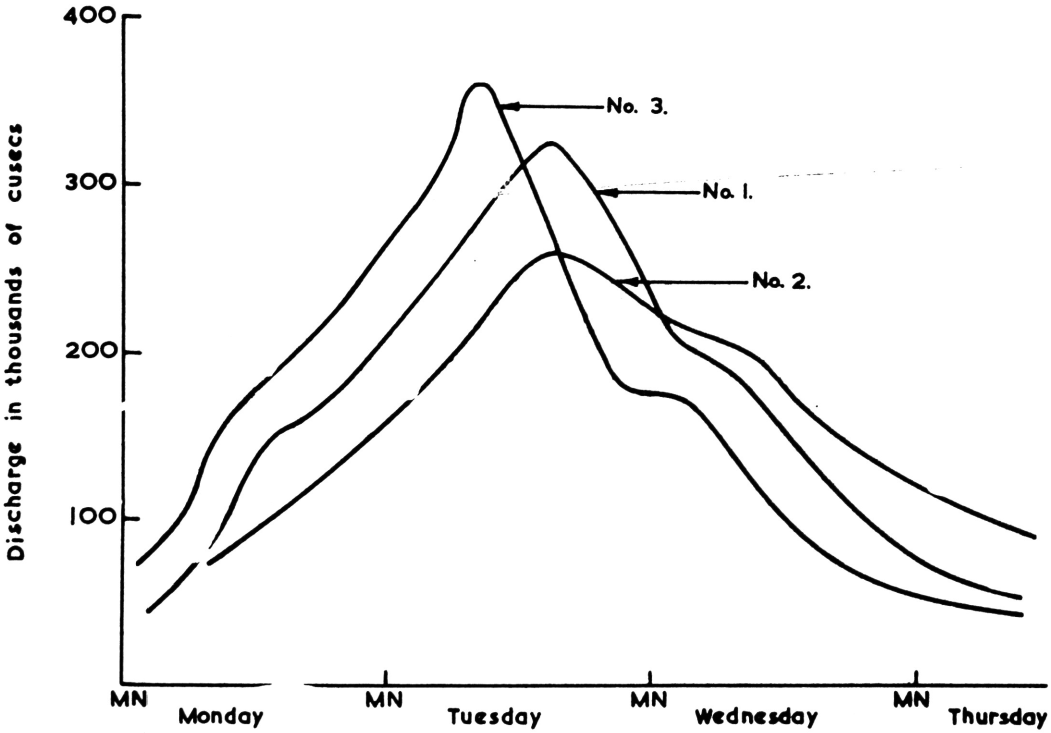
COMPARISON OF GATED AND UNGATED DAMS

FIGURE 2.

HYDROGRAPH No. I. Natural Flood at Dam site if Dam not Constructed

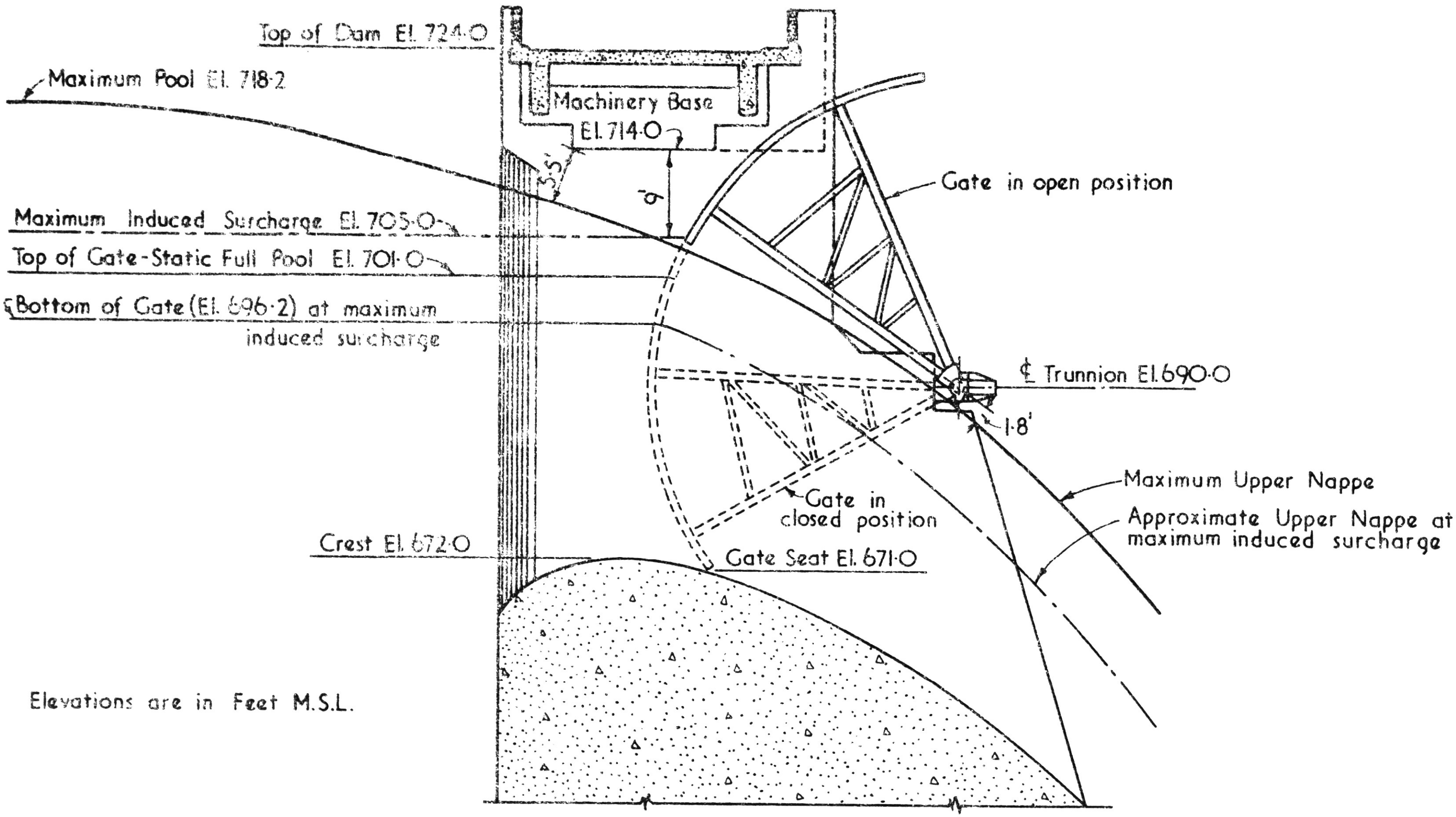
HYDROGRAPH No. 2. Actual Outflow from Dam Spillway

HYDROGRAPH No. 3. Inflow to Full Pool



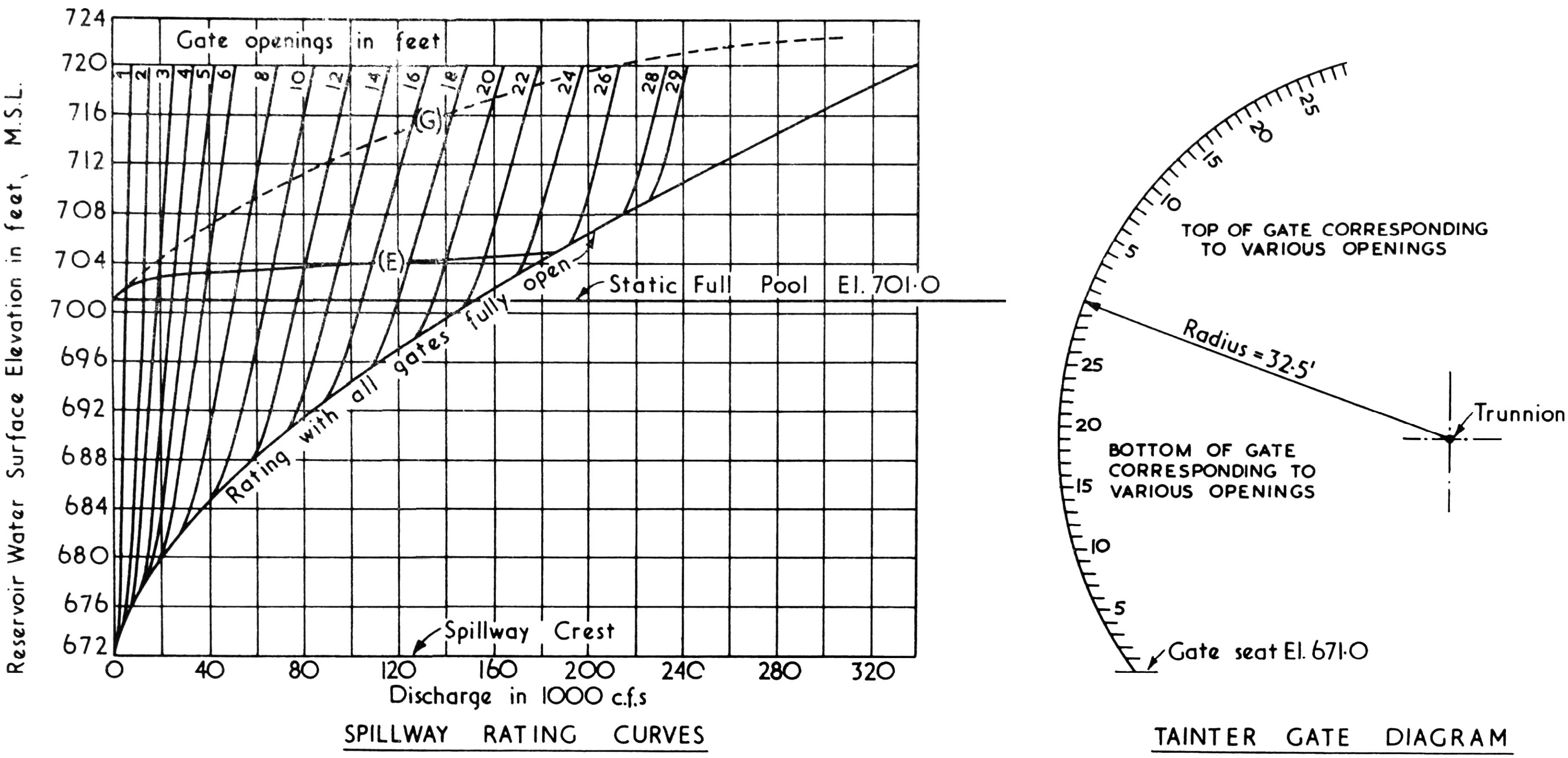
EFFECT OF UNGATED DAM ON FLOOD FLOW WITH DAM AT TOP WATER LEVEL BEFORE FLOOD

FIGURE 3.



Top of Dam El. 724·0
Maximum Pool El. 718·2
Machinery Base
El. 714·0
3·5'
9'
Gate in open position
Maximum Induced Surcharge El. 705·0
Top of Gate-Static Full Pool El. 701·0
℄ Bottom of Gate (El. 696·2) at maximum induced surcharge
℄ Trunnion El. 690·0
1·8'
Maximum Upper Nappe
Gate in closed position
Approximate Upper Nappe at maximum induced surcharge
Crest El. 672·0
Gate Seat El. 671·0
Elevations are in Feet M.S.L.

FIGURE 4.



NOTES: Spillway equipped with seven 40 ft. x 29 ft. tainter gates.
All gates are assumed to open uniformly.
Gate openings represent vertical distance of lower edge above gate seat.
Curves of partial gate openings were estimated from model tests of similar structures.
Curve (G) corresponds to top of the gates at various openings.
Curve (E) represents a limiting relation of gate opening and reservoir level.
When reservoir level corresponding to a given gate setting is below the (E) curve, outflow is regulated by the size of gate opening to not exceed natural flow conditions at the dam site.

FIGURE 5.

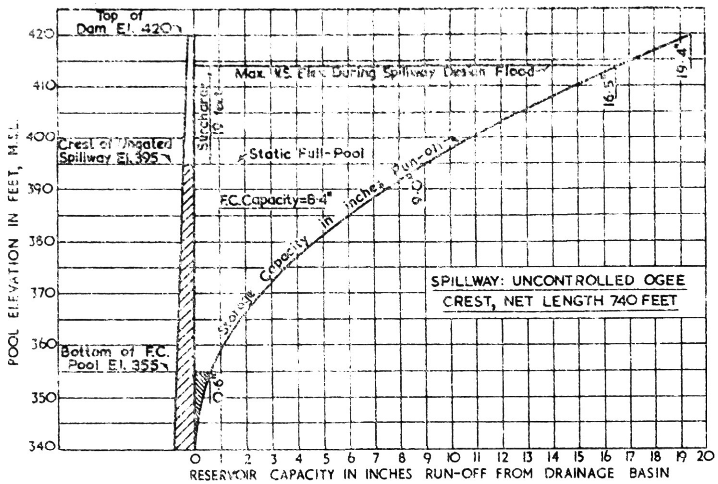


FIG. 6(a). FLOOD CONTROL RESERVOIR PLAN

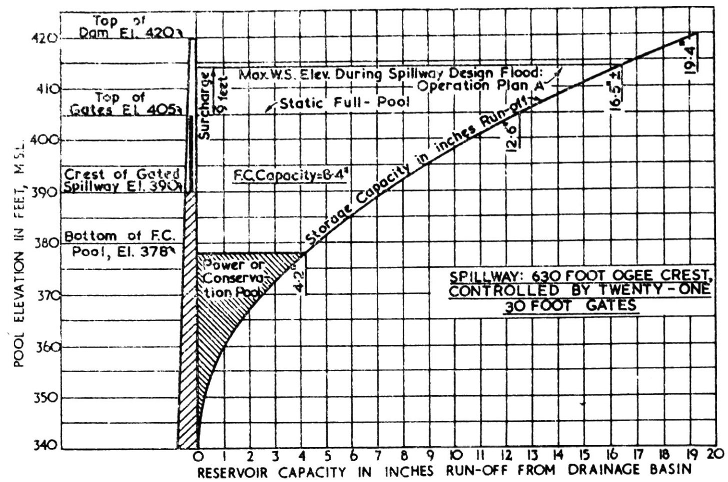


FIG. 6(b). DUAL-PURPOSE RESERVOIR PLAN

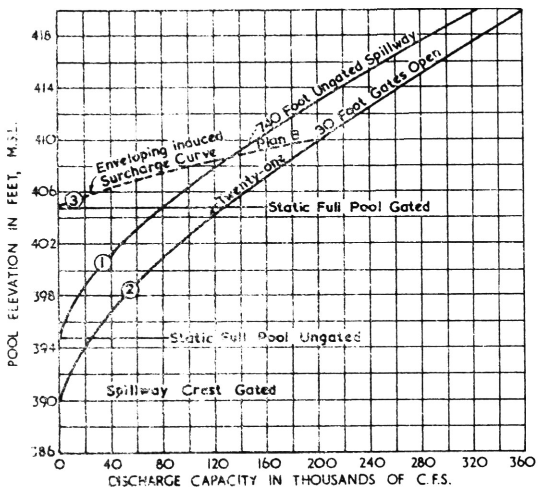


FIG. 6(c). SPILLWAY RATING CURVES

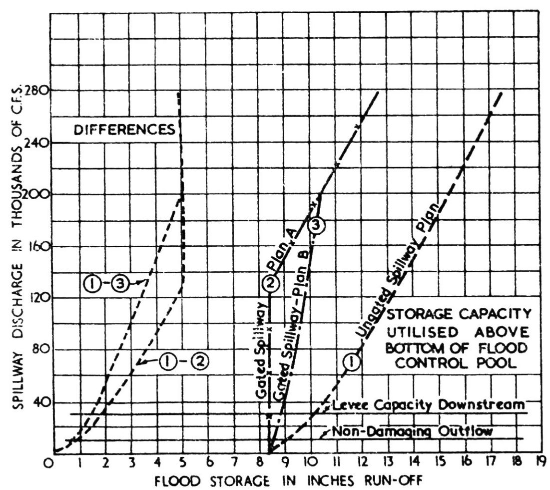


FIG. 6(d). SPILLWAY DISCHARGE vs F.C. STORAGE

NOTES:

Data shown in Fig. 6(a) and 6(b) pertain to alternative project plans for the same site.

Drainage area = 1310 sq. miles.

One inch of Run-off = 69,900 acre feet.

F.C is abbreviation for Flood Control.

Freeboard required for Wave Action = 6 ft.

COMPARISON OF FLOOD-CONTROL EFFECTIVENESS OF GATED AND UNGATED SPILLWAYS

FIGURE 6.

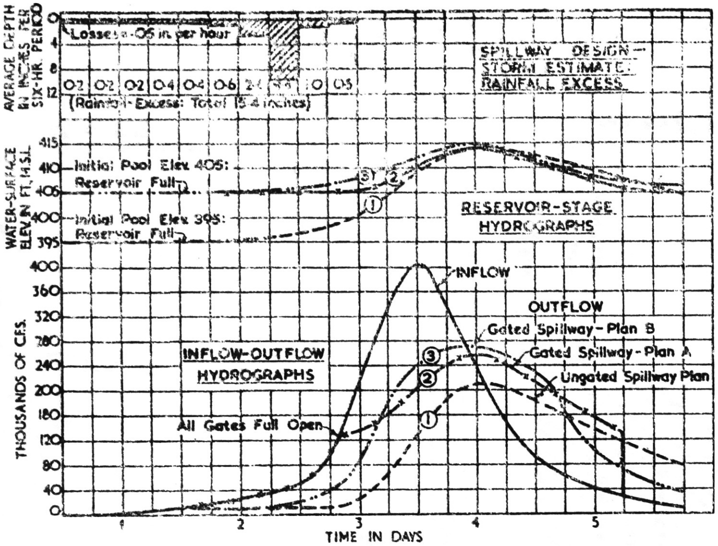


FIG. 7(a). INFLOW-OUTFLOW-STAGE HYDROGRAPHS DURING SPILLWAY DESIGN FLOOD; F.C POOL FULL AT START OF FLOOD.

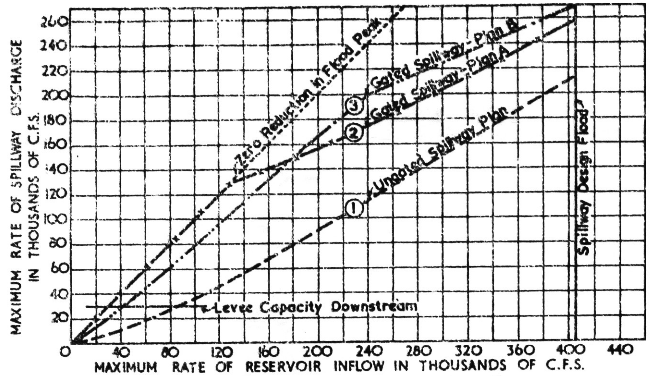


FIG. 7(b). INFLOW - OUTFLOW RATES FOR MAXIMUM RANGE OF FLOOD MAGNITUDES; F.C POOL FULL AT START OF FLOOD.

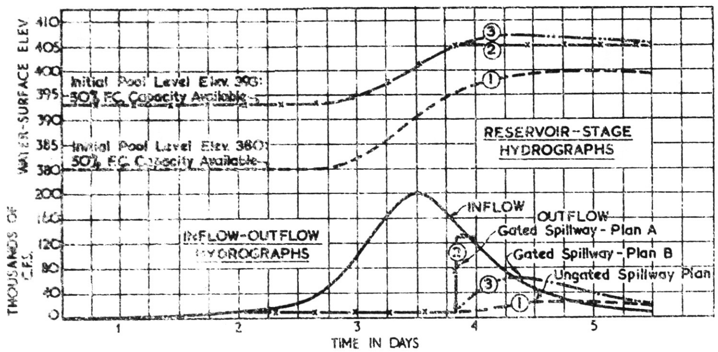


FIG. 7(c). INFLOW OUTFLOW-STAGE HYDROGRAPHS DURING FLOOD EQUAL TO 50% SPILLWAY DESIGN FLOOD; 50% F.C. CAPACITY AVAILABLE AT START OF FLOOD.

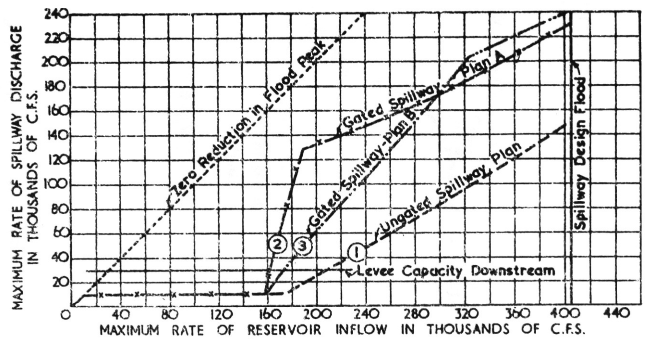


FIG. 7(d). INFLOW - OUTFLOW RATES FOR MAXIMUM RANGE OF FLOOD MAGNITUDES; 50% F.C. CAPACITY AVAILABLE AT START OF FLOOD

NOTES:

Spillway Rating Curves are shown in Fig. 6(c).

Curves shown in Fig. 7(b) and 7(d) were computed by routing floods equal to various percentages of the spillway design flood.

A minimum outflow of 10,000 c.f.s was assumed in all flood routings.

Drainage area = 1310 sq. miles.

F.C is abbreviation for Flood Control.

COMPARISON OF FLOOD-CONTROL EFFECTIVENESS OF GATED AND UNGATED SPILLWAYS

FIGURE 7.

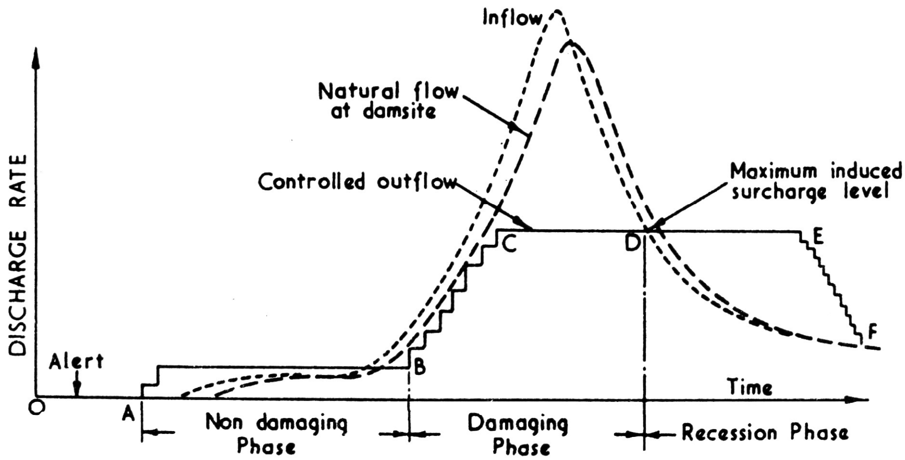


FIGURE 8

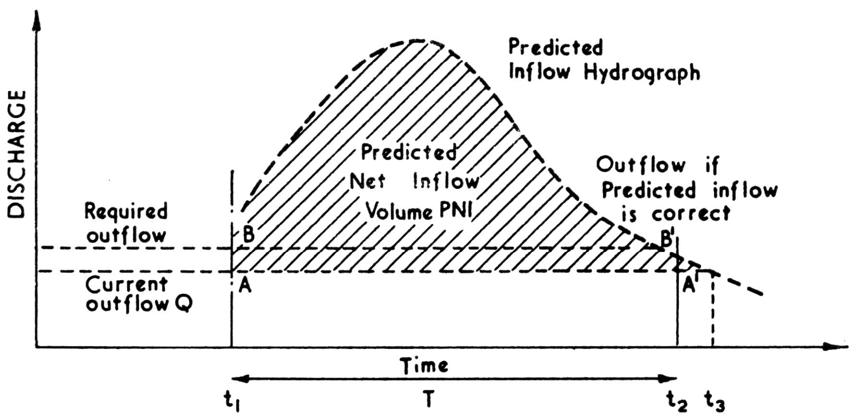


FIGURE 9

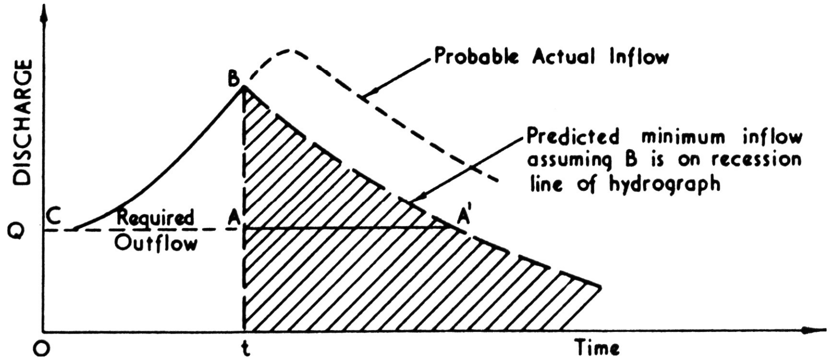
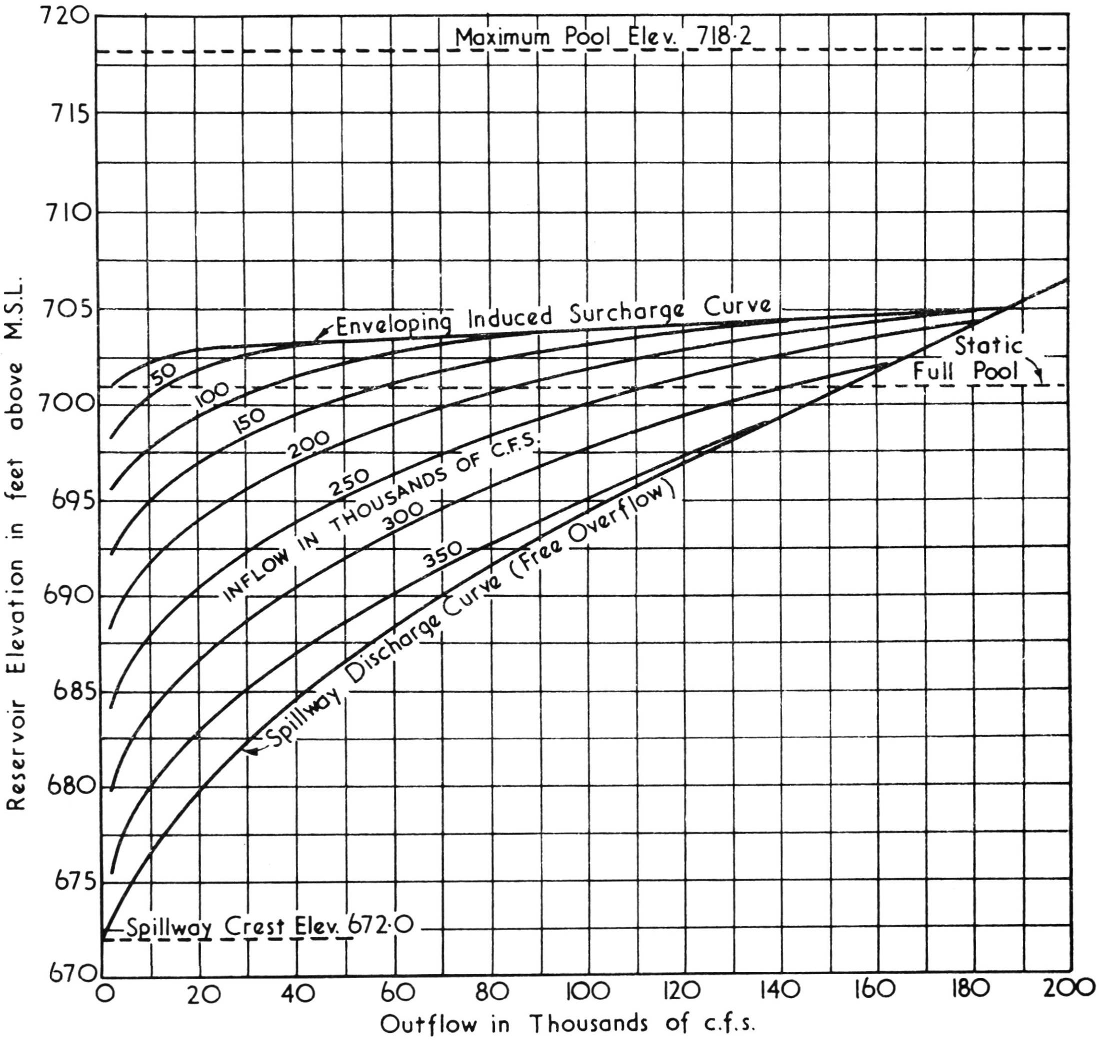


FIGURE 10



SPILLWAY GATE REGULATION SCHEDULE

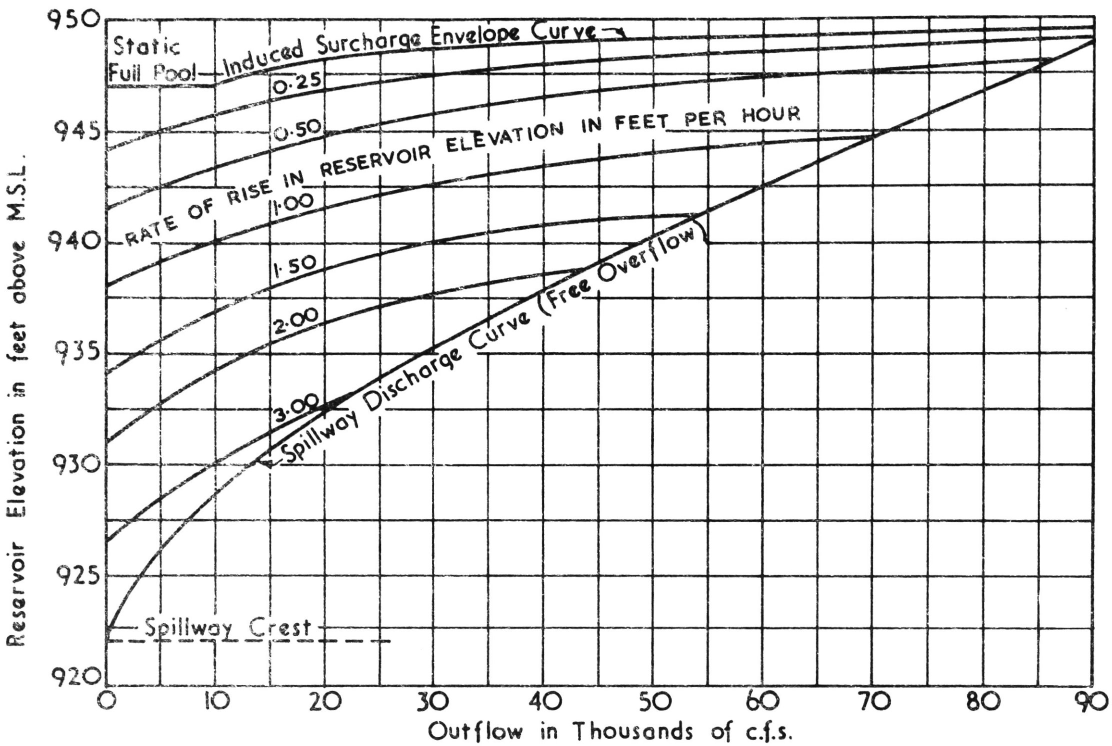
Operating Instructions:

(1) Follow Regular Flood Control Regulation Schedule until larger releases are required by this schedule.

(2) Adjust the outflow each hour on the basis of the average inflow for the preceding three hours and the current reservoir elevation as indicated by the curves.

(3) After the reservoir elevation starts to fall, maintain current gate openings until the elevation recedes to 701. When reservoir is below 701 release at the maximum allowable rate.

FIGURE 11.



SPILLWAY GATE REGULATION, SCHEDULE B

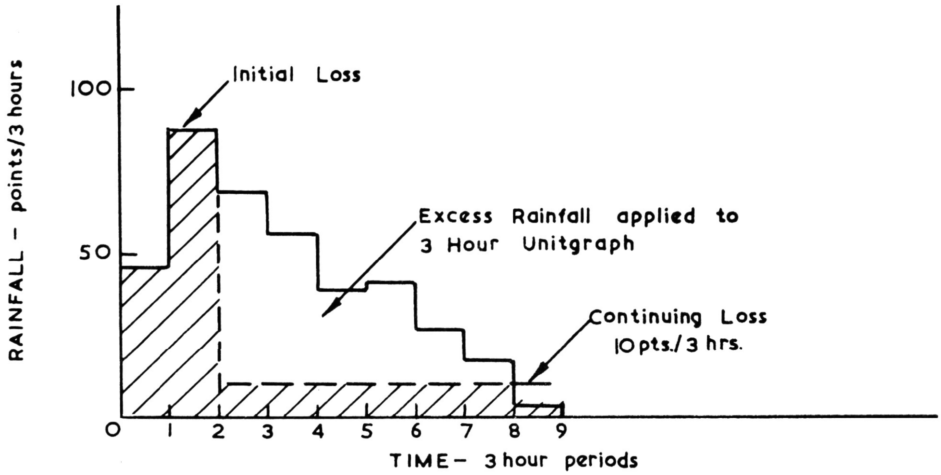
Operating Instructions:

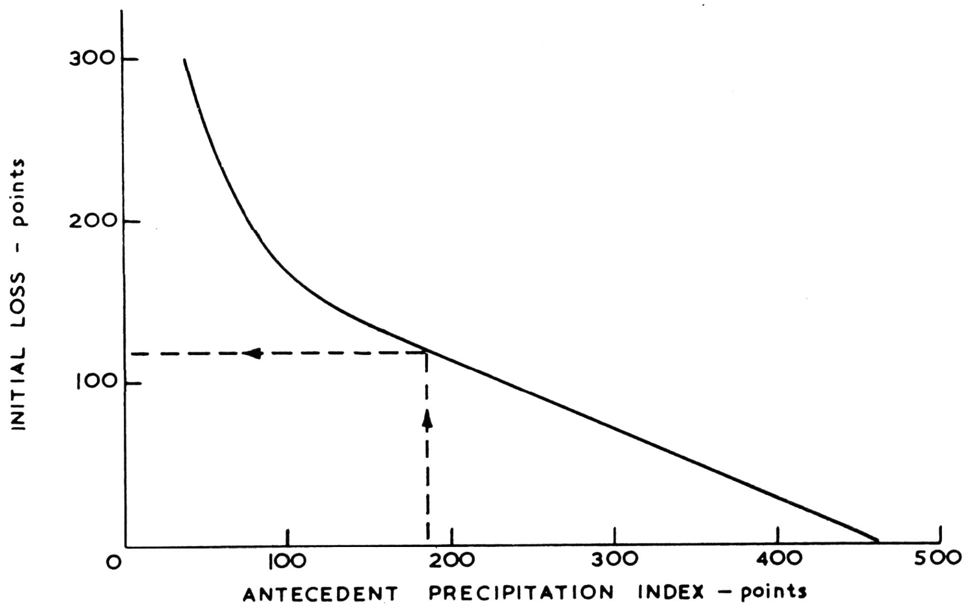
(1) Follow Regular Flood Control Regulation Schedule until larger releases are required by this schedule.

(2) Adjust the outflow each hour on the basis of the rate of rise for the preceding three hours and the current reservoir elevation as indicated by the curves.

(3) After the reservoir elevation starts to fall maintain current gate openings until the elevation recedes to 947. When reservoir is below elevation 947 release at the maximum allowable rate.

FIGURE 12.





DETERMINATION OF EXCESS RAINFALL FOR FLOOD FORECASTING

FIGURE 13.

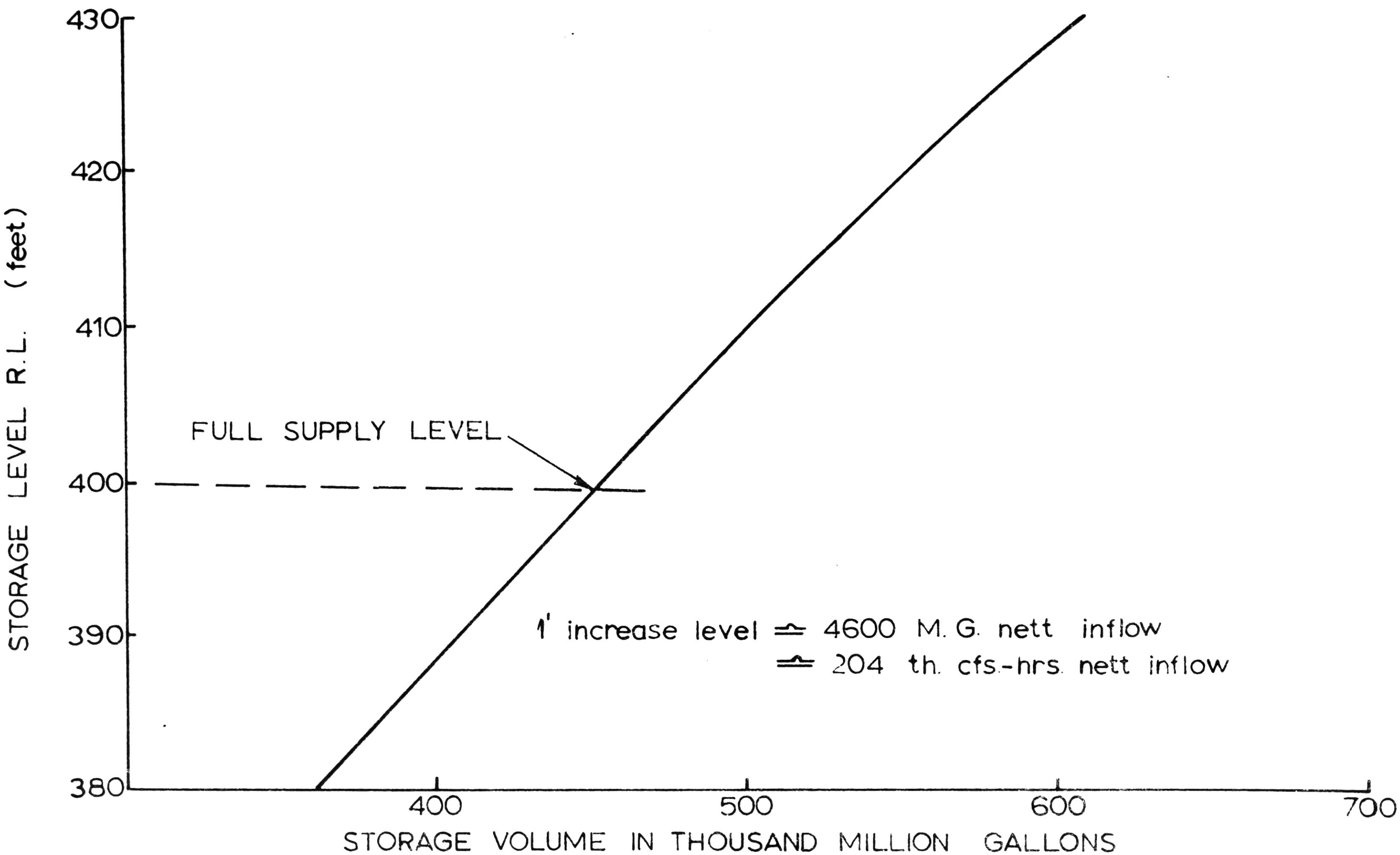


FIG. 14 BIG RIVER DAM
STORAGE-ELEVATION CURVE

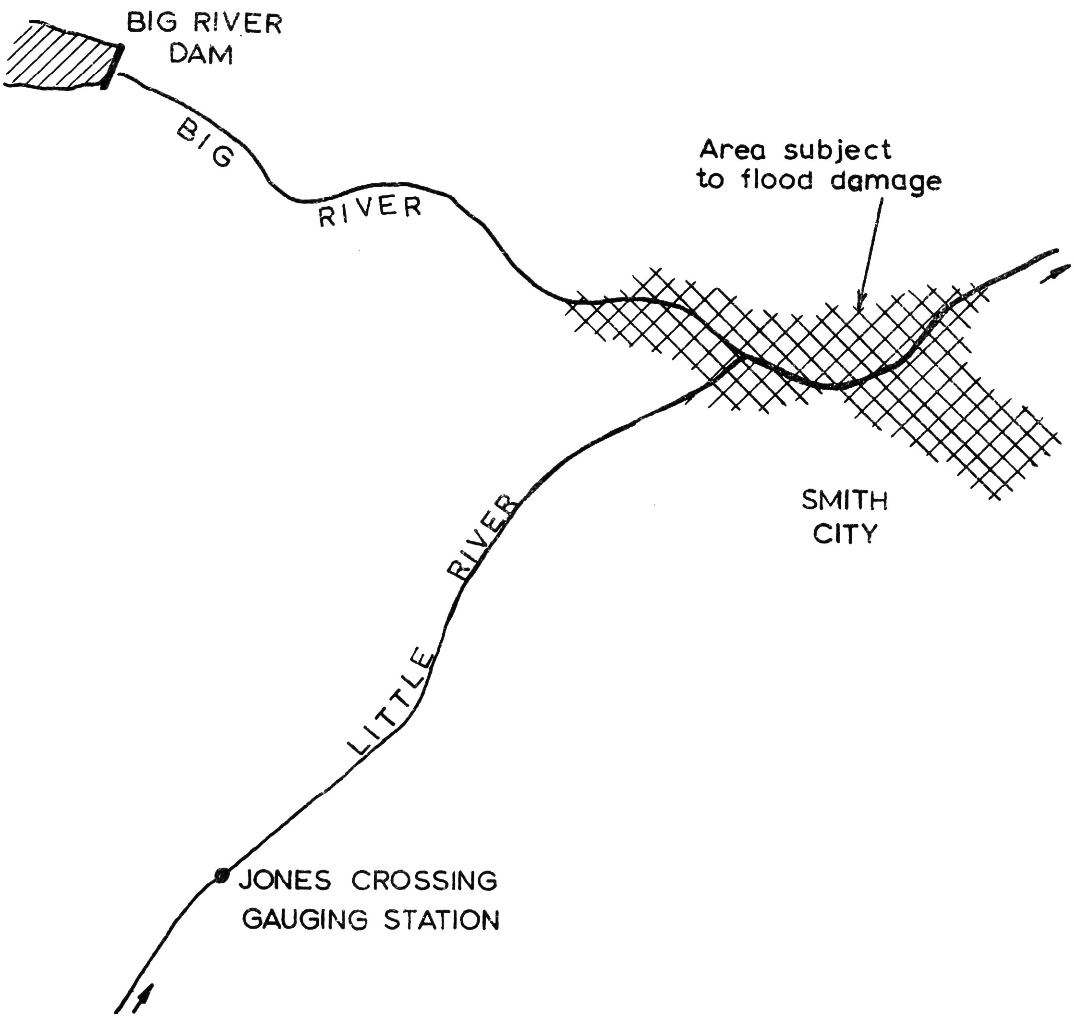


FIG. 15 RIVER SYSTEM AND LOCATION OF FLOOD DAMAGE AREAS D/S DAM

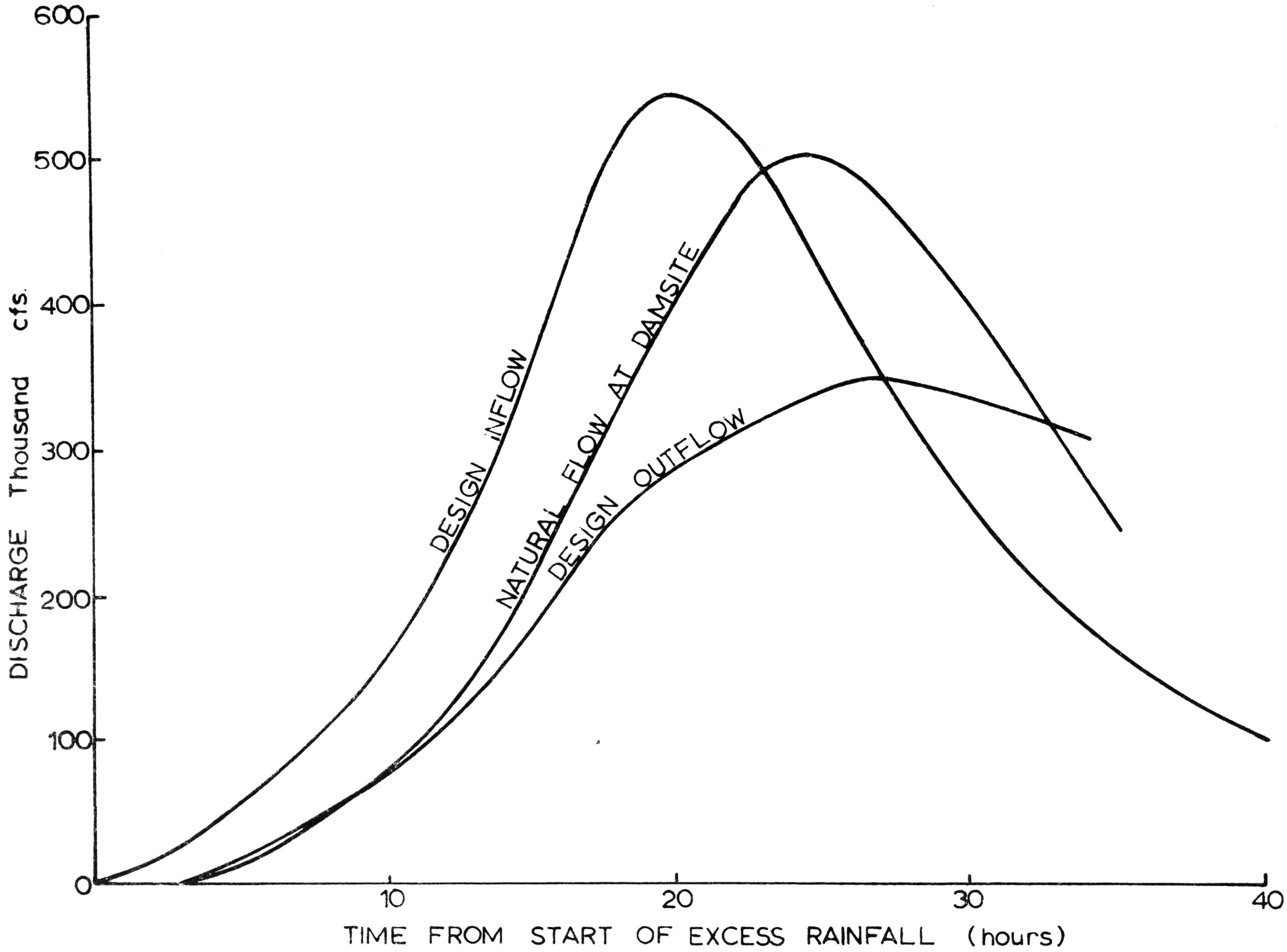


FIG. 16 BIG RIVER DAM
DESIGN HYDROGRAPHS

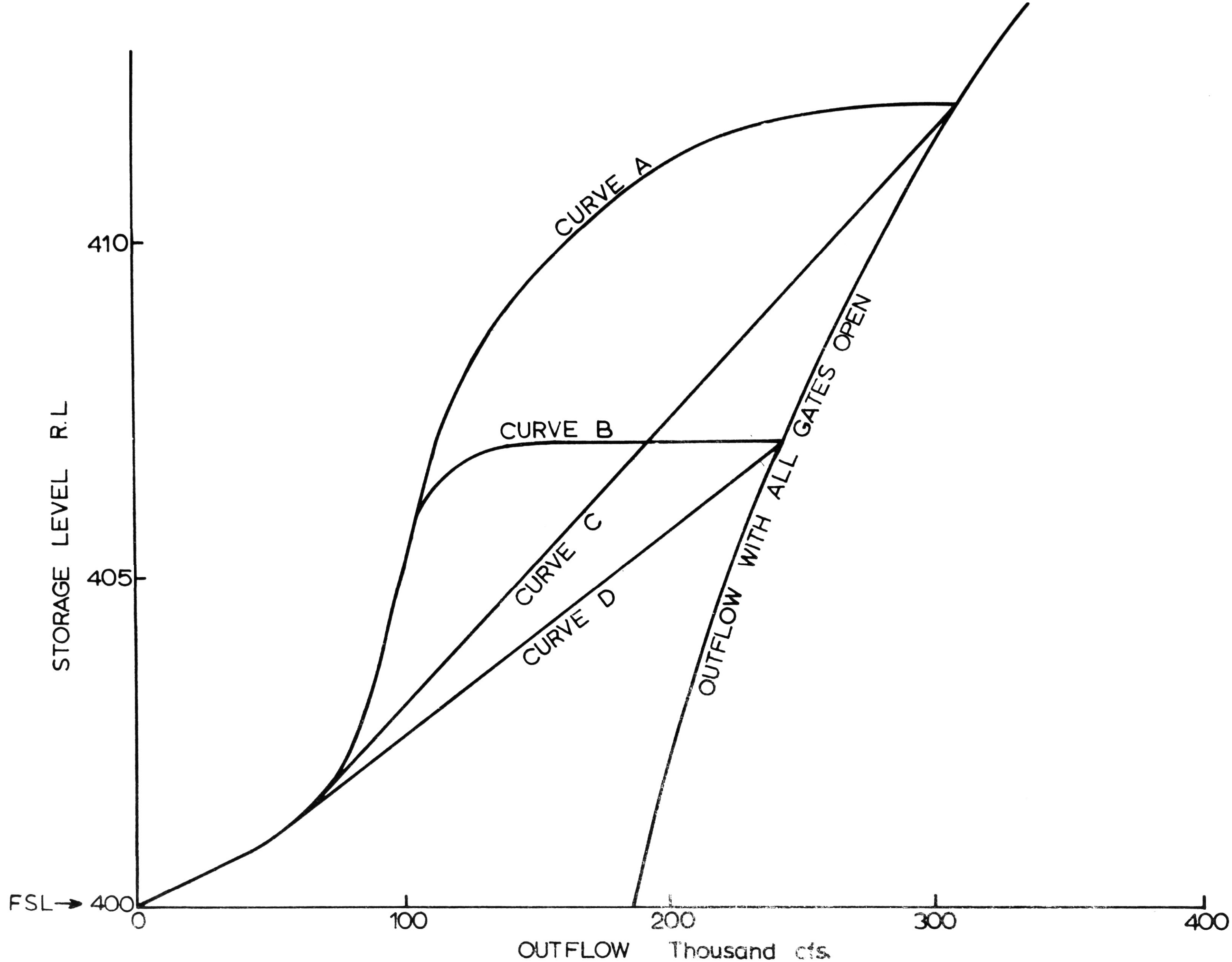


FIG. 17 FOUR ALTERNATIVE MAXIMUM INDUCED SURCHARGE CURVES.

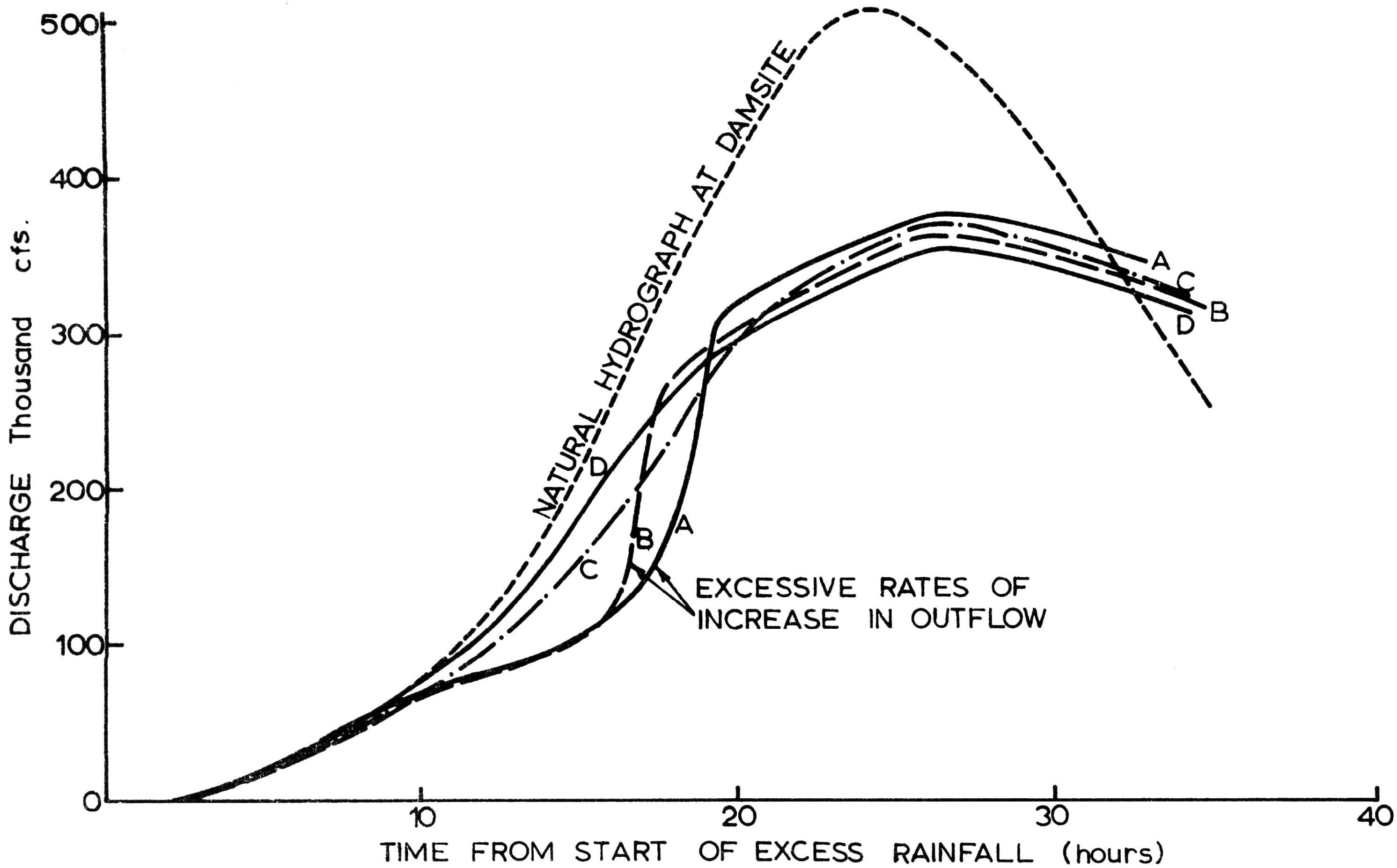


FIG.18 DESIGN OUTFLOW HYDROGRAPHS WITH FOUR ALTERNATIVE MAX. INDUCED SURCHARGE CURVES (see fig. 16)

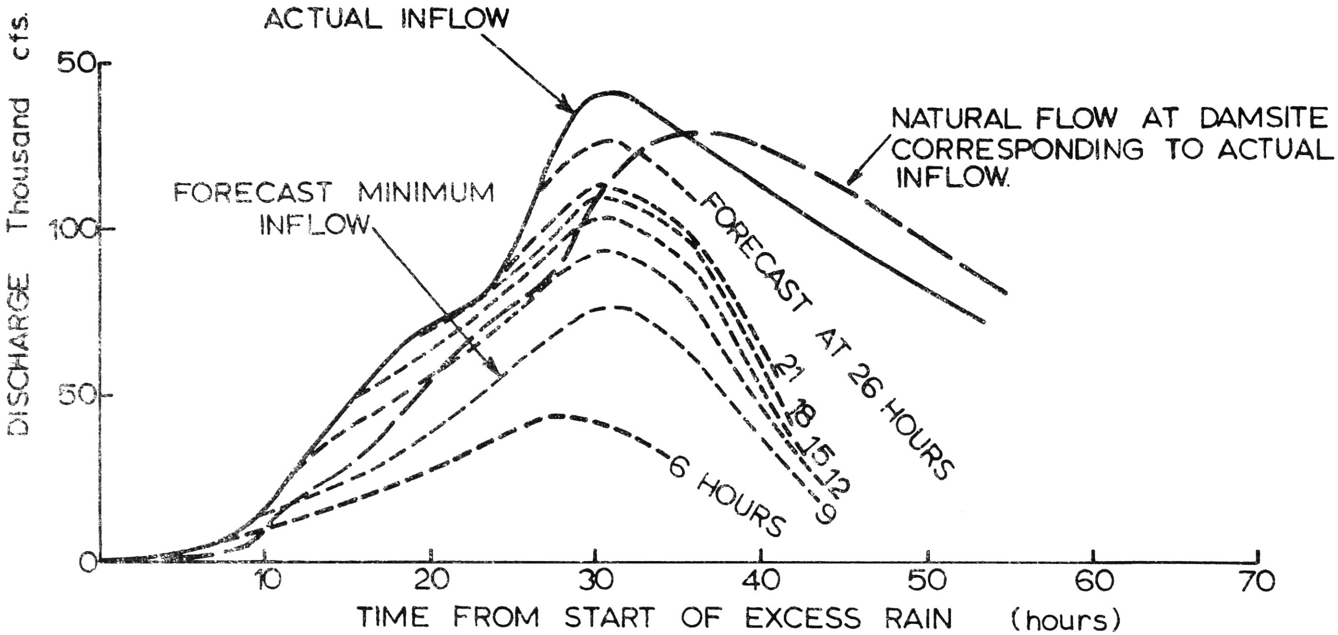


FIG. 19 TYPICAL FLOOD CAUSING MODERATE DAMAGE

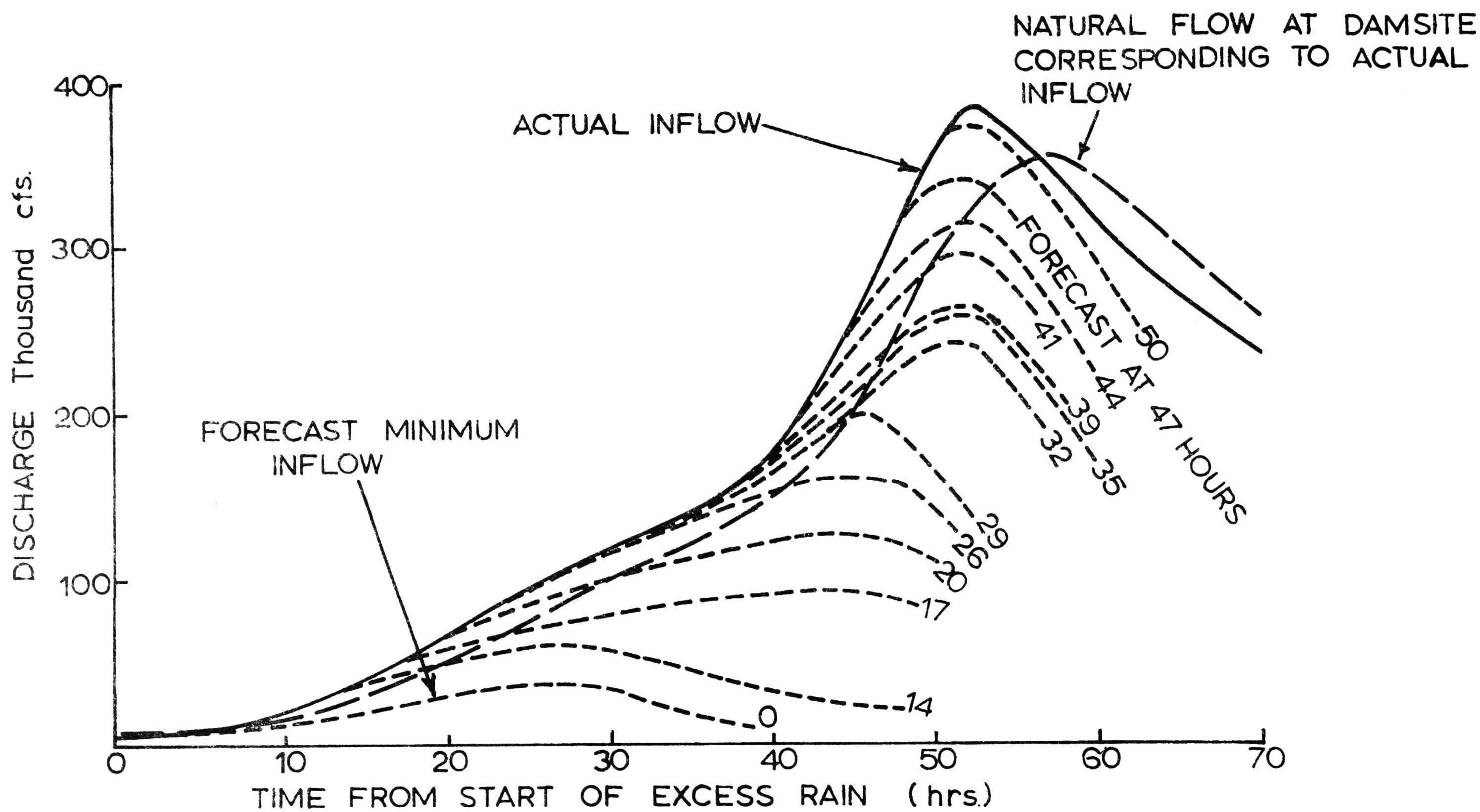


FIG. 20 TYPICAL FLOOD CAUSING EXTENSIVE DAMAGE.

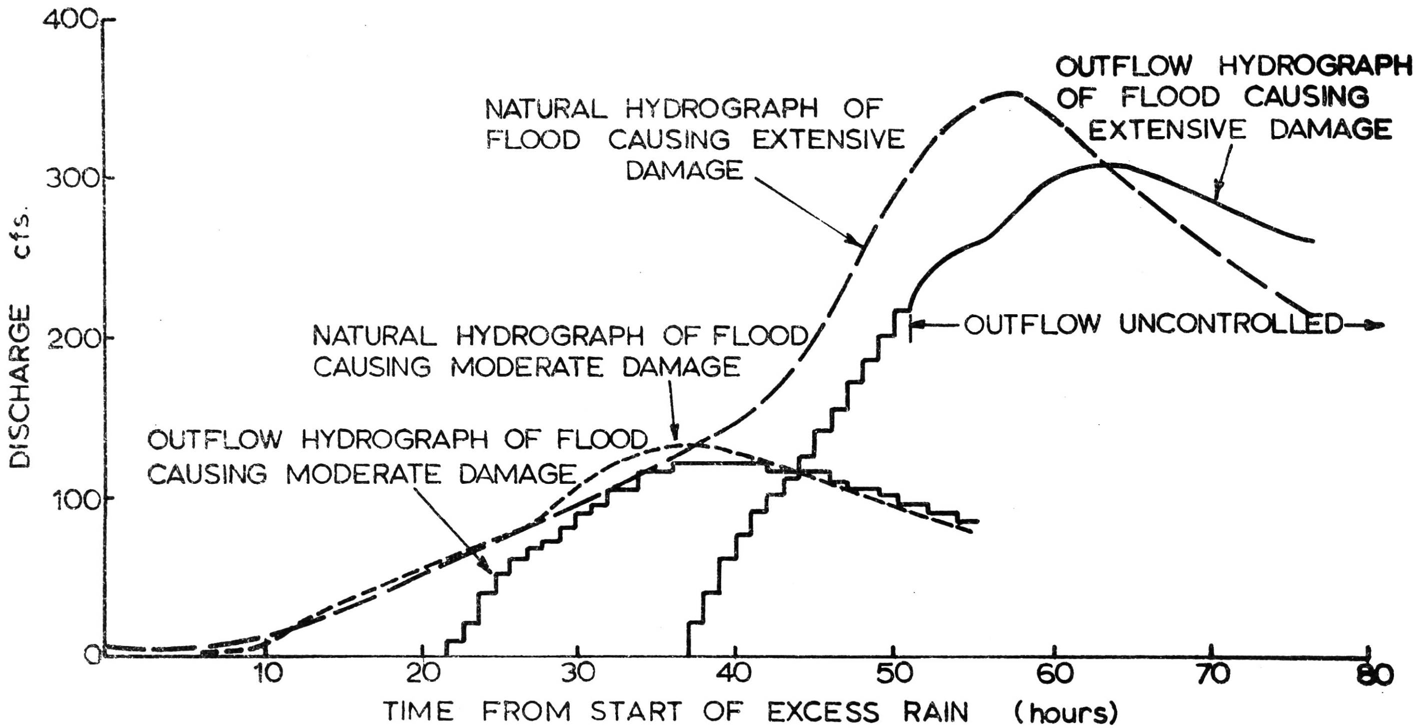
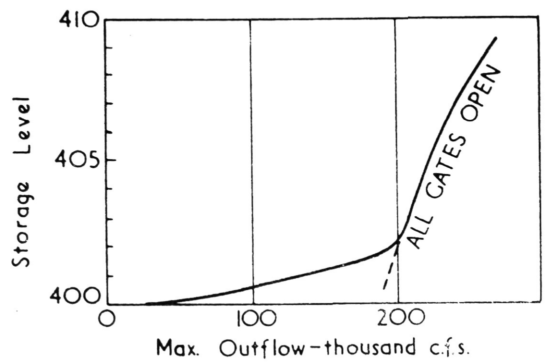
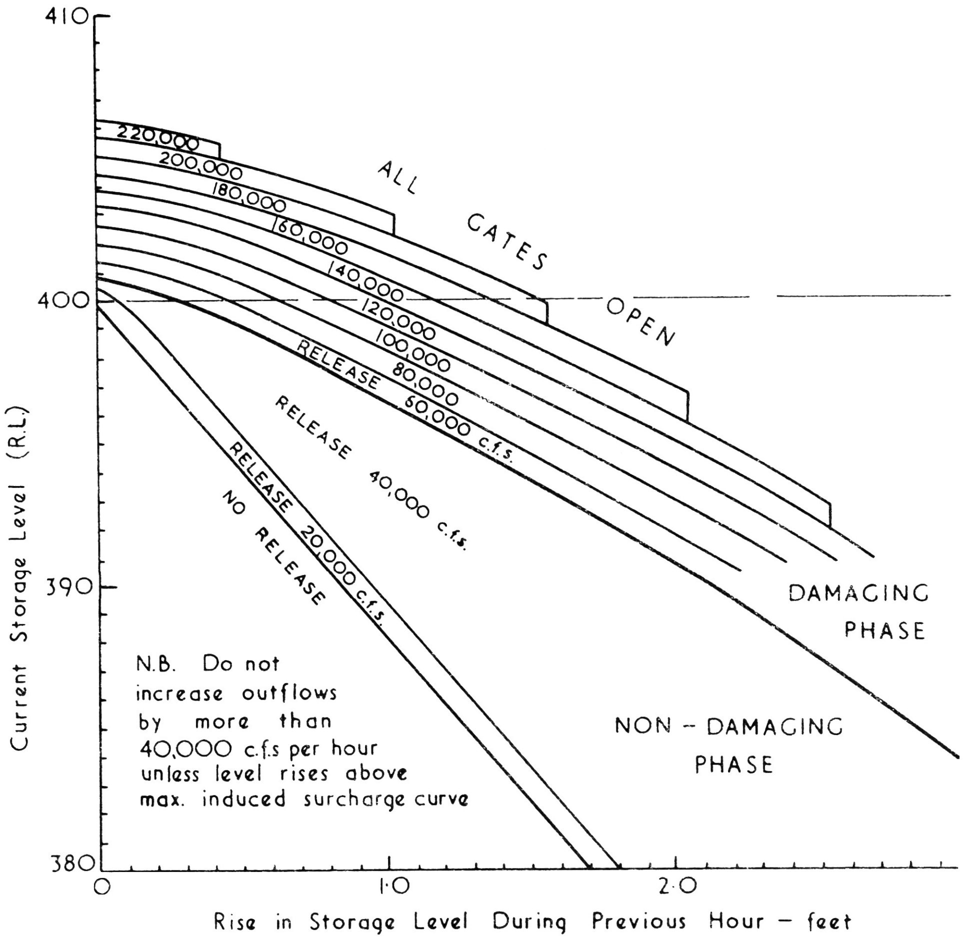


FIG. 21 GATE OPERATIONS BASED ON CURRENT RESERVOIR LEVELS ONLY.





Recession Phase:

Maintain final outflow of damaging phase until storage falls to level indicated above. Outflow should then be reduced to maintain level just above curve.

GATE OPERATIONS BASED ON RESERVOIR LEVELS AND INFLOW

FIGURE 22.

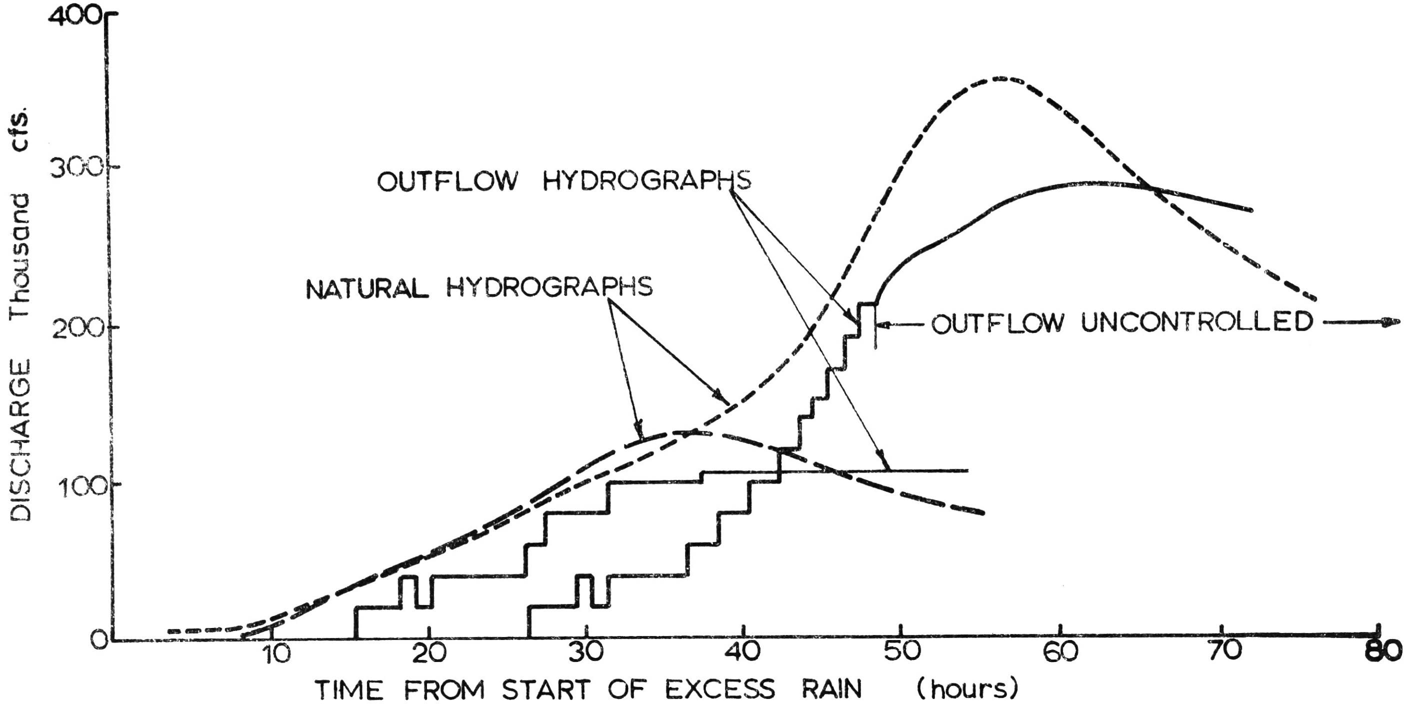


FIG. 23 GATE OPERATIONS BASED ON RESERVOIR LEVELS AND INFLOW.

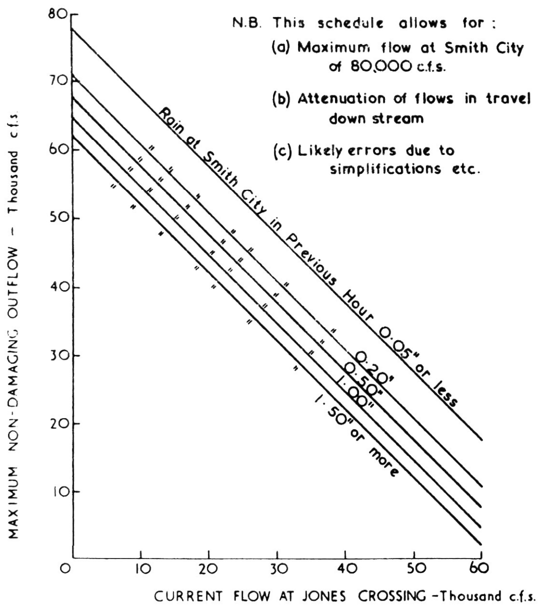


FIGURE 24: MAXIMUM NON-DAMAGING OUTFLOW FROM DAM

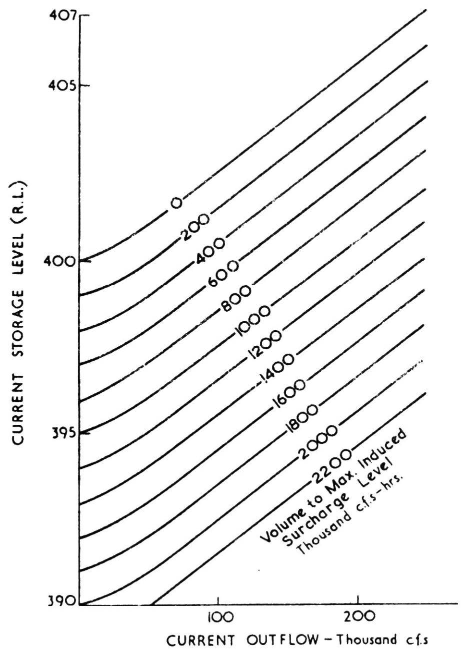


FIGURE 25: NETT INFLOW VOLUME TO BRING STORAGE TO MAXIMUM INDUCED SURCHARGE LEVEL

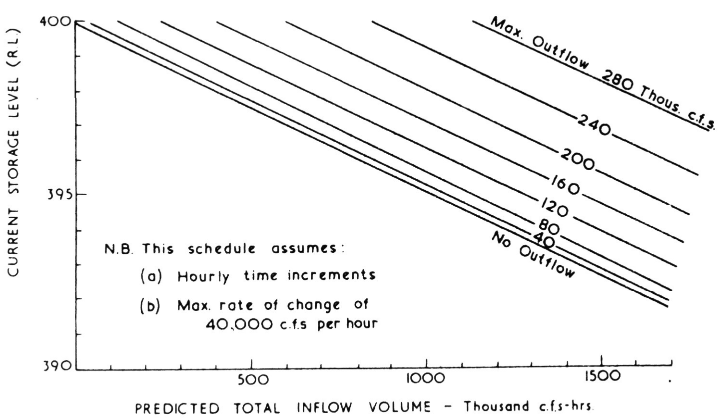


FIGURE 26: MAXIMUM OUTFLOW FOR ONE HOUR TO ENSURE A FULL DAM AT END OF FLOOD

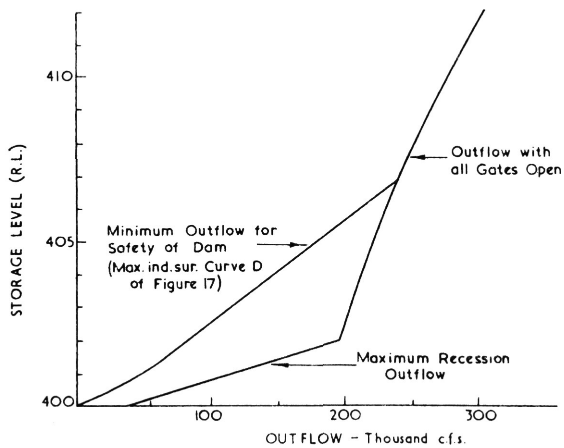


FIGURE 27: SAFETY AND RECESSION CONSTRAINTS

Q = CURRENT OUTFLOW

QC = PROGRESSIVE COMPUTATION OF NEW OUTFLOW

QF = MAXIMUM OUTFLOW FOR FULL DAM

QL = MAXIMUM OUTFLOW BY RATE OF CHANGE = Q - 40 THOUSAND CUSECS

QM = MAXIMUM NON-DAMAGING OUTFLOW

QN = NATURAL FLOW AT DAMSITE

QO = OPTIMUM FLOOD CONTROL OUTFLOW

QR = MAXIMUM RECESSION OUTFLOW

QS = MINIMUM OUTFLOW FOR DAM SAFETY

QU = MAXIMUM OUTFLOW BY RATE OF CHANGE = Q + 40 THOUSAND CUSECS

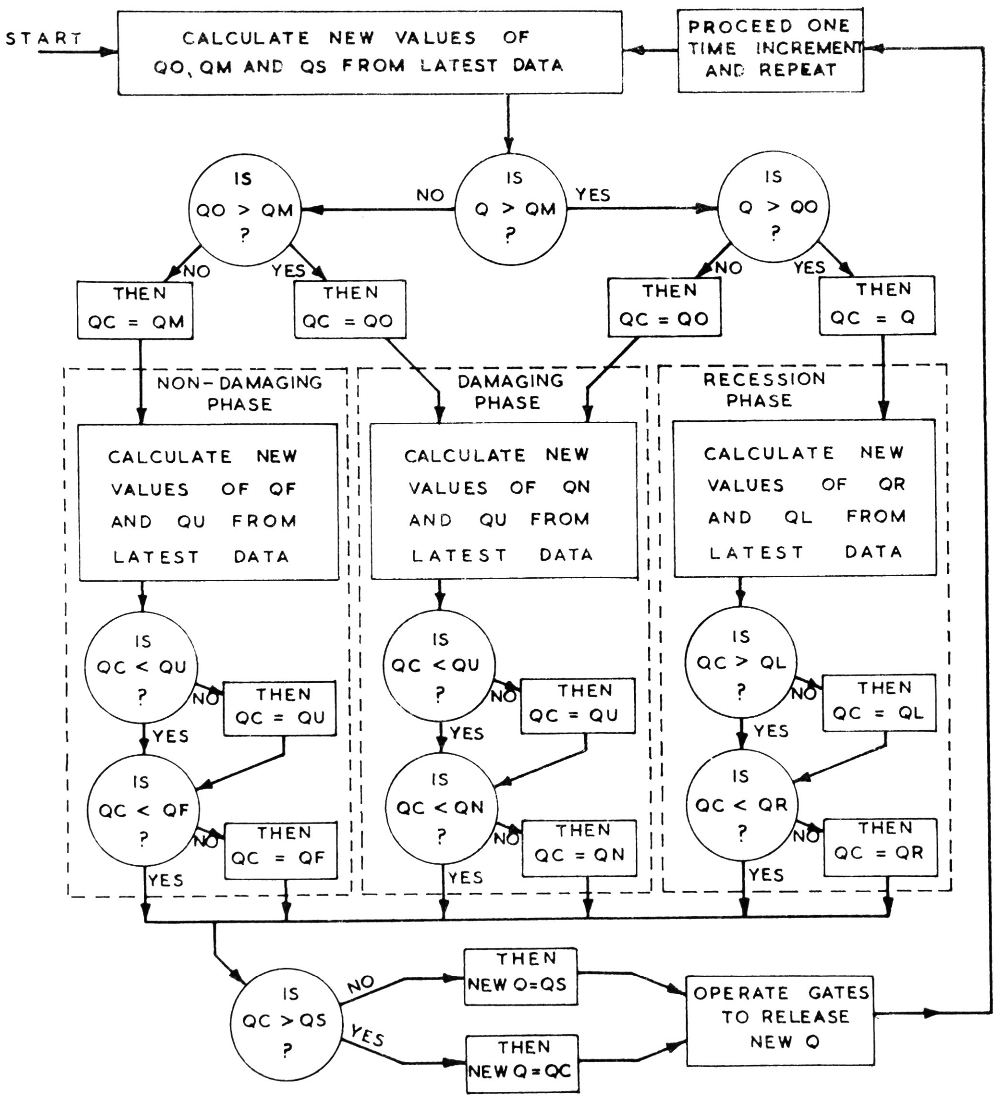


FIGURE 28: SEQUENCE OF COMPUTATIONS FOR DETERMINING GATE OPERATIONS BASED ON FLOOD FORECASTS

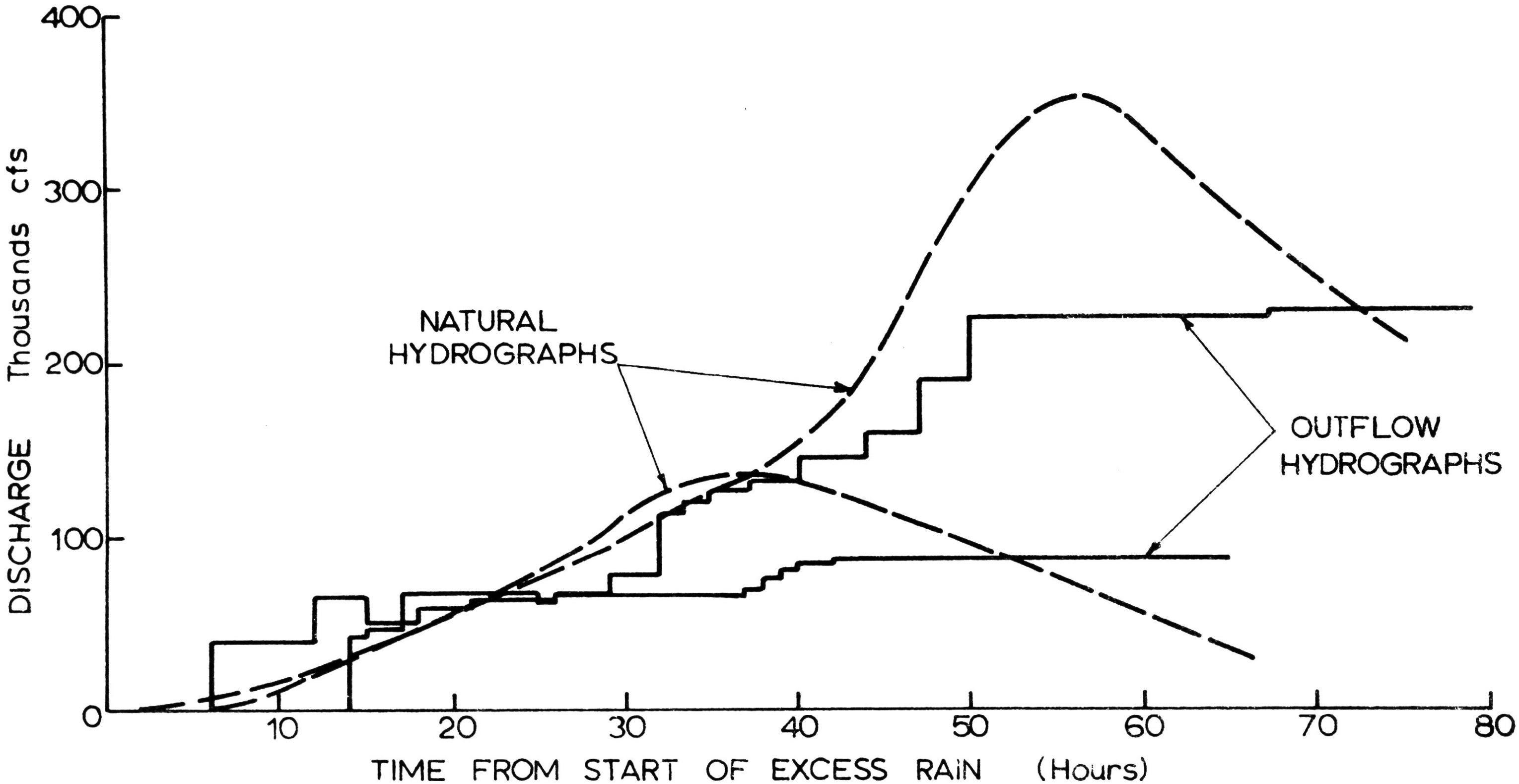


FIG. 29. GATE OPERATIONS BASED ON FLOOD FORECASTS.

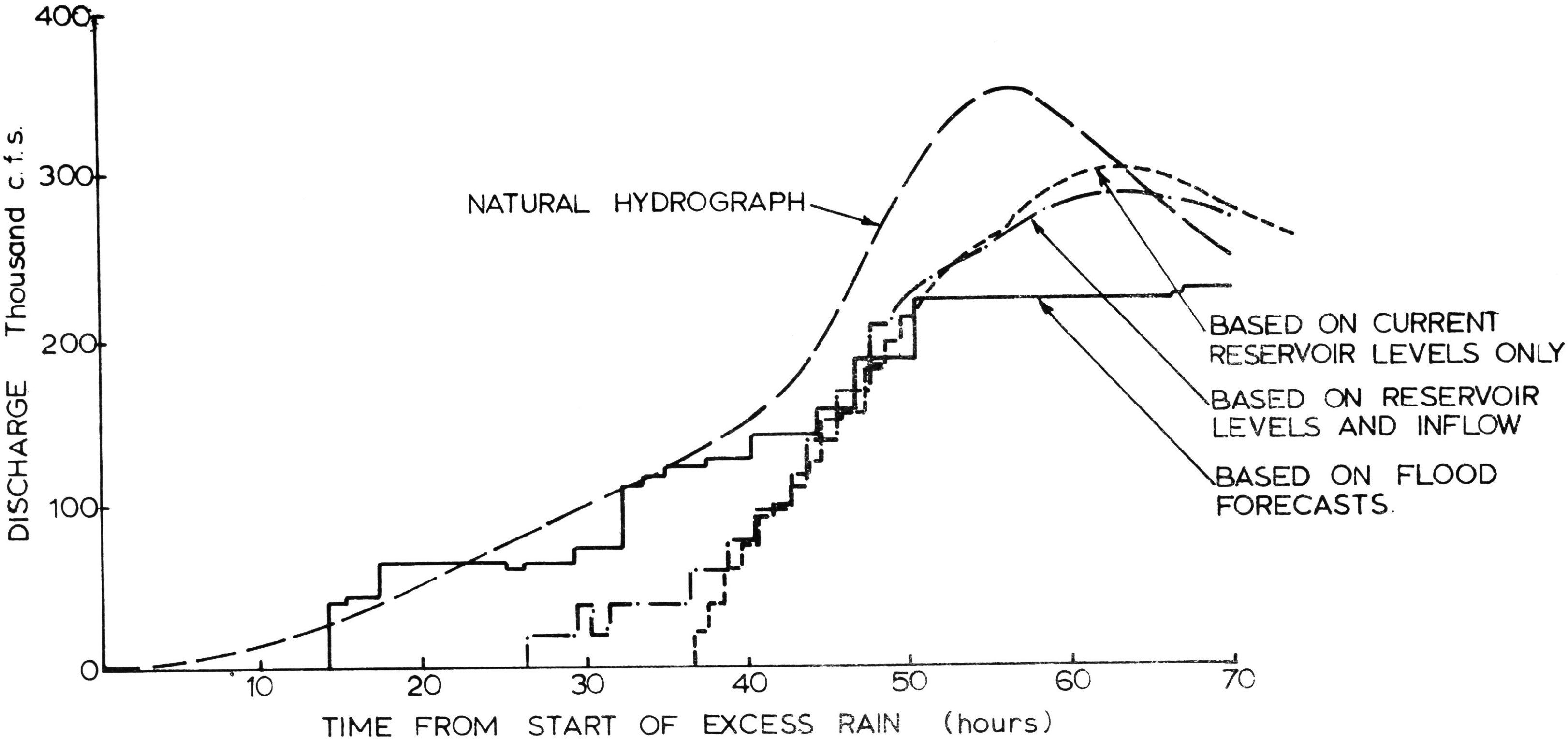


FIG. 30. COMPARISON OF METHODS OF GATE OPERATION FOR FLOOD CAUSING EXTENSIVE DAMAGE.

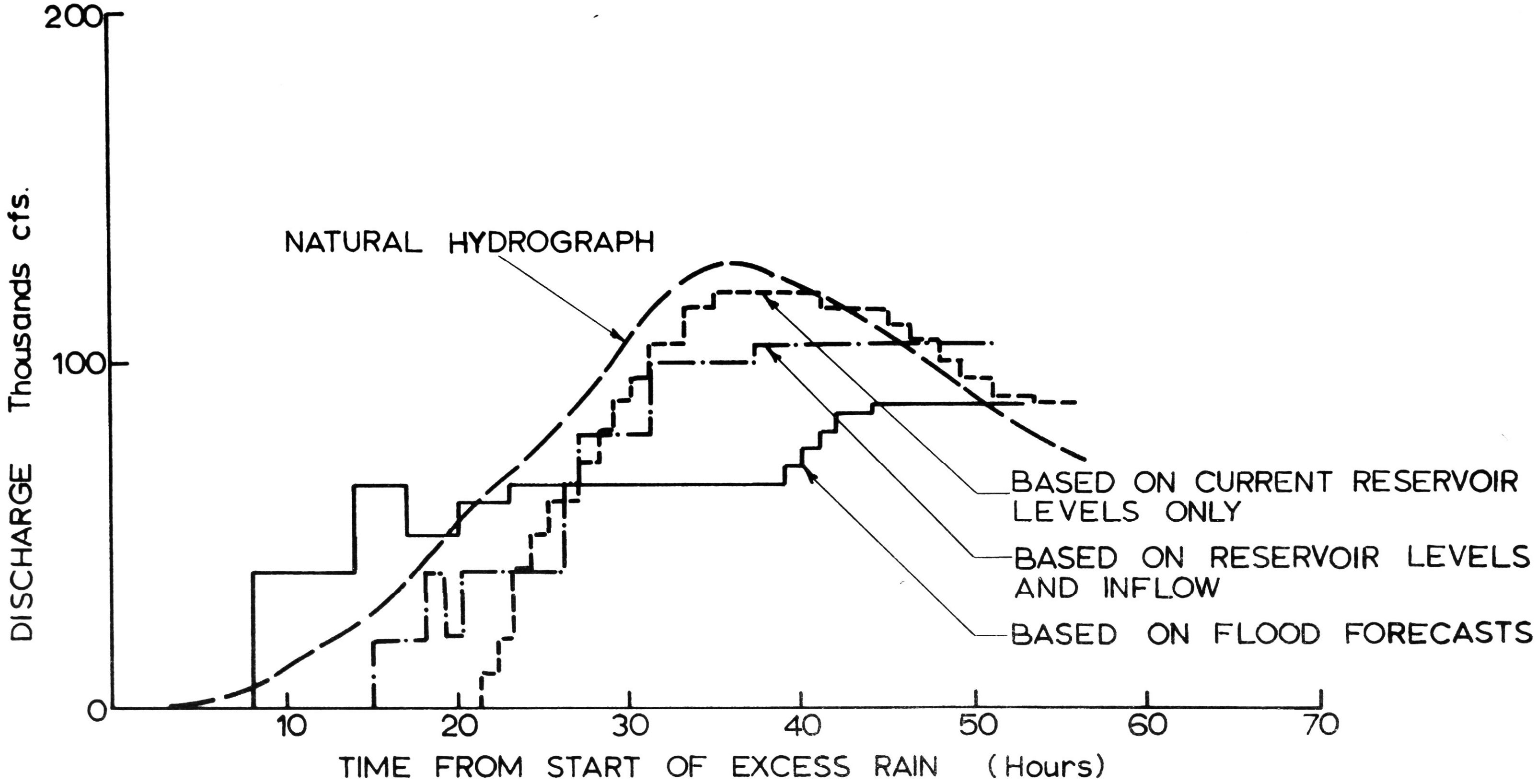
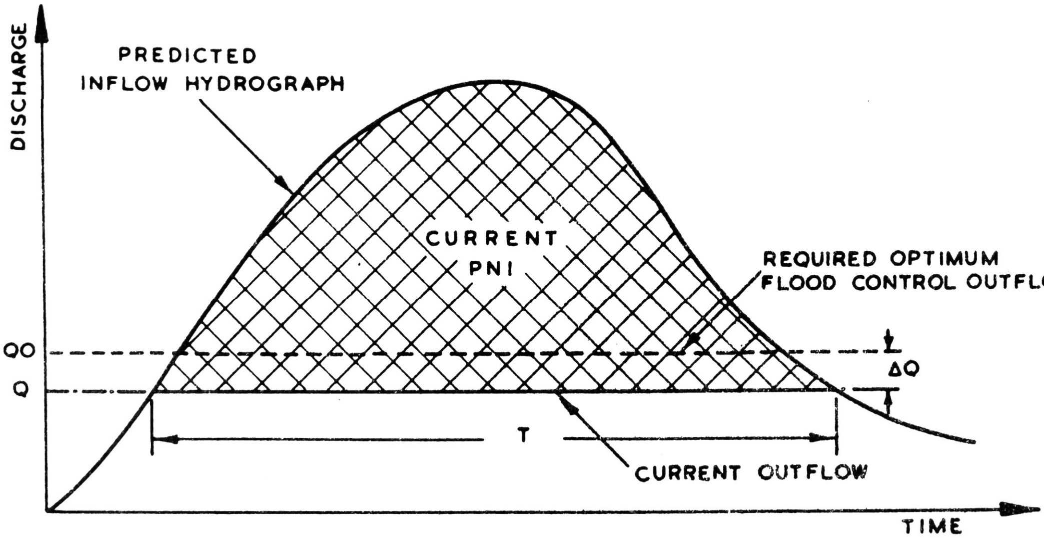


FIG. 31. COMPARISON OF METHODS OF GATE OPERATION FOR FLOOD CAUSING MODERATE DAMAGE.



PNI = PREDICTED NETT INFLOW (THOUSAND CUSEC-HRS)

VML = VOLUME TO MAX. IND. SUR. LEVEL (THOUSAND CUSEC-HRS)

Q = CURRENT OUTFLOW (THOUSAND CUSECS)

QO = REQUIRED OPTIMUM FLOOD CONTROL OUTFLOW. i.e. WHEN PNI = VML (THOUSAND CUSECS)

ΔQ = REQUIRED CHANGE IN Q TO MAKE PNI = VML

$= QO - Q$

T = TIME IN HOURS DURING WHICH THE PREDICTED INFLOW WILL EXCEED Q

CHANGE IN VML DUE TO $\Delta Q = +\Delta Q \times 6{\cdot}7$ (FROM FIG. 25)

CHANGE IN PNI DUE TO $\Delta Q \triangleq -\Delta Q \times T$

$\therefore\ PNI - \Delta Q \times T \triangleq VML + \Delta Q \times 6{\cdot}7$ FOR OPTIMUM FLOOD CONTROL

i.e. $\Delta Q \times T + \Delta Q \times 6{\cdot}7 \triangleq PNI - VML$

$$\Delta Q \triangleq \frac{PNI - VML}{T + 6{\cdot}7}$$

$$\therefore\ QO \triangleq Q + \Delta Q \triangleq Q + \frac{PNI - VML}{T + 6{\cdot}7} \quad \ldots\ldots\ldots\ldots (1)$$

FIGURE 32: DERIVATION OF EQUATION (1)

APPENDIX A

AVERAGE THREE HOUR UNITGRAPH FOR THE MACLEAY RIVER AT KEMPSEY

Period (3 hours)		Discharge cusecs
1	...	1700
2	...	9700
3	...	19800
4	...	28900
5	...	30400
6	...	34600
7	...	39300
8	...	51800
9	...	59800
10	...	66000
11	...	62800
12	...	58700
13	...	49100
14	...	39900
15	...	32600
16	...	27700
17	...	24300
18	...	22100
19	...	19700
20	...	19000
21	...	17400
22	...	14700
23	...	14000
24	...	13100
25	...	12400

APPENDIX B

FLOOD FORECAST

Macleay River at Kempsey

January 1962 - Storm 30

Date	Time	API at 9 am	Initial Loss Predicted (pts)	Initial Loss Adopted (pts)	3 hr. Rainfall Gross (pts)	3 hr. Rainfall Excess (pts)	Forecast Minimum Peak Discharge (cusecs)	Forecast Time of Peak	Weather Forecast for 12 Hours	Observed Discharge (cusecs)
11.1.62.	0900	268	140							
12.1.62.	0900	304	70						Rain	
	1200			55	55				Rain	
	1500				75	65	54000	13th-1800	Rain	11000
	1800				24	14	62400	13th-1800	Rain Easing	11000
	2100				13	3	63000	13th-1800		12000
	2400				4					14000
13.1.62.	0300									15500
	0600									17000
	0900									18000
	1200									20500
	1500									27000
	1800									49000
	2100									69000
	2400									77000
14.1.62	0300									75000
	0600									70500
	0900									63000
	1200									55000
	1500									47000
	1800									41500
	2100									38000
	2400									34000
15.1.62.	0300									30000
	0600									26000
	0900									23000
	1200									22500

FLOOD FORECAST

Macleay River at Kempsey

April 1962 - Storm 31

Date	Time	API at 9 am	Initial Loss Predicted (pts)	Initial Loss Adopted (pts)	3 hr. Rainfall Gross (pts)	3 hr. Rainfall Excess (pts)	Forecast Minimum Peak Discharge (cusecs)	Forecast Time of Peak	Weather Forecast for 12 Hours	Observed Discharge (cusecs)
6.4.62.	0900	286	80							
	1200				49					16000
	1500				15					17600
	1800			88	24					19100
	2100				22	12	23000	7th-2400	Rain	23700
	2400				26	16	34000	8th-0300	Rain	31500
7.4.62.	0300				39	29	52000	8th-0600	Rain increase	40500
	0600				33	23	66000	8th-0600	Rain	51500
	0900				30	20	76000	8th-0600	Rain	60330
	1200				27	17	83000	8th-0600	Rain increase	67800
	1500				37	27	97000	8th-1200	Rain	73500
	1800				34	24	106000	8th-1200	Rain	78400
	2100				46	36	128000	8th-1200	Rain increase	86000
	2400				12	2	129000	8th-1200	Rain	95500
8.4.62.	0300				11	1	130000	8th-1200	Rain	104800
	0600				40	30	136000	8th-1500	Rain easing	197600
	0900				32	22	137000	8th-1800	Rain easing	111900
	1200				41	31	140000	8th-2400	Rain easing	122400
	1500				39	29	148000	8th-2400	Rain easing	124100
	1800				6					133000
	2100									140000
	2400									146000
9.4.62.	0300									146000
	0600									146000
	0900									138000
	1200									133000
	1500									126000
	1800									116000
	2100									106000
	2400									92000
10.4.62.	0300									85000
	0600									75000
	0900									66000
	1200									58000
	1500									52000
	1800									46000

FLOOD FORECAST

Macleay River at Kempsey

July 1962 - Storm 32

Date	Time	API at 9 am	Initial Loss: Predicted (pts)	Initial Loss: Adopted (pts)	3 hr. Rainfall: Gross (pts)	3 hr. Rainfall: Excess (pts)	Forecast: Minimum Peak Discharge (cusecs)	Forecast: Time of Peak	Weather Forecast for 12 Hours	Observed Discharge (cusecs)
10.7.62.	0900	186	120						Rain	
	1200				45				Rain	
	1500			132	87				Rain	
	1800				69	59	40000	11th-2100	Rain	1000
	2100				56	46	69000	11th-2400	Rain	4000
	2400				39	29	86000	11th-2400	Rain	20200
11.1.62.	0300				41	31	102000	11th-2400	Rain	29000
	0600				26	16	108000	11th-2400	Rain easing	43000
	0900				17	7	111000	11th-2400		60300
	1200				2					74700
	1500									86800
	1800									110500
	2100									125900
	2400									127700
12.1.62.	0300									124100
	0600									117500
	0900									109000
	1200									98300
	1500									88400
	1800									78000
	2100									70000
	2400									61800
13.7.62.	0300									54000
	0600									48000
	0900									42000

FLOOD FORECAST

Macleay River at Kempsey

January 1963 - Storm 33

Date	Time	API at 9 am	Initial Loss Predicted (pts)	Initial Loss Adopted (pts)	3 hr. Rainfall Gross (pts)	3 hr. Rainfall Excess (pts)	Forecast Minimum Peak Discharge (cusecs)	Forecast Time of Peak	Weather Forecast for 12 hours	Observed Discharge (cusecs)
2.1.63.	0900	403								
3.1.63.	0900	418	25		0				Rain	
	1200				3				Rain	
	1500			17	14				Rain	
	1800				69	59	61000	4th-2400	Rain	22000
	2100				3	0	61000	4th-2400	Rain	22000
	2400				65	55	92000	5th-0300	Rain	22000
4.1.63.	0300				49	39	112000	5th-0300	Rain easing	22000
	0600				14	4	114000	5th-0300	Rain easing	24000
	0900				12	2				24200
	1200				22	12				26100
	1500				1					29400
	1800									34400
	2100									40000
	2400									49800
5.1.63.	0300									61000
	0600									71000
	0900									75300
	1200									75900
	1500									73000
	1800									69000
	2100									62000
	2400									55000
6.1.63.	0300									48000
	0600									42000
	0900									36000

FLOOD FORECAST

Macleay River at Kempsey

April 1963 - Storm 34

Date	Time	API at 9 am	Initial Loss: Predicted (pts)	Initial Loss: Adopted (pts)	3 hr. Rainfall: Gross (pts)	3 hr. Rainfall: Excess (pts)	Forecast: Minimum Peak Discharge (cusecs)	Forecast: Time of Peak	Weather Forecast for 12 Hours	Observed Discharge (cusecs)
25.4.63.	0900	126								
26.4.67.	0900	146	142						Heavy rain	3000
	1200								developing	
	1500								"	
	1800								"	
	2100			109	109				Rain	
	2400				44	34	25000	28th-0300		
27.4.63.	0300				16	6	29000	28th-0300	"	21000
	0600				16	6	32000	28th-0300	"	28000
	0900				22	12	37000	28th-0300	"	31000
	1200				30	20			"	40000
	1500				43	33	Forecast		"	61000
	1800				14	4	Failure		"	84200
	2100				44	34			Rain easing	101000
	2400				12	2			"	110000
28.4.63.	0300				27	17			"	107600
	0600				43	33			"	103000
	0900				82	72	121000	29th-0600	"	99200
	1200				9	0				106200
	1500				14	4				109000
	1800									98300
	2100									90500
	2400									83000
29.4.63.	0300									76000
	0600									69000
	0900									62000
	1200									53000
	1500									47000
	1800									42000
	2100									37000
	2400									33000

FLOOD FORECAST

Macleay River at Kempsey

May 1963 - Storm 35

Date	Time	API at 9 am	Initial Loss Predicted (pts)	Initial Loss Adopted (pts)	3 hr. Rainfall Gross (pts)	3 hr. Rainfall Excess (pts)	Forecast Minimum Peak Discharge (cusecs)	Forecast Time of Peak	Weather Forecast for 12 Hours	Observed Discharge (cusecs)
6.5.63.	0900	376	40							
	1200									
	1500				30					
	1800			49	19				Rain easing	
	2100				49	39	30000	7th-2400	"	4000
	2400				18	8	34000	7th-2400	"	(6000)
7.5.63.	0300				13	3	36000	7th-2400	Rain increase	(9000)
	0600				16	6	39000	7th-2400	"	(12000)
	0900				5	0	39000	7th-2400	"	(16000)
	1200				16	6	41000	8th-0300	Rain easing	20000
	1500				32	22	48000	8th-0600	"	23000
	1800				32	22	54000	8th-0600	Rain increase	33000
	2100				74	64	89000	8th-2400	"	45000
	2400				54	44	116000	8th-2400	"	62700
8.5.63.	0300				78	68	152000	8th-2400	"	80300
	0600				101	91	200000	8th-2400		100100
	0900				63	53	224000	9th-0900		120700
	1200				39	29	239000	9th-0900		131200
	1500				52	42	255000	9th-0900		142900
	1800				49	39	269000	9th-0900		169500
	2100				39	29	277000	9th-0900		211000
	2400				40	30	286000	9th-0900		270300
9.5.63.	0300				31	21	315000	9th-0900		314200
	0600				13	3	346000	9th-0900		345300
	0900				23	13				348800
	1200				10	0				331500
	1500				19	9				307300
	1800									280100
	2100									253800
	2400									231800
10.5.67.	0300									201000
	0600									181400
	0900									160700

FLOOD FORECAST

Macleay River at Kempsey

January 1964 - Storm 36

Date	Time	API at 9 am	Initial Loss		3 hr. Rainfall		Forecast		Weather Forecast for 12 Hours	Observed Discharge (cusecs)
			Predicted (pts)	Adopted (pts)	Gross (pts)	Excess (pts)	Minimum Peak Discharge (cusecs)	Time of Peak		
13.1.64.	0900	155	135							
14.1.64.	0900	309	70	169	169				Rain easing	
	1200				104	94	62000	15th-1500	"	0
	1500				62	52	93000	15th-1800		0
	1800				3					0
	2100									0
	2400									0
15.1.64.	0300									0
	0600									0
	0900									0
	1200									20700
	1500									41000
	1800									55000
	2100									63500
	2400									61000
16.1.64.	0300									55000
	0600									47000
	0900									40000
	1200									34000
	1500									29000
	1800									25000
	2100									21000
	2400									18000
17.1.67.	0300									15000

FLOOD FORECAST

Macleay River at Kempsey

March 1964 - Storm 37

Date	Time	API at 9 am	Initial Loss		3 hr. Rainfall		Forecast		Weather Forecast for 12 Hours	Observed Discharge (cusecs)
			Predicted (pts)	Adopted (pts)	Gross (pts)	Excess (pts)	Minimum Peak Discharge (cusecs)	Time of Peak		
8.3.64.	0900	299	73							2000
	1200				0					
	1500			74	74					
	1800				60	50	35000	9th-2100		19800
	2100				87	77	84000	9th-2400	Heavy rain	23730
	2400				32	22	97000	9th-2400	Rain easing	31460
9.3.64.	0300				17	7	101000	9th-2400	"	39060
	0600				22	12	110000	9th-2400		48000
	0900				22	12	110000	9th-2400		57700
	1200				10					69700
	1500				5					77200
	1800				2					84200
	2100									91000
	2400									94600
10.3.64.	0300									94600
	0600									91000
	0900									85100
	1200									77800
	1500									71500
	1800									64300

LIST OF REFERENCES

1. Hathaway, G.A. — "Determination of Spillway Requirements for Large Dams". Proc. 4th International Conference on Large Dams New Delhi, January 1951 - Distributed by U.S. Corps of Engineers with Civil Works Engineering Bulletin 50-6 issued 9th June 1950.

2. U.S. Corps Engineers Manual EM 1110-2-3600 — "Reservoir Regulation" 1951 Revised 25th May 1959 - on Engineering and Design.

3. Watson, B. — "The Development of a Flood Forecasting System for the Macleay River at Kempsey, N.S.W.". Hydrology Symposium, April 1962, Hobart, Tasmania.

4. Bureau of Meteorology — "Report on Development of a Flood Forecasting System for the Lower Macleay Valley, New South Wales" May 1963 (4 volumes).

5. Body D.N. and Rainbird A.F. — "Predicting Flood Flows from Rainstorms" from "Water Resources Use and Management", Melbourne University Press, 1963.

6. Langford, D.T.A. — "Radio Reporting Rain Gauge System". Bureau of Meteorology Paper presented to I.R.E.E. (Australia) Convention, Canberra, March 1965.

7. Body, D.N. — "Adjustment to Antecedent Precipitation Values Following Dry Spells". Unpublished Memorandum, Bureau of Meteorology, 1963.