DEVELOPMENT OF AUTOMATED FLOOD OPERATING PROCEDURES FOR ALBERTA DAMS

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Abstract

Automated procedures have been developed for the major provincially owned and operated dams that reduce the risk of downstream flooding during flood events while ensuring the safety of the structures. Because several calculations are required to determine the optimum discharge, previous flood operating procedures required the operator to use tables and graphs and to perform interpolations. To reduce the operator's workload and the probability of committing errors, the procedures were automated, by developing an application which consists of an Excel spreadsheet that calls functions from a pre-compiled dynamic link library (DLL). During each hour of the flood event, the operator enters the reservoir elevation and the spillway gate settings, and, if available, the magnitude and the time of the forecast peak inflow. By synthesizing an inflow hydrograph, either with or without an estimated value for the peak inflow, the application calculates the optimum discharge, which will use the available storage. It is demonstrated that the application is capable of reducing downstream flooding, while preserving the safety of the dam, for a variety of historic floods.

Résumé

En vue d'assurer la sécurité des barrages et de réduire le risque d'inondations en aval pendant les périodes de crue, des procèdes automatises ont été conçus pour les principaux barrages sous tutelle provinciale. Une série de calculs est nécessaire pour déterminer le débit optimum. Auparavant les procèdes opérationnels de crue exigeaient que les opérateurs utilisent des tableaux et des graphiques et qu'ils interpolent les données. Dans le but de réduire leur poids de travail ainsi que la probabilité d'erreurs, les procèdes furent automatises en développant une application consistant d'un tableur Excel qui invoque des fonctions d'un lien de librairie dynamique (DLL) pre-compilé. A chaque heure de crue, l'opérateur prend note de l'élévation du réservoir et de la position de la vanne de l'évacuateur de crue et, si possible, des prévisions de l'ampleur et du moment où sera atteint le débit de pointe entrant. En synthétisant l'hydrogramme du débit entrant, avec ou sans estimation du débit de pointe entrant, l'application calcule le débit optimal qui n'affectera que le volume excédentaire de retenue d'eau. En faisant référence à diverses crues historiques, il est démontre que l'application est capable de réduire les crues en aval tout en préservant la sécurité du barrage.

1. INTRODUCTION

The Waterton, St Mary, Oldman and Dickson Dams are located on the Waterton, St. Mary, Oldman and Red Deer Rivers, respectively, in southern Alberta. These provincially owned dams are zoned earth filled structures, which are considered to be very high consequence of failure structures according to the Canadian Dam Association (CDA) guidelines, and are therefore required to safely pass the Probable Maximum Flood (PMF).

In addition to their primary role, which is to supply water for irrigation and other conservation purposes, Alberta Environment structures are operated to mitigate flood risks to people and property by reducing downstream flood peaks. To achieve this goal, the River Forecasting Centre of Alberta Environment is required to establish detailed procedures to safely operate the structures during flood events. The development of these procedures, and the tool created to automate their use is described in this paper

2. DEVELOPMENT OF FLOOD OPERATING PROCEDURES

The flood operating procedures require many calculations, as well as looking up values in tables, for each hour of a given flood event. This is a time-consuming exercise and prone to mistakes when done manually in the course of an event. A Microsoft Excel spreadsheet has therefore been developed for each dam to automate these computations. The spreadsheet has two main functions:

1. It automates the calculation of inflows from measured discharges and reservoir elevations, which eliminates the use of cumbersome tables, plots and computational interpolation during a flood event.
2. It calculates the optimum discharge, which minimizes the downstream flooding while obeying the rules of operation.

To determine the optimum discharge, the available reservoir storage at each dam is divided into three zones (see Figure 1).

1. Freeboard. This small (approx. 1 m) zone is located between the top of the flood pool and the top of the dam. It is set aside to prevent wind and wave action from overtopping the dam.

2. Flood pool for PMF routing. Because the spillways of the dams are unable to directly pass their respective PMF, a portion of the storage must be set aside to route these large flows. This is the zone between the full supply level (FSL) and the top of the flood pool.

3. Flood control pool. This is simply the storage available between wherever the reservoir level happens to be, and FSL. Because the reservoir elevation fluctuates, the amount of storage varies. It is also the storage that is available for mitigating downstream flooding.

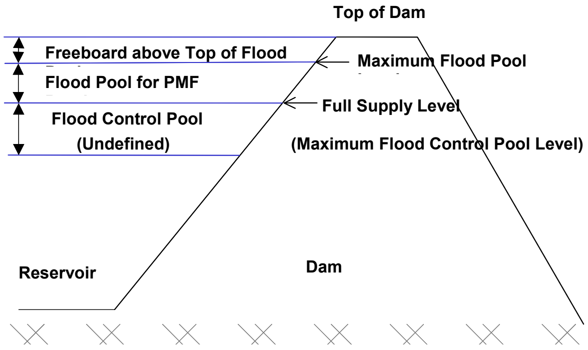

Figure 1. Schematic diagram of significant reservoir levels

If the reservoir elevation is ever inside the Flood Pool zone required for managing the PMF, then the operator must set the discharge to be as close as is physically possible to the inflow. This storage region exists only to protect the structure. When the reservoir elevation is in this zone, it must be brought down to the bottom of the PMF Flood Pool as quickly as possible.

If the reservoir elevation is below the bottom of the Flood Pool zone, then the available storage between the current elevation and the bottom of Flood Pool Storage can be used to reduce the peak flow downstream of the dam. Taking advantage of the storage in this region requires calculation of the optimum discharge, which is the outflow rate for which the required storage is exactly equal to Flood Control Pool Storage. Calculation of the required storage requires knowledge of the hydrograph of inflows to the reservoir.

2.1 Determining Inflows

By continuity,

$$\bar{I}_{(t)} = \bar{O}_{(t)} + \frac{\Delta S_{(t)}}{\Delta t} \qquad [1]$$

where:

$\bar{I}_{(t)}$ = mean inflow to reservoir over time interval Δt,

$\bar{O}_{(t)}$ = mean outflow from reservoir over time interval Δt, and

$\Delta S_{(t)}$ = change in reservoir storage over time interval Δt.

For flood operations at Alberta dams, the time interval has been fixed at one hour.

The mean inflow over a time period is calculated as a residual from Eq 1, based on three assumptions (Simons et al.)

1. Evaporation and seepage losses are negligible.

2. The stage-storage relationship does not change over time (i.e. the effects of sedimentation are negligible).

3. The time lag from the reservoir inlet to the outlet is negligible.

The first two conditions are satisfied by the flashiness of the reservoir flood events, which generally peak in only one or two days. The third condition is satisfied by the relative shortness of the Alberta Reservoirs. The Waterton reservoir, for example is approximately 7 km in length.

2.2 Modeling Inflow Hydrographs

The reservoir inflow forecasts issued by the Alberta Environment River Forecasting Centre to the dam operators generally consist of the time and value of the peak inflow, rather than a complete hydrograph. To convert the forecast peak to a complete inflow hydrograph, the inflow values occurring between the current time and the peak time are linearly interpolated. Because the rising limbs of inflow hydrographs tend to be concave, this is a slightly conservative assumption.

The shape of the recession limb of an inflow hydrograph is caused by the rate at which runoff drains from the watershed area upstream of the dam. Because the rate of drainage depends on the characteristics of the basin, the recession curve tends to be very similar for all events in a single watershed (Gray 1973). Therefore, given the peak inflow, it is possible to estimate the recession limb, by using curves derived from other events.

During a flood event, it is expected that the dam operators will always have an up-to-date forecast of the peak inflow. Operating the dam with a forecast provides for the optimal use of available storage to reduce downstream impacts, while preserving the integrity of the structure. In the event that a forecast of the peak inflow is not available, it is possible to operate the dam safely. However, this method

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of operation provides less reduction of the peak inflow and should never be used if a forecast is available.

In the absence of a forecast of peak inflow to the reservoir, the operator's only information about the inflow hydrograph is the calculated current inflow to the reservoir. The without-forecast inflow hydrograph is based on the assumption that the current inflow is the peak inflow, which obviously is not the case most of the time. Fortunately, the procedure is self-correcting. In the next hour, the new optimum discharge will be calculated, that takes into account the new reservoir elevation, inflow and release. Eventually, the current inflow will be the peak inflow, and the calculated optimum discharge will be correct. At this point, all of the remaining storage will be used to mitigate the peak inflow.

This method is slightly different from that of Lewin and Denham (1983) who extrapolated inflows to assume a peak inflow and applied a linear recession limb. Although assuming that the current inflow is the peak wastes storage on the rising limb of the inflow hydrograph, it allows for accurate determination of the optimum flow when the peak actually occurs. In contrast, Lewin and Denham's algorithm (assuming it used an accurate recession limb) preserves storage on the rising limb but will incorrectly calculate the optimum discharge for the peak inflow.

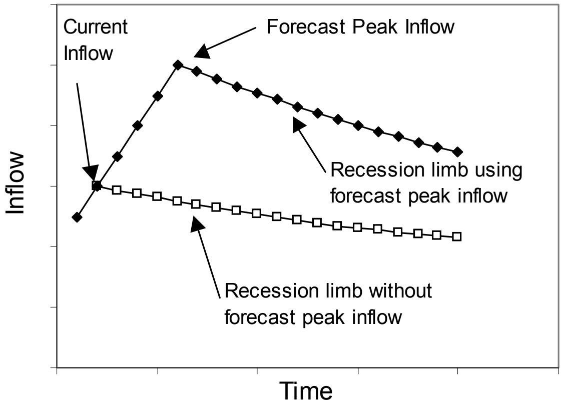

Figure 2. Estimation of future inflows to reservoir (a) with a forecast of peak inflow and (b) without a forecast.

2.3 Calculating Optimum Discharge

Given an assumed inflow hydrograph, it is necessary to calculate the optimum discharge that minimizes the peak downstream while preserving the integrity of the structure.

Equation 1 can be written in finite difference form as:

$$\frac{I_i + I_{i+1}}{2} = \frac{O_i + O_{i+1}}{2} + \frac{S_{i+1} - S_i}{t_{i+1} - t_i}, \qquad [2]$$

where i is a given time step.

Given the assumed inflow hydrograph, it is necessary to compute the optimum discharges subject to the following constraints:

1. On the rising limb, once the inflow have passed the point where flooding begins downstream of the dam, the discharges cannot exceed inflows. Before inflows have reached the downstream flood level, the operators are free to pre-release. However, because the inflows are calculated from the known outflows and the measured changes in storage, the calculated mean inflow value will always lag the set outflows by one time step. Therefore:

$$O_{(i+1)} \le \frac{I_{(i)+}I_{(i-1)}}{2}\,. \qquad [3]$$

2. The rate of discharge is limited primarily by the spillway capacity (C), which is a function of reservoir stage (E), which, in turn, is a function of storage (S). Therefore,

$$O_{(i)} \le C(E(S_{(i)})) + G_{(i)} + T_{(i)}\,, \qquad [4]$$

where:
$G_{(i)}$ = Generator discharge, and
$T_{(i)}$ = Tunnel discharge.

The optimisation must also take into account two effects, which alter the set discharges:

1. When the spillway is in use, the discharge from the spillway will change over the time interval from the value initially set by the operator, as the head on the spillway gates changes.
2. Operation of the spillway gates takes a finite period of time, during which flow is in transition from the previous discharge to the newly set discharge. If the spillway gates are not required to be operated during a time interval, for example if all the gates are open and are to remain open, then this effect must be omitted.

Because all of the factors that influence the optimum discharge change each hour (the reservoir elevation, the inflow hydrograph, the constraints in Equations 3 and 4, the alterations of the discharge) the optimum discharge will also change each hour.

All of the relationships required by the optimization (inflow hydrograph, spillway gate rating curves, reservoir volume-elevation relationships) are stored as sets of points, which are generally interpolated linearly. Therefore, a simple analytical solution of the optimum discharge is not possible, and a numerical solution was required. Unfortunately, because of the very large number of calculations required, the numerical solution was very slow using the spreadsheet's built-in programming language. Instead, the optimization code was compiled to functions within a dynamic link library (DLL), which are called by the spreadsheet as required. This results in a system that executes 10 to 30 times faster. Another advantage is that the optimisation code can be used by other programs, such as hydrologic models to route floods through the reservoirs.

2.4 Manual Operations Curves

Manual Operation Curves were developed to allow safe operation of the dam during a flood event, without the use of a computer. The curves were developed using a variation of the Operator's Spreadsheet, which calculated the optimum discharge for a wide variety of inflows, outflows and reservoir elevations.

Since the Manual Operation Curves are only to be used in case of failure of both the local computer and all communications systems, they are based on the without-forecast algorithm. Using the curves will result in discharges which are conservative but which will reduce the downstream discharge while protecting the integrity of the dam.

A typical manual operations curve, for a given range of inflows, is shown in Figure 3. The graph is divided into three regions. The left-hand (shaded) region of the graph is not used, as it would require the current discharges to exceed the spillway capacity, which is physically impossible. In Region 1 (the top region of the graph labelled "Hold Current Discharge"), the optimum discharge is less than the current discharge. In Region 2 (the bottom region labelled "Get New Discharge"), the operator calculates the new required discharge using the optimum discharge curve, which is the line dividing the two regions of the graph.

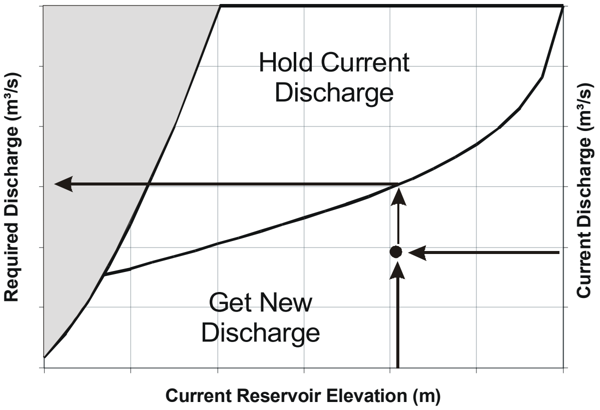

Figure 3. Typical manual flood operation curve.

3. SIMULATION RESULTS

As described below, the optimisation algorithms were tested by routing flood events. However, the inflow hydrographs of historical events are generally available as recorded flows, while the spreadsheet requires records of reservoir elevation, which also depends on the spillway discharge. This required the reservoir elevation for each time step to be solved numerically, before the optimum discharge is calculated.

The advantage of using forecasts to calculate outflows is demonstrated in Figure 4. It plots the releases from the Waterton reservoir for the estimated 100-year flood using both methods of operation. In each case the reservoir was initially set at an elevation of 1.4 m below FSL, with the initial outflow set equal to inflow.

In the non-forecast simulation, the outflow is consistently much less than the inflow, because the reservoir storage is being used to minimise the assumed peaks, which occur each hour. The result is that when the actual peak does occur, there is far less storage available to manage it. In this example, the non-forecast method reduced the downstream peak from 1044 m^3/s to 998 m^3/s – a reduction of 4%.

On the other hand, when the forecast peak is used, the reservoir outflows more closely follow the inflows. The outflow values lag the inflows by an hour until the inflow is greater than or equal to the optimum value, at which point the outflow is set equal to the optimum. This results in more storage being available to reduce the flood peak. In the example, the with-forecast method reduced the downstream peak to a value of 749 m^3/s, which is a 28% reduction. Not using the forecast wastes the reservoir storage by premature storing of the rising limb instead of the peak.

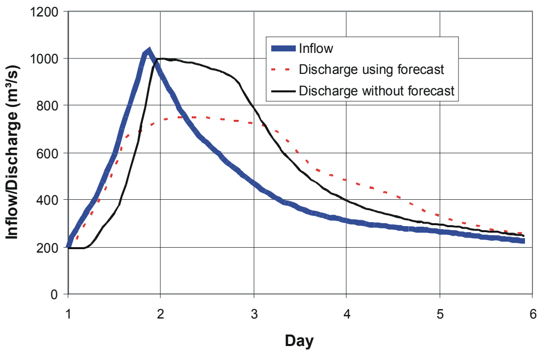

Figure 4. 100 Year Flood Hydrographs with and without Forecasts

3.1 Imperfect Forecasts

The previous example used a perfect flood forecast. However, the with-forecast method also works when the forecast peak inflow is in error. Figures 5 and 6 demonstrate the way the technique deals with under-estimation and over-estimation, respectively. Once again, the 100-year flood hydrograph was used in the simulation tests.

In the first case, which is plotted in Figure 5, the peak inflow was forecast to be 600 m^3/s, which is a 43% underestimate of the peak flow of 1044 m^3/s. In the first part of the event, the system operates as though the forecast were accurate. When the inflow exceeds the forecast peak, if there is no updated forecast, the forecast is considered obsolete and is no longer used. From this point, the dam is operated without a forecast as described above. The overall result is a combination of the two methods. The peak outflow in this example was 939 m^3/s, which was greater than when using a perfect forecast, but was still better than when using the without-forecast method.

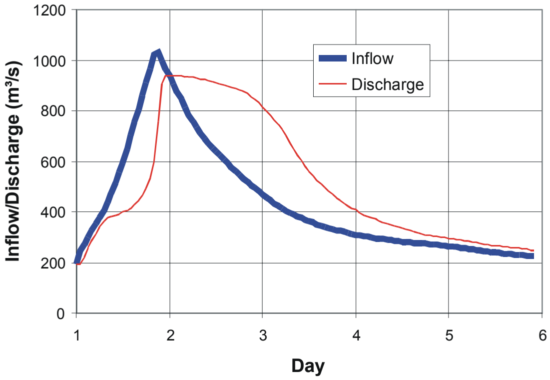

Figure 5. 100 Year Flood Hydrograph with a Too-Low Forecast of Peak Inflow.

In the second case, which is plotted in Figure 6, the flood peak was incorrectly forecast to be 1500 m³/s. When the inflow peak was found to be much less than was forecast, the procedure switched to the without-forecast method. The peak discharge was 910 m³/s, which is slightly less than that obtained by ignoring the peak forecast. The with-forecast method has the additional advantage of being more conservative by saving storage to mitigate any subsequent peaks.

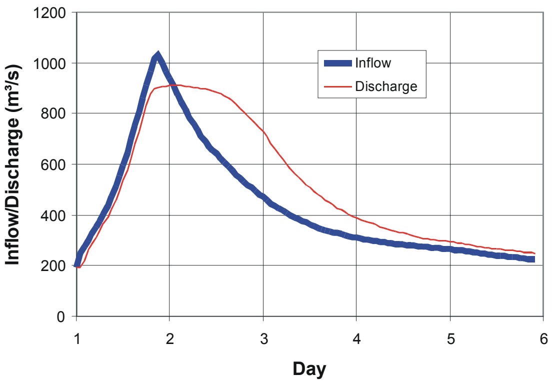

Figure 6. 100 Year Flood Hydrograph with a Too-High Forecast of Peak Inflow

3.2 Routing an actual event

To test the ability of the flood operating procedures to handle an actual event, the flood of June 5, 1995 was routed using the Waterton Operator's Spreadsheet. The peak hourly inflow to the reservoir during this event was 1212 m^3/s, which is the largest inflow on record for the reservoir. Because the reservoir was not operated hourly during this event, it was not possible to directly input the reservoir elevations and discharges recorded during this event. The inflows to the reservoir were reconstructed using a computer model.

As discussed above, the Waterton Operator's Spreadsheet is able to route a specified hydrograph through the reservoir. Using this, the 1995 flood was routed though the reservoir. The results of this simulation are shown graphically in Figure 6. In brief, the spreadsheet successfully handled the event without any errors and reduced the flood peak. With a perfect forecast, the spreadsheet simulation shows that outflow peak could have been reduced to 733 m^3/s. In the absence of a forecast, the peak outflow would have been 815 m^3/s.

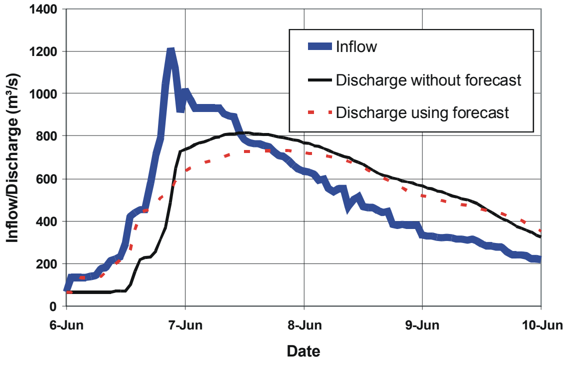

Figure 7. Routing the June 1995 Flood of Record through the Waterton Reservoir

4. CONCLUSIONS

Spreadsheets have been developed to automate the operation of the major government-owned and operated dams in Alberta. By automating the calculation of optimum discharges, the spreadsheets reduce the operator's workload, while ensuring the safety of the structure and reducing flooding downstream of the dam. The spreadsheets function with or without a forecast of the peak inflow and are able to cope with inaccurate forecasts. In case of failure of the operator's computer and communication systems, a simplified manual system may be used.

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