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MODERNIZATION OF BC HYDRO'S INUNDATION MODELLING AND MAPPING: A HIGH RESOLUTION, FLEXIBLE AND RESPONSIVE APPROACH

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ABSTRACT:

This paper describes BC Hydro's program to update the flood inundation models and maps for all of its dams to improve the information available for emergency planning and response. The program uses high resolution hydrodynamic models to assess flow release scenarios ranging from high spillway discharges to dam breach. BC Hydro has 41 dams and most of the dam breach inundation modelling and mapping was completed in the 1980s following guidelines that no longer represent the state of technology or practice. Furthermore, the extent of inundation for high spillway discharges are currently not well understood for many projects for different combinations of downstream tributary and lake conditions. The program is guided by recent research and advancements in the areas of broad-scale data collection techniques, dam breach analysis, one-dimensional and two-dimensional hydrodynamic modelling for flood routing, inundation mapping procedures and uncertainty assessment. A key requirement for this project is that the hydrodynamic models and procedures used to update the inundation maps for any river system are flexible and allow for a responsive approach so that additional scenarios can be quickly assessed given forecast information in an emergency situation. An objective of the program is that in an emergency, scenarios can be run and inundation maps produced within 24 hours of notice to be available in GIS electronic format for external emergency planning agencies. Some of the strategies employed and challenges encountered thus far in this inundation modelling and mapping project are also presented.

RÉSUMÉ:

Cet article décrit le programme de BC Hydro afin d'actualiser les modèles d'inondation de crue et les cartes pour tous ses barrages et d'améliorer l'information disponible pour la planification et la réponse en cas d'urgence. Le programme utilise des modèles hydrodynamiques à haute résolution pour évaluer des scénarios de crues allant de déversements élevés par évacuateurs de crues aux ruptures de barrages. BC Hydro a 41 barrages et la majorité de la modélisation et cartographie d'inondation par rupture de barrages a été accomplie au cours des années 1980 en utilisant des guides ne représentant plus l'état de la technologie ou de la pratique. De plus, l'étendu des inondations causées par de grandes décharges par évacuateurs n'est pas bien comprise pour plusieurs projets pour différentes combinaisons d'affluents en aval et conditions de réservoirs. Le programme est guidé par de récentes recherches et avancements dans les domaines suivants : techniques de collecte de données à grande échelle, analyse de rupture de barrage, modélisation hydrodynamique unidimensionnel et bidimensionnel de détournement de crues, méthodes pour cartographier les inondations, et évaluation d'incertitudes. Une exigence clé pour ce projet est que les modèles hydrodynamiques et les procédures utilisées pour actualiser les cartes d'inondation pour n'importe quel système riverain soient flexibles et permettent que des scénarios supplémentaires puissent être évalués rapidement en situation d'urgence selon des informations prévisionnelles. Un des objectifs du programme, lors d'une situation d'urgence, est que les scénarios peuvent être évalués et les cartes d'inondations produites avec 24 heures de préavis et être disponibles dans le format électronique GIS pour les agences de planification d'urgence externes. Quelques stratégies employées et défis rencontrés jusqu'ici dans ce projet de modélisation et de cartographie sont aussi présentés.

1. INTRODUCTION

Inundation mapping of regions downstream from dams is required to better understand the consequences of project spillway and dam breach outflows and for effective emergency management. Inundation mapping provides information on the extent and magnitude of flooding as well as key input for developing warning and evacuation strategies.

Dam breach modelling and inundation mapping studies for most of BC Hydro's projects were completed in the 1980s, however there has been no previous system-wide program to map downstream areas impacted from high spillway discharges. Since the 1980s, there has been significant improvement in hydrodynamic modelling and mapping techniques allowing for higher resolution analysis than previous studies and improved consideration for downstream land use changes. Also, there has been considerable work devoted to improving the understanding of dam breach development.

The project to modernize BC Hydro's inundation maps has been termed Flood Simulation and Mapping Model, or FloodSiMM. The scope of the FloodSiMM project includes development of hydrodynamic models and production of updated inundation maps for all main dams for select breach scenarios and spillway discharge events.

In addition, it has been recognized that there is a need for an "on-demand" or responsive reservoir routing and inundation mapping model for each system to improve emergency response capability. To achieve this, modelling tools should be flexible enough to map downstream inundation areas on a near-real time basis for any combination of predicted project outflow and downstream tributary conditions for any of the river systems. Therefore, given forecast information in an emergency, it is an objective of the FloodSiMM project that updated inundation maps for any river system can be provided to external emergency planning agencies within 24 hours of notice of an emergency.

This paper provides an overview of the FloodSiMM project including discussion of the methodology and some of the challenges involved in producing flexible and computationally efficient hydrodynamic models and updated inundation maps for a wide range of flow scenarios.

2. RIVER SYSTEMS APPROACH

BC Hydro operates 30 hydroelectric generating stations utilizing 41 main dams. The dams range in size from the 183 m high and 2042 m long W.A.C. Bennett Dam, which is an earthfill dam on the Peace River in northern British Columbia, to the 15 m high and 41 m long Spillimacheen Dam, which is a concrete gravity dam in southwest British Columbia. For the FloodSiMM project, these dams have been grouped into 26 river systems. This grouping reflects the fact that inundation models are to be developed for river systems, which may consist of one or more dams, rather than for just individual dams.

The study areas downstream of these dams include more than 1,600 km of rivers and reservoirs. The rivers vary in width and morphology from shallow gradient rivers greater than 500 m wide to incised rocky channels in steep canyons that are less than 20 m wide. The topography of adjacent floodplain areas considered for inundation modelling and mapping are also highly variable. For example, overbank flooding due to dam breach through steep, incised canyon rivers will be quite limited while in other cases very wide, flat floodplain areas greater than 10 km wide could be inundated under spillway discharges. An additional consideration in developing models for inundation consequence assessment is the mixed land use which can include highly populated urban areas, recreational sites with variable vegetation density and agricultural fields. The effects of roads, railways, dykes, bridges, culverts, ditches and other floodplain features which may influence inundation also need to be considered, although a compromise is often required between modelling detail and computation time. In general, the high degree of variability in river and floodplain areas poses both data acquisition and model development challenges which are discussed later in this paper.

3. INUNDATION MODELLING AND MAPPING METHODOLOGY

3.1 Background

Most of the existing inundation mapping for BC Hydro's dams was completed in the 1980s following guidelines developed at that time which prescribed the use of the U.S. National Weather Service's one-dimensional DAMBRK model. The DAMBRK model was used to compute the outflow hydrograph through the breach and perform the routing through the downstream valley. Rather than physically simulate the processes involved in a dam breach, DAMBRK treated the breach process parametrically with the final breach shape, size and failure time being key inputs provided by the user.

While the DAMBRK model was able to simulate the failure of both concrete and earthfill dams as well as perform the downstream routing, the program has several limitations which led to many approximations in the previous modelling studies. Some of the limitations include:

- Inability to simulate flood wave propagation through multiple channels;
- Inability to simulate multiple breaches in the same reservoir;
- Supercritical and subcritical reaches have to be run separately;
- The one-dimensional approach may not be suitable for some applications where two-dimensional effects such as super-elevation around a bend, or flow through a floodplain may not be adequately represented;
- The output from DAMBRK does not allow for the graphical visualization of flood propagation, which is possible with more detailed two-dimensional hydrodynamic models; and,
- The output from DAMBRK is difficult to incorporate into life safety risk models such as the Life Safety Model (Johnstone et al., 2005).

The inundation mapping guidelines from the 1980s indicated that specific information such as peak elevations, flood arrival and peaking times should be given at selected locations. While this information is of value, other important parameters such as flood depths and the product of depth and velocity, which can be used to assess toppling thresholds for people and withstand thresholds for buildings, can aid in risk assessment and emergency planning. Unlike DAMBRK, two-dimensional models allow all of these parameters to be extracted both spatially and temporally and be presented on inundation maps.

The guidelines also recommended that the maximum extent of flooding be shown on contour maps at a scale of 1:50,000 or larger. Most of the previous inundation simulations were based on older (~ 1950 to 1975) Energy and Mines maps. These maps, which were 1:50,000 scale, provided topographical features and some development information. In recent years, the use of remote sensing techniques such as aerial photogrammetry and LiDAR has greatly improved the ability to collect accurate topographic data over large areas. Reasonable vertical accuracy of data collected through these methods is approximately ±0.25 m, and even better in some cases. Therefore, the fidelity of the terrain models and also other topographical features shown on the previous inundation maps can be updated based on more recent data collection and cadastral databases.

The modernization of inundation maps requires that the techniques for data collection, assumptions for dam breach analysis, approaches for hydrodynamic modelling and standards for inundation mapping are consistent with current best practices and are also commensurate with the consequences of dam failure. The methodology for inundation modelling and mapping in the FloodSiMM project is presented below while the overall process is summarized in Figure 1.

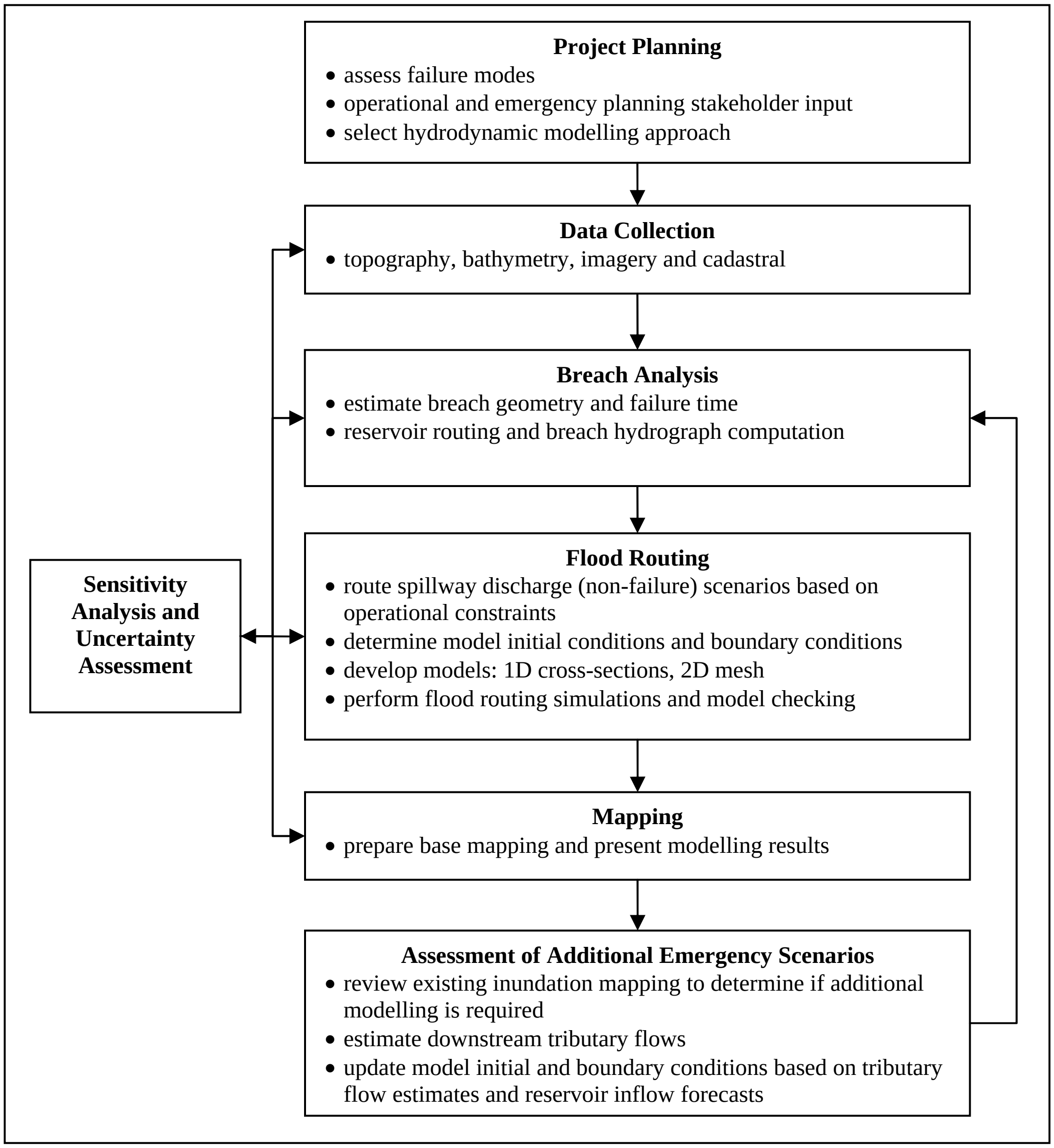

Figure 1: FloodSiMM inundation modelling and mapping process

3.2 Data Collection

For dams with a higher consequence of failure that could affect downstream populations and key infrastructure, the vertical and horizontal accuracy of topographic data used for inundation base mapping and also for hydrodynamic modelling should be approximately ±0.25 m or better. Typically, this level of accuracy and required spatial coverage can be obtained from either photogrammetrically-derived digital elevation data using 1:10,000 scale photos taken with a camera focal length of approximately 150 mm or from Light Detection and Ranging (LiDAR) surveys. For both methods, the data accuracy diminishes in areas with dense vegetation and in the case of photogrammetric data, shadows from surrounding terrain and trees can affect the interpretation of elevations from aerial photography.

Photogrammetric DEMs (Digital Elevation Models) often include points which define key linear features within the stream corridor and in the floodplain, such as river banks, dykes, roads and railways. Although LiDAR DEMs may not distinguish features such as dykes and railway embankments separately, the greater density of points that are collected (up to several points per square meter) usually captures the important floodplain elements which can influence inundation. While both methods are capable of providing high resolution elevation information for inundation modelling and mapping, LiDAR DEMs often require thinning algorithms to be applied to create more manageable data sets to use for hydrodynamic model development.

In areas where minimal overbank flooding would be expected and / or the consequences of failure are minor, topographic data from publicly available sources such as 1:20,000 scale BC Provincial TRIM (Terrain Resource Information Management) maps or 1:50,000 scale NTS (National Topographic System) maps can be considered. The accuracy of these data sets are in the ±5 m to ±10 m range.

Bathymetric data in rivers are usually obtained from a boat survey with depth sounder linked to GPS. Conventional surveying techniques are used for shallower, lower velocity channels where boat accessibility and navigation are not possible. Consideration is also given to whether a one-dimensional (1D) or two-dimensional (2D) hydrodynamic modelling approach is to be used. Cross-sections or transects are preferred for 1D model development while more detailed bathymetric data that includes coverage near banks and edges of islands and bars can provide a better three-dimensional representation of the channel bottom for 2D models.

3.3 Dam Breach Analysis

3.3.1 Failure Modes

An assessment of failure modes is typically done near the beginning of dam breach inundation studies to help determine the breach characteristics and the assumptions for dam break flood routing. To standardize the approach for modernizing inundation maps as part of the FloodSiMM project, hypothetical failures of all main dams due to both extreme floods and earthquakes are considered.

For earthfill dams, failure due to an extreme flood is assumed to occur at the maximum reservoir level during routing of the Inflow Design Flood (IDF), which for higher consequence dams is the Probable Maximum Flood (PMF). Reservoir inflows and downstream tributary flows concurrent with IDF or PMF conditions are assumed. Failure in these scenarios could be due to either overtopping or internal erosion since the adopted approach for breach parameter prediction described in Section 3.3.2 generally does not differentiate between failure modes. An earthfill dam failure due to an earthquake, also referred to as a sunny day failure, is considered at both the maximum normal and average reservoir levels with seasonally consistent corresponding reservoir inflows and downstream tributary inflows.

A stability assessment of concrete dams is performed by structural engineers to provide the breach geometry for the inundation studies. The level of detail in these analyses is commensurate with the emergency planning focus of the exercise, i.e. a relatively high level, conservative assessment of potential breach geometries. Therefore, it is not necessary to calculate the actual loading conditions that could cause failure. Instead, the blocks or sections of the dam that are deemed weakest under both the maximum reservoir level during routing of the IDF and due to an earthquake are identified and are assumed to fail. Assumptions for reservoir inflows and downstream tributary flows are similar to those for earthfill dams.

Dam breach analysis of composite dams, which include both earthfill and concrete sections, must consider failure of the portion of the dam that would produce the largest peak outflow.

3.3.2 Breach Parameters for Earthfill Dams

The estimation of dam breach parameters represents one of the largest sources of uncertainty in dam break flood inundation modelling. Although recent research has improved the understanding of the physical processes involved in earthfill dam breach formation, numerical modelling which incorporates both soil mechanics and hydraulics principles is still in the development phase. Investigators are currently working towards refining existing embankment dam failure simulation software using breach test data from laboratory and field experiments for model validation. Some of the key research initiatives in the area of earthfill dam breach prediction include CADAM (Morris, 2000), IMPACT (Morris et al., 2005), the collaborative effort currently underway between the FLOODSite and Dam Safety Interest Group projects (Morris et al., 2008) and on-going research conducted by the U.S. Department of Agriculture (Temple et al., 2006).

In the interim, while numerical breach prediction tools are being refined, agency guidelines and empirical equations developed from documented dam failures (Wahl, 1998 and Wahl, 2004) are often used to estimate breach widths and formation times. BC Hydro's current strategy for earthfill dam breach analysis includes the use of four dam breach parameter prediction equations, shown in Table 1, to provide a range of possible breach sizes and failure times, combined with geotechnical engineering input and findings from some of the recent research noted above.

Table 1: Breach Parameter Prediction Equations for Earthfill Dams

Reference	Breach Width	Failure Time
1. USBR (1988)	$B_{avg} = 3h_w$	$t_f = 0.011B_{avg}$
2. MacDonald and Langridge-Monopolis (1984)	$V_{er} = 0.0261(V_w h_w)^{0.769}$	$t_f = 0.0179V_{er}^{0.364}$
3a. Von Thun and Gillette (1990) (highly erodible)	$B_{avg} = 2.5h_w + C_b$	$t_f = 0.015h_w$ $t_f = B_{avg}/(4h_w + 61)$
3b. Von Thun and Gillette (1990) (erosion resistant)	$B_{avg} = 2.5h_w + C_b$	$t_f = 0.02h_w + 0.25$ $t_f = B_{avg}/(4h_w)$
4. Froehlich (1995)	$B_{avg} = 0.1803K_0V_w^{0.32}h_b^{0.19}$	$t_f = 0.00254V_w^{0.53}h_b^{-0.9}$

B_{avg} = average breach width (m)
C_b = offset factor, varies with reservoir storage (e.g. 54.9 for reservoir storage > 1.23×10^7 m^3)
h_b = height of breach (m)
h_w = depth of water above breach invert at time of failure (m)
K_o = overtopping multiplier: 1.4 for overtopping, 1.0 for piping
t_f = failure time (h)
V_{er} = volume of embankment material eroded (m^3)
V_w = volume of water stored above breach invert at time of failure (m^3)

The potential failure modes of earthfill dams are studied on a case by case basis with appropriate sensitivity analysis and uncertainty assessment of the breach parameters. A range of possible breach parameters should be considered in order to understand how the variability in the outflow hydrograph (Section 3.3.3) affects the magnitude and extent of downstream inundation. The assessment of uncertainties is an important aspect of dam breach inundation studies since the historical dam failure data used to develop the relationships shown in Table 1 may or may not be representative of other dams and failure scenarios. For example, the database of dam failures is lacking information from large dam failures with about 75% of cases having a height less than 15 m (Wahl, 1998).

The breach depth is usually taken as the height of the dam with the bottom elevation of the breach corresponding to either bedrock or the river bed at the downstream toe of the dam. While most previous dam breach studies have assumed that the shape of an earthfill dam failure is trapezoidal, with side slopes of 1:1, recent field and laboratory testing has indicated that breach side walls are typically vertical during breach development and the trapezoidal shape develops after the breach formation process when embankment material dries and slumps (Morris et al., 2005). Therefore, the breach shape is simply taken as a rectangular section, with the average breach width, B_{avg}, estimated from the empirical equations equal to the total width of the breach section.

Although the failure modes for earthfill dams include overtopping, piping and sunny day (e.g. earthquake) failures, the majority of research to date has focussed on overtopping failures. Additional research is required to improve the understanding of the processes involved with failures initiated by piping. Furthermore, it is difficult to predict how a breach would develop in a given dam as a result of seismic activity, since the breach would be magnitude-dependent. An earthquake could potentially cause almost instantaneous lowering of an embankment dam crest, as in the Lower San Fernando Dam failure following a magnitude 6.7 earthquake in 1971. Such a crest collapse could lead to further breach development due to overtopping depending on the reservoir level at the time of the earthquake.

Since geotechnical engineering input specific to the dam being considered is incorporated in the estimation of breach parameters, in some cases the assumed breach size may differ from that estimated from the relationships shown in Table 1. An example would be a dam that is comprised of highly erodible soils which may be expected to have a substantially larger breach than predicted by empirical equations given a certain magnitude and duration of overtopping.

Additional considerations involve the potential differences in breach formation processes between earthfill and rockfill dams. The empirical equations in Table 1 do not distinguish between earthfill and rockfill dams, although slightly modified equations for breach width and failure time estimation of rockfill dams are provided in USBR (1988) and MacDonald and Langridge-Monopolis (1984). Since the mechanics of breach formation in a rockfill dam can be different than in an earthfill dam failure, the preferred method of analysis would be to employ a numerical tool that considers key factors such as material size, gradation, composite structure if not homogeneous and soil properties. As part of the interim strategy until such a tool becomes available, the approach described for earthfill dams is also applied for rockfill dams. However, additional guidance on the stability of rockfill dams is provided by Leps (1973), Olivier (1967) and Solvik (1991).

The interim strategy used by BC Hydro for embankment dam breach analysis follows the approach described above for estimating the breach parameters for all failure modes (overtopping, piping, sunny day) combined with geotechnical engineering input and findings from recent research. The initial hydrologic conditions that should be considered when performing a dam breach analysis and subsequent downstream flow routing vary with the assumed failure mode, as described in Section 3.3.1.

3.3.3 Outflow Hydrograph Computation

A range of estimated breach widths and failure times that represent conservative yet physically realistic breach outcomes is used as input to a one-dimensional hydrodynamic model such as MIKE 11 to compute outflow hydrographs. This technique treats breach development parametrically rather than attempting to simulate any of the governing complex physical processes. Breach growth, in this case, starts at a user specified reservoir elevation and continues until the maximum size is reached corresponding to the specified breach formation time. The timing of the modelled peak flow through the breach will usually coincide with the maximum breach size, unless the reservoir storage volume limits the peak outflow before the breach is fully developed.

Chauhan et al. (2004) noted that this approach may be quite conservative for dams with large reservoirs since the database of historic dam failures used to develop the regression equations are based on final breach dimensions which are the result of passing the entire breach hydrograph, including the falling limb, through the breached section. Observations of actual dam failures and model studies have indicated that breaches continue to grow during the falling limb of the hydrograph indicating that the peak flow would be expected to occur prior to the breach reaching its maximum size.

3.4 Hydrodynamic Modelling

3.4.1 General

The methods for applying computational hydrodynamic models to simulate flood propagation and inundation extents are generally much better understood as compared to dam break parameter estimation. While the scientific principles involved with the mathematical representation of water flowing through a river valley or floodplain are already well established, there exists residual uncertainty in dam break flood routing due to the potential for large morphological changes and debris and sediment-laden flow effects.

Practically, there are three types of hydrodynamic models that can be used for the downstream routing of dam break outflow hydrographs for inundation modelling: one-dimensional (1D) models, two-dimensional (2D) models and hybrid models. The key input into any hydrodynamic model is the data that describes the bathymetry and topography of the modelled domain as well as the hydraulic boundary conditions.

One-dimensional models use cross-sections that are spaced along the river in an attempt to capture the spatial variation in the terrain which may influence the propagation of a flood wave. The cross-section data required for 1D models can either be surveyed in the field at specified locations or can be extracted from a DEM. One-dimensional models assume a uniform velocity distribution and a horizontal water surface at each cross-section so any centrifugal effects due to channel curvature are ignored. While 1D models are relatively computationally efficient, there are some limitations which require consideration during the model selection process. The spreading of a flood wave across a floodplain may not be accurately simulated by a 1D model since flow in the floodplain is assumed to be parallel to the channel. Also, there is no simulation of physical processes between modelled cross-sections. Therefore, flood inundation maps are produced by superimposing the predicted water levels over a DEM and interpolating the results between cross-section locations.

Unlike one-dimensional approaches, 2D models can simulate lateral variations in velocity and can account for superelevation of the water surface around bends. Velocities, which are depth-averaged, are computed in two horizontal, orthogonal directions. The channel bathymetry and floodplain topography is represented as a three-dimensional surface by a DEM, or model mesh. The topographic data necessary to build a 2D model can be extensive, especially if large floodplain areas are to be considered. However, broad area mapping techniques such as LiDAR or aerial photogrammetry can be used to provide the topographic data with adequate spatial coverage and accuracy.

The modelled area is discretized using either structured or unstructured elements that are either triangular or quadrilateral in shape, depending on the particular 2D model. Results from a 2D model are generally easier to visualize and model output can potentially be incorporated into life safety risk models which may require spatially distributed depth and velocity information. However, due to their numerical formulation and the size and discretization of the modelled domain, 2D models can be very computationally intensive which can lead to long execution times, sometimes in the order of days. Therefore, a key consideration in 2D model development is balancing the requirements for sufficient detail in the model mesh to adequately represent all necessary bathymetric and topographic features, while being able to run the model in a reasonable length of time given available computational resources.

Hybrid models offer a potential compromise between 1D and 2D models by dynamically coupling the two model types. Hybrid models simulate channel flows using a one-dimensional cross-sectional approach while floodplain flows are treated with two-dimensional model domains. By dynamically linking 1D and 2D flow simulation in one software package, the resolution of 2D grids does not have to be refined to represent channels thus increasing model efficiency.

3.4.2 Hydrodynamic Model Evaluation

In the early stages of the FloodSiMM project, several 2D and 1D2D hybrid hydrodynamic models were evaluated to select the most appropriate software to meet the project objectives. The evaluation software included Telemac2D, MIKE 21, MIKE FLOOD, SOBEK (both 2D and hybrid versions) and InfoWorks RS (both 2D and hybrid versions). Software versions that were current as of mid-2008 were tested using a common study area. The primary evaluation criteria was computing time but consideration was also given to other factors including numerical robustness (i.e. stability), pre- and post-processors, mesh generation features, compatibility with other relevant software and customer support.

Care was taken to develop models with a similar density of computational nodes to allow as direct a comparison as possible. All models were tested on a 3.4 GHz single processor PC. Since some of the models are capable of running on multiple processors, a 2.1 GHz dual processor PC and also a dual quad server consisting of 8 - 3.16 GHz processors were also used for testing.

Due to its superior parallel processing capability, Telemac2D had the fastest computing time of all models evaluated. The hybrid models performed reasonably well but were not as fast as Telemac2D. Numerical robustness and mesh generation features were other areas in which Telemac2D received high scores. Based on the overall evaluation, Telemac2D was selected for implementation in the FloodSiMM project for river systems requiring a 2D model approach. Based on previous experience within BC Hydro Engineering, the continued use of MIKE 11 was recommended for rivers that can be represented by a 1D model.

3.5 Inundation Mapping

Inundation maps which delineate the area that would be flooded due to a breach of an upstream dam or spillway discharge are prepared for each river system based on hydrodynamic modelling results. These maps aid in the consequence assessment of a dam breach and assist in the preparation of emergency procedures for warning and evacuation of the downstream population at risk. In addition to the extent of flooding, information on flood arrival times, peak flows, initial and maximum water levels at key locations along the river and adjacent floodplain are also included on the maps.

The base maps often include digital imagery as an underlay to inundation information. Additional base mapping details include the main road networks, dykes and other infrastructure which may be affected such as railways and transmission lines. Other cadastral data can also be used to indicate the locations of schools, community centres, utilities and key emergency response buildings (fire, police, hospitals).

An example of part of an updated inundation map prepared in the FloodSiMM project is shown in Figure 2 with geographic location names removed. Additional hydrodynamic modelling results such as peak water levels, flows and flood arrival times at key locations are not shown in Figure 2 but are typically tabulated and presented on the inundation maps.

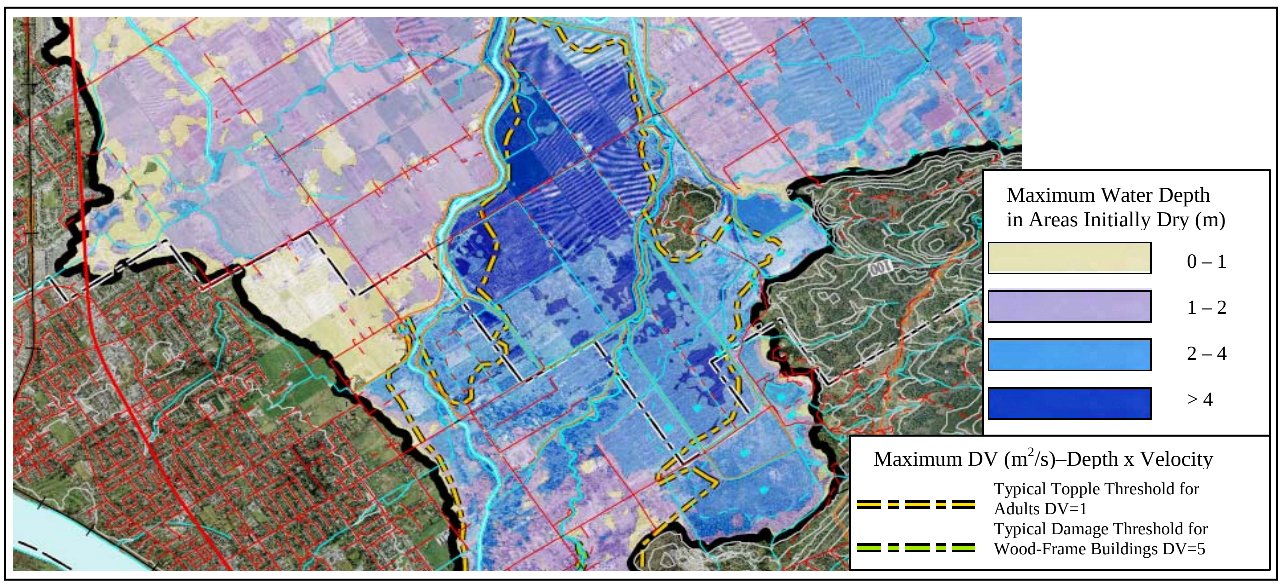

Figure 2: Example of part of an updated inundation map

4. A HIGH RESOLUTION, FLEXIBLE AND RESPONSIVE APPROACH

4.1 General

The specifications for both bathymetric and topographic data ensure that input information for inundation studies is of high accuracy with adequate spatial coverage. Data includes definition of key river and floodplain features which can be practically included in high resolution modelling and mapping. The flexible and responsive nature of FloodSiMM models to assess a wide range of dam discharge scenarios and varying operational and hydrologic conditions provides a powerful emergency planning tool. While inundation mapping for several pre-packaged failure and non-failure scenarios are prepared and incorporated into the emergency plans for each dam, the FloodSiMM models are developed to allow quick assessment of different combinations of reservoir levels, inflows and downstream tributary flows.

In some cases, the main populated areas that may be affected by a dam break are several hundred kilometres downstream of the dam. These cases require developing very large hydrodynamic models capable of routing flood flows through the downstream river valley and across the floodplain in order to determine the overall inundation impacts. For example, failure of one of BC Hydro's dams could impact several communities in the Fraser River valley, several hundred kilometres downstream. A 1D MIKE 11 model is used to route the flood flows down the Fraser River canyon to the Fraser River valley, downstream of Hope. Flow through the Fraser River valley is best represented with a 2D modelling approach which considers an area approximately 1,400 km^2 and includes large lakes, several rivers and vast floodplain regions. Balancing 2D model mesh detail with computational efficiency for such a large area is a key challenge.

The models can be used to assess potential dam failure at different times of year for varying reservoir levels and downstream tributary flows (e.g. Fraser River, Thompson River and Harrison River). The consequences of different failure modes (extreme flood versus earthquake) can also be assessed. The inundation impacts in the Fraser River valley communities could be very different if dam failure were to occur in October when reservoir levels and downstream tributary flows are low compared to the early summer during the annual freshet.

Some of the additional challenges in developing updated inundation models and mapping to meet the project objectives are described below.

4.2 Challenges

4.2.1 Hydrodynamic Model Development For a Wide Range of Flow Scenarios

Dam breach inundation studies are commonly performed by dam owners to assess the downstream impacts as a result of a potential failure. The conveyance of the downstream river is often small compared to the overall valley when routing worst case dam failure floods and therefore detailed definition of channel areas is usually not required for dam breach studies

The FloodSiMM models, however, are developed to not only model dam break flows but also to estimate inundation resulting from a range of spillway discharges for different combinations of downstream tributary and lake conditions. Routing of lesser spillway discharges and estimating the threshold of overbank flooding at various downstream locations requires an accurate representation of river bathymetry, islands, banks and adjacent dyking systems. These are important considerations during data collection and model development, discussed further in the next section.

Models are calibrated for the largest observed historical floods and available imagery and anecdotal information are often used to provide insight into historic flood extents and levels. Due to a lack of observed data, it is often difficult to calibrate the modelled rivers and floodplains for high spillway discharge scenarios and impossible for dam break flows. Therefore, sensitivity analyses and an assessment of uncertainties are required to better understand the effects of study assumptions and parameters on inundation results.

4.2.2 Estimation of Tributary Flows

For assessment of flood-related (i.e. non-earthquake) scenarios, it is particularly important to use reasonable assumptions for concurrent flows in downstream tributaries. For gauged tributaries, observed historical flow data is correlated with reservoir inflows which may be done using either hourly or daily data, depending on basin characteristics. Tributary flows for ungauged basins can be estimated from reservoir inflows or nearby gauged rivers using scaling ratios derived from basin-area relationships and areal reduction factors as appropriate.

Correlating flows in larger downstream tributaries to reservoir inflows may not be suitable in some cases. Frequency analysis of maximum daily tributary flows can be performed to provide 100-year discharge estimates, for example, to use concurrently with PMF conditions upstream of the dam.

4.2.3 Varying River Geometry and Floodplain Topography

Related to the challenges of developing hydrodynamic models for assessing a wide range of flow scenarios is the issue of dealing with difficult combinations of river geometry and floodplain topography. An example would be a very narrow river channel within a large floodplain area that requires a two-dimensional modelling approach to better simulate overbank flooding. Careful attention is required during data collection and 2D mesh generation so sufficient detail is included for channel areas. The simulation of dam break scenarios can be computationally demanding to maintain numerical stability, which can be related to the minimum spacing between nodes and the computational time step. Since the minimum node spacing will often occur within a channel where greater detail is desired, river mesh areas can play a large role in overall model stability. The mesh generator used for Telemac2D gives the user the flexibility to generate detailed river meshes with gradually coarser node spacing defined through floodplain areas while preserving key flood control features such as dykes, roads and railway embankments which could influence the extent of inundation. Figure 3 provides a concept sketch showing the variability in inundation modelling and mapping corridors with an example 2D model mesh.

Downstream of one of BC Hydro's dams, the river is only about 10 m wide but the overall study area for 2D model development is greater than 140 km^2. Balancing the overall number of nodes in the 2D model to minimize simulation times while including adequate representation of river bathymetry was a challenge that was addressed with using a minimum of five nodes to define the channel bottom and using the mesh generation options to gradually decrease the density of nodes through the floodplain, while preserving dykes and key elevated roads and railways. The total number of nodes in the 2D model is approximately 73,000 and the mesh characteristics were refined to produce a relatively computationally efficient model.

4.2.4 Produce Inundation Maps within 24 hours in an Emergency

An objective of the FloodSiMM project is that additional scenarios can be run and inundation maps produced in GIS electronic format for transfer to external emergency planning agencies within 24 hours of notice. In order to meet this objective, tested and documented procedures to operationalize the hydrodynamic models for any river system will be available to simulate any spillway discharge or potential failure scenario which may not be covered by the existing inundation maps. In these situations, reservoir inflow forecast information would be provided by the forecasting group within BC Hydro Generation Resource Management. On-line gauge flow data would be checked to get a sense of how regional relationships which have been previously derived perform for current conditions. Model input files would then be revised based on current or forecast conditions, depending on the emergency scenario. The computationally intensive 2D models are run on servers using up to eight processors to significantly reduce run times. For example, simulations for a model of approximately 73,000 nodes for 24 hour periods can be completed using the FloodSiMM servers in about 2 hours. Modelling results can be processed and presented on existing base maps to provide inundation maps for emergency planning agencies.

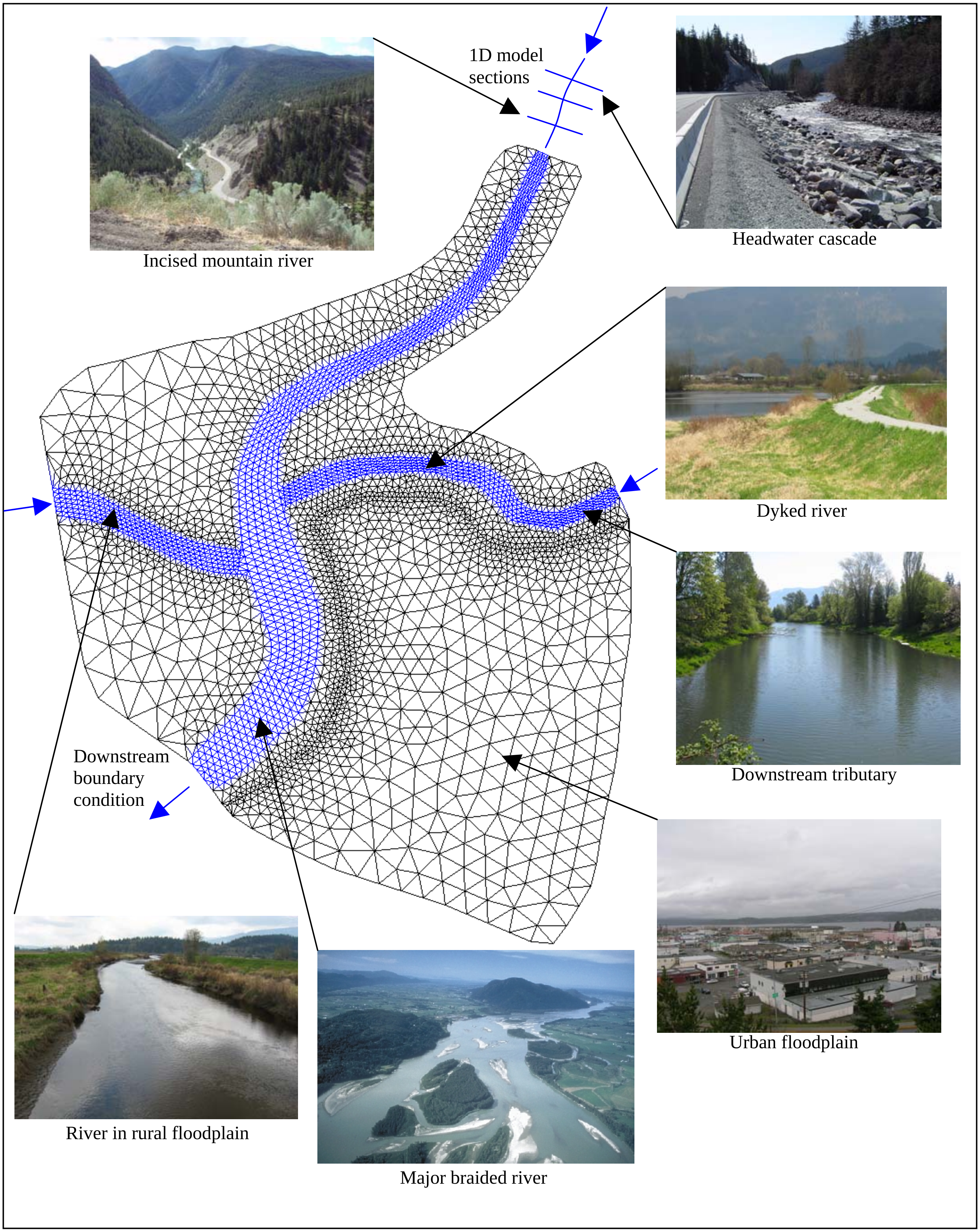

Figure 3: Concept sketch of 2D model mesh and variability in inundation modelling and mapping corridors

4.2.5 *Uncertainties*

Given the numerous uncertainties involved in inundation modelling, including accuracy of topographic and bathymetric data, dam breach parameter estimation and hydrodynamic modelling approximations, an important component of these studies is an assessment of uncertainties. Morris and Hassan (2005) summarized the results of investigations into quantifying modelling uncertainty in dam breach inundation studies and some of their recommendations have been incorporated into the FloodSiMM project.

To maintain a balance between computational efficiency and mesh detail, some floodplain elements such as elevated roads, culverts, bridge crossings and ditches may not be fully represented in the hydraulic models. Also, flood control structures such as dykes are assumed to remain intact.

Although it is recognized that dam breach discharges will erode and reshape downstream channels and landscapes, current modelling tools can not reliably simulate sediment erosion and solid material transport during extreme flood events. Further research is required to provide estimates of the potential magnitude of these effects and how they may influence the prediction of flood water levels and arrival times.

5. CONCLUSIONS

BC Hydro's FloodSiMM project will provide updated, high resolution inundation models and maps for all main dams for select breach scenarios and spillway discharge events. The modernization of inundation maps requires that the techniques for data collection, assumptions for dam breach analysis, approaches for hydrodynamic modelling and standards for inundation mapping are consistent with current best practices and are also commensurate with the consequences of dam failure. In addition, the modelling tools are to be flexible enough to map downstream inundation areas within 24 hours of notice of an emergency for any combination of predicted project outflow and downstream tributary conditions for any of the river systems. There are a number of challenges in meeting these objectives as described in this paper.

The development of updated hydrodynamic models and inundation maps for all river systems is to be performed over a three and a half year period with completion by March 2012. This will provide BC Hydro with a powerful tool for more effective emergency preparedness and management.

6. ACKNOWLEDGEMENTS

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IMPACTS OF HYDROPOWER-INDUCED FLOWS: ESTIMATION AND MITIGATION OF HUMAN AND ENVIRONMENTAL RISKS

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ABSTRACT:

Floods due to natural events and technical failures pose a significant hazard to downstream communities and environments throughout the world. Vulnerability is increasing due to changing settlement patterns along coastlines, below dams, and behind levee/dike systems. Increasing exposure of aquatic species to different hazards (e.g., waste emissions, natural floods, high or low flows resulting under typical hydropower plant, and over-fishing) and the inter-relationships among these hazards have increased their vulnerability to each of these hazards. This work develops a framework for identifying the most appropriate modeling tools and data required, and several tools for addressing planning questions of asset owners and communities at risk. Such tools will aid in the estimation of risks imposed on downstream communities and the environment due to operating decisions undertaken at dam sites. Methods for assessing alternative structural and non-structural mitigation approaches for reducing such risks are also identified, and performance indicators for evaluating their effectiveness are developed.

RESUME:

Les inondations dues à des phénomènes naturels et des problèmes techniques représentent un danger significatif pour les communautés qui habitent en aval des sources d'eau et pour les environnements à travers le monde. La vulnérabilité augmente dans le cas des tendances d'aménagement en mutation le long des côtes, sous les barrages, et derrière les systèmes de digue et les écluses. L'exposition en hausse des espèces aquatiques à différents dangers (par exemple les émissions de déchets, les inondations naturelles, les écoulements puissants ou faibles qui ont lieu sous les centrales hydrauliques classiques, et la pêche à outrance) et les interrelations de ces dangers ont accru leur vulnérabilité à chacun de ces dangers. Ce travail développe un cadre à même d'identifier les outils de modélisation et les données nécessaires les plus adéquats, et de nombreux outils pour aborder les questions de planification des propriétaires d'actifs et des communautés exposées. De tels outils rendront possible l'estimation des risques imposés aux communautés qui habitent en aval des sources d'eau ainsi qu'à l'environnement en raison des décisions d'exploitation prises sur les sites des barrages. Des méthodes propres à l'évaluation d'approches de limitation de tels risques structurels et non-structurels alternatifs sont également identifiées, et des indicateurs de performance pour évaluer leur efficacité sont développés.

1 INTRODUCTION

This research will develop a framework, supported by quantitative models, for analyzing the downstream human safety environmental impacts of dam failure or large uncontrolled dam release events. Such events are referred to as extreme events throughout this paper. The primary focus is on impacts downstream of dams, as limited areas upstream. To understand the impacts of catastrophic flood events, a number of analytical methods have been brought to bear to estimate the population loss under dam breaches. Most approaches include a description of the distribution of inhabitants, flood characteristics, an evacuation relation, and a loss relation (Jonkman, et al. 2002). The loss relation may be either empirical, based upon historical records of previous failures, or on an analysis of physical processes such as interactions between people and the strength of the floodwaters. Examples of the former include HAZUS (Scawthorn 2006), RESCDAM (Graham 1999), and LIFESim (Aboelata and Bowles 2005), and an example of the latter is the BC Hydro Loss of Life Simulation Model, LSM, (see BC Hydro 2005, Johnstone, et al. 2005). This work is directed at evaluating the needs of asset owners and communities at risk, and identifying the tools, including the types of models and intervention approaches, that are most appropriate for addressing these needs. Advances in our understanding of evacuation, structural hardening, and in the tools necessary to evaluate the effectiveness of evacuation and shelter-in-place mitigation strategies are being investigated.

Although there is as yet no consensus on which elements of the ecosystem are the key ones to be analyzed in evaluating the environmental impacts of dam releases, the need to conduct analyses of the immediate ecosystem impacts of, and long-term recovery from, extreme release events and operational strategies for mitigating these, is widely accepted (Beck 1996). While the environmental impacts of dam breaches have not been modeled to date, the impacts of different operational strategies which include controlled high (and especially low) releases have been studied in the last two decades. These studies and their resulting methodologies may be categorized as habitat methodologies, see Bovee et al (1998) and US Geological Survey (2001); fish population simulation models, see Railsback et al (2006); and holistic methodologies in which a team of stakeholders define performance measures for all attributes of interest and evaluate performance measures of all attributes for different flow scenarios, see King et al (2003) and BC Hydro (2004). Our work identifies the environmental indicators of interest in the management of dam releases and advances the development of a suite of models to estimate the immediate impacts on, and long-term recovery of, these indicators.

2 ASSESSING AND REDUCING RISK TO COMMUNITIES

In the defense against floods, many stakeholders show a preference for repairs or upgrades of flood defense structures. Non-structural alternatives, such as the development of evacuation plans, may be implemented as a stand-alone mitigation or in conjunction with structural alternatives, but rapid-onset, catastrophic events, such as dam failures, pose a special challenge because it is possible that the community may not be capable of evacuating in sufficient time. Therefore, an additional alternative might be the option of the population sheltering-in-place within strong buildings while the flood passes through the community.

Many existing evacuation plans appear to be untested, overly optimistic, and may not meet the performance objectives of the stakeholders. Furthermore, the set of detection, warning, evacuation, and sheltering subsystems is typically not designed to work as an integrated system, thus raising doubts as to whether the combined elements would perform as expected during an actual event. Simply exercising selected parts of the system to ensure that a warning has been issued, passed down the line, or received by some portion of the community does not provide sufficient proof that losses will be reduced. Field tests of an evacuation plan are rarely undertaken and can be cost-prohibitive or infeasible to perform in a meaningful way. Given these issues, two research pathways are being pursued, they are to develop a Conceptual Model of Evacuation and Sheltering Systems (ESS), and two new Measures of Effectiveness (MOEs), *evacuability* and *shelterability*. The ESS model is being developed to provide a unifying framework for models and methods used to estimate loss and survival outcomes, with and without mitigations in place. The MOEs will provide a means of estimating and communicating the potential for risk reduction to the population facing the hazard, and of assessing the effectiveness and reliability of community-based mitigations such as evacuation and shelter-in-place.

Evacuability appears to be a relatively new term first used by Vassalos and Vassalos (2004) in a discussion of passenger ship evacuation guidelines published by the International Maritime Organization (IMO 2002). No lineage of the term *shelterability* for risk mitigation has been found in the literature.

Figure 1 presents a representation of the flood hazard identification, loss estimation and mitigation system for an example based on a tsunami impacting a coastal community. The hazard (A) and community vulnerabilities (B) are used to develop the initial set of loss estimates. To mitigate the potential losses, detection and warning systems (C), and an evacuation route system and safe havens (D) are planned and implemented. Upon detection of the hazard onset by a government monitoring system, local Emergency Operations Centres are activated and the community is warned and mobilized to safe havens. While this approach appears to be workable, in many cases these subsystems are not designed to work in an integrated manner as a reliable system, and broad assumptions about their potential effectiveness are made without analysis or verification. There is a need to design mitigations by taking an integrated systems perspective to ensure that the various subsystems can work together effectively.

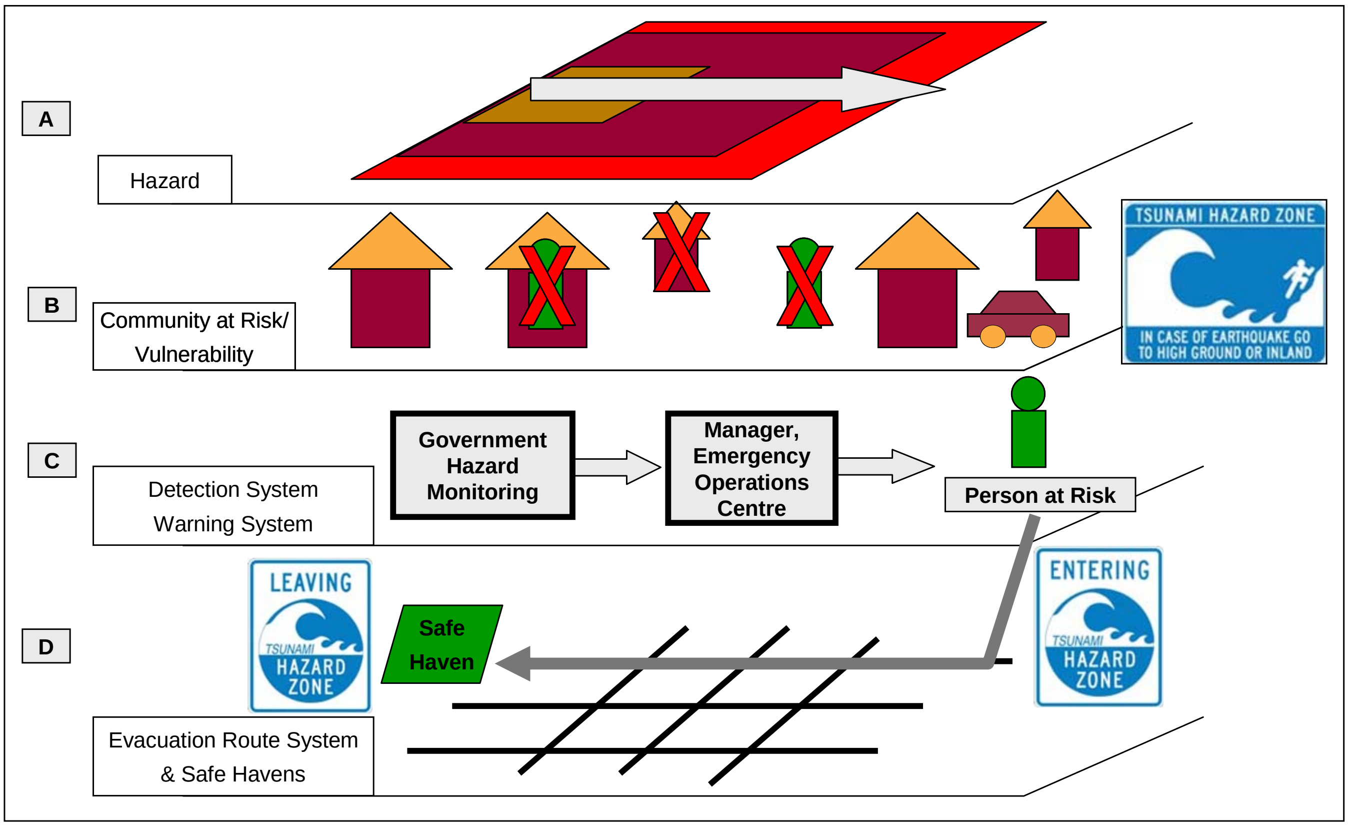

Figure 1: Flood Hazard Mitigation

2.1 A Conceptual Model of Evacuation and Sheltering Systems (ESS)

The Conceptual Model of the ESS decomposes the ESS into subsystems. It is being designed to:

- Characterize the objects, behaviours and interactions, both within and between subsystems.
- Provide a basis for selecting and integrating various empirical, analytic or simulation-based submodels.
- Guide the development, validation and correct use of models to assess if an ESS can meet life safety performance objectives.
- Support the design, analysis and optimization of mitigations for rapid-onset, catastrophic floods.
- Provide a means for clarifying failure mechanisms and causal dependencies that lead to losses.
- Help make proposed ESS more robust, fault-tolerant, and cost-effective.

Figure 1 introduced a conceptual view of loss estimation and mitigation of a tsunami hazard. The two key systems shown are the model of the hazard system and the ESS. A model of the tsunami hazard integrates seismic events and the waterbody over which the hazard propagates. A model of a dam failure might include a dam and its reservoir, and failure processes such as piping, an extreme meteorological event, or human-caused mis-operation of the facility, which in turn cause a catastrophic loss of reservoir containment. The boundaries between the various subsystems in the ESS could be defined a number of ways. As an example, consider the four subsystems of a simple ESS shown in Figure 2. Services, damage pathways and event timings are shown. The seismic event damages a dam, and also affects critical infrastructure such as roads and communications which provide services that support community evacuation. The dam breaches and the flood wave arrives at the community 60 minutes after the initial earthquake. The dam owner has put a monitoring and diagnostic system in place which allows them to detect the failure, confirm it, and notify those responsible for warning the community. The warning cascades through various channels, and the community mobilizes and evacuates.

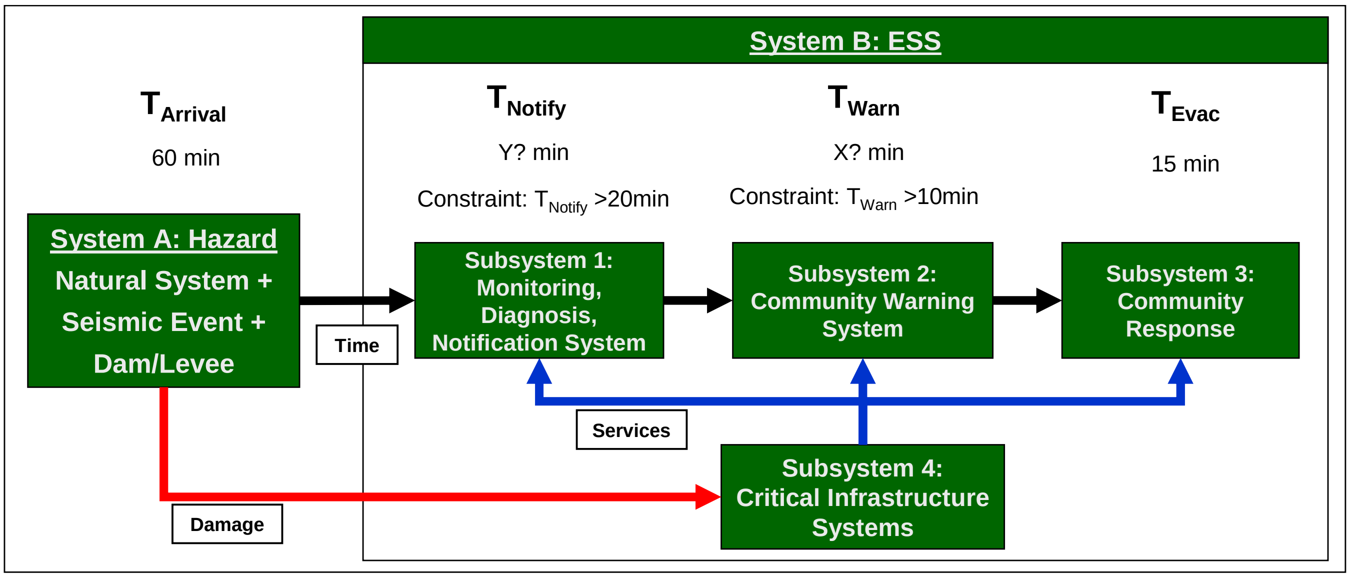

Figure 2: Example of Application of Hazard Onset and Evacuation and Sheltering System (ESS)

A rudimentary limit-state function which considers the various times can be developed as shown in Figure 3. Failure occurs if the total combined time to notify, warn and evacuate is longer than the flood arrival time. Using a simple deterministic approach, the event durations and constraints shown in Figure 2 can be used to determine the feasible region for notification and warning times that do not lead to failure. While this example demonstrates an approach for assessing whether evacuation is feasible, it greatly simplifies the many internal factors and uncertainties that contribute failure of the ESS. A more comprehensive reliability-based approach is currently being explored.

2.2 Measures of Effectiveness for Evacuation and Sheltering

MOEs are being developed that measure the potential for a population at risk to reach and shelter in a safe haven within the impact zone, i.e., *shelterability*, and the potential of a population to reach safety outside the impact zone, i.e., *evacuability*. These MOEs would:

- Help planners and engineers to develop, assess, rank and optimize alternative ESS designs.
- Would be applicable to both hazard and community-based mitigations.
- Help communities at risk develop a better understanding of the value of proposed mitigations.

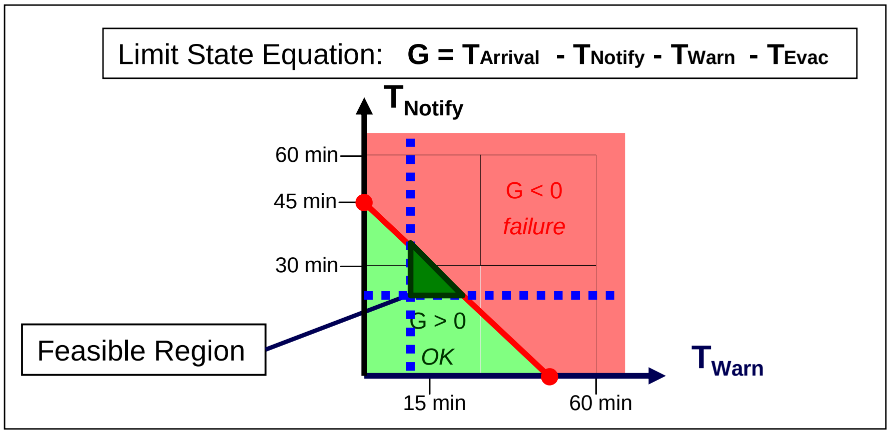

Figure 3: Determination of Feasible Region for Evacuation Timing

The basic concept of the proposed MOEs is shown in Figure 4. A community at risk is shown as having three impact zones and three safe haven zones. Each impact zone connects to one or more haven zones, and there may be a number of shelter-in-place, safe havens, locations within the impact zones. The middle diagram of the figure shows the concept of a transport system model which simulates the movement of people from the impact zones to the save haven zones. The only impact zone which has access to two possible safe haven zones is IZ_B.

The rightmost diagram of the figure indicates the *evacuability/shelterability* objective space. Each dimension is initially described as an ordinal scale ranging from bad to neutral to good. The estimated MOE for each impact zone is shown, with IZ_C offering the potential for both evacuation or sheltering, IZ_B being evacuable but not shelterable, and IZ_A offering neither; therefore, some additional mitigation is required for IZ_A. If the proposed mitigation can only enhance *shelterability* but not *evacuability*, then the status of IZ_A may be changed as indicated by the arrow in Figure 4.

The results of this analysis would be used to inform the community. The people in IZ_A would be advised to shelter within the impact zone (stay), the people in IZ_B would be advised to evacuate because remaining in their homes or businesses is unsafe, and the people in IZ_C would be advised that they have the most options in terms of how to avoid being killed by the floodwaters.

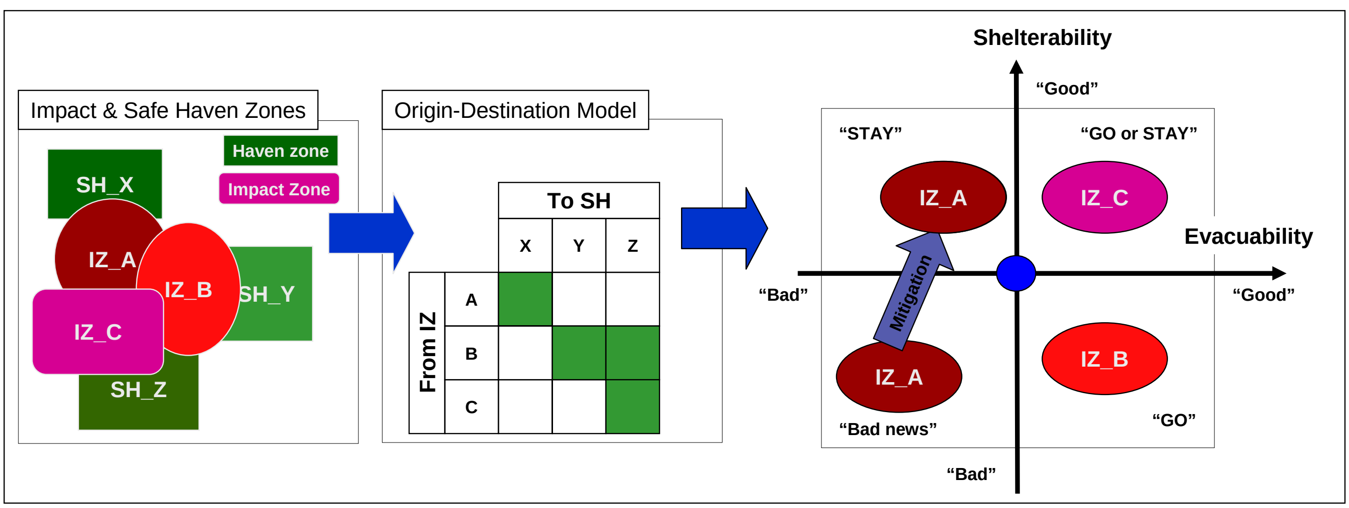

Figure 4: *Evacuability* and *Shelterability* MOEs

Figure 5 explores what can occur at the most detailed physical level in which structures and individuals are interacting directly with the flood waters. A person at risk is shown living in a house sited along the bank of a river. The person can choose to stay or go. Probability curves are shown for a number of variables such as the building floor level, the flood level, the person's height, and the time required to reach the nearby safe haven. In Scenario A, the "stay" decision can be explored by estimating the probability that the flood level (shown in red) will rise above the floor level (shown in green). For this discussion, assume that simple flooding is sufficient to cause a loss of the structure and the person inside. Similar calculations can be evaluated for the "go" decision, in which the person is toppled either as a pedestrian or in a vehicle while evacuating, and may or may not reach the target safe haven before being caught by the flood wave. Here the probability that the person will reach a safe haven in a specific time is shown in green and it may be compared with the probability of the wave of a specified height reaching the person in a specified time, which is shown in red. In Scenario B, a mitigation has been implemented: the house has been moved some distance away to higher ground. The probability of failure of the "stay" decision has now been reduced, and the probability of failure for the "go" decision has also been reduced because the building is closer to the save haven. Before and after assessments of this type can be made for every structure and person located in the hazard zone.

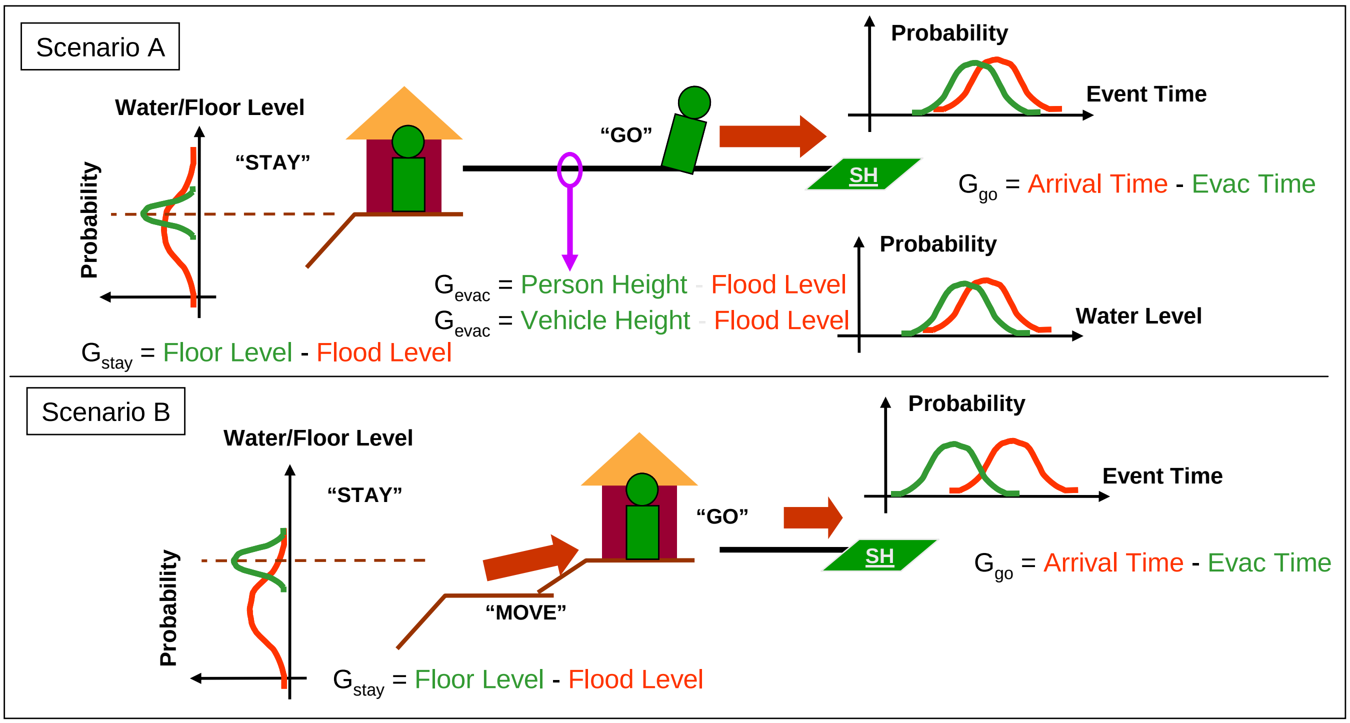

Figure 5: Estimating Changes in *Evacuability* Using Reliability-Based Approach

The detailed analysis at the individual person and structure level presented in Figure 5 can be summarized as shown in Figure 6. The left diagram is a plot of the individual outcomes in the *evacuability/shelterability* objective space. It may be useful to summarize the outcomes as a simple distribution. In this case, Scenario C has some people who can go or stay, but also a number who will not be able to get out in time. Scenario D shows how it is possible to enhance the *evacuability* for those people, but not their *shelterability*. The right diagram of the figure combines the MOE analysis with the cost for each mitigation and estimated losses. *The* evacuability and *shelterability* MOEs would provide information that could enhance the decision process. For example, while scenario B is optimal in terms of cost, scenario C offers both reduced human losses as well as significantly improved *shelterability*.

3 ASSESSING ENVIRONMENTAL IMPACTS OF EXTREME EVENTS

In order to estimate environmental impacts of an aquatic system, environmental attributes of interest and the means of describing the attributes, i.e., performance measures, should be identified first. In the EPA Guidelines for ecological risk assessment (EPA 1998), environmental attributes and performance measures are referred to as assessment endpoints and measures of effect, respectively. When investigating river systems, different environmental attributes may be considered. The most common attributes include:

- Fish abundance and diversity, or fish population dynamics. Numerous species and different life stages of fish may be considered.
- Diversity and density of aquatic plants and invertebrates.
- Abundance of fish habitat.
- Geomorphology and sedimentation (scour and deposition).
- Water quality and temperature.
- Existence of terrestrial vegetation and wildlife.

All of these attributes, except the last one, may be considered as a means (i.e., as parameters, indicators, or descriptive variables) of estimating the first one, i.e., fish abundance and diversity which is the most holistic attribute (see EPA 1998). For this attribute, for example, the population of a specific species at a specific age may be selected as its performance measure. Fish habitat has been the most commonly applied attribute in the literature, probably due to its simplicity.

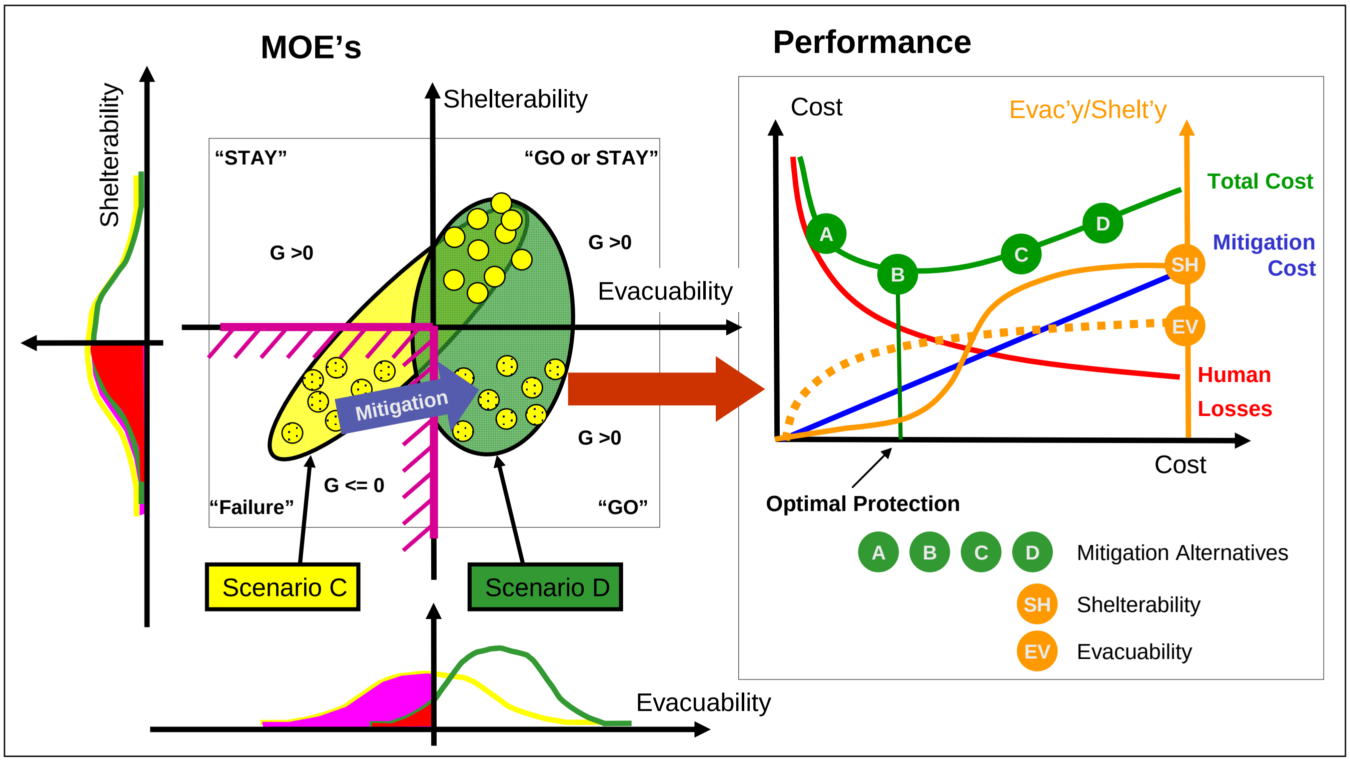

Figure 6: Integration of MOEs in Support of Decision-Making

There are a range of existing tools for estimating the long-term environmental performance of water resources systems, given the flow regime and geomorphologic and biological background conditions. The choice of appropriate estimation tool depends on the environmental attribute of interest and its performance measure. PHABSIM is the most commonly used fish habitat model in the literature, developed by the US-Geological-Survey (2001). inSTREAM is an individual-based model which simulates the fish population dynamics under different scenarios of flow, turbidity, and temperature time series. Details of the inSTREAM model are described by Railsback et al. (2006).

Although extreme events may have significant impacts on environmental attributes of a river system, e.g., fish kills and geomorphologic changes, neither fish habitat, nor fish population dynamics models could properly estimate the effects of such events. Figure 7 represents the fish population dynamics for a river system before and after an extreme event. Fish habitat changes during and after an extreme event also follow the same pattern as fish population and are not discussed separately here. The fish population of the river system is initially in a steady-state condition. At point 1, an extreme event starts and some immediate effects including direct mortality of fish and eggs, and changes in geomorphology, food, and water quality, occur. Under such conditions, long-term fish population models, e.g., inSTREAM, cannot be employed to simulate the fish population dynamics since their fundamental assumptions are not valid. As an example of the violations of the assumptions of inSTREAM, the stream cross section and sediment composition are not constant. At point 2, such immediate effects may be considered to be complete, and thereafter, a physical equilibrium is reached, and long-term fish population models can simulate the dynamics of the system. However, the system is likely to be facing biologically transient conditions from the time of the event until it reaches a biologically steady-sate condition, i.e., at point 3, which may or may not be the same as the pre-event steady-state condition.

Obviously, ignoring such extreme events in estimating the long-term status of environmental attributes is not an appropriate solution, and may result in unrealistic estimates of environmental attributes. Also, there is no published model or tool to assess immediate environmental effects of single extreme events, e.g., extreme floods. Available data are limited to opportunistic sampling before and after such events (see Nislow 2002).

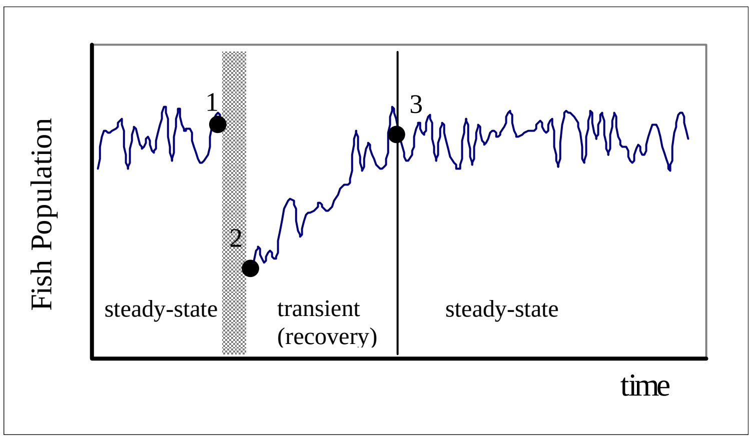

Figure 7: Conceptual fish population time series for a river system before and after an extreme event (point 1: when the extreme event occurs, point 2: when immediate effects are complete, point 3: when the system returns to steady-state conditions)

A conceptual model is being developed to investigate the immediate effects between points 1 and 2 in Figure 7. The time-dependant likelihood of the system recovery to its pre-event or to new steady-state conditions (i.e., between point 2 and 3 in Figure 7) given the immediate effects of the extreme event as the initial conditions is also being investigated. Before point 1 and after point 3 in Figure 7, one may employ either a fish population model or a habitat-based model to simulate the environmental status of a system.

3.1 A Conceptual Model for Assessing Immediate Impacts

When the flow reaches a threshold such that the long-term model assumptions are violated (i.e., point 1 in Figure 7), the long-term model should stop and a short-term model should then be used to estimate the conditions, or a range of plausible conditions, at the end of the extreme event when the long-term fish population dynamics model assumptions are valid again (i.e., point 2 in Figure 7).

A conceptual representation of the processes being addressed in this research is shown in Figure 8. Initial biological conditions (including spatial distribution of different species of fish, eggs, invertebrates, and aquatic and riparian plants), and physical and chemical conditions (including, e.g., the reservoir water level, flow rate in the river, geometry of the channel and the floodplain, sediment composition and depth, and water quality conditions) should first be assigned. Given an extreme event scenario (e.g., a dam breach), hydrodynamic, geomorphologic, and water quality models may be used to estimate the temporal and spatial distribution of hydraulic (e.g., velocity, depth, and shear forces), geomorphologic (e.g., sediment composition and depth, and channel shape), and water quality (e.g., DO and turbidity) changes in the river system. Based on the outputs of such models, the changes to different environmental objects will be investigated.

3.1.1 Eggs and Juvenile Fish within the Gravel

If the extreme event occurs during the incubation period, the eggs may be affected. Given the hydrodynamic parameters, if sediment is deposited on redds (i.e., strings of laid eggs), they may not have enough oxygen, and eggs will die. Also, if a scour as deep as the redds depth occurs, they will be destroyed. The effects of immediate changes in the water quality (e.g., turbidity and DO) on eggs and juvenile fish should also be investigated.

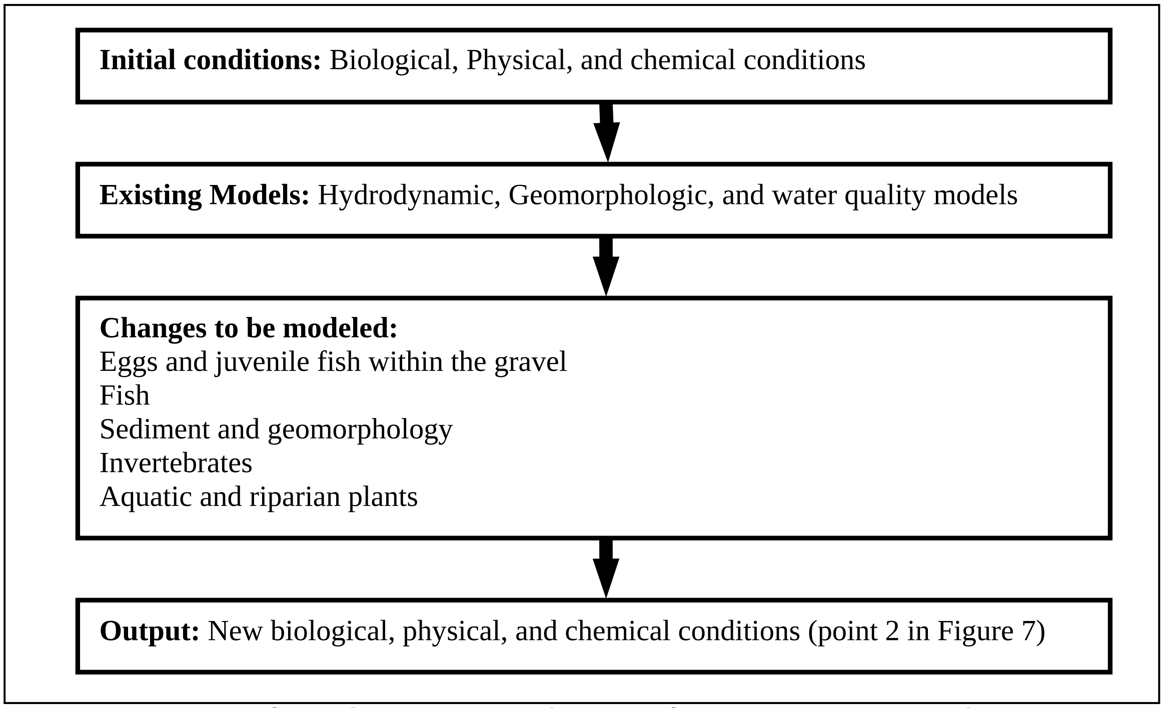

Figure 8: Assessment of immediate environmental impacts of extreme events- conceptual representation

3.1.2 Fish in the Water

Past research, focused on fish mortality in turbines and spillways, suggests that fish mortality may be caused by the: shearing effect; turbulence; sudden acceleration or deceleration; very sudden variations in pressure and cavitation; abrasion against surfaces; physical impact against energy dissipaters; and the possibility of shocks from moving or stationary parts of the turbo machinery (Marmulla, 2001). Among these effects, the shearing effect at different conditions, and abrasion against the surface after a spillway fall, have been quantified in experiments. Other effects such as turbulence, sudden acceleration or deceleration, sudden pressure change, and collision (e.g., abrasion against rocks) should be investigated. Burst speed, which is less than ten times the length of the fish for almost all species (Beamish, 1978), may serve as a key indicator for estimating the fish mortality or injury.

Other factors that may be of less importance, including effects of immediate changes in the water quality (e.g., turbidity and DO) on fish, and the fish behavior under extreme speeds should be investigated. The latter will help to estimate the probability of collision and stranding. Also the effect of extreme events on adult fish depends on the time of year, and on whether they are anadromous.

3.1.3 Sediment and Geomorphologic Changes

Due to the complexity of sediment transport and geomorphologic models, it may not be practical to model the whole downstream river system affected by an extreme event at a microhabitat scale. The options to overcome such a complexity may be categorized as follows:

- To use a probabilistic approach for characterizing scour and deposition (see Haschenburger 1999).
- To estimate the macro-scale geomorphologic changes, and use them for all microhabitats within each macro-scale simulated reach.
- To estimate a range of plausible changes given an extreme event.

3.1.4 Invertebrates, and Aquatic and Riparian Plants

Estimating the changes to these objects requires further investigations, and are not addressed in this research.

3.2 A Framework to Estimate Recovery to Equilibrium

If an extreme event occurs, environmental consequences include not only the immediate impacts, but also the transient impacts (e.g., to evaluate the shortage of sports fish in the river for several months before the river reaches a new biological equilibrium) and long-term impacts (e.g., to evaluate whether the new equilibrium has less desirable environmental attributes than those of the pre-event equilibrium). In order to account for such transient and long-term impacts, the length of the recovery period, and the ultimate equilibrium (i.e., status of point 3 in Figure 7) must be estimated.

A conceptual representation of the processes being addressed in this research is shown in Figure 9. Initial biological conditions (including spatial distribution of different species of fish, eggs, invertebrates, and aquatic and riparian plants) are set based on the results of the immediate impact estimates (i.e., output in Figure 8). Physical and chemical conditions (including the geometry of the channel and the floodplain, sediment composition and depth, and water quality) are also assigned based on the output of immediate impact estimates. Such physical and chemical conditions are assumed to be in an equilibrium status at the beginning of the recovery period. Due to the complexities of estimating the effects of extreme events, these initial conditions may know be known. Rather, accessible data may be in the form of probability density functions or simply a subjective range of data. Thus, borrowing the concept of resilience from either the ecological or engineering context is being considered this research. However, this concept has not been uniquely defined in the literature, and its existing varieties may not be suitable for this problem. Rather, new definitions and expressions for this concept may need to be developed.

Resilience, as an ecological risk indicator, is usually introduced to measure ecological stability. Holling (1973) defines ecological resilience as the ability of an ecological system to absorb a disturbance without moving from one equilibrium status to another. Hashimoto et al. (1982) introduced risk-based performance indicators in the water resources engineering context. These indicators are reliability (the probability of a system success), vulnerability (the expected magnitude of a system failure), and resilience (the expected value of system recovery speed). Holling (1996) refers to engineering resilience as “environmental resilience”. The difference between ecological and engineering approaches stems from the fact that ecologists are interested in whether a system recovers to its equilibrium status after the external disturbance is removed or not; while engineers are interested in the recovery speed.

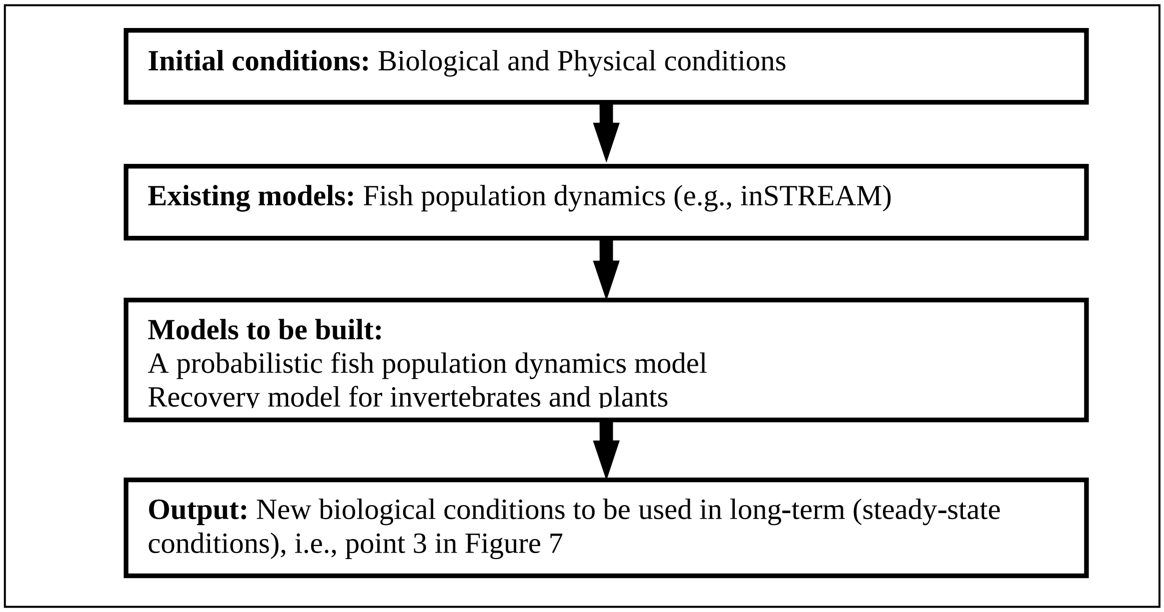

Figure 9: Assessment of environmental impacts recovery of extreme events- conceptual representation

Specific features of the case investigated in this research which may make most existing resilience definitions inappropriate to use are listed as follows:

- The results of the research will ideally be used in an integrated impact assessment where other impacts, e.g., economic and life safety, are also estimated. Therefore, the results should be quantifiable indices.
- Both ecological and engineering aspects of resilience are important. Not only is it important whether the system recovers to its previous equilibrium or not, but also the recovery speed is a key factor.
- A single execution of a population fish dynamics model takes about 1-10 hours. Thus using stochastic simulations, e.g., Monte Carlo simulation, that require thousands of simulations may not be practical.
- The environmental failure due to an extreme event is a rare event, and historical records will not likely be useful for statistical purposes. Thus, methods of deriving reliability and resilience from counting the failure events do not work here. Also, approaches that are based on recoverable systems with frequent success and failure may not be employed.
- If a FORM-based model is used to estimate the resilience, it should be noted that the limit-state-function (i.e., the function which describes whether the system is in safe or failure mode) is not an explicit function. Rather, the value of such a function should be estimated by running a fish population dynamics model, e.g., inSTREAM, every time one such value is needed.
- The downstream system in its transient period is non-stationary.

A time-dependant reliability method is being used to estimate resilience. A key step in reliability analysis which may lead to significant error, if not handled properly, is the definition of safe and fail conditions. The system is considered as safe if, for example, the 7-day moving average of the population (i.e., resistance) is greater than or equal to the fish population without the extreme event (i.e., load). Otherwise, the system is in failure mode. The load is also a time dependent variable which can be generated by running the fish population dynamics model under normal operation policies without the extreme event. The limit-state function is a function that estimates whether the system is in safe or fail mode. In this case, the limit-state function should be based on a fish population dynamics model, e.g., inSTREAM.

Time-dependant reliability may be estimated using a FORM-based method to estimate the reliability of the system at different times. The process of reliability estimation may be repeated for different initial conditions, i.e., different levels of event severity. The result may be represented as in Figure 10 which shows whether the system recovers to its previous equilibrium, or falls into a new one. At some point (i.e., when the extreme event is greater than a specific flood), the system does not recover to pre-event conditions even after years. If that happens under a 500-year flood event, for example, such an event will be the maximum load under which the system is ecologically resilient.

Figure 10 also shows the probabilistic speed of the system to reach such equilibrium. For example, one may estimate that there is an 80% chance that the system recovers to its previous steady-state condition after one year given a 5-year flood. Therefore, this index may be referred to as a probabilistic time-dependant resilience that addresses both ecological and engineering aspects of resilience.

Since the estimation of the index is FORM-based, sensitivity analysis may be conducted to investigate the effect and importance of the input uncertainty on the estimated index. For example, one may investigate whether reducing the uncertainty in the estimated range of fish population at point 2 of Figure 7 has a significant effect on resilience.

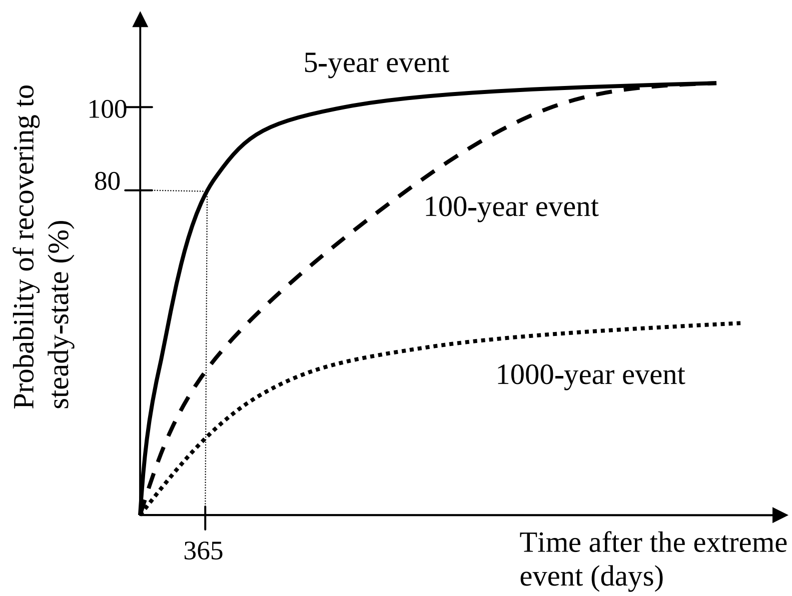

Figure 10: Conceptual representation of the proposed resilience index

4 CONCLUSIONS

4.1 Human Safety

The results of this work will provide planners and engineers with the tools that they need to develop, assess, rank and optimize alternative ESS designs and will help communities as risk develop a better understanding of the value of proposed mitigations. The Conceptual Model of ESS may be used to assess the impacts of dam failures on populations at risk, as well as the impacts of other rapid-onset, catastrophic floods, such as levee failures, tsunami, flash floods and lahar. Definition and estimation of MOEs, such as *shelterability* and *evacuability* may also be used to enhance existing flood loss estimation models.

4.2 Environmental Impacts

The process followed in this research may be used to advance the Life Safety Model (LSM) developed by BC Hydro (2005) so that it can address the environmental impacts of an extreme event. The model currently accounts for loss of life based on a given set of initial location of the objects (i.e., people), the execution of a hydrodynamic model, and estimating the Object Damage and Loss Functions (ODLFs). The advancement may include adding new components to the model. These are:

- *New objects*: Fish species at different life stages (e.g., eggs, and juvenile and adult fish), invertebrates, and plants may be considered as new objects.
- *New models*: Geomorphologic and water quality models may be added to the existing hydrodynamic model.
- *New ODLFs:* damage and loss functions may be assigned to each object based on the results of this research (i.e., short-term environmental impacts of extreme events)

5 ACKNOWLEDGEMENTS

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APPROCHE RELATIVE À LA SÉCURITÉ DES BARRAGES EXISTANTS

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RÉSUMÉ:
De nos jours, l'exploitation des barrages est un enjeu considérable. Peu importe son utilité, une retenue d'eau est toujours un bien précieux, que ce soit pour l'exploitation municipale, industrielle, commerciale ou personnelle. Il est donc primordial d'assurer son intégrité en réalisant un suivi et des entretiens réguliers afin d'en tirer les bénéfices escomptés.

Les gouvernements ont ainsi établi des lois et règlements afin d'assurer une bonne gestion de ces infrastructures. Au Québec, et à une fréquence prédéfinie, les barrages font l'objet des études de sécurité conformément aux exigences du Ministère du Développement Durable, de l'Environnement et des Parcs, selon la loi en vigueur. Le but principal de ces études est de rendre l'ensemble des barrages conformes aux normes et exigences du Ministère et, par conséquent, assurer la sécurité des biens et des personnes considérés potentiellement à risque en cas de bris soudain de ces ouvrages.

Le présent document aborde des discussions sur plusieurs sujets d'actualité. Dans un premier temps, nous élaborons les méthodes de classement des barrages. Nous présentons ensuite une étude de sensibilité de trois modèles numériques connus à la fois sur la scène nord-américaine et européenne, soit les modèles FLDWAV, MASCARET et HEC-RAS. Le but de cet exercice est de sensibiliser les utilisateurs de ces logiciels à l'interprétation des résultats issus de l'utilisation de l'un ou l'autre de ces modèles numériques. Dans le troisième volet nous présentons l'approche utilisée pour assurer la sécurité par la conception, sous l'aspect structure, pour deux types de barrages existants, en terre et en béton et à la fin les interprétations des résultats obtenus lors de l'étude de sécurité sont présentées.

ABSTRACT:
Nowadays, the dam development is a major undertaking. No matter what it is designed for, a dam is a very valuable structure, whether used for municipal, industrial, commercial or other purposes. The owner should ensure its integrity by carrying out monitoring and regular maintenance in order to obtain the anticipated benefits.

Several laws and regulations have been adopted by various governments. In Quebec, the dams should be undergo periodic safety review according to the requirements of the Ministère de Développement Durable, de l'Environnement et des Parcs. The principal purpose is to make those dams conform to the new regulation and so ensure the security of property and of persons considered potentially at risk in case of a sudden breaking of those structures.

This paper addresses and explores some issues related to dam operation and management. First, we explain the dam classification methods. Then we present a sensitivity study of three well-known numerical models, used to simulate the dam break phenomena : FLDWAV, MASCARET and HEC-RAS. This will help users of these models to reach a better understanding of the differences that might occur by using one of these models. Thirdly, we consider the approach that we employ for structural aspects of both existing concrete and embankment dams to ensure safety by design, and finally, the resulting interpretation of dam safety studies is presented.

1. INTRODUCTION :

Les barrages construits par l'être humain depuis des décennies font partie de la nature et leur démolition devient improbable et même inacceptable. Cependant comme toute construction, il est requis de l'entretenir pour conserver son intégrité. A cet effet, le gouvernement du Québec a promulgué une Loi et un Règlement sur la sécurité des barrages. Le Ministère du Développement Durable de l'Environnement et des Parcs (MDDEP) a été mandaté pour assurer leur respect par les propriétaires des barrages. Pour Hydro-Québec, Alcan et autres institutions majeures, la sécurité de leurs ouvrages fait partie de leurs activités. Cependant les petits propriétaires, y compris certaines municipalités ne sont pas équipés pour assurer le respect de la Loi et doivent avoir recours à des professionnels ayant une expertise dans ce domaine pour s'y conformer.

Le présent article traite justement de barrages appartenant à cette catégorie de propriétaires.

Au Québec, la gestion des régimes hydriques est l'une des responsabilités du Centre d'expertise hydrique du Québec (CEHQ) qui représente une agence du Ministère de l'Environnement et de Développement Durable et des Parcs. Le 11 avril 2002, le CEHQ a mis en place une Loi sur la Sécurité des barrages (LSB) qui constitue une réforme du régime juridique encadrant l'établissement et l'exploitation des barrages dans la province. L'origine de cette loi provient des événements qu'a connus le Québec, notamment le déluge de Saguenay en juillet 1996 et la commission Nicolet qui s'en est suivie.

Le CEHQ est également responsable de la gestion foncière et de l'intégrité du domaine hydrique. Les principales missions du CEHQ sont la sécurité des biens et des personnes, le développement durable et l'équité. En parallèle avec cette loi (LSB), le CEHQ a établi un Règlement sur la sécurité des barrages qui sert à l'application de cette dernière et qui fournit le cadre technique sur la base duquel les propriétaires des barrages devront réaliser des études d'évaluation de la sécurité de leurs ouvrages.

Le Québec compte près de 6 000 ouvrages de retenue de dimensions variées. Ces ouvrages nécessitent un entretien, une surveillance et un contrôle rigoureux étant donné qu'ils peuvent être soumis à plusieurs conditions probables comme par exemple les crues, le séisme, la force des glaces, le vandalisme ou encore le vieillissement des structures en béton. Les petits barrages existants présentent un grand nombre de spécificités et autant de difficultés que les très grands barrages. Ils peuvent être difficiles à analyser parce qu'ils datent généralement du début de siècle et sont peu documentés. Pour les rendre conformes aux nouvelles normes, ces barrages doivent être réexaminés afin d'assurer leur pérennité ainsi que la sécurité publique.

Le présent document met l'emphase sur l'application de la Loi et le Règlement relatif à la sécurité des barrages au Québec. Le domaine d'activité porte plus sur les petits barrages dont l'usage est plus de type municipal (prise d'eau), récréatif, villégiature ou autre. Deux projets de barrages existants, l'un en béton l'autre en remblai seront présentés selon l'approche d'évaluation utilisée. Lorsque le contexte de l'étude le permet, un rappel de la réglementation de l'Association Canadienne des Barrages (ACB) fera l'objet d'élément de discussion à titre comparatif seulement.

2. CLASSEMENT DES BARRAGES :

Tel que prescrit par la Loi, il existe deux types de barrages au Québec: les barrages à faible contenance et les barrages à forte contenance. Les barrages à faible contenance (le Québec en compte 2399) sont des ouvrages de retenu d'une hauteur de 2 m ou plus et dont la capacité d'emmagasinement se trouve au-dessous de 30 000 m^3. Les barrages à forte contenance (environ 1982 au Québec) peuvent être définis selon l'une ou l'autre des trois catégories suivantes :

- Barrage d'une hauteur de 1 mètre ou plus dont la capacité de retenue est supérieure à 1 000 000 m^3.
- Barrage d'une hauteur de 2,5 mètres ou plus dont la capacité de retenue est supérieure à 30 000 m^3.
- Barrage d'une hauteur de 7,5 mètres ou plus, sans égard à la capacité de retenue.

La Loi et Règlement sur la sécurité des barrages concernent, en particulier, ce deuxième type d'ouvrages pour qui leurs propriétaires sont tenus de réaliser des études de sécurité.

Le principal objectif de ces études est de s'assurer de la stabilité et la fonctionnalité du barrage, la conformité de sa conception et de sa construction par rapport aux règles de l'art et aux normes de sécurité ainsi qu'à déterminer, le cas échéant, les correctifs appropriés. Ceci passe par la révision du niveau de conséquence et de la classe du barrage.

On entend par le « niveau de conséquence », l'impact le plus important qui résulte du pire scénario de rupture réalisé parmi un ensemble de scénarios étudiés. Le terme « impact » fait référence aux types d'infrastructures et à la population affectée. Pour l'Association Canadienne des barrages, le niveau de conséquence est caractérisé par la notion « perte de vie ».

Le tableau suivant illustre les 6 catégories qui définissent le niveau de conséquence.

Tableau 1 : Sommaire des niveaux de conséquence considérés au Québec

Niveau de conséquence	Caractéristiques très sommaire du territoire affecté	Point(s)
Minimal	Terre agricole / chemin d'accès aux ressources	1
Faible	Moins de 10 chalets / route locale	2
Moyen	Moins de 10 résidences / ligne de chemin de fer	3
Important	Plus de 10 résidences et moins de 1 000 habitants / école	5
Très important	Entre 1000 et 10 000 habitants / parc industriel	8
Considérable	Hôpital / site d'entreposage de matières dangereuses	10

Le classement du barrage est réalisé lors de la première étude d'évaluation de sa sécurité et il est révisé lors des études subséquentes [(1)].

En plus du « niveau de conséquence », la « Vulnérabilité » de l'ouvrage représente un élément d'analyse incontournable dans la détermination de la classe d'un barrage. L'évaluation de la vulnérabilité repose sur des paramètres physiques, dits constants : hauteur, type de barrage, capacité de retenue et fondation; ainsi que sur les paramètres variables dans le temps : âge, zone sismique, état et fiabilité du système d'évacuation. La particularité rencontrée dans l'élaboration qualitative de l'ensemble de ces paramètres réside dans l'évaluation de la fiabilité du système d'évacuation, avec ou sans gestion requise. Le tableau suivant offre, à titre d'information seulement, un exemple de classement d'un barrage quelconque, en fonction des différents paramètres discutés ci-dessus.

[(1)] La fréquence de réalisation des études de sécurité subséquentes dépend du niveau de conséquence du barrage : Considérable ou très important (10 ans), moyen ou important (10 ans), faible (15 ans) et minimal (20 ans). Pour l'association Canadienne des Barrages, la fréquence est de 10, 7 et 5 ans.

Tableau 2 : Exemple de classement d'un barrage

Barrage	Paramètres physiques constants					Paramètres variables							
	Hauteur (m)	Capacité de retenue (10^6 m³)	Type	Terrain de fondation	Valeur moyene (A)	Âge (années)	Séismicité	État	Fiabilité des appareils d'évacuation	Valeur moyene (B)	Vulnérabilité (V=AxB)	Conséquence (C)	Pointage (P=VxC)
	10,0	78,4	Béton gravité	Roc		26	2	Bon	Adéquate			Important	
Point	2,0	3,06	2,0	2,0	2,27	4,2	1	3	1	2,3	5,22	5	26,1

Classe du barrage : ☐ A (P ≥ 120) ☐ B (70 ≤ P < 120) ■ C (25 ≤ P < 70) ☐ D (P < 25)

Ainsi, le propriétaire d'un barrage dont le niveau de conséquence est « important », est tenue de produire une étude d'évaluation de la sécurité de son ouvrage à un intervalle régulier de 10 ans.

3. ÉTUDES DE SENSIBILITÉ SUR TROIS MODÈLES NUMÉRIQUES :

3.1 Généralités

La modélisation de rupture d'un barrage réalisée pour différentes conditions (normales et crues) a pour but de déterminer l'impact d'un tel scénario sur les infrastructures situées en aval de l'ouvrage, et ce, dans la perspective de réviser le niveau de conséquence dudit barrage. Il s'agit d'un des principaux volets de l'étude de sécurité d'un barrage (en plus du volet « stabilité »).

Dans ce sens, l'ingénieur responsable de l'étude évalue les données en sa possession et procède au montage du modèle numérique en fonction de l'outil de modélisation qu'il utilise. Dans un premier temps, il devra statuer sur le type de modèle le plus représentatif selon le cas traité en répondant à la question incessante : utiliser un modèle unidimensionnel (1D) ou bien opter pour un modèle de deux dimensions (2D) ? La réponse n'est toujours pas évidente, mais peut être dictée par les contraintes budgétaires du mandat ou bien la précision désirée en termes de niveau d'eau et de vitesse d'écoulement. Par exemple, le choix d'un modèle 2D est plus approprié lorsqu'il est question d'analyser les champs de vitesses dans une zone urbaine affectée par la rupture d'un barrage.

Dans la plupart des cas, l'utilisation des modèles unidimensionnels est largement suffisante. Ces derniers offrent une simplicité d'utilisation et un temps de modélisation relativement court. Les modèles numériques (1D) utilisés par les firmes d'ingénieurs au Québec sont FLDWAV (pour Flood wave), MASCARET, et dans une moindre mesure, le logiciel HEC-RAS. Le premier est développé par D. L. Fred du National Weather Service des États Unis. Sa conception repose sur une résolution des équations unidimensionnelles complètes de Saint-Venant.

Le logiciel MASCARET est un outil de modélisation numérique hydraulique (1D), développé par EDF – France. Il s'agit d'un logiciel de modélisation unidimensionnel à surface libre, basé également sur les équations de Saint-Venant.

Développé par le centre des ingénieurs hydrologues de l'« US Army corps of Engineers », le logiciel HEC-RAS offre une résolution des écoulements permanents et non permanents. Il contient un modèle d'analyse du phénomène de transport de sédiment et offre également une analyse de la qualité de l'eau.

La présente section traite d'un cas de rupture de barrage réalisé avec les trois modèles numériques proposées, et offre une synthèse des résultats pour des fins de discussion et de comparaison. Notons que seul le scénario de rupture en temps de crue sera considéré. Mais tout d'abord, voici un bref rappel des notions et équations mathématiques utilisées pour le développement des codes sources de ces modèles.

3.2 Le modèle FLDWAV

Les équations gouvernantes du modèle FLDWAV se composent de trois éléments : les équations unidimensionnelles complètes d'un écoulement non permanent provenant des dérivés des équations de Saint-Venant, l'assortiment des conditions aux limites internes à travers une structure tel un barrage localisé le long d'un cours d'eau et finalement les équations externes des conditions aux limites (amont et aval).

La forme complète des équations de Saint-Venant de conservation de la masse et des moments s'écrit comme suit :

$$\frac{\partial Q}{\partial x} + \frac{\partial S_C\left(A + A_0\right)}{\partial t} - q = 0 \qquad (1)$$

Où :

- Q : débit (m³/s).
- A : Surface active de la section mouillée.
- A_0 : Surface inactive de la section associée à la zone topographique.
- S_e : Coefficient de sinuosité de la profondeur du cours d'eau.
- q : débit latéral (positif si entrant et négatif si sortant).
- t et x représentent les paramètres temporel et spatial.

L'équation de la conservation des moments d'écrit :

$$\frac{\partial\left(S_M\ Q\right)}{\partial t} + \frac{\partial\left(\frac{\beta Q^2}{A}\right)}{\partial x} + gA\left(\frac{\partial h}{\partial x} + S_F + S_E + SI\right) + L - W_F B_a = 0 \qquad (2)$$

Avec :

- S_M : coefficient de sinuosité de la profondeur du canal.
- β: coefficient adimensionnel caractérisant la variation de la vitesse dans la section.
- g : accélération gravitationnelle.
- h : élévation du niveau d'eau.
- L : effet de l'écoulement lateral.
- S_F : coefficient de friction.
- S_E : coefficient expansion/contraction.
- S_I : coefficient de viscosité du fluide.
- B_a : largeur supérieure basse active.
- W_F : effet de résistance de l'eau sur la surface de l'écoulement.

Les coefficients S_F, S_E et S_I s'écrivent comme suit :

$$S_F = |Q|\frac{Q}{K^2}; avec K = \frac{1{,}49}{n} AD^{2/3} S^{1/2} \quad (3)$$

$$S_E = \frac{K_E}{2g} \frac{\partial \left(\frac{Q}{A} \right)^2}{\partial x} \quad (4)$$

$$S_I = \frac{\kappa}{\gamma} \left[(b+2) \frac{Q}{AD^{b+1}} + \frac{(b+2)}{(2D^b)} \left(\frac{\tau_0}{\kappa} \right)^b \right]^{\frac{1}{b}} \quad (5)$$

K_E représente un coefficient de perte par contraction/expansion;

$D = \frac{A}{B}$ Où B est la largeur mouillée à la surface; K la viscosité, γ la masse unitaire du fluide et τ la contrainte initiale du fluide. L'équation interne aux frontières est caractérisée par la formule suivante :

$$Q_I - Q_{I+1} = 0 \quad (6)$$

Le débit est fonction des élévations de niveau d'eau des cellules amont et aval ainsi que des propriétés de la structure de contrôle. Notons que les suffixes i et i+1 indiquent respectivement les sections en amont et en aval. Les équations aux limites externes peuvent se traduire par un hydrogramme de débit ou de niveau d'eau en amont, associées en aval, par exemple, à une relation empirique entre le niveau d'eau et le débit.

La résolution des équations de Saint-Venant par le modèle FLDWAV est initiée par l'application de la technique itérative Newton-Raphson (Fred, 1985). La sensibilité du modèle FLDWAV par rapport à la dimension de la brèche a toujours été un sujet de débat ces dernières années. Ainsi, il en ressort que pour des larges réservoirs, le débit de brèche est sensible à la largeur de cette dernière, et dans une moindre mesure, au temps de sa formation. Dans le cas des petits réservoirs, ce débit est fortement lié au temps de formation de la brèche mais demeure quasi insensible à la largeur de cette dernière (Réf.)

3.3 Le modèle MASCARET

Le modèle MASCARET est composé de trois noyaux de base de calcul hydrodynamique filaire : Fluvial permanent, Fluvial non permanent et transcritique non permanent. Les discussions sur le modèle MASCARET sont basées sur ce troisième type de noyau de calcul.

Le principal objectif du modèle consiste à déterminer les niveaux d'eau et les débits dans les différents affluents d'un réseau hydraulique. Il est basé sur les équations de Saint-Venant unidimensionnelles (équations 1 et 2 légèrement modifiées) utilisant un schéma de différences finies en supposant que le flux de l'écoulement suit une direction privilégiée suivant l'axe du lit mineur de la section. La répartition des pressions est supposée hydrostatique et la vitesse est considérée homogène dans toute la section d'écoulement.

Le code du modèle MASCARET utilise le Schéma de Roe, un schéma explicite de volume fini (Goutal N. et Maurel F. 2002).

3.4 Le modèle HEC-RAS

Le modèle HEC-RAS 4.0 Beta a été conçu pour réaliser des simulations hydrauliques unidimensionnelles pour les trois régimes d'écoulement : torrentiel, fluvial et mixte. Le code numérique du modèle se base sur une résolution de l'équation d'énergie selon une procédure itérative appelée « Standard Step Method ».

$$Z_1 + Y_1 + \frac{a_1 V_1^{\,2}}{2g} + h_e = Z_2 + Y_2 + \frac{a_2 V_2^{\,2}}{2g} \tag{7}$$

Où :

- Z_i : Élévation du cours d'eau.
- Y_i : Profondeur de l'écoulement.
- V_i : Vitesse moyenne.
- A_i : coefficient de poids de la vitesse d'écoulement.
- g : accélération gravitationnelle.
- h_e : perte d'énergie (perte par friction et par contraction ou expansion).

La résolution des écoulements non stationnaires se fait principalement à l'aide des mêmes équations utilisées par FLDWAV (1 et 2), sans considérer le terme de l'effet du vent, le coefficient de contraction et d'expansion (S_E) et le coefficient de viscosité (S_I). HEC-RAS utilise la méthode « sparse matrix linear algebra solver » dans la résolution de l'équation de différence finies, qui est générée par l'utilisation d'un schéma implicite « four points », également de différence finie (comme pour FLDWAV).

Le calcul de l'écoulement lors de la formation de la brèche dans le modèle HEC-RAS est réalisé en utilisant l'équation d'un déversoir avec une correction de la submergence pour évaluer l'élévation de la surface de l'eau en aval réduisant ainsi le débit de l'écoulement lorsque le niveau d'eau s'approche de la côte d'eau en amont.

3.5 Étude de cas : Rupture d'un barrage en béton

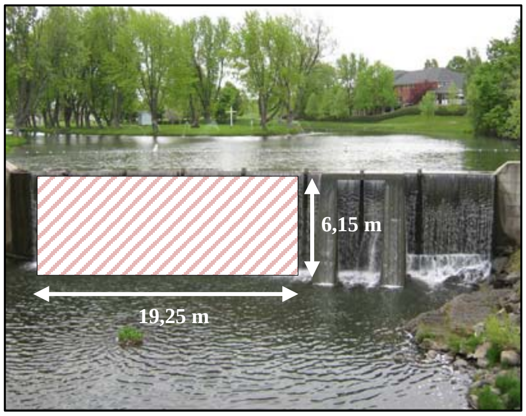
Le barrage proposé pour la présente étude est un ouvrage poids en béton d'une longueur de près de 52 m et d'une hauteur de 6,8 m. Il est composé principalement d'une section centrale de type déversoir, munie d'une vanne demi-fond d'une largeur de 2,6 m et d'une hauteur de 1,2 m.

Figure 1 : Différentes vues du barrage à l'étude

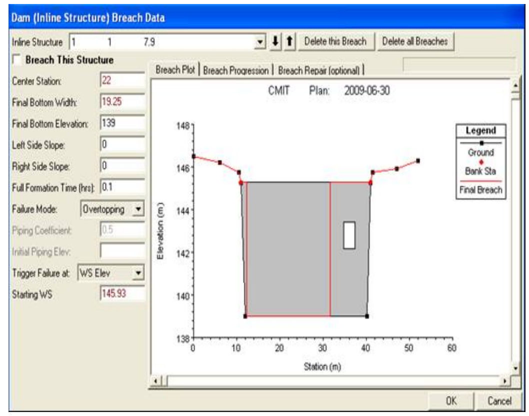
Le montage du modèle numérique est réalisé en considérant 10 sections de rivière, incluant celle du barrage. Le calibrage des modèles tient compte de la rugosité du terrain naturel des lits mineur et majeur. La capacité d'évacuation du barrage a été calculée à l'aide des formules théoriques standards pour un déversoir (muni de brise-jet) et une vanne complètement ouverte. La crête du déversoir est à l'élévation 145,25 m alors que le seuil de la vanne se trouve à l'élévation 142,18 m.

Un hydrogramme de crue, dont la pointe correspond au débit de la crue de 51,0 m³/s, représente la condition aux limites amont, alors qu'en aval, l'écoulement est considéré libre. La rupture du barrage est initiée lorsque le niveau d'eau atteint la côte 145,93 m. Cette côte représente le niveau d'eau maximum atteint au passage de l'hydrogramme de crue au droit du barrage sans considérer sa rupture. La formation de la brèche est instantanée

(6 minutes) et ses dimensions sont montrées dans la figure 2-a. La figure 2-b illustre la définition de la brèche dans le modèle HEC-RAS.



(a) Emplacement et dimension de la brèche



(b) Définition de la brèche dans HEC-RAS

Figure 2 : Caractéristiques de la brèche

Les tableaux suivants illustrent les résultats de niveau d'eau, de débit et de vitesse d'écoulement pour chacun des sections du modèle.

Tableau 3 : Niveaux d'eau maximums

Chaînage (Km)	Lit du cours d'eau (m)	Niveau initial (m)	Niveau maximal (m)		
			FLDWAV	MASCARET	HEC-RAS
-0+020	142,14	145,80	145,93	145,93	145,93
0+000	139,65	145,80	145,93	145,93	145,93
0+065	138,92	140,47	141,82	141,52	141,69
0+245	137,67	139,19	139,87	139,21	140,03
0+405	137,08	138,46	138,95	138,29	138,99
0+500	136,43	137,47	137,96	137,41	138,03
0+535	135,51	137,22	137,62	137,31	137,95
0+835	134,56	136,15	136,35	135,85	136,38
1+165	133,26	134,32	134,60	133,79	134,61
1+400	130,86	132,58	133,19	131,91	132,47

Tableau 4 : Synthèse des débits et des vitesses d'écoulement

Chaînage (km)	Débit maximum (m³/s)			Vitesse d'écoulement maximale (m/s)		
	FLDWAV	MASCARET	HEC-RAS	FLDWAV	MASCARET	HEC-RAS
-0+020	51	51	51	2,7	0,3	2,6
0+000	182	249	196	2,3	0,3	2,8
0+065	182	206	168	4,1	5,4	4,4
0+245	97	72	83	2,2	2,1	1,9
0+405	79	47	88	2,5	2,3	2,2
0+500	74	44	81	3,3	3,1	2,8
0+535	73	47	80	2,4	2,1	1,9
0+835	62	44	71	1,2	2,2	2,0
1+165	59	30	67	2,1	2,7	1,9
1+400	57	19	48	2,0	2,1	3,3

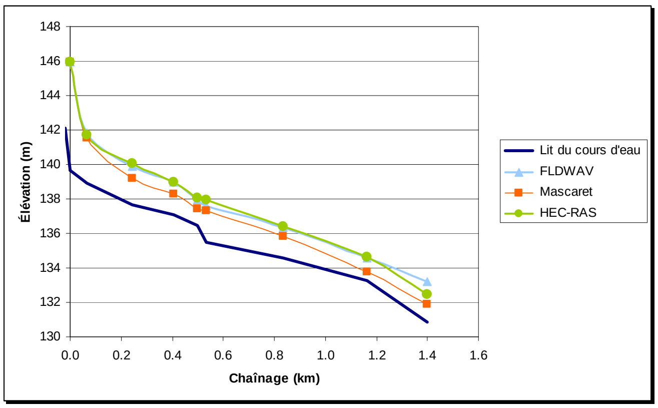


Figure 3 : Synthèse des profils longitudinaux des niveaux d'eau maximums dans la vallée en aval du barrage résultants des trois modèles utilisés

Les vitesses d'écoulement pour les trois modèles sont du même ordre de grandeur, alors que le débit maximum, au barrage, obtenu avec le modèle MASCARET est d'environ 28 % à 37 % supérieur comparativement aux autres modèles (à cause des différences dans les schémas numériques adoptés et aussi le traitement de la submergence par l'aval qui peut affecter le débit de pointe à la rupture et ainsi le niveau d'eau juste en aval du barrage). La différence de niveau entre MASCARET d'un côté et FLDWAV et HEC-RAS de l'autre est assez remarquable. Elle est située entre 0,3 m et plus de 1,0 m.

Bien que les deux modèles HEC-RAS et FLDWAV traitent les mêmes équations hydrodynamiques en régime non stationnaire, les résultats peuvent différer en raison des techniques de résolution propres à chacun. Nous limitons dans le présent document à citer deux éléments majeurs, à savoir la définition du coefficient de Manning et l'évaluation du coefficient de friction, notamment dans le calcul du coefficient K (équation 3). Pour le modèle HEC-RAS, la définition et le calcul du coefficient K se font pour les trois composantes de la section : rive droite, rive gauche et cours d'eau « central » ou « principal ». Une valeur est ensuite établie pour la section d'écoulement, alors que FLDWAV évalue ces paramètres selon des couches d'élévation horizontales (largeur-élévation). L'implémentation des valeurs de Manning pour le cours d'eau « central » et les rives droite et gauche est une option dans le modèle qui est principalement utilisée pour le calibrage du modèle numérique. Toutefois, les diverses discussions que nous avons eu avec d'autres utilisateurs FLDWAV montrent que cette option est rarement utilisée du fait que la précision demandée ne le justifie tout simplement pas.

En ce qui concerne le modèle MASCARET, il n'offre qu'une définition du coefficient de Manning pour le lit mineur et le lit majeur; ce qui est, dans certains cas, loin d'être représentatif.

Un élément évident à constater lors de la comparaison de ces modèles réside dans le temps de calcul. En effet, le modèle MASCARET se distingue par un temps de calcul largement supérieur par rapport à FLDWAV et HEC-RAS, dont la résolution des schémas numériques se fait quasi-instantanément. Il faut dire que le schéma numérique de MASCARET implique beaucoup d'interactions entre cellules voisines issues du maillage, ce qui fait affecte considérablement le temps de réponse à chaque itération.

4. APPROCHE POUR LA SÉCURITÉ PAR LA CONCEPTION DES BARRAGES EXISTANTS :

4.1 Généralités

Un barrage est en général une structure complexe, dont le comportement durant son cycle de vie est influencé par plusieurs facteurs et phénomènes plus ou moins bien connus : - caractéristiques de matériaux (vieillissement), - comportement de la fondation,- les effets chimiques de l'eau,- sollicitation sismique,
- risque hydrologique,- mode d'exploitation, et dont les exigences quant à la sécurité sont extrêmes que ce soit en conception, en réalisation et en exploitation. Les barrages présentent des enjeux importants sur le plan de la sécurité, car leur rupture aurait des conséquences catastrophiques en raison de l'importance des coûts et des travaux de maintien en état. En conséquence les fonctions attendues des barrages, stabilité structurelle et étanchéité doivent être contrôlées, pour s'assurer en permanence de leur intégrité et de leur performance. En effet les processus de vieillissement peuvent à la longue altérer ces fonctions essentielles. L'évaluation de la sécurité sur les structures existantes peut présenter des problèmes uniques et spéciaux. Les petits barrages présentent autant de difficultés que les très grands barrages et de ce fait peuvent être difficiles à analyser. Les critères de conception étant devenus plus exigeants et intégrant les avancements des règles de l'art, ces barrages doivent être réévalués sur les critères de conception, les conditions géologiques, structurales actuels. Nous présentons brièvement les problématiques rencontrées dans les barrages en béton et en terre.

4.2 Problématiques rencontrées dans les barrages existants

Un barrage étant une structure s'opposant au passage de l'eau et résistant à sa poussée, différents types de structures peuvent être utilisés mais ceux rencontés dans notre cas sont les barrages poids en béton et les barrages en terre. Ces structures sous l'effet de diverses sollicitations, en particulier par la présence de l'eau, vieillissent et se dégradent. En effet les processus de vieillissement internes peuvent à la longue altérer la stabilité structurelle et l'étanchéité. Les causes de vieillissement étant multiples, il importe donc au plus tôt d'identifier ces processus de vieillissement et de diagnostiquer leur niveau gravité.

Les barrages poids, établis en général sur des fondations rocheuses, sont surtout sensibles aux phénomènes suivants :

- le passage d'une crue dépassant la crue de projet pourrait compromettre la stabilité d'ensemble;
- le colmatage des drains de fondation entraînant une augmentation des sous pressions sous la base du barrage et diminuant la stabilité;
- le vieillissement du corps du barrage par réaction chimique des constituants de béton avec l'eau et par l'action du gel développant les maladies du béton. Ce béton devient à la fois moins étanche et moins résistant mécaniquement. Ce phénomène a pour conséquence d'augmenter les sous-pressions et de diminuer la stabilité d'ensemble.
- les dégradations éventuelles au niveau des joints.

Pour les barrages en remblai dont la fonction étanchéité est assurée par la terre, les principales pathologies susceptibles de conduire à des désordres, voire à des ruptures sont :

- des pressions interstitielles excessives apparaissant lors de la construction du remblai peuvent diminuer la résistance de cisaillement,
- les tassements de la crête du remblai entraînant une diminution de la revanche;
- le colmatage des drains qui, à terme, peut atteindre le talus aval et mettre en danger la stabilité du remblai;

- les circulations d'eau à travers le remblai ou la fondation, non contrôlés par le système de filtration et de drainage, peuvent par érosion interne conduire à un phénomène de renard;
- la pression transmise par l'eau peut soulever le sol par sous pression et ruiner l'ouvrage.

Les Figures 4 et 5, prises lors des inspections effectuées en 2008, montrent les dégradations subies au cours de son cycle de vie, d'un barrage en béton type gravitaire au Québec (datant de 1895, H= 5m, V= 453 200 m^3). Ce barrage est actuellement en cours d'étude d'évaluation de sa sécurité y compris les études géotechniques et matériaux.

Figure 4 : Désagrégation avec une zone de fuite

Figure 5 : Désagrégation du béton des contreforts

La stabilité d'ensemble est alors devenue l'élément prépondérant pour dimensionner ces ouvrages. Le barrage et sa fondation doivent résister avec une sécurité suffisante aux forces qui les sollicitent et l'ensemble de l'ouvrage doit être étanche de telle sorte que les fuites soient sans danger pour sa résistance. L'approche traditionnelle simplifiée et conservative est utilisée pour la première étape de l'évaluation de la stabilité.

Les barrages pouvant induire un potentiel de risques très significatif, définir un processus de surveillance et d'évaluation pour réduire au maximum les probabilités de défaillance devient nécessaire. De nombreux pays se dotent alors de lois, d'agences et de mécanismes de contrôle des barrages et intensifient leurs efforts de recherche et de développement :

- La Commission Internationale des Grands Barrages (CIGB) définit un ensemble d'activités concomitantes, bien coordonnées et raisonnablement agencées (Kert C, 2008);
- L'US Army Corps of Engineers (USACE, 2005) propose des procédures pour l'évaluation et l'amélioration de la stabilité sur des structures existantes;
- L'Association Canadienne des barrages ACB/CDA définit des guidelines pour la sécurité des barrages;
- Le Québec, une loi de sécurité dont les procédures que nous avons appliquées sont décrites dans le paragraphe suivant.

4.3 Procédures pour l'évaluation de la sécurité des barrages selon loi de sécurité du Québec

Pour assurer la sécurité des barrages, leur pérennité, leur intégrité et pour protéger les personnes et les biens contre les risques associés aux barrages, l'approche adoptée suit les articles 48 et 49 de la loi de la sécurité du Québec. Pour cela nous nous intéressons dans cette section à la vérification :

- de l'état et du comportement au moyen d'inspections détaillées des composantes du barrage, analyse des résultats des activités de surveillance, d'auscultation;
- de la conception du barrage au moyen au moyen la vérification des critères de conception, les données, et méthodes d'analyse considérées de la stabilité et du terrain de fondation, réalisation d'études géotechniques;

- de la fonctionnalité et de la fiabilité des appareils d'évacuation : le contrôle des organes d'évacuation constitue une activité primordiale pour la sécurité des barrages. L'historique des incidents de barrages montre que le sous dimensionnement et/ou le mauvais fonctionnement des organes d'évacuation sont responsables d'une part prépondérante de ces incidents;
- des dispositifs de sécurité notamment les systèmes d'urgence, les systèmes de détection des situations d'urgence et des systèmes d'appoint.

5. ÉTUDE SUR DEUX (2) BARRAGES EXISTANTS

5.1 Barrage en béton

Pour ce barrage présenté ci- dessus, certaines données ont pu être récoltées grâce aux documents existants : devis de construction, des plans avec coupes et détails pour travaux, composante du système de levage, documents pour travaux de réfection précédents et notre étude a consisté en :

5.1.1 État et comportement

- Programme d'investigation : deux (2) inspections visuelles des composantes du barrage (déversoir, vanne en mode d'ouverture, murs d'ailes droite et gauche) ont permis de constater des composants en bon état. Les petits barrages n'étant pratiquement pas auscultés, l'inspection visuelle de sécurité est encore plus nécessaire et plus utile.
- Étude de stabilité : elle a été effectuée en utilisant plusieurs combinaisons de charges pour les cas Normal, de crue et de séisme pour vérifier des indicateurs de performance à l'aide du logiciel CADAM (Leclerc M., Leger P., Tinawi R., 2002). Les résultats de l'analyse ont montrée un non respect au glissement et en renversement surtout dans le cas de chargement opérationnel avec glace.
- Fiabilité et Fonctionnalité des appareils d'évacuation : les essais de fonctionnement du déversoir et de l'ouverture et fermeture de la vanne à l'aide d'un mécanisme d'alimentation hydraulique externe. La fiabilité des appareils d'évacuation (déversoir et vanne) est adéquate.
- Anomalies et autres observations : présence de petits morceaux de bois du coté de la rive gauche.

5.1.2 Recommandations

- de renforcer la structure (contreforts, ancrage).
- mise en place d'un programme d'inspection.
- Tenue de visites régulières pour procéder au nettoyage et à l'entretien régulier du barrage et de ses abords.

5.2 Barrage en terre

C'est un ouvrage en terre construit dans les années *quarante* et modifié en 1988 par la mise en place d'un déversoir en béton avec poutrelles. Sa hauteur est de 1,75m avec une hauteur de retenue de 1,25m. L'analyse des données s'est faite sur le peu de données et d'informations existants, et notre étude a porté sur :

5.2.1 État et comportement du barrage

- Études géotechniques : 2 forages ont été effectués sur les digues rive droite et gauche à partir desquels les caractéristiques des sols et les paramètres hydrauliques ont été déterminées par des essais de laboratoire. L'analyse a révélé l'existence de sols organiques au bas de la structure de la digue.
- Étude de stabilité : les analyses de stabilité ont été réalisées à l'aide de la suite GeoStudio de GEO_SLOPE International Ltée. Une étude pour l'établissement du réseau d'écoulement en régime permanent et une analyse de la stabilité de la digue ont été réalisées. Toutes les analyses de stabilité ont été effectuées en mode statique à long terme dans les conditions d'exploitation normales et en période de crues avec les caractéristiques des sols et les paramètres hydrauliques évaluées à partir de l'étude géotechnique. Les résultats d'analyse ont montré qu'il y a non-conformité; le niveau d'eau en période de crue dépasse la crête

de la digue et les facteurs de sécurité ne sont pas respectés. L'ensemble de l'ouvrage est mis en jeu à cause de l'effet de renard.
- Fiabilité et fonctionnalité des appareils d'évacuation : n'a pas la capacité suffisante d'évacuation de crues.
- Anomalies et autres observations : présence de petits morceaux de bois du coté de la rive gauche. Les digues sont envahies de végétation indésirable (arbres, toute sorte de végétation etc.).

5.2.2 Recommandations

- nécessité de reconstruction de l'évacuateur, l'évacuateur actuel sous dimensionné
- revoir la composition complète de la digue sur ses deux cotés de la rive et concevoir une nouvelle digue
- un suivi régulier devrait être effectué par un technicien/ingénieur en géotechnique

Une recommandation pour travaux de mise en norme est de ne pas obstruer l'écoulement par la route et les 2 ponceaux en aval

Des plans de gestion de l'eau et des mesures d'urgence ont été adressés au propriétaire du barrage dans les 2 cas.

5.3 Sécurité du site et des alentours

Nous avons constaté que les barrages pour lesquels nous réalisons des études d'évaluation de la sécurité sont souvent de type soit municipal (prise d'eau) et villégiature. Ils sont souvent entourés de résidences principales et d'infrastructure (ponts, routes, pistes cyclables etc..) proche du barrage. Ils sont par conséquent susceptibles d'être inondées en cas de rupture du barrage. Par conséquent, des mesures et moyens de sécurité aux environs des barrages doivent être envisagées et adoptés après avoir identifié les dangers potentiels. Nous avons eu à proposer :

- la mise en place d'estacades ou câble d'acier à l'amont du barrage pour prévenir des accidents dus aux embarcations des villégiateurs qui peuvent être entrainées en cas de crue vers les vannes du barrage;
- Surélévation du garde-corps;
- Protection par une gaine métallique des fils d'alimentation électrique de la prise de courant;
- Installation d'éclairage sur le barrage pour des manœuvres de nuit;
- Éviter l'accès aux appareils d'évacuation aux personnes non autorisées;
- Permettre l'accès à certaines parties du barrage pour inspection et entretien;
- Signalisation avec un message mentionnant le degré du risque;
- Signalisation pour guider le public aux points d'accès;
- Élimination de la végétation sur les structures du barrage.

5.4 Impacts potentiels

L'application de la loi de la sécurité des barrages, outre le fait qu'elle permet une meilleure gestion du risque : à savoir une surveillance constante du barrage par de fréquentes inspections entrainant la détection prématurée des zones d'érosion interne et des fuites, l'amélioration de la stabilité et de la tenue d'ouvrages, l'amélioration des aspects environnementaux et un contrôle concernant les mesures de sureté prises, ainsi que l'information préventive de la population et l'alerte, une meilleure organisation des secours, et la coordination entre les besoins d'ordre commerciale et productif développe chez les propriétaires ou gestionnaires une prise de conscience et les rend plus responsable.

6. INTERPRÉTATION DES RÉSULTATS DES ÉTUDES DE SÉCURITÉ

Bien qu'il existe d'autres modèles numériques également conçus pour la modélisation de rupture de barrages, les trois modèles présentés dans le présent document demeurent les plus utilisés en Amérique du Nord, et surtout au Québec. Les problèmes d'instabilités rencontrés lors des essais réalisés avec HEC-RAS montrent qu'il n'est pas encore au point pour faire face à des scénarios de rupture plus complexes.

Dans le cas de l'étude de rupture traitée dans ce document, plusieurs éléments doivent inciter l'ingénieur responsable de l'étude à faire une réflexion quand au choix des modèles numériques à utiliser. À titre d'exemple, nous mentionnons qu'au chaînage 0+065 se trouve une résidence principale accotée sur la rive gauche de la vallée en aval du barrage. Son niveau de terrain est à l'élévation 142,40 m. Il existe donc une dénivelée de 0,58 m par rapport au résultat avec le modèle FLDWAV, 0,88 m pour MASCARET et 0,71 m pour HEC-RAS. La modélisation de rupture avec le modèle FLDWAV contraindra l'ingénieur à considérer cette maison à risque surtout sachant la faible dénivelée et aussi le potentiel d'érosion des berges.

Par ailleurs, et selon la figure 1, le niveau d'eau obtenu avec le modèle MASCARET est inférieur à celui de FLDWAV ou HEC-RAS. Il y a donc une surestimation de niveau d'eau par ces deux derniers modèles, ou bien une sous-estimation par le modèle MASCARET. Le choix d'un outil de modélisation peut donc engendrer des différences remarquées lors du tracé des zones inondées, ce qui peut modifier considérablement le niveau de conséquence attribuable à une éventuelle rupture du barrage. Ainsi, en utilisant le modèle FLDWAV, le niveau de conséquence du barrage étudié serait « moyen », alors qu'il sera établi à « faible » si l'étude de rupture serait réalisée avec les modèles HEC-RAS ou MASCARET.

Par ailleurs, il importe de mentionner que l'introduction du volet rupture de barrage dans le modèle HEC-RAS est assez récente, ce qui ne fait pas de ce modèle, le choix privilégié pour la modélisation de rupture de barrages, du moins pour le moment. Le modèle HEC-RAS demeure toutefois le meilleur outil pour les études de conception hydraulique des structures (pont, pertuis, vannes, etc…) et des canaux (largeur, longueur, type, etc…) étant donné sa capacité à intégrer et à manipuler, très facilement, ces paramètres.

Actuellement, bon nombre des études de ruptures sont ainsi réalisées avec les modèles FLDWAV et MASCARET. Mis à part le temps de calcul qui favorise largement l'utilisation de FLDWAV, ces deux modèles sont assez robustes pour supporter des cas de rupture assez complexes, y compris les ruptures en cascades, les écoulements latéraux et l'étude de plusieurs biefs simultanément. Globalement, les réponses en termes de niveau d'eau et de débit sont quelque peu différentes, et nécessitent une profonde réflexion de l'ingénieur responsable de l'étude. Dans des cas particuliers, le recours aux modèles numériques de deux dimensions (2D) s'impose.

La méthode d'analyse de la stabilité pour les ouvrages poids en béton utilisée est l'approche conservatrice dans la détermination des charges, des cas de charges et des propriétés des matériaux. Pour les barrages en terre l'approche utilisée est celle de l'équilibre limite intégrée dans le modèle Géo-Slop. La stabilité est vérifiée pour plusieurs scénarios. Certaines hypothèses ont été prises lorsque l'information est inexistante surtout dans le cas de petits barrages qui ne comportent pas d'outils d'auscultation. Cependant la stabilité globale est vérifiée par l'imposition de critères de performance sur des indicateurs prédéfinis (contraintes, facteurs de sécurité, etc.) pour assurer une marge de sécurité contre la rupture.

7. CONCLUSION

Les événements d'inondations de barrages survenus dans les années 80 et 90 ont suscité une mobilisation sans précédent des gouvernements de tous les pays dans le but de mieux cadrer l'exploitation et la gestion des barrages. La Loi et le Règlement adoptés par le gouvernement du Québec établissent les normes minimales de sécurité des barrages selon une méthodologie et une approche bien définies. Le propriétaire d'un barrage de catégorie forte contenance, est soumis, selon la classe et le niveau de conséquence de l'ouvrage, à des exigences particulières quant à l'exploitation et la gestion des eaux retenues. La réalisation d'une étude d'évaluation de la sécurité, par un ingénieur, est devenue l'une des obligations qui découle de la nouvelle Loi. À l'issue de cette étude, le niveau de conséquence du barrage est révisé et un nouveau classement du barrage est établi suivant le processus présenté dans ce document.

L'étude de sécurité est composée de deux volets principaux, soit l'hydrologie/hydraulique et la structure. D'un point de vue hydraulique, et dans la majorité des cas, l'ingénieur responsable de l'étude est tenu de réaliser des modélisations de rupture de barrages pour plusieurs scénarios extrêmes. Le choix d'un modèle numérique doit être bien réfléchi puisque la diversité des résultats, tel que montré dans le présent document, peut conduire à des conclusions bien différentes. Il en est de même lorsqu'il est question d'évaluer la stabilité d'un barrage qu'il soit en béton ou en terre. Les différentes hypothèses posées par l'ingénieur doivent être fondées, le plus possible, sur des éléments et des observations de terrain. Toutefois, la principale problématique qui se pose pour les barrages existants est le manque de données (forage, composition du terrain de fondation etc..). De plus, il n'existe pas de plans pour ces ouvrages; l'ingénieur est donc confronté à explorer d'autres sources pour combler le manque d'information. Les résultats obtenus à l'aide d'un modèle numérique sont également à examiner avec rigueur car il présente souvent des schémas simplistes utilisés pour absorber la complication d'un cas à l'étude (géométrie et paramètres physiques par exemple).

Le présent document met l'emphase également sur la sécurité par la conception d'un point de vue structure pour les barrages existants (terre ou béton). Nous avons tout d'abord introduit brièvement les problématiques rencontrées par les barrages en béton et en terre, ensuite nous avons introduit les procédures de l'évaluation de la sécurité des barrages selon la loi du Québec que nous avons appliquées à deux (2) barrages, l'un en terre l'autre en béton. Il en résulte que la sécurité des biens et des personnes passe par une série de processus visant non seulement le barrage proprement dit, mais aussi le site et ses alentours.

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TESTING RIVER2D AND FLOW-3D FOR SUDDEN DAM-BREAK FLOW SIMULATIONS

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ABSTRACT:

The two-dimensional flow model River2D and the three-dimensional Computational Fluid Dynamics (CFD) model Flow-3D were tested for the first time to simulate instantaneous dam-break flows using high-quality experimental data. The computer simulations were challenging because the sudden dam-breaks generated highly unsteady and rapidly varied flows moving over initially dry beds, including large obstacles and non-flat bed topography. The flows were characterized by the presence of strong hydraulic jumps and waves moving over the computational domain. River2D performed well in cases when the flood wave moved over a flat bed; however, when there was a rapid rise in bed levels its performance was not satisfactory. It was found that River2D's groundwater model, intended for handling wetting and drying in normal (low slope) river flows, caused an excessive amount of surface water to artificially flow underground in these situations. This present limitation would prevent River2D from being used to simulate dam-break flows over natural bed topography. Flow-3D performed well in the three test cases analyzed, without requiring excessive computational times. Flow-3D seems well suited for practical applications, especially when the flow details near the dam are sought.

RÉSUMÉ:

La modélisation de ruptures subites de barrages a été évaluée pour la première fois en utilisant les modèles d'écoulement bidimensionnel River2D et tridimensionnel Flow-3D avec un ensemble de données expérimentales. La modélisation fut complexe car les écoulements résultant d'une rupture sont transitoires et varient rapidement sur des surfaces initialement sèches, souvent accentuées, et qui comprennent habituellement des obstacles. Les écoulements sont caractérisés par des ressauts hydrauliques et des ondes qui se déplacent sur le domaine de modélisation. Le modèle River2D représentait bien les conditions lorsque l'onde de crue se déplaçait sur un terrain plat, mais les résultats étaient moins satisfaisants lorsque les niveaux du lit étaient accentués. L'étude a aussi démontré que le modèle de nappe phréatiques de River2D, qui prend en considération les cycles d'humidification et de dessiccation, a été responsable pour une quantité excessive et artificielle d'eau souterraine. Cette contrainte limite l'utilisation du modèle River2D pour la simulation d'écoulement sur des terrains naturels suite à la rupture de barrages. Le modèle Flow-3D a donné de bons résultats dans tous les trois cas à l'étude sans requérir un temps de calculs excessif. Pour les applications pratiques, Flow-3D semble être convenable, surtout lorsqu'il y a un intérêt pour l'écoulement près du barrage à une échelle plus détaillée.

1. INTRODUCTION

It is well known that the propagation of a flood wave generated by a sudden dam-break event can have catastrophic effects downstream (Hervouet and Petitjean 1999; Begnudelli and Sanders 2007; Alcrudo and Mulet 2007). The dynamics of the dam-break wave propagation is quite complex and its behaviour does not comply with the common assumptions of conventional steady and gradually-varied open channel flows. Dam-break flows are highly unsteady and rapidly varied, typically with mixed subcritical and supercritical flow regimes. The flow dynamics can be further complicated if the wave front advances over areas that were initially dry. An analytical description of these complex flows can only be achieved for very simple cases of limited practical application.

In the past decades, the ability to predict the dam-break flows has increased with the application of numerical models. In most practical dam-break applications, one-dimensional (1D) numerical modelling is commonly used to simulate the flood wave propagation downstream from the dam. However, in certain cases the simplifications assumed by 1D models may be too restrictive to accurately reproduce the flood wave dynamics; for example when the flow is not confined to a single channel, the channel has sharp bends or large obstacles are present in the flooded area (Hervouet and Petitjean 1999). Two-dimensional (2D) models offer the possibility of providing a better description of the flow, and although their application is not as widespread as 1D models, 2D models have been successfully applied to simulate real dam-break events (Hervouet and Petitjean 1999; Valiani et al. 2002; Begnudelli and Sanders 2007). Although 2D depth-averaged flow models neglect the vertical velocity component and assume a hydrostatic pressure distribution, such simplifications are valid in most cases. However, on certain localized areas the vertical velocity may be important - for example right at the dam section where the flow is rushing down or when the flood wave hits a large obstacle (like a building, a bridge or a powerhouse). Such special cases may require the application of a full three-dimensional (3D) flow model that solves for the three velocity components under non-hydrostatic conditions. However, the applications of 3D flow models for dam-break flows seem very limited at present, with only a few examples could be found in the literature.

DeMaio et al. (2004) applied the commercial model Fluent and a 2D vertical mesh to compute the water surface profile approximately 1 second after an instantaneous dam-break represented by a sudden gate opening. To compute the location of the free surface, Fluent used the Volume-of-Fluid (VOF) method. Lohner et al. (2006) developed a 3D VOF flow model that was applied to simulate the free surface movement after a sudden dam-break, as well as the interaction of the flood wave with a cylindrical pier, all under laminar flow conditions. Recently, Crespo et al. (2008) used a 2D vertical Smoothed Particle Hydrodynamics (SPH) model to simulate an instantaneous dam break caused by a sudden gate opening. More detailed validations of full 3D turbulent flow models with experimental data of sudden dam-break were not found.

The purpose of the work presented here is to test and verify two popular numerical flow models, River2D and Flow-3D, by using high quality experimental data of sudden dam-breaks flows. First, a brief description of the numerical models is presented. Later, three experimental test cases are used to validate the models. The results showed that River2D only provides good results when the bed is flat; while Flow-3D provided good results in every case.

2. NUMERICAL MODELS

The 2D depth-averaged flow model River2D and the 3D computational fluid dynamics (CFD) model Flow-3D were selected for the numerical simulations. A brief description of both models follows.

2.1 River2D

River2D is a freely available (www.River2D.ca) flow model developed by the University of Alberta (Steffler and Blackburn 2002). The model uses a triangular Finite Element (FE) computational mesh to compute water depth (*h*) and the two depth-averaged velocity components (*u*,*v*) in the horizontal plane. The unstructured triangular mesh provides great flexibility to model complex and irregular plan-form geometries. Some comments on how River2D compares to other FE flow models are found below.

The first generation of FE flow models, such as the US Army Corp's RMA2 or the US Federal Highway Administration's FESWMS, relied on artificial eddy viscosity for numerical stability, these models were mainly limited to steady subcritical flow conditions and had difficulties in simulating wetting and drying; therefore, they could not be used to model sudden dam-breaks. Newer FE flow models, such as River2D and Electricté de France's Telemac-2D, use modern shock-capturing techniques to achieve stable solutions, even in cases with supercritical flow and hydraulic jumps. Telemac-2D has successfully been applied to model real-case dam-break flows (Hervouet and Petitjean 1999), while River2D has been applied successfully to simulate dam-break flows in laboratory flumes with flat beds only (Ghanem et al. 1995a, 1995b). One important difference between Telemac-2D and River2D is the manner in which they handle wetting and drying. River2D uses a physically-based groundwater model that activates whenever the water depth falls below a minimum value h_{min}. In this way a continuous and smooth water surface is computed above and below the ground, avoiding some of the problems in partially-dry elements exhibited by Telemac-2D (Hervouet and Janin 1994, Ghanem et al. 1995a). River2D computes the groundwater discharge per unit width in the aquifer q_a as

$$q_a = TS \tag{1}$$

Where T is the aquifer transmissivity (permeability times aquifer thickness) and S is the water surface slope in the aquifer. For normal river flows, q_a is in the order of 1% of the surface flow. Hence, only a small portion of the total river discharge is expected to flow into the aquifer.

River2D is in fact a suite of three executable programs: a topographic bed interpolation model (Bed.exe), an automatic mesh generation model (Mesh.exe) and the hydrodynamics model (River2D.exe). Bed.exe is an optional program intended to help generate the bed file, which contains the topographic and roughness data. Bed.exe has tools to generate triangular irregular networks (TIN) from scatter topographic data points. Bed break-lines can also be incorporated to better define linear topographic features, such as banklines or thalweg. Mesh.exe is used to generate the triangular mesh and impose boundary conditions; it has several tools and options to optimize the mesh geometry. All the programs are written in Visual C++ and have their own graphical user's interface with good graphic capabilities. River2D can generate colour plots of several computed variables, as well as vector velocity plots. It is also possible to extract profile information using user-defined data points. Although the suite of programs eliminates the need for third-party pre- and post-processing software, the input and output files are written in a simple ASCII format that make it easy to re-format the files for transferring to other software, if desired.

Among several consulting companies and research institutions, Northwest Hydraulic Consultant (NHC) has been using River2D extensively over the last 10 years. NHC has compared River2D with field data collected in several rivers in Canada and South America, and in every case the model has performed notably well (Vasquez and Lima 2009). For example, in an unsteady tidal simulation of the Lower Fraser River, River2D matched observed water levels with 3 cm accuracy, even when the tidal amplitude exceeded 1 m. Vasquez (2005) showed that River2D was able to reproduce reasonably well the recirculation zone in a channel bifurcation. Vasquez and Leal (2006) showed that River2D compared well with experimental data of sudden dam-breaks in straight and curved laboratory channels. Despite these good results for general river flow modelling, River2D has not been thoroughly tested for sudden dam break flows with obstacles and varying bed elevation.

2.2 Flow-3D

Flow-3D is an advanced 3D CFD model developed by Flow Science Inc. in Santa Fe, New Mexico. Flow-3D computes the three velocity components (u,v,w) and pressure at the nodes of a orthogonal Finite Difference Grid, using different turbulence models (k-ε, RNG, LES). Among the general-purpose CFD programs commercially available in the market, Flow-3D stands out for its capabilities intended for hydraulic engineering applications: it has excellent capabilities for modelling free surface flow; it can read topographic files in (x,y,z) format; boundary conditions can be specified as a discharge hydrograph or water level hydrograph; and, local scour at bridge piers can also be simulated (Vasquez and Walsh 2009). Flow-3D has been used extensively to model structures such as spillways, stilling basins, water intakes, fish ladders and similar instream structures.

As an interesting fact, Flow Science Inc. was founded by Dr. C.W. Hirt, who pioneered the VOF method, a unique free surface tracking technique. The VOF is at present the best method available to simulate the movement of rapidly-varying water surfaces and it is used by all commercial CFD programs when modelling free surfaces (e.g. Fluent, Star-CCM+, CFX). However, references where Flow-3D has been tested for dam-break flow simulations could not be found in the literature.

In summary, both River2D and Flow-3D seem in principle capable of simulating sudden dam break flows. However, probably due to the lack of good quality experimental data, the real capabilities of both models to simulate dam-break flows has not been properly verified. Recently, as a result of the efforts of a multinational European research project named IMPACT -Investigation of Extreme Flood Processes and Uncertainty- (www.impact-project.net), several high quality experimental data sets on dam-break flows have been made available to the general public (Zech and Soares-Frazao 2007). Some of these data sets have been used as test cases for verifying the performance of River2D and Flow-3D, as presented in the following sections.

3. STRAIGHT CHANNEL WITH BED STEP

The first case considered in this work involved a relatively simple straight channel configuration with a change in bed levels at the dam location. Since sediment tends to naturally deposit upstream from a dam, in the event of a dam-break, it is likely that the bed elevation upstream from the dam will be higher than the bed level downstream. To study the effect of higher bed elevations upstream from a dam, Leal et al. (2002) performed dam-break experiments in a rectangular horizontal flume with a 0.12 m high bed step to represent sediment deposition upstream from a dam. The flume was 19.2 m long, 0.5 m wide and 0.7 m high, and the dam was simulated by a vertical slide gate installed at the downstream end of the bed step. The bed level downstream from the dam was set at z_d = 0.071 m; while the bed level upstream was z_u = 0.190 m. The water level (WL) upstream from the dam was set to 0.59 m. The water depth downstream from the dam was zero (dry bed) for Test Ts-25 and 0.075 m (wet bed) for test Ts-28. These tests were actually performed using a movable bed (sand) and were reproduced by Vasquez and Leal (2006) using River2D-MOR, the movable-bed version of River2D. However, the duration of the experiment is too short for the movable bed to have a very strong influence on the water levels, and hence the bed is assumed as fixed for the present simulations.

The computational mesh in River2D was L_x = 20 m long and L_y = 0.5 m wide with roughness height k_s = 0.0164 m. Two different meshes with a node spacing of about 0.025 m and 0.050 m were used in a preliminary analysis. Both meshes led to identical results, indicating that the solution was mesh independent. The results reported here are for the coarser 0.050 m, whose features are summarized in Table 1. Flow-3D's rectangular grid had a cell size of 0.02 m in the horizontal (x,y) direction and 0.01 m in the vertical direction. The number N of cells in the three spatial directions, as well as the total number of nodes, are detailed in Table 2. Since this is basically a one-dimensional flow experiment and in order to save computational time, the Flow-3D mesh was made narrower (L_y = 0.2 m) than the width of the experimental flume. The Flow-3D model used the same roughness height used by the River2D model (Table 1).

Figure 1 shows the longitudinal profiles of water levels at 1 s and 4 s after dam break for the two experiments, Ts-25 and Ts-28. Each plot shows the experimental data points, and the profiles computed by both River2D and Flow-3D. For experiment Ts-25, both models capture very well the movement of a sharp water front over the initially dry bed downstream from the dam. For experiment Ts-28, both models correctly simulate the formation of a hydraulic jump and a bore that migrates downstream; although Flow-3D seems to better reproduce the shape of the hydraulic jump right downstream from the bed step.

In general, the agreement between the experiment and the two numerical models is quite good and it is difficult to state which model performs better. At time t = 1 s, it appears that Flow-3D produces a better agreement with experimental data; perhaps because the vertical velocity and non-hydrostatic effects –both ignored by River2D–

are more important at this moment in time when water is quickly rushing down from the reservoir. However, by $t = 4$ s, it appears the River2D produces a better agreement with the experimental data.

Table 1: Mesh features and roughness of River-2D models

Dam-break Experiment	Number of nodes	Number of elements	Roughness height (m)
Channel with bed step	6,076	10,935	0.0164
Triangular sill	5,586	9,880	0.0005
Isolated obstacle	10,933	20,852	0.0002

Table 2: Domain dimensions and number of cells of Flow-3D meshes

Dam-break	Dimension (m)			Number of cells			Total No.
Experiment	L_x	L_y	L_z	N_x	N_y	N_z	of cells
Channel with bed step	20.0	0.20	0.60	1000	10	30	300,000
Triangular sill	5.6	0.50	0.16	560	50	32	896,000
Isolated obstacle	17.7	3.64	0.46	501	182	46	4,194,372

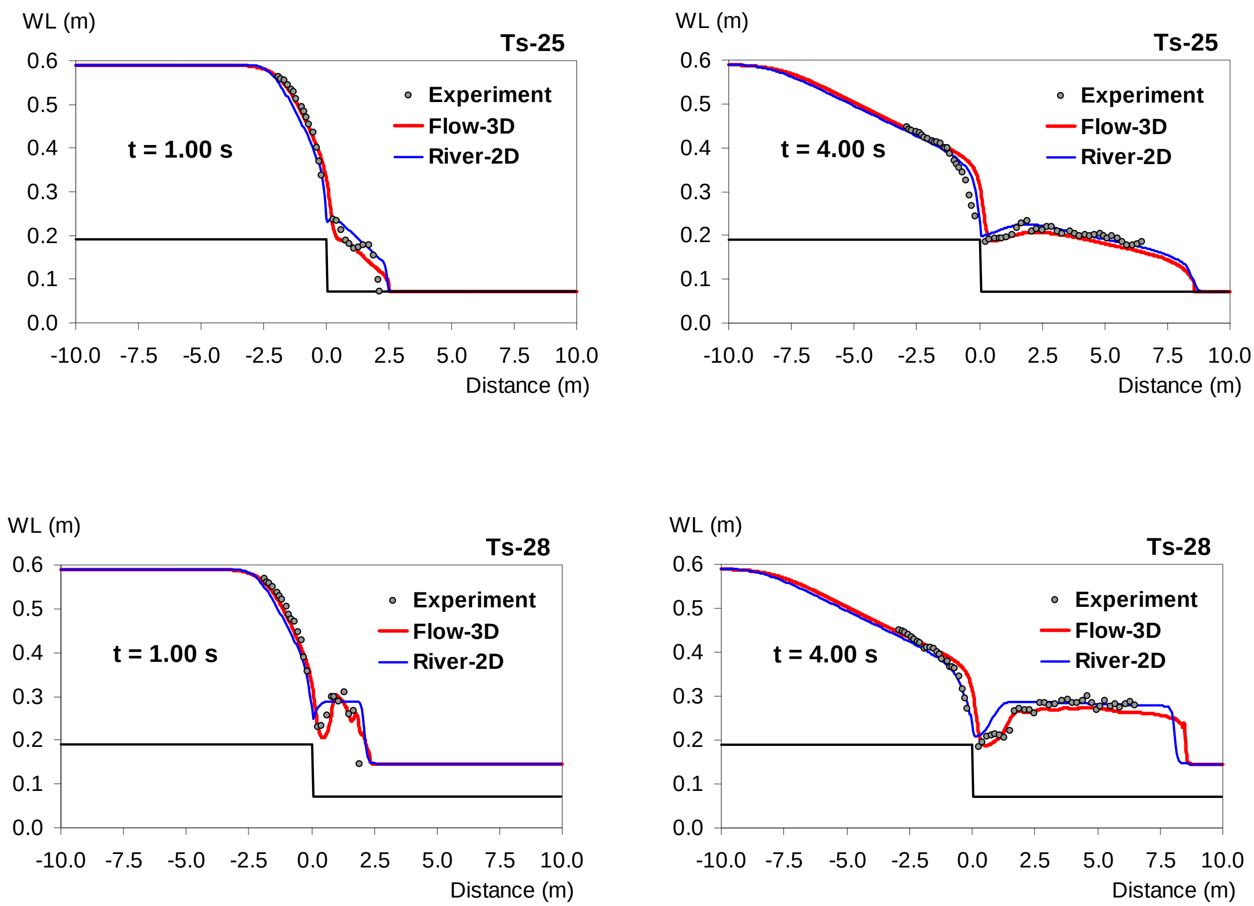

Figure 1: Longitudinal water surface profiles computed by River2D and Flow-3D at 1.0 s and 4.0 s for the straight channel with a bed step. The channel downstream was both initial dry (Ts-25, top) and initially wet (Ts-28, bottom).

4. ISOLATED OBSTACLE

The second case evaluated the presence of an isolated obstacle downstream of the dam. After a dam breaks, the flood wave could hit structures located in its path, such as powerhouses, bridges or buildings, especially if the dam is located near an urban area. Soares-Frazao and Zech (2007) performed experimental flume studies of the interaction between a dam-break flood wave and an isolated obstacle. Details of the experimental set-up can be found in the original publication, and only a brief description is provided here. The channel was 3.6 m wide with a mildly trapezoidal cross sectional shape. The dam – represented by a vertical lift gate – was only 1 m wide at the centre of the channel width. The initial water level in the reservoir was 0.4 m, while it was 0.02 m downstream from the dam. The obstacle was a rectangular block 0.8 m by 0.4 m, skewed 64^{o} relative to the channel centreline and located about 3.5 m downstream from the dam. Six pressure gauges recorded the dynamic water levels during the experiment - five gauges (G1 through G5) were located around the obstacle and the sixth gauge (G6) was located within the reservoir. After the gate was lifted to simulate the sudden dam-break, a very complex flow pattern developed downstream, with recirculation zones, hydraulic jumps and waves reflecting from the solid walls. Snapshots of the water surface computed by Flow-3D during the first five seconds of the experiment –including the initial conditions– are shown in Figure 2. The main features of the River2D and Flow-3D models are shown in Tables 1 and 2.

Figure 3 shows a comparison between the recorded water levels at the five gauges surrounding the obstacle and the water levels computed by both Flow-3D and River2D. Each plot shows also the location of the pressure gauge. The experimental results show how complex the flow pattern is, with strong fluctuations in water levels, with several peaks. The first peak is probably caused by the direct hit of the flood wave, while the following peaks are probably caused by waves reflected from the walls.

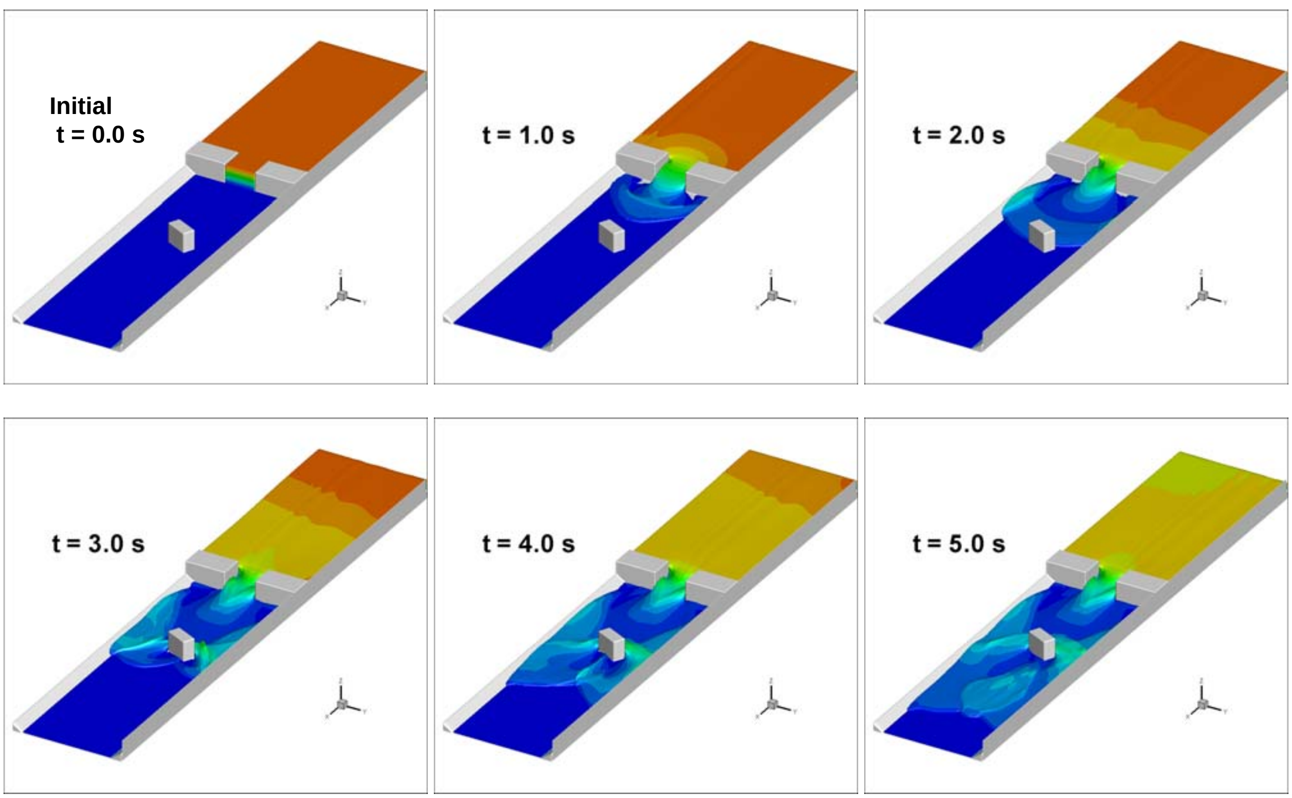

Figure 2: Water surface computed by Flow-3D for the first 5 s of the Isolated Obstacle test case

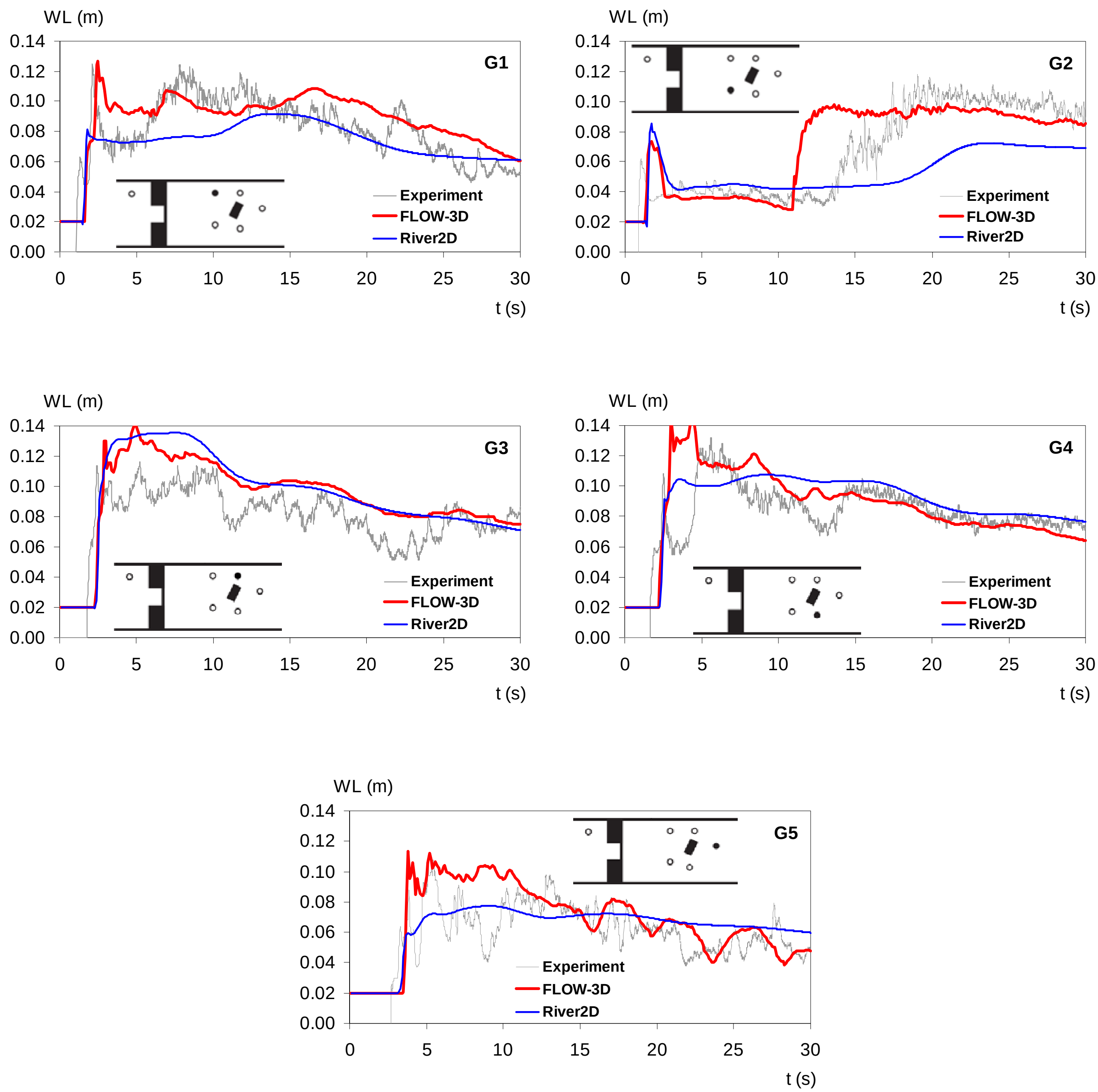

Figure 3: Water levels computed by River2D and Flow-3D at the five pressure gauges (G1 to G5) for the Isolated Obstacle test case.

The isolated obstacle experiment is much more complex than the dam-break in a straight flume described previously and the agreement between the experiment and the numerical models is not as good. By visual inspection, it appears that Flow-3D produces better results than River2D, which should be expected considering that the flood wave hitting the obstacle should produce strong vertical velocities. River2D missed the first water level peaks at gauges G1 and G5, but it did capture the sudden rise at G2. A few sensitivity tests were performed in Flow-3D changing some parameters in the VOF model and increasing the order of the convection computation, but none produced any significant improvement to the results presented in Figure 3.

5. TRIANGULAR BOTTOM SILL

The third test case considered the presence of a raised triangular sill in the channel floor downstream of the dam. Two-dimensional depth-averaged flow models that solve conservation of mass and momentum equations, do not compute the (u,v) velocity components directly; but instead they solve the equations for the unit discharge discharge per unit width) in the two horizontal directions $q_x = uh$ and $q_y = vh$. Velocity is then computed by dividing the unit discharge by the water depth: $u = q_x / h$ and $v = q_y / h$. An unsolved problem with this approach is that when q is high and the water depth decrease rapidly ($h \rightarrow 0$), the velocity tends to overshoot, making the numerical model unstable. This can occur for example, when water flows over a shallow submerged obstacle like a bar or bump and hence the importance of testing dam-break flow models in such cases.

Soares-Frazao (2007) conducted a dam-break experiment in a straight flume which had a triangular bottom sill (bump), located downstream (Figure 4). The initial water level in the reservoir upstream from the dam (gate) was 0.111 m. The symmetric sill was located 1.61 m downstream from the dam; it had a length of 0.90 m and a maximum height of 0.065 m, with upstream and downstream slopes of 0.14. Downstream from the sill there was a pool of water with initial depth of 0.02 m. Three pressure gauges recorded the time evolution of water levels upstream (G3) and downstream (G1 and G2) from the bottom sill. High speed digital cameras were also used to capture snapshots of the instantaneous water surface profile around the sill.

The channel was closed at the both ends. After the gate was lifted, a wave front moved quickly over the initially dry bed upstream from the bottom sill. Around t = 1.5 s the wave reached the sill and started climbing it (Figure 4). A portion of the water flow made it over the sill's crest and plunged into the still pool downstream from the sill generating a strong hydraulic jump that migrated downstream. The remaining of the flow was reflected by the sill and started to migrate upstream towards the dam. After several wave reflections from both ends of the flume, the water depth started to asymptotically stabilize around 0.06 m.

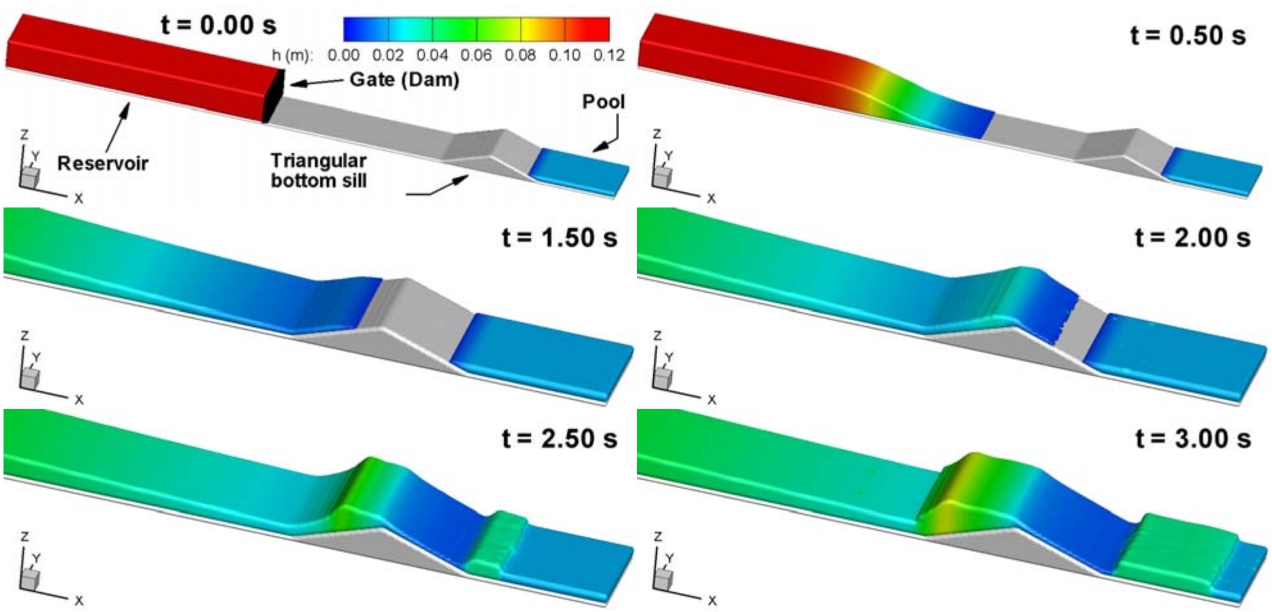

Figure 4: Initial conditions and water levels computed by Flow-3D
for the triangular bottom sill test case

Figure 5 shows a comparison of water levels measured at the three gauges and the results computed by Flow-3D and River2D up to 45 s after dam-break; while Figure 6 shows snapshots of the water surface profiles at t = 1.8 s and t = 3.0 s, both measured and computed. As before, the details of the meshes for both model are shown in Tables 1 and 2.

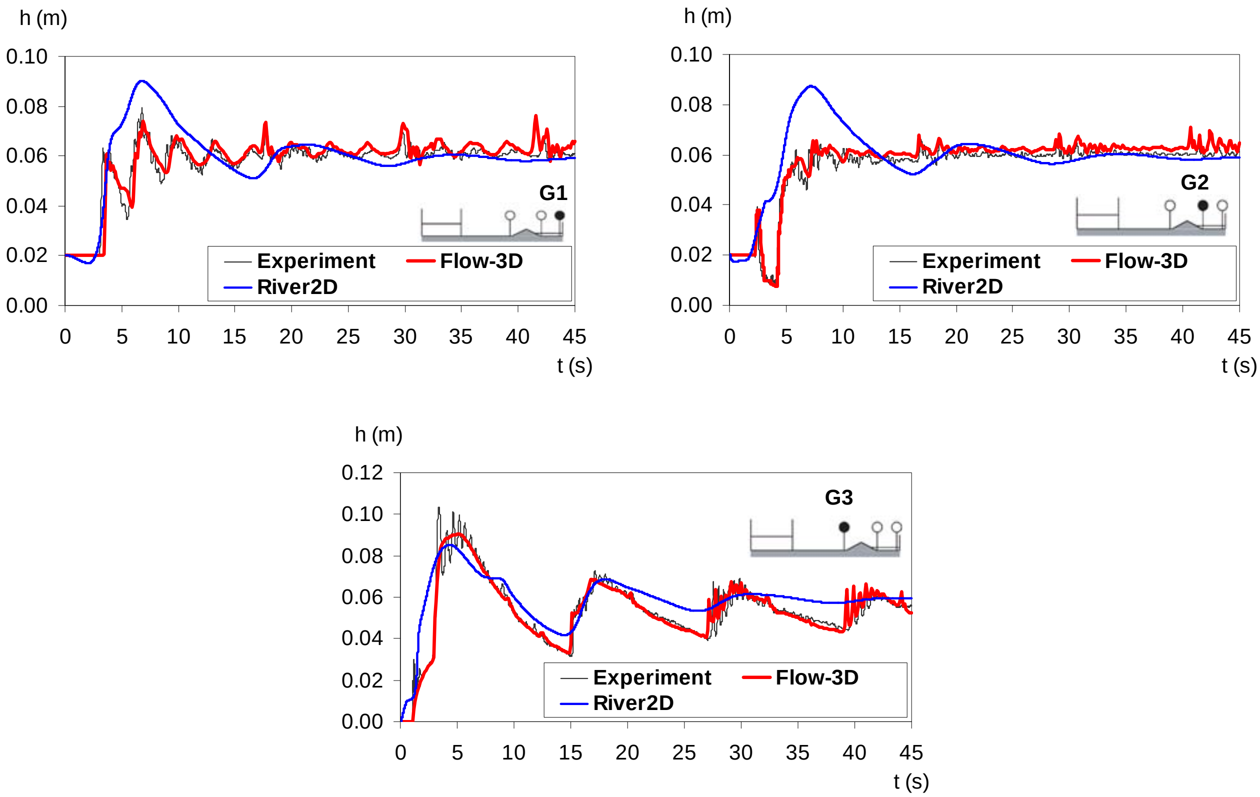

Figure 5: Comparison of water levels computed by River2D and Flow-3D
with experimental data at three pressure taps for the triangular bottom sill test case.

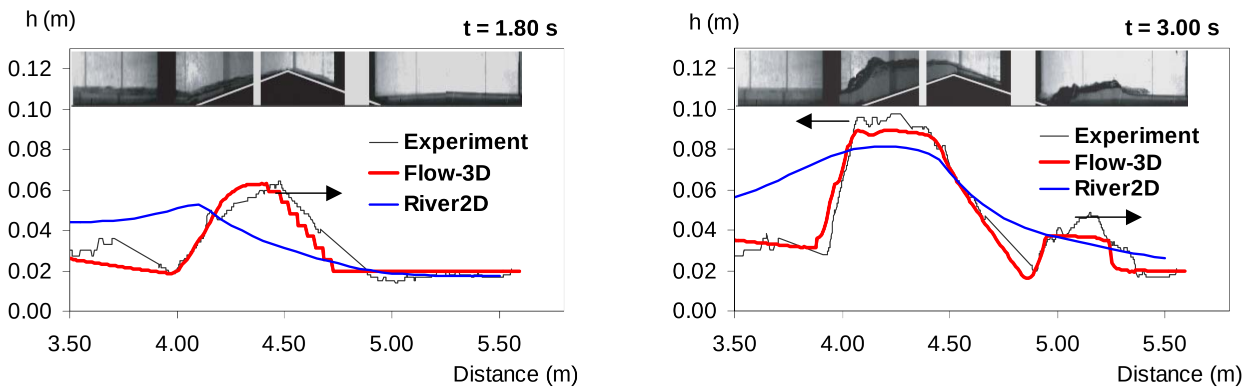

Figure 6: Comparison of longitudinal water level profiles computed by River2D and Flow-3D
at 1.8 s and 3.0 s around the triangular bottom sill. Notice migrating hydraulic jump at $t = 3.0$ s.

Flow-3D produced an excellent agreement with the experimental data. The 3D model was able to accurately capture the water level fluctuations at the three gauges (Figure 5). The sudden drop of water depth at gauge G2, caused by the formation of a hydraulic jump is well reproduced. This is also visible in the longitudinal profile at $t = 3.0$ s (Figure 6). The discrepancies between Flow-3D and the experiment shown in Figure 5 are in the order of the experimental error (see Figures 5 and 10 in Soares-Frazao 2007) and it can be claimed that for this particular experiment, Flow-3D produced results with the same level of accuracy as the physical model.

However, the results computed by River2D were not as good. Although the model captured some of the oscillations measured at Gauge G3 (Figure 5), it grossly over-predicted the maximum water levels downstream from the sill (G1 and G2). River2D missed completely the hydraulic jump formed right downstream from the sill, as shown in the profile at $t = 3.0$ s (Figure 6). These results are further discussed in the next section.

6. LIMITATIONS OF RIVER2D FOR DAM-BREAK FLOWS

River2D's computed water levels for the triangular bottom sill experiment (Figures 5 and 6) were surprising and rather unexpected. River2D did not exhibit any stability problem or velocity overshoot; however, the agreement with the experimental data was not satisfactory. The first test in the straight flume with a bed step described in Section 3 demonstrated that River2D is capable of modelling hydraulic jumps; therefore the surface flow model cannot be responsible for the failure to model the hydraulic jump in the triangular bottom sill experiment (Figure 6), the problem lies in the groundwater flow model.

As explained in Section 2.1, River2D assumes that the bottom sill is pervious and part of an aquifer. As the flood wave tries to climb over the upstream face of the sill, a strong water slope S develops across the sill, as illustrated in Figure 7 for t = 1.8 s. For that particular time, the slope was S = 0.034, much larger than the typical values in normal river flows, which are in the order of 10^{-3} to 10^{-5}. Since the discharge into the aquifer is proportional to S (equation 1), and in this case the slope is very high, all the incoming flow is forced into the aquifer (sill) with no water left to move over the sill crest. In other words, the entire flood wave passes through the sill, instead of over its crest.

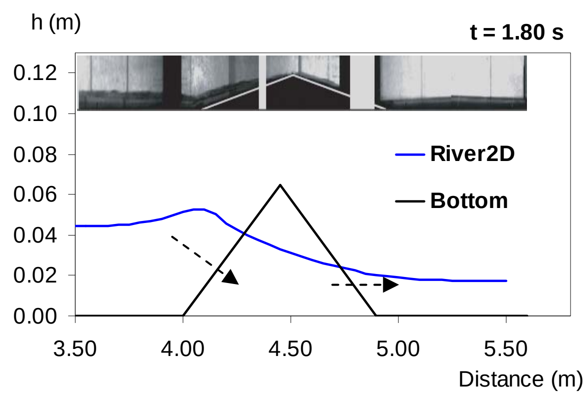

Figure 7: Water levels computed by River2D at 1.8 s show groundwater flow (arrows) through the triangular bottom sill.

As water flows through the sill, it feeds the pool downstream rapidly, but gradually, generating a higher than observed water level at gauges G1 and G2 (Figure 5), but without a hydraulic jump. It is not possible to eliminate or reduce substantially the groundwater flow; attempts to reduce the value of transmissivity T (equation 1) led to numerical instabilities. It is likely that some surface flow also went underground into the inclined trapezoidal faces of the isolated obstacle flume (Figure 2). If that were the case, it would help explain some of the deviations between River2D and the experimental data (Figure 3).

These rather unexpected results of River2D illustrate the need for testing and validating numerical models, previous to their application in practical problems. This is especially important when the application is something for which the model has not been used before or it has only been used under simplified conditions.

7. POTENTIAL APPLICATION OF FLOW-3D FOR REAL DAM-BREAK FLOWS

Compared with a 2D model, the main limitation of any 3D model for practical applications is the significant increase in the mesh size caused by including the vertical dimension (compare Tables 1 and 2), which translates in longer computational times. For the isolated obstacle test case, which required the largest of the Flow-3D mesh tested (Table 2), Flow-3D version 9.3 required 9.5 hours to run in an 8-core computer (with Xeon E5430 processors running at 2.66 GHz under Windows XP). Although computational times for the three laboratory experiments simulated were manageable, their geometries were rather simple and not as complex as expected in a real dam break case. For that reason, a qualitative simulation was performed using real topographic data to test how Flow-3D would perform under a more realistic scenario.

The computational domain was L_x = 540 m long, L_y = 420 m wide and L_z = 70 m high and included a narrow valley were a concrete dam was assumed to be initially in place. A powerhouse building, immediately downstream from the dam, was also included in the model (Figure 8). The computational grid was made of 5'439,000 cells. Assuming that the dam vanished instantaneously at t = 0 s, Flow-3D was applied to simulate the flow conditions during the first 100 s after that event. The water surface computed at t = 10 s is shown in Figure 8. The total computational time was about 7 hours.

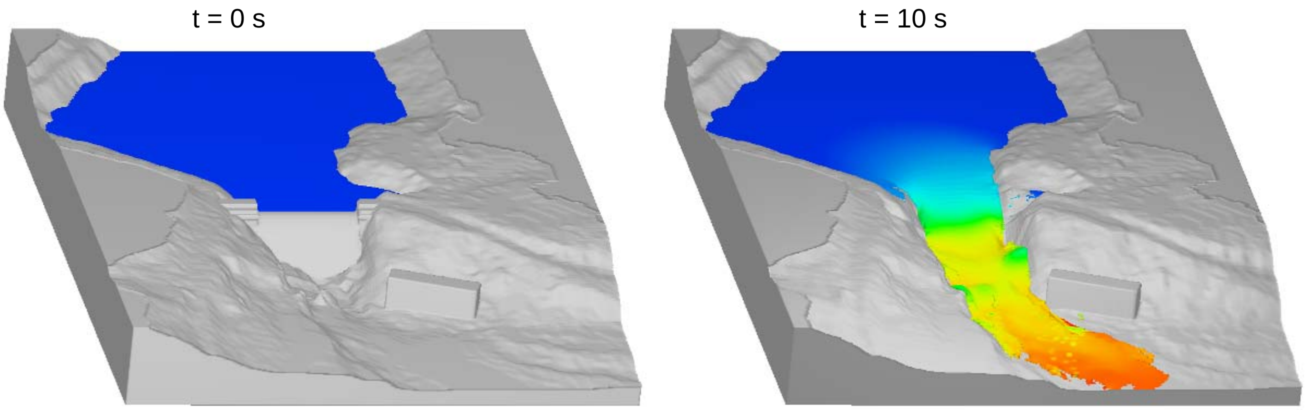

Figure 8: Qualitative simulations of a hypothetical instantaneous dam-break using Flow-3D.

Although this was just a qualitative simulation, it has proved for the first time that Flow-3D could be used to model dam break flows under realistic conditions within reasonable computational effort. These encouraging results pave the way for future practical engineering applications, where the details of the dam-break flow in the vicinity of the dam are sought. However, it seems unlikely that Flow-3D could be used to route the flood wave a long distance away from the dam, because that would demand a very large computational time in a present-day computer; a 2D model is probably more suitable for such a task.

8. SUMMARY AND CONCLUSIONS

The 2D depth-averaged flow model River2D always assumes that water moves over an alluvial pervious bed with a defined groundwater transmissivity T. This assumption allows some surface water to flow into an assumed underlying aquifer when the water depth drops below a minimum low value, in such a way that wetting and drying processes under normal river flow conditions can be efficiently modelled. The amount of aquifer flow is governed by the product of T and the water surface slope S in the aquifer. If a dam-break flood wave moves over an initially dry bed which is flat, S will also remain flat (S = 0) and the aquifer flow will be negligible. However, a flood wave moving over a non-flat bed will generate high values of S, forcing a large amount of surface flow into the aquifer. In cases when the bed slope intersects the flood wave, (e.g. up-sloping bed in the downstream direction), up to a 100% of the incoming flood wave can disappear into the groundwater aquifer. Since it is not possible to set T = 0, or even T ≈ 0, without making River2D unstable, this model should not be applied to simulate sudden dam-break flows over natural bed topography until this problem is fixed. This however, does not mean that River2D should not be used for normal river flows; experience using River2D for those types of applications has shown that the model is quite accurate and very reliable. Sudden dam-breaks are a special case River2D was not intended for.

Flow-3D was able to achieve a very good agreement with the experimental data in the three experimental test cases analyzed. One important factor in the success of Flow-3D was its robust VOF solver used for tracking the location of the free surface. The computational effort for all the test cases analyzed (three experiments plus hypothetical dam) was reasonable, adding to that Flow-3D's ability to accommodate the complex geometries typically found at dam sites (Figure 8), suggest that Flow-3D can be a valuable tool for real dam-break flow modelling. Flow-3D seems perfectly suitable for analyzing the details of the dam break flows near the dam; for example to investigate different failure scenarios, for calculating the discharge hydrograph caused by the dam failure and especially for studying the interaction between the flood wave and nearby structures (e.g. powerhouse, bridge). However, flood routing far away from the dam would be too computationally demanding and probably not practical at the moment.

9. ACKNOWLEDGEMENTS

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