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**A NEW ONE-DIMENSIONAL NUMERICAL MODEL FOR THE
HYDRAULICS OF SEDIMENTS IN RIVERS**

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**A NEW ONE-DIMENSIONAL NUMERICAL MODEL FOR THE
HYDRAULICS OF SEDIMENTS IN RIVERS**

présentée par **Iman Hamza AlQasimi**

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DEDICATION

To my greatest blessing, my children.

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RÉSUMÉ

Un modèle numérique unidimensionnel est développé pour simuler l'évolution morphodynamique des rivières. Ce modèle, UMHYSER-1D (Unsteady Model for the HYdraulics of SEdiments in Rivers 1D), est un modèle hydromorphodynamique unidimensionnel capable de représenter les profils de la surface d'eau dans une rivière ou un réseau de chenaux, avec différents régimes d'écoulement en prenant compte du transport de sédiments, cohésifs ou non cohésifs. UMHYSER-1D simule des écoulement permanent ou non, avec ou sans transport de sédiments. Il a été développé en plusieurs étapes.

Dans la première étape, une bonne partie du code du modèle quasi-permanent MHYSER (Modèle de l'HYdraulique des SÉdiments en Rivières) a été adopté. Il s'agit de la lecture des données d'entrée, comme la lecture des section transversales et leurs caractéristiques hydrauliques, et le module de résolution des équations de continuité et d'énergie, pour la ligne d'eau, et d'Exner, pour le transport solide, adoptée pour des simulations à long terme. En effet, dans ce cas, les termes non permanents de l'équation de continuité du transport des sédiments sont ignorés. De plus, une méthode de transport des sédiments hors équilibre est utilisée. Cette première étape quoique facile, est très importante puisqu'elle permet de valider surtout la phase d'écoulement permanent et le pré-traitement de UMHYSER-1D. Sa validation a été faite en comparant les solutions avec des cas de la littérature.

La deuxième étape consiste à résoudre les équations de la phase liquide en régime non permanent. En effet, l'approche découplée a été adoptée. Pour les écoulements non permanents, les équations de Saint Venant sont résolues en utilisant le schéma numérique NewC. Ce choix déroge au choix classique, schéma de Preissmann, adopté par la plupart des modèles disponibles. Ensuite, l'ajout

de structures hydrauliques et de conditions aux limites internes (courbes de tarages, déversoirs, ponts et vannes) a été fait. La validation à ce stade est surtout numérique, par comparaison des résultats à ceux du modèle SRH-1D qui utilise le même schéma numérique.

L'étape suivante est la résolution de l'équation de transport des sédiments. Il s'agit d'une équation de convection-diffusion, avec un terme source résultant de l'érosion/dépôt de sédiments, adaptée d'une formulation originale d'un modèle bidimensionnel, SRH-2D. La résolution numérique de cette équation de convection-diffusion 1D, valable surtout pour des simulations à court terme, fait appel à la méthode des pas fractionnaires. D'abord, l'équation sans le terme source est résolue avec une méthode volumes finis implicite, puis l'équation avec le terme source est résolue pour obtenir la concentration au pas de temps suivant. Des équations de transport de sédiments non cohésifs et des processus physiques de sédiments cohésifs sont implémentés pour calculer le dépôt/érosion des sédiments en utilisant l'équation de conservation de masse de la phase solide. Pour la validation de cette étape, un recours au modèle SRH-1D a été fait.

La quatrième étape du développement de UMHYSER-1D consiste en la prise en compte de la simulation du pavage des rivières et de leurs ajustements horizontal/latéral. Le pavage est pris en compte en utilisant le concept de la couche active, qui permet une érosion sélective. De plus, UMHYSER-1D tient compte empiriquement des ajustements de la géométrie du lit en utilisant une relation entre la largeur d'érosion et le débit, un angle de repos pour la stabilité des berges et trois théories de minimisation. En validant cette étape utilisant une expérience de laboratoire, UMHYSER-1D montre des capacités prometteuses quant à son utilisation à petite échelle, celle du laboratoire.

UMHYSER-1D se veut un outil destiné à l'ingénieur hydraulicien. En effet, son application, dans cette thèse, au cas extrême du déluge du Saguenay 1996, a donné des résultats présentant une anomalie qui ne pouvait pas être expliquée physiquement. En effet, les différences entre les résultats des simulations et les observations relevées sur le site présentent des différences acceptables sauf sur quatre kilomètres où la différence atteignait vingt mètres. Ceci a forcé la validation des données de la rivière Ha ! Ha ! pour découvrir que les sections transversales publiées depuis 2007 ne sont pas toutes bonnes, pire encore, l'une des sections après déluge n'est pas située dans le lit de la rivière initiale mais plutôt est située dans le nouveau chenal créé après que la rivière a débordé. De plus, l'application de UMHYSER-1D à l'évaluation de l'impact des sédiments en suspension, sur la rivière Aux Sables, montre bien que ce modèle peut bien répondre aux besoins des ingénieurs praticiens.

Enfin, UMHYSER-1D se veut aussi un laboratoire numérique destiné aux chercheurs en hydraulique fluviale. Par exemple, tester une nouvelle équation de transport solide pour un tronçon de rivière de plusieurs kilomètres de longueur, tester les limites d'un nouveau schéma numérique ou comparer la performance de deux schémas de discrétisation sont des exemples d'utilisation de ce modèle. La structure du code de UMHYSER-1D, adoptant les concepts de sous-programmes, modules, "subroutines" et "functions", permet de tester d'autres résultats de recherches théorique ou expérimentale en les incorporant dans le code du modèle.

ABSTRACT

A one-dimensional numerical model is developed to simulate the evolution of rivers. This model, UMHYSER-1D (Unsteady Model for the HYdraulics of SEdiments in Rivers 1D), is a one-dimensional hydro-morpho-dynamic model capable of representing the surface profiles of water in single river or a multiriver network, with different flow regimes taking into account cohesive or non-cohesive sediment transport. UMHYSER-1D simulates steady and unsteady flows, with or without sediment transport. It was developed in several steps.

In the first step, a good part of the code for the quasi-permanent model MHYSER (Model of Hydraulics of SEdiments in Rivers) was adopted. These are the reading of the input data, such as the cross sections and their hydraulic characteristics, and the module solving the equations of mass and energy conservation, for the water surface, and Exner, for the sediment transport, adopted for long-term simulations. Indeed, in this case, the non-permanent terms of the sediment transport continuity equation are ignored. In addition, a non-equilibrium sediment transport method is used. This first step, although easy, is very important since it mainly validates the permanent flow phase and the pre-treatment of UMHYSER-1D. Its validation was made by comparing the solutions with cases from the literature.

The second step consists in solving the equations of the liquid phase for unsteady state flow. Indeed, the decoupled approach has been adopted. For unsteady flows, the de Saint Venant equations are solved using the NewC numerical scheme. This choice deviates from the classic one, Preissmann scheme, adopted by most of the available models. Then, the addition of hydraulic structures and internal boundary conditions (rating curves, weirs, bridges and gates) was made. The validation at this stage is mainly numerical, by comparison of the results with those of the SRH-1D model which uses the same numerical scheme.

The next step is to solve the sediment transport equation. It is a convection-diffusion equation, with a source term resulting from sediment erosion/deposition, adapted from an original formulation of a two-dimensional model, SRH-2D. The numerical resolution of this 1D convection-diffusion equation, valid especially for short-term simulations, uses the fractional step method. First, the equation without the source term is solved with an implicit finite volume method, then the equation with the source term is solved to obtain the concentration at the next time step. Non-cohesive sediment transport equations and cohesive sediment physical processes are implemented to calculate sediment deposition/erosion using the solid phase mass conservation equation. To validate this step, use was made of the SRH-1D model.

The fourth step in the development of UMHYSER-1D consists in considering the simulation of armoring of rivers and their lateral/vertical adjustments. The armoring is modeled using the concept of the active layer, which allows selective erosion. In addition, UMHYSER-1D empirically takes into account the adjustments of the bed geometry using a relationship between the erosion width and the flow rate, a repose angle for bank stability and three minimization theories. By validating this step using laboratory experiment, UMHYSER-1D shows promising capabilities for its use on a small scale, that of the laboratory.

UMHYSER-1D is intended as a tool for the hydraulic engineer. Indeed, its application, in this thesis, to the extreme case of the Saguenay flood 1996, made it possible, in view of the results, to force data validation of the Ha! Ha! river to discover that the cross sections published since 2007 are not all good, even worse, one of the post-flood cross-sections not located in the initial river but rather is located in the new channel created after the river overflowed. In addition, the application of UMHYSER-1D to address suspended sediment impact assessment, for the Aux Sables river, shows that this model can well meet the needs of practicing engineers.

Finally, UMHYSER-1D is also a numerical laboratory for river hydraulics researchers. For example, testing a new solid transport equation for a river reach of several kilometers, testing the limits of a new numerical scheme or comparing the performance of two discretization schemes are examples of using this model. The code structure of UMHYSER-1D, adopting the concepts of sub-programs, modules, "subroutines" and "functions", makes it possible to test other results of theoretical or experimental research by incorporating them into the model's code.

CONDENSÉ EN FRANÇAIS

Dans cette thèse un modèle numérique unidimensionnel est développé pour simuler l'évolution morphologique des rivières. Ce modèle hydromorphodynamique unidimensionnel, UMHYSER-1D (Unsteady Model for the HYdraulics of SEdiments in Rivers 1D), résout les équations unidimensionnelles de conservation de la masse et de la quantité de mouvement, équations de Saint Venant pour calculer les lignes d'eau d'une rivière ou un réseau de chenaux. En présence de sédiment, cohésif ou non, l'équation de transport est prise en compte aussi. Si l'écoulement est permanent, UMHYSER-1D traite la résolution des équations de continuité et d'énergie, pour la ligne d'eau, et d'Exner, pour le transport solide. La validation est faite par comparaisons des résultats à ceux de cas classiques.

Pour les écoulements non permanents, l'approche découplée a été adoptée. D'abord la phase liquide est traitée puis la phase solide. Ce découplage sous-entend qu'une attention particulière doit être apportée au choix du pas de temps en plus des critères de stabilité des schémas numériques utilisés. Pour la phase liquide, les équations de Saint Venant sont résolues en utilisant le schéma numérique NewC qui est un schéma aux différences finies capable de modéliser les conditions d'écoulement sub-, super- et trans-critique sans nécessiter des modifications des équations de Saint Venant. L'analyse de stabilité linéarisée utilisant les séries de Fourier montre que le schéma numérique est stable pour une gamme de nombres de Courant et même pour des nombres de Froude supérieurs ou égaux à un. Cependant, dans cette thèse, il a été remarqué que, pour des nombres de Froude supérieurs à 1,5, des oscillations numériques apparaissent et pourraient s'amplifier pour donner des solutions instables. En guise de remède, deux options sont proposées: utiliser l'inertie partielle locale (LPI, en anglais) pour affaiblir le terme d'inertie de l'équation de quantité de mouvement, ou introduire une certaine diffusion numérique pour lisser la solution. De plus, par sa

construction, le schéma NewC a une limitation dans la résolution de l'écoulement torrentiel. En effet, comme il nécessite une condition aux limites à chaque extrémité du domaine, un écoulement supercritique ne doit pas se produire aux limites amont ou aval de tout tronçon de rivière modélisé. Par ailleurs, l'effort de calcul par NewC se compare favorablement aux autres schémas aux différences finies disponibles et NewC est facilement incorporable dans les algorithmes de résolution d'écoulements dans un réseau de rivières. Après l'ajout de structures hydrauliques et de conditions aux limites internes (courbes de tarages, déversoirs, ponts et vannes), la validation numérique par comparaison des résultats à ceux du modèle SRH-1D, utilisant le même schéma numérique, est faite.

Pour modéliser le transport de sédiments, UMHYSER-1D, utilise une équation, de convection-diffusion avec un terme source résultant de l'érosion/dépôt de sédiments, qui permet de basculer automatiquement entre les différents modes de transport. Ainsi le transport par charriage, par suspension ou un transport mixte peut être calculé en fonction d'un paramètre de mode de transport tenant compte de l'hydraulique locale de l'écoulement. En effet, le plus grand rapport, vitesse du sédiment (en charriage ou en suspension) à la vitesse de l'écoulement, impose le mode de transport des sédiments, permettant ainsi le passage d'un mode de transport à l'autre dans le même tronçon dépendamment des caractéristiques hydrauliques locales de l'écoulements. De plus, cette équation peut être appliquée à des fractions de tailles multiples et est généralisée au transport des sédiments cohésifs.

La résolution numérique de cette équation de convection-diffusion 1D fait appel à la méthode des pas fractionnaires. D'abord, l'équation sans le terme source est résolue avec une méthode volumes finis implicite. Le terme convectif est discrétisé en utilisant la méthode de Lax-Wendroff TVD, et le terme diffusif est approximé par le schéma aux différence central pour avoir une discrétisation

du second ordre. Par la suite, l'équation avec le terme source est résolue pour obtenir la concentration au pas de temps suivant. Cette résolution peut se faire soit analytiquement ou en utilisant la méthode numérique de Runge-Kutta d'ordre 4. La mise à jour du lit de la rivière, à la suite du dépôt/érosion, se fait en utilisant l'équation de conservation de masse de la phase solide en faisant appel à l'une des 13 équations de transport de sédiments non cohésifs et des processus physiques de sédiments cohésifs implémentés dans UMHYSER-1D. La validation de cette étape est jugée très satisfaisante après comparaison des résultats de UMHYSER-1D à ceux du modèle SRH-1D.

Ce code a été étendu en à la simulation du pavage des rivières. Le pavage est pris en compte en utilisant le concept de la couche active, qui permet une érosion sélective. UMHYSER-1D calcule le transport des sédiments par fraction selon les tailles des sédiments. Ainsi, des particules de différents diamètres sont transportées à des taux différents. Dépendamment des paramètres hydrauliques, de la distribution des sédiments entrants dans une section transversale et ceux composant le lit, certaines particules peuvent être érodées, d'autres déposées et certaines rester immobiles. La méthode utilisée, celle de Bennett et Nordin, utilise trois couches du lit pour le dépôt et deux couches pour l'érosion. Sous la couche active, la couche supérieure qui contient le matériau du lit disponible pour le transport, se trouve la couche inactive, utilisée pour le stockage. Le lit intact, avec la composition initiale du matériau, se trouve en dessous de ces deux couches.

La couche active est le concept le plus important dans cette procédure. Elle contient tous les sédiments disponibles pour le transport à chaque pas de temps. Dans UMHYSER-1D, L'épaisseur de la couche active est définie comme étant proportionnelle à la moyenne géométrique de la plus grande classe granulométrique contenant au moins 1% du matériau du lit à cet endroit. L'érosion

d'une classe particulière du matériau de lit est limitée par la quantité de sédiments, de cette classe, présente dans la couche active. Si la capacité de transport pour une classe donnée est supérieure à ce qui est disponible pour le transport dans la couche active, l'expression "disponibilité limitée" est utilisée. Cependant, si plus de matériel est disponible que celui nécessaire pour répondre à la capacité de transport calculée par une équation de transport solide, l'expression "capacité limitée" est employée. En cas de dépôt net, La couche inactive est utilisée. L'épaisseur de dépôt de chaque classe, de la courbe granulométrique, est ajoutée à la couche inactive, qui à son tour est ajoutée à l'épaisseur de la couche active. La composition granulométrique et l'épaisseur de la couche inactive sont calculées en premier, après quoi une nouvelle couche active est recalculée et l'élévation du lit mise à jour. Ainsi, puisque les matériaux disponibles pour le transport sont ceux exposés à la surface du lit, le concept de couche active est fondamental. En effet, lors des simulations numériques, pour chaque pas de temps, le seul matériau disponible pour l'érosion est celui de la couche active. De ce fait, l'épaisseur de la couche active doit toujours être au moins aussi épaisse que la profondeur maximale d'affouillement prévue. Pour ce faire, il est recommandé de définir l'épaisseur la couche active d'abord, puis bien choisir le pas de temps pour calculer le transport des sédiments. Un processus d'essai-erreur peut être utilisé pour garantir cela.

La dernière étape dans le développement de UMHYSER-1D est l'ajustement des rivières après chaque épisode d'érosion/dépôt. Pour ce faire trois options y sont incorporées en utilisant une relation entre la largeur d'érosion et le débit, un angle de repos pour la stabilité des berges et trois théories de minimisation.

Pour la première option, largeur d'érosion, UMHYSER-1D étant un modèle 1D, il ne simule pas directement le transport latéral des sédiments ; cependant, il tient compte empiriquement des

processus impliqués en utilisant une relation entre la largeur d'érosion et le débit. En effet, pour les modèles 1D, puisque la contrainte de cisaillement ne varie pas transversalement dans une section d'écoulement, il est difficile d'estimer l'érosion non uniforme qui se produit en cas d'incision du lit de la rivière.

Pour le recul des berges, la deuxième option est basée sur l'angle de repos. Pendant la progression de l'érosion, la pente de la rive est limitée par les valeurs au-dessus et sous l'eau de l'angle de repos. À la fin de chaque pas de temps, si des ajustements verticaux ou horizontaux ont rendu les berges trop raides, les deux points adjacents aux segments des berges sont ajustés verticalement jusqu'à ce que les pentes soient égales aux pentes critiques. Le matériel prélevé sur les berges sera ajouté au cours du prochain pas de temps sous forme de débit solide latéral.

La dernière option offre le choix parmi trois théories de minimisation pour l'ajustement de la profondeur ou la largeur des rivières : minimisation de la puissance totale de la rivière, minimisation de la pente d'énergie et la maximisation de la débitance (ou minimisation de la pente du lit). Selon la théorie de minimisation de la puissance totale, si une puissance totale minimale résulte d'une modification des largeurs de la rivière, alors des ajustements de la section transversale se font dans la direction latérale. Sinon, les ajustements ont lieu verticalement. Pour la minimisation de la pente énergétique (pente du fond), si celle-ci, à une section transversale donnée, est supérieure à la pente énergétique (pente du fond) moyenne pondérée de ses deux sections adjacentes, la largeur du canal à cette section est réduite pendant le dépôt ou la profondeur est augmentée pendant l'érosion. Cependant, si la pente d'énergie (pente du fond) est plus petite, la profondeur du canal à cette section est diminuée pendant le dépôt ou une augmentation de largeur se produit pendant l'érosion.

Avec tous ces développements, UMHYSER-1D est un laboratoire numérique prêt à être utilisé. Tester des options incorporées dans UMHYSER-1D, comme le concept de tubes de courants, ou comparer d'autres, comme deux théories de minimisation, ou incorporer d'autres nouveautés, comme une nouvelle équation de transport solide, sont des exemples d'exploitation de ce nouveau modèle qui lui promettent une longue vie productive. Dans cette thèse, au lieu de pencher sur ces applications, le choix a été plutôt d'appliquer UMHYSER-1D à trois situations : un cas de laboratoire, au modifications morphologiques de la rivière Ha ! Ha ! suite au déluge du Saguenay 1996 et au cas de restauration de la rivière Aux Sables après ce même déluge. Le premier cas est un défi car il traite de l'érosion à petite échelle spatiale (canal de laboratoire), alors que le second cas est un défi car durant inondation de 1996, les rivières ont charrié des troncs d'arbres sur plusieurs kilomètres. L'intérêt de l'utilisation de UMHYSER-1D à ces deux cas est de voir jusqu'à quel point UMHYSER-1D peut être utilisé pour des cas extrêmes. Le dernier cas représente un exemple d'application de UMHYSER-1D pour de l'ingénieur hydraulicien.

Pour le premier cas, des expériences à l'Université du Minnesota ont été effectuées pour simuler le démantèlement de barrage. Un canal d'une largeur de 0,61 m et une pente de 0,018 a été rempli de sédiments pour reproduire le dépôt de sédiments en amont d'un barrage. Il est à noter que le dépôt, au début de la simulation utilisant UMHYSER-1D, présente la forme typique d'un delta dans les réservoirs, avec deux régions de pentes différentes séparées par une zone à forte pente. Dans cette zone, les équations de Saint Venant ne sont pas valides car alors l'hypothèse de pression hydrostatique est clairement non respectée. N'empêche, les résultats de UMHYSER-1D, bien qu'ils ne concordent pas totalement avec les profils observés, sont très satisfaisant sauf peut-être au niveau de la zone à forte pente. Avec l'utilisation des théories de minimisation, seule la minimisation de la puissance totale du cours d'eau a amélioré les résultats de simulation. Avec

cette expérience de laboratoire, UMHYSER-1D montre des capacités prometteuses quant à son utilisation à petite échelle, celle du laboratoire.

L'application de UMHYSER-1D, dans cette thèse, au cas extrême du déluge du Saguenay 1996, donne des résultats présentant une anomalie qui ne peut pas être expliquée physiquement. En effet, les différences entre les résultats des simulations et les observations relevées sur le site présentent des différences acceptables sauf sur quatre kilomètres où la différence atteint vingt mètres. Ceci a forcé la validation des données de la rivière Ha ! Ha ! pour découvrir que les sections transversales publiées depuis 2007 ne sont pas toutes bonnes, pire encore, l'une des sections après déluge n'est pas située dans le lit de la rivière initiale mais plutôt est située dans le nouveau chenal créé après que la rivière a débordé. Cette application montre que les processus physiques pris en compte par UMHYSER-1D permettent de capturer les tendances d'érosion/dépôt même pour des cas d'érosion extrême.

Pour le dernier cas d'application, UMHYSER-1D est utilisé pour aider l'ingénieur à minimiser le risque de pollution en prenant une décision quant à la vitesse d'excavation dans la rivière Aux Sables lors de son creusement. En effet, à la suite du déluge du Saguenay 1996, la solution retenue pour minimiser le risque d'inondation était de creuser la rivière pour augmenter sa débitance. Cette solution soulève un autre problème puisque ce creusage libère des sédiments en suspension qui peuvent menacer la qualité des eaux au niveau de la prise d'eau de la ville de Jonquière situé à une dizaine de kilomètres du site des travaux. Ainsi, après calibration du modèle (coefficients de Manning et de diffusion pour les sédiments) UMHYSER-1D a permis d'établir une relation entre le débit solide à relâcher en amont en fonction du débit liquide de la rivière pour que la concentration au niveau de la prise d'eau ne dépasse pas la valeur tolérée de 19 mg/l. Cette

application montre que UMHYSER-1D ce modèle peut bien répondre aux besoins des ingénieurs hydrauliciens.

UMHYSER-1D est un modèle complexe qui peut résoudre une multitude de problèmes. Par exemple, il peut être utilisé pour tester une nouvelle équation de transport solide pour un tronçon de rivière de plusieurs kilomètres de longueur, tester les limites d'un nouveau schéma numérique ou comparer la performance de deux schémas de discrétisation. Aussi, il peut être utilisé dans les application réelles d'ingénierie fluviale pour des écoulements 1D avec ou sans transport de sédiment. L'utilisation d'une programmation orientée objet rend les extensions/améliorations du code faciles ce qui permet de tester d'autres découvertes théoriques ou expérimentales.

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LIST OF SYMBOLS AND ABBREVIATIONS

Uppercase variables

$1D$	One-dimensional
$2D$	Two-dimensional
$3D$	Three-dimensional
A	flow area
A_b	cross-sectional area of the mixing layer
$A_{\delta k}$	Cross-sectional area of the bedload zone, for the k -th sediment size class
B	Top width (or storage width)
C	Cross-sectional sediment concentration by volume (Chapter 4)
C_T	Cross-sectional sediment concentration by volume (Chapter 2)
$\hat{C}$	Equilibrium volumetric sediment concentration carrying capacity of the flow
$\hat{C}_t$	Total transport capacity (potential)
$\hat{C}_k$	Transport capacity of k -th sediment size class if it is the only available class
$\hat{C}_{tk}$	Concentration capacity by volume for k -th size class, $\hat{C}_{tk} = p_k \hat{C}_k$ (Chapter 2)
C_k	Cross-sectional sediment concentration by volume for the k -th sediment size class if it is the only available class
C_{T_k}	Cross-sectional sediment concentration by volume, for the k -th sediment size class
C^*	Cross-sectional sediment concentration capacity by volume
C_{*k}	Concentration capacity by volume for k -th size class, $C_{*k} = p_k C_k^*$ (Chapter 4)
C_k^*	Transport capacity by volume for k -th sediment size class (Chapter 4)
C_c	Cross-sectional cohesive sediment concentration by volume
C_{eq}	Equilibrium cohesive cross-section sediment concentration by volume

$C_{\delta k}$	Cross-sectional bedload concentration by volume, for the k -th sediment size class
CFL	Courant–Friedrichs–Lewy (stability condition)
C_1, ω_1	Point on the curve cohesive fall velocity vs concentration
C_2, ω_2	Point on the curve cohesive fall velocity vs concentration
C_3, ω_3	Point on the curve cohesive fall velocity vs concentration
C_4, ω_4	Point on the curve cohesive fall velocity vs concentration
D	Dispersion coefficient in the longitudinal direction (Chapter 4)
D_l	Dispersion coefficient in the longitudinal direction (Chapter 2)
D_{l_k}	Dispersion coefficient in the longitudinal direction, for the k -th sediment size class
D_{b_k}	Width averaged sediment deposition rates, for the k -th sediment size class, at the interface between the bedload and the suspended load zones
E_{b_k}	Width averaged sediment entrainment rate, for the k -th sediment size class, at the interface between the bedload and the suspended load zones
F_r	Froude number
F	Froude number
H	Average cross-sectional depth (Chapter 4)
K	Conveyance
K	Constant in the coefficient of diffusion (Chapter 4)
L_{tot}	total effective adaptation length
L_b	bedload adaptation length
LPI	Partial Local Inertia
N	Number of a river's reaches (a river has $N+1$ cross-sections)
$NewC$	Numerical scheme for the de Saint Venant equations
P_s	Cohesive surface erosion constant (kg/m ² /s)

P_m	Cohesive mass erosion constant (kg/m ² /s)
$Q_{s,i,k}$	Flow carrying capacity for size class k during time step i
Q	Stream flow
Q_s	Cross-section bedload transport rate
S_0	Bed slope
S_f	Energy slope ($= Q Q /K^2$)
$ T_{i,k} $	Amount of material of size class k present in the active layer
U	Cross-sectional velocity,
V_D	Deposition velocity
V_E	Erosion velocity
W	Channel top width (Chapter 4)
W_{er}	Erosion width
Z	Water surface elevation (Chapter 4)

Lowercase variables

a	Constant for erosion depth computation
b	Constant for erosion depth computation
b_L	coefficient defining bedload adaptation length
d	Particle diameter
d	Particle diameter
f	Fraction of suspended load, ratio of suspended portion to the total sediment concentration
f_k	Weighting coefficient ($\sum_k f_k = 1$)

g	Gravity acceleration
h	Water depth
k	k -th sediment size class
m	Exponent in the LPI method
n	Exponent in the LPI method
n_k	Porosity for the k -th sediment size class
p_c	Percentage of the cohesive sediment in the riverbed
p_k	Percentage of k -th size class material
q	Lateral discharge
q_{tot}^*	equilibrium capacity for total load transport rate (per unit width)
q_{tot}	Actual total load transport rate (per unit width).
s	Specific gravity of sediments
t	Time independent variable
u_*	Shear velocity
x	Spatial independent variable
y	Water surface elevation
z	Suspension parameter ($= \omega_f/(\kappa u_*)$)
z_b	Bed elevation

Greek variables

$ \Delta Z_k $	Amount of material in size class k eroded during time step i
$\Delta z_{b,k}$	Change in bed elevation due to sediment class k
Δz_b	Total depth of deposition/erosion

Γ_k	Source term for class k
ϕ	Relative shear stress ($= \theta/\theta_r$)
Γ	Source term
ψ	Spatial weighting factor
α	Velocity distribution coefficient, or Boussinesq coefficient (Chapter 2)
α	sediment recovery factor (Chapter 4)
β	Ratio of sediment velocity to flow velocity
β_{bed}	Ratio of bedload sediment velocity to flow velocity
β_{sus}	Ratio of suspended sediment velocity to flow velocity
γ	Specific weight of water
κ	Von Karman constant ($=0.41$)
ρ_s	Sediment density (kg/m^3)
ω, ω_f	Sediment fall velocity
ω_s	Suspended sediment fall velocity
$\omega_{s,k}$	Fall velocity of k -th size class particle
τ_b	Bed-shear stress
θ	Shields parameter (Chapter 4)
θ	Temporal weighting factor (Chapter 2)
θ_r	Reference non-dimensional shear stress
τ	Bed shear stress (Pa)
τ_{df}	Critical shear stress for full deposition (Pa)
τ_{dp}	Critical shear stress for partial deposition (Pa).
τ_{es}	Critical surface erosion shear stress (Pa)

τ_{em}	Critical mass erosion shear stress (Pa)
σ	Multiplicative coefficient of the LPI method, $\sigma = \max(0, 1 - F_r^m)$
ν	Kinematic viscosity of water

Numerical schemes variables

A_P	Value of A at center P of finite volume
E, P, W	Centers of volume elements
WW, EE	Centers of volume elements
a_i, b_i, c_i, d_i	Coefficients in the momentum discretized equation
a_P^c, a_E^c, a_W^c	coefficients in the finite volume discretization of the convective term
a_P^d, a_E^d, a_W^d	coefficients in the finite volume discretization of the diffusive term
a_P, a_E, a_W	coefficients in the finite volume discretization of the convection-diffusion equation
e	east face of finite volume element
i	Cross-section
n	Time step
r	variable in the limiter function of Van Leer
w	west face of finite volume element
$\alpha_{i+1/2}$	Coefficient in the continuity discretized equation
$\beta_{i+1/2}$	Coefficient in the continuity discretized equation
$\gamma_{i+1/2}$	Coefficient in the continuity discretized equation
Δx_P	Width of finite volume
ψ	Limiter function of Van Leer

Software

CCHE1D	software of Center for Computational Hydrosience and Engineering-1D
CONCEPTS	Conservational Channel Evolution and Pollutant Transport System
FLDWAV	Flood Wave routing model
HEC-RAS	Hydrologic Engineering Center's River Analysis System
MHYSER	Model of the Hydraulics of Sediments in Rivers
SRH-1D	Sedimentation and River Hydraulics - 1D
UMHYSER-1D	Unsteady Model for the HYdraulics of SEdiments in Rivers - 1D

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CHAPTER 1 INTRODUCTION

There are several numerical models which solve the conservation equations in open channel flows. New scientific findings improve not only the numerical technics to solve the basic equations, but also our understanding of the involved physical processes. However, not all the processes are resolved or fully understood. For instance, when it comes to non-cohesive sediment transport, all the existing models use the developed equation under steady state conditions. New equations for high flow velocity are under study and soon one needs to test them. This brings the need for an available performant numerical laboratory. Then, once the new findings are tested and improved to be approved, one needs a tool to incorporate these finding to be used in real engineering cases.

In this thesis the development of a one-dimensional numerical model is proposed, even though, presently, the trend is toward developing bidimensional or three-dimensional solvers. The main reason of this choice are, from one hand, to deliver a tool to test or use new laboratory findings via the availability of the model's source code. In fact, most of laboratory research findings in river engineering involve one dimensional experiments. From the other hand, the much longer computation times required for 2D models compared to 1D models, the least amount of field data required by 1D models, the stability of the 1D numerical schemes used to solve the governing equations with the gain in computational efficiency make the use of 1D hydraulic models still needed in river engineering particularly for applications with long rivers

This thesis is divided into six chapters consisting of:

- Chapter 1 – Introduction: the present chapter.
- Chapter 2 – Literature review: starting with a general review of literature, this chapter focuses then on specific review of literature for the steps taken during development of the model.
- Chapter 3 – Methodology: It provides the details of the methodological approaches used in this thesis to develop an up-to-date numerical model.

- Chapter 4 – Model presentation: This is the core of the thesis. The model is presented, validated and the results of its two applications are summarized. These applications are detailed in the next chapters.
- Chapter 4 – First application: Application of the model to the Ha! Ha! River during the Saguenay 1996 flood. Published data are used to model the river morphological changes caused by the flood.
- Chapter 5 – Second application: Application of the model to risk of pollution minimization when increasing the conveyance of Aux Sables River after the Saguenay 1996 flood.
- Chapter 6 – General discussion: This chapter provides a review of the research work done during this thesis.
- Conclusion and recommendations: In this section, the whole scientific work done during this thesis is concluded, and some recommendation for future work are suggested to improve further the capabilities of the developed model.

Main objective:

The main objective of this thesis is developing a new unsteady numerical model for the hydraulics of sediments in rivers having twofold goals: to be used as a performant numerical research laboratory to test new findings and to be a continuously improved model to solve real engineering cases.

Specific objectives:

Through the model's development stages, the specific objectives are observed. The main ones are:

- Use of a performant numerical scheme, different of the classical Preissmann one. This is part of the creation of a numerical laboratory objective.
- Use of the same sediment convection-diffusion for different sediment transport modes (bedload, suspension load or total mode) and types (cohesive or non-cohesive).

- Model flows and sediments in rivers and channel networks with or without movable boundaries.
- Model both steady and unsteady flow conditions, model cohesive or non-cohesive sediment transport.
- Model rivers' width changes using at least one minimization theory and riverbank retreat method. This objective improves the capabilities of the present one-dimensional numerical laboratory.
- Consider hydraulics structures such as weirs, bridges, and gates along with other internal boundary conditions (such as time-stage tables, rating curves).
- Account for bed material mixing. This is an important feature for riverbed sorting and armoring modeling
- Add visualization windows to monitor the solution during the simulation. This will save time during numerical investigations

Originality justification:

The originality of this thesis resides in the development of a new performant numerical model. The developed model brings together several published ideas to produce a performant tool for researchers and engineers, and this is precisely the novelty of this thesis, as mentioned by D'Alembert¹, the French Mathematician, in his paper "Découverte": "...the discoveries which are the fruit of genius (and it is of these especially that it must be question) are made in three ways: by finding one or more entirely new ideas, or by joining a new idea to a known idea , or by bringing together two known ideas ...".

¹ Jean Le Rond D'Alembert dans *Encyclopédie : ou, Dictionnaire raisonné des Sciences, des Arts et des Métiers*. Tome IV, Paris, M.DDC.LIV, pp.705-706, article "Découverte". Mis en ordre, et publié entre 1751 et 1765, par M. d'Alembert, Paris, Briasson et autres, 17 volumes.

CHAPTER 2 LITERATURE REVIEW

Sediment transport and river flows are still among the most and least understood processes in nature. Before the 1970s, most of river engineering problems called for field investigations and laboratory physical models. Since then, advancement computational models took over. Presently, numerical modeling is widely used in river engineering studies. Determination of the flood's risk (Mahdi, 2007), investigations of river morphology changes (Formann et al., 2007; Mahdi, 2009), stream restoration projects and sediment deposition studies (Langendoen, 2000; Lauer and Parker, 2008) are some examples of river engineering problems involving numerical modeling. Several numerical models were developed during the last decades: 1D models have been used to simulate flow and sediment transport in short- and long-term simulation, and 2D and 3D models have been used to predict more details flow conditions in more detail. Even if two-dimensional (2D) models are gaining in popularity, the much longer computation times required for 2D models compared to 1D models, the least amount of field data required by 1D models, the stability of the 1D numerical schemes used to solve the governing equations with the gain in computational efficiency (Papanicolaou et al., 2008), make the use of 1D hydraulic models still needed in river engineering particularly for applications with long rivers (Deal, 2016). Finally, it should be noted that, in open channel modeling, for 1D unsteady flow equations, the finite difference method dominates the available solving methods (Szymkiewicz, 2010). For example, the following non exhaustive schemes list was presented by Cunge et al. (1980): Lax scheme, leap-frog scheme, Abbot-Ionescu scheme, Delf Hydraulic Laboratory scheme, Vasiliev scheme, Gunaratman-Perkins scheme, Preissmann scheme.

2.1 Governing equations

For 1D hydromorphodynamic modeling the governing equations are derived from the principles of mass and momentum conservation for the water and sediment phases. The used equations are, for the flowing water, the continuity and momentum equations, known as the de Saint Venant (1871) equations, and mass conservation and sediment transport equations for the solid phase.

2.1.1 De Saint Venant equations

The de Saint Venant equations, formulated in the 19th century by two mathematicians, de Saint Venant and Bousinnesque, describe unsteady, gradually varying 1D open channel flow. For example, Singh (1996) and Cunge et al. (1980) give an extensive overview of these equations which can be expressed in term of discharge and wetted area, discharge and water depth (elevation), velocity and wetted area, or velocity and water depth (elevation). These equations can be derived by averaging the three-dimensional Reynolds equations over the cross-section of the channel or by applying conservation laws to a control volume (Szymkiewicz, 2010). The basic assumptions for the analytical derivation of the de Saint Venant equations are the following:

- the flow is one-dimensional, i.e. the velocity is uniform over the cross-section and the water level across the section is represented by a horizontal line
- the streamline curvature is small, and the vertical accelerations are negligible, so that the pressure can be taken as hydrostatic
- the effects of boundary friction and turbulence can be accounted for through resistance laws analogous to those used for steady state flow
- the average channel bed slope is small so that the cosine of the angle it makes with the horizontal may be replaced by unity.

These hypotheses do not impose any restriction on the shape of the cross-section of the channel and on its variation along the channel axis, although the latter is limited by the condition of small streamline curvature.

The commonly used form of de Saint Venant equations (Figure 2.1) is:

$$\begin{cases} \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q & (2.1a) \\ \frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left(\alpha \frac{Q^2}{A} \right) + gA \frac{\partial y}{\partial x} + gAS_f = 0 & (2.1b) \end{cases}$$

where, Q = discharge, A = cross-sectional area for the flow, q = lateral discharge, t = time independent variable, x = spatial independent variable, g = gravity acceleration, α = velocity

distribution coefficient, y = water surface elevation, S_0 = bed slope, S_f = energy slope ($= Q|Q|/K^2$), and K = conveyance.

The de Saint Venant equations are hyperbolic. To be solved, this system needs two initial conditions and one upstream and one downstream boundary conditions for sub-critical flow, and two initial conditions and two upstream boundary conditions for super-critical flow.

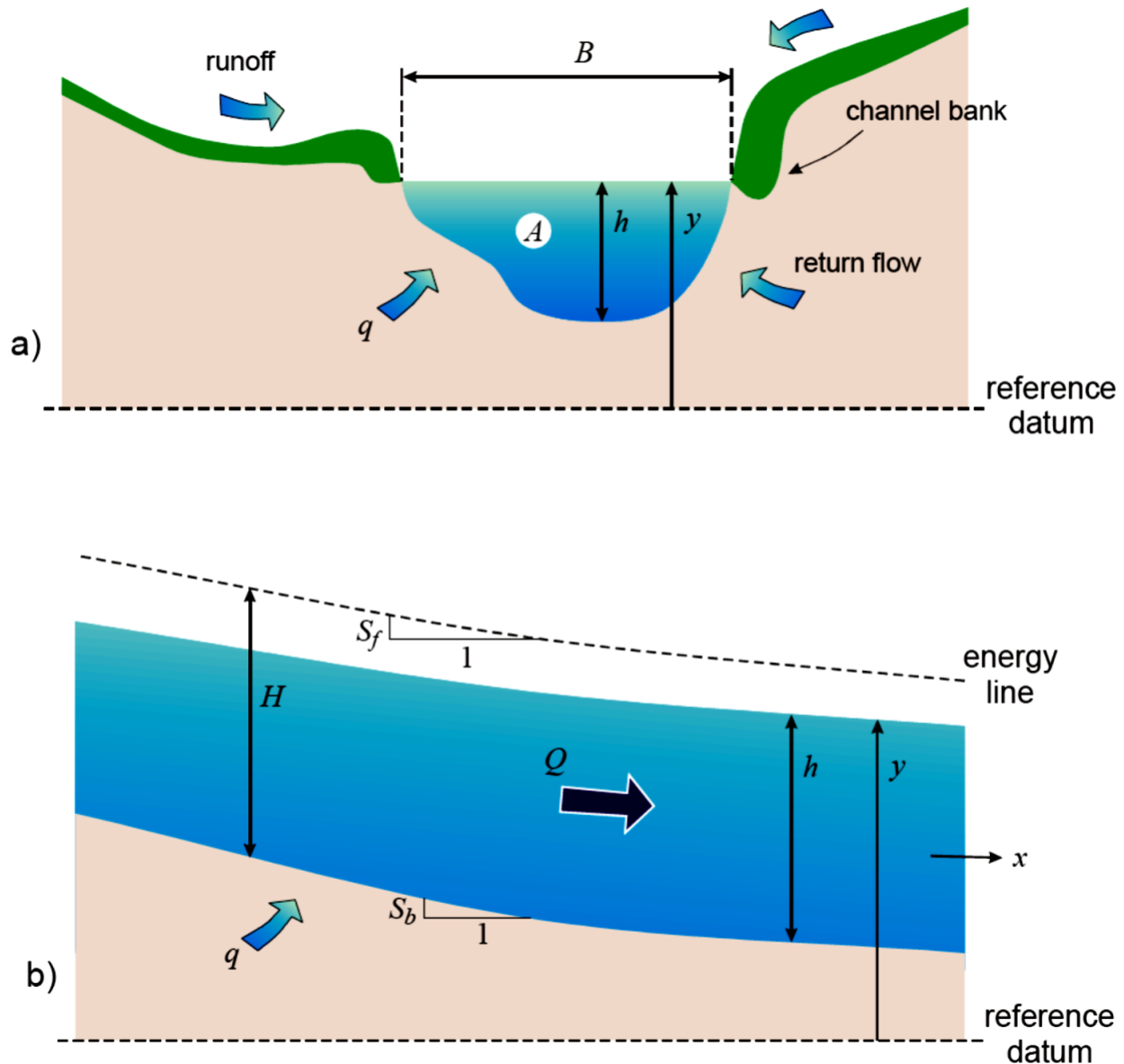


Figure 2-1 Definition sketch of hydraulic variables used in the governing equations: a) cross-sectional view and b) longitudinal view

2.1.2 Sediment equations

Two approaches are used to study sediment transport in rivers. *The first one*, distinguishes two layers across the water column and one or more layers covering the streambed (see Figure 2.2; e.g., Armanini and Di Silvio, 1988; van Niekerk *et al.*, 1992; Graf, 1981; Yang 2003).

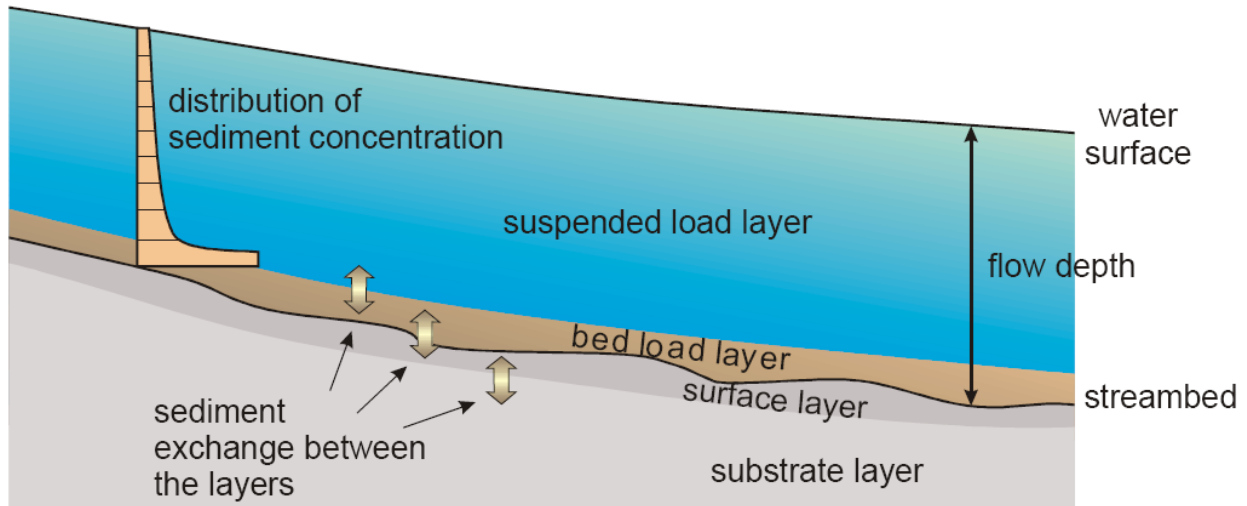


Figure 2-2 Multi-layer approximation of sediment transport (adapted from Mahdi, 2003)

The two layers across the water column are (see Figure 2.3):

- a layer near the bed where sediment particles roll, slide, or saltate, and are transported as *bed load*, and
- a layer spanning the remainder of the water column where particles are in suspension and are transported as *suspended bed-material load* or *wash load*.

The second approach consists of considering sediment transport as a total load (one phase) represented by a concentration by volume C_T (Figure 2.4).

This *two-layer approach* may be a more accurate description of sediment transport mechanics because of the difference in suspended and bed load transport modes. However, the expressions for the sediment fluxes between the layers are empirical and are mainly derived from data acquired from sand-bed rivers and flume experiments (e.g., Garcia and Parker, 1991). The *one-layer approach* cannot distinguish the different behavior of sediment particles in the suspended and bed loads, but it is computationally more efficient because of its simplicity.

Mathematical model for sediment transport is based on conservation laws; i.e., conservation of suspended load, bedload, and bed-material for each size fraction class. Additional auxiliary equations are needed, such as for bed material sorting (the process of exchange of sediment particles between the water stream and some conceptual model of a layered bed), or sediment transport capacity. Wu (2007) summarized the different formulations for the sediment transport equations. Without being exhaustive, but keeping a good level of generality, the set of basic 1D partial differential equations governing the transport of sediments can be written in the following manner:

First approach, suspended load and bedload transport:

Suspended load transport by size fraction is

$$\frac{\partial AC_k}{\partial t} + \frac{\partial QC_k}{\partial x} = \frac{\partial}{\partial x} \left(D_l A \frac{\partial C_k}{\partial x} \right) + B(E_{b_k} - D_{b_k}) \quad (2.2a)$$

where A = cross-sectional area for the flow, Q = water discharge, C_k = cross-sectional suspended sediment-load concentration by volume, E_{b_k} and D_{b_k} = width averaged sediment entrainment and deposition rates of particles at the interface between the bedload and the suspended load zones, respectively, D_l = dispersion coefficient in the longitudinal direction, B = storage width. The subscript k denotes the k -th sediment size class.

Bedload transport (*sediment continuity*) by size fraction is

$$\frac{\partial A_{\delta k} C_{\delta k}}{\partial t} + (1 - \lambda_k) \frac{\partial A_{b_k}}{\partial t} + \frac{\partial Q_{s_k}}{\partial x} = B(D_{b_k} - E_{b_k}) \quad (2.2b)$$

Where n_k = porosity and A_b = cross-sectional area of the mixing layer, Q_s = bedload transport rate at the cross-section, $C_{\delta k}$ = cross-sectional averaged bedload concentration by volume, $A_{\delta k}$ = cross-sectional area of the bedload zone. The subscript k denotes the k -th sediment size class.

In most circumstances, the change in suspended sediment concentration in any cross-section is much smaller than the change in riverbed; i.e.,

$$\frac{\partial A_{\delta k} C_{\delta k}}{\partial t} \ll \frac{\partial A_{b_k}}{\partial t}$$

Eq. (2.2b) simplifies then to:

$$(1 - \lambda_k) \frac{\partial A_{b_k}}{\partial t} + \frac{\partial Q_{s_k}}{\partial x} = B(D_{b_k} - E_{b_k}) \quad (2.2c)$$

Second approach, considering sediment transport as total load (one phase) represented by a concentration by volume C_T , the equation describing sediment transport is similar to Eq. (2.2a):

$$\frac{\partial A C_{T_k}}{\partial t} + \frac{\partial Q C_{T_k}}{\partial x} = \frac{\partial}{\partial x} \left(D_{l_k} A \frac{\partial C_{T_k}}{\partial x} \right) + B(E_{b_k} - D_{b_k}) \quad (2.2d)$$

And bed updating is given by:

$$(1 - \lambda_k) \frac{\partial A_{b_k}}{\partial t} = B(D_{b_k} - E_{b_k}) \quad (2.2e)$$

where C_{T_k} = Cross-sectional total load (one phase) concentration by volume, for the k -th sediment size class, and D_{l_k} = Dispersion coefficient in the longitudinal direction, for the k -th sediment size class.

In principle, Eqs. (2.2a) and (2.2d) are applicable only for fine sediments and low concentrations (Greimann and Holly, 2001). It is commonly accepted that if the sediment concentration is lower than 0.1 by volume, Eqs. (2.2a) and (2.2d) can be used. Because the sediment concentration usually is not high in most natural rivers, the diffusion model has been widely adopted in river dynamics.

A promising approach, by Greimann (2008), proposes an unsteady total load equation for depth-averaged sediment transport models. This equation does not require the load to be segregated a priori into bed and suspended but rather automatically switches to suspended load, bed load, or mixed load depending on a transport mode parameter consisting of local flow hydraulics. This ‘‘new’’ approach can handle cohesive and non-cohesive sediments using the same form of the diffusion equation.

2.1.3 Equilibrium and non-equilibrium sediment transport

The first developed software adopted the equilibrium or saturated sediment transport model which assumes that the actual bed-load transport rate is equal to the transport capacity under the equilibrium condition at every cross-section. The equilibrium bed-load transport rate can be determined using an empirical formula such as those given in Table 2.1.

Table 2-1 Example of sediment transport functions and its type

Equation	Type
Meyer-Peter and Müller (1948)	Bed load
Laursen (1958)	Bed-material total load
Modified Laursen's Formula (Madden, 1993)	Bed-material total load
Toffaleti (1969)	Bed-material total load
Engelund and Hansen (1972)	Bed-material total load
Ackers and White (1973)	Bed-material total load
Ackers and White (HR Wallingford, 1990)	Bed-material total load
Yang (1973) + Yang (1984)	Bed-material total load
Yang (1979) + Yang (1984)	Bed-material total load
Parker (1990)	Bed load
Brownlie (1981)	Bed-material total load
Yang et al. (1996)	Bed-material total load

For cohesive sediment, the two counterfeited types of erosion include; surface and mass erosion (Mehta et al., 1989). Surface erosion is visible in cases when the bed shear stress has just began exceeding a critical value. In elevated cases when the level of stress increases, the value bed shear stress overtakes that of the bulk shear strength of a layer of material, in which the layer is prone to mass erosion.

Figure 2.5 reflects the most idealistic case of erosional and depositional characteristics that are retractable from physical tests. In general, both erosion rate constant M and the critical erosion shear stress τ_e varies along with the bed properties in two factors; depth and time (Ariathurai & Krone, 1976; Mehta et al., 1989). To determine empirical model parameters, the most reliant sources include field studies and laboratory flume studies.

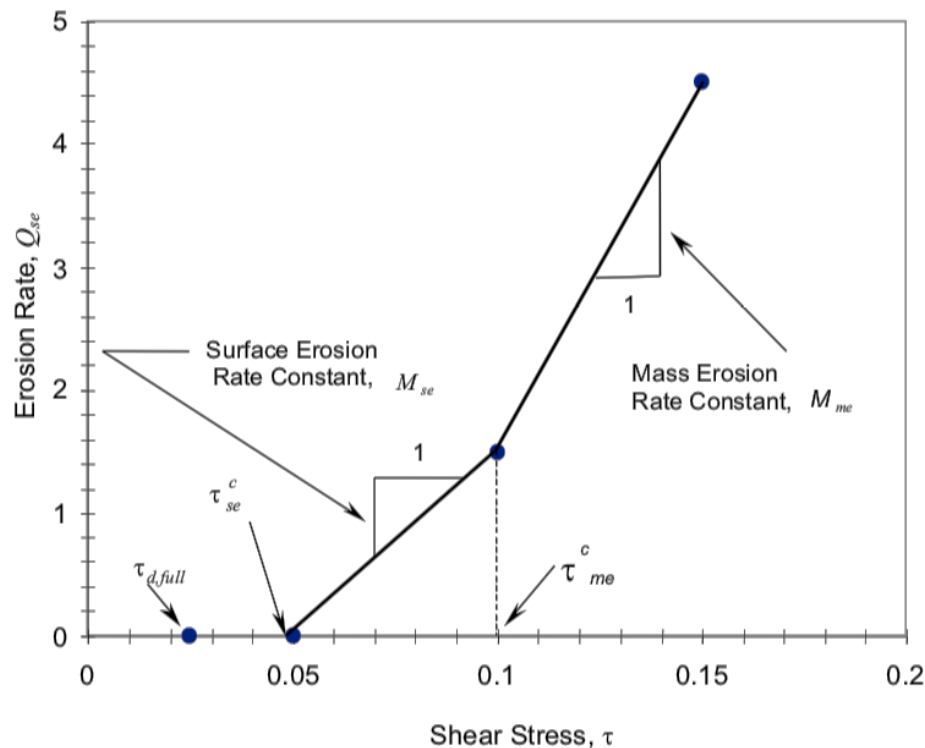


Figure 2-5 Erosional characteristics requirements (after Vermeyen, 1995)

However, sediment transport in natural rivers usually is not in states of equilibrium because of variations in flow conditions and channel properties. In fact, sediment cannot reach new equilibrium states instantaneously, due to the temporal and spatial lags between flow and sediment

transport (Figure 2.6). A more realistic and general approach is the non-equilibrium (or unsaturated) sediment transport model.

The sediment flux between streambed and overlying fluid, not only depends on flow conditions and size fractions available in the bed or transported by the flow, but also on the interparticle forces in the presence of clay particles on the streambed. The entrainment rate of sediment particles on *cohesive* streambeds markedly differs from that of *cohesionless* streambeds.

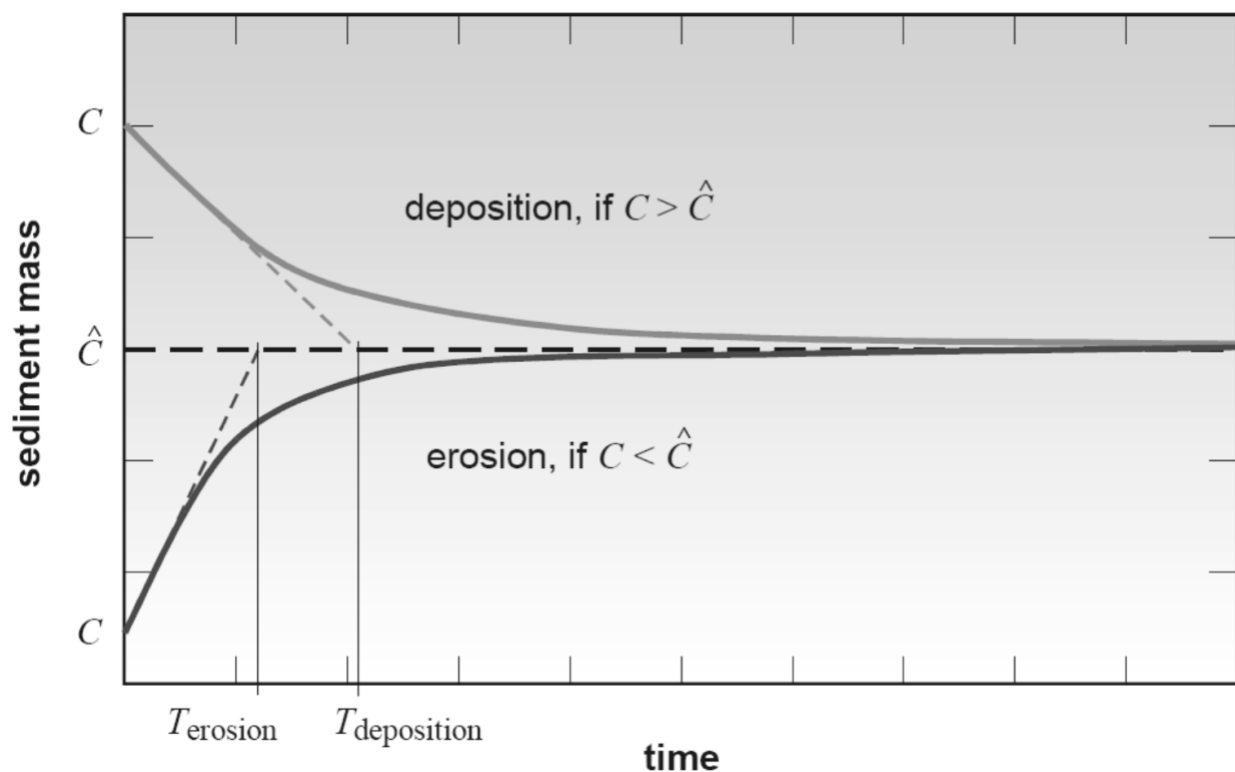


Figure 2-6 Progress of sediment mass toward equilibrium for the case of constant flow conditions (adapted from Mahdi 2003)

Several equations exist in the literature for the erosion and entrainment rates of sediments such as the formulations proposed by Bennett (1974) and Foster and Meyer (1972) for non-cohesive sediments, and by Ariathurai and Arulanandan (1978), Parchure and Mehta (1985), and Krone's (1962) for cohesive sediment. Detailed review of the available formulations can be found in Wu (2007).

2.1.4 Transport and sorting of non-uniform sediment mixture

Most of transport capacity formulas were developed for uniform size gradations. For non uniform sediment, the transport capacity is calculated as follows:

$$\hat{C}_{t_k} = p_k \hat{C}_k \quad (2.3a)$$

Where $\hat{C}_k$ = transport capacity for size fraction k , computed from uniform sediment formula as if the size fraction was the only sediment present in the bed, p_k = percentage of the material belonging to size class k , and $\hat{C}_{t_k}$ = actual fractional transport potential for k -th size class.

The total transport potential is given by:

$$\hat{C}_t = \sum_k \hat{C}_{t_k} \quad (2.4)$$

Eq. (2.4), the most widely used form of fractional transport in numerical modeling, has many shortcomings (Hsu and Holly, 1992). It predicts zero transport capacity for fractions that are not present in the bed. To overcome this problem, instead of p_k , some models use $\alpha p_k + (1 - \alpha)p_j^*$, where α = a weighting factor ($0 \leq \alpha \leq 1$) and p_j^* = percentage of the k -th size class entering the reach. Another less used method (e.g., Karim and Kennedy, 1982) uses a distribution function to compute the transport capacity for each individual size class: $C_{t_k} = f_k C_t$ (with $\sum_k f_k = 1$).

As a result of computing sediment transport by size fraction, particles of different sizes are transported at different rates. Hence, some particle sizes may be eroded, while others may be deposited or may be immovable, depending on the hydraulic parameters, the incoming sediment distribution, and the bed composition. Consequently, several different processes may take place. For example, all the finer particles may be eroded, leaving a layer of coarser particles for which there is no carrying capacity. No more erosion may occur for those hydraulic conditions, and the bed is said to be armored. This armor layer prevents the scour of the underlying materials and the sediment available for transport becomes limited to the amount of sediment entering the reach. However, future hydraulic events, such as an increase of flow velocity, may increase the flow carrying capacity, causing the armor layer to break and restart the erosion processes in the reach. Many different processes may occur simultaneously within the same channel reach. These depend not only on the composition of the sediment entering the reach, but also on bed composition within that reach.

There are many approaches to bed sorting. Some of the most common are the ones by Hirano (1971), Bennett and Nordin (1977) and Borah et al. (1982). Other approaches exist in the literature such as the ones by Bayazit (1975), Karim and Kennedy (1982), Rahuel *et al.* (1989), Armanini and di Silvio (1988), and van Niekerk *et al.* (1992). For more detailed treatment of the subject see, for example, Armanini (1995) and Wu (2007).

2.2 Numerical solution of de Saint Venant equation

The current literature describes several numerical techniques that are suitable for solving the de Saint Venant equations including the method of characteristics, explicit difference methods, fully implicit methods, Godunov methods (Cunge et al., 1980; Hirsh, 1990b).

The first method used to solve the de Saint Venant equations was the method of characteristics which is very efficient in the treatment of boundary conditions. Abbott (1966) details its description for open channel flow. This method was not used because of some technicalities. In particular, the method of characteristics does not guarantee volume and momentum conservation and it has difficulties caused by the nonlinearity of equations as well as by the required non-equally spaced nodes typical for rivers. The Godunov's type methods (see, e.g., Toro, 1997) are very effective on simple channel geometries with flat bottom and rectangular cross-section. For varying bottom profiles, the bottom slope appears as a source term that may generate artificial flows (Toro, 2001; Garcia-Navarro and Vazquez-Cendon, 2000; Vasquez-Cendon, 1999). Moreover, Godunov's type methods are explicit in time and the allowed time step is restricted by a CFL stability condition, relating time step to the spatial step and the wave speed.

2.2.1 Preissmann scheme

In fluvial hydraulics, it is commonly accepted that the most performant numerical scheme is Preissmann (Preissmann, 1961; Abbott, 1979; Abbott and Basco, 1989; Cunge et al., 1980), also known as the Box Scheme (Figure 2.7a). Till now, this scheme is still used in many softwares such as the widely used HEC-RAS, the USACE's software (Brunner GW, 2016), even if several authors pointed out the limitations of Preissmann's scheme. In particular, this scheme encounters numerical instability in the transition between supercritical and subcritical flow regimes as described by Meselhe and Holly (1993) and Kutija and Hewett (2002).

In fact, the Von Neumann condition for numerical stability implies (Figure 2.7b):

$$\left(\psi - \frac{1}{2}\right)\frac{1}{C_r} - \left(\theta - \frac{1}{2}\right) \geq 0 \quad (2.5)$$

When C_r is the Courant number, and where ψ and θ are respectively spatial and temporal weighting factors ranging from 0 to 1.

If $\psi \neq \frac{1}{2}$, the stability of the scheme will depend on the sign of C_r , or equivalently, on the direction of travel of a characteristic wave. Even if $\psi = \frac{1}{2}$ and $\theta \geq \frac{1}{2}$, because the above analysis is based on the linearization of the basic equations, in practice, instability does occur in the transition between flow regimes.

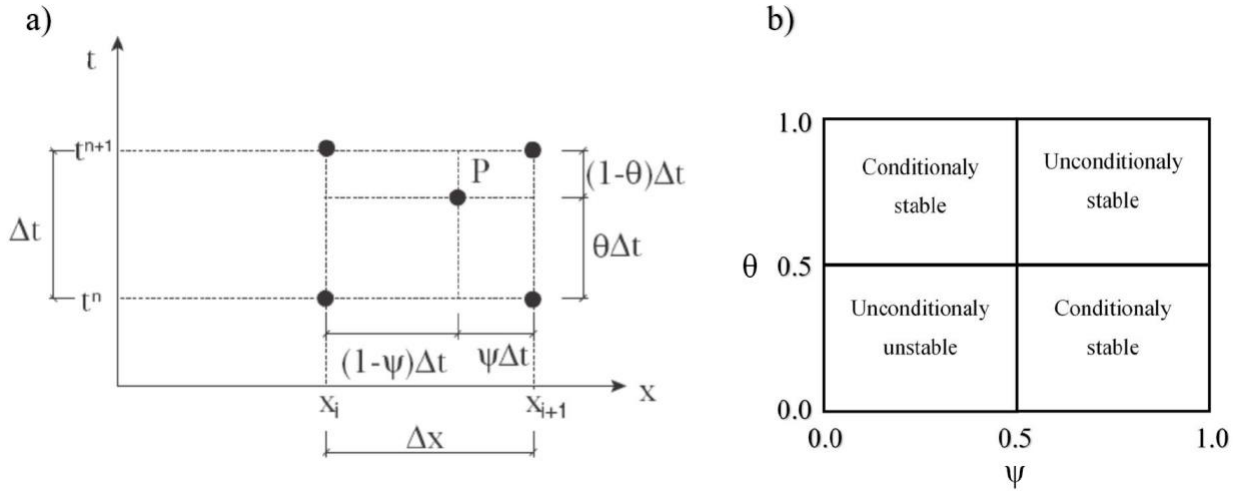


Figure 2-7 Preissmann's numerical scheme. (a) Values at point P are weighted using the four close points on the x-t grid, (b) Regions of stability in ψ - θ plane when $C_r \geq 0$

However, in practice, whenever the transition between flow regimes occurs, Preissmann scheme is used by minimizing the effect of the inertia term of the momentum equation. This numerical “trick” consists of multiplying the acceleration term of the momentum equation by a coefficient, $\sigma = f(F_r)$, function of the Froude number, F_r . This approach is known as Partial Local Inertia (LPI) technique. For example, in the model FLDWAV, Fread and Lewis (1998) use the expression $\sigma = \max(0, 1 - F_r^m)$, $m > 0$. Hence, for fluvial flow $\sigma \approx 1$ (for $m > 30$), and for supercritical flow, $\sigma = 0$ and FLDWAV solves the diffusion equation. Most of the famous and worldwide developed

1D models use the Preissmann scheme with the LPI technique. To name few: FLDWAV (Fread and Lewis, 1998), CONCEPTS (Langendoen, 2000), CCHE1D (Vieira and Wu, 2002) and HEC-RAS (Brunner GW, 2016).

2.2.2 NewC scheme

Kutija and Hewett (2002) proposed a numerical scheme, NewC, to overcome the Preissmann's scheme limitation. NewC scheme, able to model sub-, super- and trans-critical flow conditions, uses a staggered grid, where the computational points for flow, Q , are located at the cross-sections, and water surface elevations, Z points, are located halfway between the cross-sections. NewC scheme has some limitations in solving for supercritical flow. In fact, it requires one boundary condition at each end of the domain. Hence, supercritical flow should not occur at the upstream or downstream boundaries of any modeled river reach. Moreover, in some modeling cases, for higher values of Froude number ($F_r \geq 1.5$), sometimes numerical instability is observed. This is because the stability analysis Kutija and Hewett (2002) is based on linear analysis using Fourier series expansions (Hirsh, 1990a; Pletcher et al., 2012).

At this stage an important remark is required: To model transcritical flow, the conservative form of the momentum equation should be solved (Meselhe et al. 2005). Transcritical flows are generally encountered in cases of dam-break flows or in steep mountain streams where it is difficult to obtain sufficiently detailed topography to resolve hydraulic jumps. Moreover, when they occur, the 1D flow assumptions are not valid. Therefore, the de Saint Venant equations are not recommended to obtain detailed hydraulic information near the transition between sub- and super-critical flows.

However, the NewC scheme is stable for both sub- and super-critical flow and will be accurate sufficiently far away from the transition. The advantage to not using the non-conservative form of the momentum equations is that mass conservation of flow can easily be ensured. In addition, for complicated natural channels it is analytically and computationally difficult to discretize the source terms of the conservative form of the momentum equations to ensure that the scheme preserves a stationary solution (Sanders et al., 2003). That is, it is difficult to discretize the pressure forces of the nonconservative form without inducing artificial flow for cases where the initial water surface elevation is constant.

2.3 Sediment transport modeling

Solving a convection-diffusion for sediment transport is very challenging because, the used numerical scheme should introduce no, or very little, numerical diffusion and this is not obvious. In fact, even if there is no diffusion during sediment transport, the transport equation will be an advection equation with a source term, hence the solution is not obvious.

There exists much literature on methods that have been developed to integrate the advection equation. Though the character of the advection equation with constant coefficients is well understood, its numerical integration may be quite difficult, especially if the solution displays large gradients (e.g., Hirsch, 1988a). Also, if the coefficient in the advection term is varying, as is the case for suspended sediment transport, the complexity of the numerical integration increases. Further, one may use a stable scheme to integrate the advection equation, however, they will obtain a *diverging* solution because of the behavior of the *source term*. If the time step is much larger than the needed time scale for the concentration to reach the equilibrium concentration (see Figure 2.5), the discretized source term will be much larger in magnitude than the discretized unsteady and advection terms. Therefore, the source term controls the sediment concentration, and any solution method of the advection equation will be either unstable or inaccurate. The source term must be approximated by an implicit method to assure a stable solution (LeVeque and Yee, 1990). When the diffusive term is present, the situation becomes critical, hence numerical diffusion introduced by some numerical schemes should be avoided.

A very effective method used to solve the convection-diffusion, Eq. (2.2a), is the fractional step method (Yanenko, 1971). Solving Eq. (2.2a) is equivalent to solving the following set of equations:

$$\frac{\partial AC_k}{\partial t} + \frac{\partial QC_k}{\partial x} = \frac{\partial}{\partial x} \left(AfD \frac{\partial C}{\partial x} \right) \quad (2.6)$$

$$\frac{\partial AC_k}{\partial t} = B(E_{b_k} - D_{b_k}) \quad (2.7)$$

Eq. (2.6) is solved, using a finite volume method, to obtain intermediate solution, AC_k , which will be used as initial condition to solve, in the next step, the initial value problem in Eq. (2.7) to obtain the solution at the next time step.

Very powerful numerical schemes for Ordinary Differential Equations, such as the very famous Runge-Kutta 4th order method (Chapra and Canale, 2020), can be used to solve Eq. (2.7).

2.4 River Morphological changes

Once the volume of erosion or deposition is computed, the final step consists of applying it to the cross sections. For 1D modeling, different models offer different options; some allow vertical adjustment only like MIKE11 (DHI, 2009), while others allow for lateral adjustment too like GSTARS3 (Yang and Simoes, 2003).

Two options are available for channel geometry adjustment: vertical adjustment and width adjustment. Channel widening or narrowing can take place only at the stream tubes adjacent to the banks. For stream tubes that are not adjacent to the banks, i.e., interior tubes, bed adjustments can be made only in the vertical direction. The process is briefly schematized in figure 2.8. A vertical adjustment will move all the cross-section points below the water surface the same vertical distance as shown in Figure 2.8a. A width adjustment will move the cross-section points, below the water surface, in the horizontal direction as shown in Figure 2.8b. If there is only one stream tube (the whole river), for width adjustments, the maximum bed geometry change occurs near the bank and the thalweg elevation remains unchanged. Criteria are needed to decide whether the adjustment will proceed in the vertical or lateral direction at a given time step of computation. These criteria are given by theories of minimization.

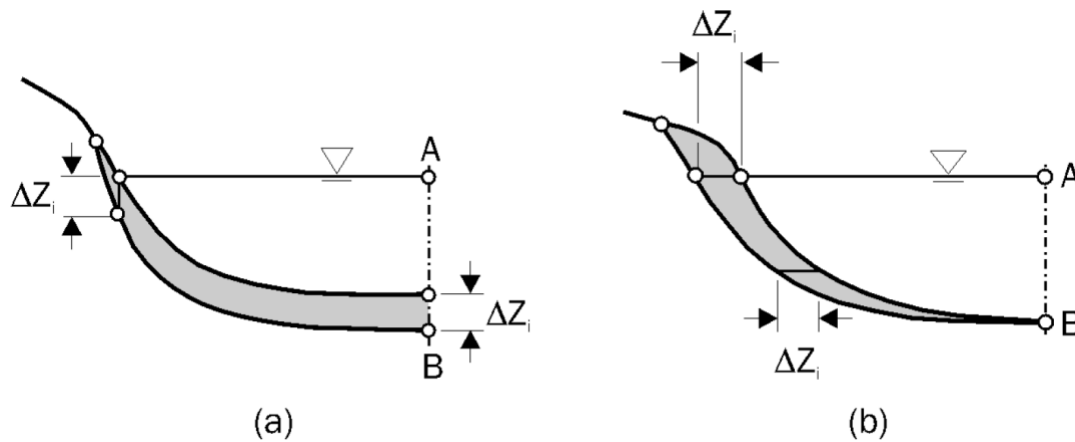


Figure 2-8 Schematic representation of channel changes: (a) vertical adjustment due to scour or deposition; (b) width adjustment due to scour or deposition. Line AB denotes the stream tube boundary (adapted from Yang and Simoes, 2003)

2.4.1 Minimization theories for Channel Narrowing and Widening

Among the minimization theories (Yang, 1972; Chang, 1988) for channel narrowing and widening adjustment, three have been used and are promising: minimization of total stream power, minimization of energy slope, and minimization of bed slope. According to the minimization of total stream power theory, if lower total stream power is the result of alteration of the channel widths, then channel adjustments are made in the lateral direction. Otherwise, the adjustments progress in the vertical direction. While adopting the minimization of energy slope, if the energy slope at a cross-section is greater than the weighted average energy slope of its adjacent sections, then the channel width at this section is reduced during deposition or the depth is increased during erosion. However, if the energy slope is smaller, the channel depth at this section is decreased during deposition or an increase of width occurs during erosion. Finally, for the minimization of bed slope, if the bed slope at a cross-section is greater than the weighted average bed slope of its adjacent cross-sections, then the channel width at this section is reduced during deposition, or the depth is increased during erosion. Otherwise, the channel depth at this section is decreased during deposition or the width is increased during erosion.

2.4.2 Riverbank retreat

Bank erosion occurs through variety of processes, which could be divided into two main groups: fluvial erosion and mass failure processes.

Fluvial erosion process

The three main types of hydraulically induced failure are (Figure 2.9): **bed degradation** (vertical) which follows a process where there is a major increase in the erosive power of flowing water than eventually leads to mobilization and erosion of the sediments on a channel bed. The erosion process typically occurs in the vertical direction which leads to an increase in the bank height and steepness of the bank slope, exposing the bank to possibility of undercutting and mass failure. **Basal cleanout** is defined as a process where the supportive or protective materials (could be placed or from mass failures) located at the bank toe are withdrawn, which is typically triggered through high flows. It is notable to mention that repetition of a cycle involving basal cleanout, undercutting, mass failure, and bank toe accumulation all controls the intensity of retreat rate of a stream bank. **Undercutting** is explicitly the removal of bank materials (through the flow of water). Some of the major causes of undercutting are related to effects such as the presence of debris and vegetation, characteristics such as poor drainage, or presence of non-cohesive materials.

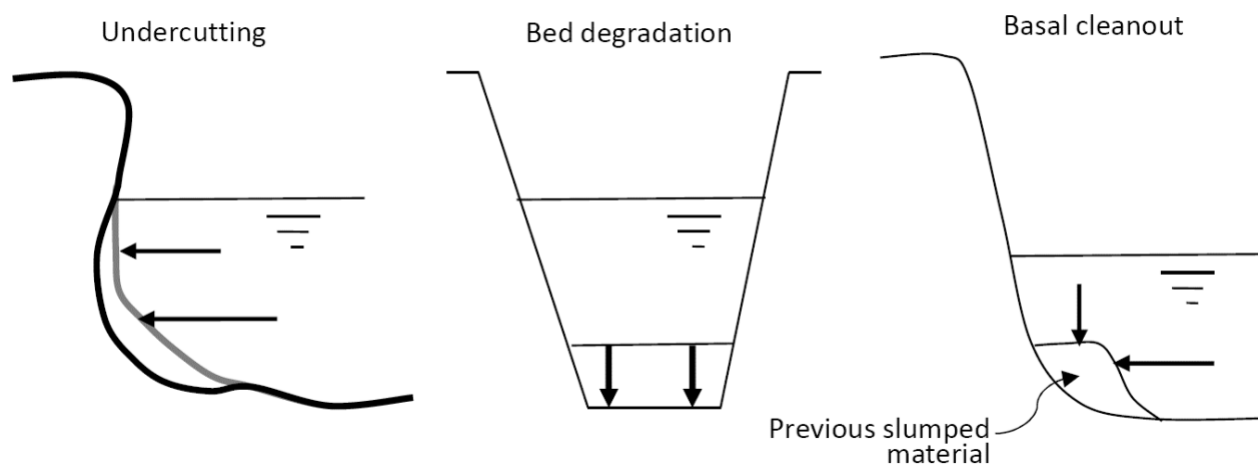


Figure 2-9 Types of hydraulically induced erosions

Following the excess shear stress approach set forth by Arulanandan *et al.* (1980), Osman and Thorne (1988) developed a method to predict the *lateral-erosion rate* at the bank toe.

Excess shear stress is defined as the difference between the shear stress exerted by the flowing water on the bank, and the critical shear stress at which the soil particles are entrained.

Mass failure process

It is considered with certitude that numerous existing numerical models neglect the presence and effect of bank erosion, hence causing insufficient research base references in geomorphic response of a channel. Figure 2.10 summarizes the most popular riverbanks failure mechanisms.

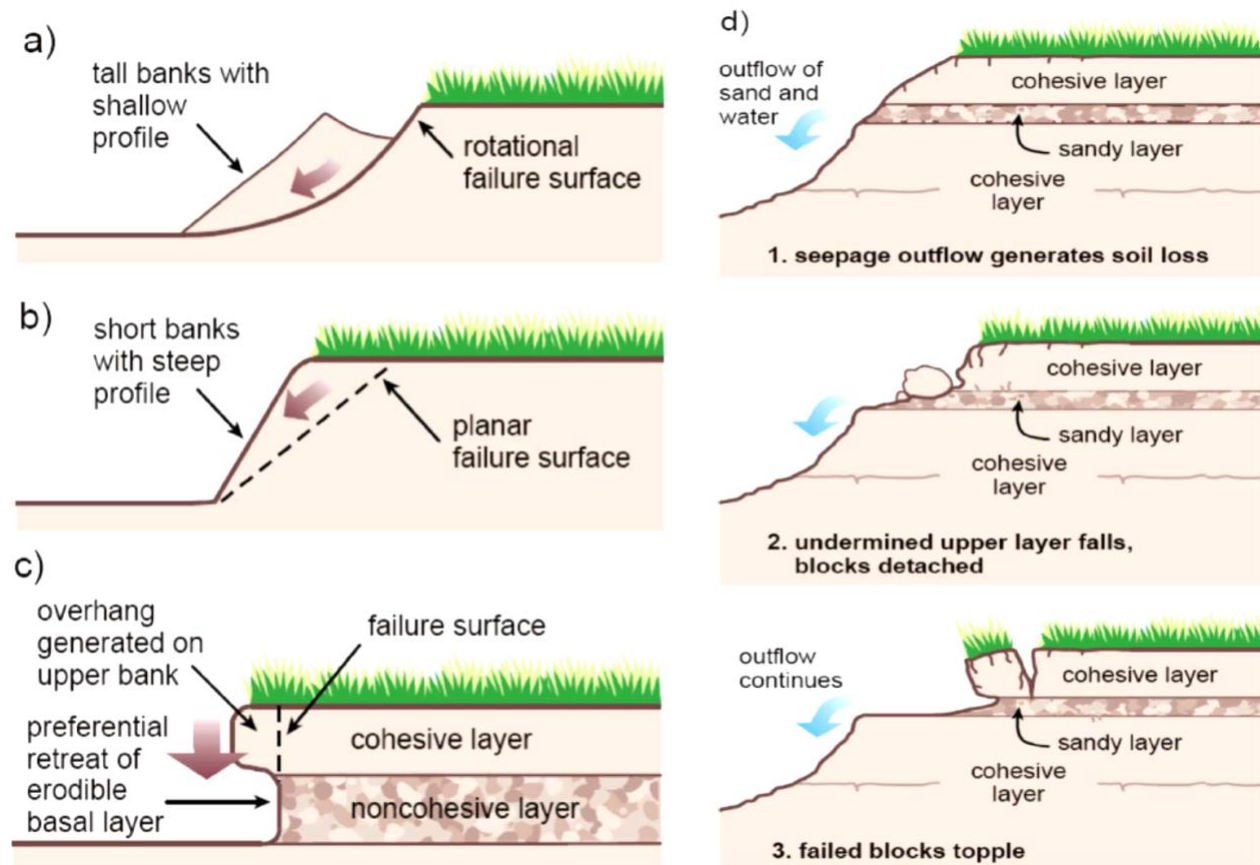


Figure 2-10 Bank failure mechanisms: a) rotational failure, b) planar failure, c) cantilever failure, and d) piping or sapping failure (adapted from Hagerty, 1990)

Many classifications have been accredited as to the key roles in mass failure. According to Watson and Basher (2006), the concepts of mass failure mechanisms can be classified through seven categories: **Dry Granular Flow**, occurring typically on non-cohesive bank at the angle of repose, where the individual sediment grains roll, slide and bounce down the bank in a layer. **Shallow Slide** takes place on the bank where a layer of material moves along a plane that is parallel to the bank surface. This type failure is native to areas with soils of low cohesions and moderately steep angles and when the angle of the bank is greater the angle of internal friction of the bank material. **Rotational failure** is described through a deep-seated movement of material in up and down-ward directions along a curved slip surface. **Planar (Slab) failure** is a sliding and forward toppling of a deep-seated mass into the channel, that is caused mainly from; scour at the bank toe, high pore pressure in the bank material, and tension cracks present at the top of the bank. In general slab failures are likely to occur in cohesive banks with steep bank angles ($>60^\circ$), with low flow conditions. According to Simon et al. (2000) planar failure are typically more common in order of occurrence. **Cantilever Failure**, it occurs when an overhanging block collapses into the channel, due to significant case of undercutting, and is most likely to occur on banks with composite layers of fine/coarse and or cohesive/non cohesive materials. **Wet Earthflow** occurs when soil (of a bank) flows in a form of highly viscous liquid, where it flows down the bank and forms an accumulation of material at the toe. The main causes of wet earthflow are significant loss of strength mainly due to saturation and increased bank weight. It is most common for wet earthflow to occur on low-angle banks, and ones exposed to strong seepage and with relatively poor drainage. Finally, **Piping Failure** is expressed as the collapse of a certain part of the bank, caused by high groundwater seepage pressures and seepage flows which leads to the removal of specific sections of the bank. The major cause of the failure is ground water flow along inter-bedded saturated layers.

Few models take into account mass failure processes. GSTARS (Yang and Simoes, 2003) suite uses the angle of repose approach, MHYSER (Mahdi, 2009) uses the rotational approach and CONCEPT (Langendoen, 2000) considers the planar approach. However, when coupling mass failure process to hydraulics erosion process there is the time scale issue to be dealt with. In fact, during the modeling process, mass failure occurs instantly which means that the cross-section should be adjusted within one-time step which doesn't reflect what occurs in reality.

CHAPTER 3 PROJECT DESCRIPTION

The present thesis presents the work done to develop a new software for one-dimensional sediment transport in rivers, UMHYSER-1D (Unsteady Model for the HYdraulics of SEdiments in Rivers, one-dimensional).

UMHYSER-1D is a one-dimensional mobile boundary computer model for rivers and manmade canals. It is able to simulate steady or unsteady flows, internal boundary conditions, looped river networks, cohesive and non-cohesive sediment transport, and lateral inflows. UMHYSER-1D can estimate sediment concentrations throughout a river reach given the sediment inflows, bed material, hydrology, and hydraulics of this river reach.

When this project started, the objective was twofold: to develop a software that can be used by researchers to test or implement new findings in fluvial hydraulics and/or computational hydraulics, and use it in river engineering to solve complex, real engineering cases.

To this end, the following steps were taken:

- Start by computing water surface profiles in a single channel or multi-channel looped networks in steady state, then add unsteady flow routing.
- Use of numerical scheme able to handle subcritical, supercritical, and transcritical flows in an unsteady hydraulic simulation.
- Add steady and unsteady sediment transport routings: Transport of non-cohesive sediments and cohesive sediment aggregation, deposition, and erosion.
- Implement thirteen different non-cohesive sediment transport equations that are applicable to a wide range of hydraulic and sediment conditions.
- Allow cross stream variation in hydraulic roughness.
- Implement fractional sediment transport, bed sorting, and armoring.
- Add computation of width changes using theories of minimizations: minimum stream power, minimum energy slope and minimum of bed slope.
- Add point and non-point sources of flow and sediments.

- Add several internal boundary conditions, such as time-stage tables, rating curves, weirs, bridges, and gates.

The main issue during this research project was the validation process. Application of UMHYSER-1D to simple steady state flow cases, from the literature, gave very good results, but for unsteady flow condition, with or without sediment transport, a validation strategy was needed. In fact, a reliable existing model had to be chosen for comparison purposes. This task was not an easy one because the model to be chosen needed to cover most of the proposed outputs of UMHYSER-1D. The US Bureau of Reclamation software SRH-1D was chosen for this end because it includes most of the capabilities UMHYSER-1D and it is developed by a well-known entity with great experience in computational hydraulics. Numerical validation, as presented in the appendices, by using the two software for the same case, gives confidence in the use of UMHYSER-1D. One published laboratory case was used too to test UMHYSER-1D capabilities at small scale (laboratory dimensions).

Finally, two applications of the developed software are completed. The first is modeling the famous Ha!Ha! River flooding during the 1996 Saguenay flood (Quebec, Canada). This is an extreme test case, where the flooding caused major erosion, hence there was debris flow in the river. The idea was to test the ability of UMHYSER-1D to capture the erosional trend. The second application is a real engineering case. After the flooding of the Saguenay region (Quebec, Canada) in July 1996, the retained option to address flood mitigation of the Aux Sables River consisted of increasing the flow by excavating the river. Using UMHYSER-1D, the suspended sediment impact assessment for the Aux Sables River provided the maximum permissible sediment discharge that avoids any pollution risk.

The next chapter presents UMHYSER-1D, its validation and summarizes the two applications. Chapter 5 is dedicated to the Ha!Ha! River flooding modeling, and Chapter 6 covers the Aux Sables River application.

CHAPTER 4 ARTICLE 1: A NEW ONE-DIMENSIONAL NUMERICAL MODEL FOR UNSTEADY HYDRAULICS OF SEDIMENTS IN RIVERS

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4.1 Abstract

This paper presents the model UMHYSER-1D (Unsteady Model for the HYdraulics of SEDiments in Rivers 1-D), a one-dimensional hydromorphodynamic model capable of representing water surface profiles in a single river or a multiriver network, with different flow regimes considering cohesive or non-cohesive sediment transport. It has both steady and unsteady flow and sediment modules. For steady gradually varied flows, UMHYSER-1D uses the standard step method to solve the energy equation and the “NewC” scheme for the de Saint Venant equations. For sediment transport, UMHYSER-1D uses two methods: for long-term simulation, the unsteady terms of the sediment transport continuity equation are ignored, and a non-equilibrium sediment transport method is used. For short-term simulation, the convection-diffusion equation, with a source term arising from sediment erosion/deposition is solved using the fractional step method. The equation without the source term is solved with an implicit finite-volume method, then the equation with source term is solved. Internal boundary conditions, such as time-stage tables, rating curves, weirs, bridges, and gates are simulated. Incorporated is the active layer concept, which allows selective erosion, enabling the simulation of bed armoring. Non-cohesive sediment transport equations and cohesive sediment physical processes are applied to calculate the sediment deposition and erosion. Finally, UMHYSER-1D empirically accounts for bed geometry adjustments by using a relationship between erosion width and flow rate, an angle of repose condition for bank stability and three minimization theories. The presented validation and application cases show UMHYSER-1D’s capabilities and predicts its promising role in solving complex, real engineering cases.

Keywords: One-dimensional model; UMHYSER-1D; Fractional step method; Implicit finite-volume method; Unsteady sediment transport; Bed geometry adjustment; Minimization theories.

4.2 Introduction

Numerical modeling is widely used in river engineering studies. Determining the risk zone caused by floods [1], investigations of river morphology changes [2,3], stream restoration projects and sediment deposition studies [4,5] are some examples of river engineering problems involving numerical modeling. Several numerical models were developed during the last decades, spanning from the simplest ones to more complex solvers. The less complicated ones are the one-dimensional (1D) models. Some do not consider riverbed erosion, such as FLDWAVE [6], by solving unsteady flow equations, while others are developed for mobile riverbeds such as GSTARS [7] or MHYSER [2] using the flow quasi-steadiness hypothesis. Others more complicated, were developed and still under improvement such as the one by El Kadi and Paquier [8], CONCEPTS [5], SRH-1D [9], CCHE1D [10], MIKE11 [11], HEC-RAS [12] or BASEMENT [13]. Even if two-dimensional (2D) models are gaining in popularity, the much longer computation time required for 2D models compared to 1D models, the least amount of field data required by 1D models, the stability of the 1D numerical schemes used to solve the governing equations with the gain in computational efficiency [14], make the use of 1D hydraulic models still needed in river engineering, particularly for applications with long rivers [15].

This paper presents a new 1D model, Unsteady numerical Model for HYdraulics of SEdiments in Rivers, UMHYSER-1D, developed at Polytechnique Montreal and still under improvement. Since its development, UMHYSER-1D went through different validation stages before its application in real engineering cases ([16,17,18]), however its numerical procedures were never published. The authors objectives are to create an efficient tool to solve real engineering problems and to have access to a numerical laboratory to be used to test new scientific findings. In fact, the advantage of developing a new tool resides in the accessibility to the source code which makes it easy to improve the model by adding new research ideas or testing new findings such as a new sediment transport equation.

UMHYSER-1D is a 1-D model developed to simulate flows in rivers and channels with or without movable boundaries. It can compute water surface elevations in complex channel networks. UMHYSER-1D is able to model both steady and unsteady flow conditions, cohesive and non-cohesive sediment transport, and rivers' width changes using minimization theories and riverbank

retreat method. Moreover, hydraulics structures such as weirs, bridges, and gates along with other internal boundary conditions (such as time-stage tables, rating curves) are accounted for.

In steady flow conditions, UMHYSER-1D is identical to MHYSER, Model for HYdraulics of SEdiments in Rivers, developed by Mahdi [2], where the continuity and energy equations are applied, when there are no changes in the flow regime, while the momentum and the continuity equations are used when there are changes from supercritical to subcritical flows. For sediment transport, MHYSER models long-term situations and uses the non-equilibrium sediment transport method of Han [19].

UMHYSER-1D performs five groups of operations including water phase, sediment phase, stream tubes, riverbank stability analysis, and stream power minimization, respectively. The flowchart of the unsteady module of UMHYSER-1D, presented in this paper, is illustrated in Figure 4.1.

The remainder of this article is divided into six sections. In section two, the flow routine is presented, followed by the sediment routing, bed material mixing, and bed geometry adjustment presented in section three. Section four presents the numerical solution procedure for the water and sediment equations. Finally, in section five UMHYSER-1D is applied to an experimental test of a reservoir deposits' erosion following dam removal, followed by the conclusion.

4.3 Unsteady flow routing

For a simple channel or a complex channel network, unsteady UMHYSER-1D solves the de Saint Venant equations, using appropriate boundary conditions.

4.3.1 Unsteady flow equations

For unsteady flows UMHYSER-1D solves the 1D de Saint Venant equations [20]:

$$\begin{cases} B \frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = 0 \\ \frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left(\alpha \frac{Q^2}{A} \right) + gA \frac{\partial Z}{\partial x} + gAS_f = 0 \end{cases} \quad (4.1a)$$

$$(4.1b)$$

where, Q = discharge, B = storage width, A = cross-sectional area, t = time independent variable, x = spatial independent variable, g = gravity acceleration, α = velocity distribution coefficient, Z =

water surface elevation, S_0 = bed slope, S_f = energy slope ($= Q|Q|/K^2$), and K = conveyance.

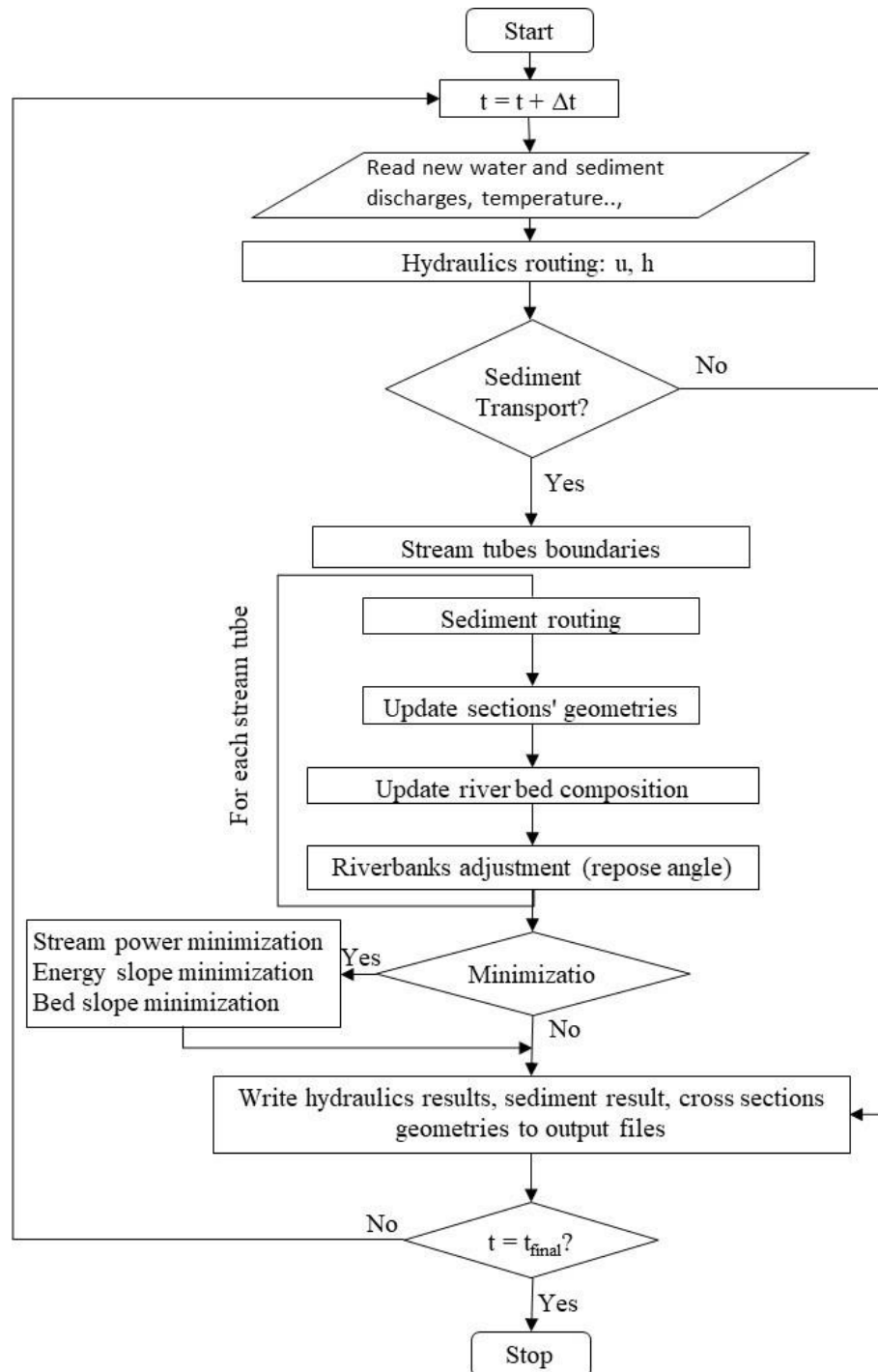


Figure 4-1 Flowchart of UMHYSER-1D

For river networks UMHYSER-1D uses the simultaneous solution procedures described by

Chaudhry [21]. The unknowns are flow depths and flow discharges in each channel. For a river with $N+1$ cross-sections, there are $2(N+1)$ unknowns where N river reaches provide $2N$ equations. To have a unique solution, two boundary conditions provided by rivers' connections are required.

4.3.2 Boundary conditions

While the most used boundary conditions are a known discharge at the upstream boundary and a rating curve at the downstream boundary, in UMHYSER-1D the upstream boundary condition can be either a water discharge or a river stage, and the downstream boundary condition can be a river stage or a rating curve.

For a network, in addition to the upstream and downstream boundary conditions, the continuity equation is applied at each node with no storage allowed, and momentum equation is applied at each junction, imposing the same water level correction to all cross sections associated with the junction.

Hydraulic structures such as bridges, weirs and dams may exist along a river reach. At each internal structure, two unknowns are introduced: discharge and water surface elevation. The continuity equation along with an extra equation depending on the particular structure provides the set of equations to solve for the unknowns. UMHYSER-1D supports 4 types of internal boundary conditions. A stage time series represents a controlled pool or lake and a rating curve represents any structure having a unique relationship between flow rate and water surface elevation. Moreover, weirs and bridges are implemented in the same way as done in [12] and [6] respectively.

4.4 Sediment routing

After the water surface characteristics are calculated, the cross sections are divided into sections of equal conveyance or stream tubes (Figure 4.2).

These stream tubes act as conventional 1-D channels with known hydraulic properties where sediment routing can be carried out within each stream tube almost as if they were independent channels. Once the top widths are determined, the velocities of the stream tubes are calculated by giving a crosswise velocity distribution. Stream tube locations can vary with time. Therefore, although no material can cross stream tube boundaries during a time step, lateral movement of

sediment is described by lateral variations in the stream tube boundaries. For a short-term simulation, the governing equation for sediment transport is a convection-diffusion equation with a source term arising from sediment erosion/deposition.

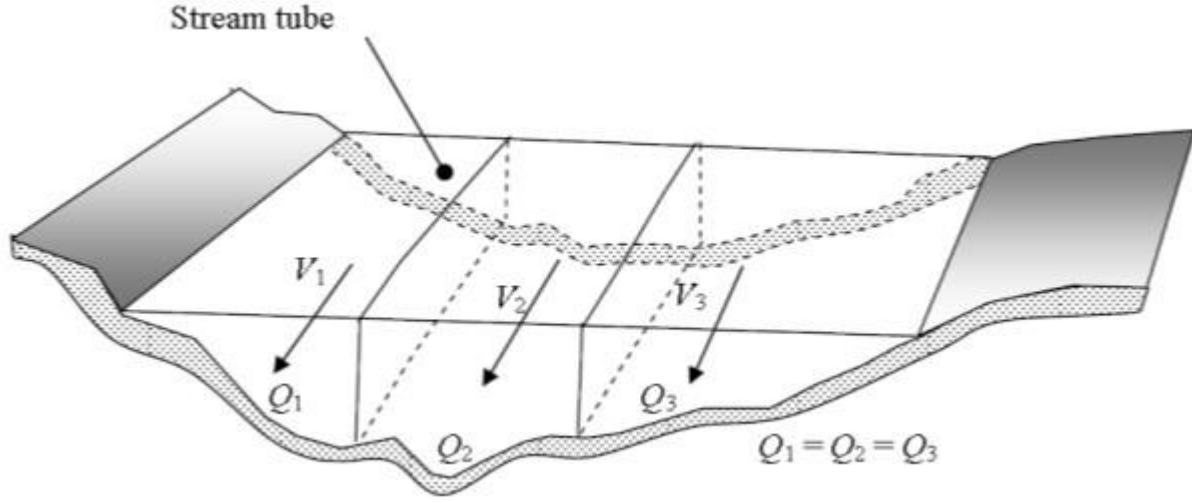


Figure 4-2 Schematic representation of stream tubes [2]

4.4.1 Total load convection-diffusion equation

In the present paper, the 1-D version of the 2-D unsteady total load convection-diffusion equation to model depth-averaged non cohesive sediment transport, developed in [22], automatically switches to suspended load, bed load, or mixed load depending on a transport mode parameter consisting of local flow hydraulics. Moreover, this equation can be applied to multiple size fractions and is generalized to cohesive sediment transport.

The 1-D version of Greimann et al. equation [22], cross-sectional averaged convection-diffusion equation is:

$$\frac{\partial AC}{\partial t} + \frac{\partial \beta QC}{\partial x} = \frac{\partial}{\partial x} \left(AfD \frac{\partial C}{\partial x} \right) + \Gamma \quad (4.2)$$

Where A = cross-sectional area, C = cross-sectional averaged sediment concentration by volume, Q = flow rate, D = longitudinal diffusion coefficient, Γ = source term, x = longitudinal distance and t = time.

To provide model closure, three variables still need to be determined: the transport mode parameter f , the velocity ratio β , and the source term Γ . The f parameter represents the ratio of suspended portion to the total sediment concentration for a single size class; it ranges from 0 for pure bed load to 1 for pure suspended load, while β is the ratio of sediment velocity to flow velocity. Two expressions for β are needed, one for bed load, β_{bed} , and the other for suspension case, β_{sus} . The following expressions for f and β are given in [19]:

$$f = \min(1, 2.5e^{-z}) \quad (4.3)$$

$$\beta_{bed} = \frac{u_*}{V_t} \frac{1.1\phi^{0.17}(1-e^{-5\phi})}{\sqrt{\theta_r}} \quad (4.4a)$$

$$\beta_{sus} = 1 + \frac{u_*}{2\kappa V_t} (1 - e^{2.7z}) \quad (4.4b)$$

$$\beta = \max(\beta_{bed}, \beta_{sus}) \quad (4.5)$$

where $z = \omega_f/(\kappa u_*)$ is the suspension parameter, and $\phi = \theta/\theta_r$, $\theta = \tau_b/[\gamma(s-1)d]$ = Shields parameter (τ_b = bed-shear stress; γ = specific weight of water; d = particle diameter; and s = specific gravity of sediment); and θ_r = reference non-dimensional shear stress, the Shields parameter at which there is a low but measurable reference transport rate, such as defined in [23].

Note that z must be less than 1 when computing β_{sus} , and ϕ must be less than 20 when computing β_{bed} [19]. Further, Equation (4.5), permitting the use of the same Equation (4.2) for simulating bed load and suspended load, avoids the discontinuity in sediment velocity between bed load and suspended load, using Equations (4.4a) and (4.4b). In addition, the longitudinal diffusion coefficient, D , is computed as in [24]:

$$D = K \frac{W^2 U^2}{H u_*} \quad (4.6)$$

where W = channel top width, U = cross sectional velocity, H = average cross-sectional depth, u_* = shear velocity, and K = user specified value ([24] recommend 0.011).

Source term Γ

Non cohesive sediment

For non-cohesive sediment the following source term is used [25,26]:

$$\Gamma = \frac{1}{L_{tot}} (q_{tot}^* - q_{tot})W \quad (4.7)$$

where L_{tot} = total effective adaptation length, W = channel top width, q_{tot}^* = equilibrium capacity for total load transport rate (per unit width), and q_{tot} = actual total load transport rate (per unit width).

UMHYSER-1D offers a choice of 12 equilibrium capacity equations [23, 27-38], widely used in river engineering. Using the sediment recovery factor, α , related to L_{tot} [22]:

$$L_{tot} = (1 - f)L_b + f \frac{uh}{\alpha\omega_s} \quad (4.8a)$$

where $L_b = b_L h$ = adaptation length for bed load, b_L = a calibration coefficient, h = water depth.

L_{tot} might be calculated as [39]

$$L_{tot} = \max(L_b, L_s) \quad (4.8b)$$

with $L_s = \frac{uh}{\alpha\omega_s}$ = adaptation length for suspended load. If bed load and suspended load coexist, L_b is generally smaller than L_s and $L_{tot} = L_s$; in fact, L_b is set to one or two times the grid spacing [40]. But since L_b and L_s (i.e., α) are used as calibration parameters, the difference between and is negligible.

For suspended load ($f=1$), from Eq. (4.8a):

$$\alpha = \frac{uh}{L_{tot}\omega_s} \quad (4.8c)$$

where ω_s = suspended particle fall velocity, Eq. (7) takes the form:

$$\Gamma = \alpha\omega_s(C^* - C)W \quad (4.9a)$$

$$\text{or} \quad \Gamma = (V_E - V_D C)W \quad (4.9b)$$

where C^* = cross-section averaged sediment concentration capacity by volume and C = cross-section averaged sediment concentration by volume, $V_E = \alpha\omega_s C^*$, and $V_D = \alpha\omega_s$.

In the case of multiple size fractions, Eq. (9b) is valid for each material of size k :

$$\Gamma_k = (V_{E,k}p_k - V_{D,k}C_k)W \quad (4.10)$$

where p = percentage of material of size k , C_k = cross-section averaged sediment concentration by volume for material of size k , $V_{E,k} = \alpha\omega_{s,k}C_k^*$, $V_{D,k} = \alpha\omega_{s,k}$, $\omega_{s,k}$ = fall velocity for particle of size class k , and C_k^* = sediment concentration capacity by volume computed if the bed was composed entirely of this k size fraction ($C_{*k} = p_kC_k^*$, sediment concentration capacity by volume of size class k).

The complexity of the sediment recovery factor, α , is well discussed in [39]. Suggested values for α are about 0.25 for strong deposition, and 1 for strong erosion [19,41]. In UMHYSER-1D, b_L and α are user defined calibration parameters and different values of α are used for deposition versus erosion.

Cohesive sediment

Treated as a single sediment class, cohesive sediment is still modeled by Eq. (4.2) but the expression of the source term, Γ , is given by:

$$\Gamma = (V_E p_c - V_D C_c)W \quad (4.11)$$

where V_E and V_D are the velocities of erosion and deposition, respectively, p_c is the percentage of the cohesive sediment in the riverbed, and C_c is the cross-section averaged cohesive sediment concentration by volume.

UMHYSER-1D deposition of cohesive sediment is based on Krone's equation [42], while particle and mass erosion are based on the work of Parthenaides [43] and adapted by Ariathurai and Krone [44]. V_E is defined by:

$$V_E = \begin{cases} 0 & \text{if } \tau \leq \tau_{es} & (4.12a) \\ \frac{P_s}{\rho_s} \left(\frac{\tau - \tau_{es}}{\tau_{em} - \tau_{es}} \right) & \text{if } \tau_{es} \leq \tau \leq \tau_{em} & (4.12b) \\ \frac{P_s}{\rho_s} + \frac{P_m}{\rho_s} \left(\frac{\tau - \tau_{em}}{\tau_{em}} \right) & \text{if } \tau_{em} \leq \tau & (4.12c) \end{cases}$$

P_s = surface erosion constant (kg/m²/s), P_m = mass erosion constant (kg/m²/s), τ = bed shear stress (Pa), ρ_s = sediment density (kg/m³), τ_{es} = critical surface erosion shear stress (Pa), τ_{em} = critical mass erosion shear stress (Pa).

The deposition velocity, V_D , is given by:

$$V_D = \begin{cases} \left(1 - \frac{\tau}{\tau_{df}}\right) \omega & \text{if } \tau \leq \tau_{df} & (4.13a) \\ \left(1 - \frac{\tau}{\tau_{dp}}\right) \omega \left(1 - \frac{C_{eq}}{C_c}\right) & \text{if } \tau_{df} \leq \tau \leq \tau_{dp} \text{ and } C_{eq} \leq C_c & (4.13b) \\ 0 & \text{if } \tau_{dp} \leq \tau \text{ or } C_{eq} \geq C_c & (4.13c) \end{cases}$$

C_c = cross-section averaged sediment concentration by volume, τ = bed shear stress (Pa), C_{eq} = equilibrium cohesive cross-section averaged sediment concentration by volume, τ_{df} = critical shear stress for full deposition (Pa), ω = fall velocity of the cohesive sediment (m/s), and τ_{dp} = critical shear stress for partial deposition (Pa).

4.4.2 Fall velocity

For sediment fall velocity, UMHYSER-1D adopts the approach of the US Bureau of Reclamation [9, 45] for both cohesive and non-cohesive sediments. More specifically, for non-cohesive sediment recommended values of [46] for particle diameter less than 10 mm are adopted. For particle's diameter higher than 10 mm, the following formula is used [45]:

$$\omega = 1.1\sqrt{(s-1)gd} \quad (4.14a)$$

where ω is the fall velocity, g = acceleration due to gravity, s = specific gravity of sediments, and d = particle diameter.

For particles in the silt and clay size ranges, that is, with diameters between 1 and 62.5 μm , the unhindered sediment fall velocity is computed from [45]:

$$\omega = \frac{(s-1)gd^2}{18\nu} \quad (4.14b)$$

Where ν = kinematic viscosity of water. This equation is valid for a user defined value of the concentration, C_1 .

In fact, the fall velocity for cohesive sediments depends on the concentration. UMHYSER-1D defines cohesive fall velocity according to Figure 4.3 [9], where the user defined a set of site specific 4 points data $(C_1, \omega_1, C_2, \omega_2, C_3, \omega_3, C_4, \omega_4)$.

4.4.3 Bed material mixing

UMHYSER-1D uses the method of Bennett and Nordin [47] for the bed composition accounting procedure by dividing the bed into conceptual layers. The top layer, or active layer contains the bed material available for transport, beneath which is the storage layer or inactive layer, and finally the undisturbed bed. The active layer is the most important layer in this procedure. Erosion of a particular size class of bed material is limited by the amount of sediment of this size class present in the active layer.

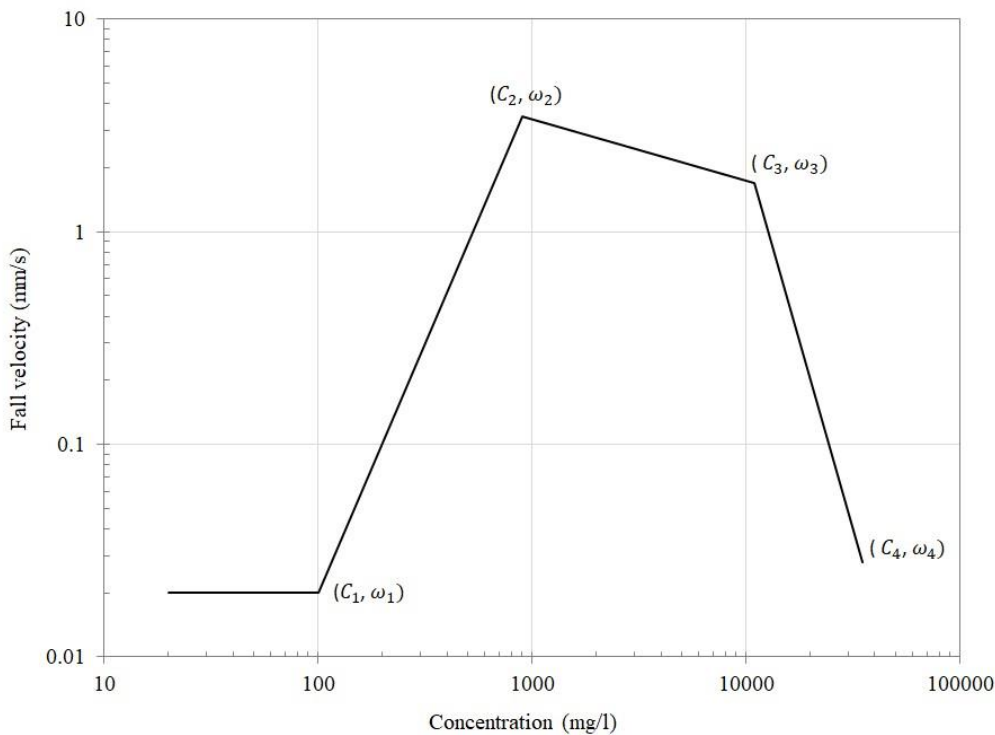


Figure 4-3 Input data illustration for cohesive settling velocity

If the flow carrying capacity for a particular size class is greater than what is available for transport in the active layer, the term availability limited is used [47]. On the other hand, if more material is available than the computed carrying capacity by a sediment transport equation, the term capacity limited is used. At the end of each time step, bed material is calculated in each stream tube. At the beginning of the next time step, after the new locations of the stream tube boundaries are determined, these values are used to compute the new layer thickness and bed composition. Figure 4.4 illustrates this procedure.

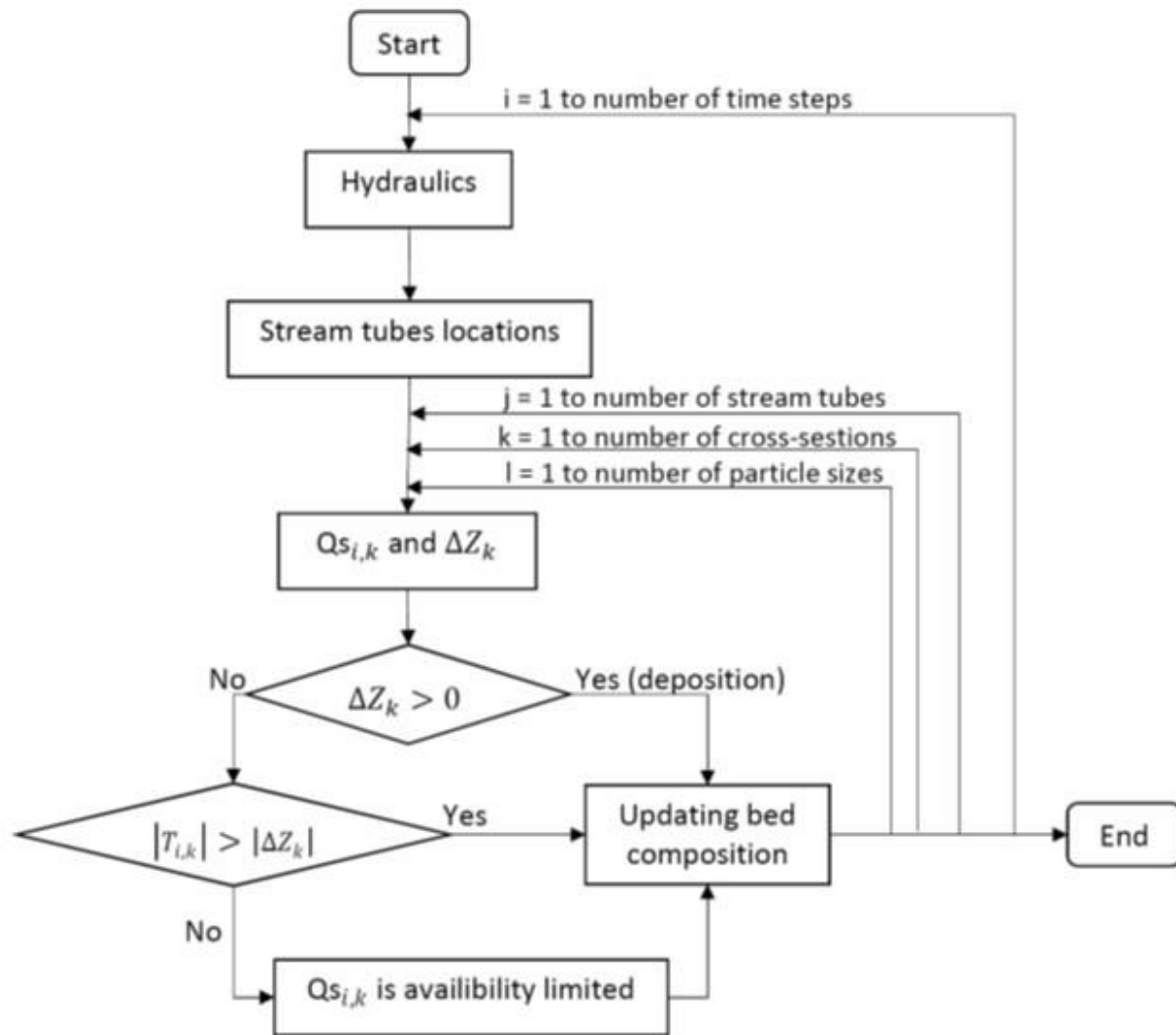


Figure 4-4 Simplified diagram for the bed sorting and armoring processes, adapted from [42]

$Qs_{i,k}$: flow carrying capacity for size class k during time step i , $|\Delta Z_k|$: amount of material in size class k eroded during time step i , and $|T_{i,k}|$: amount of material of size class k present in the active layer, i.e., available for erosion, during time step i

4.4.4 Bed geometry updating and adjustments

By solving mass conservation equation, Eq. (15), the average depth of erosion/deposition is obtained for each size class:

$$(1 - n_k)W \frac{\partial z_{b,k}}{\partial t} = -\Gamma_k \quad (4.15)$$

where n_k = porosity for the k -th size class in the active layer, and $\frac{\partial z_{b,k}}{\partial t}$ = change in the bed elevation, z_b , due to sediment class k . The source term, Γ_k , is given by Eqs. (10) and (11) for non-cohesive and cohesive sediment respectively. The total depth of deposition/erosion in a cross-section, Δz_b , is the sum of all the bed changes of the different size classes, $\Delta z_{b,k}$.

The erosion width is an important parameter in estimating the erosion details. Because 1-D models do not have a shear stress that varies across a cross-section, it is difficult to estimate the non-uniform erosion that occurs during incision. Since UMHYSER-1D is a 1-D model, it does not directly simulate lateral transport of sediment; however, it empirically accounts for the processes involved by using a relationship between erosion width and flow rate, and an angle of repose condition for bank stability. The erosion width, centered at the centroid of the cross section is given by:

$$Wer = aQ^b \quad (4.16)$$

where Wer = erosion width, Q = stream flow, and a and b are user defined constants.

During erosion progression, the steepness of bank slope is limited by the above and under water values of the angle of repose. At the end of each time step, if vertical or horizontal adjustments have caused the bank to become too steep, the two points adjacent to the segments of the banks are adjusted vertically until the slopes equal the critical slopes. The material taken from the banks will be added during the next time step as a lateral sediment discharge.

Moreover, as an option UMHYSER-1D offers the choice of 3 minimization theories for the determination of depth and width adjustments, at a given time step: minimization of the total stream power [48], minimization of the energy slope [49] and minimization of the bed slope (or conveyance maximization).

According to the minimization of total stream power theory, if lower total stream power is the result of alteration of the channel widths, then channel adjustments are made in the lateral direction. Otherwise, the adjustments progress in the vertical direction.

While adopting the minimization of energy slope, if the energy slope at a cross-section is greater than the weighted average energy slope of its adjacent sections, then the channel width at this section is reduced during deposition or the depth is increased during erosion. However, if the energy slope is smaller the channel depth at this section is decreased during deposition or an increase of width occurs during erosion.

Finally, for the minimization of bed slope, if the bed slope at a cross-section is greater than the weighted average bed slope of its adjacent cross-sections, then the channel width at this section is reduced during deposition, or the depth is increased during erosion. Otherwise, the channel depth at this section is decreased during deposition or the width is increased during erosion.

4.5 Numerical solution procedure

In the case of steady flow, for backwater computations, the standard step method is used [50]. Computations proceed in the upstream direction for subcritical flows and in the downstream direction for supercritical flows. The Exner equation is then solved for bed updating [2].

For unsteady flow conditions, the set of PDEs, Eqs. (4.1a), (4.1b) and (4.2), are solved using a decoupled approach. First, for the liquid phase, the de Saint Venant equations are solved using the NewC numerical scheme [51], which assures numerical stability in the transition between different flow regimes, then the solid phase equation is solved using the fractional step method [52].

4.5.1 De Saint Venant equations

Eqs. (4.1a) and (4.1b) are solved using the NewC scheme [51] which uses a staggered grid, where the computational points for flow, Q , are located at the cross-sections, with a weighting implicit factor θ in the time dimension ($0 \leq \psi \leq 1$), and Z points are located halfway between the cross-sections (Figure 4.5).

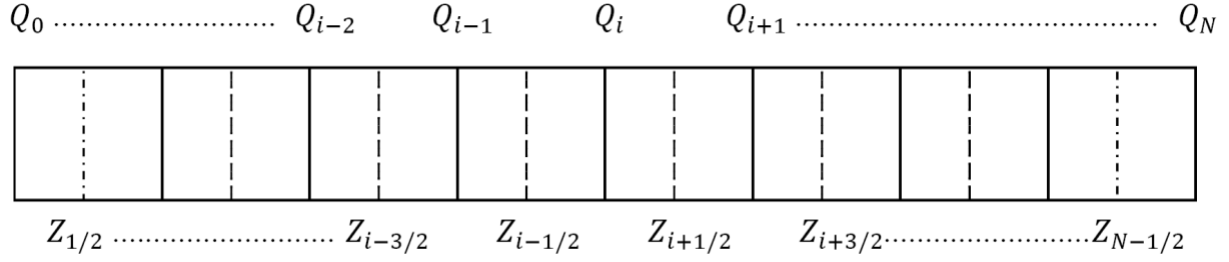


Figure 4-5 Discretization grid for NewC numerical scheme

Application of the modified NewC scheme leads to the following set of equations:

$$a_i Q_{i-1}^{n+1} + b_i Q_i^{n+1} + c_i Q_{i+1}^{n+1} = d_i \quad (4.17a)$$

$$Z_{i+1/2}^{n+1} = \varepsilon_{i+1/2} Q_i^{n+1} + \beta_{i+1/2} Q_{i+1}^{n+1} + \gamma_{i+1/2} \quad (4.17b)$$

All the details are given in [46], but for completeness, the expressions of the coefficients in Eqs. (4.17a) and (4.17b) are recalled here:

$$a_i = -\frac{\alpha\psi\Delta t}{\Delta x} \frac{Q_i^{n+\frac{1}{2}}}{A_i^{n+\frac{1}{2}}} - g \frac{\psi\Delta t}{\Delta x} A_i^{n+\frac{1}{2}} \varepsilon_{i-1/2}^{n+\frac{1}{2}} \quad (4.18a)$$

$$b_i = 1 + g\Delta t \frac{A_i^{n+\frac{1}{2}}}{K_i^{n+\frac{1}{2}}} |Q_i^n| + g \frac{\psi\Delta t}{\Delta x} A_i^{n+\frac{1}{2}} (\varepsilon_{i+1/2}^{n+1} - \beta_{i-1/2}^{n+1}) \quad (4.18b)$$

$$c_i = \frac{\alpha\psi\Delta t}{\Delta x} \frac{Q_i^{n+\frac{1}{2}}}{A_i^{n+\frac{1}{2}}} + g \frac{\psi\Delta t}{\Delta x} A_i^{n+\frac{1}{2}} \beta_{i+1/2}^{n+1} \quad (4.18c)$$

$$d_i = Q_i^n - gA_i^{n+\frac{1}{2}} \frac{\Delta t}{\Delta x} \left[\psi \left(\beta_{i+\frac{1}{2}}^{n+1} - \beta_{i-\frac{1}{2}}^{n+1} \right) + (1-\psi) \left(Z_{i+\frac{1}{2}}^n - Z_{i-\frac{1}{2}}^n \right) \right] + \left[-\frac{\alpha(1-\psi)\Delta t}{\Delta x} \frac{Q_i^{n+\frac{1}{2}}}{A_i^{n+\frac{1}{2}}} (Q_{i+1}^n - Q_{i-1}^n) \right] \quad (4.18d)$$

$$\varepsilon_{i+1/2} = -\beta_{i+\frac{1}{2}} = \frac{\psi\Delta t}{\Delta x B_{i+1/2}^{n+1/2}} \quad (4.18e)$$

$$\gamma_{i+1/2} = h_{i+1/2}^n - \frac{(1-\psi)\Delta t}{\Delta x B_{i+1/2}^{n+1/2}} (Q_{i+1}^n - Q_i^n) \quad (4.18f)$$

Written for all $i = 0, N$; the system of equations, Eq. (17a), with an upstream and a downstream boundary conditions, is solved using the double sweep algorithm [53, 54]. Then, for each i , Eq. (17b) provides the water surface elevations.

The numerical scheme is able to model sub-, super- and trans-critical flow conditions, but even if linearized stability analysis using Fourier series expansions [55] shows that the numerical scheme is always stable for $(0.5 < \psi \leq 1)$ [51], it has been noticed that, for Froude numbers of more than 1.5, flows start to show some wiggles that might grow into instability. As a remedy, the Local Partial Inertia [6] consisting of multiplying the acceleration term of equation (1b) by $\max(0, 1 - F^n)$, where F is Froude number and n is a user specified integer, is applied. Another option consists of introducing some numerical diffusion (for example, for $\psi = 0.7$) to smooth the solution [51]. Moreover, as by its construction, the NewC scheme has some limitations in solving for supercritical flow. In fact, it requires one boundary condition at each end of the domain. Hence, supercritical flow should not occur at the upstream or downstream boundaries of any modeled river reach.

4.5.2 Convection-diffusion equation

The convection-diffusion, Eq. (4.2), is solved using the fractional step method [52]. Solving Eq. (4.2) is equivalent to solving the following equations:

$$\frac{\partial AC}{\partial t} + \frac{\partial \beta QC}{\partial x} = \frac{\partial}{\partial x} \left(AfD \frac{\partial C}{\partial x} \right) \quad (4.19a)$$

$$\frac{\partial AC}{\partial t} = \Gamma \quad (4.19b)$$

Eq. (4.19a) is solved, using a finite volume method, to obtain intermediate solution, AC , then the initial value problem in Eq. (4.19b) is solved to obtain the solution at the next time step. This section details the solution of Eq. (4.19a).

Using the finite volume method, the stream is discretized into cells centred on the cross sections (Figure 4.6), and Eq. (4.19a) is transformed to an integral one to conserve mass:

$$\frac{\partial}{\partial t} \int_x AC dx + \sum(\beta QC) = \sum \left(AfD \frac{\partial C}{\partial x} \right) \quad (4.19c)$$

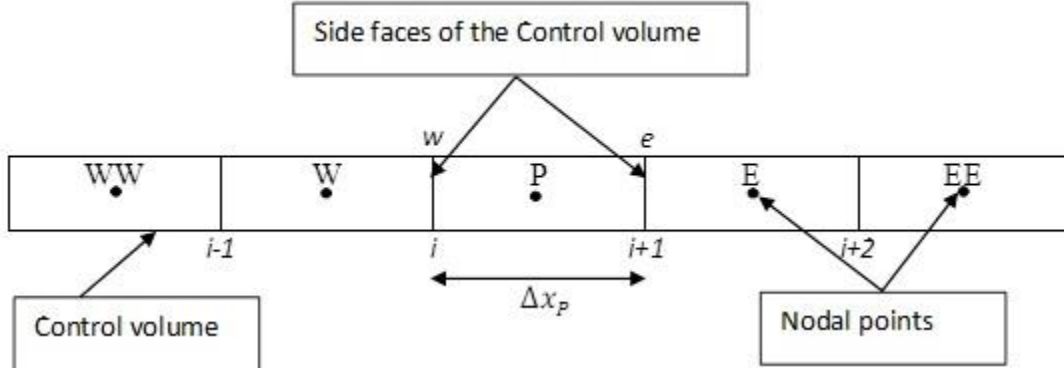


Figure 4-6 Discretization grid for convection-diffusion equation

Unsteady term

Using the explicit Euler method, the unsteady term is discretized as:

$$\frac{\partial}{\partial t} \int_x AC dx = \frac{C_P^{n+1} - C_P^n}{\Delta t} A_P \Delta x_P = a_P^u C_P^{n+1} - a_P^u C_P^n \quad (4.20a)$$

where n and $n+1$ = current and next time steps respectively. The subscript P indicates where the terms are approximated, at the center of the control volume, and:

$$a_P^u = \frac{A_P \Delta x_P}{\Delta t} \quad (4.20b)$$

Convective term

Following Pletcher et al. ([56], pp 182-184), the convective term:

$$\Sigma(\beta QC) = (\beta QC)_e - (\beta QC)_w \quad (4.21)$$

is discretized using a Lax-Wendroff TVD method:

$$(\beta QC)_e = \frac{1}{2} \{ (\beta Q)_e [C_P + C_E] - Y_e [C_E - C_P] \} \quad (4.22)$$

with $Y_e = |(\beta Q)_e| \left[1 - \varphi \left(1 - \frac{\Delta t}{A_P \Delta x_P} |(\beta Q)_e| \right) \right]$, where φ is the flux limiter. In the literature, different limiters exist with the same performance [57]. In this paper, the limiter function of Van Leer [58] is used:

$$\varphi = \frac{r + |r|}{1 + r}; r = \begin{cases} \frac{C_E - C_P}{C_P - C_W} & \text{if } (\beta Q)_e > 0 \\ \frac{C_P - C_E}{C_E - C_{EE}} & \text{if } (\beta Q)_e < 0 \end{cases} \quad (4.23)$$

To get second order accuracy in time, the Crank-Nicolson method is used:

$$C = \psi C^{n+1} + (1 - \psi)C^n = C^n + \psi(C^{n+1} - C^n) = C^n + \psi\Delta C \quad (4.24)$$

where θ = implicit factor ($0 \leq \psi \leq 1$). Equation (22) takes the final form:

$$(\beta Q C)_e = \frac{\psi}{2}[(\beta Q)_e + Y_e]C_P^{n+1} + \frac{(1-\psi)}{2}[(\beta Q)_e + Y_e]C_P^n + \frac{\psi}{2}[(\beta Q)_e - Y_e]C_E^{n+1} + \frac{(1-\psi)}{2}[(\beta Q)_e - Y_e]C_E^n \quad (4.25)$$

and the convective term, equation (21) can be written as:

$$\Sigma(\beta Q C) = a_P^c C_P^{n+1} + a_E^c C_E^{n+1} + a_W^c C_W^{n+1} + \frac{1-\psi}{\psi} (a_P^c C_P^n + a_E^c C_E^n + a_W^c C_W^n) \quad (4.26)$$

where

$$a_P^c = \frac{\psi}{2}[(\beta Q)_e + Y_e] - \frac{\psi}{2}[(\beta Q)_w - Y_w] \quad (4.27a)$$

$$a_E^c = \frac{\psi}{2}[(\beta Q)_e - Y_e] \quad (4.27b)$$

$$a_W^c = -\frac{\psi}{2}[(\beta Q)_w + Y_w] \quad (4.27c)$$

Diffusive term

To get a second-order accurate discretization in both space and time, the diffusive term is approximated using the central differential scheme in space [56]:

$$\Sigma\left(AfD \frac{\partial C}{\partial x}\right) = (Af)_e D_e \frac{C_E - C_P}{\Delta x_e} - (Af)_w D_w \frac{C_P - C_W}{\Delta x_w} \quad (4.28)$$

then, followed by the Crank-Nicolson method in time, using equation (22):

$$\Sigma\left(AfD \frac{\partial C}{\partial x}\right) = -a_P^d C_P^{n+1} - a_E^d C_E^{n+1} - a_W^d C_W^{n+1} - \frac{(1-\psi)}{\psi} [a_P^d C_P^n + a_E^d C_E^n + a_W^d C_W^n] \quad (4.29)$$

where:

$$a_E^d = -\psi \frac{(Af)_e D_e}{\Delta x_e} \quad (4.30a)$$

$$a_W^d = -\psi \frac{(Af)_w D_w}{\Delta x_w} \quad (4.30b)$$

$$a_P^d = -(a_E^d + a_W^d) \quad (4.30c)$$

Integral equation

Using Eqs. (4.20a), (4.26) and (4.29), the integral equation (4.19c) takes the form:

$$a_P C_P^{n+1} + a_E C_E^{n+1} + a_W C_W^{n+1} = -\frac{(1-\psi)}{\psi} [a_P C_P^n + a_E C_E^n + a_W C_W^n] \quad (4.31)$$

where, using equations (20b), (27abc) and (30abc):

$$a_P = a_P^u + a_P^c + a_P^d \quad (32a)$$

$$a_E = a_E^c + a_E^d \quad (32b)$$

$$a_W = a_W^c + a_W^d \quad (32c)$$

The solution, C_i^{n+1} , at every cross-section i , is the intermediate solution to be used in Eq. (4.18b) which is solved analytically, or using Runge-Kutta 4th order method [59] to obtain the new solution $C_{i,*}^{n+1}$ at the next time step.

4.5.3 Bed elevation change

Finally, combining Eq. (19b) and mass conservation, Eq. (15), the average depth of deposition/erosion for each size fraction k , at a cross section i , is calculated as:

$$\Delta z_{b,k} = -\frac{(C_{i,*}^{n+1} - C_i^{n+1})A_i}{(1-n_k)W} \quad (4.33)$$

where: $\Delta z_{b,k}$ = bed elevation change due to sediment class k , n_k = porosity for the k -th size class in the active layer, A_i = Cross-sectional area and W = channel top width.

The sediment transport is computed for each individual sediment size fraction within each stream tube then the total bed changes, Δz_b , are computed as a sum of the bed changes due to each particle size, $\Delta z_{b,k}$.

Even if the NewC scheme is unconditionally stable for $0.5 < \psi \leq 1$, to ensure the validity of the uncoupled approach used by UMHYSER-1D, numerical experimentation is required to determine a suitable time step to be used.

4.6 Application

Cantelli et al. [60] performed experiments, of 1.5 hours duration, at the University of Minnesota to simulate dam removal. A flume of a width of 0.61 m and a slope of 0.018 was filled with uniform coarse sand, of $d_{50} = 0.8$ mm, to replicate the sediment deposit behind a dam. The maximum depth of the sediment layer is 0.12 m. To ensure that the erosion occurred in the middle of the flume, a channel 1 cm deep and 27.5 cm wide was cut in the middle of the deposit. Constant water flow rate of 0.3 l/s and sediment rate of 0.002 kg/s ensure an equilibrium supply rate at the given slope and width. During bed erosion, precise measurement of the bed profile of the dam were done, but there were no cross-sectional data available to compare the results. Because the vertical erosion in the middle of the channel was faster than the additional sediment supply from the banks, Cantelli et al. [60] observed a rapid narrowing of the channel followed by a gradual widening.

Model inputs

In UMHYSER-1D, the points representing each cross-section are spaced 1 cm and the cross-sections are spaced 10 cm apart. Several simulations were performed to achieve the “best” one. For the base case simulation, Manning’s roughness coefficient is assumed equal to 0.025, Parker sediment transport capacity equation is used, and the angles of repose used for bank failure modeling are set to 70 degrees for the angle of repose above water, and to 45 degrees for the angle of repose below water. Moreover, the erosional width is assumed to be 24 cm, and the default value of the non-dimensional critical shear stress needed to use Parker’s equation is used ($\theta_c = 0.0386$) along with the adaptation length for bed load, $L_b = 10$ cm. Table 1 summarizes the main input data for the base case and for the 23 simulations performed by changing one parameter at a time.

Model Results:

To find the best data set that gave the best results (closest simulated longitudinal profile to the observed one), sensitivity analysis is performed. Figures 4.7 to 4.14 show the effects of sediment transport capacity equation, angles of repose, erosional width, non-dimensional critical shear stress, minimization theories, adaptation length for bed load, diffusive wave approximation, and Manning coefficient respectively. The observed final longitudinal profile is compared to the simulated ones at time 1.5 hours.

Table 4-1 Sensitivity analyses' input data sets

Simulation	Flow Routing	Sediment Equation	θ_{c1}	Angle of repose (°)		Wer ₂ (cm)	Lb ₃ (cm)	Minimization	Manning
				Above water	Bellow water				
Base run	Dynamic	Parker ₄	0.0386	70	45	24	10	-	0.025
Sim 1	Dynamic	MPM ₅	-	70	45	24	10	-	0.025
Sim 2	Diffusive	Parker	0.0386	70	45	24	10	-	0.025
Sim 3	Dynamic	MPM-WP ₆	-	70	45	24	10	-	0.025
Sim 4	Dynamic	Parker-SSC ₇	0.0386	70	45	24	10	-	0.025
Sim 5	Dynamic	Parker	0.03	70	45	24	10	-	0.025
Sim 6	Dynamic	Parker	0.045	70	45	24	10	-	0.025
Sim 7	Dynamic	Parker	0.0386	70	40	24	10	-	0.025
Sim 8	Dynamic	Parker	0.0386	70	30	24	10	-	0.025
Sim 9	Dynamic	Parker	0.0386	70	20	24	10	-	0.025
Sim 10	Dynamic	Parker	0.0386	40	40	24	10	-	0.025
Sim 11	Dynamic	Parker	0.0386	70	45	24	00	-	0.025
Sim 12	Dynamic	Parker	0.0386	70	45	24	20	-	0.025
Sim 13	Dynamic	Parker	0.0386	70	45	16	10	-	0.025
Sim 14	Dynamic	Parker	0.0386	70	45	22	10	-	0.025
Sim 15	Dynamic	Parker	0.0386	70	45	20	10	-	0.025
Sim 16	Dynamic	Parker	0.0386	70	45	30	10	-	0.025
Sim 17	Dynamic	Parker	0.0386	70	45	40	10	-	0.025
Sim 18	Dynamic	Parker	0.0386	70	45	24	10	TSP ₈	0.025
Sim 19	Dynamic	Parker	0.0386	70	45	24	10	ES ₉	0.025
Sim 20	Dynamic	Parker	0.0386	70	45	24	10	BS ₁₀	0.025
Sim 21	Diffusive	Parker	0.045	70	30	30	10	TSP ₈	0.025
Sim 22	Diffusive	Parker	0.045	70	30	30	00	TSP ₈	0.022
Sim 23	Diffusive	Parker	0.045	70	30	30	00	TSP ₈	0.028

1: non-dimensional critical shear stress needed to use Parker's equation [23]; 2: Erosion width; 3: adaptation length for bed load; 4: Meyer Peter and Muller [27]; 5: Parker's equation [23]; 6: Meyer Peter and Muller corrected by Wong and Parker [28]; 7: Parker's equation with shear stress correction [23]; 8: Total Stream Power minimization; 9: Energy Slope minimization; 10: Bed Slope minimization

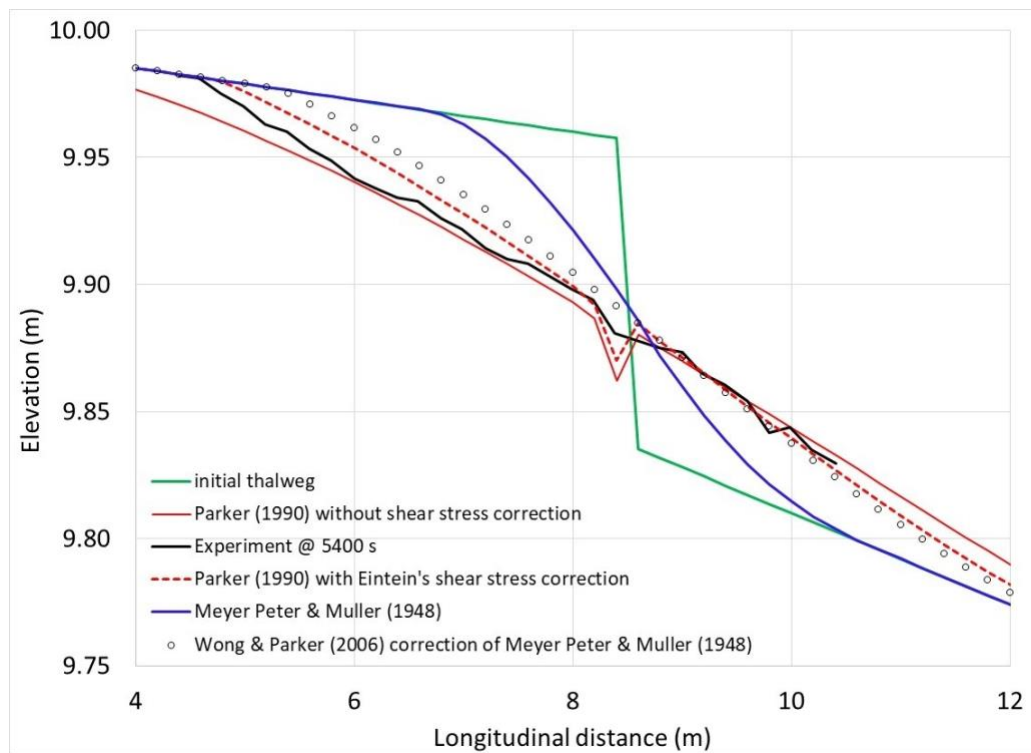


Figure 4-7 Sensitivity analysis: Effects of sediment transport capacity equation on longitudinal profile evolution

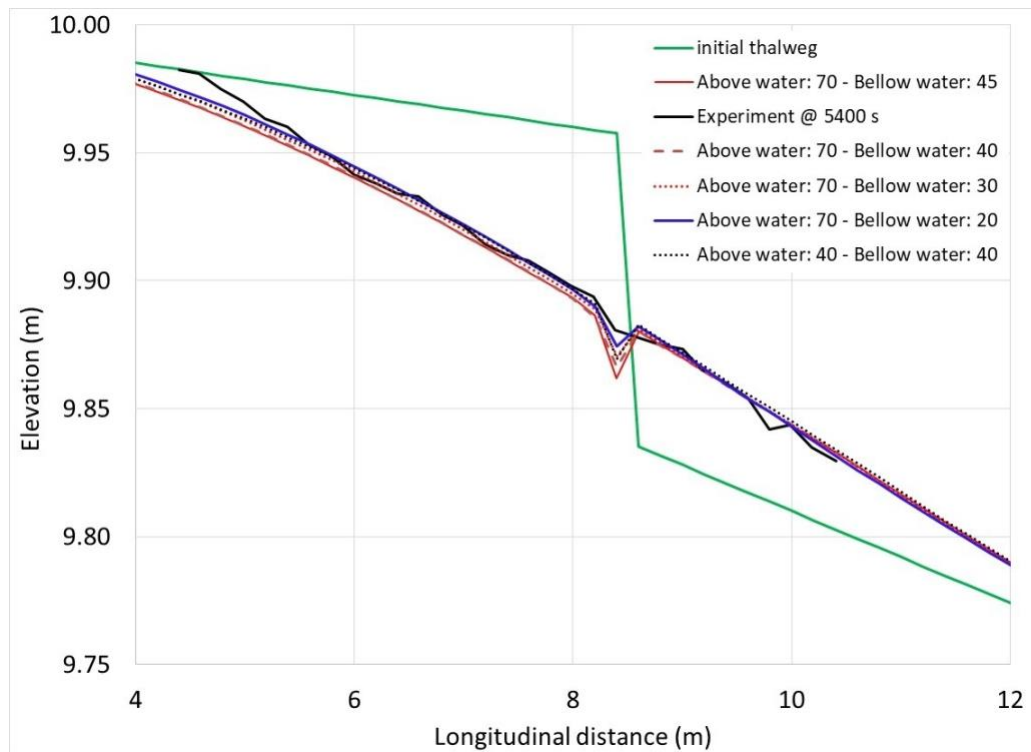


Figure 4-8 Sensitivity analysis: Effects of angles of repose on longitudinal profile evolution

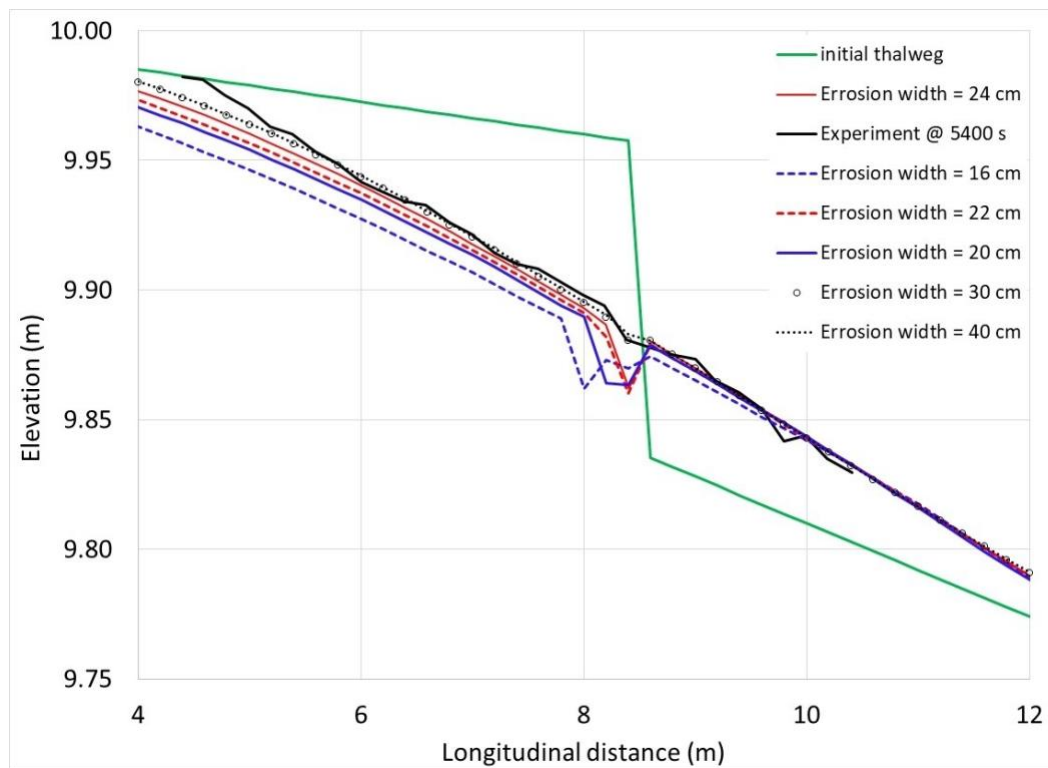


Figure 4-9 Sensitivity analysis: Effects of erosional width on longitudinal profile evolution

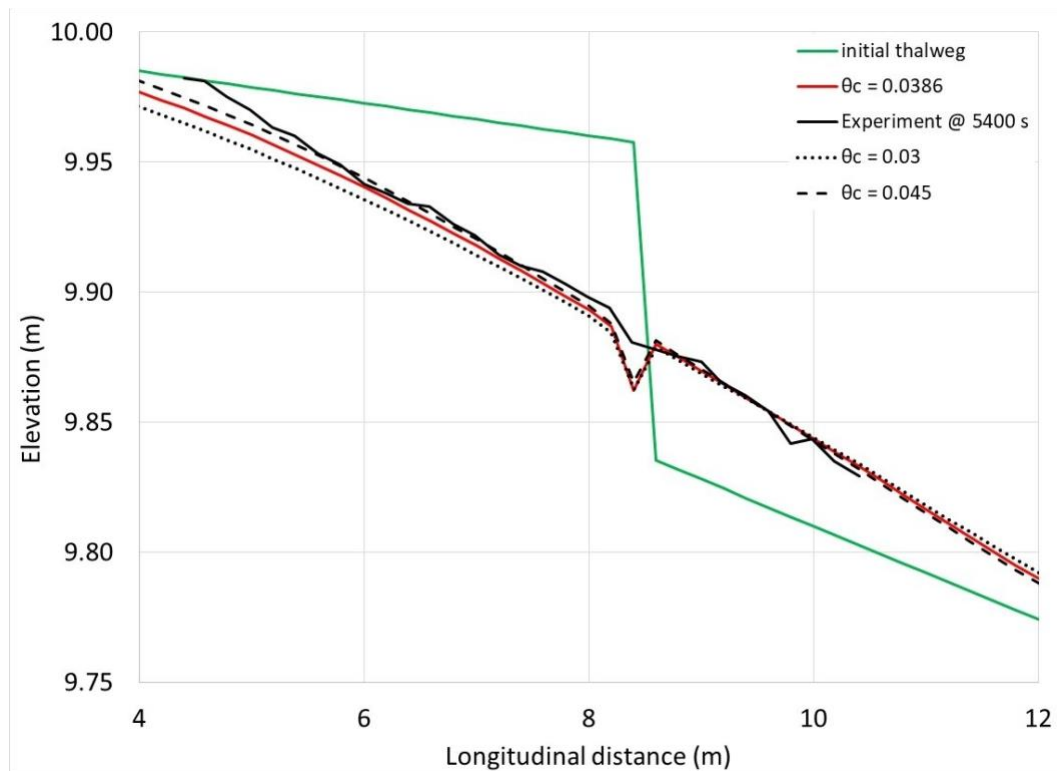


Figure 4-10 Sensitivity analysis: Effects of non-dimensional critical shear stress on longitudinal profile evolution

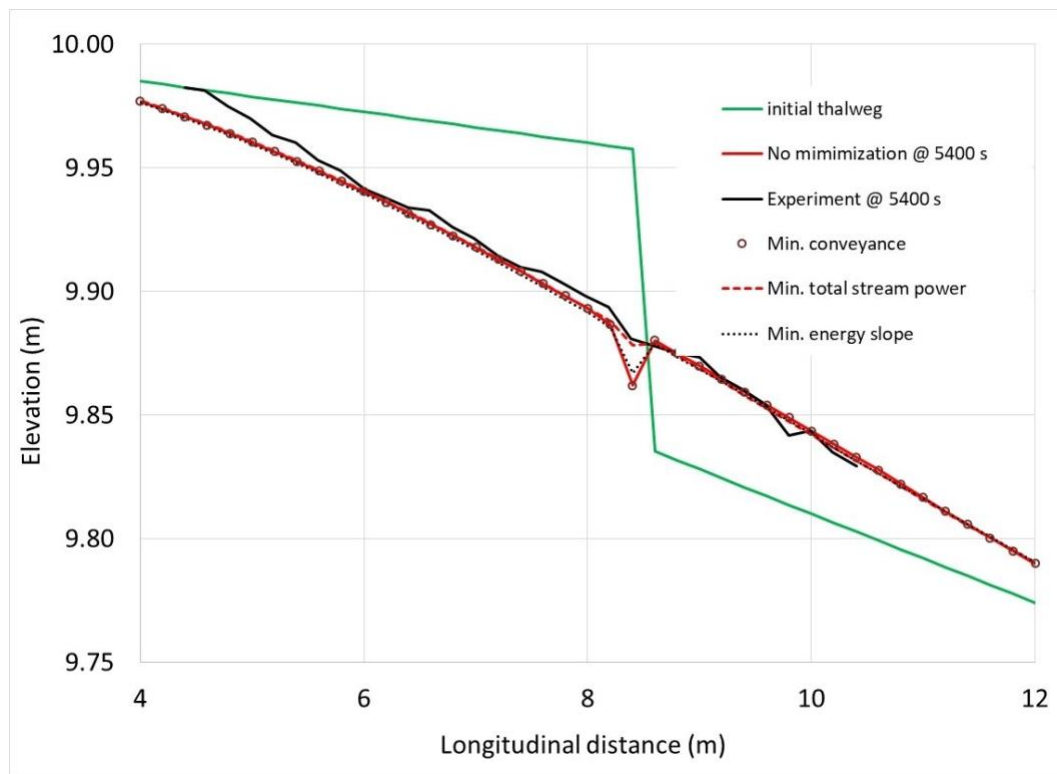


Figure 4-11 Sensitivity analysis: Effects of the minimization theories on longitudinal profile evolution

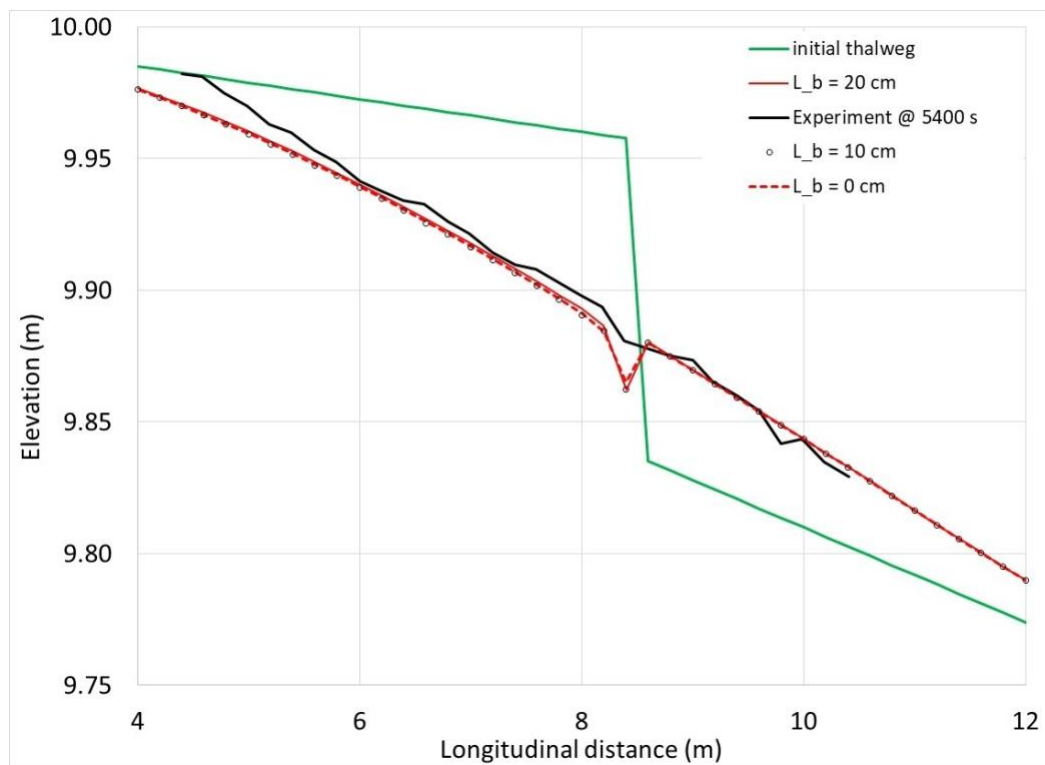


Figure 4-12 Sensitivity analysis: Effects of adaptation length on longitudinal profile evolution

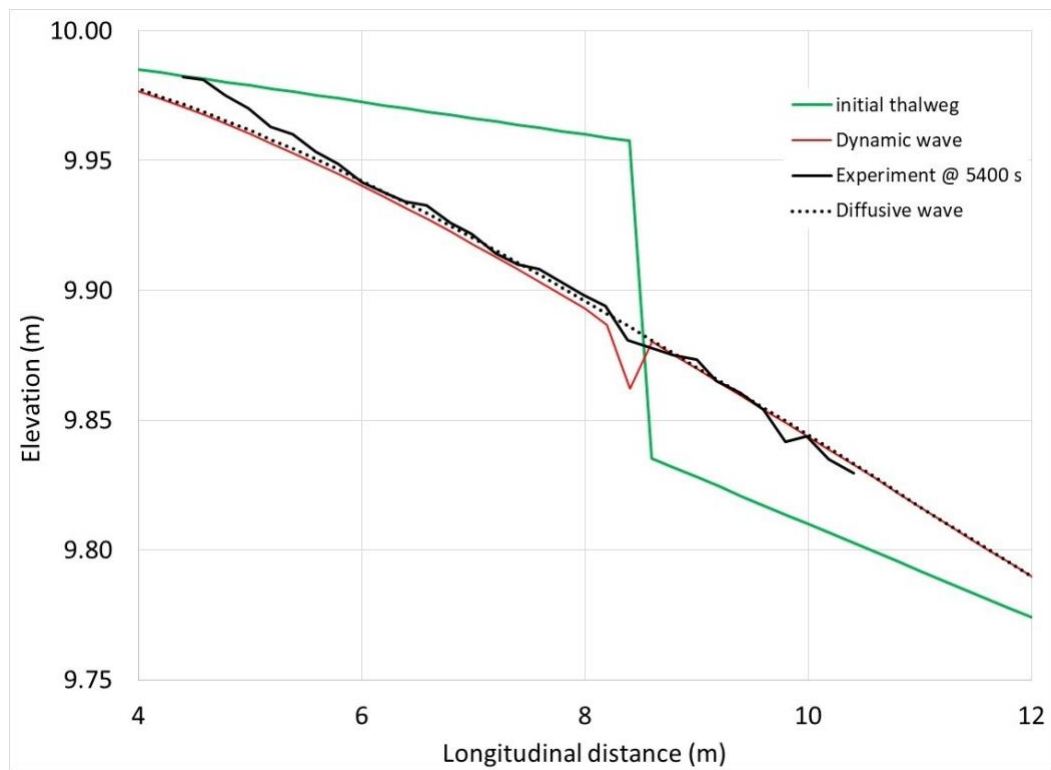


Figure 4-13 Sensitivity analysis: Effects of the diffusive wave approximation on longitudinal profile evolution

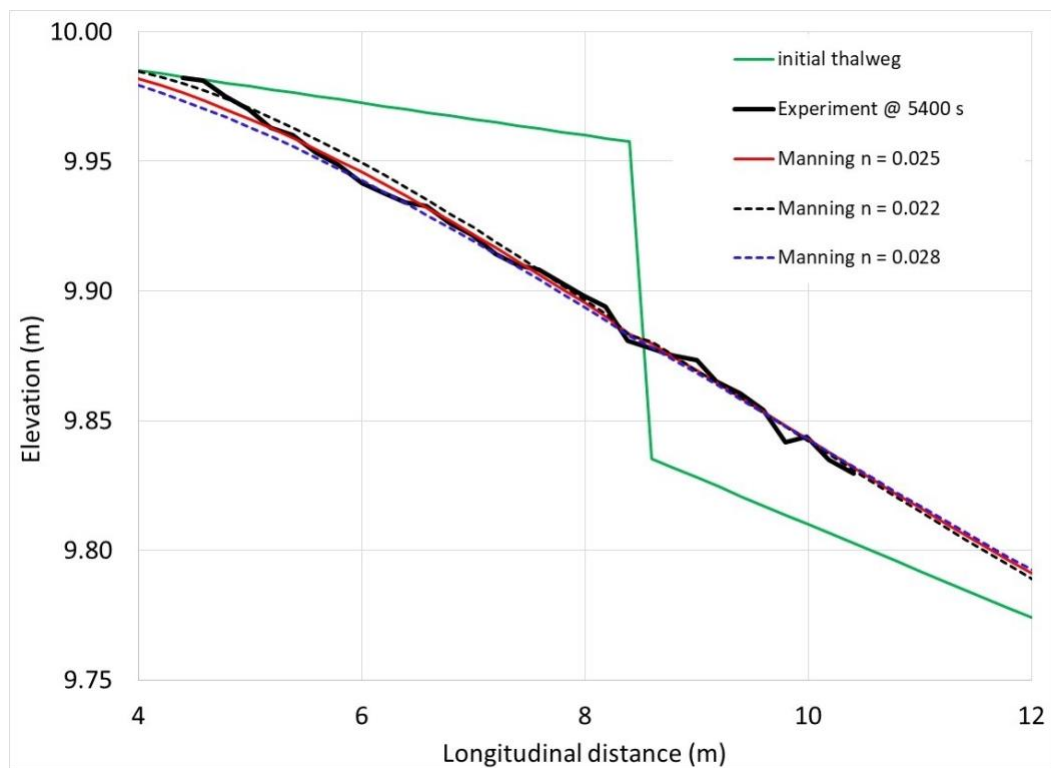


Figure 4-14 Sensitivity analysis: Effects of Manning coefficient on longitudinal profile evolution

The Manning coefficient of 0.025 offers better results while Parker's bedload transport equation, developed for gravel bed sediment, presents the best results even if the sediments in this experiment is coarse sand. It is not surprising that Parker's [23] equation performs well since the used sediments were transported mainly in bedload mode. The non-dimensional critical shear stress needed to use Parker's equation was assigned a value of $\theta_c = 0.03$. Note that the value of hiding factor, α , is not important because only a single size class is being simulated. The angles of repose used for bank failure modeling are set to 70 degrees for the angle of repose above water, and to 30 degrees for the angle of repose below water. The adaptation length for bed load non-equilibrium sediment transport have little impact on bed evolution. This is mainly because the max water depth (less than 1 cm) is smaller than the distance between two cross-sections (10 cm). In this experiment, equilibrium sediment transport prevailed. Finally, erosional width is used to empirically accounts, via Eq. (16), for the observed rapid narrowing of the channel followed by a gradual widening. Better results are achieved using an erosional width of 30 cm. If the minimization of bed slope and energy slope show practically no improvement over the initial simulation, the minimization of total stream power improved significantly the bed profile at the knickpoint. Finally, the diffusive wave equation approximation produces better results than the full dynamic wave equation by smoothing the bed waves at the knickpoint.

Figure 4.15 shows the results of the best simulation for the best input data corresponding to simulation Sim 21. The simulated results are compared to the experiment results of Run 6 of [60]. Overall, the agreement between the measured and simulated longitudinal profiles is very satisfactory. For cross-sections changes, Figure 4.16 shows the example of the cross-section corresponding to the knickpoint. Note that there were no cross-sectional data to compare the model against, but Cantelli et al. [60] observed a rapid narrowing of the channel followed by a gradual widening. UMHYSER-1D cannot model the observed cross-section evolution since it does not directly simulate lateral transport of sediments. In fact, it cannot model the rapid narrowing of the channel followed by a gradual widening.

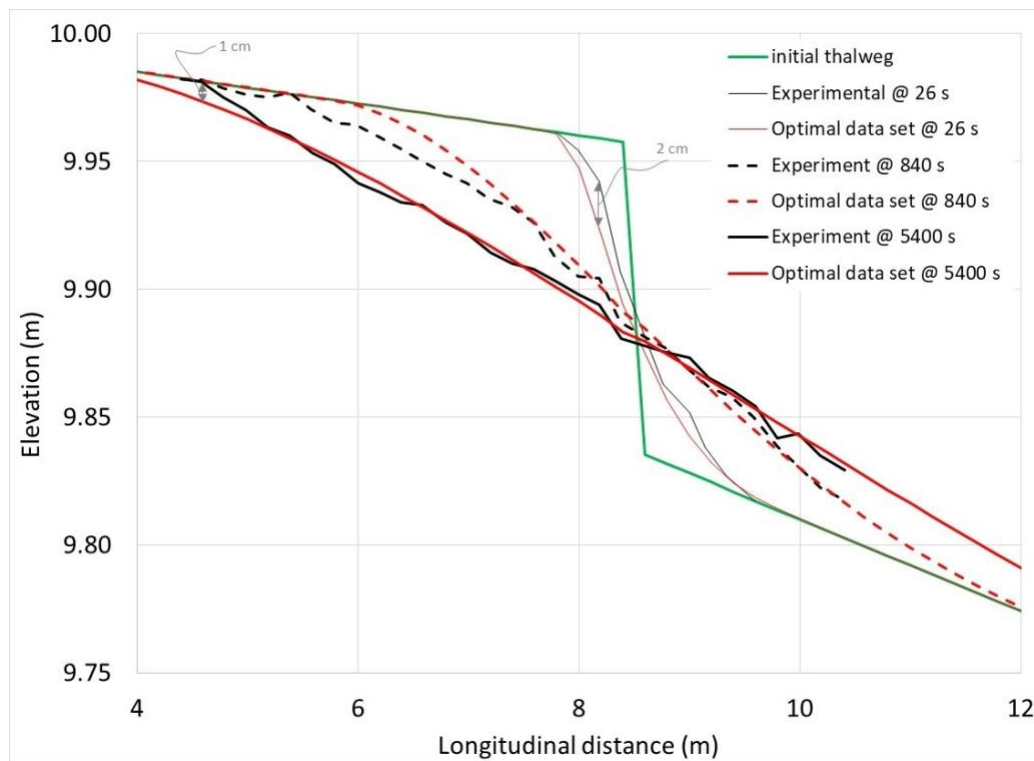


Figure 4-15 Simulated and observed Longitudinal profiles' comparison between Run 6 of [59] and UMHYSER-1D optimum input data set

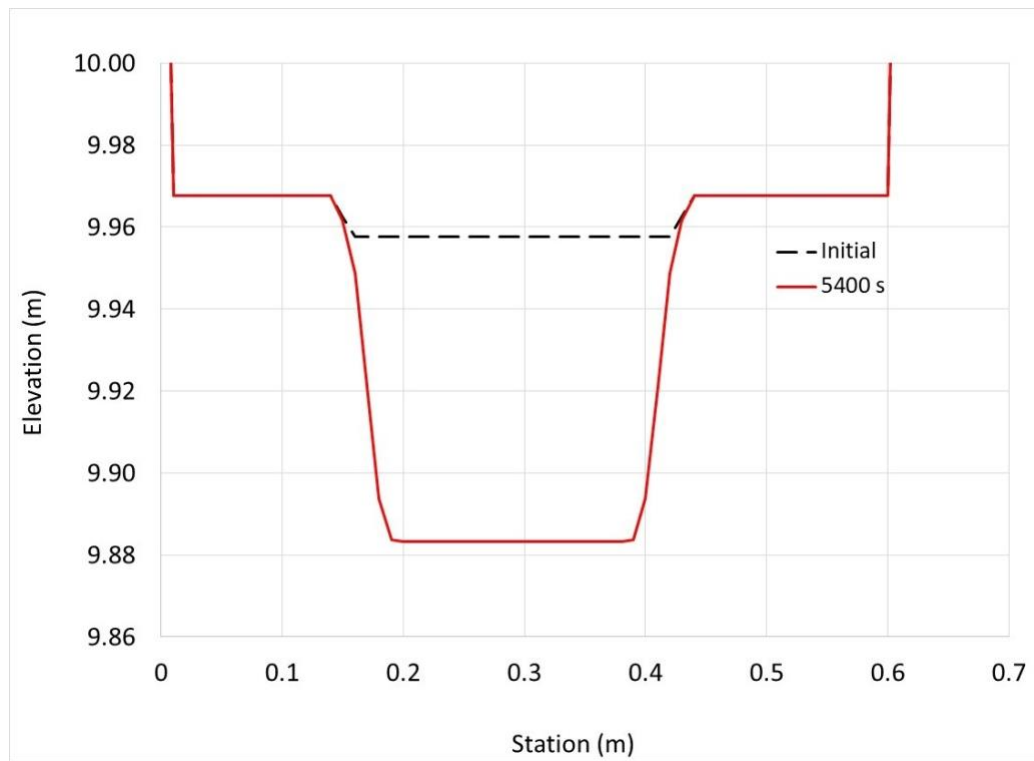


Figure 4-16 Example of transverse changes: cross-section corresponding to the knickpoint (Sim 21)

4.7 Conclusion

This paper presents UMHYSER-1D, a newly developed 1-D hydromorphodynamic model. It handles subcritical and supercritical regimes and cohesive and non-cohesive sediments. Moreover, UMHYSER-1D allows modeling of a single natural channel or multichannel looped networks with different types of internal boundaries and hydraulics structures.

In steady flow conditions, UMHYSER-1D is identical to MHYSER, (Mahdi, 2009). Under unsteady flow conditions, UMHYSER-1D uses a decoupled approach: First, the “NewC” scheme is used to solve the de Saint Venant equations, then the total load convection-diffusion equation is solved using the fractional step method. The equation without the source term is solved with an implicit finite-volume method, then the equation with source term is solved to obtain the sediment concentration at the next time step. The active layer, concept and sediment physical processes are applied to calculate the sediment deposition and erosion. Finally, UMHYSER-1D empirically accounts for bed geometry adjustments by using a relationship between erosion width and flow rate, an angle of repose condition for bank stability and three minimization theories.

An experimental erosion test is used to test the capabilities of UMHYSER-1D to predict the erosion of reservoir deposits following dam removal. Since UMHYSER-1D is a 1D model, it cannot model satisfactory transversal cross-sectional evolution, but sensitivity analysis allowed the identification of the best input data set that reproduced very satisfactory the longitudinal profile evolution. Hence, even at small scale (laboratory flumes), UMHYSER-1D performs very well. For this case, no observed cross-sectional data were available. This is an important issue because the greatest uncertainty in applying 1D model reside in the estimation of the streamwise sediment transport.

Finally, the previous applications of UMHYSER-1D and the present experimental test case show the capabilities of this model and predicts its promising role in solving complex real engineering cases and its use as a numerical laboratory to test available or new scientific findings.

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CHAPTER 5 ARTICLE 2: FLOODING OF THE SAGUENAY REGION IN 1996: PART 1 – MODELING RIVER Ha!Ha! FLOODING

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5.1 Abstract

This paper presents an application of the model UMHYSER-1D (Unsteady Model for the HYdraulics of SEdiments in Rivers One-Dimensional) to the representation of morphological changes along the Ha! Ha! River during the 1996 flooding of the Saguenay region. UMHYSER-1D is a one-dimensional hydromorphodynamic model capable of representing water surface profiles in a single river or a multiriver network, with different flow regimes considering cohesive or noncohesive sediment transport. This model uses fractional sediment transport, bed sorting and armoring along with three minimization theories to achieve riverbed and width adjustments. UMHYSER-1D is applied to the Ha! Ha! River (Quebec, Canada), a tributary of the Saguenay River, for the 1996 downpour. The results permit forcing data verification and prove that some cross sections are not the right ones. UMHYSER-1D captures the trends of erosion and deposition well although the results do not fully agree with the collected data. This application shows the capabilities of this model and predicts its promising role in solving complex, real engineering cases.

Keywords: Saguenay flood 1996; One-dimensional model UMHYSER-1D; Data validation; Ha! Ha! River

5.2 Introduction

Precipitation-runoff floods and dam failure floods result in unusually rapid water surface rises and high-velocity outflows through the downstream river. The inundation of riverbanks may cause significant erosion and important riverbank retreats and creates potentially unstable embankments, as those observed in the aftermath of the Saguenay floods in 1996 (Lapointe et al., 1998).

From July 18 to 21, 1996, unusually heavy rain affected the Saguenay region of Québec, Canada, between Lake St. Jean and the St. Lawrence River (Figure 5.1). These torrential rains are the largest meteorological event recorded in Québec for almost a century. Between 150 and 280 mm of rain fell during more than 48 hours over a territory of several thousand square kilometers, affecting the watersheds of the southern part of the Gaspé Peninsula, Charlevoix, Haute-Mauricie, Haute-Côte-Nord and Saguenay-Lac-Saint-Jean, leading to widespread flooding and damage including extensive erosion in the region, significant riverbank retreat and destruction of many run-of-the-river dams on rivers discharging into the Saguenay River and Saguenay Fjord.

The Saguenay-Lac-Saint-Jean region was affected the most. For example, water discharge to the Kenogami Reservoir Lake reached 2780 m³/s on July 20, 1996, while the historical maximum observed before this event was 997 m³/s.

The situation was particularly dramatic for a few rivers and streams: the Saint-Jean River at Anse-Saint-Jean, the Petit Saguenay River in the municipality of the same name, the A Mars River in the municipality of La Baie, the Ha! Ha! River in the municipalities of La Baie and Ferland-et-Boilleau, the Moulin River in the municipalities of Laterrière and Chicoutimi, the Belle River in the municipality of Hébertville, the Chicoutimi River in the municipalities of Laterrière and Chicoutimi and the Aux Sables River in the municipality of Jonquière. Major damage affected local populations (houses flooded, buildings destabilized and washed away, infrastructure torn apart, etc.) and other effects had negative impacts on rivers through multiple repercussions: riverbeds were lowered, riparian and aquatic vegetation was destroyed, loose soil suffered deep erosion, great amounts of sediment were deposited in some places, multiple beds were created, the majority of habitats were destroyed, aquatic fauna were washed away and so on.

This paper analyses the Ha! Ha! River 1996 floods through modeling by using a newly developed numerical model, UMHYSER-1D (One-Dimensional Unsteady Model for the HYdraulics of SEdiments in Rivers) (AlQasimi and Mahdi, 2018). The second section presents UMHYSER-1D, and section 3 describes the study case, a reach of the Ha! Ha! River, which is a tributary of the Saguenay River, along with the available data; section 4 includes the results and discussion of the simulations, followed by the conclusion.

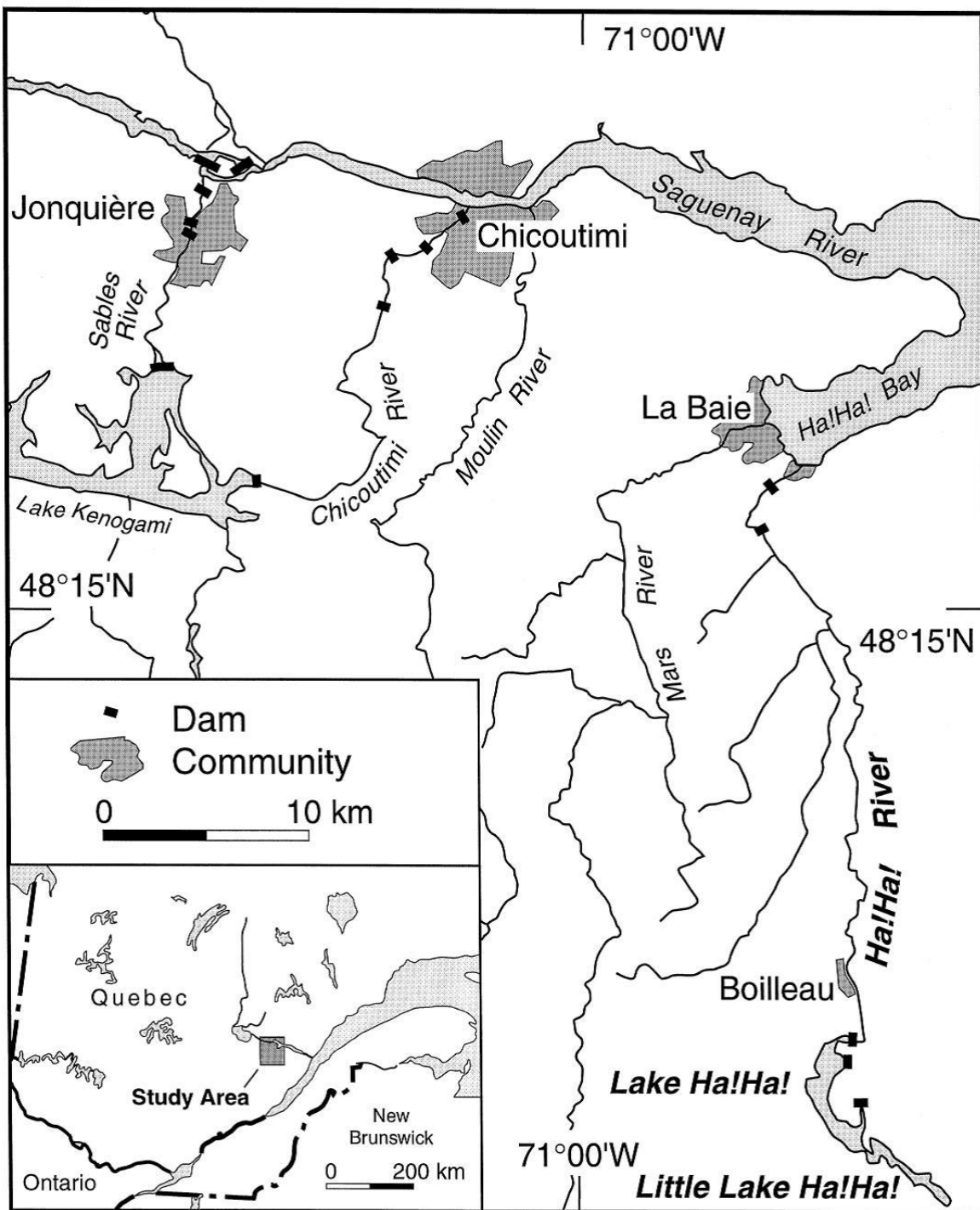


Figure 5-1 Location map showing Ha!Ha! River and the Ha! Ha! Reservoir (Brooks and Lawrence, 1999)

5.3 Overview of UMHYSER-1D

UMHYSER-1D is an unsteady one-dimensional model that represents the water and sediment phases by solving the one-dimensional de Saint Venant equation for the water phase and the Exner/one-dimensional convection-diffusion equation for the solid phase. UMHYSER-1D performs five groups of operations: water phase, stream tubes, sediment phase, riverbank stability analysis and cross section adjustments.

UMHYSER-1D uses the continuity equation and the energy equation when there are no changes in the flow regime, while the momentum equation is used with the continuity equation when there are changes from supercritical to subcritical flows, or vice versa. In the case of steady flow, for backwater computations, the standard step method is used (Henderson, 1966), and the friction losses are computed by a uniform flow formula as generally admitted (Jain, 2000). Under steady-state conditions, the capabilities of UMHYSER-1D are similar to those of the MHYSER model developed by Mahdi (2009). The de Saint Venant equations are used for unsteady flow computations. Irregular cross sections can be handled regardless of whether the river reach consists of a single channel or multiple channels. For the latter case, the variables related to the cross-sectional geometry are computed for each subchannel and are summed to obtain the total values. Moreover, internal conditions such as weirs, falls and sluices are modeled by rating curves. UMHYSER-1D uses the NewC scheme (Kutija and Newett, 2002), which assures numerical stability in the transition between different flow regimes.

After the water surface characteristics are calculated, the cross sections are divided into sections of equal conveyance or stream tubes. These stream tubes act as conventional one-dimensional channels with known hydraulic properties where sediment routing can be carried out within each stream tube almost as if they were independent channels. Once the top widths are determined, the velocities of the stream tubes are calculated by giving a crosswise velocity distribution for every cross section.

Stream tube locations are allowed to vary with time. Therefore, although no material is allowed to cross stream tube boundaries during a time step, lateral movement of sediment is described by lateral variations in the stream tube boundaries. For noncohesive sediment transport, UMHYSER-1D uses the transport functions of Meyer-Peter and Müller (1948) and Parker (1990), Laursen (1958), modified Laursen (Madden, 1993), Toffaleti (1968), Engelund and Hansen (1972), Ackers

and White (1973), modified Ackers and White (HR Wallingford, 1990), Yang (1973, 1979, 1984) and Yang et al. (1996). When the unsteady term of the suspended sediment transport continuity equation is ignored, the Exner equation is solved to update the bed changes. Both the spatial and temporal derivatives are approximated by first-order finite difference operators (Hirsh, 1990).

UMHYSER-1D deposition of cohesive sediment is based on Krone's equation (1962), while particle and mass erosion are based on the work of Partheniades (1965) and adapted by Ariathurai and Krone (1976). For the convection-diffusion equation, the Lax-Wendroff TVD scheme is used to discretize the convective term; a central difference scheme is used for the diffusion term (Tannehill et al., 1997), and the source term discretization is similar to the one used by Vetsch et al. (2017).

For bed changes, the sediment transport is computed for each individual sediment size fraction within each stream tube. The bed changes are computed as a sum of the bed change due to each particle size. To maintain numerical stability, the time step is determined by a Courant-Friedrichs-Lewy (CFL) condition (Cunge et al., 1980). Since the kinematic wave speed of the bed changes is not easily quantified, numerical experimentation is required to determine a suitable time step to be used for a simulation.

UMHYSER-1D uses the method of Bennett and Nordin (1977) for the bed composition accounting procedure by dividing the bed into conceptual layers. The top layer, or active layer, contains the bed material available for transport, beneath which is the storage layer or inactive layer, and finally the undisturbed bed. The active layer is the most important layer in this procedure. Erosion of a particular size class of bed material is limited by the amount of sediment of this size class present in the active layer. At the end of each time step, bed material is calculated in each stream tube. At the beginning of the next time step, after the new locations of the stream tube boundaries are determined, these values are used to compute the new layer thickness and bed composition.

Finally, UMHYSER-1D offers the choice of 3 minimization theories for the determination of depth and width adjustments, at a given time step: minimization of the total stream power (Yang, 1972), minimization of the energy slope (Chang, 1988) and minimization of the bed slope.

5.4 Application: the 1996 Lake Ha! Ha! flood

5.4.1 Site description

The study area is an 8.4 km reach of the Ha! Ha! River, the most severely affected river during the 1996 floods. This river drains a catchment of 610 km². The Ha! Ha! River links Lake Ha! Ha! to the Ha! Ha! Bay, an arm of the Saguenay Fjord (Figure 5.2). The study reach extends from the Cut-away dike at Lake Ha! Ha! to the first falls encountered, 6 km beyond the village of Boilleau (8.4 km from the Cut-away dike that broke during the events).

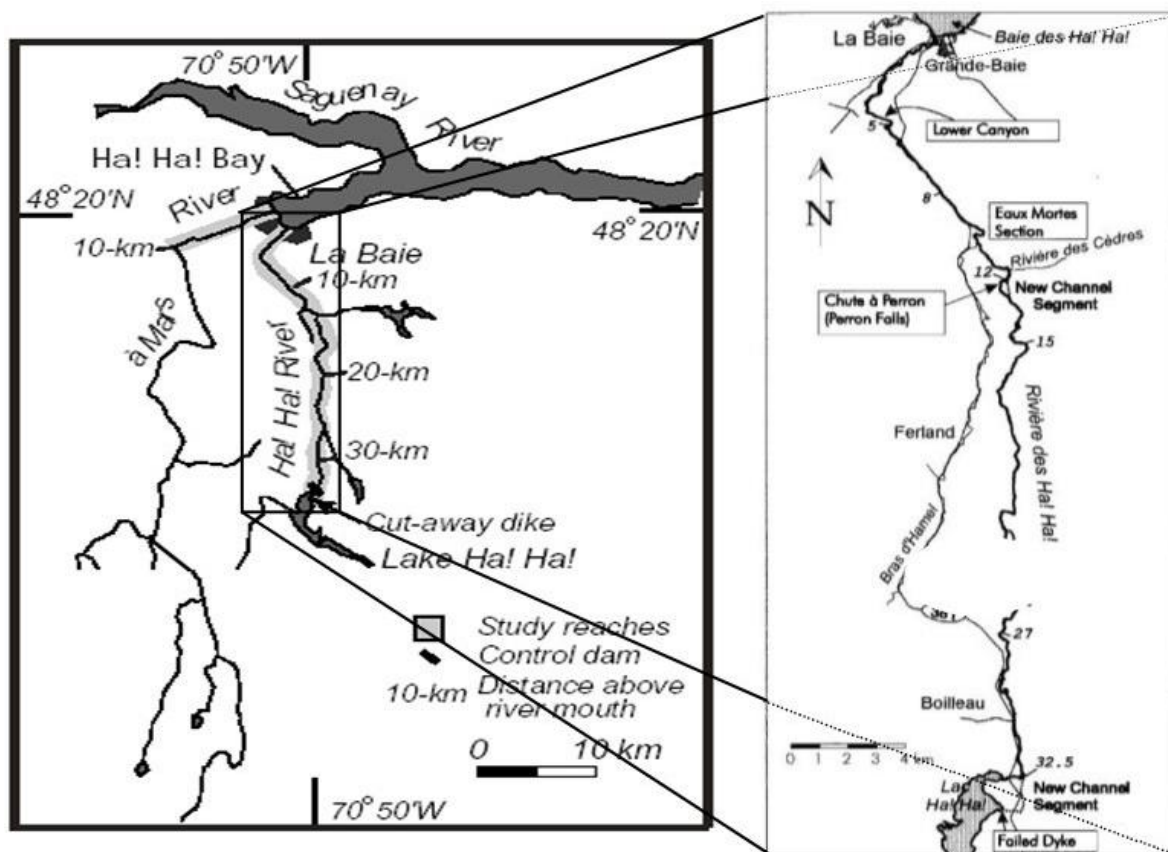


Figure 5-2 Study area: reach of 8.4 km downstream of the Ha! Ha! Lake (modified after Couture and Evans, 2000 and El Kadi and Paquier, 2004)

On July 19, 1996, the watershed of the Ha! Ha! River began to receive an exceptional rain: on average, more than 210 mm of rain fell on this mountain basin of 608 km² and increased the contributions to Lake Ha! Ha! from 10 to 160 m³/s.

Lake Ha! Ha! is impounded by a concrete dam that suffered minor damage during the flood. The dam maintained a high elevation in the water body (380 m) and allowed only a small spill (less than 30 m³/s).

Lake Ha! Ha! was impounded by three structures: the concrete dam as the main dam, the evacuator, and two secondary dikes (Left Bank and Cut-away). The crest elevation of the Cut-away dike was 380.65 m, 40 cm lower than that of the main dam and 35 cm lower than the Left Bank dike (Nicolet Commission 1997).

The rising water level of the lake under the effect of the increased inputs and their partial retention caused on July 20, at approximately 6:00 am, an overtopping on the Cut-away dike, leading to its gradual erosion. The dike breach that developed during the morning of July 20 led to its failure.

As a result, the incision of a new outlet channel occurred, bypassing the concrete dam and leading to rapid drainage of the main lake. Due to the incision, the lake level dropped from a level of 381 m to a new level of 370 m (above average mean sea level). Further details of the flood and the corresponding damage are given by Brooks and Lawrence (1999). They estimated the peak outflow to be in the range of 1080-1260 m³/s at a surveyed cross section 27 km downstream from the dike.

5.4.2 Available data

The 37 km river reach, from the Cut-away dike to the Ha! Ha! Bay (Figure 5.1), is discretized into 370 cross sections before and after the flood, and the rock elevations along the river (Figure 5.3) are provided by Capart et al. (2007).

The hydrograph of the breach flood (Figure 5.4) and a size gradation curve (Figure 5.5) for the first 10 km of this river reach are provided by Mahdi and Marche (2003). This sediment distribution is assumed to be valid for the entire river. Note that to ensure numerical stability, a time step of 10⁻⁴s is used.

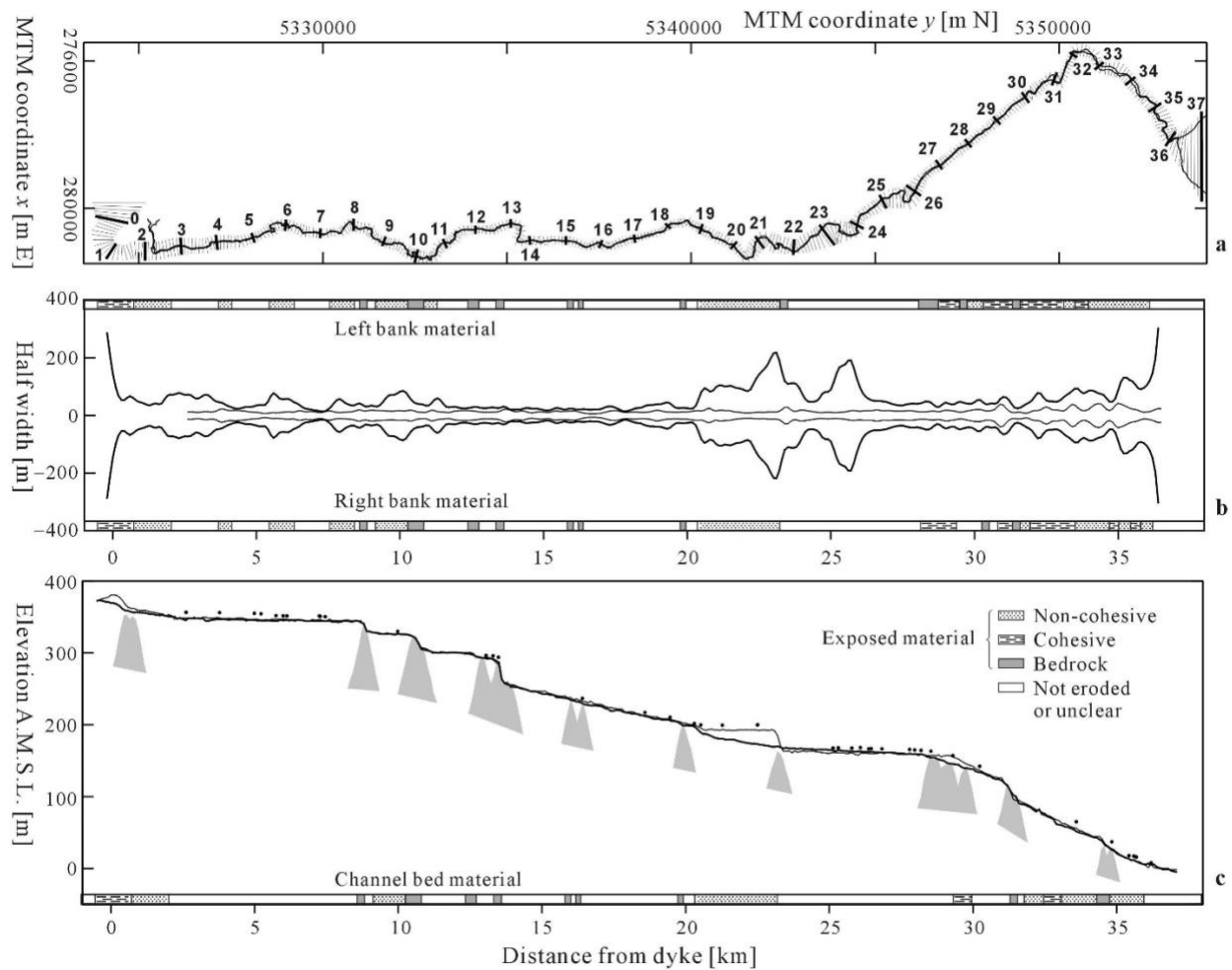


Figure 5-3 Longitudinal river profiles: (a) evenly spaced valley cross-sections, numerals in km indicate distance from breached dyke; (b) width changes induced by the flood (pre-flood and post-flood corridors); and (c) elevation data: pre-flood thalweg profile (thin line), post-flood thalweg profile (thick line), surveyed high-water marks (dots), and reconstructed bedrock surface (grey); (Capart et al., 2007)

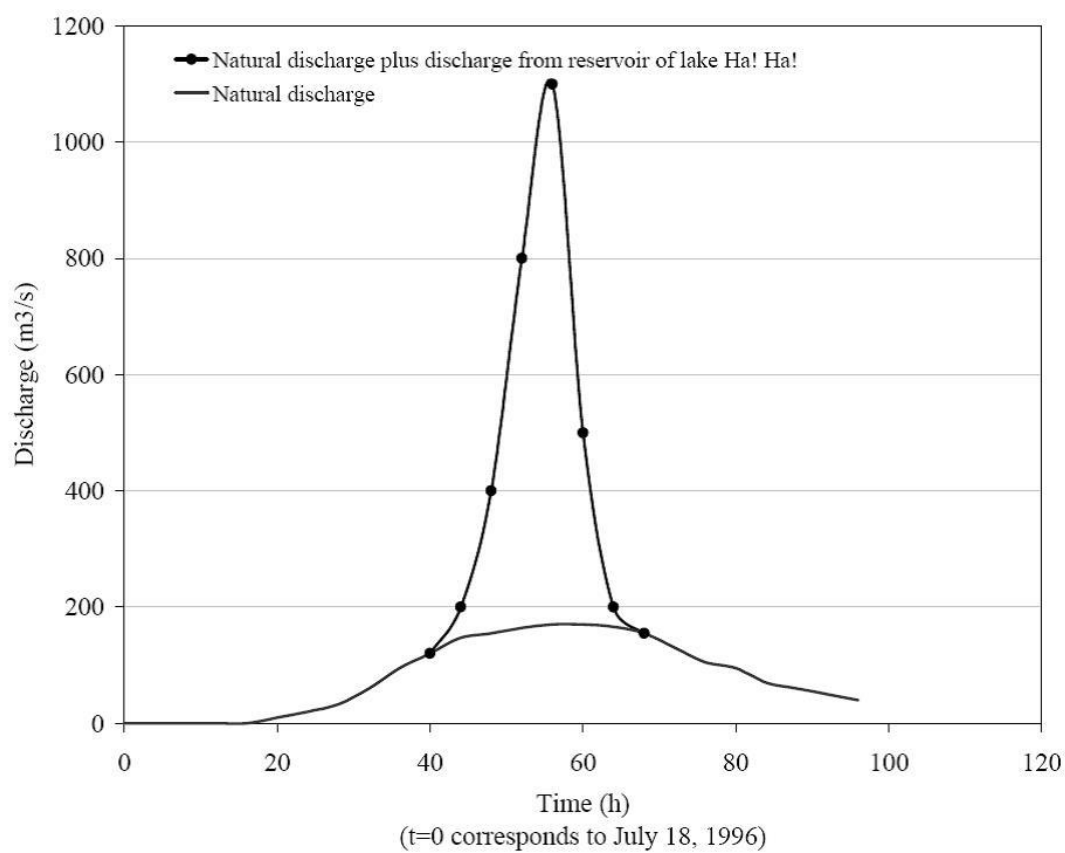


Figure 5-4 Outflow discharge hydrograph (Mahdi and Marche, 2003)

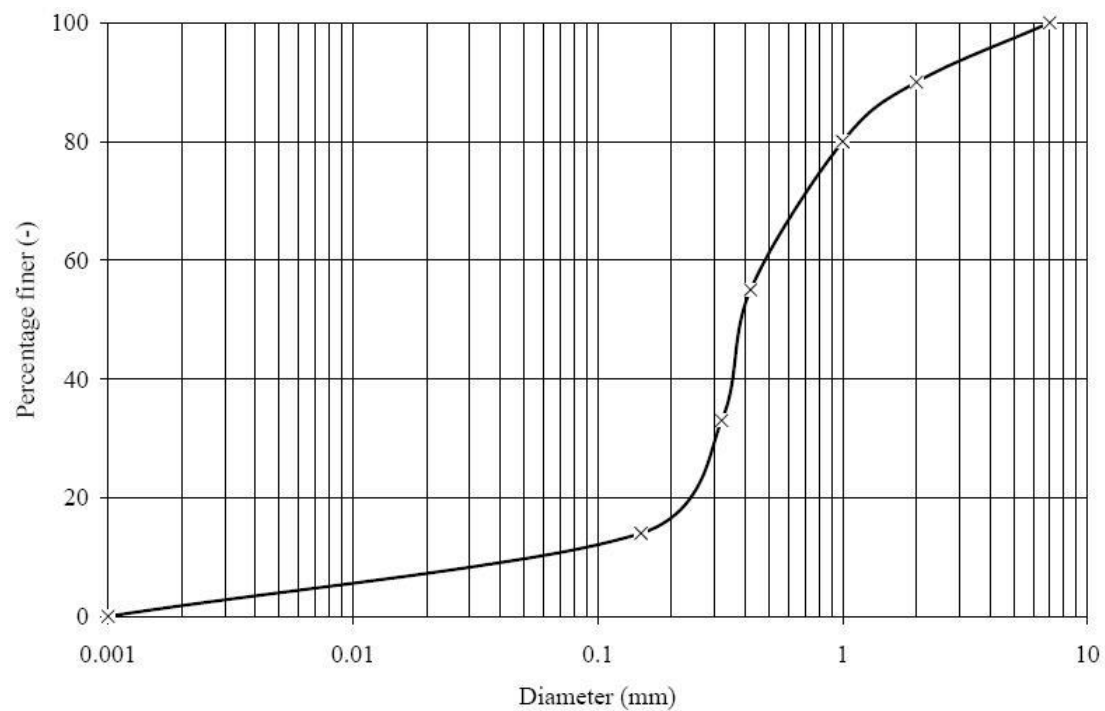


Figure 5-5 Sediment particle size distribution (Mahdi and Marche, 2003)

5.5 Results and discussion

As the available data cover the river's cross sections before and after the flood, the only interesting results are the longitudinal profile and the comparison of the cross sections to the observations.

5.5.1 Cross sections data validation

Performing data validation of the cross sections reveals that not all the available cross sections provided by Capart et al. (2007) and covering the entire river can be used. Figure 5.6 shows an example of a cross section that cannot be used for the simulation. After removing several similar cross sections, the first simulations aimed to model the whole Ha! Ha! River, since the available cross sections covered the entire river.

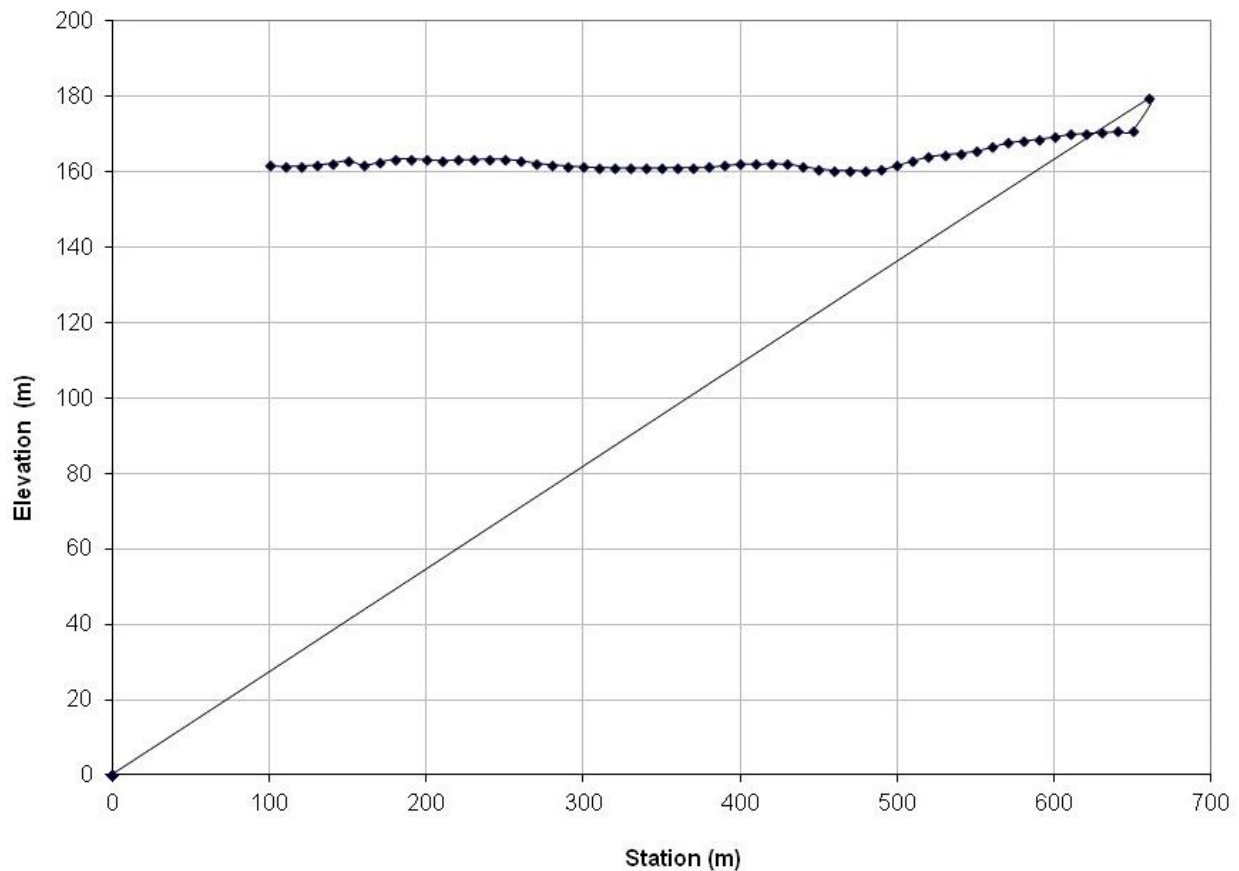


Figure 5-6 Example of an invalid initial cross-section: cross-section 263 from Capart et al. (2007)

Figure 5.7 shows an example of a simulated thalweg and the observed one (Capart et al., 2007). As shown, major differences between the simulated and observed thalwegs are noted approximately 22 km downstream from the Cut-away dike.

These major differences, appearing at a specific zone and reaching a maximum value of 20 m, cannot be attributed to modeling errors. As seen from Figure 5.2, approximately 22 km downstream of the Cut-away dike and just upstream of Perron Falls, a new river path was created during the 1996 flood. The available initial cross sections are along the old riverbed, and the available postflood cross sections are along the new river path. Hence, in this zone, the preflood cross sections of Capart et al. (2007) are not the right ones to use for the simulations. Once this river zone is excluded, only the upper reach, 8.4 km long, can be modeled.

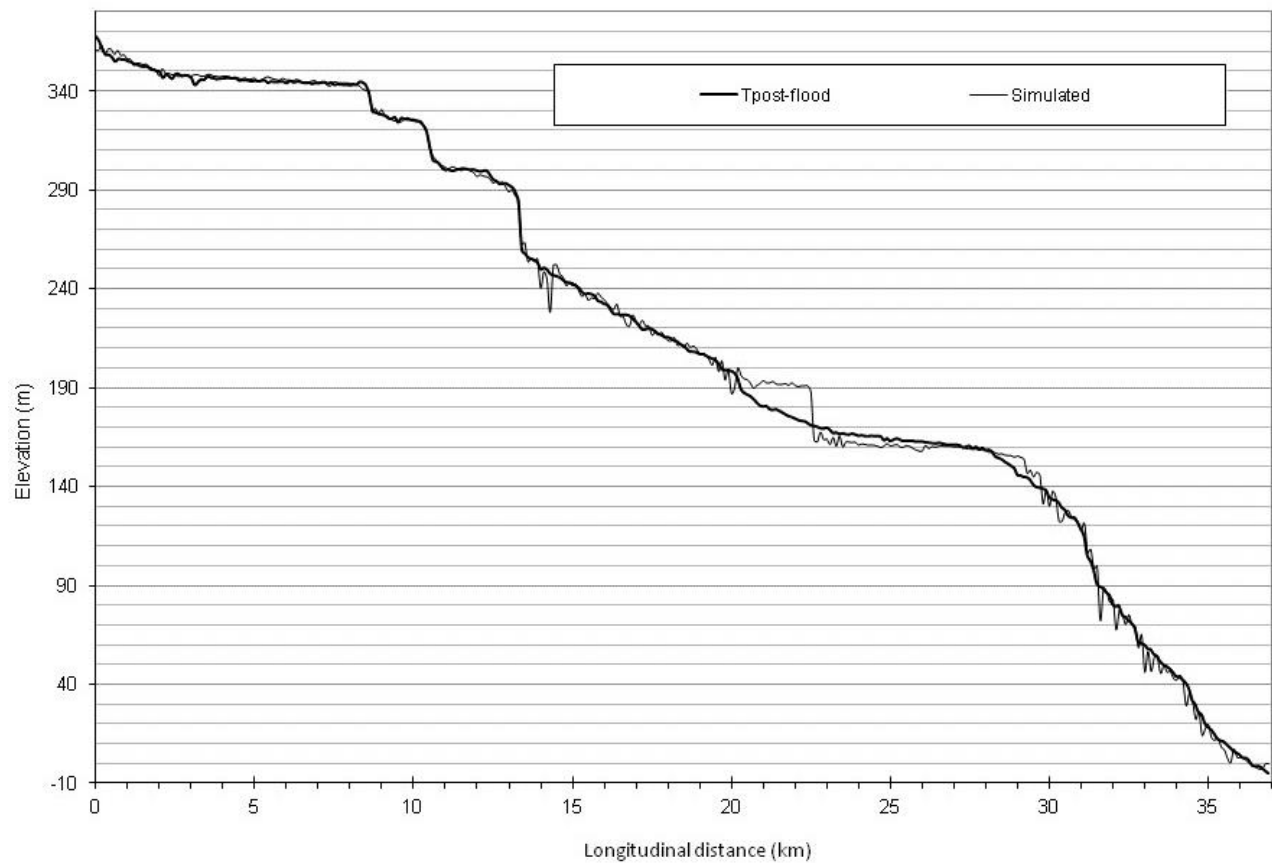


Figure 5-7 Example of observed and simulated thalwegs

5.5.2 Longitudinal profile

Figure 5.8 shows the initial, simulated and observed final longitudinal profiles. UMHYSER-1D captures the trends of erosion and deposition well, although the simulated profile underestimates erosion for almost the whole river reach, except at the first cross section and the last 1.2 km (Figure 5.9). For the first cross section, the predicted thalweg is 1.5 m (12%) deeper than the observed one.

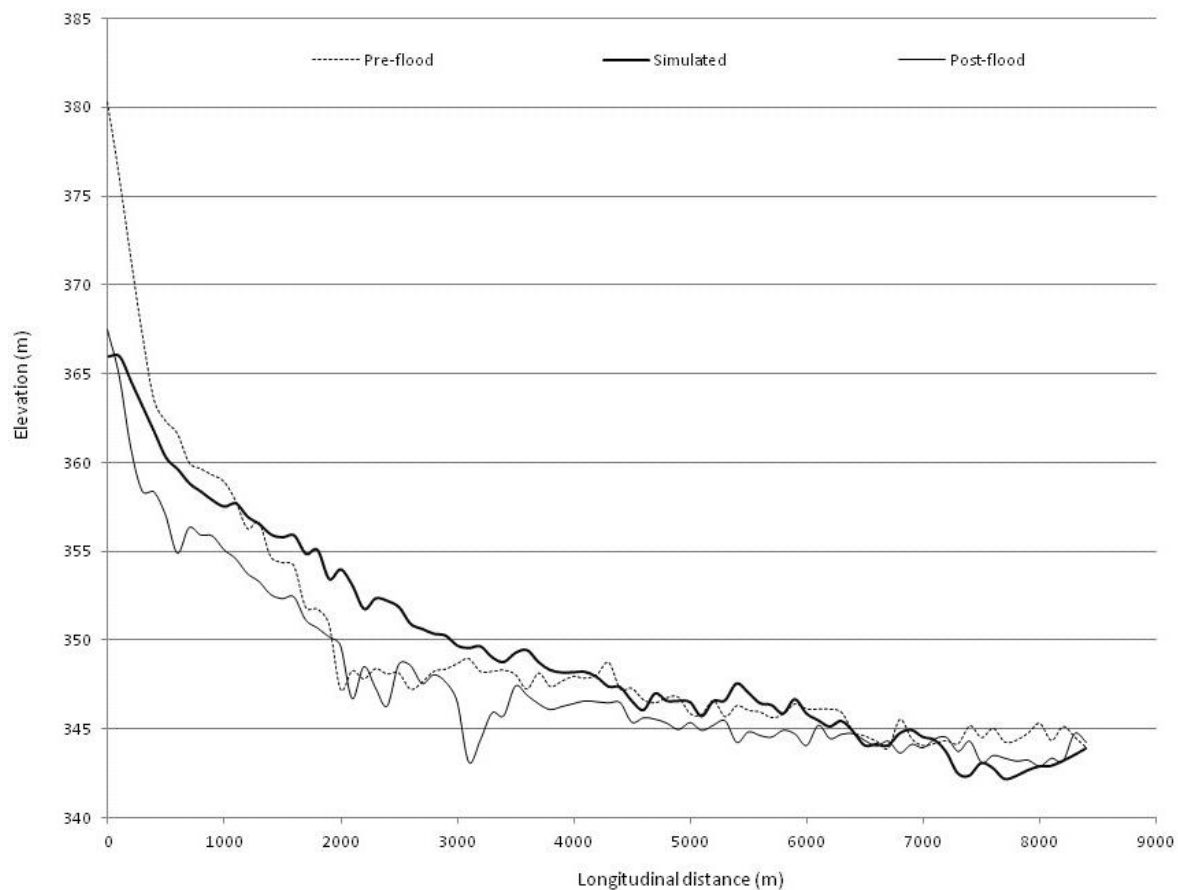


Figure 5-8 Initial, observed and simulated longitudinal profiles

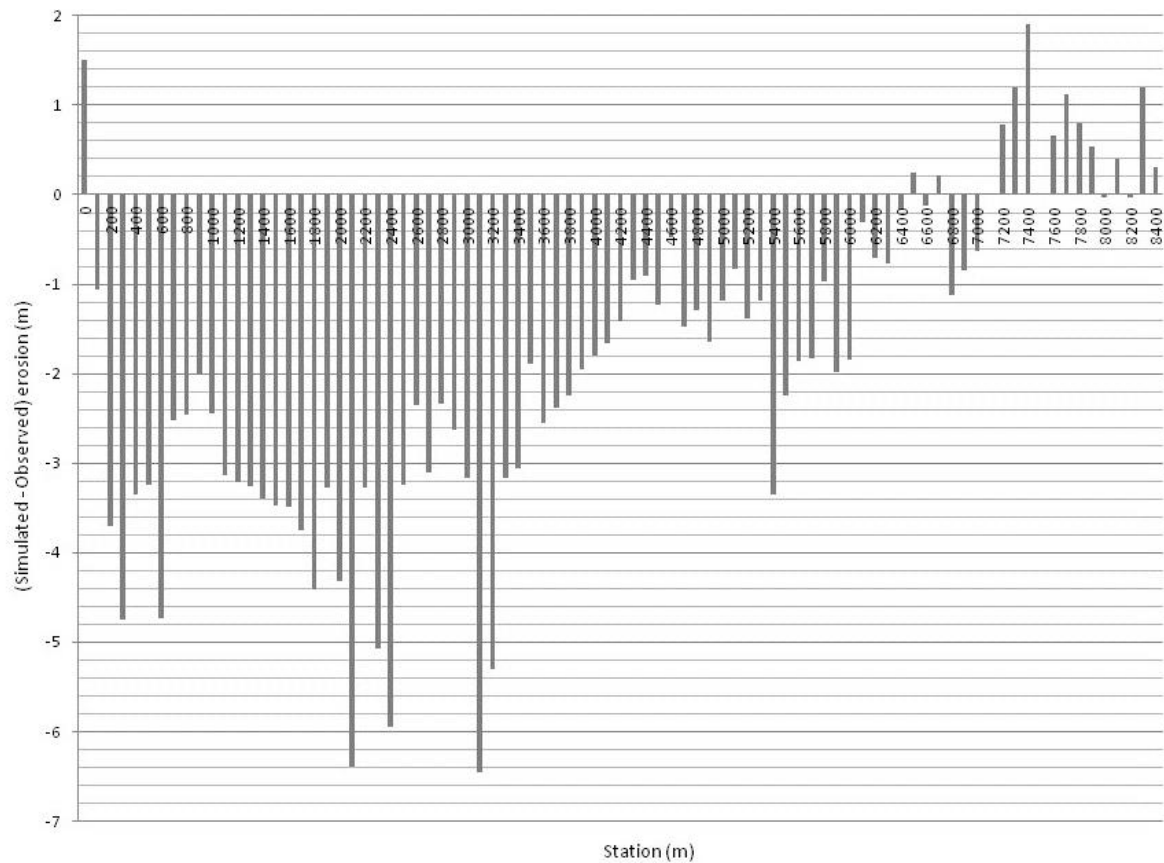


Figure 5-9 Difference between simulated and observed erosion

5.5.3 Evolution of the cross sections

Figures 5.10 to 5.14 show examples of simulated and measured cross sections after the flood passage. Using UMHYSER-1D, the trends of cross sections' evolution are well captured, although the erosion is underestimated for all the cross sections except those of the last 1.2 km. Note from Figure 5.11, the unusual shape of the observed final cross section where the right riverbank experienced sediment deposition of more than 40 m.

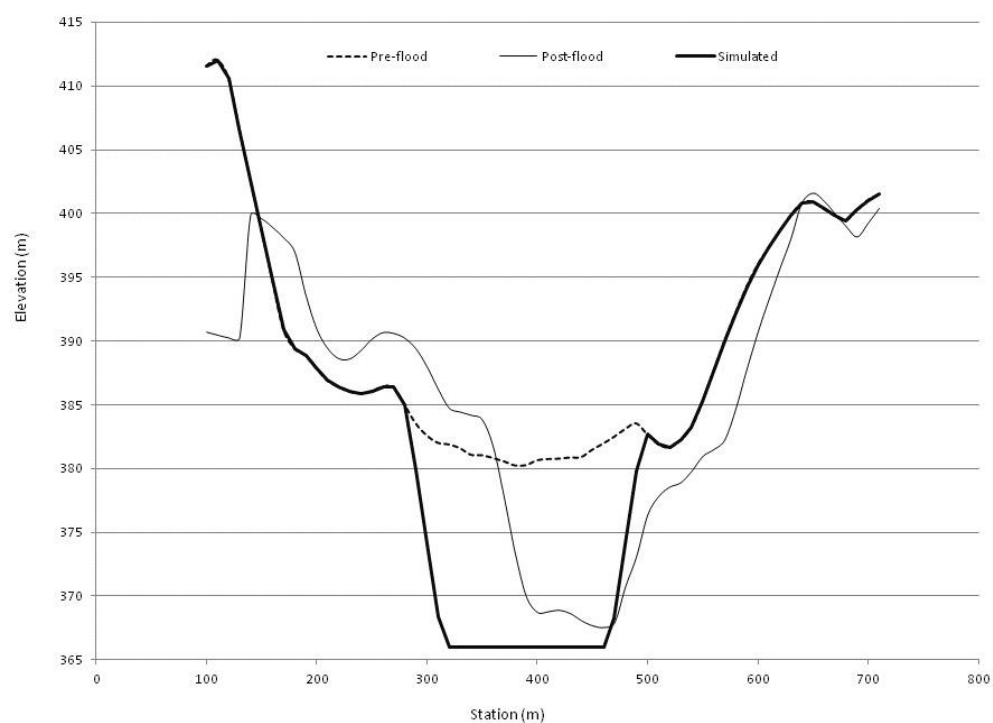


Figure 5-10 Simulated and observed first cross-section (0 km)

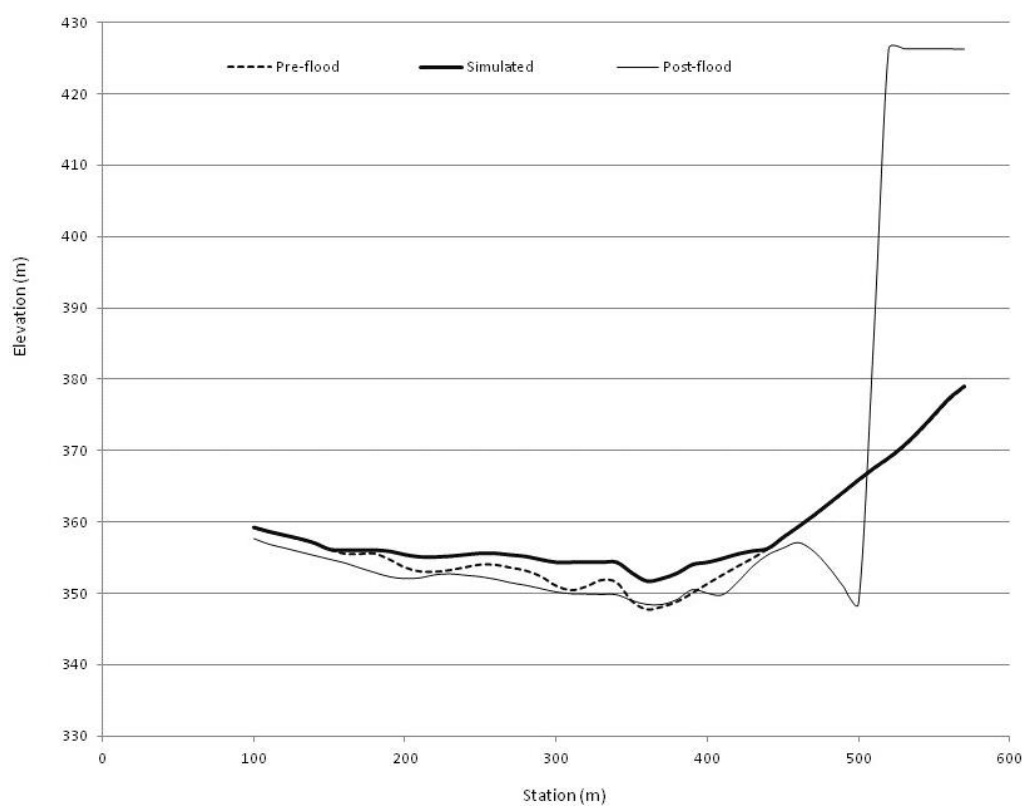


Figure 5-11 Simulated and observed cross-section 23 (2.2 km from upstream)

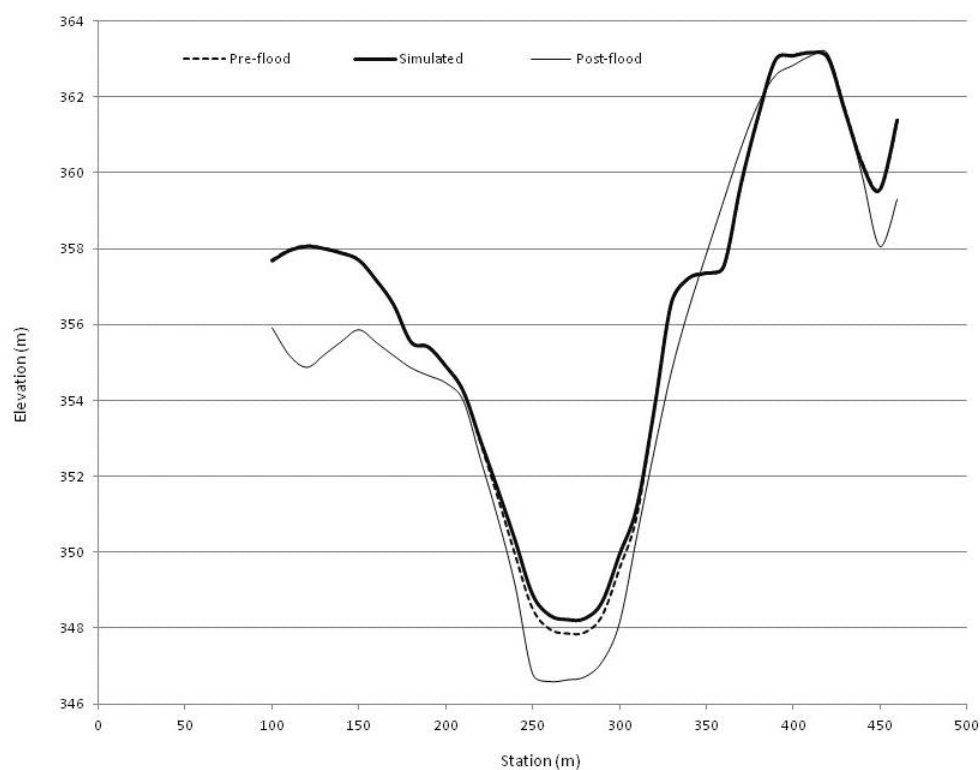


Figure 5-12 Simulated and observed cross-section 42 (4.1 km from upstream)

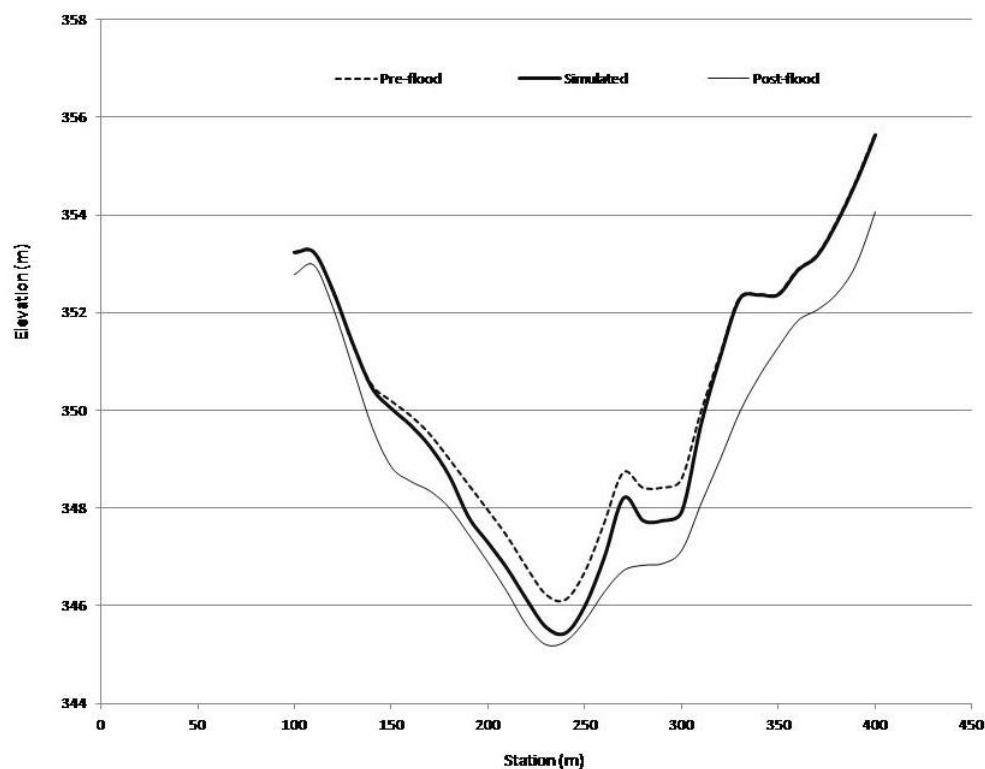


Figure 5-13 Simulated and observed cross-section 62 (6.8 km from upstream)

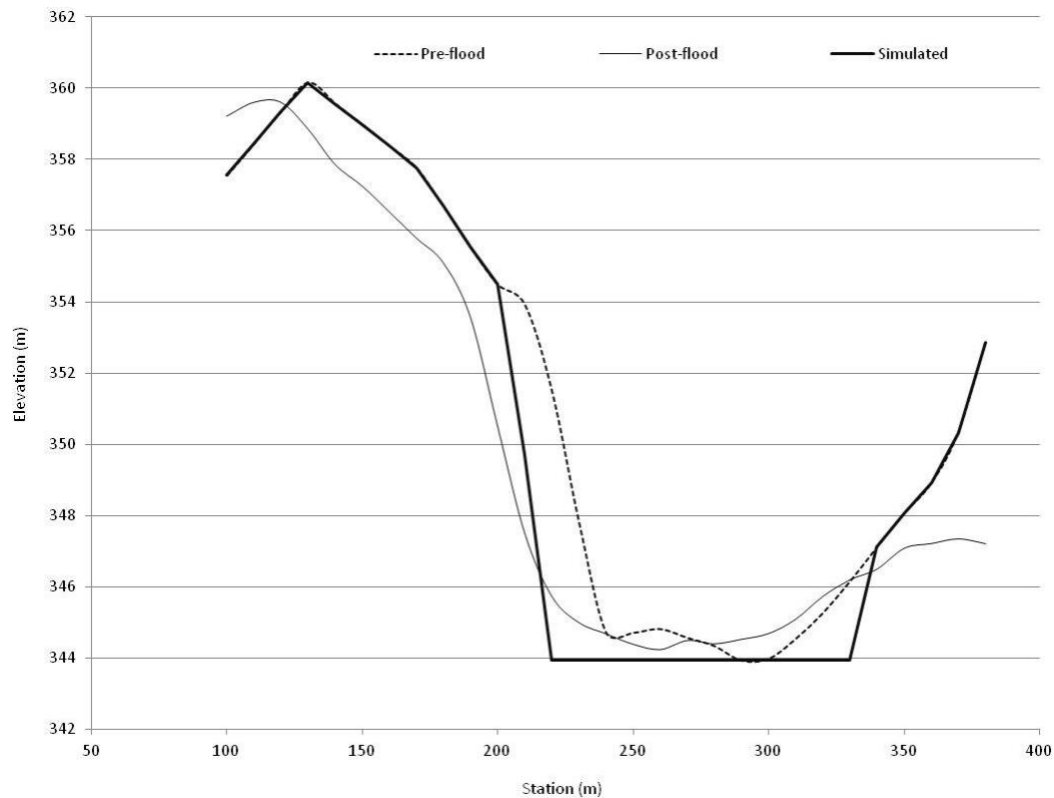


Figure 5-14 Simulated and observed last cross-section (8.4 km from upstream)

5.6 Conclusion

Several river systems could suffer extensive catastrophic floods in the event of a dam break. This paper presents UMHYSER-1D, a newly developed one-dimensional hydromorphodynamic model that solves the de Saint Venant equations, the sediment Exner equation and a convection-diffusion equation for suspended sediments. The model handles subcritical and supercritical regimes and cohesive and noncohesive sediments. Moreover, UMHYSER-1D allows modeling of a single natural channel or multichannel looped networks with different types of internal boundaries. Applied to the Ha! Ha! River (Quebec, Canada) for the 1996 flood, UMHYSER-1D predicts the trends in the river changes well. Furthermore, based on a set of simulations, a doubt was raised about the quality of the cross sections used along a reach of 3 km. This question was confirmed after finding evidence in the literature that the Ha! Ha! River overflowed from its original channel to a secondary valley just before Perron Falls. Thus, the original cross sections used cannot be considered as they belong to the old river path and do not cover the secondary valley where the

post flood river flows. Although UMHYSER-1D captures the main features by predicting the evolution trends of the longitudinal river profile and cross sections, the numerical results are not in full agreement with the observations. Several reasons can explain this shortcoming. First, UMHYSER-1D is a one-dimensional model based on the de Saint Venant equations, which assume small bed slopes and neglect vertical accelerations. Second, the sediment transport equations used in the model are developed under quasi-uniform and steady flow conditions with small water velocities, which was not the case during the 1996 Ha! Ha! River flooding. Finally, several assumptions were used for the input data: some cross sections had bizarre shapes, a single gradation curve was used for the entire river reach with a single roughness coefficient, and debris flows were ignored. Indeed, after the breaching of the Cut-away dike, water flowed in a forest, and a new channel was created after trees were uprooted. UMHYSER-1D was used in an extremely complicated case and was able to predict the trends of deposition/erosion using simplified assumptions for the input data. Knowing the different sources of sediment transport uncertainty, the performance of UMHYSER-1D is encouraging. The application of UMHYSER-1D to the 1996 Ha! Ha! River flooding shows the capabilities of this model and predicts its promising role in real engineering cases.

5.7 Acknowledgement

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CHAPTER 6 ARTICLE 3: FLOODING OF THE SAGUENAY REGION IN 1996: PART 2 – AUX SABLES RIVER FLOOD MITIGATION AND ENVIRONMENTAL IMPACT ASSESSMENT

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6.1 Abstract

After the flooding of the Saguenay region in July 1996, several rivers, including the Aux Sables River, experienced unusual water discharge, causing flooding and morphological damage. This paper addresses flood mitigation and environmental impact assessment of the Aux Sables River following the July 1996 flooding. The consequences of the flood are summarized followed by a review of various proposed solutions for a similar flood. The option of dredging the Aux Sables River to increase discharge without causing flooding raises the issue of suspended sediment concentration, since the intake water at Jonquiere would be at risk. Utilizing newly developed software, UMHYSER-1D, the suspended sediment impact assessment for the Aux Sables River enables the maximum permissible sediment discharge to be released into the river while avoiding any risk of pollution for the population of Jonquiere city. Using UMHYSER-1D to mitigate this risk confirms the important role of numerical modeling in solving complex engineering problems.

Keywords: Saguenay flood July 1996, Aux Sables River, Flood mitigation, Suspended sediment impact assessment, UMHYSER-1D software

6.2 Introduction

During the July 1996 Saguenay flood, several rivers experienced unusual water discharge, causing flooding and morphological damage. In addition to the impact to the Ha! Ha! River, the most extreme flooding occurred along the Aux Ecorces, Pikauba, and Cyriac Rivers, which flow into Lake Kénogami and are tributaries of the Aux Sables, Chicoutimi, Du Moulin, and A Mars Rivers (Figure 6.1). The Chicoutimi and Aux Sables Rivers, two outlets of the Kenogami Reservoir, were particularly affected by the flooding.

For Lake Kenogami, the estimated maximum inflow was 2780 m³/s, exceeding the outflow and causing the reservoir to rise to unprecedented levels. The water level reached a maximum level of 166.08 m, exceeding the 165.7 m crest of the concrete dams (Environnement et Faune Quebec, 1996). As a result, a number of dikes and all three dams (two discharging in Aux Sables River) controlling the reservoir level were overtopped by Lake Kenogami waters (Environnement et Faune Quebec, 1996a). The outflow spilled primarily into the Aux Sables and Chicoutimi Rivers.

The four sections of this paper address flood mitigation and environmental impact assessment for the Aux Sables River. Section 2 presents first the consequences of the 1996 flood on the Aux Sables River and the proposed solutions to reduce the consequences of a similar flood are reviewed and the retained option is explained. Section 3 addresses the suspended sediment impact assessment for the Aux Sables River during implementation of the retained solution. The results and discussion are presented in section 4 followed by the study's conclusions.

6.3 Aux Sables River flood mitigation

6.3.1 Study reach

Flowing northwards into the Saguenay valley, the Aux Sables River is a tributary of the Saguenay River (Figure 6.1). The study reach is situated along the lower 11.1 km of the Aux Sables River, from Lake Kenogami to the Saguenay River, with three run-of-the-river dams: the Jonquière, Ville-de-Jonquière and Chute-à-Besy dams (Figure 6.2).

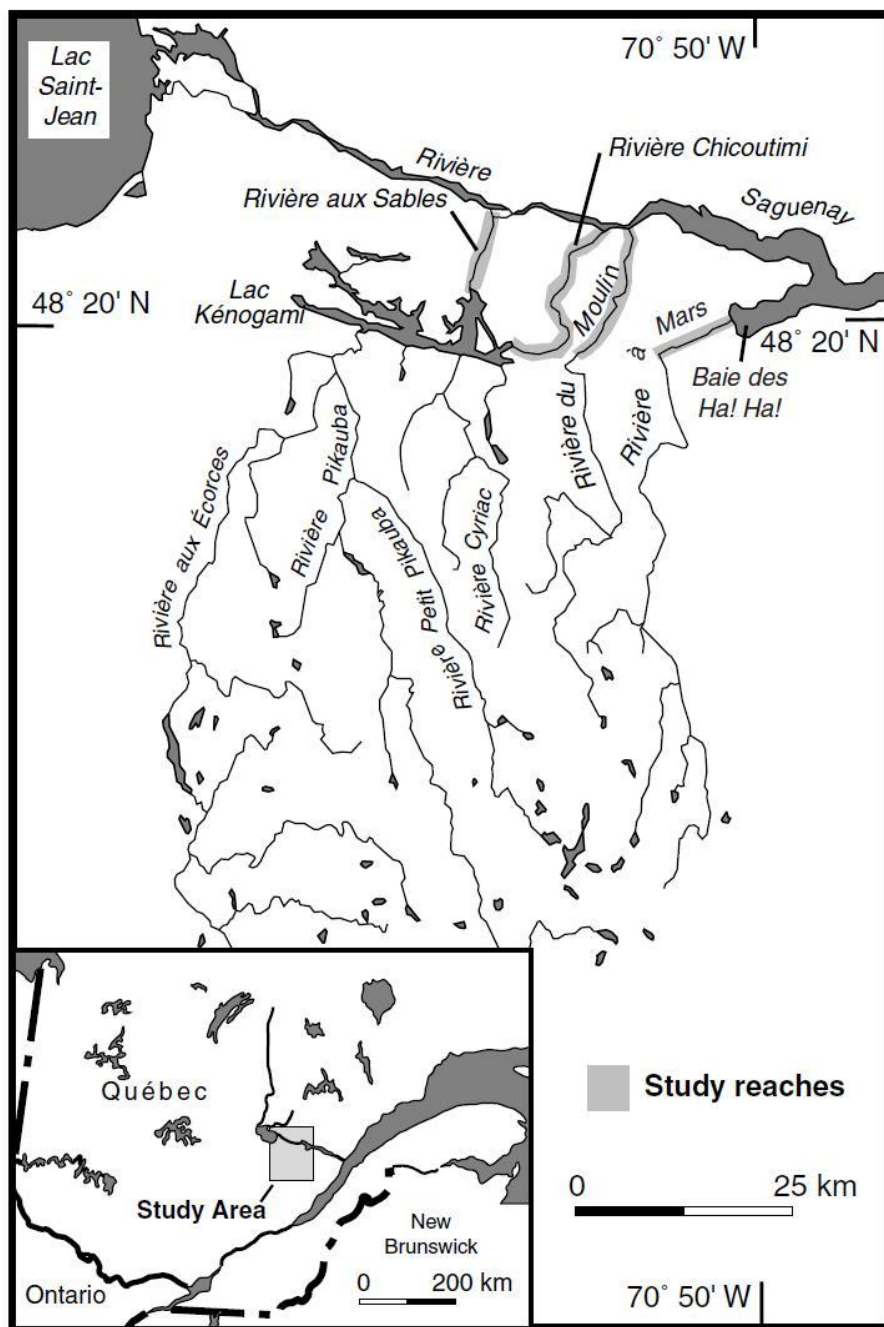


Figure 6-1 Location map depicting Lake Kenogami, Chicoutimi and Aux Sables Rivers (Brooks and Lawrence, 2000)

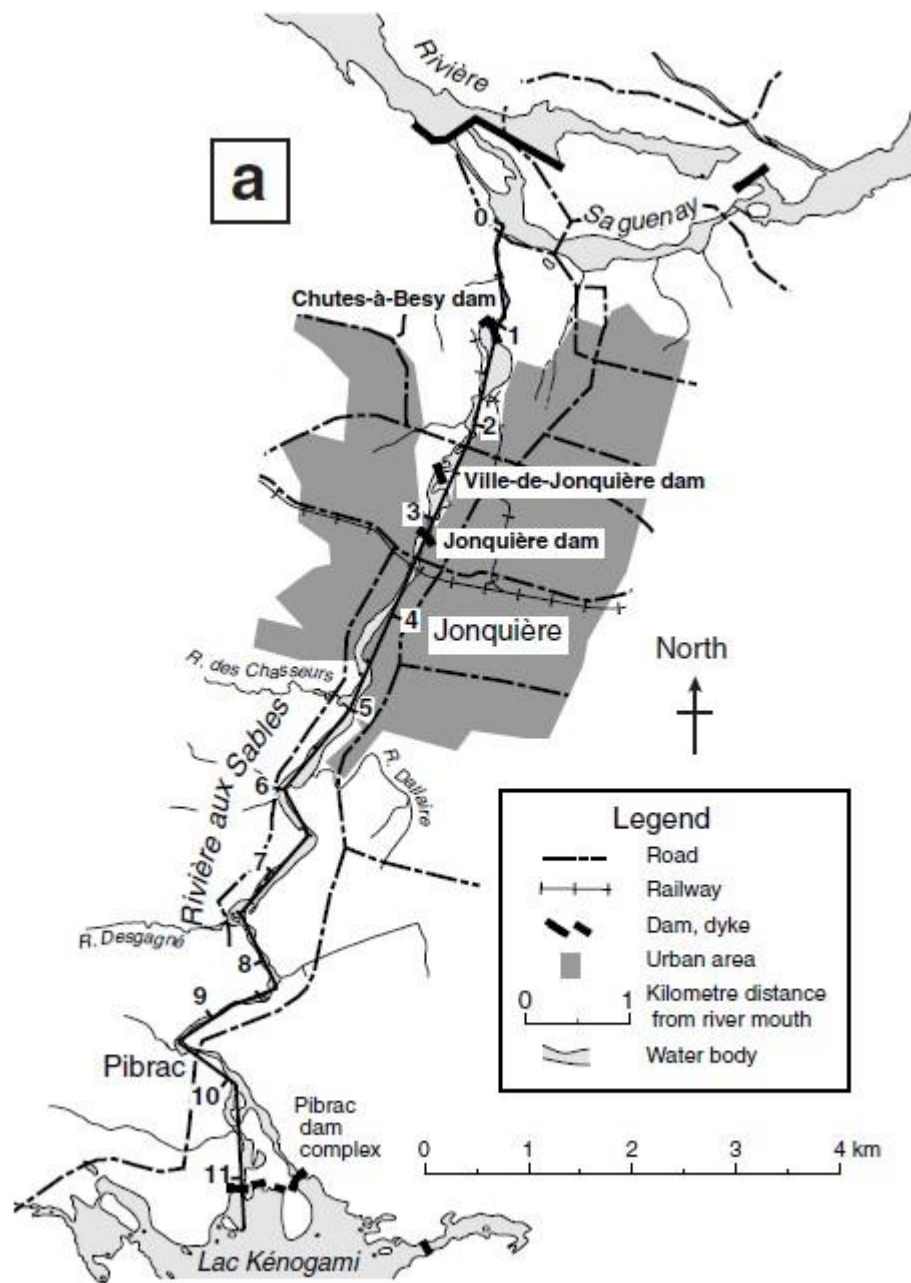


Figure 6-2 Aux Sables River: Pre-flood maps depicting the study reach along with the location of the run-of-the-river dams (Modified after Brooks and Lawrence, 2000)

The longitudinal profile of the river is shown in Figure 6.3, where the gentle gradient steepens markedly along the last 3 km, characterized by the presence of bedrock.

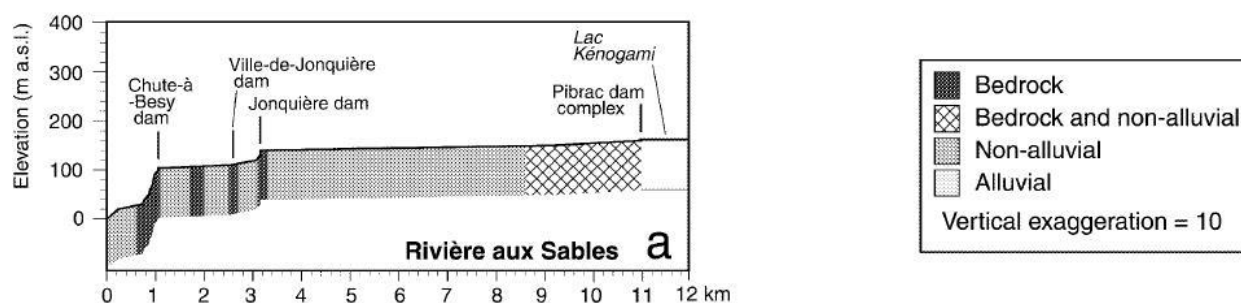


Figure 6-3 Longitudinal profiles and channel morphologies along the Aux Sables River
(Modified after Brooks and Lawrence, 2000)

6.3.2 Discharge

The discharge of the Aux Sables and Chicoutimi rivers, which are outlets of the Kenogami Reservoir, are regulated by control dams. The rainstorm of July 18 to 21, 1996, produced the hydrograph in Figure 6.4, on the Aux Sables River where two control dams and two dikes form the Pibrac dam complex (Figure 6.2).

6.3.3 Consequences

When the discharge along the Aux Sables River reaches $150 \text{ m}^3/\text{s}$, property inundation occurs, and when $170 \text{ m}^3/\text{s}$ is exceeded, flooding of homes begins (Environnement et Faune Quebec, 1996). During the July 1996 flooding, not only was the peak flow discharge $653 \text{ m}^3/\text{s}$ (CSTGB, 1997), overtaking these critical values, but flood discharge also exceeded the available spilling capacity at the Jonquière and Ville-de-Jonquière dams, which are two run-of-the-river dams (Figure 6.4). Table 6.1 summarizes the spilling capacities and the eventual consequences for the three dams. Between 8.6 to 8.4 km, up to 20 m of lateral bank erosion occurred along the left bank, while a series of sand and gravel mid-channel, side, and point bars were aggraded between 8.3 and 7.2 km, and between 0.6 km and the river mouth (0 km), the channel was widened from 20-30 m to 60-130 m. Between 11 and 3.5 km numerous homes were inundated on the gently sloping upper portion of the river (Brooks et al., 1997).

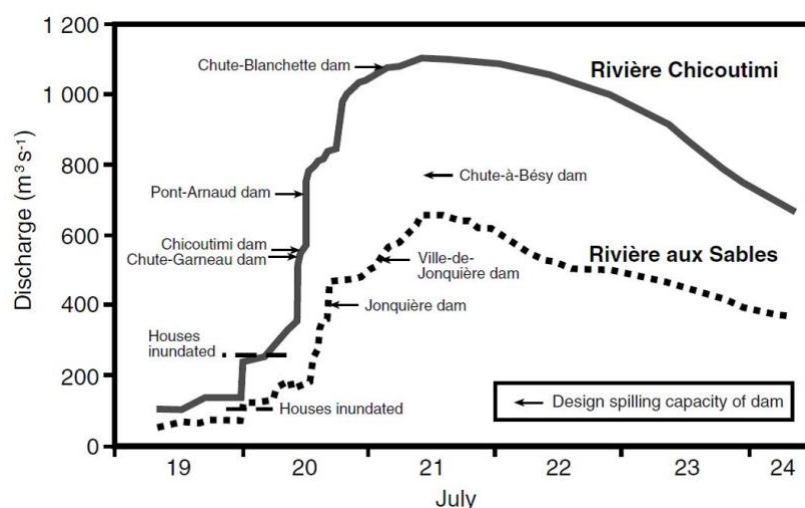


Figure 6-4 Aux Sables River's storm hydrograph (July 19-24, 1996; CSTGB, 1997). Note the minimum discharge beyond which houses begin to be flooded and the maximum spilling capacities of the run-of-the-river dams (Modified after Brooks and Lawrence, 2000)

Table 6-1 Aux Sables River run-of-the-river dams and impacts from flooding (modified after CSTGB, 1997 and Brooks and Lawrence, 2000)

Dam (Year)	Type (foundation)	Dam height (m)	Dam length (m)	Spilling capacity* (m ³ /s)	Impacts of Flooding
Jonquière (1943)	Concrete gravity (bedrock with soil abutments)	8	93	530	- abutment breached, lateral valley side erosion, reservoir drained, river flow diverted away from dam
Ville-de- Jonquière (1996)	Concrete gravity (bedrock)	< 15	~105	455	- concrete wing-wall breached, partial drawdown of reservoir
Chute-à- Bésy (1911)	Concrete gravity (bedrock) and earthen dike (soil)	< 15	~220	770	- dike breached, incision and lateral erosion of valley side, reservoir drained, river flow diverted away from dam

* spilling capacity at maximum reservoir working level

6.3.4 Flood mitigation

Mitigation options

The flooding events along the Aux Sables River, among other rivers, were analyzed by the ``Commission scientifique et technique sur la gestion des barrages`` (CSTGB, 1997). Afterward, Hydro-Quebec, Ministère de l'Environnement and a consortium of consultants assessed various options to mitigate floods resulting from extreme conditions in Lake Kenogami, and the Chicoutimi and Aux Sables Rivers (Ministère des Ressources Naturelles et Hydro-Quebec, 2002a, 2002b, 2002c, 2002d, 2002e; Hydro-Quebec, 2001, 2002a, 2002b and Groupe-Conseil GENIVAR, 2002).

The options were analyzed according to the following criteria:

1. A flood comparable to the one of July 1996 should not exceed the major flood levels on the Chicoutimi and Aux Sables Rivers, corresponding to the discharge beyond which a home begins to be flooded;
2. For a maximum probable flood, the water level of Lake Kenogami must be less than 166.67 m;
3. All existing or new structures must conform to the Dam Safety Act;
4. During summer the water level of Lake Kenogami must be stabilized at $163.86 \text{ m} \pm 0.1 \text{ m}$.

Different scenarios were proposed. The first proposed raising and consolidating the Lake Kenogami retaining structures, raising the flood level on the downstream rivers and implementing an improved flood forecasting system.

The second scenario suggested the construction of a reservoir upstream of Lake Kenogami on the Pikauba River, the consolidation and modernization of existing Lake Kenogami structures, and the construction of a sill in the Aux Sables River, in addition to the implementation of an improved flood forecasting system.

Finally, the third scenario involved the construction of two reservoirs on the Pikauba and Aux Ecorces Rivers, the consolidation and modernization of existing Lake Kenogami structures and the implementation of an improved flood forecasting system.

Digging a sill in the Aux Sables River

In this paper only the flood mitigation option directly related to the Aux Sables River will be reviewed. The following options were considered to increase Lake Kenogami's discharge capacity (Ministère des Ressources Naturelles, de la Faune et des Parcs, 2003):

Digging 600 m along the Aux Sables River;

1. Digging several kilometers along the Chicoutimi River;
2. Building an outlet toward Lake Saint-Jean along several kilometers via the Belle-Rivière;
3. Building a canal in the Jean-Dechêne stream that flows for several kilometers before reaching the Saguenay;
4. Digging a tunnel several kilometers long toward the Saguenay.

After several studies, the retained option was digging a single sill in the Aux Sables River to ensure the protection of homes that are susceptible to flooding on either side of the river upstream of Pibrac Bridge (Ministère des Ressources Naturelles et Hydro-Quebec, 2002a, 2002b, 2002c, 2002d, 2002e). In 2009, GENIVAR (2009) proposed a new concept by eliminating the Pibrac Bridge pillar (Figure 6.5), which acts as an obstruction to flowing water at high discharges.

The proposed concept (Figure 6.6) consists of

1. Increasing the flow section downstream of the bridge (downstream of km 10.29) by excavating the left bank for approximately 60 m;
2. Increasing the flow to the right of the bridge by excavating the riverbed approximately 2 m and on the left bank;
3. Replacing the Pibrac Bridge with a span of 50 m (without a central pillar);
4. Increasing the flow upstream of the bridge by excavating a 440 m channel with maximum width of 60 m, requiring excavation of the riverbed varying from 0.5 to 2 m;

According to the above, the Aux Sables River will be able to carry a discharge of 650 m³/s without flooding the riparian houses.



Figure 6-5 Pibrac Bridge: for high discharges, the pillar is an obstruction to the flow, increasing the water level upstream. Note the cofferdam installed to limit suspended sediment release



Figure 6-6 Excavation limits from the Pibrac Bridge: yellow limits correspond to option replacing the bridge with a longer one with no pillar (Modified after GENIVAR, 2009)

6.4 Suspended sediment impact assessment

6.4.1 Site description

The study area (Figure 6.7) is approximately 5.8 km long from the Pibrac Bridge (km 10.32) to the Jonquiere water intake (km 4.9). The outlet of the small shallow lake, just after the bridge (steep 400 m long reach), Rapid du CEPAL, forces suspended sediment mixing (Figure 6.8). The tributaries to this river reach do not contribute significantly to the river discharge except tributary C at km 5.4, 500 m from the water intake station. As can be seen in Figure 6.9, the color difference between the water of this tributary and the river water suggests that it provides substantial suspended sediment.

6.4.2 Problem presentation

The excavation of the river involves suspended sediment release. Depending on the concentration, these sediments can be harmful at the filtration plant in the city of Jonquiere eventually leading to a significant risk of drinking water pollution and therefore a health risk for the population.

The suspended sediment concentration at the Jonquiere water intake should be less than 19.29 mg/l to ensure drinkable water. To minimize suspended transport downstream from the Pibrac Bridge, cofferdams were installed just after the bridge (Figure 6.5). The question is the following: for a given constant discharge controlled by the Pibrac dam at km 11.1, what is the maximum sediment concentration released at the bridge to ensure that the suspended sediment concentration at the Jonquiere water intake is less than 19.29 mg/l?



Figure 6-7 Study area from Pibrac Bridge to the Jonquiere water intake (courtesy of GENIVAR)



Figure 6-8 CEPAL rapid at km 9.7, looking upstream. Suspended sediment mixing allows adopting one-dimensional approach



Figure 6-9 Suspended sediment inflow from tributary C (km 5.4) (Courtesy of GENIVAR)

6.4.3 Available data

Some data are easily retrieved such as river bathymetry, upstream and downstream boundary conditions, riverbed composition, and geometric cross-sections describing the river. All these data were secured by GENIVAR thanks to the field campaigns carried out on the site.

Turbidity

Turbidity surveys were done at various locations along the river. A turbidity probe was installed in the Aux Sables River near Highway 70, 1 km upstream of the Jonquiere water intake. Moreover, turbidity measurements took place at three sites (Figure 6.7), the Jonquiere water intake (site A), near the CEPAL rapids (site B) and just upstream of the Pibrac Bridge (site C).

Boundary conditions

The upstream condition is located just after the Pibrac Bridge, at km 10.3, where the discharge will be specified. The river discharge is controlled by the Pibrac dam, at km 11.1. During the work period, the released discharge from Lake Kenogami is quasi-permanent, varying between 5 m³/s and 50 m³/s. For the downstream boundary condition, at the Jonquiere dam and km 3 (Figure 6.2), the water level is maintained constant by the dam at 140.2 m.

There is no internal boundary condition for the water phase. A suspended sediment concentration depending on the water discharge, will be specified at km 5.4.

Riverbed composition

The armored riverbed is made of large pebbles with the exception of the rapids, where water flows over bedrock. There is no bed load sediment transport. During riverbed excavation cohesive sediments beneath the armored layer are released and transported downstream.

Cross-sections

The river reach from the Pibrac Bridge at km 10.3, to the Jonquiere dam at km 3.0, is discretized into 86 cross-sections provided by GENIVAR.

Water height

GENIVAR provided the water height corresponding to the following discharges: 5, 10, 15, 20, 25, 30, 35, 40, 45 and 50 m³/s.

6.5 Numerical model: UMHYSER-1D

For a constant water discharge released from Lake Kenogami, the convection-diffusion equation is solved to determine the concentration of suspended sediments at the Jonquiere water intake. To this end the software UMHYSER-1D, presented in a companion paper, is used. For unsteady sediment transport, UMHYSER-1D solves the convection-diffusion equation with source terms from sediment erosion/deposition:

$$\frac{\partial AC}{\partial t} + \frac{\partial QC}{\partial x} = \frac{\partial}{\partial x} \left(AD \frac{\partial C}{\partial x} \right) + \Sigma \quad (6.1)$$

where: C = depth averaged concentration; A = cross section area, Q = flow rate, D = longitudinal diffusion coefficient, a calibration parameter, and Σ = source (erosion, excavation, lateral inflow) and sink (deposition) terms for one sediment class.

6.6 Results and discussion

Numerical modeling of the Aux Sables River should answer the following question: under which hydraulic conditions (flows) would the concentration of sediments released at the Pibrac Bridge be lower than 19.29 mg/l at the Jonquiere filtration station (water intake)?

To answer this question, the following numerical simulations are made:

- Permanent flow modeling without sediment transport: this step is necessary for determining the Manning coefficients,
- Modeling of suspended transport (released sediments): first, calibrate the model to determine the right diffusion coefficient. Then, validate the model.
- Using the model to assess suspended sediment transport and answer the previous question.

6.6.1 Calibration and validation

Liquid phase

Knowing the flow discharge and corresponding water height, simulations are carried out for a steady flow at different discharge rates, and the results are compared to the water height measurements. For each of the available discharge rates (5, 10, 15, 20, 25, 30, 35, 40, 45 and 50

m³/s), the maximum difference between observed and simulated water height does not exceed 3 cm for the Manning's coefficients listed in Table 6.2. Figure 6.10 shows an example of the calibration for a discharge of 30 m³/s.

Table 6-2 Calibrated Manning's coefficients

River km	Right bank	River	Left bank
0 - 5.3	0.08	0.03	0.08
5.3 - 6.2	0.08	0.03	0.04
6.2 - 9.0	0.06	0.03	0.04
9.0 - 10.0	0.08	0.05	0.08
10.0- 10.3	0.06	0.02	0.06

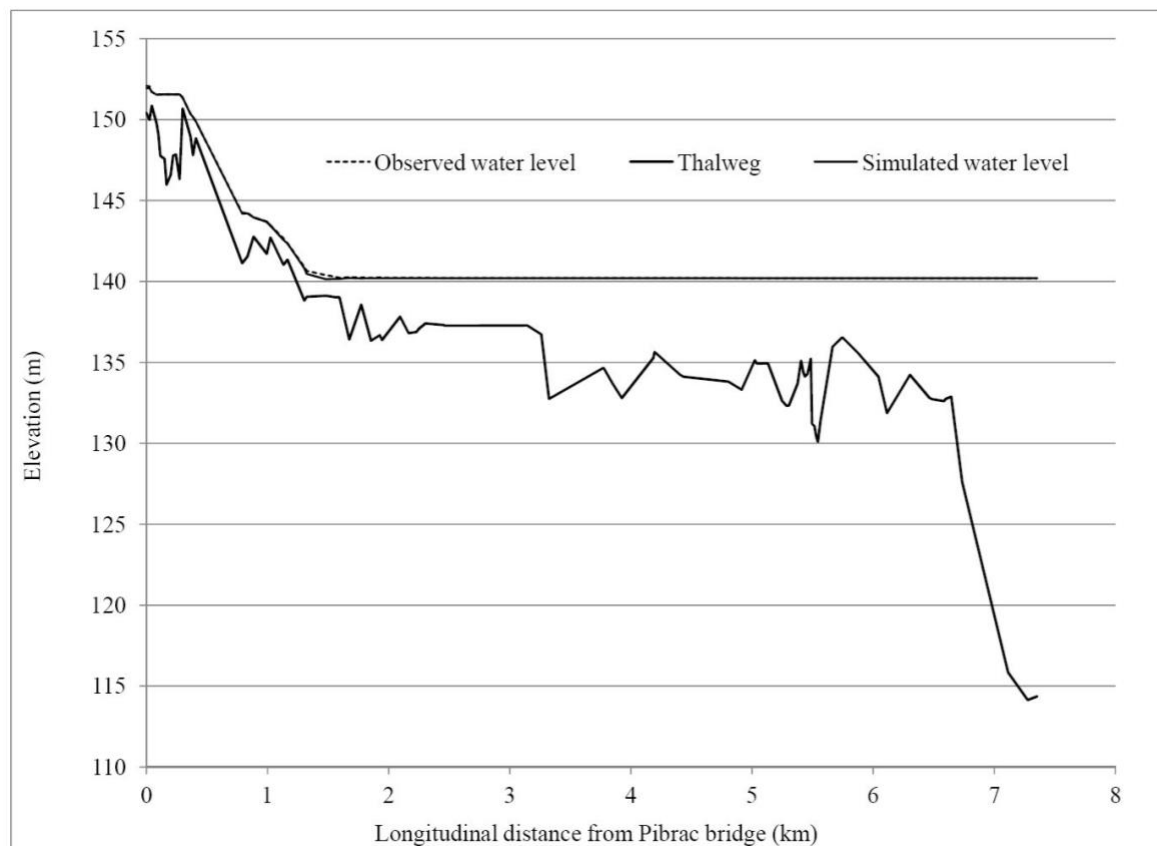


Figure 6-10 Example of calibration results for a discharge of 30 m³/s

Solid phase

From March 13 to 16, 2009, no rain was observed, thus the sediment input from the tributary at km 5.4 is assumed nil. The river discharge is almost constant, varying between 24.7 m³/s and 26.4 m³/s, and the observed turbidity, at km 6.15, varies between 1.43 and 1.95 NTU. The calibration is achieved by varying the diffusion coefficient given by Fischer's equation (Fisher, 1975):

$$D = 0.011 \frac{U^2 W^2}{H U_*} \quad (6.2)$$

Where: W= channel width, U = cross-sectional average velocity, H = water depth, and U* = shear velocity.

The best results (Figure 6.11) are obtained by changing the proportionality constant of Fisher's equation from 0.011 to 0.0135. Note that the simulated concentration starts at 0 mg/l since the model computations start with a nil concentration. Figure 6.12 shows a validation example for the period of March 2-3, 2009.

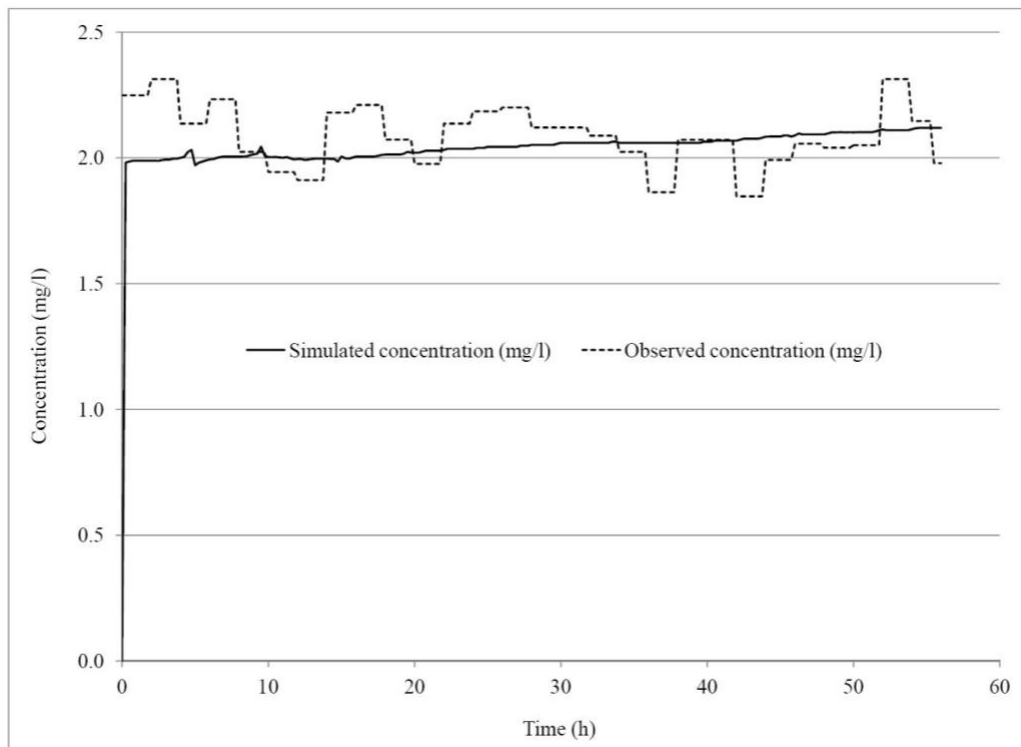


Figure 6-11 Calibration of the diffusion coefficient for a liquid discharge varying between 24.7 m³/s and 26.4 m³/s during the period March 13-16, 2009

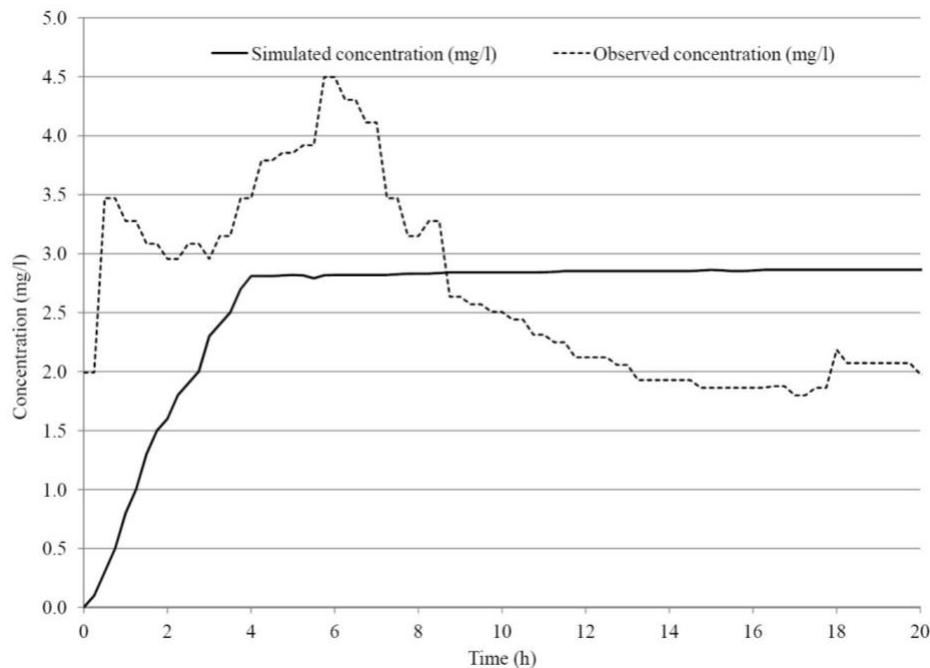


Figure 6-12 Validation of the calibrated model. Observed and simulated concentration from March 2, 22:00 to March 3, 18:00

6.6.2 Assessment of suspended sediment deposition

The calibrated and validated model will be used to find the maximum sediment concentration to be released at the bridge that will ensure the suspended sediment concentration at the Jonquiere water intake is less than 19.29 mg/l.

According to the available means on the site, the maximum possible quantity of suspended sediment to be released at the Pibrac Bridge is 53.80 tons per day and 2.2417 tons is dumped per hour. It is therefore considered that during the work, the maximum sediment discharge released will not exceed 2.5 tons per hour.

A series of simulations are performed as follows:

- a- The sediment discharge is constant at 2.5 tons/h;
- b- Several simulations with different water discharge are performed looking for the minimum discharge giving a concentration at the Jonquiere intake of 19.29 mg/l.
- c- Redo step b) with a smaller sediment discharge (2, 1.5, 1, 0.5 tons/h) looking for the corresponding minimum water discharge

Table 6.3 and Figure 6.13 summarize the results. According to the available river discharge capacity controlled by the Pibrac dam, one can choose the maximum quantity to be released into the river.

Table 6-3 Maximum tolerable released sediment load into the Aux Sables River

Upstream released sediment (ton/h)	Minimum water discharge (m ³ /s)	Concentration at water intake (mg/l)
0.5	5	19.15
1.0	10	19.10
1.5	18	19.13
2.0	25	19.08
2.5	32	18.96

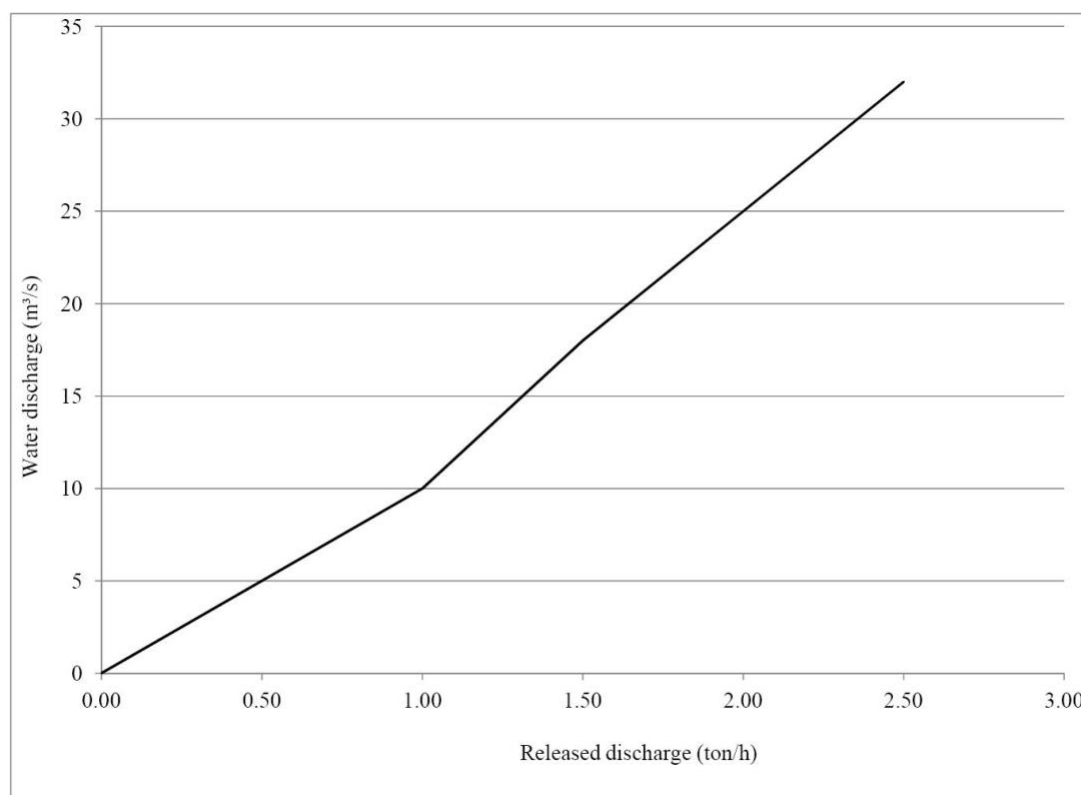


Figure 6-13 Maximum sediment load released upstream corresponding to the minimum water discharge of the Aux Sables River

6.7 Conclusion

The severe rainstorm of July 1996 caused extreme flooding in the Saguenay region, Quebec. Among the storm's numerous consequences, along the Aux Sables River, the flood discharge exceeded the design or available spilling capacity at two run-of-the river dams. Flooding in an urban area damaged or destroyed buildings and infrastructure. The inundation discharge threshold was exceeded by a factor of 3.8.

The retained option for flood mitigation on the Aux Sables River consists of: increasing the flow downstream of the Pibrac Bridge by excavating the left bank for a length of approximately 60 m; increasing the flow to the right of the bridge by excavating the riverbed approximately 2 m and excavating the left bank; replacing the Pibrac Bridge with a span of 50 m (without a central pillar) and increasing the flow upstream of the bridge by excavating a 440 m channel, with a maximum width of 60 m, and requiring riverbed excavation to vary from 0.5 to 2 m.

This option raised a pollution risk at the Jonquiere city water intake, located a few kilometers downstream from the work area. Thanks to a newly developed software, UMHYSER-1D, an Aux Sables River suspended sediment impact assessment provides the maximum permissible sediment discharge that will avoid any pollution risk for the population of Jonquiere city. Using UMHYSER-1D to mitigate water pollution risk confirms the important role of numerical modeling in solving complex engineering problems.

6.8 Acknowledgement

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CHAPTER 7 GENERAL DISCUSSION

This thesis deals with the development, the validation and application of a new 1D hydro-morphological model, UMHYSER-1D, for use in natural rivers and manmade canals. It is a mobile boundary model with the ability to simulate steady or unsteady flows, internal boundary conditions, looped river networks, cohesive and non-cohesive sediment transport, and lateral inflows. It borrows some ideas and some computer code from the quasi-state model MHYSER (Mahdi, 2009), but it has been mostly rewritten and updated.

Many other sediment and water routing models, such as the HEC-RAS (US Army Corps of Engineers; Brunner, 2016), FLUVIAL-12 (Chang, 1998), CONCEPTS (Langendoen, 2000), and CCHE1D (Wu and Vieira, 2002) have also been developed to solve one-dimensional alluvial river problems. These models have many of the same capabilities as UMHYSER-1D. However, few differences are present: bedload, suspended load, and mixed load are computed using the same partial differential equation. This equation automatically switches to suspended load, bed load, or mixed load depending on a transport mode parameter consisting of local flow hydraulics. The second important difference, compared to the above models, resides in the used numerical scheme, able to model sub-, super- and trans-critical flow conditions, while the above-mentioned models use Preissmann scheme.

This being said, UMHYSER-1D is still a 1D model. It should not be applied to situations where a two-dimensional or three-dimensional model is needed for detailed simulation of local hydraulic conditions. Appendix G shows a case study where the use of UMHYSER-1D is unsuitable. In this case, a 2D model is used to simulate sediment deposition in a confluence of two rivers. In fact, in this case, 2D phenomena such as secondary currents and transverse sediment movement cannot be ignored.

UMHYSER-1D has been validated using several cases from the literature. Appendix A presents another validation case from the Users' Manual of SRH-1D version 4.0 (Greimann and Huang, 2018). This case validates numerically UMHYSER-1D simulating sediment transport in a simple trapezoid channel. Its results compare very well to those of SRH-1D. Appendices B and C detail other results of numerical validations, while Appendix D tests the implemented minimization

theories of UMHYSER-1D. In fact, the simulation of the experimental validation case of Chapter 4 is done using the three theories. The results show that in this case the minimization of total stream power improves the results at the knick-point, but the bed slope and energy slope minimization have no effect on the results. This conclusion should be taken with great care: at the knick-point, the flow equations are not valid (violation of small slope and hydrostatic pressure hypotheses). The minimization of total stream power minimizes an integral over the total length of the river reach which has a smoothing effect on the solution.

Finally, UMHYSER-1D has been applied to two cases in this thesis. The first one is an extreme case study where the 1996 River Ha!Ha! flood was simulated. UMHYSER-1D capture the erosion/deposition trend. More importantly, forced data validation after showing huge difference between the results and the “observed” data (more than 20 m) at a cross-section, to find out that this difference occurred because this cross-section from a published data set doesn’t belong to the initial water course. Other researchers used this same data set to validate their models and they proudly show the perfect match between the results of their models and the “observed” ones (see for example El Kadi and Paquier, 1999).

The second application is a real engineering case. Using UMHYSER-1D, suspended sediment impact assessment study provided the maximum permissible sediment discharge to avoid any pollution risk for the population of Jonquiere city, downstream the river reach. This is another example showing the important role of numerical modeling in solving complex engineering problems.

This new software has great potential. At the research level, it can be used as a numerical laboratory to test or validate new findings, and at the practicing engineers' community level, it can be used to solve complex real river engineering cases.

CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

The main objective of the present thesis is the development of a new 1D hydro-morphological model for unsteady flow conditions. UMHYSER-1D is validated, its performance is verified, and its capabilities are promising. A summary of my research contributions is made, followed by an inventory of the limitations of UMHYSER-1D, and finally recommendations for further research are listed.

8.1 Contributions

The main contributions of our research work are mainly defined through the initial specific objectives of this project:

- **Use of a performant numerical scheme, different of the classical Preissmann one. This is part of the creation of a numerical laboratory objective:** After a review of numerical schemes available in the literature, the NewC was chosen and used to numerically solve the shallow water equations. This is a plus in computational hydraulics where most of the available software use the same numerical scheme, Preissmann scheme.
- **Use of the same sediment convection-diffusion for different sediment transport modes (bedload, suspension load or total mode) and types (cohesive or not):** This is a very good contribution in the field: it unifies the different sediment transport modes by one modeling partial differential equation. In fact, the convection-diffusion equation describing sediment transport integrates the two sediment modeling approaches: depending on hydraulics conditions, the model switches from bedload to suspended load or mix load calculations.
- **Model flows and sediments in rivers and channel networks with or without movable boundaries:** This option is added as an extra to make UMHYSER-1D more general as a numerical tool. Most of engineering cases don't deal with network sediment transport, but in the development process this part was added to improve the capabilities of the model.
- **Model both steady and unsteady flow conditions, cohesive or non-cohesive sediment transport:** this is the core of the project. In fact, once done, all the other options of

UMHYSER-1D were added. Even if UMHYSER-1D is equivalent to MHYSER, in steady flow conditions, most of the code was rewritten, and MYHSER is considered to be a special case of the steady flow conditions.

- **Model rivers' width changes using at least one minimization theory and riverbank retreat method. This objective improves the capabilities of the present one-dimensional numerical laboratory:** Three options are present in UMHYSER-1D: minimization of total stream power, of bed slope, and of energy slope. Moreover, riverbank retreat is done using the angle of repose adjustments. This is a simple yet an effective approach.
- **Consider hydraulics structures such as weirs, bridges, and gates along with other internal boundary conditions (such as time-stage tables, rating curves):** This was successfully implemented. A special attention is drawn to the implemented river stage upstream boundary condition because very few models allow this upstream condition (the very popular condition is water discharge). In fact, on some rivers, such as the Richelieu River (QC), the most upstream station, has a water stage gauge, while several kilometers downstream, it has a discharge gauge.
- **Account for bed material mixing. This is an important feature for riverbed sorting and armor modeling:** This is implemented using a proven algorithm to simulate bed material mixing process that occur in natural river systems. This option improves the capabilities of UMHYSER-1D
- **Add visualization windows to monitor the solution during the simulation. This saves time during numerical investigations:** when a simulation starts, a user defined number of windows open in the computer screen showing the river longitudinal profile along with user selected cross-sections. This way, the modeler can visualize the results during the simulation and there is no need to wait for the end of the simulation to open the output data file and go through the post-treatment to visualize the results. This option is also helpful with diverging solutions: no need to lose time waiting for diverging solutions.

- **The code structure of UMHYSER-1D:** adopting the concepts of sub-programs, modules, "subroutines" and "functions", makes it possible to test other results of theoretical or experimental research by incorporating them into the model's code.
- **Last but not least:** UMHYSER-1D is developed and ready to be used as numerical laboratory for fluvial hydraulics and/or a powerful tool to solve real engineering problems. UMHYSER-1D can be used for river morphological changes due to natural or human influences. It simulates unsteady, one-dimensional flow, graded-sediment transport, and bank-erosion processes in stream corridors. It computes channel evolution by tracking bed changes and channel widening and riverbank erosion. It is able to predict the dynamic response of flow and sediment transport to instream hydraulic structures. Selectively by size classes UMHYSER-1D simulates transport of cohesive and cohesionless sediments as suspended load, bedload or total load. UMHYSER-1D includes three minimization theories for channel geometry adjustments and allows for channel-boundary roughness varying along a cross section.

Finally, during the last four years, my research contributions have expanded to include a total of 3 journal papers and 2 conference papers. Moreover, I have 2 more submitted journal papers and 1 conference paper.

8.2 UMHYSER-1D limitations

UMHYSER-1D is an engineering tool for solving fluvial hydraulic problems with the following limitations:

- one-dimensional flow: it should not be used in cases where 2D flow conditions dominates.
- straight channels or channels of very low sinuosity;
- two types of upstream boundary conditions:
 - water discharge
 - river stage
- two types of downstream boundary conditions:

- rating curve
 - river stage
- five types of internal boundary conditions:
 - time stage table
 - elevation versus discharge table
 - weir
 - bridge
 - gate
- different hydraulic roughness for bed, banks, and floodplains
- total load sediment transport
- Suspended load sediment transport
- bedload sediment transport
- homogeneous cohesionless or cohesive bed-material in transverse direction
- 11 pre-defined sediment particle-size classes
- non-equilibrium transport of cohesionless and cohesive bed-material
- bed material mixing
- consolidation of deposited cohesive sediments is unavailable
- bank retreat based on angle of repose
- river adjustment based on three minimization theories (optional):
 - minimization of total stream power
 - minimization of bed slope
 - minimization of energy slope
- stream tube approach (optional)
- metric or imperial systems of units

8.3 Recommendations for further research

I may suggest few recommendations for short/mid term of UMHYSER-1D life span:

- 1- Graphical User Interface: UMYHSER-1D is a complex software. It has several input data groups (Appendix F). Some of them are required, others, are optional. Few of the optional input data may become required if other data are inputted. This is to say that GUI will ease the use of the software.

- 2- Riverbank enlargement

As mentioned in the literature review, bank erosion occurs through mainly two groups of processes: fluvial erosion and mass failure processes. The former is dealt with, in UMHYSER-1D, with the help of the angle of repose approach. Mass failure processes should be implemented using mechanistic approaches. The planar and circular failures approaches can be implemented in UMHYSER-1D and then compared. The method used by Langendoen (2000) can be easily implemented in UMHYSER-1D for planar failure modes. For circular failures, the approach by Mahdi (2003) can be successfully added to UMHYSER-1D capabilities. Appendix E presents the methodology to implement any mass failure mode in UMHYSER-1D.

- 3- Stream tubes approach

In UMHYSER-1D, the stream tubes approach is implemented. The modeler can choose a number of stream tubes to model a river in a quasi-2D way. It is a good opportunity to investigate deeply the limitation of this approach. The question to address is: to what extent this approach improves 1D modeling? This investigation can be a good master thesis.

- 4- Heat transfer implementation in UMHYSER-1D

Since the heat transfer 1D equation has the same form of the sediment convection-diffusion equation used by UMHYSER-1D, the implementation of 1D heat transfer in the model should be easily done. The only difference between the two equations reside in the source terms. The same numerical technics to solve the convection-diffusion equation can be used.

5- Contaminants transport in UMHYSER-1D

A contaminants transport equation described by a convection-diffusion equation can be implemented in UMHYSER-1D, following the same way as for 1D heat transfer in rivers. The only difference between the two equations reside in the source terms. The same numerical technics to solve the convection-diffusion equation can be used.

6- Automatic calibration tool

UMHYSER-1D is a complex tool incorporating many parameters (Manning coefficients, local loss coefficients, sediment transport equation and their parameters...). When used, its calibration will be a very challenging task. I strongly recommend the development of a new tool dedicated to automatic calibration.

7- Numerical laboratory

UMHYSER-1D is ready to be used and/or improved. Many opportunities can be explored with the model. As examples, one can implement new sediment transport equations to be compared to the existing ones or to be used in river modeling, investigate the performances of new bed sorting algorithms, and new numerical schemes can be tested.

8- River engineering software

UMHYSER-1D is ready to be solve in real engineering problems. I recommend the development of workshops to disseminate it among hydraulics engineer's community. It has few added capabilities compared to the existing ones such as the minimizations theories for river morphology adjustment.

9- Validation

UMHYSER-1D has been validated numerically using the software SRH-1D. It is recommended to use other available 1D models such as HEC-RAS the widely used one. Moreover, using a 2D model as a reference can help validating UMHYSER-1D since 2D models are more precise.

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APPENDIX A VALIDATION CASE 1: TRAPEZOID CHANNEL

This example, taken from the Users' Manual of SRH-1D version 4.0 (Greimann and Huang, 2018), is used to validate numerically UMHYSER-1D under steady flow condition in a simple channel. The results of SRH-1D and UMHYSER-1D are compared.

The case: SRH-1D and UMHYSER-1D are used to simulate sediment transport in a simple trapezoid channel (Figure A.1), with a width of 200 ft and side slopes of 1V:2H. The channel has a length of 5000-ft and a slope of 0.001. The water discharge is 14,900 ft^3/s and the downstream water surface elevation is set at normal depth. The input sediment load is 48420 tons/day and 11 sediment sizes are used ranging from silt to small cobble. Two bed layers, one active layer and one inactive layer, are used.

In both models, SRH-1D and UMHYSER-1D, the upstream and downstream cross sections were inputted and then 9 cross sections were interpolated between them.

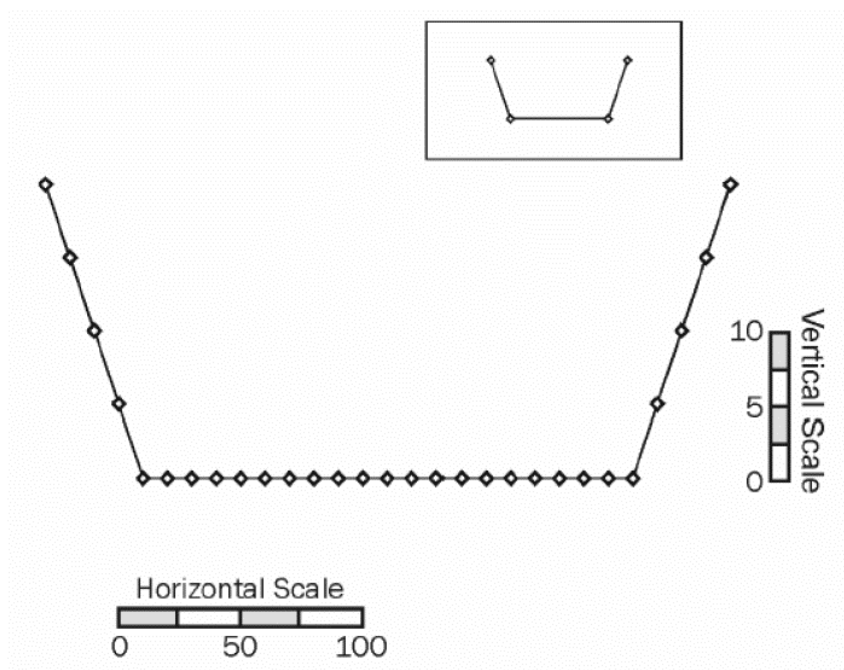


Figure A.1 Sketch showing the discretization points used in the cross-section template to define the channel. The smaller insert shows an equivalent cross section using the minimum possible number of discretization points (Greimann and Huang, 2018)

Results

Figure A.2 shows the initial and final bed elevation and water surface elevation profiles. The incoming sediment load being near equilibrium condition, the bed elevation change is small. The results of UMHYSER-1D are identical to those of SRH-1D except at the first cross-section where the difference is less than 0.2 ft (6 cm) for the thalwegs and 0.16 ft (4 cm) for the water surface elevations.

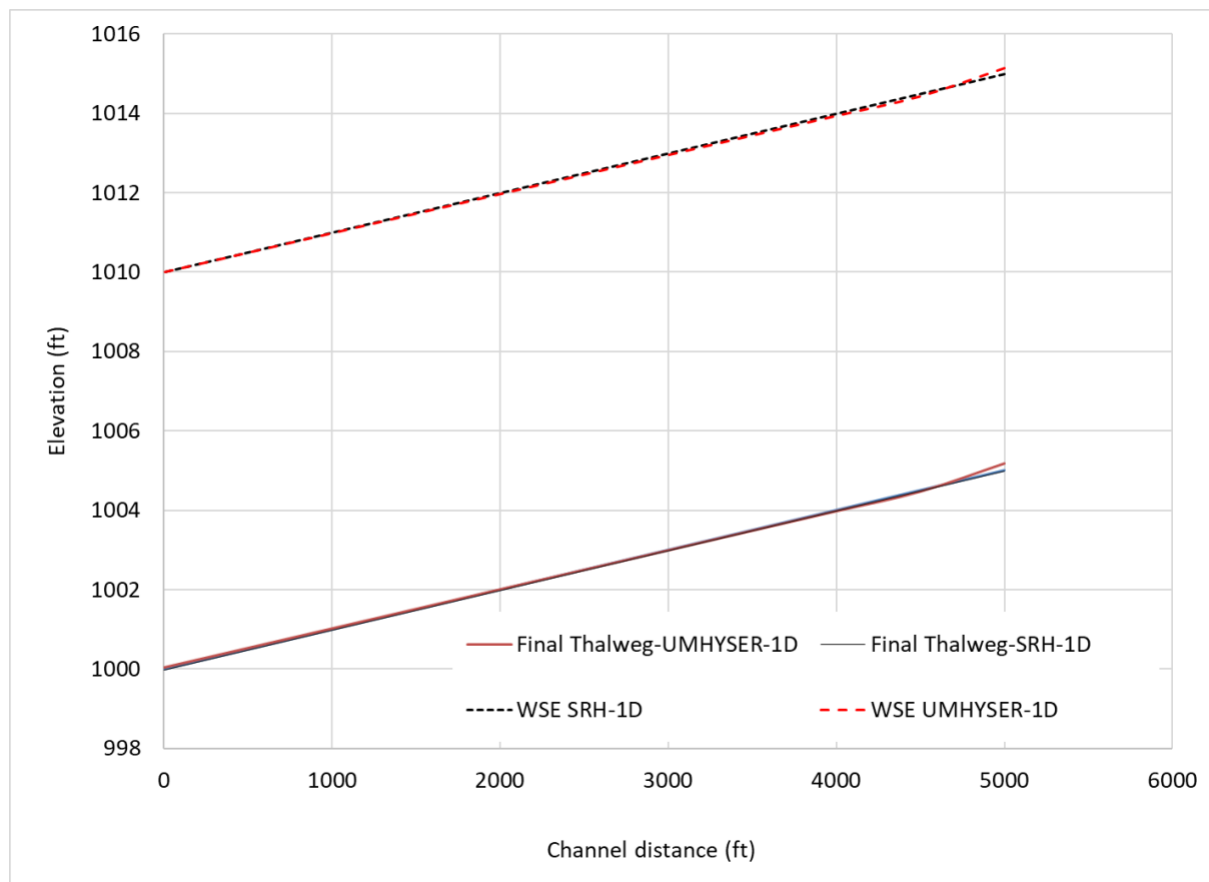


Figure A.2 Bed elevation and water surface by SRH-1D and UMHYSER-1D

References

Greimann B, Huang JV (2018). SRH-1D 4.0 User's Manual (Sedimentation and River Hydraulics-One Dimension, Version 4.0). Technical Report SRH-2018-07, Technical Service Center, US Bureau of Reclamation, Denver (CO)

APPENDIX B VALIDATION CASE 2: CHANNEL NETWORK

This appendix is part of the published conference paper: *AlQasimi E, Mahdi T (2018). Unsteady Model for the Hydraulics of Sediments in Rivers. 26th Annual Conference of the Computational Fluid Dynamics Society of Canada. June 10-12, Winnipeg, MB, Canada*

This example, taken from the Users' Manual of SRH-1D version 4.0 (Greimann and Huang, 2018), is used to validate numerically UMHYSER-1D under steady flow condition in a network of 4 channels. The results of SRH-1D and UMHYSER-1D are compared.

The case: SRH-1D and UMHYSER-1D are used to simulate sediment transport in a network composed of 4 trapezoidal channels (Figure B.1).

Each channel (1 mile long) has a trapezoidal cross section of 200 ft bottom width and side slopes of 1V:2H. The upstream water discharge is 14,900 cfs and the downstream water surface elevation is fixed at 1010 ft. The input sediment load is 57,709 ton/day and 11 sediment sizes are used, ranging from silt to small cobble. Two bed layers, one active and the other inactive, are used.

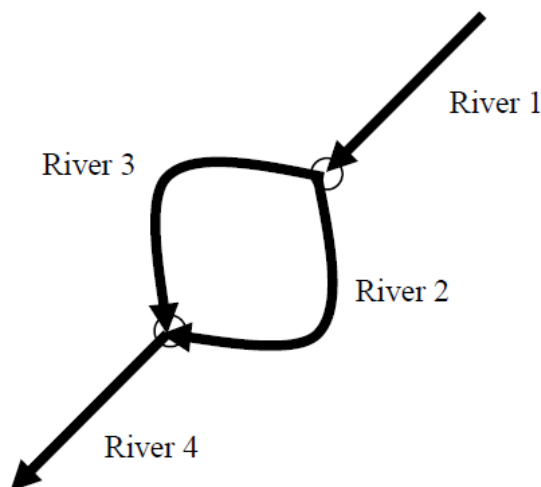


Figure B.1 Sketch of the river network (Greimann and Huang, 2018)

Results

Figure B.2 shows the initial and final bed elevation and water surface elevation profiles obtained by SRH-1D and UMHYSER-1D.

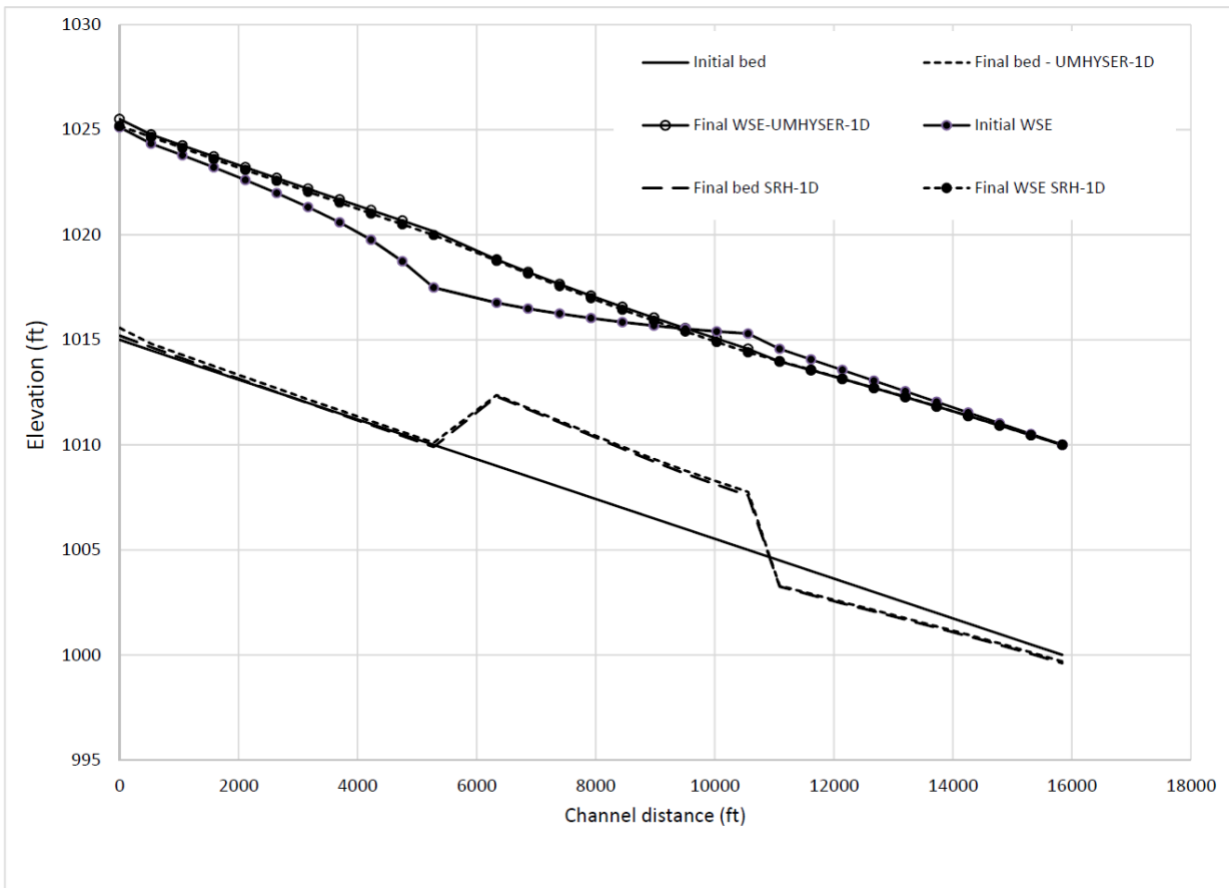


Figure B.2 Comparison of bed and water surface elevations by MHYSER-1D and SRH-1D (AlQasimi and Mahdi, 2018)

Rivers 2 and 3, identical in calculations, are the middle section from a river distance of 5800 feet to 10560 feet in Figure B.2. Sediment deposition occurs in rivers 2 and 3 due to reduced flow rate and lower energy slopes. River 4 experiences erosion because there is not enough sediment supply and some of the sediments are deposited in rivers 2 and 3. Moreover, the deposition in river 2 and 3 raises the water surface elevation in river 1, causing sediment deposition in river 1. Figure B.3 shows the comparison of the simulation of the first cross section by SRH-1D and UMHYSER-1D.

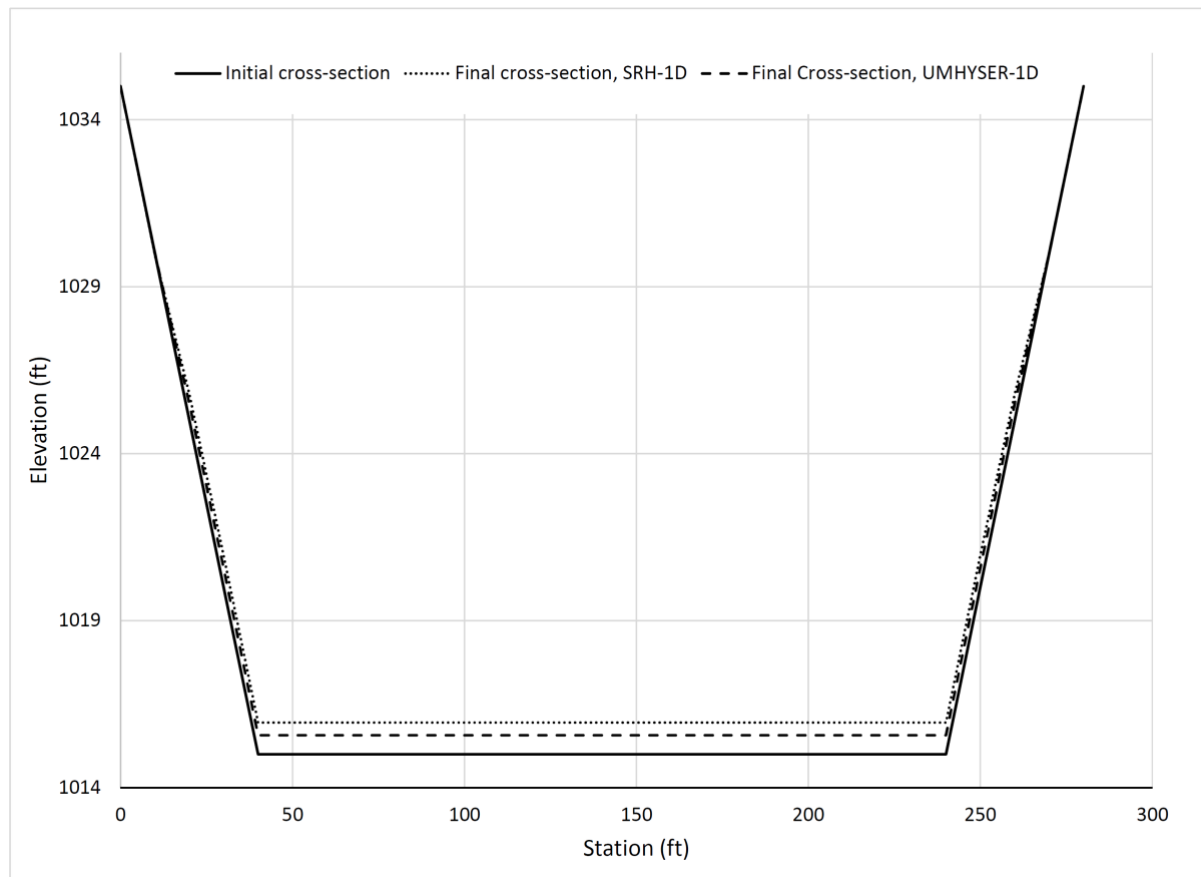


Figure B.3 Comparison between the results of SRH-1D and UMHYSER-1D at the first cross-section (AlQasimi and Mahdi, 2018)

The results of UMHYSER-1D are the same as those of SRH-1D except at the first cross-section (Figures B.2 and B.3) where the difference between the water surface elevation and bed elevations are 3.8 in and 4.44 in which represent a difference of 4% and 9% respectively. These differences are very acceptable.

References

Greimann B, Huang JV (2018). SRH-1D 4.0 User's Manual (Sedimentation and River Hydraulics-One Dimension, Version 4.0). Technical Report SRH-2018-07, Technical Service Center, US Bureau of Reclamation, Denver (CO).

APPENDIX C VALIDATION CASE 3: LONG RIVER REACH

This example, taken from the Users' Manual of SRH-1D version 4.0 (Greimann and Huang, 2018), is used to validate numerically UMHYSER-1D under unsteady flow and sediment transport conditions in a long river reach. The results of SRH-1D and UMHYSER-1D are compared.

The case: The US Bureau of Reclamation modeled the reach of the Rio Grande from San Acacia Diversion Dam downstream to the Narrows of Elephant Butte Reservoir (Holmquist-Johnson, 2005), a distance of about 70 miles, which has changed significantly through recorded history from the early 1900's to the present (Figure C.1).



Figure C.1 Location of the modeled river reach: from San Acacia to Elephant Butte Reservoir in New Mexico, USA (adapted from Padilla and Baird, 2010)

UMHYSER-1D uses the input data of SRH-1D to simulate the historic period 1992-2002 of the Rio Grande reach from San Acacia Diversion to Elephant Butte Reservoir. This reach includes:

1. Downstream reservoir with variable water surface elevation downstream boundary condition
2. Cohesive sediment transport
3. Variable transport parameters
4. Four lateral inflows with sediment

Results

Figure C.2 shows that UMHYSER-1D gives almost the same results as SRH-1D, as can be seen in Figure C.3 comparing the final simulated thalwegs and the water surface elevations (WSE) of SRH-1D and UMHYSER-1D. The maximum differences, from Figure C.2, are observed at cross-sections 100 and 183. Figures C.4 and C.5 show these differences which are very acceptable (the maximum difference is less than 5% at cross-section 183, Figure C.5).

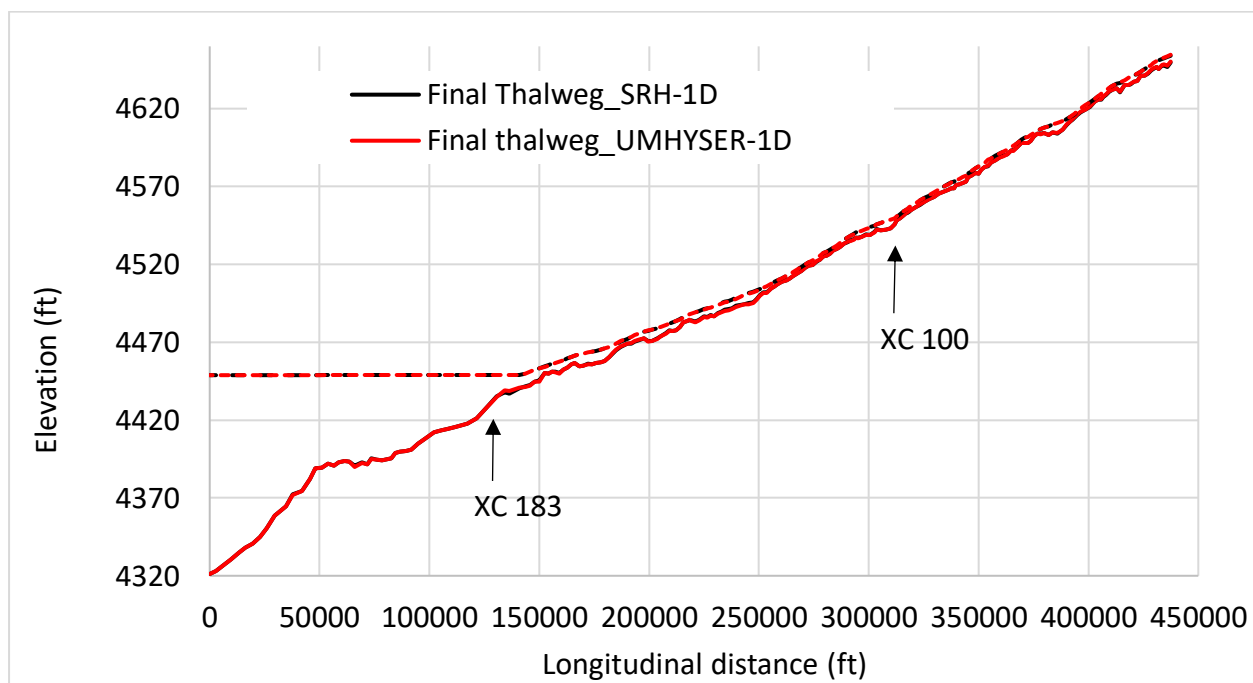


Figure C.2 Comparison of longitudinal profiles by SRH-1D and UMHYSER-1D

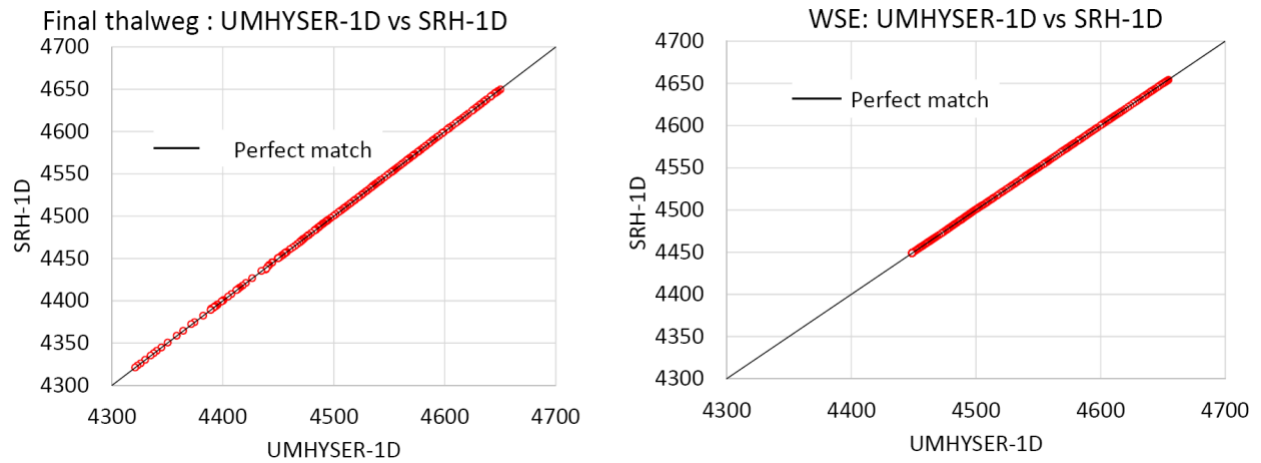


Figure C.3 Comparison of longitudinal profiles by SRH-1D and UMHYSER-1D

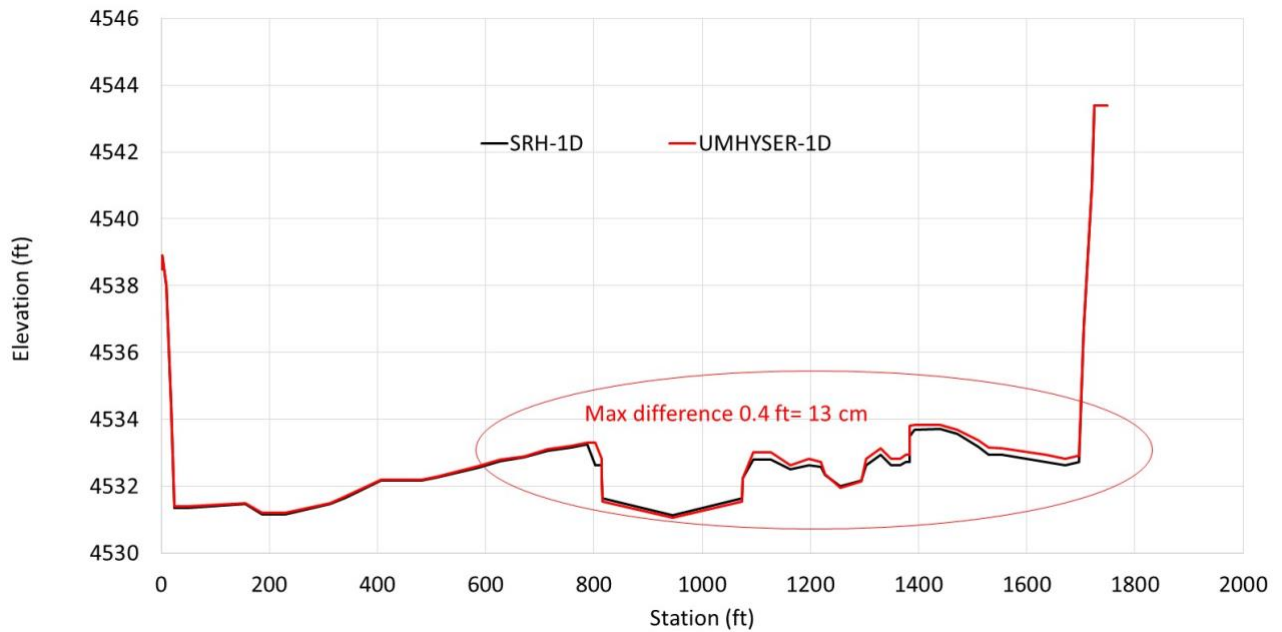


Figure C.4 Cross-section 100 evolution simulated by SRH-1D and UMHYSER-1D

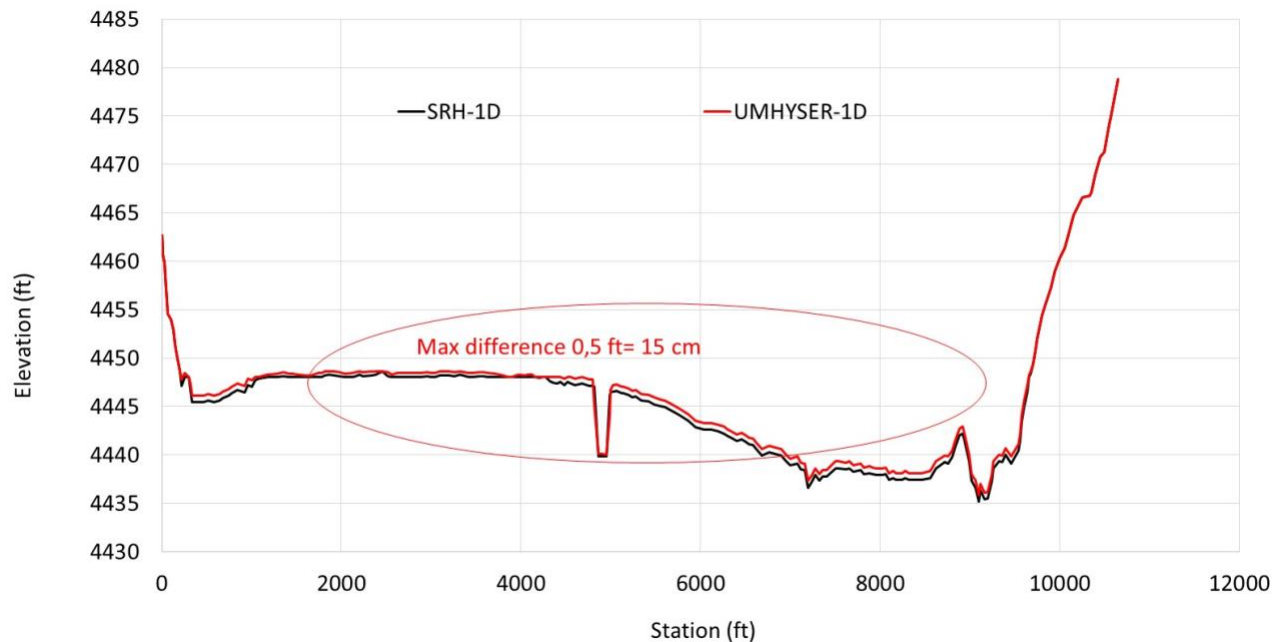


Figure C.5 Cross-section 183 evolution simulated by SRH-1D and UMHYSER-1D

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APPENDIX D COMPARISON OF THREE MINIMIZATION THEORIES FOR RIVER MORPHOLOGICAL ADJUSTMENTS

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Abstract: For river morphology changes, several minimization theories exist allowing bed and width adjustments. Three of these theories, namely minimization of bed slope, minimization total stream power and minimization of energy slope, are compared in this study using an experimental study. The bed slope minimization has no effect on thalweg evolution and energy slope minimization doesn't improve significantly the results. The minimum total stream power option improved the simulation results at the slope change only. It has to be noted that at the slope break, the total stream power minimization smooths the numerical solution even if, at this location, the de Saint Venant equations are not valid since the slope is not small anymore and the hydrostatic pressure distribution is violated.

D.1 INTRODUCTION

Several minimization theories exist for the adjustments of river's bed and width. Three of these theories are compared in this paper using an experimental study by Cantelli et al. (2004).

Minimization of total stream power (Yang, 1972): if lower total stream power is the result of alteration of the channel widths, then channel adjustments are made in the lateral direction. Otherwise, the adjustments progress in the vertical direction.

Minimization of energy slope (Chang, 1988): If the energy slope at a cross-section is greater than the weighted average energy slope of its adjacent sections, then the channel width at this section is reduced during deposition or the depth is increased during erosion. However, if the energy slope is smaller, the channel depth at this section is decreased during deposition or an increase of width occurs during erosion.

Minimization of bed slope: If the bed slope at a cross-section is greater than the weighted average bed slope of its adjacent sections, then the channel width at this section is reduced during deposition, or the depth is increased during erosion. Otherwise, the channel depth at this section is decreased during deposition or the width is increased during erosion.

The methodology is described in Section 2, while the results and discussion section are presented in Section 3, followed by the conclusion.

D.2 METHODOLOGY

Figure D.1 summarizes the experimental case of Cantelli et al. (2004). The longitudinal profile has 2 different slopes and the material on the flume's bed is uniform (d_{50} between 0.79 mm to 0.81 mm). The upstream water and sediment discharges are 0.3 l/s and 7.5 g/h respectively.

The numerical modeling is done using the model UMHYSER-1D by AlQasimi and Mahdi (2018, 2019) where the three minimization options are implemented. The model is run for each minimization option and the results are compared. UMHYSER-1D solves the de Saint Venant equations and a convection-diffusion equation for sediment mass conservation. For the present case, even if the bed slope smallness is locally violated, the de Saint Venant equations are used in this case as an extreme case to show the performances of the model.

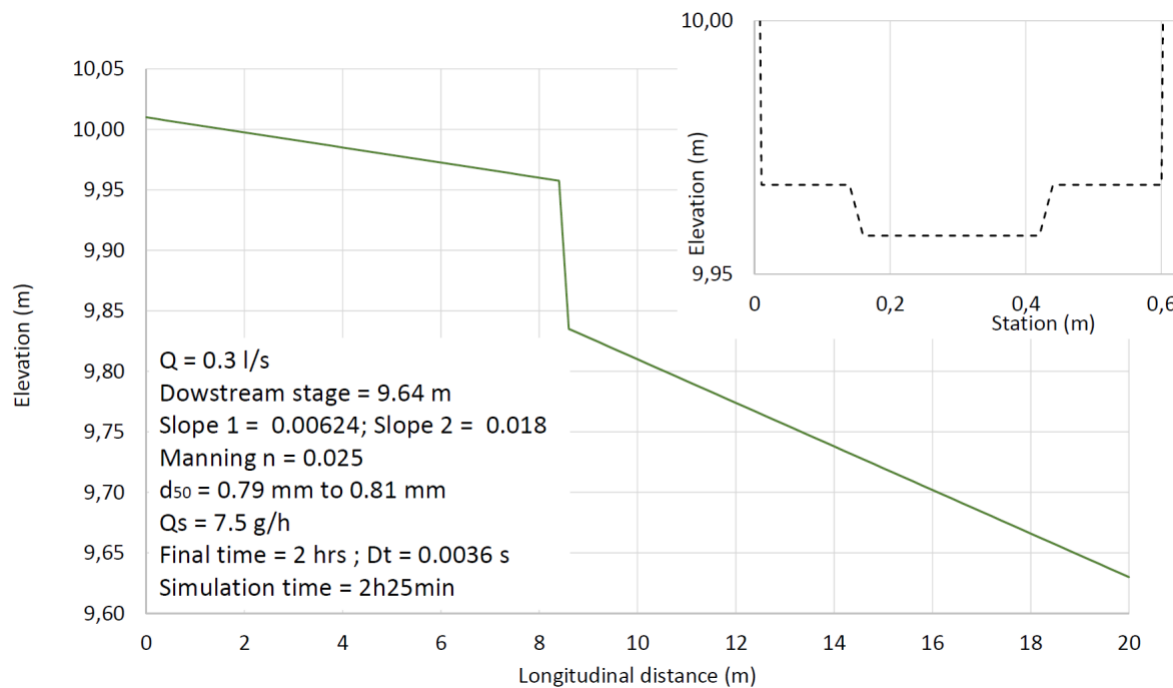


Figure D.1 Initial longitudinal profile of the experimental flume and typical cross-section.

D.3 RESULTS AND DISCUSSION

Figure D.2 shows simulated longitudinal profiles without minimization. The longitudinal profile evolution trend is well captured by UMHYSER-1D. However, at the slope break, the bed evolution is surely false. At the knickpoint, the de Saint Venant equations are not valid, but the numerical algorithm minimizes error growth which make UMHYSER-1D able to model a river reach with sudden thalweg changes. Overall, UMHYSER-1D is able to well reproduce the experimental results except at the slope break.

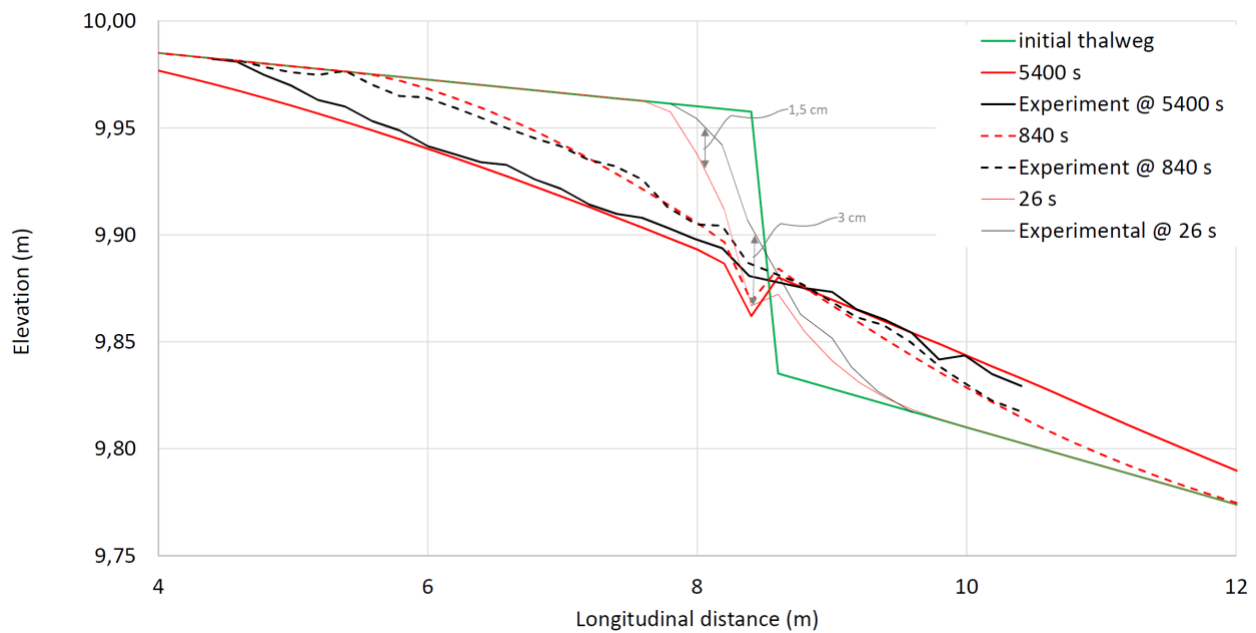


Figure D.2 Thalweg evolution without minimization options

Figure D.3 shows simulated longitudinal profiles with three minimization options: minimization of bed slope, minimization of total stream power and minimization of energy slope. The best results are achieved by the minimum total stream power option while bed slope minimization has no effect and energy slope minimization doesn't improve significantly the results.

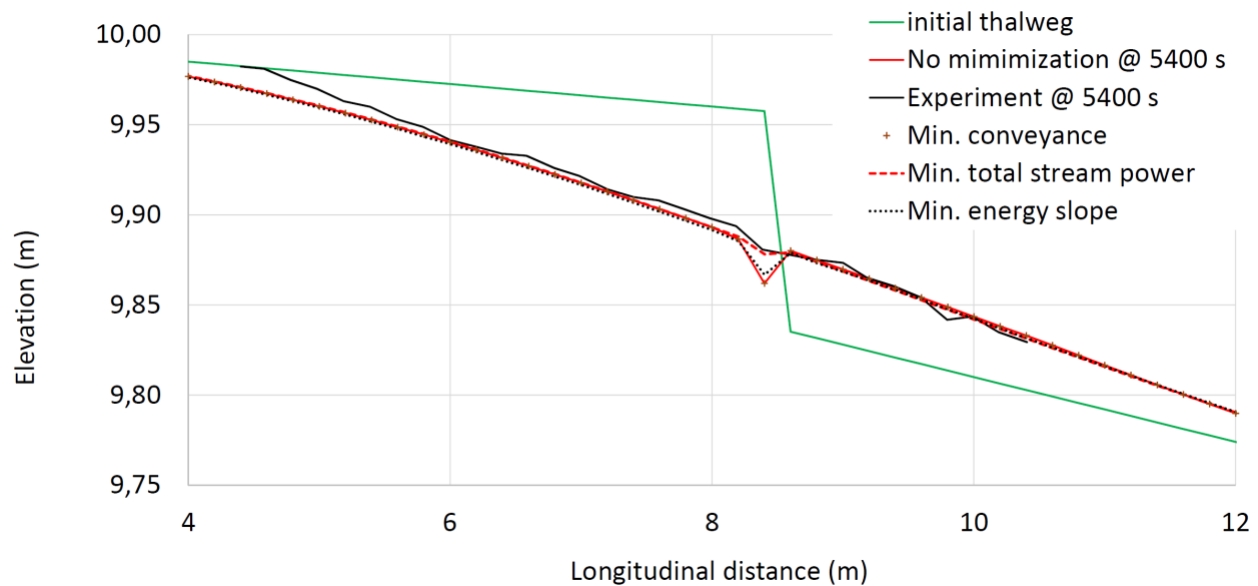


Figure D.3 Thalweg evolution using three minimization options

D.4 CONCLUSION

To improve numerical modeling of rivers' longitudinal profile evolution, three minimization options, minimum of bed slope, minimum total stream power and minimum of energy slope, were compared in this study using an experimental study.

The bed slope minimization has no effect on thalweg evolution and energy slope minimization doesn't improve significantly the results. The minimum total stream power option improved the simulation results at the slope change only: the difference is about 30%. It has to be noted that at the slope break, the total stream power minimization smooths the numerical solution even if, at this location, the de Saint Venant equations are not valid since the slope is not small anymore and the hydrostatic pressure distribution is violated. More investigations are needed to demonstrate the role of minimization theories to predict river morphology under unsteady flow conditions.

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APPENDIX E CIRCULAR FAILURE IMPLEMENTATION

Figure E.1 presents the methodology for a successful implementation of the circular failure mode in UMHYSER-1D. This methodology is adapted from Mahdi (2003, 2007)

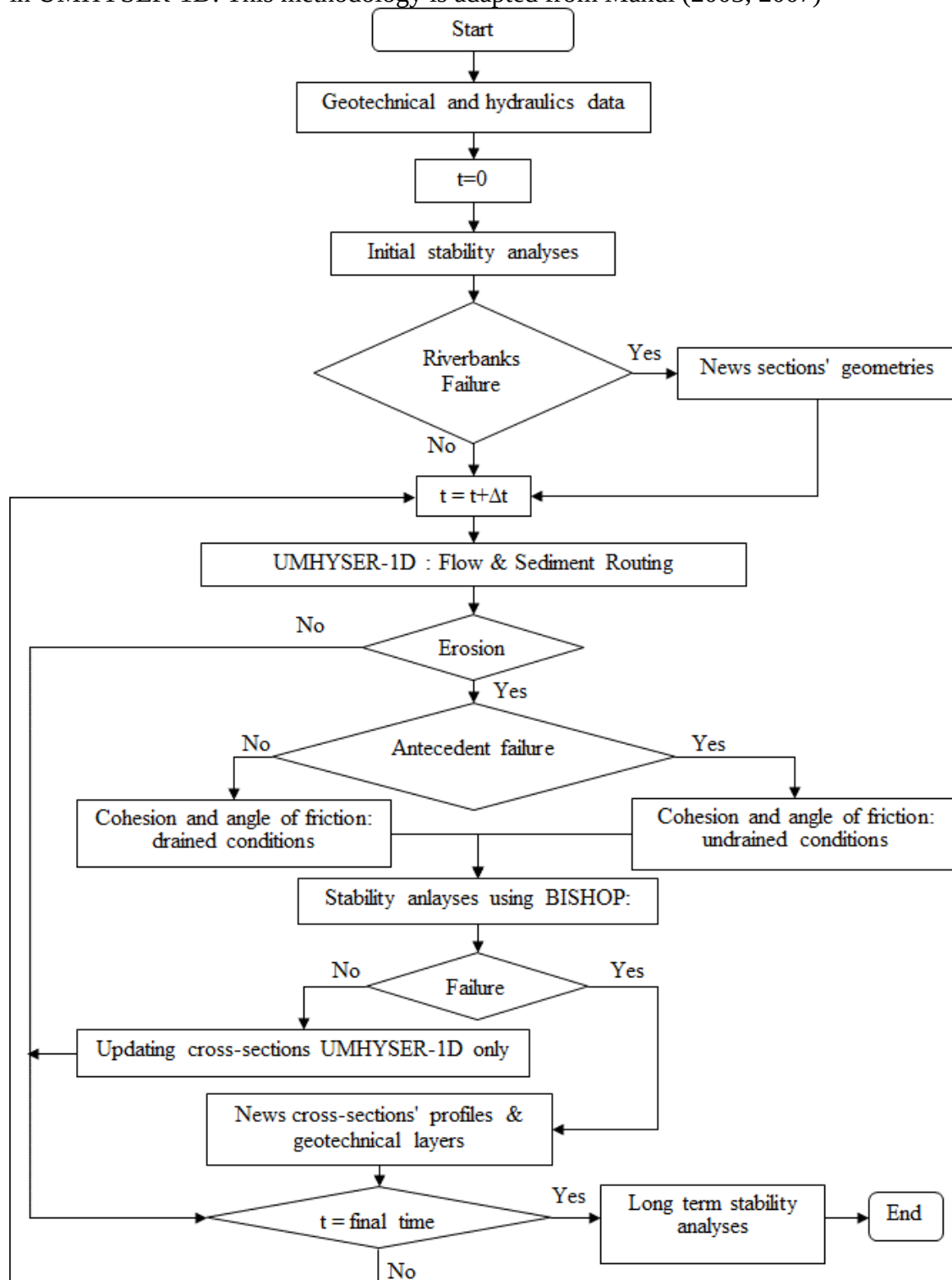


Figure E.1 Detailed analysis methodology to implement circular/planar failure in UMHYSER-1D

APPENDIX F UMHYSER-1D INPUT DATA GROUPS

The input data necessary to run the UMHYSER-1D model are separated into 14 data groups:

- **WATER TEMPERATURE**
 - Water temperature
- **MODEL PARAMETERS**
 - Title of case study
 - Solution parameters (dimensions, convergence criteria, θ)
 - Time - time step
- **CHANNEL GEOMETRY AND FLOW CHARACTERISTICS**
 - Number and name of river
 - Initial condition
 - Cross section location & geometry
 - Roughness coefficients
 - Bank location & cross section endpoint location
 - Cross section energy loss coefficient
- **UPSTREAM BOUNDARY CONDITIONS**
 - Junction
 - Time-stage table
 - Time-discharge table
- **DOWNSTREAM BOUNDARY CONDITION**
 - Junction
 - Time-stage table
 - Time-discharge table
 - Discharge-stage table
 - Weir
 - Normal depth
 - Rating curve
- **INTERNAL BOUNDARY CONDITIONS**
 - Location and type
 - Time-stage table
 - Time-discharge table
 - Discharge-stage table
 - Weir
 - Normal depth
 - Rating curve
 - Gate
- **LATERAL FLOW INPUTS**
 - Location and type
 - Time-discharge table

- **SEDIMENT MODEL PARAMETERS**
 - Sediment solution parameters (convergence criteria, θ)
 - Sediment size group

- **SEDIMENT BED MATERIAL**
 - Bed thickness
 - Location of size fractions
 - Sediment size fractions

- **SEDIMENT TRANSPORT PARAMETERS**
 - Number of sub-channels
 - Minimization option
 - Sediment transport equation
 - Sediment transport properties (angle of repose, active layer thickness, diffusion coefficient)

- **SEDIMENT BOUNDARY CONDITIONS**
 - Junction
 - Sediment transport equation
 - Rating curve
 - Flow-sediment discharge table
 - Time-discharge table
 - Sediment size distribution

- **LATERAL SEDIMENT INFLOWS**
 - Location of lateral sediment input
 - Rating curve
 - Flow-sediment discharge table
 - Time-discharge table
 - Time-discharge table for each size fraction
 - Sediment size distribution

- **COHESIVE SEDIMENT PARAMETERS**
 - Deposition parameters
 - Erosion parameters
 - Fall velocity
 - Density in bed
 - Dry bulk density in bed

- **EROSION AND DEPOSITION LIMITS**
 - Bed limitation locations
 - Bed limitations

APPENDIX G RIVER'S CONFLUENCE MORPHOLOGICAL MODELING USING SRH-2D

Rivers' confluence morphological modeling using SRH-2D

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Abstract. Efficient river morphology modeling is needed for hydraulics engineers. Several one-dimensional software supporting sediment transport are available and widely used, regardless of their limitations. However, two-dimensional sediment transport is still not widely used in practice for several reasons. Besides the high simulation times, the efficiency of such tools is not widely accepted. This paper tests the ability of the US Bureau of Reclamation two-dimensional sediment transport software, SRH-2D, to simulate the morphological evolution of two rivers' confluence. Even if no quantitative data are available to judge the results' quality, SRH-2D is able to capture the trend of the observed morphological evolution for this complex case.

Keywords: Rivers' confluence, 2D numerical simulation, SRH-2D, Morphological evolution.

G.1 INTRODUCTION

In river engineering, efficient morphological modeling tools are needed to predict river evolution after a dam construction or failure, for instance. Several one-dimensional sediment transport software have been developed during the last decades. For example, GSTARS (Yang and Simoes, 2002) and MHYSER (Mahdi, 2009) use the flow quasi-steadiness hypothesis to address erosion/deposition problems, while CONCEPTS (Langendoen, 2000), SRH-1D (Huang and Greimann, 2012), MIKE11 (DHI, 2009), CCHE1D (Vieira and Wu, 2002) or ISIS (Wallingford,

2001) solves the de Saint Venant and sediment transport equations. More recently, AlQasimi and Mahdi (2018a) developed UMHYSER-1D, Unsteady Model for HYdraulics of SEdiments in Rivers-One Dimensional, to predict river morphology changes and successfully applied it to the 1996 river Ha!Ha! flooding (AlQasimi and Mahdi, 2018b) and to address the Aux Sables river flood mitigation solutions (AlQasimi et al., 2018). For two-dimensional river morphology modeling, even if few software exist, their use is not widely spread within the practicing engineers' community.

In this work, two rivers' confluence undergoing flow regime changes, will be modeled using the USBR two-dimensional software SRH-2D (Lai 2008). This case might be used to test the capabilities of other two-dimensional models or to address the performances of existing one-dimensional software.

G.2 MATERIALS AND METHODS

Two-dimensional software: SRH-2D

SRH-2D solves the shallow water equations and the convection-diffusion equation over a mobile bed and Boussinesq equations are used to compute the turbulence stresses (Lai 2008, 2010; Greimmann et al., 2008).

SRH-2D proposes two turbulence models: k-epsilon and depth-averaged parabolic models. SRH-2D uses a wetting–drying front limit of 0.001 m. An implicit scheme is used to solve the finite-volume numerical method.

Pre-processing and post-processing of the model is executed in SMS (AQUAVEO, 2018), a modelling software presented as a graphical user interface and analysis tool that holds all of the SRH-2D functionalities.

Application case: Petite Manouane and Manouane rivers' confluence

The study case is the confluence of the two rivers, Petite Manouane and Manouane, just upstream the Duhamen lake, located in the Quebec province of Canada (FigureG.1). The discharges are 250 m³/s and 40 m³/s for the Petite Manouane and Manouane rivers respectively. The lake level is constant at the elevation 252.45 m. The bed material is composed mainly of uniform sand, having a median size of 3 mm. The Manouane river carries no sediment while the Petite Manouane river

sediment discharge's is not limited. The sediment transport equation of Engelund and Hansen is used for this study. The domain is 4,800 m long from the Petite Manouane River upstream boundary, and 3,800 m long from the Manouane river upstream boundary.

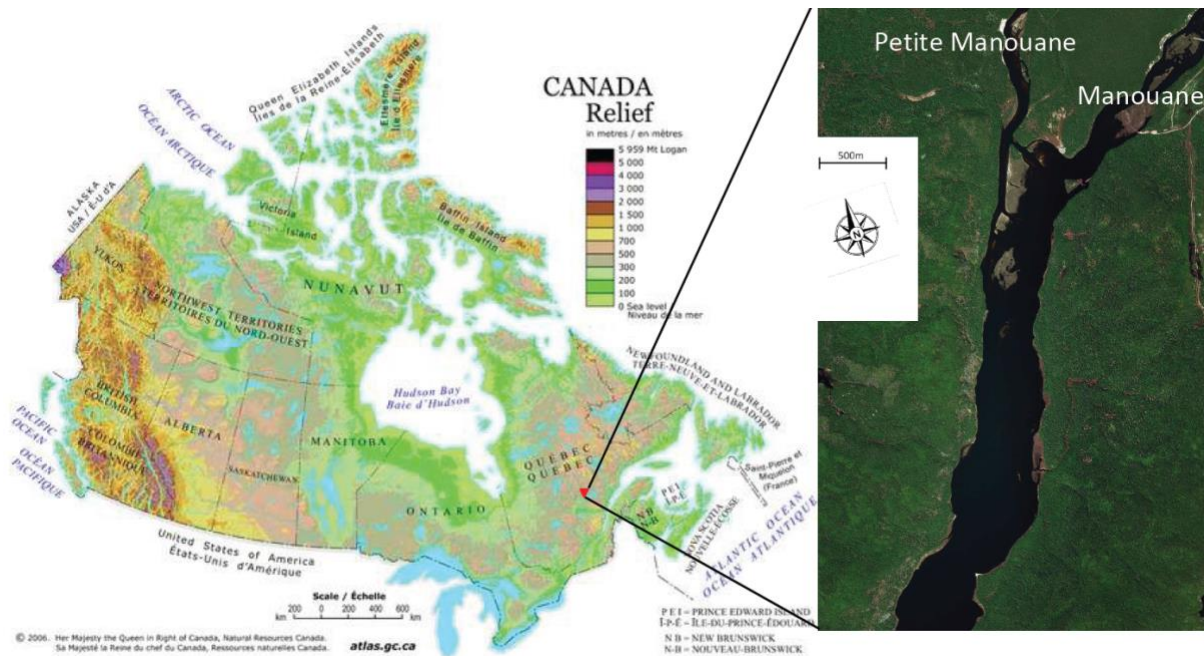


Figure G.1 Location of the rivers' confluence

For the two-dimensional simulation the numerical domain is discretized using 42,932 triangular elements. A constant value of the Manning's coefficient of 0.018, for the entire domain, is adopted according to Strickler formula (Yang, 2003), and to guarantee numerical stability, the time step is reduced to 2 s. For the downstream boundary condition, a constant water level is maintained at 252.45 m.

G.3 RESULTS AND DISCUSSION

Figure G.1 shows the sand deposition/erosion evolution for the first 16 days. No qualitative data are available for validation but, deposition has been observed to increase during the last years, when the Manouane river discharge started to decrease, at the same places as simulated.

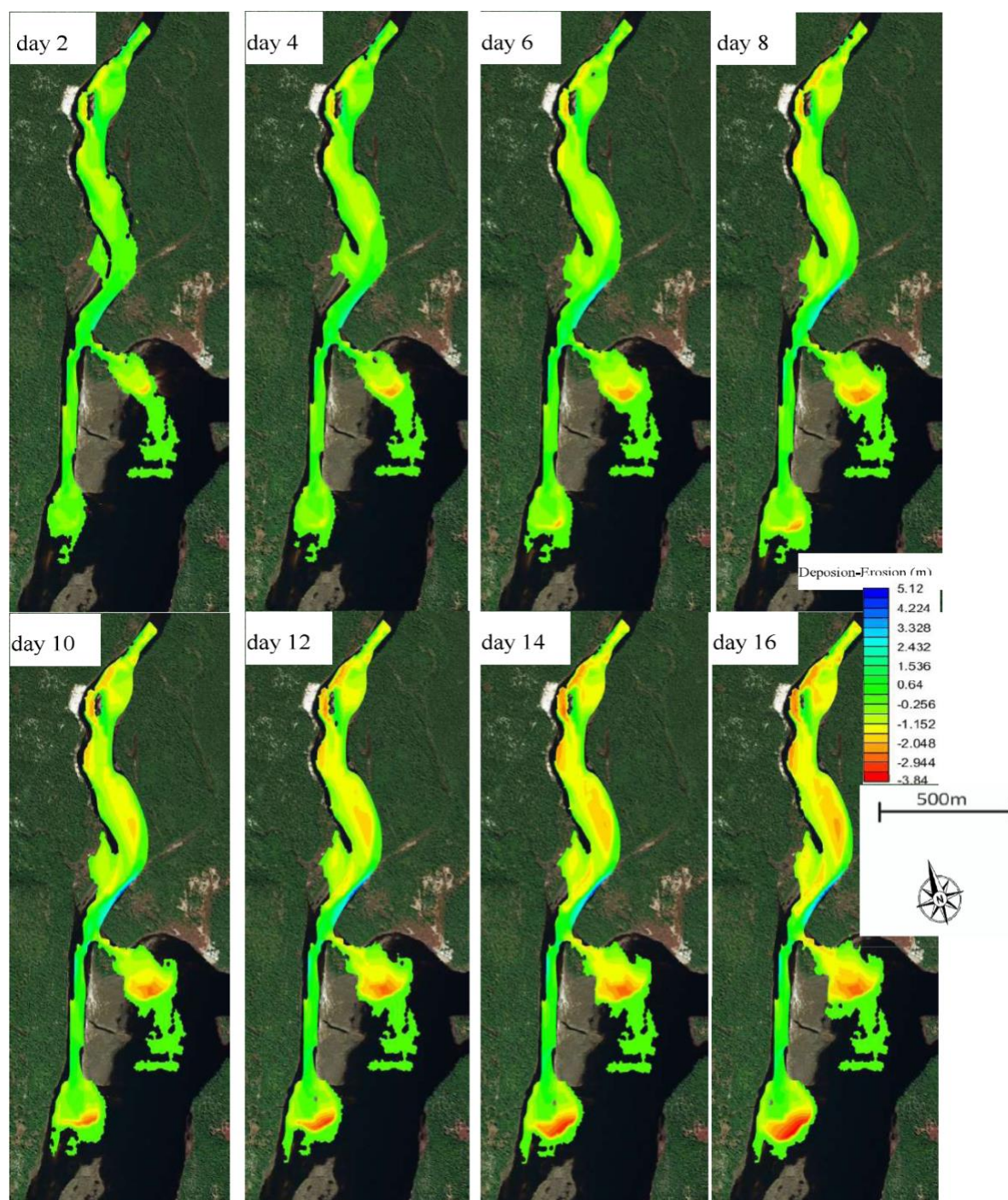


Figure G.2 Erosion/deposition evolution during the first 16 days (deposition values are negative)

The deposition and erosion zones correspond to the zones of low and high velocities respectively. At the junction of the the Petite Manouane and the Manouane rivers, the velocities' values are low, allowing sediments' deposition, as shown in Figure G.3. Note that erosion occurs

in the Petite Manouane river, where the velocities are higher (small cross-section and higher discharge).

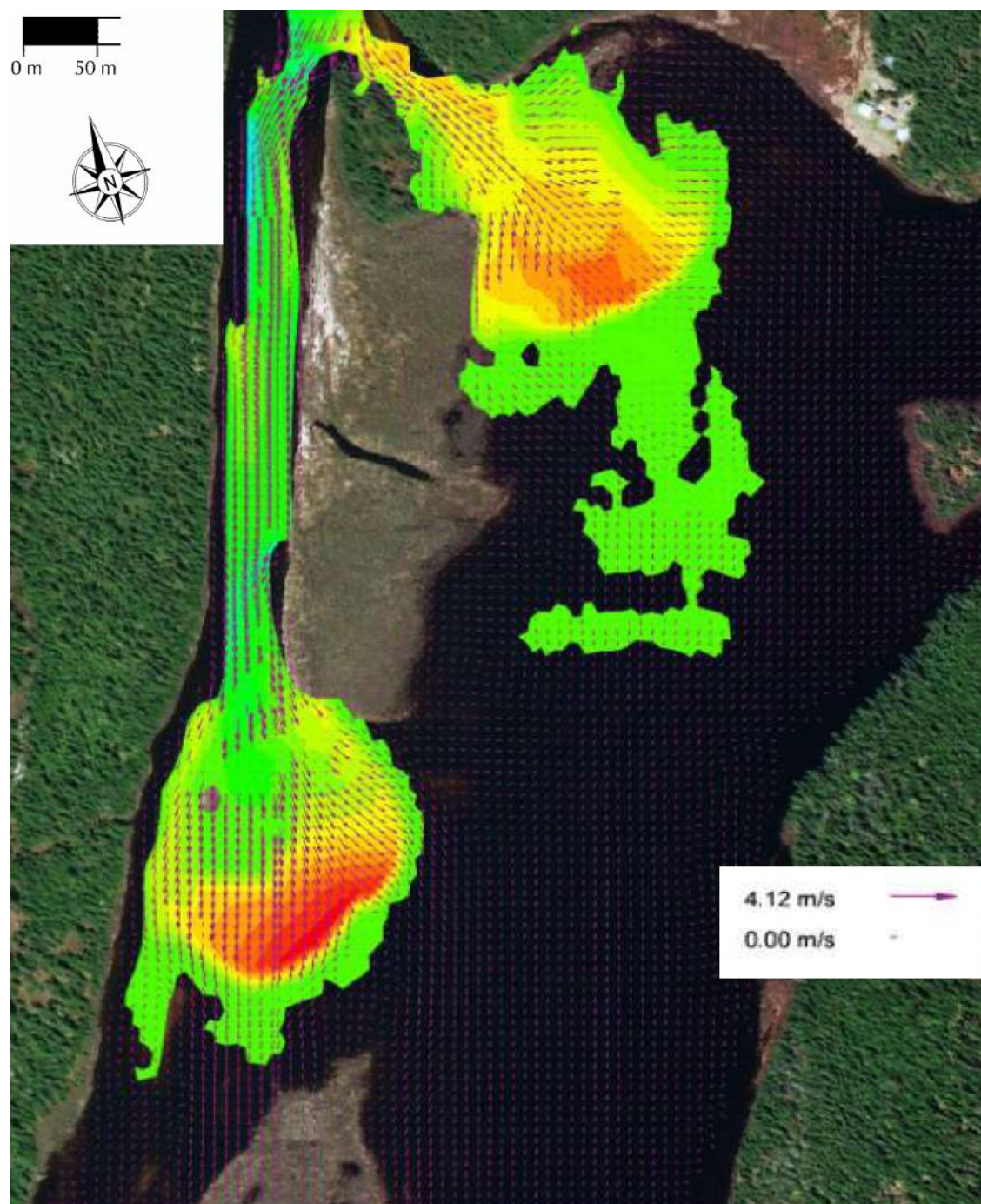


Figure G.3 Velocity field at the confluence. Deposition occurs in zones of low flow velocities

Figure G.4 shows that its last bend, before the confluence, experiments huge erosion on the left side (looking downstream), after the deposition increases in the right side. The maximum values of deposition and erosion are about 4 m to 5 m respectively

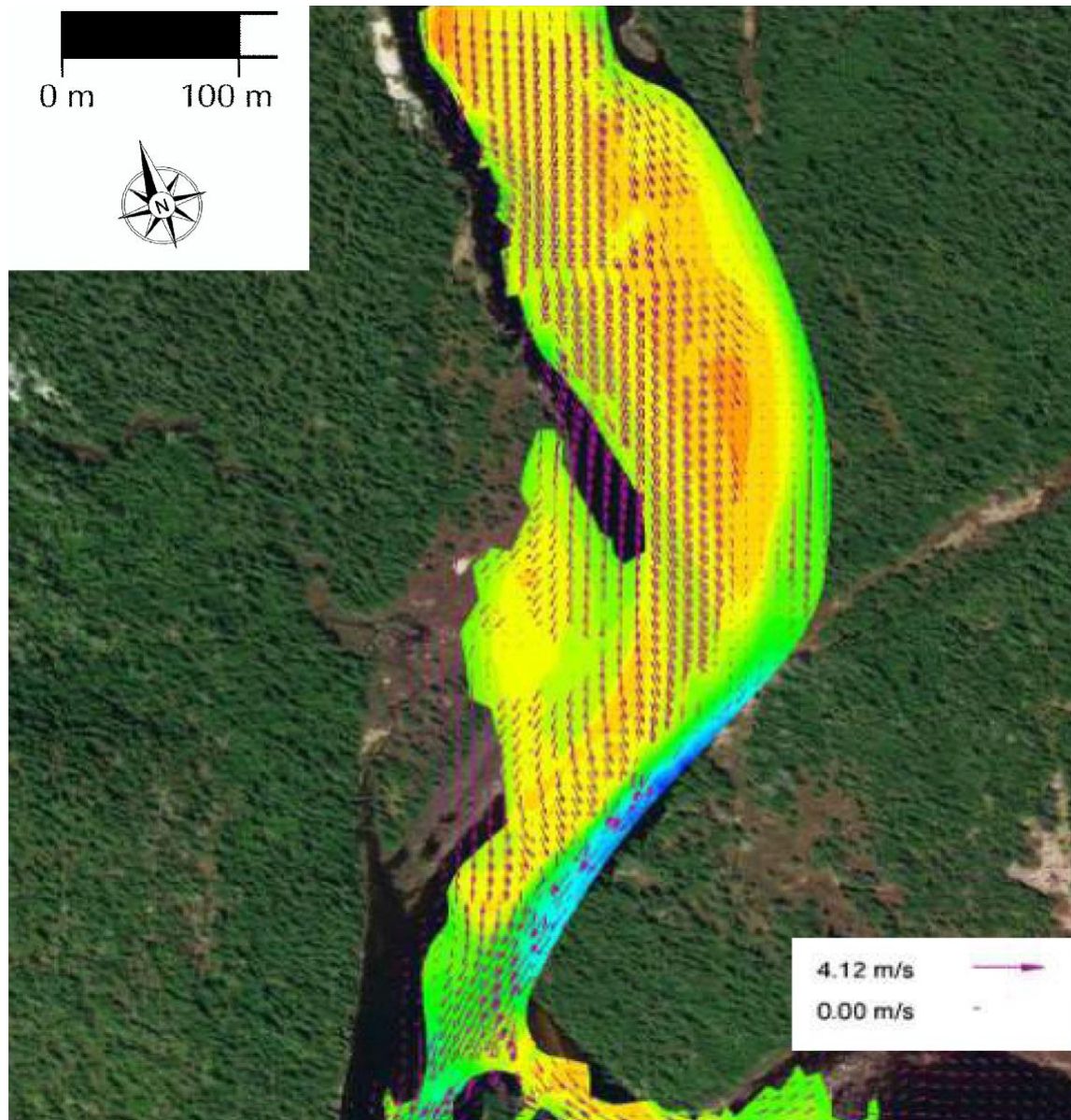


Figure G.4 High erosion occurring in zones of high flow velocities on the Petite Manouane river

G.4 CONCLUDING REMARKS

In this work the numerical software SRH-2D software was used to simulate morphological evolution of the confluence of the Petite Manouane and Manouane rivers, located in the Quebec

province, Canada. The results capture the trend of deposition/erosion observed in the field. The deposition and erosion zones correspond to the zones of low and high flow velocities. Future work will use these results to test the capabilities and limitations of the software UMHYSER-1D.

Acknowledgements

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