Diss. ETH No. 17534

Numerical Modelling of Distributed River – Aquifer Coupling in an Alpine Floodplain

A dissertation submitted to ETH Zürich
for the degree of Doctor of Science
presented by

Wolfgang Ruf

Dipl.-Hyd., Universität Freiburg i. Br.
July, 11th, 1969, German

accepted on the recommendation of
Prof. Paolo Burlando, examiner
Prof. Andreas Dittrich, co-examiner
Dr. Roland Fäh, co-examiner

2007

Diss. ETH No. 17534

Numerical Modelling of Distributed River – Aquifer Coupling in an Alpine Floodplain

A dissertation submitted to
ETH Zurich
for the degree of
Doctor of Sciences
presented by

Wolfgang Ruf

Dipl.-Hyd., Albert-Ludwigs-Universität Freiburg im Breisgau
Date of birth: July 11th, 1969
citizen of Germany

accepted on the recommendation of

Prof. Paolo Burlando, examiner
Prof. Andreas Dittrich, co-examiner
Dr. Roland Fäh, co-examiner

2007

Acknowledgement

The author wishes to thank Prof. Paolo Burlando for giving me the opportunity to work on this very interesting topic and to write this dissertation as well as for his manifold support during the last years. Special thanks goes also to Peter Molnar and Paolo Perona as supervisors of my work, but also as good collaborators in the MaVal project, who helped and supported me in so many aspects: scientific advice, fruitful discussions, personal conversations, proofing the manuscript, and many more. Thank you also to the co-examiners for spending their time and their helpful comments.

I am very grateful for all assistance, discussions and support also from the other members of the MaVal group: Laura Foglia, who developed the groundwater model and assisted in many field works, but also Matthias Sturzenegger, Marius-Viktor Bîrsan and Maurizio Savina, who worked more on the vegetation part of the project, and Andrea Salvetti, who worked at SUPSI-IST. Thank you also to Prof. Hansruedi Pfeifer, who accompanied this project.

Many thanks go to all diploma students of the MaVal project, namely Stephan Pfammatter, Pascal Zanetta, Uria Cerini, Samirah Namijeh, Virginie Favre, Lorenzo Alfieri and M. Ranieri, who all did their diploma work within the MaVal project. Further thanks goes to many students and coworkers of the institute and of SUSPI-IST for the help in the field or in the data preparation: Klemens Rosin, Jörg Kirsch, Esther Müller, Bernhard Richli, Dirk Schroer, Massimiliano Cannata, and many more...

The work was not possible without the technical support from Thomy Keller from ETH Zürich and Sergio DeBernardi from SUPSI-IST, who installed many instruments and regularly downloaded the data. Thank you also to OFIMA for the allowance of installing the meteo-station on their property and for providing all required data. Additional data were provided by SES, MeteoSwiss, Federal Office for Environment and Swisstopo.

Daniel Braun and especially Fabienne Steiner invested a lot of time and help related to the analysis of water samples in the laboratory. Analyzes for oxygen isotopes were carried out in the laboratory of Stefano Bernasconi, and silica was analyzed in the chemistry laboratory from Prof. Detlef Günther. Prof. Ingensand and Adrian Ryf provided geodetic measurement instruments and assistance in their usage. Great help in all computer related problems was granted to me by René Weber, Xaver Studerus and Roland Alber.

Special thanks goes to Roland Fäh for providing the open-channel flow model 2dMb and its support in its application. Steffen Mehl helped me in the case of questions related to MODFLOW.

There is still a large number of persons, who could be mentioned here, because such an extended project could not be carried out without the help of many helpful people everywhere and all the time. I want to say "thank you very much" to all of them. Furthermore, many good friendships were developed during the time of this project, and I hope that many of them will last also after the end of this work.

Last, but not least, I thank my parents for making my studies and this PhD work possible, but also for the personal assistance and many good words, which is also the case for my sister and many of my good friends.

This project was funded by the Canton Ticino (*Sezione Protezione Aria Acqua e Suolo*), Swiss National Fund (grant No. 21-66885.01) and BUWAL, Bern (Project-Nr. 810.4600.101).

TABLE OF CONTENTS

LIST OF FIGURES IN THE TEXT

List of Tables in the Text

List of Figures in the Appendix

List of Tables in the Appendix

List of Symbols and Abbreviations

Symbol	Dimension	Description
A	$\mathbf{M^2}$	cross-section of the flow
A_{riv}	$\mathbf{L^2}$	wet area within one MODFLOW river cell
c	$\mathbf{L\,T^{-1}}$	kinematic wave velocity; celerity
C	-	Courant number
C_{riv}	$\mathbf{L^2\,T^{-1}}$	hydraulic conductance of stream – aquifer interconnections
cfl	-	numerical variable which corresponds to the Courant number C
D	$\mathbf{L^2\,T^{-1}}$	diffusion coefficient
D/Dt	$\mathbf{T^{-1}}$	substantive derivative
$\boldsymbol{f}$	$\mathbf{L\,T^{-2}}$	body force vector
g	$\mathbf{L\,T^{-2}}$	gravity constant / gravitational acceleration
h	$\mathbf{L}$	pressure head / piezometric head
h	$\mathbf{L}$	flow depth
h_{dry}	$\mathbf{L}$	numerical threshold variable considering a cell as being dry
$h_{exchange}$	$\mathbf{L}$	numerical variable considering a cell as taking actively part in the river – aquifer interaction process
h_{gw}	$\mathbf{L}$	groundwater head
$h_{i,j,k}$	$\mathbf{L}$	piezometric groundwater head at a cell node
h_{limit}	$\mathbf{L}$	numerical threshold variable considering a cell as being suitable for infiltration
$h_{obs,i}$	$\mathbf{L}$	simulated height
h_{riv}	$\mathbf{L}$	head in the stream / water level in the river
h_{wet}	$\mathbf{L}$	numerical threshold variable considering a cell as being wet
$h_{sim,i}$	$\mathbf{L}$	simulated height
$\boldsymbol{k}$	$\mathbf{L\,T^{-1}}$	Tensor of hydraulic conductivity of the aquifer
K_{riv}	$\mathbf{L\,T^{-1}}$	hydraulic conductivity of riverbed material
k_s	$\mathbf{L}$	equivalent sand roughness / Nikuradse roughness
K_{Sat}	$\mathbf{L\,T^{-1}}$	saturated hydraulic conductivity
k_{St}	$\mathbf{L^{1/3}\,T^{-1}}$	Strickler coefficient (=1/Manning number)
k_{tot}	$\mathbf{L}$	total roughness
K_{xx}	$\mathbf{L\,T^{-1}}$	saturated hydraulic conductivity along x-direction

Symbol	Dimension	Description
K_{yy}	**$L\,T^{-1}$**	saturated hydraulic conductivity along y-direction
K_{zz}	**$L\,T^{-1}$**	saturated hydraulic conductivity along z-direction
L	**L**	length of the reach (river cell in MODFLOW)
M	**L**	thickness of riverbed
m	**-**	maximum possible number, whose cell centres lie within the area of a corresponding MODFLOW cell
$mult$	**-**	whole-numbered multiple in the context of grid transformation
n	**-**	number of active 2dMb cells
$\boldsymbol{P}$	**$M\,L^{-2}\,T^{-1}$**	tensor representing the surface forces applied on a fluid particle
q	**$L^2\,T^{-1}$**	sink or source term
q	**$L\,T^{-1}$**	groundwater flow
q_{ex}	**$L^3\,T^{-1}$**	exchange rate of a single 2dMb cell
Q	**$L^3\,T^{-1}$**	discharge
Q_{exch}	**$L^3\,T^{-1}$**	flow between stream and aquifer (positive, if directed to the aquifer) / for a given MODFLOW cell
Q_{inflow}	**$L^3\,T^{-1}$**	discharge as inflow in Bignasco
r_{bot}	**L**	bottom of streambed
r_{hy}	**L**	hydraulic radius
$RMSE$		root mean square error
S_0	**-**	bed slope
S_f	**-**	friction slope
sf	**L**	weir crest height
Ss	**L^{-1}**	specific storage of the porous material
t	**T**	time
T	**T**	Time domain
T_{exch}	**T**	time interval for calling MODFLOW and for exchange of information between MODFLOW and 2dMb
$\boldsymbol{u}$	**$L\,T^{-1}$**	flow velocity
$\bar{u}$	**$L\,T^{-1}$**	depth-averaged flow velocity
u_*	**$L\,T^{-1}$**	shear velocity
$\boldsymbol{v}$	**$L\,T^{-1}$**	velocity vector
W	**T^{-1}**	volumetric flux per unit volume representing sources or sinks
W	**L**	width of the river (in river cell in MODFLOW)
x	**L**	coordinate in space

Symbol	Dimension	Description
x	**L**	longitudinal direction
y	**L**	coordinate in space
y	**L**	flow depth
Y_u	**L**	uniform flow depth
z	**L**	coordinate in space
z	**L**	vertical datum
z_b	**L**	geodetic height of the river bed
β	-	dispersion-correction factor
β	-	form factor for the consideration of the roughness
ρ	$\mathbf{M\,L^{-3}}$	fluid density / specific weight of the fluid
τ_{bx}	$\mathbf{M\,L^{-1}\,T^{-2}}$	bed shear stress
τ_{by}	$\mathbf{M\,L^{-1}\,T^{-2}}$	bed shear stress
τ_{xx}	$\mathbf{M\,L^{-1}\,T^{-2}}$	turbulent normal stress
τ_{yy}	$\mathbf{M\,L^{-1}\,T^{-2}}$	turbulent normal stress
τ_{xy}	$\mathbf{M\,L^{-1}\,T^{-2}}$	turbulent shear stress
τ_{yx}	$\mathbf{M\,L^{-1}\,T^{-2}}$	turbulent shear stress
Ω	$\mathbf{L^2}$	space domain

Glossary and Definitions

In this glossary, definitions are given, how specific terms are used in the context of this work.

2dMb

2-Dimensional Mobile Bed Model. Horizontal 2D surface water model used in this study, developed at ETH Zürich, Switzerland.

Active Cells

Those grid cells of a model (groundwater or surface water model), which are part of the rectangular grid, having therefore also a grid numbering, but are excluded from solving the equations for the modelling, because they cannot be wetted: this is because they are either part of the hillslope area (not part of the aquifer resp. floodplain), or they are temporarily not wet (in case of no inundation in the surface water model).

Braided Area

Area of special interest. It is located within the "main valley" (see there), showing a braided river system. In this study it refers to the area between Riveo and Coglio.

Connected Aquifer

Aquifer with groundwater connected with the river by an unsaturated area. Both infiltration and exfiltration can occur, depending on the head difference between the river and aquifer.

Digital Terrain Model (DTM)

Set of coordinates in space containing also z-values (vertical geodetic information as a distance to the reference geoid). Coordinates can be regularly spaced or scattered irregularly; z-values are related to the surface of the terrain. This means that buildings and vegetation have been filtered out. Below water surfaces, the z-values refer to the bottom of the water body (river bed, lake bottom). In contrast, digital surface models (DSM) include also buildings and vegetation, the z-values refer therefore to the top of the canopy. The term "digital elevation model" (DEM), which is widely used by geographers and engineers is avoided in this text in order to avoid confusion between DTM and DSM.

Environmental Flow (Requirement) (EFR)

Environmental Flow release is the water to be released or maintained within the river below an intake station. The environmental flow requirement (EFR) is the discharge needed in order to sustain the ecological integrity of the downstream riparian ecosystems. Often, this EFR is fixed by legislative regulation.

Exfiltration

Water flux from the aquifer into the river resp. surface water.

External and Iterative Coupling

External coupling of quasi-steady-state surface water and steady-state groundwater model. The final solution is reached by iteration.

Full Coupling

Coupling scheme used for the connection between the surface water and groundwater model. The exchange of information after each time step is handled with external ASCII-files.

Groundwater

Connected subsurface water within the aquifer; here in the alluvial fill of the "main valley" (see there).

Groundwater Model

Here: quasi-3D Finite-difference model MODFLOW-2000 with a structured grid, here used in 2D.

Hillslope Flux

Diffuse lateral fluxes from the adjacent hillslopes directly into the aquifer. This could principally be overland and subsurface flow. In this study, only subsurface flow is considered, since overland flow is assumed to be negligible. In the special situation of this study, also water from tributaries, which infiltrates directly into the aquifer when reaching the alluvial deposits of the main valley is included in the hillslope fluxes.

Hydrological Model

Here: synonym for watershed model.

Infiltration

Water flux from the river resp. surface water into the aquifer.

Main River

Here, the term "main river" stands for the river course of the river Maggia within the "main valley" (see there).

Main Valley

Area of interest in this study. It is the lower part of the studied catchment starting in Bignasco at the upper part and ending in Avegno resp. Ponte Brolla at the lower part. It is the area where both the surface water and groundwater model are applied.

MaVal

Acronym for the project funded by SNF (Swiss National Science Foundation), in which this PhD research was conducted: Interactions between surface water and groundwater in an alpine environment – assessment, modelling, ecosystem response, impact analysis (Maggia Valley project; http://www.maggia.ethz.ch).

Modelling Domain

Area containing all cells of a model regardless whether they are active or inactive. The active modelling domain contains only the active model cells.

MODFLOW-2000

The U.S. Geological Survey Modular Ground-Water Model. A quasi-3D finite-difference groundwater model used in this study and developed at the U.S. Geological Survey, United States of America.

Quasi Steady-State

State in a transient model as a result of a simulation with boundary conditions being constant over time, until the solution is stable and does no longer change in time.

Recharge

Term from the perspective of the groundwater model. It is the spatially distributed input into the aquifer from precipitation and hillslope fluxes. In this study, hillslope fluxes account for almost the entire recharge and occur at the border of the aquifer.

River-Aquifer Interaction

Process of exchange of water between surface water and groundwater (in both directions).

Side Valleys

Side valleys to the main valley, containing tributaries, which are partly diverted for the purpose of hydropower production. These tributaries contribute only in case of flooding conditions to the "main river" (see there).

Surface Water

Water in the river or water inundating the floodplain.

Surface Water Model

This term is used here in contrast to the term “groundwater model”. It is a synonym for a hydrodynamic model, or more precisely: open-channel flow model. This model is used to simulate the flow in the river and the inundation in the floodplain. In this study it refers to the model 2dMb.

TOPKAPI

TOPographic Kinematic Approximation and Integration: Distributed hydrological watershed model used in this study, developed at University of Bologna, Italy.

Unconnected Aquifer

Aquifer whose groundwater is separated from the river by an unsaturated zone. Only infiltration, no exfiltration can occur.

Watershed Model

Hydrological model applied within the entire catchment. In this study, the distributed model TOPKAPI accounting for all important hydrological processes except groundwater flow is used.

Important Geographic Names

Main River: *Maggia*

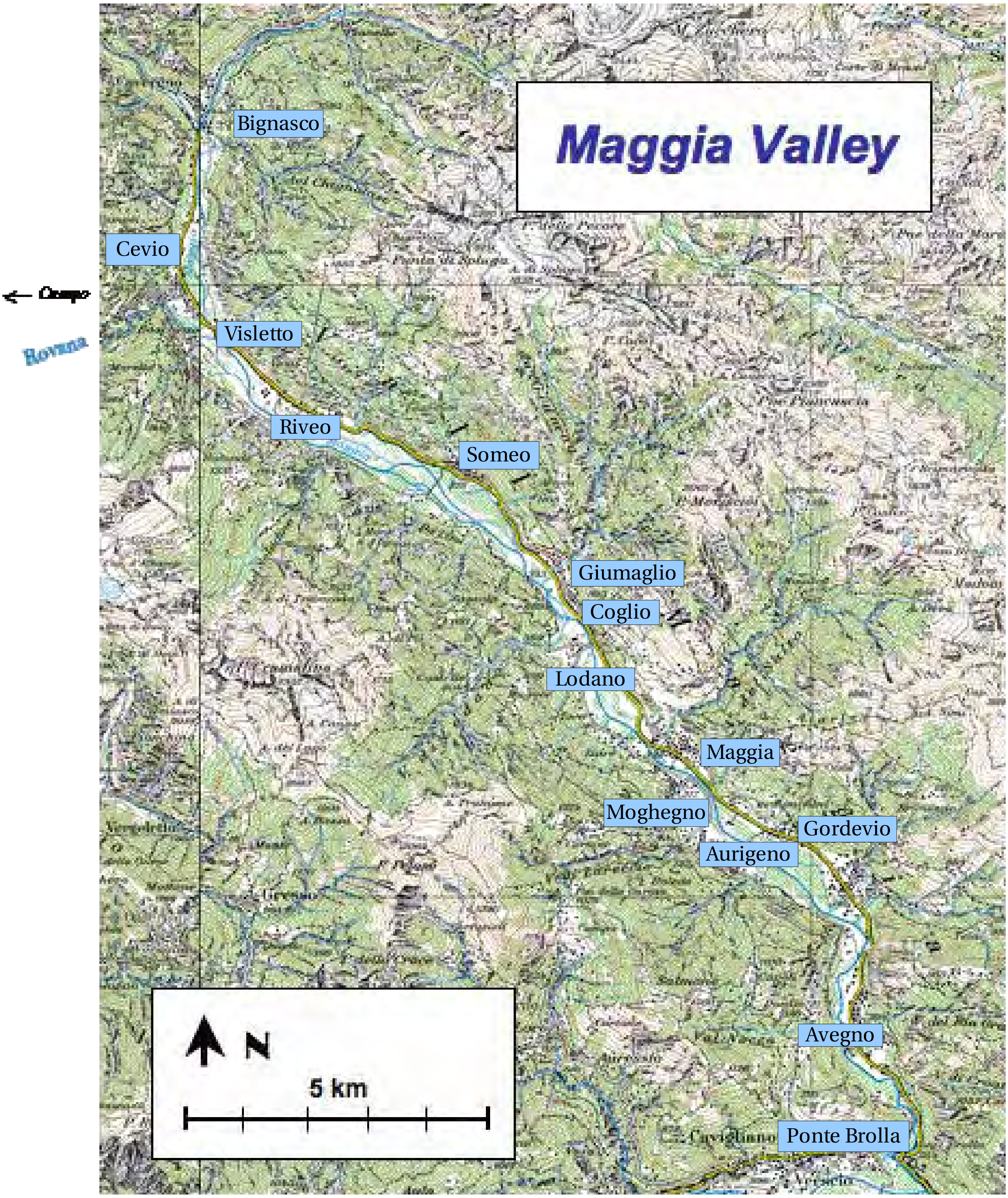
Main Tributary: *Rovana (confluence with Maggia in Visletto)*

Villages in the main valley in the order from upstream to downstream:

Cavergno, Bignasco, Cevio, Visletto, Riveo, Someo, Giumaglio, Coglio, Lodano, Maggia, Moghegno, Aurigeno, Ronchini, Gordevio, Avegno, Ponte Brolla.

Topographic Map of the Maggia Valley

(Source: swisstopo, Wabern, Switzerland)



Abstract

Many practical and research applications, both in environmental science and water management, require an integral view of the problem. For this purpose, integrated modelling becomes more important, while increasing computer power also allows for more complex modelling systems. In this context, this work provides an important step ahead in coupling a surface water model for open-channel flow with a groundwater model in order to account for the river-aquifer exchange processes.

Surface water – groundwater interaction is of great importance in many fields of ecohydrological science, water quantity and quality management problems as well as for ecologically oriented protection, mitigation and restoration projects. It is the link between river and groundwater systems. Ecohydrological research is gaining importance, but is so far mostly restricted to field observations or to modelling at the reach scale. Hydrological models on the other side are generally applied on the entire watershed scale, but neglect the ecological aspects.

This work is embedded in a project trying to bridge the gap between hydrology and ecology on the one hand, and between the different scales on the other. In an environmentally active floodplain (in the Maggia valley in Southern Switzerland), a modelling framework was developed to investigate the influence of an heavily altered flow regime as a consequence of hydropower operation on the riverine vegetation development in the floodplain.

For this purpose, the hydrological conditions had to be modelled in the entire riverine corridor, accounting both for surface water (i.e. river flow and inundation in the floodplain) and groundwater conditions and processes. The strong and fast response of the system to changing water levels in the river and piezometric heads in the aquifer leads to the target of this work, namely to the development of a coupled model accounting also for the feedback mechanisms between these two systems. Therefore, the coupling between a detailed transient two-dimensional hydraulic open-channel flow model with a transient two-dimensional or three-dimensional groundwater model was needed. None of the existing modelling approaches was able to fulfil all requirements. Therefore, the "Modular three-dimensional groundwater flow model" (MODFLOW-2000) was coupled with the "Two-dimensional mobile bed model" (2dMb). The topographic and flow conditions led to enhanced requirements in terms of numerical stability due to the presence of numerous hydraulic jumps and extensive drying and wetting of surface water cells.

Special attention in coupling the surface water model and the groundwater model had to be paid to the following issues:

- Choice of the mathematical formulation of the river-aquifer exchange process
- Grid-to-grid transformation of exchange rates and hydraulic heads considering different grid sizes and orientation, accounting also for the time and space variability of wet cells of the 2dMb grid
- Time scheme of communication between the two models and exchange of information between them, accounting also for different time-scheme requirements for the single models
- Numerical stability of the coupling scheme
- Reproducing realistic results of the river-aquifer interaction

The coupled model is embedded in the holistic modelling framework accounting also for the hydrological processes in the watershed. It provides space and time variable information on hydrological and hydraulic variables of ecological importance, such as inundated area and duration of inundation, flow depth, flow velocity, shear stress and depth-to-groundwater. The time series or statistics of these variables provide essential information for any riparian vegetation growth model to be applied in the riverine corridor.

The analysis of the simulation results shows that the coupled model is numerically robust and provides the desired information for the investigation of water-related processes and conditions which are important for the evolution of the floodplain vegetation. Furthermore, the developed modelling tool is universally applicable in other areas of investigation under similar environmental conditions and is believed to be a valuable tool for many further ecohydrological and river restoration projects.

Keywords: river-aquifer interaction, model coupling, numerical modelling, ecohydrology

Zusammenfassung

Viele praktische und wissenschaftliche Anwendungen – sowohl im Umweltbereich als auch in der Wasserwirtschaft – verlangen eine integrale Sichtweise der Probleme. Daher gewinnt die integrierte Modellierung immer mehr an Bedeutung, aber auch deswegen, weil durch die zunehmende Leistung der Computer immer komplexere Modellsysteme ermöglicht werden. In diesem Kontext bedeutet diese Arbeit der Kopplung eines Strömungsmodells für die Simulation des Wellenablaufes im Fluss bzw. von Überschwemmungen mit einem Grundwassermodell einen wichtigen Fortschritt, um die Austauschprozesse zwischen Fluss und Grundwasser erfassen zu können.

Die Interaktion Fluss – Grundwasser ist in vielen Bereichen der ökohydrologischen Forschung, bei Wasserquantitäts- und qualitätsproblemen in der Wasserbewirtschaftung wie auch bei ökologisch orientierten Schutz- und Sanierungs- und Revitalisierungsmassnahmen von grosser Bedeutung. Sie ist das Bindeglied zwischen Fluss- und Grundwassersystemen. Die ökohydrologische Forschung gewinnt immer mehr an Bedeutung, ist aber bis jetzt meist eingeschränkt auf Feldforschung oder auf die Modellierung in der Flussabschnittsskale. Hydrologische Modelle andererseits werden im gesamten hydrologischen Einzugsgebiet angewendet, allerdings unter der Vernachlässigung ökologischer Aspekte.

Die vorliegende Arbeit ist in ein Projekt eingebettet, welches einerseits eine Brücke zwischen Hydrologie und Ökologie und andererseits zwischen den verschiedenen Skalen schlagen will. Daher sollte ein Modellsystem geschaffen werden, welches es erlaubt, in einer ökologisch aktiven Flusslandschaft (dem Maggia-Tal im Kanton Tessin, Schweiz), den Einfluss eines aufgrund der Wasserkraftnutzung entscheidend veränderten Abflussregimes auf die Vegetationsentwicklung in der Aue zu untersuchen.

Aus diesem Grund mussten die hydrologischen Verhältnisse im gesamten Flusstal modelliert und dabei die hydrologischen Prozesse sowohl an der Oberfläche (Fluss und Überschwemmungen) als auch im Grundwasser berücksichtigt werden. Die starke und schnelle Systemantwort auf sich ändernde Wasserstände im Fluss und Grundwasserhöhen bedingen das Hauptziel dieser Arbeit, nämlich die Entwicklung eines gekoppelten Modells, welches auch Rückkopplungsmechanismen zwischen diesen beiden Systemen mitberücksichtigt. Aus diesem Grund war eine Kopplung zwischen einem detaillierten instationären zweidimensionalen hydraulischen Strömungsmodell und einem instationären zwei- oder dreidimensionalen Grundwassermodell erforderlich. Keine der existierenden Modellansätze konnte alle nötigen Voraussetzungen erfüllen. Daher wurde das "Modular three-dimensional groundwater flow model" (MODFLOW-2000) und das "Two dimensional mobile bed model" (2dMb) miteinander gekoppelt. Dies topographischen Gegebenheiten wie auch die Abflussverhältnisse im Fluss stellen besondere Anforderungen an die numerische Robustheit, insbesondere durch

zahlreiche Fliesswechsel und häufigen Wechsel zwischen trockenen und überfluteten Bereichen.

Besondere Beachtung musste bei der Kopplung des Strömungs- mit dem Grundwassermodell folgenden Punkten geschenkt werden:

- Wahl der mathematischen Formulierung der Austauschprozesse zwischen Fluss und Grundwasser
- Übertragung der Austauschinformationen und der hydraulischen Druckhöhen zwischen den verschiedenen Gittern der beiden Modelle, wobei unter anderem die zeitliche Veränderlichkeit der benetzten Zellen im Strömungsmodell berücksichtigt werden musste
- Zeitschema für die Kommunikation zwischen den beiden Modellen und der Informationsaustausch zwischen diesen beiden, wobei die unterschiedlichen Zeitschemata der beiden Modelle berücksichtigt werden mussten
- Numerische Stabilität des Kopplungsschemas
- Plausibilität der Simulationsergebnisse

Das gekoppelte Modell ist in ein umfassendes Modellsystem eingebettet, welches auch die hydrologischen Prozesse im gesamten Einzugsgebiet berücksichtigt. Es liefert räumliche und zeitliche Informationen über ökologisch relevante hydrologische und hydraulische Variablen wie z. B. Überflutungsfläche und -dauer, Abflusstiefe, Fliessgeschwindigkeiten, Sohlenschubspannungen und Grundwasserflurabstände. Die Zeitreihen oder statistischen Kennwerte dieser Variablen liefern die notwendigen Informationen für jedes Vegetationsentwicklungsmodell, welches in der Flussaue zur Anwendung kommen wird.

Die Analyse der Simulationsergebnisse zeigt, dass das gekoppelte Modell numerisch stabil ist und die gewünschten Informationen für die Untersuchung der wasserbezogenen Prozesse und Verhältnisse liefert, welche für die Vegetationsentwicklung in der Flussaue von Bedeutung sind. Ausserdem ist das hier entwickelte Modellwerkzeug universell, d. h. auf andere Gebiete mit vergleichbaren ökologischen Verhältnissen übertragbar, und ein wertvolles Hilfsmittel für viele zukünftige ökohydrologische und Flussrevitalisierungsprojekte.

Sommario

Molte applicazioni scientifiche, sia in campo delle scienze ambientali che in quello rivolto alla gestione delle acque, richiedono una vista integrata dei problemi. Per questo motivo, i modelli integrati divengono sempre più importanti; questa crescita è altresì favorita dalla continua evoluzione dei nuovi processori che permettono computazioni sempre più complesse. Questo lavoro è dedito all'accopiamento di un modello di deflusso superficiale con uno di falda. L'obbiettivo è quello di descrivere i processi di scambio tra fiume e acquifero.

L'interazione tra acque superficiali e sotterranee ha un importanza sempre più alta ed è materia di studio per tutte quelle scienze eco-idrologiche dedite allo studio della qualità, quantità, gestione e protezione delle acque e degli ambienti rivieraschi. Ancora oggi la ricerca eco-idrologica è molto legata alle misurazioni in campo o alla realizzazione di modelli a scala di fiume; i modelli idrologici, che usualmente sono applicati agli interi bacini idrologici, non descrivono gli aspetti ecologici.

Questo lavoro fa parte di un progetto intento a colmare le lacune tra gli aspetti ecologici e idrologici nonchè quelli tra le diverse scale di indagine. L'ambiente di studio è la Valle Maggia (nella Svizzera meridionale), un'attiva valle rivierasca colpita da un forte alterazione dei regimi idrologici a causa dell'installazione di diversi sbarramenti per la produzione idroelettrica. Allo scopo di fornire supporto ad un modello per lo sviluppo della vegetazione rivierasca, si è cercato di modellare le condizioni idrologiche per l'intera valle. Accoppiando un modello per le acque superficiali (i.e. portate superficiali e zone inondate) con un modello di falda. Cercando di sviluppare questo modello integrato si è dovuta anche considerare la differenza di risposta dei due sistemi (superficiale e sotterraneo) la quale ha permesso altresì l'approfondimento della conoscenza dell'interazione fiume-falda. Ci si è resi conto che i modelli da accoppiare dovevano essere di tipo transitorio bi-dimensionale per le acque superficiali, transitorio bi(tri)-dimensionale per quello di falda. Sik è quindi deciso di accoppiare "2Modular three-dimensional groundwater flow model" (MODFLOW-2000) con "Two-dimensional mobile bed model" (2dMb). Le particolari condizioni topografiche è idrologiche hanno richiesto un miglioramento in termini di stabilità numerica; tutto ciò è dovuto alla frequente presenza di salti idraulici e all'estensione della superficie inondabile.

In particolare, i punti critici che si incontrano quando si vuole creare un modello integrato per le acque superficiali e di falda, sono:

- scelta della formulazione matematica per i processi di scambio tra fiume e falda;
- passaggio da un griglia di calcolo all'altra, solitamente i due modelli si basano su griglie con diverse dimensioni e orientamento.
- scelta di uno schema temporale per il dialogo tra i due modelli, in considerazione anche delle diverse velocità di calcolo.
- Stabilità numerica per il modello accoppiato;
- riproduzione di risultati realistici in termini di interazione fiume-falda.

Il modello messo a punto considera ovviamente anche le caratteristiche del bacino idrologico e fornisce informazioni puntuali, in termini spaziali e temporali (sul dominio e per il tempo di indagine), di diverse variabili idrauliche e idrologiche. Le grandezze di output, che possono ad esempio essere utilizzate come input in altri modelli, sono l'area inondata, la durata dell'inondazione, l'altezza d'acqua, la velocità della corrente, le sollecitazioni taglianti della corrente e la profondità della falda. La serie storica, cosi' come le statistiche su queste variabili sono fondamentali per qualsiasi tipo di modello di vegetazione rivierasca.

I risultati delle analisi compiute mostrano che il modello accoppiato è numericamente robusto e risponde a quanto precedentemente descritto. Inoltre, il modello qui sviluppato è applicabile ad altre aree con simile caratteristiche ambientali e potrebbe essere uno strumento molto utile per studi di eco-idrologici cosi' come per studi di ripristino ambientale.

1. INTRODUCTION

1.1. Background

River-Groundwater interaction is an important process in the hydrological cycle. On the one hand it is relevant for the development and characteristics of riverine habitats (e.g. special review: Brunke and Gonser, 1997). On the other hand, knowledge of the water fluxes between surface and groundwater is essential for an optimal management of the water resources both in rivers and in the groundwater, which accounts for a balance between conservation and exploitation. Both aspects of water quality and water quantity are important.

Many investigations – often field studies – concerning the river – groundwater interaction have been carried out in the field of ecology on the reach scale, which is the scale relevant for the occurrence of certain habitats and biocoenoses. Its extension is usually between tens and hundreds of metres so that the area is small enough to gather fine resolution data (Crowder, 2000). In larger areas like watersheds, entire valleys and so forth, detailed field investigations are no longer feasible. However, for river restoration and revitalization projects, information on the temporal and spatial distribution of water fluxes between surface and aquifer are of vital interest. In these cases, modelling techniques can be a helpful tool in order to provide information needed for both scientific and management purposes.

Alpine alluvial deposits are coarse and highly permeable. Moreover, because of frequent flood events in connection with high shear stresses on the river bed due to high flow velocities, colmation is rare or limited in time until the next flood reworks the river bed completely. The exchange of water between aquifer and river tends therefore to be strong in alpine valleys. Such highly dynamic systems show a high spatial and temporal variability also in the type and strength of its natural processes. If this is the case, infiltration (downwelling water flux from the river into the groundwater) and exfiltration (upwelling water flux from the aquifer into the river) are not constant in time and space. Moreover, the sign of the flux might change with time depending on the piezometric head in the river and in the aquifer. Here, feedback mechanisms between the two subsystems (river and aquifer) may occur, requiring an integral view of the overall system. This leads to the main task of this thesis: the coupling of a two-dimensional surface water flow model with a quasi-3D-groundwater model for the application in active alpine floodplains.

1.2. Motivation

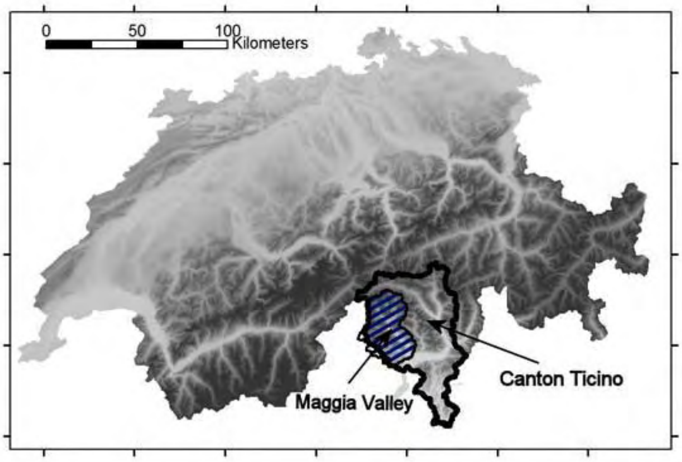


Fig. 1.1: Location of the study area Maggia valley in the South of Switzerland (from: Foglia, 2006).

In most of the watersheds in the Alps, water is extensively used for hydropower production. This phenomenon is however not restricted to the Alps, but occurs also in other regions of the world. Water for hydropower can basically be used in two ways: run-of-river power stations and storage power stations. In areas with steep reliefs, generally storage power stations are used. They are mostly characterized by water diversions downstream of the intake stations and some dams, which leads to strong streamflow regulation and often to a lack of water in the downstream river reaches. Depending on the legislation, concession and the environmental flow requirements, these impacts on the hydrological regime can be substantial up to zero water flow in the river. This is also the case in the Maggia Valley, an alpine valley in the Southern rim of the Swiss Alps, located in the Canton Ticino (Fig. 1.1). After the hydropower system went into operation in the 1950s, the released water was initially such that the Maggia River became completely dry over a reach of several kilometres. Only after negotiations, new rules have been enforced such that there is a constant minimum flow guaranteed, which allows for permanent water flow throughout the entire valley (Thorens and Mauch, 2002; Cerini, 2003). However, annually about 75 % of the original water amount is still diverted and released directly into Lago Maggiore, thereby circumventing a long reach of the Maggia River main valley. In the main valley, a constant release of 1.2 m^3/s remains during the winter season (October, 1st until June 14th), and 1.8 m^3/s during the summer season (June, 15th until September 30th) (Thorens and Mauch, 2002). Only if the capacity of the intake stations and diverting open-channel flow galleries is exceeded, the spillover ends up in the Maggia River. This means that the large flood events are not significantly effected, whereas the occurrence of moderate flow conditions is strongly diminished (Fig. 1.2).

A large part of the area affected by the changes in the streamflow regime is a braided area (Fig. 1.3) with a relatively pristine riverine landscape (Fig. 1.4). Since 1992, it is protected by federal legislation as part of the inventory of the wetlands of natural importance.

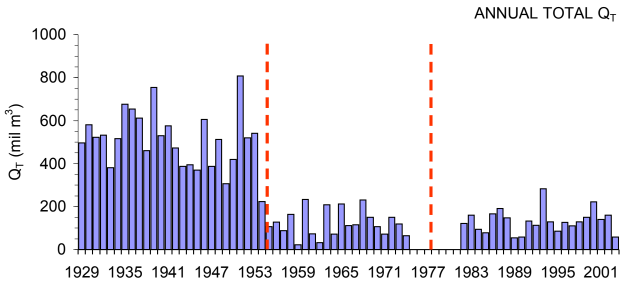


Fig. 1.2: Changes in the hydrological flow regime of the Maggia River due to hydropower operation (annual flow volume is shown). The red vertical lines separate time periods with different environmental flow regulation (see also Tab. 3.1). Data are from the station in Bignasco.

Fig. 1.3: Braided area between Riveo and Someo (aerial photography from 1933, source: Swisstopo, Wabern, Switzerland).

Fig. 1.4: The braided area of the Maggia valley.

Evaluation of aerial photography from 1933 until recent have shown a strong alteration of the spatial distribution pattern of vegetation within the braided area of the floodplain (Favre, 2004; Sturzenegger, 2005)(Fig. 1.5). On a long-term perspective, the vegetation distribution should reach – statistically speaking – steady-state dynamics, i.e. there should be a balance between destruction of vegetation due to flooding and scour, and on the other hand a resettlement of succession species due to rejuvenation, germination and growth. During the time of observation however, a strong tendency toward narrowing of the river and the exposed sediment area was observed related with a strong increase of softwood and hardwood floodplain forest. This general tendency occurs despite the fact that the river system is still able to rework the river bed and to destroy herbaceous and shrub vegetation significantly after big flood events such as in 1978. The question arises, whether the hydropower operation has had a significant contribution to these changes, be it due to the altered surface water flow regime, or to changes in groundwater levels. Another impact could also be represented by changes in the sediment supply within the catchment due to retention of sediments in artificial reservoirs in the upper part of the catchment. It should be noted that similar observations of changes in riparian vegetation due to streamflow regulation have been reported in other studies where generally a loss in the natural dynamics of the floodplain forest is a result of regulations (e.g. Johnson et al., 1995; Poff et al., 1997).

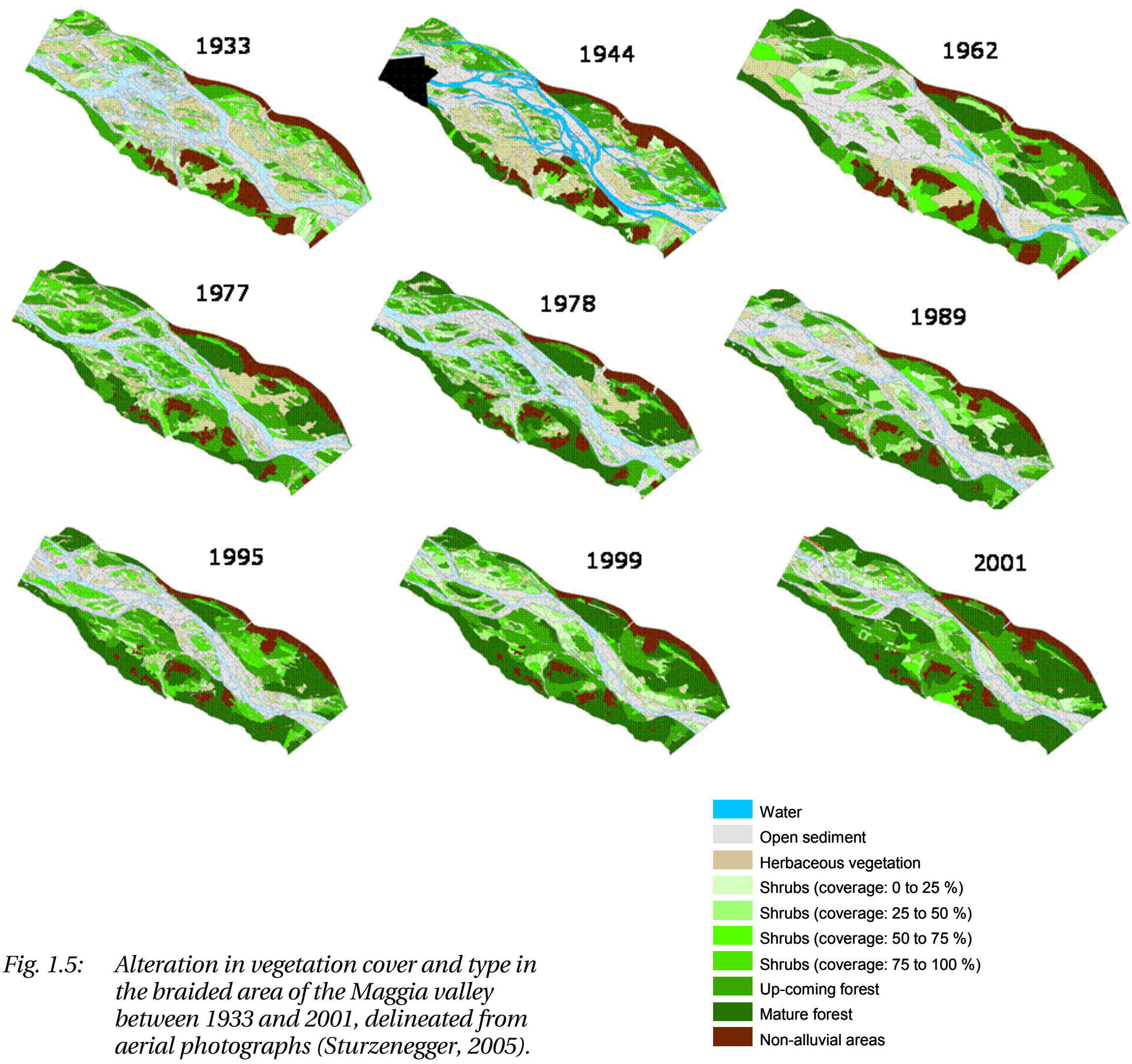


Fig. 1.5: Alteration in vegetation cover and type in the braided area of the Maggia valley between 1933 and 2001, delineated from aerial photographs (Sturzenegger, 2005).

To study the long-term impact on the floodplain vegetation, a modelling framework had to be developed, which allows to simulate the hydrological system including both surface water and groundwater in the alluvial plane. Besides the sediment transport with moving beds and gravel bars, erosion of plants, and the availability of nutrients, water is the decisive factor for the development of riparian vegetation, both in terms of suffering from periodic or episodic flooding and water stress due to low groundwater tables.

Therefore, the MaVal project (Valle Maggia Project: Interactions between surface water and groundwater in an Alpine environment – Assessment, Modelling, Ecosystem Response, Impact Analysis; www.maggia.ethz.ch) was initiated at ETH Zurich in collaboration with SUPSI-IST (*Scuola Universitaria Professionale della Svizzera Italiana; Istituto di Scienze della Terra*, Trevano, Ticino). For some special issues, other project partners were included, too (see appendix 1). This project aims at the investigation of the impact of hydropower operation onto the hydrological flow regime and its influence on ecosystems. So far, the focus is hydrologically on quantitative aspects, not on qualitative ones. This dissertation is part of this project providing the link between hydropower, surface water, groundwater and vegetation.

1.3. Rationale

1.3.1. Generalities

The final goal was to develop an integrative modelling system accounting for all relevant water fluxes, which are important to understand the ecologically relevant processes in alpine active floodplains. Therefore, a hydrological watershed model, already accounting for potential anthropogenic water diversions and abstractions, a hydrodynamic open-channel model and a groundwater model had to be combined (Fig. 1.6). The modelling outcome provides the essential information for the application of a vegetation growth model in the floodplain. Within the MaVal project, a previous work has already been carried out by Foglia (2006), setting up a groundwater model, which has been linked with a watershed model.

A strong interaction between river and aquifer (Fig. 1.7) with a high variability in space and time was observed in the field. It can be seen in the illustrated parameter transect, which is located in the centre of the valley, that piezometric heads increased up to 5 m due to a storm event related to a flood in the Maggia river. The most striking findings are that the piezometer which lies most remote from the river reacts with the highest phreatic rise although as the latest of the three piezometers. Besides this, the strongest rise occurs within the time interval of 1 to 2 days.

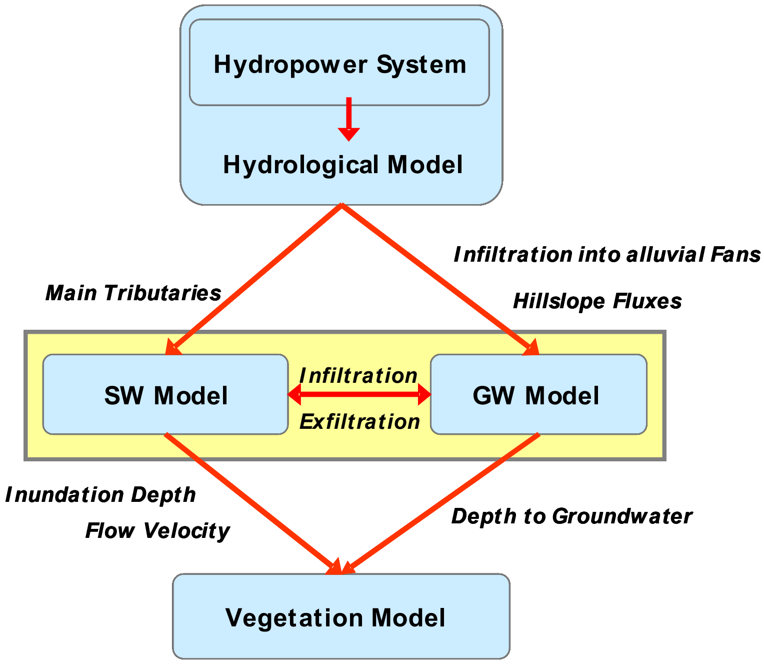


Fig. 1.6: Schematic of the modelling framework of the MaVal project. The central yellow part of the coupling of the surface water and the groundwater model is the focus of this work.

This leads to the aim of this work, which was the coupling of an open-channel flow model with a groundwater model to be applied under high dynamics of the channel flow and the surface water – groundwater interaction thus accounting for exchange and feedback mechanisms at the important time and space scales, which affect the water controlled aquatic ecosystem processes. For this purpose, a surface water model, which is suitable for simulating flow with a very high variability and relatively steep slopes, had first to be selected and implemented. Second, a new coupling scheme had to be developed to combine the surface water model with the groundwater model. The model implementation and parameter estimation of the previous study concerning the groundwater and watershed part (Foglia, 2006) was used as an input to this work. The objective of the modelling is to provide spatially and temporally distributed hydraulic and hydrological variables of ecological importance, which are required for the vegetation modelling.

Despite its application and specific target question in the Maggia Valley, the modelling framework to be developed aims at being "universally" applicable in any region, where surface water – groundwater interaction is an important process in the hydrological cycle. Fields of applications are seen not only with respect to environmental flow requirements, but in all cases, where surface water groundwater interaction plays an important role, e.g. also in river restoration projects.

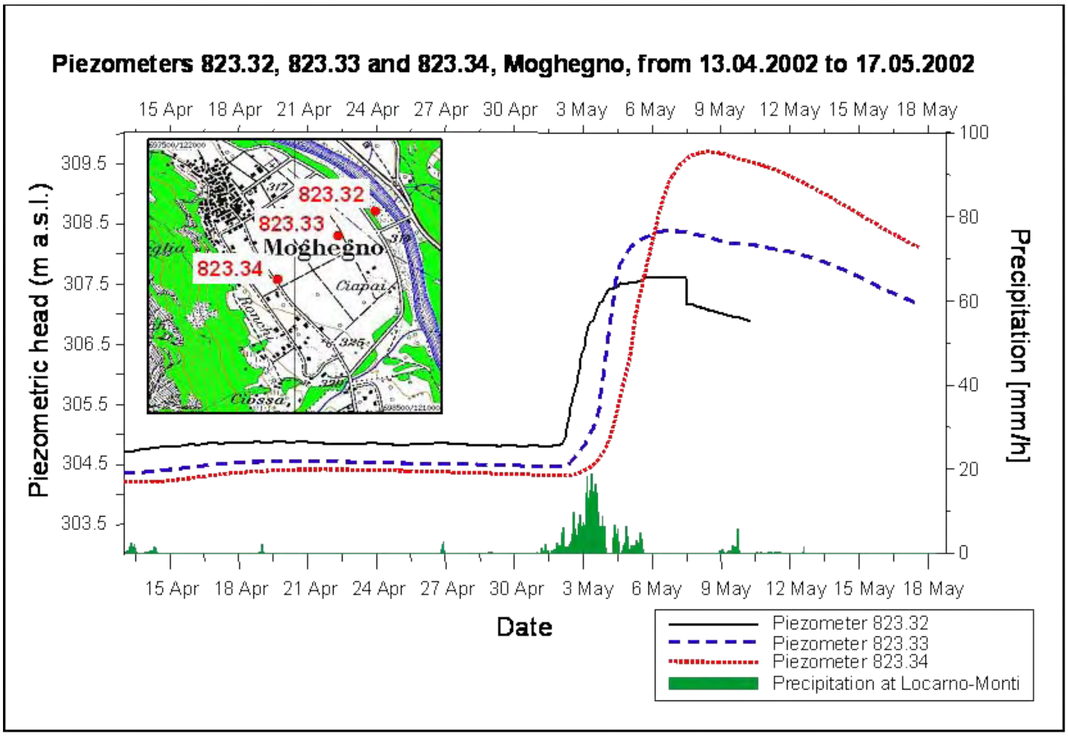


Fig. 1.7: Response of groundwater head due to variation in streamflow.

1.3.2. Specifications and Requirements

The overall modelling framework will combine a hydrological distributed watershed model, a spatially and temporally explicit hydrodynamic surface water model and a groundwater model as illustrated in Fig. 1.6. Both the surface and the groundwater model serve as boundary conditions or driving forces for a vegetation growth model in the braided area, which is currently under development within the MaVal project. The coupling of the surface water and the groundwater model had to be complete in the sense that possible exchange fluxes in both directions could be simulated, variable in space and time. The nature of the processes and the topography required at least a 2D approach for the surface water model because of the existence of a braided river system with highly variable flow conditions. Enhanced numerical requirements for the surface water model were given because of the relatively steep slope of the study reach between 0.5 % and 2 % and rather large variations in runoff (0.4 m^3/s up to 1000 m^3/s).

Because of the high dynamics of the system with strong response of the groundwater heads on variations in stream flow (Fig. 1.7) and hillslope contributions into the aquifer, the modelling system had to be capable for transient simulations. Limitations in the computer power and the complexity of the problem restricted the focus of this work so far to the simulation of single flood hydrographs. This corresponds also to the present runoff regime in the Maggia River with long-lasting constant flow conditions interrupted by single high flood events. Nevertheless, possible strategies to overcome these computational limitations, thus allowing also long–term simulations, are outlined in the outlook (§ 8.3.3), and are suggested to be applied after the demonstration of the principal suitability of the modelling system under transient conditions in this work.

The watershed model should have, in this specific case, only a one-way coupling, since the aquifer does not exert any feedback on hillslope surface or subsurface runoffs. The output from the watershed model should just be the input for the surface water model and the groundwater model by accounting for the flow in tributaries and diffuse fluxes from the hillslopes.

The modelling system must be numerically robust. Equally important, it should serve as a suitable tool to answer ecohydrological questions, especially such as the development of vegetation within the braided area of the Maggia Valley. Spatially distributed and time-variant hydrological and hydraulic variables with ecological importance can be derived. In the present case, they are: time respectively season and duration of flooding, frequency of flooding, flow depth, flow velocity, shear stress, and exchange rates between river and aquifer. Furthermore, temporal and spatial statistics can also be derived from the modelling results. Both will be an essential input for any vegetation growth model, be it more deterministic or stochastic. Besides this, the results can be used for the analysis of the processes in the past to gain a better understanding about the interrelations between hydrology, hydraulics and vegetation.

Although sediment transport and changing river bed morphology is an important factor in active alpine floodplains, it will not yet be considered in this work. This is meant to be part of future work, whereas in this work it is assumed that only major floods are responsible for significant changes of the river bed morphology and topography, and minor and locals changes will not effect the overall pattern of the river-exchange mechanism.

1.4. Innovation

The innovative parts of this work are the coupling between a transient 2D hydrodynamic stream flow model and a quasi-three-dimensional groundwater model within a modelling framework, which accounts also for the hydrological processes in the entire watershed. The specific point is the scale considered in this modelling framework. Large scale processes within a watershed are combined with meso-scale groundwater and small scale hydrodynamic processes in the river thus allowing for the simulation of all the hydrological processes affecting areas with strong interaction between surface water and groundwater. Furthermore, a range of three orders of magnitude had to be accounted for.

This work can therefore also be seen within the ecohydrological context. It allows an integrated view of the most important water related processes within a riverine corridor and goes beyond the limits of those modelling approaches, which are restricted to the reach scale.

The modelling framework developed in this work is therefore expected to allow for a more integral view of ecohydrological systems and their processes, to bring the ecohydrological research a step further, and to provide a new tool for river restoration and management projects.

2. State-of-the-Art

This chapter deals with the presentation of the state-of-the art in terms of river-aquifer interaction and the existing modelling approaches of coupling streamflow and groundwater flow. Since the implementation of the surface water model was also subject of this work, some additional description is also given here. The state-of-the art of the watershed models and groundwater models as stand-alone models is presented here only as far as it is essential to understand the entire modelling framework, its implementation and the interpretation of its results.

2.1. Watershed Modelling

Hydrological Watershed Models have been used for decades in the hydrological science. There are at least three main classification schemes: 1. lumped vs. semi-distributed and fully-distributed models, 2. black-box vs. conceptual and physically-based (= process-oriented) models, and 3. deterministic vs. stochastic models (e.g. Beven, 2001). Within the modelling framework, which had to be developed in this study, simulation of streamflow hydrographs for different subcatchments as well as spatially distributed subsurface flow should be obtained as a modelling output to connect with the groundwater and surface water model. This is achieved using spatially distributed models, which allow for a discretization in space and time, including the most relevant hydrological processes. While applying these models, the best compromise has always to be found between the type of problems to be solved, the complexity of the catchment characteristics and its dominating physical processes and data availability. A description of the most widely used watershed models can be found in Singh (2006). Recent developments are in the direction of integrated models, which combine the classical rainfall-runoff models with channel flow for groundwater components as it is described in § 2.4.3.

2.2. Surface Water Modelling

The term “surface water model” is used here in contrast to the term “groundwater model”, and open-channel flow models (i.e. flow routing models) are meant. This is due to the fact that in the literature the expression surface water – groundwater interaction is widely used for the interaction between river and groundwater.

2.2.1. Classification of Flow-Routing Models

In general, two types of flow-routing models can be distinguished: hydrological (empirical) routing methods and hydro-dynamic or hydraulic (dynamic) routing methods. The first group is mostly applied in one dimension, whilst the hydraulic models are set up in 1D, 2D, and 3D. The hydrological routing methods use the continuity equation and a certain reservoir-outflow relationship. They can be applied only for a specified location and require some parameter fitting, therefore at least one flood wave propagation has to be measured for this reason. Representatives of this family are the Muskingum or the modified puls-method. The advantage of these empirical models are their ease of use in terms of mathematical effort. Drawbacks are however that they have to be calibrated, since they do not solve the physical dynamic equations. Therefore, they cannot be used under conditions which are distinct from the calibration period, such as changes in the channel geometry, and that they have to be set up for each location separately. They are used mainly for large rivers with relatively large time steps.

A one-dimensional model of an intermediate type is the Muskingum-Cunge method (Cunge, 1969), which has its origin in a special solving scheme of the kinematic wave approach (see next section, eq. 2.7), which is one of the simplifications made by hydraulic models, but belongs from the formal aspects to the hydrological approaches. It has been shown (Cunge, 1969) that due to the special choice of the parameters, the solution is an approximation to the even less simplified diffusion equation (eq. 2.10). Changes in river geometry can be accounted for, but limitations are in the case of rapid changes in discharge or more complex river geometry. Nevertheless, it has been implemented in different software packages such as in the open-accessible HES-HMS (US Army Corps of Engineers, 2001).

Much more powerful and therefore widely used are the dynamic hydraulic models. They can give the results for each point along the river or in each grid point in case of a two- or three-dimensional model. The development of numerical models for the simulation of open surface flow began in the 1950s (Du, 1997).

2.2.2. Hydrodynamic Models

Physics

Hydrodynamic flow is fully described by the continuity equation related to the well-known Navier-Stokes equation for the conservation of momentum (2.1)(White, 2003), which is the partial differential equation for hydrodynamic flow in the general case. This equation is however very complex and cannot be solved analytically except for some special cases, i.e. simplifications and special boundary conditions.

$$\rho \frac{D\boldsymbol{v}}{Dt} = \nabla \cdot \boldsymbol{P} + \rho \boldsymbol{f} \quad (2.1)$$

where ρ is the fluid density [$\mathbf{M\,L^{-3}}$], D/Dt is the substantive derivate [$\mathbf{T^{-1}}$], $\boldsymbol{v}$ is the velocity vector [$\mathbf{L\,T^{-1}}$], $\boldsymbol{f}$ is the body force vector [$\mathbf{L\,T^{-2}}$], and $\boldsymbol{P}$ is a tensor representing the surface forces applied on a fluid particle [$\mathbf{M\,L^{-2}\,T^{-1}}$]. Using water as fluid, the simplification of incompressibility can be made.

For open channel flow, the shallow water wave equation system is applied: Depth-averaged variables are used and can be applied when the ratio between wave length and flow depth is below 20, because the velocity field in this case is still relatively uniform. The continuity equation together with one equation of motion must be coupled to fully describe physically the open-channel flow. This could principally be the momentum or the energy equation. Generally, the momentum equation is used because hydraulic jumps can be calculated this way without an explicit description of the internal energy dissipation. For the one-dimensional case, it is known as the de-St.-Venant equations. For sake of simplicity, the following considerations are made for this one-dimensional case. Different ways of its formulation exist, one possibility is the following:

continuity equation: $$\frac{\partial A(z)}{\partial t} + \frac{\partial Q(x,t)}{\partial x} = q \quad (2.2)$$

momentum equation: $$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\beta \frac{Q^2}{A}\right) + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0 \quad (2.3)$$

The variables have the following meaning: x is the longitudinal direction [$\mathbf{L}$], z is the vertical datum [$\mathbf{L}$], t is time [$\mathbf{T}$], Q is discharge [$\mathbf{L^3\,T^{-1}}$], A is the cross-section of flow [$\mathbf{L^2}$], q is a sink or a source term [$\mathbf{L^2\,T^{-1}}$], β is a dispersion-correction factor [-], g is the gravity constant [$\mathbf{L\,T^{-2}}$], and S_f is friction slope [-].

Instead of the discharge Q or correspondingly – when divided by the cross-section area A – the velocity, also the water level or flow depth can be used in the formulation of the equation system. The first case uses conservative variables, whilst the second approach uses non-conservative ones.

River geometry is reflected by the parameter A at any location x, which can be measured in the field. The dispersion-correction factor β accounts for the non-uniform horizontal velocity profile. It depends slightly on the cross-section of the profile and the degree of turbulence, especially on the ratio between flow depth and sand roughness, and lies mostly in the range between 1.01 and 1.18 (Beffa, 1994). The by far most difficult estimation is the friction slope S_f. It accounts for all energy losses like inner friction due to laminar or turbulent losses, bed friction, wind friction at the surface and any other frictions, which lead to some energy losses. This is explained in detail in § 4.5.4 in connection with the model 2dMb, which has been used in this study.

This equation system is not analytically solvable, not even in the one-dimensional case. For this reason, numerical methods have to be used. Most common are the method of characteristics, finite-differences, finite-elements and finite-volumes (see below). Extensive discussion about the theory and use of the shallow water-wave equations, its numerical models and its application can be found in Chow (1973) or Cunge et al. (1980). Chaudhry (1993) provides several numerical solving algorithms.

Since solving the entire momentum equation (2.3) is still numerically and therefore computationally demanding, several simplifications are made in order to solve it. For better illustration, the momentum equation is resolved for S_f:

$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{1}{gA}\left[\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\beta \frac{Q^2}{A}\right)\right] \quad (2.4),$$

where S_0 is the bed slope [-].

In general, the simplifications can be classified into three groups of methods. The kinematic methods, the diffusion methods and the fully-hydrodynamic methods. The kinematic methods neglect the first two terms on the right side, which are the local and convective acceleration terms of the momentum equation. The momentum equation is therefore reduced to the equation of uniform flow and reduced to the following equation system:

$$\frac{\partial A(z)}{\partial t} + \frac{\partial Q(x,t)}{\partial x} = q \quad (2.5)$$

$$S_f = S_0 \quad (2.6)$$

Using an empirical friction law leads to Q being a local function of A, and when put into the continuity equation (2.5), the equation for the kinematic wave (2.7) is obtained:

$$\frac{\partial Q}{\partial t} + c\frac{\partial Q}{\partial x} = cq \quad (2.7),$$

where c is the kinematic wave velocity at a given location [**L T**$^{-1}$]. This approach leads to the following characteristics or constraints: there is only advective motion; if c is constant, there is only translation. There is no flattening of the wave while it is propagating, and the wave can move only in the downstream direction. This approximation can therefore be used only if no backwater effects have to be taken into account, if no hysteresis effects occur in the water level-discharge relationship, if the bed slope is high and the temporal changes in the hydrograph are small. Using numerical models, the peak of a kinematic wave is nevertheless mostly flattened due to numerical diffusion.

If changes in flow depth are considered in addition to the bed slope, the diffusion method is obtained in contrast to the kinematic wave approach. It is characterized by the following equation system:

$$\frac{\partial A(z)}{\partial t}+\frac{\partial Q(x,t)}{\partial x}=q \quad (2.8)$$

$$S_f=S_0-\frac{\partial y}{\partial x} \quad (2.9)$$

After some transformation, the parabolic differential equation, namely the diffusion equation (2.10) is obtained:

$$\frac{\partial Q}{\partial t}=D\frac{\partial^2 Q}{\partial x^2}-c\frac{\partial Q}{\partial x}+cq \quad (2.10)$$

Here, c [**L T**$^{-1}$] is celerity and D is the diffusion coefficient [**L**^{2} **T**$^{-1}$], c is responsible for the propagation of the wave and D for a dampening. The diffusion approach has already a wide field of application, since it accounts for backwater effects and can be used also for relatively fast temporal changes in the hydrograph. Temporally variable backwater effects can however not be considered, neither is this method suitable for very fast changes in the hydrograph such as e.g. for dam breaks and rapid swell and sunk (hydro-peaking) after the sudden opening or closing of a weir. In these cases, the full hydrodynamic method has to be used.

The same considerations as above apply principally also for the two-dimensional case. The simplification of the kinematic wave is however not so meaningful for the 2D-modelling approaches, since they are used in case of complex river bed topography. Only in the field of surface runoff from hillslopes, some hydrological models use the kinematic wave approach as the hydrodynamic model in the surface runoff module.

So far, the physics of the problem and the governing equations to be used were treated in this paragraph. There are still several issues to be discussed when it comes to numerical modelling like dimension of the models, solving scheme, grid structure, and others. In order to avoid repetitions, these issues will also be discussed in § 4.5, when it can be set in relation to the description of the model being used.

Numerical solving schemes in surface water modelling

Since the suitability of the surface water model to be applied in the modelling framework depends both on the study site characteristics and on the numerical solving scheme, the second point is briefly discussed here. In principal, the same schemes are applied also in the field of groundwater modelling, but they are addressed here with respect to the application in surface water models.

Basically, three different numerical methods are used to solve the shallow water wave equations: finite differences, finite elements, and finite volumes. Similar to the finite element method is the finite points method (Du, 1997), which is however not yet widely used. Whilst the finite difference method requires a structured grid, finite element and finite volume methods can handle both structured and unstructured grids. Finite volume methods use the integral form of the partial differential equations, which are used for the discretization for the finite differences. Therefore, the calculations with the FD and FV schemes should converge to the same solution, if uniform grid spacing is used (Beffa, 1994). Finite elements methods are characterized by the calculation of state variables in the nodes. Between the nodes, these variables are interpolated. Fluxes over the edges of the cells are integrated regarding the values of the interpolated state-variables. Hydraulic jumps often generate problems, because discontinuities can hardly be handled by interpolation schemes. FE methods have widely been used at the beginning of numerical modelling of surface waters, because they allowed for flexible meshes, and had therefore an advantage over the FD schemes. They have originally been applied in structural engineering, and then transferred to hydrodynamic problems. Because of their limitations in handling discontinuities such as hydraulic jumps and internal wet-dry-boundaries, they have lost their attraction in favour of FV methods, which allow a more robust handling of these discontinuities and account better for the conservation of mass due to the integral form of equations. A cell-centred grid together with a FV scheme can guarantee that the water balance over the grid edges will be fulfilled.

Applications in ecology

Recently, 2D-hydrodynamic models have also been used in order to simulate hydrological and hydraulic conditions of aquatic habitats in rivers (Crowder, 2000). Often, the scale of these model applications is much smaller, but what is important, a very high precision is needed, which is higher than it is for instance the case for flooding problems. Crowder (2000) used e.g. the finite-element model RMA-2V (King, 1990) for simulating fish habitats on a reach of 61 m with an average mesh size of 0.02 to 1.3 m^2

depending on the model implementation. In the MaVal study, the focus lies on those ecological aspects, which are related to a larger spatial and temporal scale, therefore also the precision of the hydrological variables can be less.

2.3. Groundwater Modelling

The physical description of saturated flow in the aquifer (groundwater flow) is given by Darcy's law (2.11) (for extensive description about the processes see e.g. Bear (1979) or Matthess and Ubell (1983)). This assumption holds by far for most of the cases. Variation of the density is neglected, and the continuity equation of fluid flow then becomes the well-known Boussinesq equation (eq. 2.12).

$$q = -\boldsymbol{k} \nabla h \quad (2.11)$$

$$\nabla \cdot (\rho \boldsymbol{u}) = \rho \nabla \cdot \boldsymbol{u} = \nabla \cdot \boldsymbol{u} = 0 \quad (2.12)$$

q is groundwater flow [**L T**$^{-1}$], $\boldsymbol{k}$ is the tensor of hydraulic conductivity of the aquifer [**L T**$^{-1}$], h is pressure head [**L**], ρ is specific weight of the fluid (i.e. water) [**M L**$^{-3}$], and $\boldsymbol{u}$ is flow velocity [**L T**$^{-1}$].

There are differences in case of confined or unconfined aquifers. In confined layers, the water table is given by the upper bound of the permeable layer, and the transmissivity is therefore given by the geometry of the system. The water table and the piezometric heads are not identical, and the heads can fluctuate. In unconfined aquifers, the transmissivity is given by the product of the hydraulic conductivity and the thickness of the water body and is hence variable. The calculation of the water table is therefore more difficult because of the “moving boundaries”. This is the reason, why calculations of confined groundwater systems are faster.

First numerical groundwater models were used in relation to water supply questions (Kinzelbach, 1986). They were in horizontal 2D, i.e. just one single aquifer layer. Only horizontal flow components can be accounted for using this approach. Most of the recently used groundwater models use either the finite-differences or finite-elements. The implementation of finite differences is numerically easier, but has the drawback of the necessity of using structured grids. Unstructured grids are however flexible and can better follow internal structures such as rivers, which could act as internal boundaries, or local refinement is easier. Many of the models account for three dimensions, but for larger areas, they are often applied just in two-dimensions, if data availability is a constraint or if vertical flow components are not relevant. Some model developments use also a quasi-three-dimensional approach solving the equations for each horizontal layer separately and then determining the vertical fluxes as a result of the head differences in the distinct layers, such as the MODFLOW-2000, which has been used in this work.

2.4. River – Groundwater Interaction

2.4.1. Physical Process

Before describing the complex process of river-groundwater interaction, a definition of some terms shall be given, since many different expressions for the same process can be found in the literature. Sometimes, the terms *gaining* or *loosing rivers* are used. The first means that there is a water flux from the aquifer into the river, the second means the opposite. Other authors use the expressions *gaining* or *loosing aquifers*. They see the process from a different perspective, so gaining aquifers mean water flowing from the river into the aquifer, loosing aquifer the opposite. There is also a pair of terms from the perspective of the moving waters: *downwelling* and *upwelling water*. The first flows from the river into the aquifer, the latter in the opposite direction. In this study, mostly the expression *infiltration* and *exfiltration* are used, always from the perspective of the groundwater. Infiltration therefore means water flux from the river into the aquifer, exfiltration from the aquifer into the river.

The detailed description of the process of river-groundwater interaction is complicated due to its three-dimensional nature of the process. Water can flow both through the river bottom and through the banks. In any case, it must flow in the direction from the higher to the lower pressure potential. In many cases, colmation of fine material occurs on the river bed so that the water flux is hindered. If the river carries a lot of suspended solids, these fine particles will enter the porous space of the underlying bet material and clog the void space. This colmation will increase with time until a reworking of the river bed takes place. This happens during high flood events. The whole river bed will be in movement, the fine material will be washed away, which leads often to a remarkable reduction of the colmation. This effect can be shown e.g. in temperature time series of piezometers next of a river, where temperature shows daily variations, if the river water infiltrates into the groundwater, and where these variations become smaller when colmation takes place (e.g. Fette, 2005).

During flood events, bank storage can occur. High water levels in the river let water infiltrate into the bars or river banks due to an outward gradient of water level in the river and piezometric head in the adjacent groundwater. After the flood wave has passed through the cross section, this gradient can be reverted leading to releasing water from the bank storage back again into the aquifer. It is reported in the literature using numerical simulations that, under certain circumstances, this process can even influence the flood hydrograph (Pinder and Sauer, 1971).

The bank storage however leads to a local increase of the groundwater table, which will also create a gradient in piezometric head in the aquifer in direction away from the river. Hence, there will be only a part of this water flowing back into the river, another part will flow and remain in the aquifer. This proportion will depend on the duration of the flood water level and the hydraulic conductivity within the aquifer.

A distinction can be made, whether the groundwater table and the river are connected or not. In case of a connected river, the aquifer below the river bed is saturated. Unconnected rivers show an unsaturated zone in the aquifer below the river bed. In the first case, Darcy's law in 3D for the saturated flow can be applied in the zone below the river bed. In the other case, flow must be described for unsaturated conductivity, which is not trivial in three dimensions, since the spatial distribution of subsurface moisture is not known in most cases. After a while, there will be nevertheless a relatively constant vertical water flow in case of infiltrating unconnected rivers due to steady-state conditions. The horizontal flow however is difficult to describe in such a case.

Colmation takes place not just in a well-defined layer, but in a zone, which can vary in time without sharp boundaries. The loss in hydraulic pressure head with depth is therefore not constant. This has been proved by detailed field measurements in the Thur River (Switzerland) at EAWAG (*Swiss Federal Research Institute for Aquatic Sciences*), where hydraulic pressure probes have been installed in different depths below the river bed. This study has also shown that a mathematical description of this behaviour is very difficult, and that the results of these kinds of measurements cannot be used for the determination of the river bed resistance for a whole river reach due to scaling problems, additionally to the problem of the representativeness of a spot measurement for the determination of a parameter value for an area. This is due to the fact that these kinds of measurements are two-dimensional, whilst the entire process has clearly a three-dimensional character.

Another scale related issue is the occurrence of riffles and pools. Along the longitudinal profile of a river, the thalweg of the river does not have a constant slope, but shows ondulating features of different size. Riffle-pool systems can occur resulting in alternating small-scale upwelling and downwelling of water (features "a" in Fig. 2.1). Superimposed may be larger features (feature "b" in the same figure) having a similar effect until features of geological controls (Brunke and Gonser, 1997; Wright et al., 2005). Whether some river water is considered to be exfiltrated water, stemming from the aquifer or not, is therefore a definition of scale. Also water quality (chemistry and temperature) differs between river and groundwater. In case of small-scale exchange, these water characteristics will however be very similar, but will differ more and more the larger is the considered scale of this pattern of upwelling and downwelling.

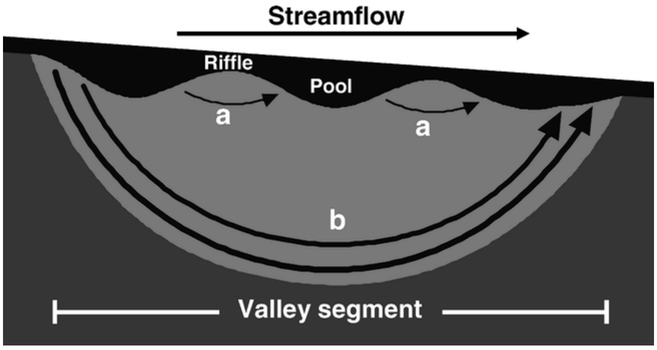


Fig. 2.1: Schematic of up- and downwelling due to geologic controls of different scales (from: Wright et al., 2005).

Field surveys have shown that a variability in the pattern of water quality in the river can result due to the different water quality of upwelling water (Holocher et al., 2001; Uehlinger et al., 2003; Burgherr and Ward, 2001; Tockner et al., 1997). These patterns underly also a temporal evolution depending on the changes in the subsurface flow paths and conditions (Malard et al., 1999). This leads to a patchy distribution of distinct riverine habitats, where different biocoenoses can develop on a lotic scale. This process of river-aquifer interaction plays therefore also a very important role in riverine ecology over different spatial scales.

Distinct situations for the exchange process occur, whether the river flow direction is parallel or perpendicular reps. oblique to the groundwater flow direction. A major effect will be especially for the water quality in case of perpendicular underflow of the groundwater because of a good mixing of the water, whilst the mixing is much weaker, but more steady in the first case.

2.4.2. Approaches for the Mathematical Formulation of the Surface Water – Groundwater Interaction

Analytical solutions for the surface water-groundwater interaction have been provided e.g. by Anderson (2003). These solutions are however restricted to special cases and are not applicable for large areas with spatial heterogeneities. These detailed descriptions are relevant for limited cases on a small scale such as pumping wells near river courses. On larger scales, these detailed mathematical formulations of the physical processes are not applicable because of the heterogeneity of the underlying parameters. In most cases, leakage factors, which describe the river-aquifer exchange, are subject to calibration, their number and thus their spatial variability is therefore kept as small as possible in order to avoid an overparameterization of the model. For investigations on a larger scale, i.e. larger than a river reach, as it is the case also in this work, it is basically important to account for the overall amount of exchanged water rather than to describe the underlying process in such detail. For this reason, mostly conceptual formulations of the river-aquifer exchange process have been developed.

The most common conceptual formulation of the exchange fluxes between river and aquifer is the linear proportionality between the water level in the river and the piezometric head in the aquifer (e.g. Matz et al., 2006). The proportionality factor is called the leakage factor. It is often considered to be constant over time and also to be the same for infiltration and exfiltration processes. As this factor is very difficult to measure and is furthermore scale-dependent, this approach has an big advantage. It yields to just only one calibrating parameter, since heads can be measured or modelled. This formulation is also used in the MODFLOW and will also be applied in this study (see eq. 4.3).

It is also possible to use different leakage factors for infiltrating and exfiltrating conditions (see eg. Wald and Schiffler, 2002). As infiltration and exfiltration takes place not only in the vertical direction, which is assumed in the approach described above (eq. 4.3), but also horizontally through the river bank, the physical description of the problem is more complex.

More sophisticated conceptual approaches which are operationally implemented are hard to find in the literature. A comprehensive discussion about different conceptual mathematical approaches to describe the surface water – groundwater interaction is indeed found in Sophocleous (2002), but they are discussed mainly from the theoretical point of view. As they all have higher data requirements than available for this study, these approaches were not considered here.

2.4.3. Existing Modelling Approaches

Different approaches to modelling the river-groundwater interaction in terms of coupling different models or modules can be found in the literature. The oldest approach to be found is probably from Freeze and Harlan (1969). The existing approaches can be divided basically concerning four aspects: 1. the river is represented in one or in two dimensions, 2. the model provides steady-state or transient solutions, 3. the exchange of water is formulated by mass exchange or by fully integrating the equations for the surface water and the groundwater into one numerical solving scheme, 4. the mathematical equation for the formulation of the coupling process. Additionally, many approaches for coupling hydrological watershed models with groundwater models exist. In all approaches, groundwater flux is represented in two or three dimensions. An overview over the most important existing coupling approaches accounting for surface and groundwater are listed in Tab. 2.1 and explained in more detail in the following text.

Tab. 2.1: Overview over important existing models coupling surface water and groundwater.

Hydrological Model	Hydraulic Model		Groundwater Model	Reference	Suitability / Advantages and Disadvantages
	1D	**2D**			
WaSim-ETH	kinematic wave	---	integrated, simplified approach	Schulla, 1997	Hydrological models with groundwater component, but not suitable for detailed hydraulic simulation of channel flow
MIKE-SHE	---	---	MIKE-SHE	Abott et al., 1986a	
ArcEGMO	---	---	MODFLOW	Sommer et al., 2002	
SWAT	---	---	MODFLOW	Arnold et al., 1993	
WaSim-ETH	---	---	MODFLOW (LAKE and RIVER Package)	Krause and Bronstert, 2002	
---	BRANCH (unsteady)	---	MODFLOW	MODBRNCH, Swain and Wexeler, 1996	full transient coupling, but only 1D channel flow
---	SFR (unsteady)	---	MODFLOW		
---	HYDRET	---	MODFLOW (RIVER package)	Wald and Schiffler, 2002	
MIKE-SHE	MIKE-11	---	MIKE-SHE	Matz et al., 2006	
---	---	RIVER package	MODFLOW	McDonald and Harbaugh, 1988	full coupling with 2D channel flow, but no transient simulation of the channel flow
---	---	kinematic wave	Boussinesq equation	Peyrard, 2006	under development
MODHMS	MODHMS	---	MODHMS	Panday and Huyakorn, 2004	fully integrated approach using one single solving scheme, but usable for the catchment scale and not for detailed hydraulic representation of the river channel
HydroGeoSphere	---	HydroGeo Sphere	HydroGeo Sphere	Sudicky, URL	

Coupling of hydrological models with groundwater models

Some hydrological models have already a groundwater component included. So e.g. the watershed model WaSim-ETH (Schulla, 1997; Schulla and Jasper, 2006; Jasper and Schulla, 2007), which includes a one-dimensional kinematic wave approach for the channel flow and a two-dimensional groundwater component. MIKE-SHE

(Abbott et al., 1986a; Abbott et al., 1986b; Graham, 2002) is a physically-based hydrological model accounting for all relevant hydrological processes in the catchment integrated with groundwater flow. The watershed model ArcEGMO has been coupled with the groundwater model MODFLOW (Sommer et al., 2002). It has been in successfully operational use for many water management applications in Germany. The quasi-distributed watershed model SWAT (Arnold et al., 1993) based on the concepts of hydrological response units was coupled with MODFLOW for long-term investigations in Korea (Chung et al., 2006). Another way of coupling was undertaken to couple WaSim-ETH together with MODFLOW-96 for accounting the exchange between saturated areas with the groundwater. It was implemented in the Elbe watershed (Germany), where groundwater levels are high and may generate large saturated areas. The lake approach (Rembe and Wenske, 1998) and the RIVER Package (see § 4.4.7) in the extended MODFLOW are used for the interaction between surface waters and groundwater (Krause and Bronstert, 2002).

MODFLOW is by far the most popular groundwater model for coupling with hydrological models. The reason is its open source code, its large applicability and its modular structure. This list of hydrological models coupled with groundwater models is not complete, but should show the research activity in this field. However, all of them have the limitation of representing the channel flow just in a simplified way, which is not capable to simulate complex river bed topography as it can be found e.g. in the Maggia valley.

The unsaturated zone is the interface or transition zone between the space domain usually modelled by hydrological models and the space domain of the groundwater models. In both cases, the simplified models neglect the unsaturated zone, the more advanced models do include it. Different approaches exist from simple storage or balance approaches up to the full solution of the three-dimensional Richards equation for unsaturated conditions.

With respect to groundwater modelling, its inclusion is neglected in many practical applications, especially because of lack of knowledge about the soil properties and moisture distribution in the unsaturated zone. It is however especially important for the time lag of recharge be it due to precipitation or to infiltration from a river, which is unconnected from the groundwater table. Also in case of evapotranspiration processes, the consideration of the unsaturated zone may become important, if sub-yearly dynamics of the system are of interest.

Examples for the consideration of the unsaturated zone accounting for the vertical water fluxes are given by some Soil-Vegetation-Atmosphere (SVAT) models, using the water balance method (e.g. the model SOWAS (Disse, 1995)), or solving the Richards equation for unsaturated flow (e.g. the model SWATRE (Nieuwenhuis, 1986;

Wesseling, 1989)). The latter approach can also be found in some hydrological models (e.g. WaSim-ETH (Schulla and Jasper, 2006)). In our study, the inclusion of the unsaturated zone has not been necessary so far, since recharge from precipitation is comparably small and most parts of the river in the main Maggia valley are directly connected with the groundwater.

Coupling of one-dimensional channel flow models with groundwater models

Different approaches have been developed to couple a two- or three-dimensional groundwater model with a one-dimensional surface water flow model. One example is MODBRNCH (Swain and Wexeler, 1996; Swain 1994; USGS, URL_c). In this case, the groundwater model MODFLOW-96 (McDonald and Harbaugh, 1988) has been coupled with the open-channel flow model BRANCH (Schaffranek et al., 1981). "BRANCH simulates steady or unsteady flow in a single open-channel reach (branch) or throughout a system of branches (network) connected in a dendritic or looped pattern by solving the one-dimensional equations of continuity and momentum for the river flow. Channel-aquifer flows are leakage through a confining layer or river bed. Computation of this leakage in the ground-water and surface-water system allows these processes to be coupled for simulation purposes." (USGS, 2007a) The ground-water flow equation is solved using the finite-difference approximation. The BRANCH model uses a weighted four-point, implicit, finite difference approximation of the unsteady-flow equations. A leakage term has been added to the equations in the BRANCH model and was coupled through the leakage quantity to the MODFLOW-96 model (USGS, URL_b).

Another example of coupling channel flow with groundwater is the Streamflow Routing Package (SFR) (Prudic et al., 2004), which is already implemented in MODFLOW-2000 (Harbaugh et al., 2000). It accounts for a stepwise (reach-by-reach) steady or unsteady flow simulation in the river branches, which have to be defined beforehand, and the three-dimensional groundwater flow.

The one-dimensional open-channel flow model HYDRET solves the St-Venant equations implicitly and has also been coupled with the three-dimensional MODFLOW using the RIVER package (explained in more detail in § 4.4.7). It is applied in the engineering praxis in Germany (Wald and Schiffler, 2002).

MIKE-11 (DHI, 2004) is a one-dimensional model developed at the Danish Hydraulic Institute (DHI), and is already integrated into MIKE-SHE using mass exchange between the surface and ground water systems (Matz et al., 2006). It is able to solve the full St.-Venant-equations, but can optionally also be run with the diffusive or kinematic wave approximation (Graham and Larsen, 2002). It has e.g. successfully been applied at the Iller river in southern Germany (Matz et al., 2006).

Coupling of two-dimensional channel flow with groundwater flow models

Coupling two-dimensional channel flow with groundwater flow is rare. One important approach is the RIVER Package (RIV) (McDonald and Harbaugh, 1988), which is already implemented in MODFLOW-2000 (Harbaugh et al., 2000). The RIVER Package accounts for a 2D representation of the surface water, but cannot account for unsteady flood wave propagation in the river. Furthermore, the extent of the wetted surface must be determined beforehand. Accounting for feedback mechanisms between surface water and groundwater due to interaction processes is therefore not possible. The technique of this approach has further been developed in order to be compatible with the target of this work, and is described in further detail in § 4.4.7.

Another approach is currently under development for its application at the Garonne river (France). There, a two-dimensional implementation of the St-Venant equations (kinematic-diffusive wave approach) for the surface flow coupled with the Boussinesq equation for the groundwater dynamics has been done (Peyrard et al., 2006). So far, no description was found by the author about the coupling implementation methods.

Fully integrated coupling combining the equations for surface water and groundwater flow

Three approaches have to be mentioned here: The first is a simultaneous coupling of a one-dimensional channel flow with a two-dimensional vertically averaged groundwater flow model, which considers lateral flow interactions beneath the channel bed and solves the two systems in a simultaneous manner within a single matrix (Gunduz and Aral, 2005). As it accounts only for one-dimensional channel flow, it is no longer considered here. The other two approaches are the MODHMS Model (HydroGeoLogic, URL; Werner et al. , 2006) and the the *HydroGeoSphere* model (Sudicky, URL), which are briefly described in the following paragraphs.

MODHMS

Panday and Huyakorn (2004) developed a fully integrated numerical scheme solving all relevant hydrological processes in a watershed in a single system of equations. As a drawback, a large number of parameters is used to fully describe all relevant processes and characteristics of the watershed. Furthermore, a huge number of cells is needed to fully describe the entire system. The numerical scheme was implemented in the Model MODHMS, which is now distributed and used by Hydrogeologic Software Inc., Herndon, VA (U.S.). Despite its many capabilities, rivers are represented in one dimension, only.

HydroGeoSphere

This model was developed at the University of Waterloo. It is based on the earlier *Integrated Hydrology Model* (InHM; e.g. VanderKwaak and Loague, 2001; Loague et al., 2005). *HydroGeoSphere* (Sudicky, URL) is a fully integrated watershed model. Also channel flow is represented in 2D, the whole system is solving all equations describing surface water processes in the watershed, 2D channel flow and groundwater flow at once. It is therefore an universal model accounting also for transport processes.

Remarks

Fully integrated physically based hydrological models like MODHMS as described by Panday and Huyakorn (2004) or *HydroGeoSphere* (Sudicky, URL) are applied at scales, which are distinct from the scale of the detailed river morphology as it can be found in alpine floodplains. They consider in each grid-cell nearly all hydrological and hydraulic processes completely. When applied under such circumstances and at small scales, they would require a huge amount of input data and an enormous computational effort to account for all processes in such a detail. Even if many of the parameters may be derived, there still remain a large number of unknown parameters. This leads on the one hand to calibration problems, on the other to over-parametrization of the problem.

The above methods are basically hydrological models accounting for all hydrological processes in the entire watershed, which are not relevant for the surface water – groundwater interaction. They are hence to be applied within the whole catchment. For this reason, they need also various distributed input data about catchment characteristics, which are usually not available while investigating studies about river–aquifer exchange. Although these models were principally capable to fully describe the surface water – groundwater interactions from the mathematical and physical point of view, there is an essential mismatch in the scales of interest and application scales.

Suitability for the Maggia valley

None of the modelling approaches described above is suitable for direct application to the conditions of the Maggia River and for the goals of the MaVal project. A numerically robust transient two-dimensional approach was needed for the surface water under conditions of complex river geomorphology with braided rivers, activating new channels and gradually increasing inundated areas during flood events, and a high streamflow variability, which required a detailed spatial resolution for the surface topography. Nevertheless, data availability was limited.

Despite all different possibilities of the modelling approaches described above, a new modelling strategy had to be found to fulfil all the requirements. Each of the model groups would be suitable for some aspects, but lack at least one essential component.

The coupled hydrological and groundwater models describe the surface water components and the groundwater components well, but are not able to account for a two-dimensional representation in a small scale of a few meters as it is required under conditions of alpine floodplains. This is possible only with a two-dimensional implementation of the shallow water equations.

The fully integrated approaches, which combine the equations for surface water and groundwater flow in one numerical solving system, are not suitable for the application in alpine floodplains such as in the Maggia valley because of the incompatibility of scales and data requirements, as discussed in the previous section.

The modelling strategy to be implemented in this work belongs to the group of coupling hydraulic open-channel flow models with groundwater models. With this kind of models, both components, i.e. the surface water and the groundwater, can be modelled at the appropriate scale, depending of the problem at hand. However, existing coupled models either account for two-dimensional channel flow, but only in steady-state, or use a transient mode, but account only for one dimensional flow. The decisive difference to the existing approaches and the innovation in this work is the coupling of a two-dimensional and simultaneously transient open-channel flow with a groundwater model.

3. STUDY SITE

In this section, the study site is described, especially focused on the specific characteristics, which identify the requirements for the modelling framework to be developed. Existing data, and data to be monitored or collected during field campaigns, which are relevant for the calibration and validation part of this work, are also described here.

The area of investigation of the MaVal project can be seen in a nested way: the “watershed” of the Maggia River, the “main valley”, and the “braided area”. They will now be defined and described.

3.1. Natural Environment

3.1.1. Location and Geology

The entire watershed is the Maggia Valley upstream of Ponte Brolla located in Canton Ticino in Switzerland. It is an alpine region at the southern rim of the Swiss Alps (Fig. 3.1). The watershed has a surface area of 592 km^2 and rises up from 216 m a.s.l. up to 3272 m a.s.l. at Mount Basòdino, which contains the only glacier in the catchment. The valleys are deeply incised with extreme slopes. Nearly the entire catchment lies in the Penninic basement, the rocks are predominantly gneiss and granite. Only in a small part of the upper catchment some marble can be found.

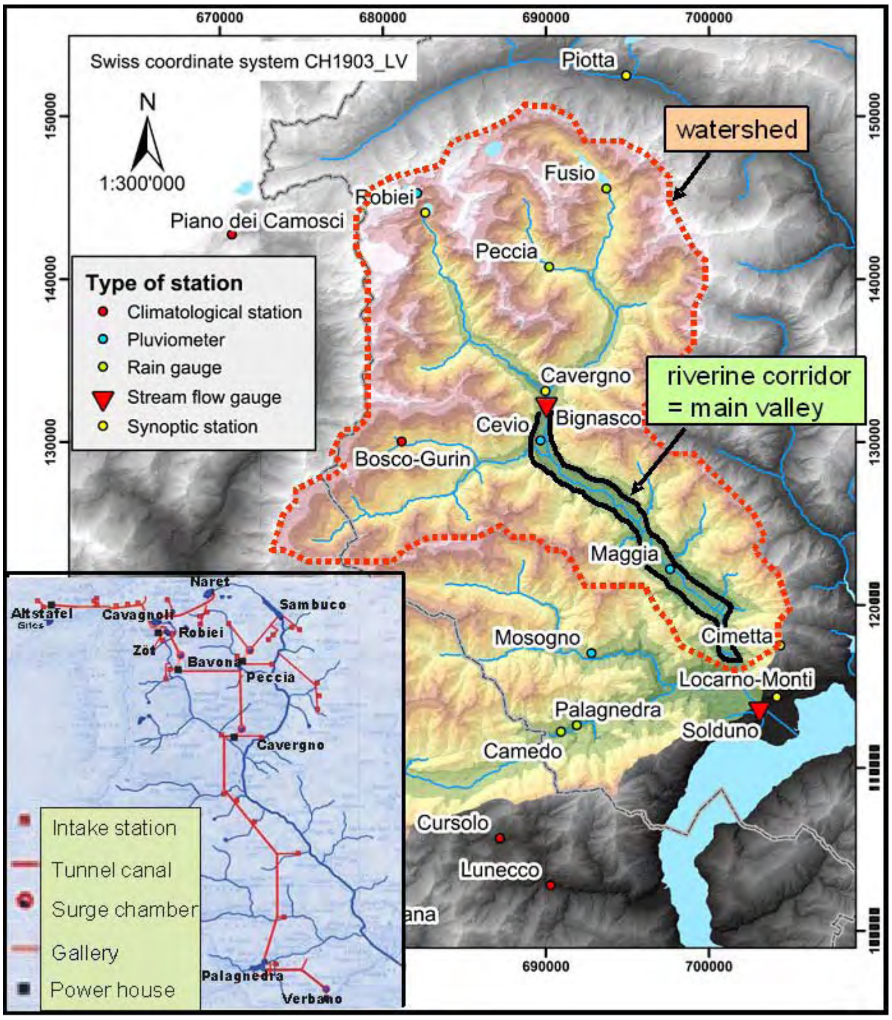


Fig. 3.1: *Study site of the MaVal project. The work focuses on the riverine corridor, which is also called the "main valley". On the lower left, a scheme of the hydropower installations is shown.*

3.1.2. Climate

The climate can be described as Mediterranean with orographic influences and, especially in the northern part of the catchment, some influence of the subpolar deep pressure zone. Sometimes, the weather is influenced by the North foehn. Precipitation is very high in autumn and fall, when the area is under the influence of the subpolar circulation. The combination of warm and humid air masses from the Mediterranean Sea with the deep pressure cyclones can create long-lasting heavy rainfall in this period. The summers are usually relatively dry under the subtropical influence. Winter precipitation is low. Because of the high elevation, most of the precipitation in the winter season fall as snow. Large spatial differences in precipitation amount are caused by orographic effects. Monthly precipitation and temperature are illustrated in Fig. 3.2.

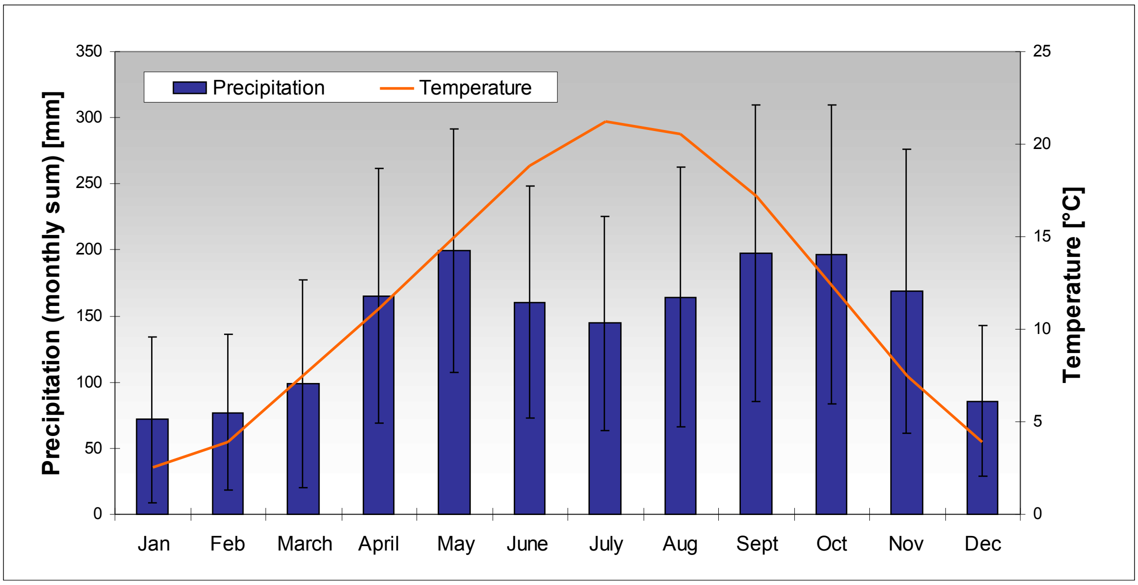


Fig. 3.2: Mean monthly precipitation and mean monthly temperature during the period 1929-2003 in Cevio, located in the main valley at 418 m a.s.l (from: Sturzenegger, 2005).

3.1.3. Streamflow and Fluvial Morphology

The natural streamflow regime of the Maggia River (streamflow gauge in Bignasco) can be described as pluvio-nival with a typical snowmelt peak in June. Streamflow is highly variable in time due to the very fast catchment response to precipitation. This is because of the very steep slopes, connected with thin soil layers and granite and gneiss as bedrock. Streamflow varies from less than 1 m^3/s under natural low flow conditions up to several hundreds of m^3/s during extreme flood events (in the braided area, peak discharge was estimated due to the contribution of the main tributary Rovana of approx. 1000 m^3). The flow duration curve is shown in Fig. 7.8.

There is one main tributary to the main valley, which is Rovana and drains approx. 21 % of the watershed, whilst the main river Maggia drains 52 %. The remaining 27 % are drained by the tributaries discharging directly from the side valley into the main valley of the Maggia River. They all enter the main valley from hanging valleys through a waterfall into an alluvial fan with a high permeability. Today, most of these are diverted and have therefore only very little water; but also other tributaries do not convey much water under dry conditions. This leads to the fact that – with the exception of just one tributary (Salto Maggia) – all other tributaries infiltrate completely into the fans before reaching the main river Maggia. Only under wet conditions, they convey enough water to reach the Maggia River.

Generally, the water of the Maggia River is extremely clear, i.e. turbidity is very low. Only the Rovana river undercuts a big landslide in Campo Valle Maggia, so that this tributary carries some suspended solids. Since under present conditions this tributary is diverted due to the hydropower operation most of the year, the Maggia River is still clear, except under flood conditions, when Rovana does deliver a considerable amount of sediment and suspended solids.

The entire floodplain in the main valley consists of coarse material: large gravel and sand with a pronounced armouring layer (see also § 3.3.4). The piezometric response to floods, the practical absence of turbidity as well as field observations suggest the absence of any colmation layer.

The main valley between Cavergno in the North and Ponte Brolla in the South is an incised valley with extremely steep slopes (Fig. 3.3) and a relatively well-defined floodplain. The valley is approx. 22 km long and 0.5 km wide. In its central part, for a length of 7-8 km (near the villages of Someo and Giumaglio), there is a braided area with alternating river channels and vegetated islands in between. Historical references report the wild character of the river Maggia with vast inundations and frequent relocations of the channels (see Cerini, 2003). From the natural point of view, this braided area is a very active floodplain with riverine vegetation undergoing its natural dynamics of erosion, seedling, succession and rejuvenation. The braided area is the part with the most gentle slope of approx. 0.5 % in the longitudinal profile, whilst in the upper and lower parts channels slopes up to 2 % dominate.

Fig. 3.3: A view of the main valley with its steep hillslopes.

3.1.4. Aquifer

The entire main valley was formed by a glacier. It is an overdeepend U-shaped valley with an aquifer depth up to 150 m containing coarse quaternary alluvial deposits (gravel and sand) at least in the approx. uppermost 30 metres, where information of borehole drillings is available. The aquifer flattens out at the outlet of the watershed in Ponte Brolla because of a geological barrier with in-situ granite and gneiss, which comes to the surface. Because of the high river bed conductivity, groundwater heads react strongly and rapidly to alterations in the water level in the river. Because of the high permeability of the alluvial material they also respond strongly to lateral recharge from the hillslopes. In the upper part of the main valley, infiltration from the river into the aquifer dominates, whilst in the lowest part, groundwater upwelling occurs due to the in-situ bedrock at the surface. In the central part, especially in the braided area, both conditions can occur alternating in space and time. Since the aquifer is relatively narrow and deep and the interaction with the river and the hillslopes is high, groundwater recharge due to precipitation and evapotranspiration from the soil and groundwater can be neglected.

3.2. Anthropogenic Influence

3.2.1. Hydropower Operation

History

First hydropower operation in the Maggia valley began in 1954. As a result, flow in the main valley was significantly reduced resulting in a dry reach of the river bed over a length of approx. 4 km between Visletto and Someo in the first post-dam period (1954-1974), when the minimum flow release was lowest. In connection to this, dramatic lowering of the groundwater table was observed endangering also the local water supply system (Fig. 3.4). The dramatic drop in the groundwater level, especially the intra-annual fluctuations in the immediate post-dam period, may have a major influence on the riparian vegetation. The drying up of the river bed created opposition by the fishermen and local population, which lead to the first environmental flow regulation in Switzerland. A stepwise increase of the environmental flow was enforced by the Cantonal authorities as it is shown in Tab. 3.1. At present, it is again the subject of discussion due to changes in legislation.

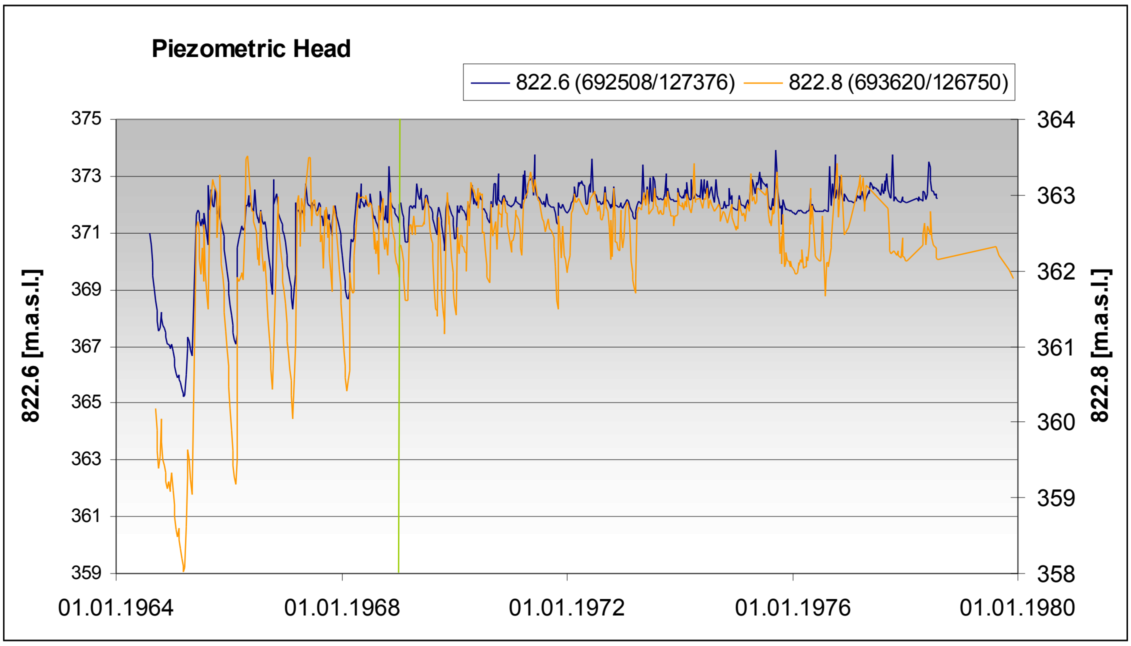


Fig. 3.4: *Groundwater heads in two piezometers near Someo (coordinates are in brackets) between 1964 and 1979 (from: Sturzenegger, 2005).*
Essential drawdown in the first years is due to the initial minimum flow in the first post-dam period. First environmental flow requirements were imposed in 1969 (green line), stricter ones in 1973 and 1982 (see Tab. 3.1), which lead to a recovery in the groundwater table.

Tab. 3.1: *Evolution of the Environmental flow (EF) enforced in the Maggia valley (from: Foglia, 2006).*

Date of implementation	**EFs established**	**Period**
January 2nd, 1969	750 l/s	all year
January 2nd, 1971	700 l/s 1200 l/s	November 1st – March 31st April 1st – October 31st
January 4th, 1973	1200 l/s	all year
April 10th, 1982	1200 l/s 1800 l/s	October 1st – June 14th June 15th – September 30th

Hydropower system and operation

Today, the hydropower system consists of eight artificial reservoirs and one natural lake with many connections between them. Some pumping stations are installed to pump the water into the upstream reservoirs during time periods of low energy consumption or low energy prices. The system is capable to store the seasonal water supply in the reservoirs and to operate the system superimposed on a daily or weekly basis due to pumping facilities. The hydropower system is shown in Fig. 3.1. The most important structures and related basic operating rules affecting the main valley are the following: An open-channel gallery from Cavergno (upstream of Bignasco) diverts the water into the reservoir of Palagnedra outside the watershed. From there it is later used to produce electricity in a power station at Verbano at Lago Maggiore. The large tributary Rovana and most of the smaller ones on the right side of the Maggia valley downstream of Bignasco, are also connected to this main gallery. If the capacity allows, the water from the tributaries is captured completely by the intake stations, which are located relatively close to the main valley, so no water is left in the tributaries. Only due to exfiltration from the adjacent hillslopes, there is little discharge when entering the main valley. From the upper part of the catchment, i.e. upstream of Bignasco, all water is diverted through this main gallery except what is required to be left as environmental flow in Bignasco concerning the regulation described above, or what exceeds the the conveying capacity of the gallery. The excess water is released into the Maggia River or can still remain in the tributaries. As a result, there are nowadays long periods with extreme and constant low flows (which were lowest in the first post-dam period between 1954 and 1974). Typical annual hydrographs (Fig. 3.5) show the alterations in the flow regime due to the hydropower operation (see additionally Fig. 1.2). Big floods are however hardly effected by the hydropower system (Fig. 3.6), but flows in the medium range, and especially its variations are significantly reduced. This is also evident from to the flow duration curves in Fig. 7.8.

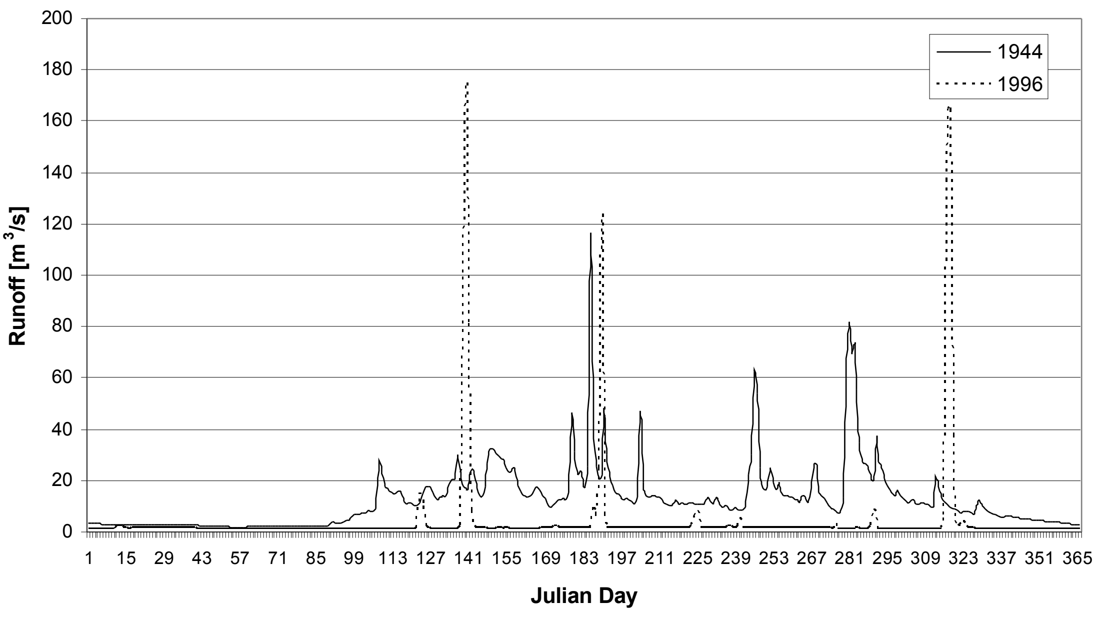


Fig. 3.5: Hydrographs of daily streamflow in Bignasco at the upper boundary of the main valley for two selected years (1944 and 1996), i.e. before and after hydropower operation started.

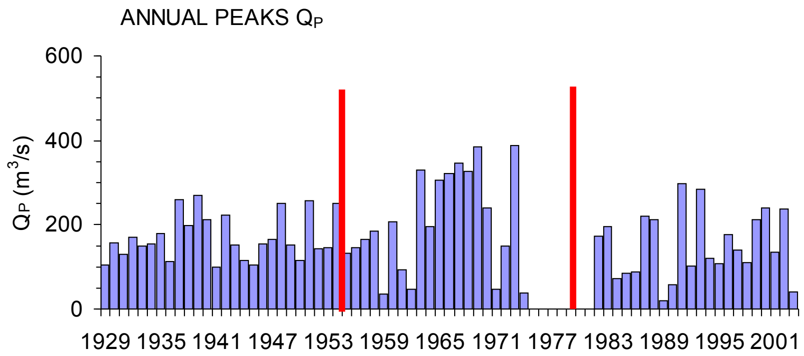


Fig. 3.6: Evolution of annual peak flow (daily values) in Bignasco for each year in the pre-dam (1929-1953), post-dam (1954-1977) and recent (1980-2003) periods. Red lines separate the three periods. There is no trend visible after the start of the hydropower operation.

3.2.2. Embankments and Deposits in the Main valley

Some embankments and anthropogenic sedimentary deposits in the mid and second half of the 19th century have partly narrowed the river bed, but there are still large areas, especially in the braided area between Riveo, Someo and Giumaglio, where the river morphology is still relatively pristine. This is the area of interest for the investigation of the vegetation development within the MaVal project. The evolution of the area of the unspoilt river system is illustrated in Fig. 3.7.

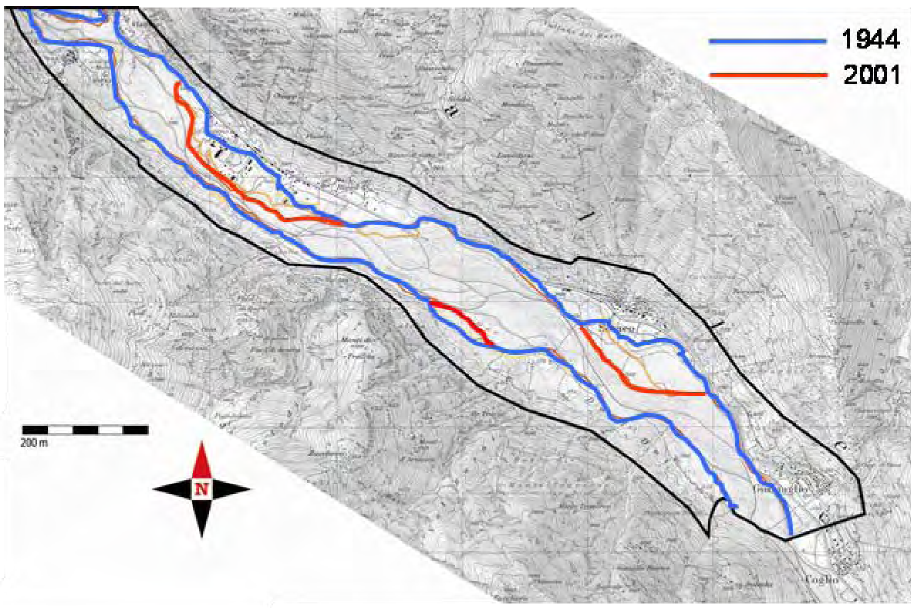


Fig. 3.7: Evolution of the area of the unspoilt river system in the Maggia valley between Visletto (confluence from Rovana into the Maggia River) and Giumaglio. Blue: floodplain extent 1944, red: artificial embankments 2001 (source: Cerini, 2003).

3.2.3. Observation of Alterations in the Riverine Vegetation

The riverine ecosystem in terms of riparian vegetation is under a strong alteration, as a series of aerial photographs taken irregularly from 1933 up to now clearly shows (see Fig. 1.5). The vegetation develops toward a more mature vegetation stage, whereas the distribution of the open gravel bars and herbaceous (pioneer) vegetation declines. At least three reasons have to be considered: variation in climate, riparian forest management, and anthropogenic influence into the water cycle. There are no indications of strong climatic variations; furthermore, there is no hint in the available literature or from the local population for any forest management in the braided area of the riverine ecosystem such as coppice management (see Rampazzi et al., 1993; Cerini, 2003), except of some afforestation in the area close to the village of Someo, where esp. douglas firs were afforested behind an artificial embankment erected after 1944. Since this part was not considered in the investigation of the vegetation development due to its human influence, forest management can be so far excluded to explain the changes in the distribution of vegetation in the braided area. Accordingly, the hydropower operation with all its consequences is considered to be the most important factor for these alterations in the vegetation.

3.2.4. Legislative Implications

In 1992, the new federal order for the protection of riverine landscapes (Verordnung vom 28. Oktober 1992 über den Schutz der Auengebiete von nationaler Bedeutung (Auenverordnung), legislation number 451.31 (28. Oktober)[1]) was enforced. Special areas for riverine landscape protection were defined in a specific legislative inventory (entry of Maggia in 1992) (Bundesinventar der Auengebiete von nationaler Bedeutung (Aueninventar)[2]). The braided area of the Maggia River is one of the largest in Switzerland. In connection with the Federal Water Protection Act (Bundesgesetz vom 24. Januar 1991 über den Schutz der Gewässer (Gewässerschutzgesetz, GschG; legislation number: 814.20)[3], stricter environmental flow requirements have to be released according to the necessity for ecological integrity of the protected riverine landscapes. The question about an optimal water management accounting for environmental flow is risen, which optimizes the ecological needs for the maintenance of the riverine vegetation in the protected braided area and the economics of the electricity production. For this reason, the understanding between hydrological conditions and the vegetation development as well as their mutual interaction with the sediments in the riverine corridor is essential. This is a strong motivation for the MaVal project.

---

[1] in Italian: Ordinanza del 28 ottobre 1992 concernente la protezione delle zone golenali d'importanza nazionale (Ordinanza sulle zone golenali)

[2] in Italian: L'inventario federale delle zone golenali d'importanza nazionale (inventario delle zone golenali)

[3] in Italian: Legge federale del 24 gennaio 1991 sulla protezione delle acque (LPAc)

3.3. Data Acquisition and Monitoring

3.3.1. Previous Studies in the Maggia Valley

First studies in the context of the target of Environmental Flow Requirements were carried out by the Federal and Cantonal Authorities in the 1960s with the intention to estimate the flow required for a continuous water course in the Maggia River. The first report by the Federal Office of Water Management (1966) (cited in: Foglia, 2006) contains a short introduction and some information about the hydrological situation related to the hydropower operation, together with a first assessment of its consequence for the natural river system. The second study (Beatrizotti, 1973) was carried out in the context of the development of a first groundwater model. The reason was that 20 years after the begin of the hydropower operation the groundwater table had dropped up to 10 m in the upper part of the valley, thus causing problems for some private wells and for irrigation. Despite obtaining some useful results, the models suffered from insufficient data on bedrock elevation and hydraulic conductivity of the aquifer. As a consequence, a ten-year monitoring network including 28 piezometers along the main valley was established which weekly recording. Additionally, some geophysical investigations for the evaluation of the aquifer bed level were carried out. These studies were published as internal reports of the former Geological Cantonal institute, now SUSPI-IST, Trevano, Switzerland (Foglia, 2006).

Additionally, some groundwater studies including the installation of a locally dense network of piezometers and pumping tests were carried out in the floodplain area between Gordevio and the Maggia River. This was a preliminary study for a possible installation of wells for water supply, which have finally never been constructed (internal reports at SUPSI-IST).

3.3.2. Streamflow Data

Continuous measurements

Maggia River

The existing streamflow gauging network in the main valley was relatively poor before the project started. A long time series of runoff recording exists only at the top of the study reach in Bignasco, a streamflow gauging station run by the FOEN (*Federal Office of the Environment*, Berne) since 1929 with a short interruption after the big flood in 1978. Close to the outlet in Avegno, some runoff data are available from a hydropower company, but only up to streamflows of 10 m^3/s. One stage level device in Lodano (in the middle of the main valley) and in Ponte Brolla (at the outlet of the main valley) were no longer in operation since 1921 resp. 1965.

As only the streamflow stations in Bignasco and Avegno were available, it was necessary to install further gauging stations for the monitoring of the runoff behaviour, especially under consideration of infiltration and exfiltration of the Maggia River along the longitudinal profile. The topography and dynamics of the river bed limited the possible locations. Finally, three new gauging stations were installed (from upstream to downstream): in Cevio, Riveo and Lodano.

The station in Cevio was installed for the following reason: When hydropower operation started, the Maggia River lost all its water during its run from Bignasco to Cevio, which is the uppermost longitudinal section of the main valley. In this part, the river is channelized. This station allows insight into the strong infiltration behaviour of the river in this section. This station was intended to be used only under low flow conditions, since water measurements under flood conditions required for the complete rating curve, are not possible at this cross-section. Three measurement under low flow conditions (salt dilution method and propeller) have been carried out to build the rating curve under additional hydraulic considerations.

The station in Riveo is located next to the quarry site, which is an anthropogenic deposit within the natural river bed. It is situated just at the upper boundary of the braided area, where the whole project is focused on. Here, a pressure transducer was used to measure the water level recording in time intervals of 10 min. The station is meant to be used only under low and moderate flow conditions, because it is not possible to perform runoff measurements under high flow conditions in order to build the rating curve. It is also not feasible to compute the runoff with hydraulic methods because of some backwater effects. The runoff under high flow conditions can be estimated using the runoff in Bignasco and the main tributary Rovana. Under flood conditions, these two rivers contribute by far most to the runoff in Riveo so that the relative error is assumed to be small. Under low flow conditions, this is different: Rovana does not contribute anything due to water diversion by the hydropower company. Furthermore, the strong river-aquifer interaction made the estimation of the low flow at the location of the station impossible. Two measurements with the propeller have been carried out in order to build the rating curve, again under additional hydraulic considerations.

Lodano is situated almost in the centre of the main valley. A radar measuring the stage level was installed directly under the bridge. Principally, this station is capable to measure low and high flow. The rating curve has been generated using flow measurements with the propeller with runoff ranging from 1.4 m^3/s to 62.4 m^3/s. For higher runoff, the rating curve was extrapolated using rough estimates for extrapolation up to 570 m^3/s, reconstructed from runoff measurement Rovana and Maggia in Bignasco, together with some correlation analysis between these two stations.

Tributaries

Data from two tributaries were also available: The spillover of Rovana river at the intake station, just 2 km before Rovana enters the main Maggia valley. Within this distance, a little amount of water fed by the hillslope is conveyed there. The time resolution is daily, up to 1999, from then on mostly 5 min.

A cantonal gauging station is located at Salto Maggia, just below a small reservoir, in order to monitor the environmental flow at this point, where the water is diverted into the hydropower station in Giumaglio. Salto Maggia enters the Maggia River at the village of Maggia with flow conditions continuous in time; under normal, i.e. low flow conditions, a big portion of this water infiltrates into the aquifer before reaching the Maggia River. Under flood conditions the assumption can be made that the largest portion discharges into the Maggia River.

Episodic or Single Measurements

Discharge Measurements in the Maggia River

In connection with the monitoring of the evolution of the river bed, a few discharge measurements were carried out by SUPSI-IST (Cantone Ticino) between 1971 and 1976[4] (Foglia, 2006). Additionally, these measurements were repeated in 1996[5] in order to prove the effectiveness of the recent environmental flow requirements (Fig. 3.8). During an extensive field campaign in June 2005 under low flow conditions (1.2 m^3/s in Bignasco), runoff in the Maggia River was measured with the propeller at 10 locations along the longitudinal profile of the entire valley for the validation of the surface water model under low flow conditions in different sections of the river bed (Fig. 3.9).

The spot measurements both in the post-dam period (1974-1976) and in the present period (1996 and 2005) showed that the variation in streamflow along the longitudinal profile is high (2005: starting from 1.3 m^3/s in Bignasco to a minimum of 0.7 m^3/s in the Riveo at the begin of the braided area and increasing again to 3.1 m^3/s in Avegno at the outlet of the valley).

[4] The exact dates are: 20.12.1973, 24.01.1974, 08.11.1974, 11.12.1974, 12.12.1974, 19.12.1974, 30.01.1976, 05.03.1976, 31.03.1976, 25.06.1976, 02.07.1976.

[5] Date: 13.06.1996.

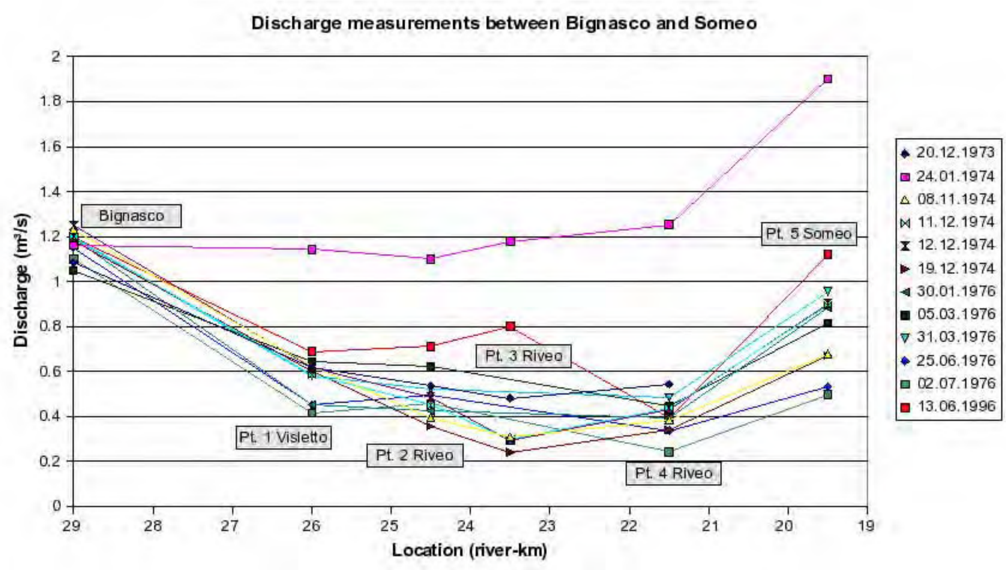


Fig. 3.8: *Discharge measurements under low flow conditions in the upper part of the main valley and the corresponding environmental flows imposed in Bignasco (source: Foglia, 2006). Locations of discharge measurements are ordered from upstream to downstream.*

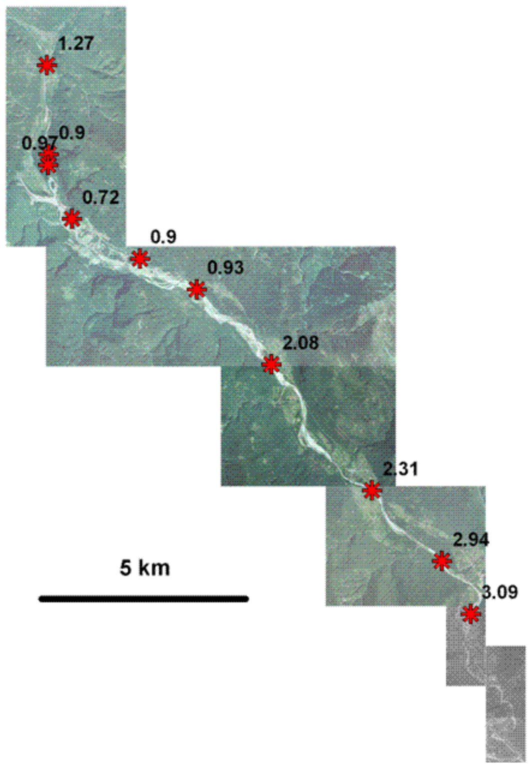


Fig. 3.9: *Runoff in the Maggia River along the longitudinal profile under low flow conditions (June, 2005) (Flow direction is from the North to the South). The uppermost measurement in Bignasco with 1.27 is from the federal gauging station, the others are own field measurements.*

Discharge measurements in the tributaries

Over the time span of the project, the discharge of most of the tributaries was measured under low flow conditions for two reasons: 1. As they infiltrate completely into the aquifer before reaching the main river, it is a direct measurement of the amount of infiltrated water, and while measuring the distance until the river bed is empty, the infiltration capacity can be estimated. 2. These measurements can be used for the estimation of the hillslope fluxes in the following way: Some of the tributaries have an intake station further upstream, where the entire amount of water is captured. The discharge must therefore stem from the hillslope contribution of the side valley between the intake station and the point of entering the main valley, where the measurements took place. Considering similar geology, soils, vegetation, and topography, these measurements can serve as a surrogate for measurements of the hillslope fluxes into the aquifer of the main valley, which had finally to be modelled by the watershed model and calibrated together with the groundwater model being therefore important for the modelling framework.[6]

Water lines and flood levels in the Maggia River

Available data for the validation of the surface water model under flood conditions (see § 6.3.4) are sparse, because only the runoff stations in Bignasco (inlet) and Lodano (in the middle of the valley) are capable to measure these kind of events. Furthermore, the rating curve in Lodano had to be extrapolated for high flood events. Besides this, spatial information is needed.

For this reason, two field measurement campaigns were carried out during this project. Just after the floods of May 3rd, 2002, and November 16th, 2002, indicators of the highest flood water stage were searched for in the field and measured by GPS. Because of the large area, only selected reaches could be monitored.

Additionally, aerial photographs of 1978 taken just five days after a large flood of August 7th, 1978, were evaluated. Three types of areas could be discriminated: 1. certainly flooded, 2. probably flooded, 3. certainly not flooded. This spatial information was gained for the braided area around Riveo/Someo.

[6] These measurements of the discharge of the tributaries are not very precise for methodological reasons. Upstream of the cascades, the creeks are often not or only hardly accessible. Just below the cascades when entering the fan in the main valley, big rocks with a riffle-pool system are present, before reaching a irregular channel bed. No regular cross-section for using the propeller or similar techniques could be found. Therefore, the salt dilution method had to be used, but also here, the pools were creating problems as retention reservoirs, and – because of several flow paths – the distance between injection and measuring point had to be sometimes relatively large, so that a substantial part of the discharge has already infiltrated during this reach. Despite their inaccuracy, these data provide a useful information of the order of magnitude of the processes where the tributaries are involved.

3.3.3. Surface Topography

Different data sources were available with respect to surface topography: digital terrain models, river cross-section profiles and geodetic measurements from own field campaigns. All of them were used to build a common combined digital terrain model to be used for the surface water model.

Digital terrain models

The *Federal Office of Topography swisstopo* (Wabern, Switzerland) provides a digital terrain model covering entire Switzerland based on the topographic map 1:25 000. The final product consists of a regular grid with a spacing of 25 m. This DTM was available and used for the watershed model TOPKAPI, the surface topography (upper bound of upper layer) for MODFLOW, and for the surface water model 2dMb at those locations, where no other detailed topography data were available.

Additionally, some digital terrain model data for particular and restricted areas in the main valley were provided by the Canton Ticino (SUPSI-IST, *Scuola Universitaria Professionale della Svizzera Italiana; Istituto di Scienze della Terra*, Trevano, Ticino). They differ substantially in their spatial resolution.

Additionally to the already existing data, a new detailed DTM was generated by *GEOFOTO* (Bellinzona, Switzerland). A flight was undertaken in summer 2002 accounting for the central part between Riveo and Maggia. There were several reasons for it: First, the hydrodynamic model needs precise data. High resolution data had been available only for restricted parts of the floodplain. Second, because of the active processes in the river morphology, recent data had to be used to compare with recent measurements. For this purpose, stereoscopic photographs were evaluated in order to build a digital terrain model. The final product consisted of scattered data along contour lines with a vertical distance of 0.5 m. If the horizontal distance was large, or in case of singular points, additional scatter data were set. Using these data, the standard deviation is expected to be in the order of 1 m in the horizontal direction and 30–40 cm in the vertical direction (Foglia, 2006).

Cross-section data

In the past, Canton Ticino assigned different geodetic companies to measure the topography of several cross-sections of the Maggia River. Data were available from the years 1979/80, 1988 and 1996 for different parts of the river and different distance between them, and in the order of half a kilometre.[7] All these data were provided by the Canton Ticino (SUPSI-IST).

These data were used to reconstruct a DTM along the river bed of the Maggia River, where no other data were available. The problem however exist that the river bed changed significantly in many parts of the valley, so that they do not necessarily match the recent surface topography.

These river bed DTMs were combined with the other DTMs mentioned in the previous section to build one single DTM for the entire main valley. As breaklines like the thalweg, were not yet included, they were introduced in a mixed manual-automatic procedure using GIS. Then, the scattered data could be interpolated onto a rectangular grid for the use in the hydrodynamic model. For the groundwater model, only the already existing DTM from swisstopo with a common spatial resolution of 25 m was used for representing the ground surface, which was basically used for initial conditions and for the calculation of the depth-to-groundwater.

River topography from field measurements

In June, 2005, an extensive geodetic measuring campaign for the river bed was carried out for the following reasons: 1. checking the existing DTM and its accuracy against conventional methods, 2. measuring flow depth and width for the validation of the surface water model, 3. measuring lateral and vertical changes in the river cross-section compared to the time when DTM data where collected, since after the flood in 2002, which occurred already at the time of the project, some changes could be registered even by visual observation. Nine longitudinal profiles have been measured for this purpose (Fig. 3.10).

3.3.4. Remote Sensing, Vegetation and Sediment Data

Remote sensing

Aerial photography

A set of nine aerial photographs dated from 1933 to 2001 was provided by the *Federal Office of Topography swisstopo* (Wabern and Dübendorf; Switzerland)[8]. All of them are in black-and-white except the pictures from 1999, which are in colour. The covered area, flight height, resolution etc. differ between the pictures. For most years, the

[7] Data were provided by *Istituto Scienze della Terra*, *SUPSI-IST*, Canobbio, Switzerland.
Additional cross-section data are available data based on some geodetic survey carried out by the personnel of the same cantonal geological and hydrological office (SUPSI-IST) for the period between 1971 and 1976, for the purpose of monitoring the geomorphological dynamics of the river bed, in some years related with some discharge measurements with the propeller.

[8] The dates of the flights are: 27.06.1933, 10.08.1944; 28.08.1962: 27.06.1977; 12.08.1978; 19.07.1989; 26.06.1995; 26.07.1999; 06.09.2001.

overlapping pictures for the interesting areas are available so that a stereoscopic evaluation would be possible.

While concentrating on a reach of 2.8 km including the braided area, all nine aerial photographs from the years 1933, 1944, 1962, 1977, 1978, 1989,1995, 1999, and 2001 were evaluated by Sturzenegger (2005) (see also Fig. 1.5). The picture from 1978 is special in the sense that is is taken just five days after the biggest flood ever occurred since the beginning of the stream flow measurements in 1929.

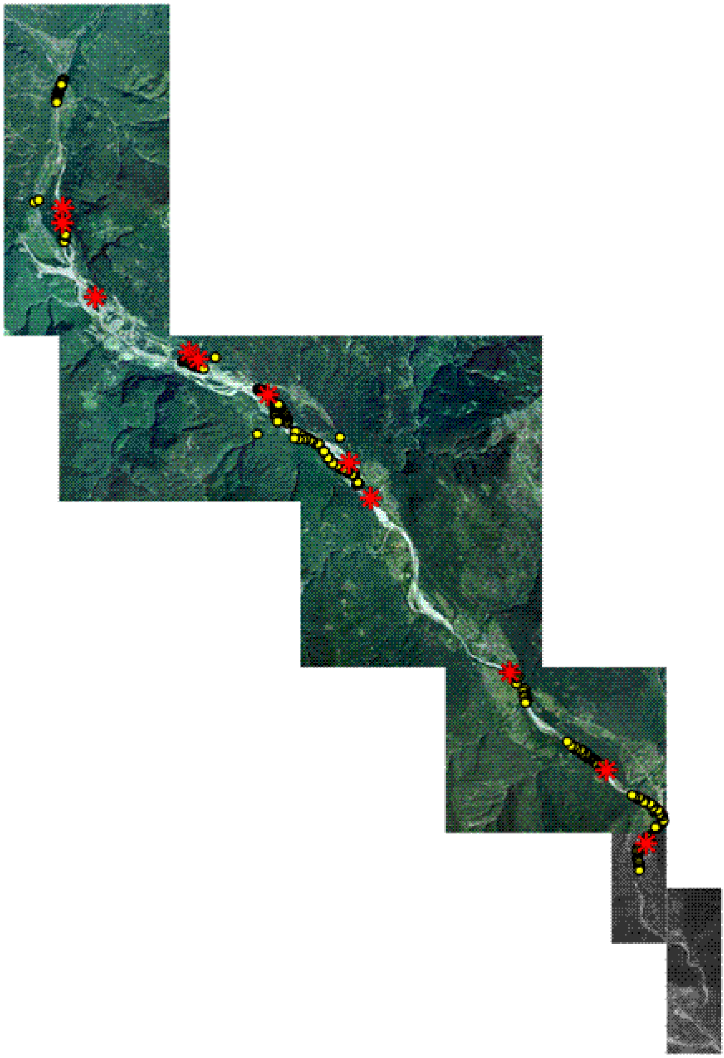


Fig. 3.10: Overview of the field campaign in June, 2005: geodetic measurements of longitudinal profiles of the river bed (yellow dots) and locations of runoff measurements (red stars), always under low flow conditions (1.2 m^3/s in Bignasco). The flow direction of the river is from North to South.

In addition to these photographs from *swisstopo*, a coloured orthophoto set covering almost the entire main valley from the summer 2000 was provided by *Cantone Ticino* (*Sezione Bonifiche e Catasto*). For the remaining area (municipalities of Gordevio and Avegno at the lower end of the valley), a composite of older black-and-white orthophotos could be used (provided by the same Cantonal office). These pictures were used for identifying land surface classes being related to surface roughness values for the surface water model. Additionally, an infrared-composite was available and used for vegetation analysis by Favre (2004).

Satellite images

Satellite images can potentially be used for the determination of the extent of the inundated area during flood events. During an extensive search however, no suitable data could be found. As high flood events are caused by intense rainfall and the response to it is very fast in this watershed, the likelihood of cloud cover during the flood in the main valley is very high. Radar images are another potential source of information. In this case however, the deep incision of the valley is a major problem because of the shading effect. It is detectable for a radar sensor only under a very special direction of the orbit. After all, is must be concluded that no information from satellite was available to use as data source for flood events.

Vegetation data

The vegetation development in the braided area has been observed and investigated by several authors (e.g. an inventory of plants by Rampazzi et al. (1993). First studies about the connection to flood events were studied by Bayard and Schweingruber (1991). A recent detailed plant inventory was built by Stoessel (2006) in order to relate the vegetation classes delineated from aerial photography (Sturzenegger, 2005) to specific or dominating plant species. The purpose was – among others – to gather information about the habitats and the ecological needs of the plants existing on these habitats.

Sediment Data

Direct measurements of sediment transport relevant for the processes in the river bed of the main valley do not exist. Only the comparison between geodetic cross-section data for different years allow for some estimation of erosion and sedimentation reaches as illustrated in Fig. 3.11. A comparison of old and new topographic maps as well as the aerial photographs shows remarkable horizontal changes in the river bed. This fact was also verified by own geodetic measurements (see § 6.3.4, esp. Fig. 6.12).[9]

Sediment supply in the Maggia River stems mainly from the tributary Rovana, which undercuts a landslide below Campo (Valle Maggia) and provides therefore loose

[9] If sediment transport rates and a mobile river bed were to be investigated in future studies, different DTMs could be generated from the aerial photographs, however requiring a substantial effort.

material. It is easily visible at the confluence of the Rovana river into the Maggia River in Visletto that under flood conditions, the turbidity between these two rivers is extremely different. The main river Maggia has still a relatively low turbidity even during flood events.[10]

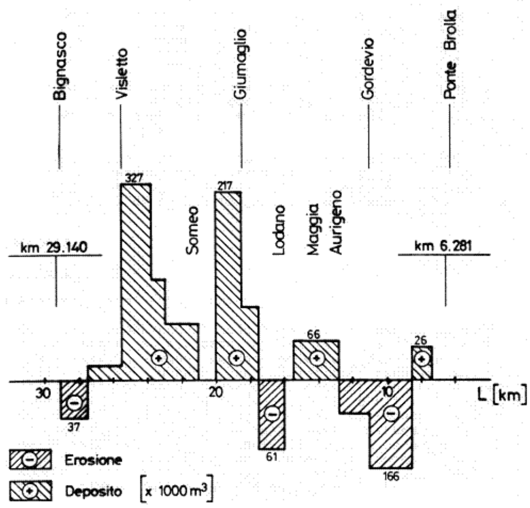


Fig. 3.11: Deposition and erosion in the Maggia River along the longitudinal profile between the years 1979 and 1989 (from: Anastasi, 1990).

For the determination of the roughness in the river bed and the adjacent gravel bars, grain size distributions were measured. Anastasi (1990) report a range between 0.12 and and 0.22 m for d_m of the upper layer[11] (d_m is the median diameter) (Tab. 3.2). The smallest grain size was found in the braided area between Visletto and Lodano, which is the braided area, but has also the least slopes and is the area downstream of the mouth of Rovana into the Maggia river. In the area of main interest, i.e. the braided area, the grain size does not much differ.

Tab. 3.2: Evolution of median grain size of the surface (armouring) layer of the Maggia River in the main valley along the longitudinal profile from upstream to downstream (from: Anastasi, 1990). As the method of evaluation of the measured data could not be detected, they were not used for the determination of the bed roughness. They show however the spatial variability along the longitudinal profile in the river bed.

Location	L [km][12]	d_m [m]
Bignasco	28.63	0.17
Cevio	26.30	0.14
Visletto (confluence with Rovana)	25.12	0.13
Giumaglio	19.05	0.12
Lodano	16.50	0.13
Aurigeno	12.75	0.17
Gordevio	10.15	0.21
Avegno	6.93	0.22

[10] For future investigations focusing more on sediment related issues, sedimentation data of the reservoirs of the hydropower company OFIMA would be available, which would allow for the estimation of the sediment supply in the upper part of the catchment, which does no longer feed the main valley.

[11] The method to obtain these values is based on line samples, but their evaluation is not further described in this report. For this reason, they are not directly used, but own measurements and evaluation and those done by Sturzenegger (2005). They provide however a useful information about the relative spatial variability in the longitudinal direction.

[12] L is the distance along the thalweg of the Maggia river from the mouth into Lake Maggiore.

Measurements by Sturzenegger (2005) and own investigations were carried out also using the method of line-sampling ("Linienproben“) (Fehr, 1987a; Fehr, 1987b)[13] for different locations (see Fig. 3.12) along the longitudinal profile both in the river bed and on the adjacent gravel bars. For this, the evaluation had to be carried out in three steps: 1. the transformation from line sampled data which account only for number to a volumetric (mass) consideration, 2. the consideration that the fine material was neglected in the field survey, and 3. the adaption of a *Fuller* distribution for the estimation of the part of the fine material in the volumetric sample. The results are representative for the lower layer. It is assumed that the d_{90} of the lower layer corresponds to the median grain size d_m of the upper layer which is the relevant parameter for the determination of the roughness parameter k_s (equivalent sand roughness (§ 4.5.4, see Bezzola, 2003)). The results are shown in Fig. 3.13 and Tab. 3.3. The variation between the different locations is not very high, but there is a small tendency toward downstream fining. With the dominance of relatively large grain sizes in the upper layer, the choice of the *Fuller* distribution did not largely effect the result.

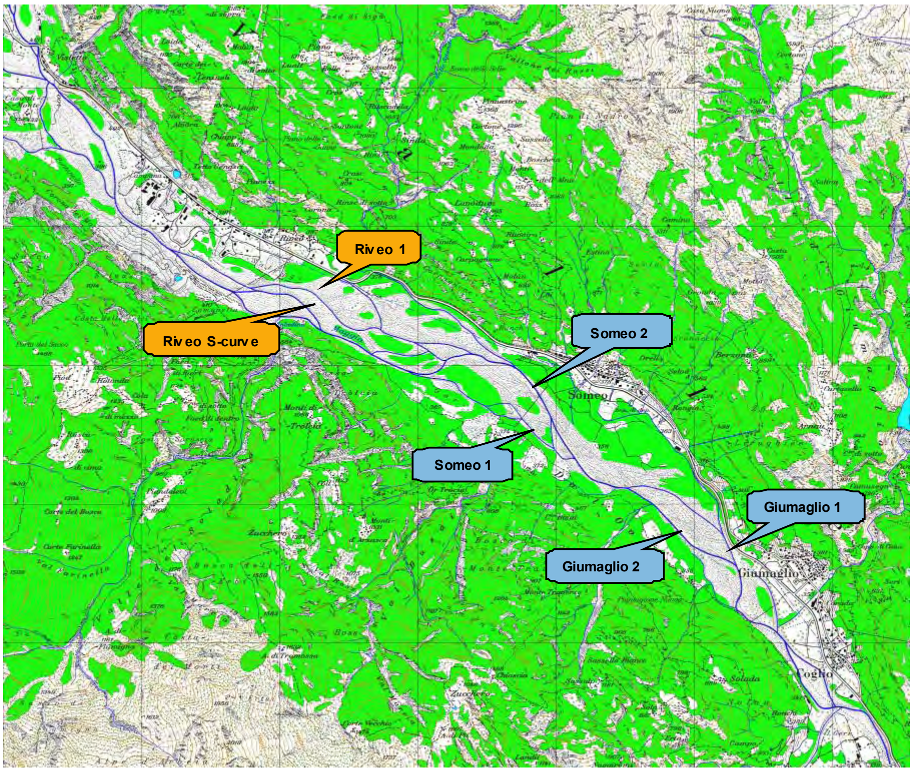


Fig. 3.12: Location of line samples for the determination of grain size distribution of the river bed material (from: Sturzenegger, 2005).

[13] With this method, all pebbles and gravels > 1 cm under a straight line, which are visible from the top, are counted and used for generating a relative frequency distribution.

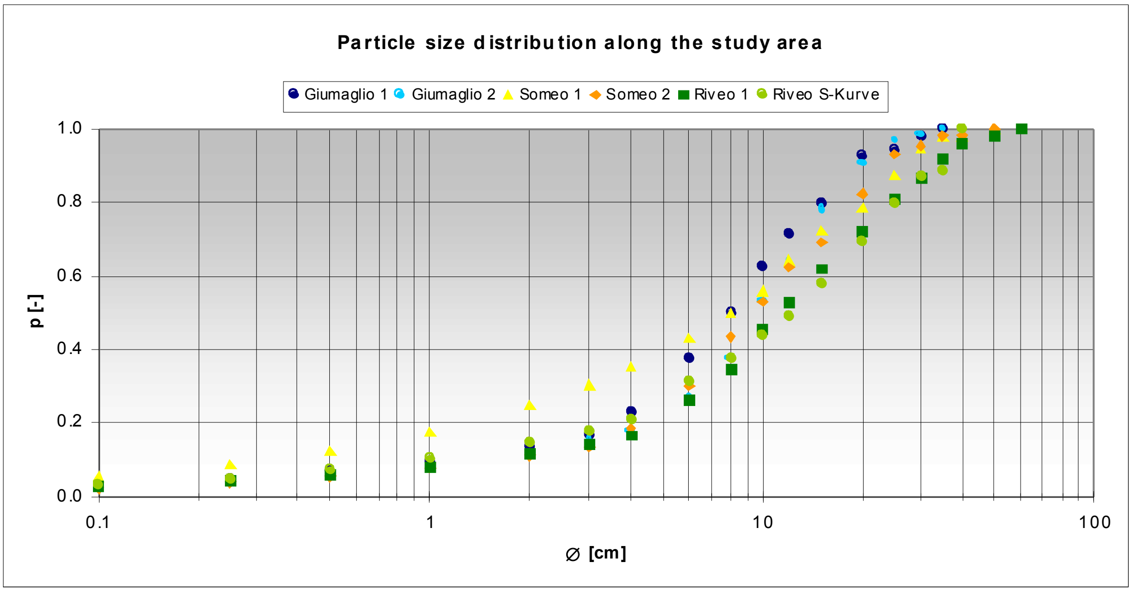


Fig. 3.13: Volumetric grain size distribution of the sediment in the lower layer of the river bed at different locations (from: Sturzenegger, 2005; field measurements by Sturzenegger and Ruf).

Tab. 3.3: Median grain size distribution d_m at different locations (see Fig. 3.12) of non-vegetated area in the main Maggia valley incl. their standard deviation (from: Sturzenegger, 2005; field measurements by Sturzenegger and Ruf).

	Riveo	Someo	Giumaglio
d_m (armouring layer) [m][14]	0.147 ± 0.002	0.114 ± 0.005	0.100 ± 0.007

As mentioned already, a pronounced armouring layer can be found in the river corridor of the main valley. In order to check this assumption, a qualitative inspection was carried out at several locations on the gravel bars next to the main channel[15]. It was found that the armouring layer consists of one or two layers of big gravel without any sand, the lower layer is a mixture of sand and a high number of embedded gravels, which have approx. the same size as those of the armouring layer. Finer material (silt or clay) was not found indicating a relatively high vertical permeability of the river bed.

3.3.5. Groundwater and Aquifer Data

Piezometers and groundwater observation

As reported in § 3.3.1, first piezometers were installed in the valley in relation to the hydropower operation. Furthermore, a dense collection of piezometers was installed in

[14] The bandwidth is given by the standard deviation of the measurements.

[15] For practical reasons, it was not possible to do this investigation directly in the river. The locations were selected as close to the river channel as possible (under low flow conditions), but in such a way that the digging holes were still dry.

the floodplain of Gordevio for preliminary analysis about possible water exploitation for drinking water supply. During the large flood in 1978, many of these piezometers were destroyed or covered by sediment. Even knowing the coordinates and using metal detectors, it was not possible to find them any more. For this reason, and to be more project target specific, seven new piezometers were installed in connection with the MaVal project, whereas five are in a bundle in the floodplain next to the braided area, the other two directly next to the river course in Cevio in the North and Gordevio in the South.

In the 1960s and 1970s, piezometric heads were recorded weekly. Since the start of the MaVal project, they were equipped with automatic data loggers recording hourly values of piezometric heads and temperature (Tab. 3.4). The location of the old and new piezometers in the main valley can be seen in Fig. 3.14. For all of them, the borehole profile is available. More details about the groundwater monitoring can be found in Foglia (2006).

Tab. 3.4: Historical weekly groundwater measurements (partly from Foglia, 2006).

Period	**Locations**	**Measure**	**recording interval**
23.01.57 - recently	Well in Cevio (OFIMA)	groundwater head	weekly
1965-1974	in 8 piezometers between Bignasco and Someo	groundwater head and temperature	weekly
1971-1978	in 28 piezometers between Bignasco and Aurigeno	groundwater head	weekly
2002–2007	in approx. 28 piezometers between Bignasco and Aurigeno	groundwater head and temperature	hourly

Stratigraphy

Information about the shape and the stratigraphy of the aquifer was collected from three sources: Analysis of core samples, geoelectric and gravimetric measurements.

Stratigraphic data from the aquifer were available by *core samples* gathered while drilling the piezometers and wells in the main valley, mostly from the 1970s. The depth is between 10 and 50 m, on average around 30 m. New piezometers from this decade have a depth of less than 10 m. On summary, there is a alternation of sandy and gravel layers, as it is typical for old fluvial sediments in mountainous regions. All information can be found at the Cantonal Geological Office, *IST-SUPSI* (Trevano, Switzerland).

Geoelectric measurements have been performed in the upper and central part of the main valley. Despite some inaccuracy of their results due to problems with the instruments as reported by Pfamatter et Zanetta (2003) (cited in: Foglia, 2006), a general statement is given that the aquifer is very deep (between 70 and 100 m) and consists of at least two layers, which can be distinguished: an upper layer of approx. 30 m with coarse sediments, and a lower layer with approx. 100 m on average with fine sediments.

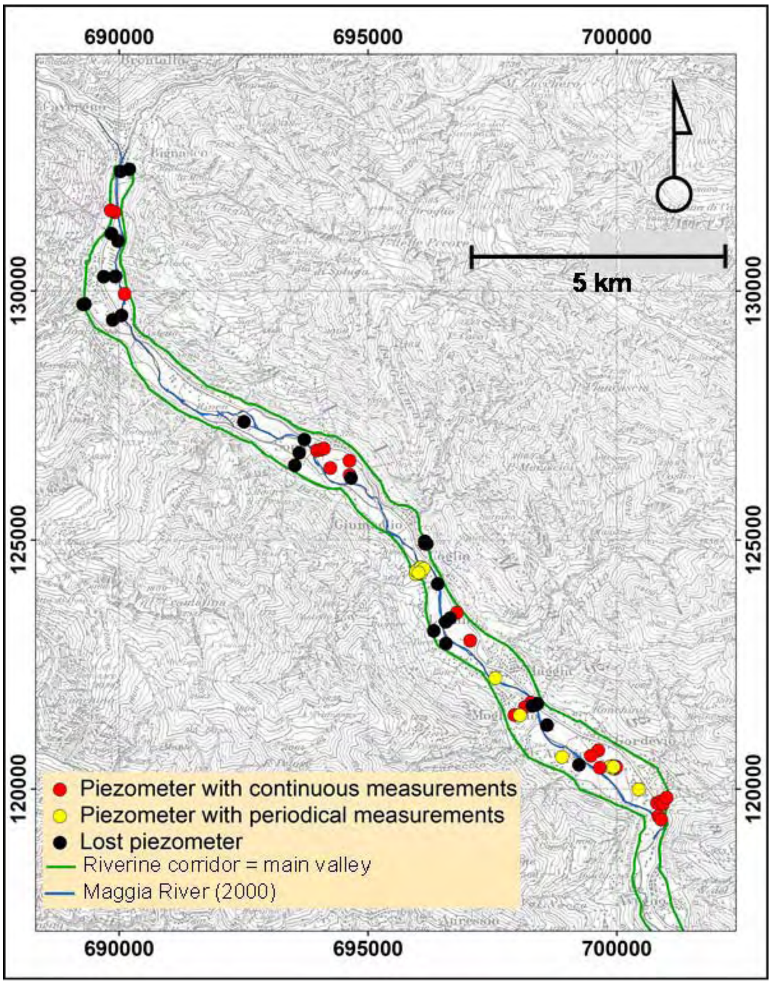


Fig. 3.14: Map of the locations of old and new piezometers (from: Foglia, 2006)

To complement the analysis of the geoelectrical soundings, *gravimetric measurements* have been performed in the lower part of the valley between Someo and Gordevio (Foglia, 2006). Their results were consistent with the geoelectric data. The depth of the aquifer can therefore be estimated in the longitudinal profile as shown in Fig. 3.15. The depth is approx. 120 m, there is an upper layer of about 30 m with coarse sediments, and a lower layer with relatively fine sediments. The depth of discriminating the two layers was additionally based on the analysis of core samples of deep piezometer boreholes (Foglia, 2006).

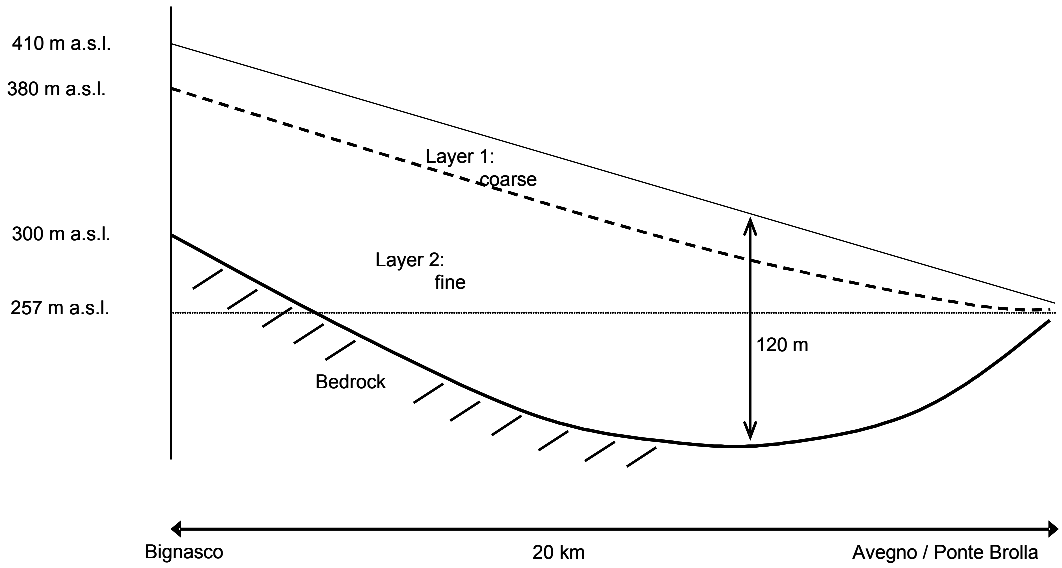


Fig. 3.15: Estimation of the bedrock and layering of the aquifer of the main valley in the longitudinal profile, based on geoelectric and gravimetric measurements (from: Foglia, 2006).

3.3.6. Data for the watershed model

Distributed soil data, land use data and a network of hydro-climatic and streamflow data were available for use in the watershed model. Some data have a daily resolution, others are recording continuously. Furthermore, numerous data from the hydropower operation system, like the time-evolution of the storage volumes of the reservoirs, water abstraction and spillways, are available to reconstruct the natural behaviour of the catchment as well as to quantify and investigate the anthropogenic influence within the watershed.

3.3.7. Hydropower Operation Data

All relevant hydropower operational data such as turbined water, released water, water level in the reservoirs, diverted and overflow discharges within the entire system resp. watershed were made available by the hydropower company OFIMA (*Officine Idroelettriche della Maggia SA*, Locarno, Switzerland), partly in daily, partly in subhourly time resolution, depending on the location and year.

Data on the turbined water in the hydropower plant in Giumaglio, run by the second hydropower company in the Maggia Valley, SES (*Società Elettrica Sopracenerina*, Locarno, Switzerland) were also available. Daily data have been digitized, hourly data are principally available on paper. Using a conversion factor provided by SES, the turbined water could be calculated from the energy production data.

At the outlet of the catchment between Avegno and Ponte Brolla, all water up to a maximum of 10 m^3/s from the Maggia River except of a minimum of 200 l/s is diverted and released back just 1 km further downstream (AET: *Azienda Elettrica Ticinese* (Ponte Brolla) since 2002, formerly SES). Also here, daily data were digitized, whereas hourly data are available on paper.

3.3.8. Overview over Data used in the Modelling System

Tab. 3.5 summarizes the most important data used in this study for the coupled modelling system in the Maggia valley. All these data ware described above in more detail. The role of the different models within the modelling framework is extensively discussed in the following chapter. Some of the data have already been used in the previous study by Foglia (2006), but then included in the overall modelling system.

Tab. 3.5: Overview over the most important data used for modelling in this study in the Maggia valley.

Type	Location	used for
Streamflow	- inflow in Bignasco - tributaries	- model input surface water model
	- gauging stations in main valley - episodic measurements along the Maggia River	- determination of infiltration / exfiltration - model validation surface water model
	- gauging station Locarno-Solduno	- calibration / validation of watershed model
Groundwater heads	piezometers in the main valley	- calibration / validation of groundwater / watershed model (Foglia, 2006)
- Hydro-climatic data - Soil and land use maps - DTM 25µm x 25 m	entire watershed	- model input watershed model (Foglia, 2006)
DTMs of different resolutions river cross-sections	different parts in the floodplain resp. river bed of the main valley	- surface topography for surface water model - initial conditions for groundwater model (Foglia)
- geophysical data - borehole profiles	- different parts / cross-sections of the main valley - piezometers in the main valley	- geometry and properties of the aquifer (Foglia)
line samples	selected locations in the river bed or gravel bars	- surface roughness
aerial photography	- entire main valley	- land surface types for the determination of surface roughness classes
	- time series of braided area	- delineation of vegetation types and their temporal development (accompanying study)

4. Development of the Modelling framework

4.1. Requirements

Spatially distributed and temporally variable patterns of infiltration and exfiltration are on the one hand important for the development of different aquatic habitats or for the detection of possible sources of pollution in the ground or surface water. On the other hand, they are either an essential boundary condition for modelling both surface water and groundwater systems, or they are the system's response to this forcing, depending on the implementation of the modelling system. Because of their variability in time and space, these boundary conditions cannot be treated independently, but must be determined by a common consideration of both systems, the fluvial and the groundwater system, together. Only in this way, the existing feedback mechanisms and effects can be considered. This is the essential point for the necessity of coupling the surface water and groundwater models together. As it has been pointed out in the state-of-the art, § 2.4.3, no existing coupling approach was available, which could fulfil the needs of the target of the study. Therefore, a new approach had to be developed, which is suitable for the implementation in alpine floodplains under conditions comparable to those found in the Maggia valley. This modelling framework is described in the following sections.

The spatial and temporal scale to be considered arises from the physical processes involved and from the task of investigation. The key area of interest is the braided area of the Maggia River within the riverine protection area. The delimitation of a modelling system restricted to the braided area alone would be very difficult, because of the absence of natural well specified upstream and downstream boundaries for the groundwater system in this case. The boundary conditions, i.e. groundwater flow into or out of the domain of the braided area, would therefore be unknown. Preliminary studies with the steady-state groundwater model have shown that these boundary conditions are quite influential for the solution (Foglia, pers. comm.). Therefore, the modelling domain for the groundwater and the hydrodynamic surface flow model had to be chosen as the entire main valley.

4.2. Structure of the Modelling System

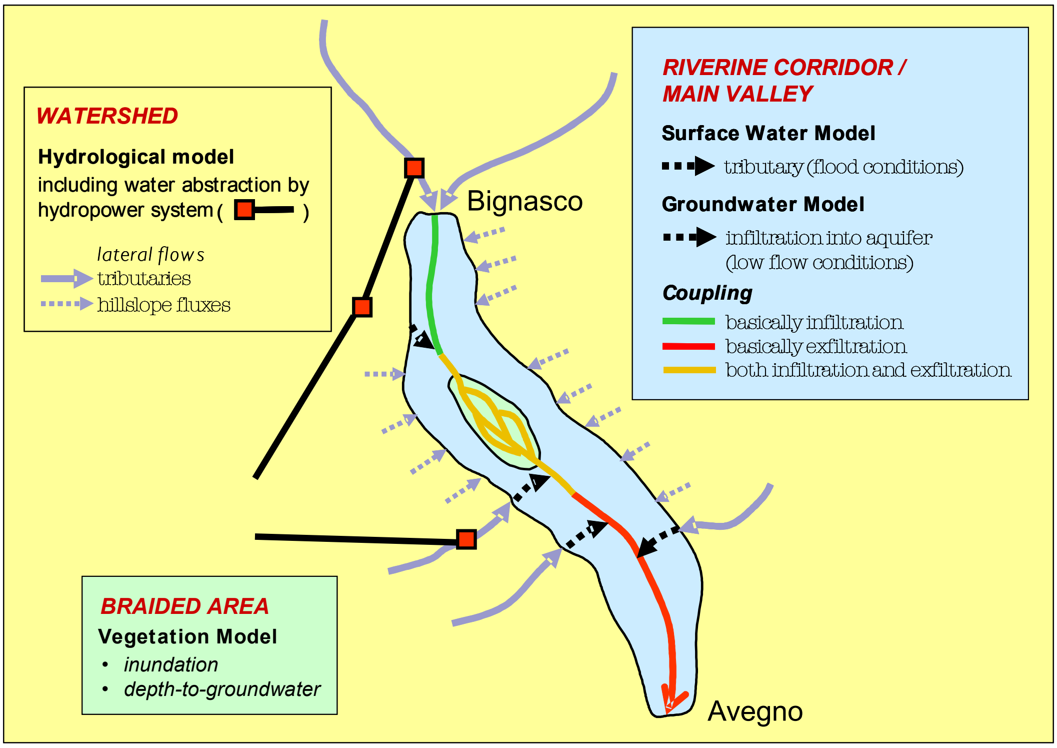


Fig. 4.1: *Modelling Framework (as implemented in the MaVal project): Spatial connection and alignment of the single models (different colours) within the modelling framework. See also Fig. 1.6 for the schematic interrelations between the models.*

For modelling the water fluxes in the system, a modelling framework has been developed (Fig. 1.6 and Fig. 4.1) consisting basically of three components: 1. a hydrological watershed model accounting for the entire watershed, 2. a hydrodynamic surface water flow model for the main valley, and 3. a groundwater model, also for the main valley. The coupled surface water – groundwater model will provide the information for the vegetation model, which is intended to simulate the development of vegetation in the braided area under consideration of episodic flooding and inundation events as well as groundwater table fluctuations, which are essential for the availability of water and nutrients for the germination and growth of riparian vegetation.

Water abstraction and diversions have already been considered in the hydrological watershed model. The output of this model are independent boundary conditions for the hydrodynamic surface water and for the groundwater model. Independent in this context means that there are no feedback mechanisms from the river or the aquifer into the watershed components. This is true in reality because of the steep slopes of the main valley, which restrict backwater effects both on the surface for the tributaries and in the groundwater for the hillslope subsurface flow.

The interface for the hydrodynamic surface water model are the tributaries on the surface, both at the upper boundary of the modelling domain in Bignasco, and the tributaries from the side valleys into the main valley. These tributaries from the side valleys however contribute only under high flow conditions to the Maggia River, since otherwise, most of the water is being infiltrated into the aquifer in the alluvial fans before reaching the main river. Lateral continuous hillslope fluxes on the surface, and especially in the subsurface into the aquifer of the main valley are modelled also by the hydrological watershed model. These fluxes act as boundary conditions for the groundwater model at the hillslope – aquifer interface.

The main focus of the dissertation lies in the interaction between river and groundwater, which is relevant in the main valley. For this purpose, the open-channel model is applied on the surface, the groundwater model (saturated flow) in the subsurface. These two are interconnected through the interface of infiltration resp. exfiltration. Exchange of water is hereby not limited to a single stream line, but distributed in space, in all cells which are wetted in the surface water model. These connection cells vary therefore in space and time during a simulation. Water flux can be upwards or downwards, depending on the difference in piezometric head of the river and groundwater.

An overview over the different models used in this modelling framework is given in Tab. 4.1 and will be explained in detail in the following sections.

Tab. 4.1: Overview over the models in the modelling scheme.

Type	Name	Characteristics	Area of implementation	used for
hydrological / watershed model	TOPKAPI	distributed continuous precipitation-runoff model	entire watershed	- hillslope fluxes into aquifer (one-way coupling with MODFLOW) - potential use for scenarios: discharge in tributaries
open-channel flow / hydraulic / surface water model	2dMb	2D shallow water equations	main valley (river and floodplain)	- simulation of water level, flow velocity and shear stress - full coupling with MODFLOW
groundwater model	MODFLOW-2000	Darcy-equation	main valley (aquifer)	- groundwater heads - full coupling with 2dMb

4.3. Hydrological Watershed Model TOPKAPI

The continuous and distributed rainfall-runoff model TOPKAPI (Todini and Ciarapica, 2001; Ciarapica and Todini, 2002; Liu and Todini, 2002) was used to model the runoff in the tributaries into the main valley and the diffuse surface and subsurface flow from the hillslopes of the main valley into the aquifer (Foglia, 2006). The model was originally (versions of 2001/2002) built for the purpose of flood forecasting, but has been adapted for continuous modelling and also for allowing the distributed simulation of subsurface flow.

Essential input data are a digital terrain model, soil and land use maps, and hydro-climatic data based on ground stations. As an output of the model, a spatial distribution of all water fluxes and state variables can be provided based on its grid structure. Hourly time steps were used in this study. Water abstraction or release of the hydropower operation was accounted for as external input or output and the operation so far not explicitly integrated.

A more detailed, nevertheless comprehensive description of TOPKAPI as used in the context of this project can also be found in Foglia (2006).

4.4. Groundwater Model MODFLOW-2000

4.4.1. Overview

MODFLOW-2000 (Harbaugh et al., 2000; Hill et al., 2000) was used for the groundwater modelling within this project. This acronym stands for MODular three-dimensional finite-difference groundwater FLOW model (version from year 2000). It solves Darcy's equation in 2D, but allows a 3D representation by dividing the aquifer into horizontal layers with spatially variable thicknesses. The numerical solving scheme is an implicit finite-difference one, which allows any time step, which is compatible with the temporal scale of the physical processes, to be chosen.

MODFLOW-2000 has been developed during many years at the U.S. Geological Survey. The first version came out in 1988 (USGS, URL_d) and has always been further developed since then. The model including the source code is public domain. The graphical user interface developed in PMWIN (Chiang and Kinzelbach, 2001) facilitated the dissemination application of the model. Due to its power for application in nearly all existing groundwater problems related to saturated flow, it has been widely used both in the scientific community and in practice and can be seen as a standard in groundwater modelling.

Its structure is modular, which allows adaptations relatively easily and makes it suitable also for coupling with other models or modelling components. The latter reason also allowed the usage in this study. The model is a flow model, however modules have also been developed for particle tracking or accounting for transport phenomena.

The model uses a structured grid allowing for variable grid spacing. In this study however, only a constant grid spacing was used. The Darcy-equation is solved in each horizontal layer including sinks and sources. The vertical dimension is accounted for by calculating the vertical mass transfer between overlaying cells depending on the head differences between these cells, the vertical conductivity and an eventual resistance layer between these cells. The water table can be confined or unconfined. Because of lack of sufficient data containing vertical information, finally only one single layer was used within the coupled model implemented in the Maggia valley (see also Foglia, 2006)[16]. Furthermore, MODFLOW can be run in steady-state or in transient mode. The structure and the solving procedure remains the same in both modes, just a few input parameters have to be changed and the time-dependent terms in the equations are zero in steady and non-zero in unsteady mode. All processes regardless if input or output routines, numerical solvers or the implementation of physical processes, are organized in so-called "packages". Some of them are mandatory, others are optional or there is the choice between different packages for the same task, e.g. for the selection of the solver. The packages which are of special interest in this work and which are described later are listed in Tab. 4.2.

Tab. 4.2: MODFLOW packages used in this work.

Package	Description	Usage
WELL	specification of sink and source terms in any selected cell	- flow boundary condition - link between surface water and groundwater model
RECHARGE	matrix of input or output of water for all active cells	- recharge from the hillslope - potential use for percolation and evapotranspiration
RIVER	specification of water level in the river and streambed elevation for all wet cells	- used as a tool in order to implement the full coupling

A detailed description about the basic structure of MODFLOW can be found in McDonald and Harbaugh (1988), which is the description of the first officially released version of MODFLOW. Changes, which had been made for MODFLOW-2000, and new packages are fully described in Harbaugh et al. (2000). A summary of the most important characteristics and the most relevant features of MODFLOW, which were

[16] *Foglia* technically implemented the stand-alone groundwater model with two layers, but used exactly the same parameter values for both layers. This is practically identical with the implementation of one layer. These two layers were combined to one layer when using it for the coupling as described in § 6.3.5.

applied in this study, can be found in Foglia (2006). Those features, which are relevant for understanding the coupling procedure and the interpretation of the results will nevertheless be explained in the following paragraphs.

4.4.2. Governing Equations and Solving System

Based on Darcy's law and the conservation of mass, the Boussinesq equation (4.1) is used to describe groundwater flow under steady-state conditions. (McDonald and Harbaugh, 1988; for the derivation of the equations (4.1) and (4.2) see e.g. Rushton and Redshaw, 1979).

$$\frac{\partial}{\partial x}(K_{xx}\frac{\partial h}{\partial x})+\frac{\partial}{\partial y}(K_{yy}\frac{\partial h}{\partial y})+\frac{\partial}{\partial z}(K_{zz}\frac{\partial h}{\partial z})+W=0 \quad (4.1)$$

Under transient conditions, the storage and the time derivative term is added to eq. (4.1) leading to the following eq. (4.2), which is first order in time:

$$\frac{\partial}{\partial x}(K_{xx}\frac{\partial h}{\partial x})+\frac{\partial}{\partial y}(K_{yy}\frac{\partial h}{\partial y})+\frac{\partial}{\partial z}(K_{zz}\frac{\partial h}{\partial z})+W=Ss\frac{\partial h}{\partial t} \quad (4.2),$$

where: x, y, z are the coordinates in space [**L**]; K_{xx}, K_{yy}, K_{zz} are the values of the saturated conductivity of the aquifer along the x-, y-, and z-direction [$\mathbf{LT^{-1}}$]; h is piezometric head [**L**]; t is time [**T**]; W is a volumetric flux per unit volume representing sources or sinks [$\mathbf{T^{-1}}$]; Ss is the specific storage of the porous material [$\mathbf{L^{-1}}$].

These two equations are standard in groundwater modelling. For internal calculation, they were transformed in order to solve them more easily (McDonald and Harbaugh, 1988; Harbaugh et al., 2000). Different solvers are implemented in MODFLOW-2000 and can be selected as single packages. All of them use an implicit scheme for solving the partial differential equations.

Under transient simulations, the time steps can be adaptive or kept constant. The structure of MODFLOW is such that it runs as a sequence of so-called stress periods. The result of a preceding stress period is the initial condition for the following stress period. Each stress period can be defined as steady-state or as transient in arbitrary sequence. Additionally, most parameters, boundary conditions etc. can be defined separately for each stress period. This allows a very high flexibility to the users.

4.4.3. Grid

As most finite-difference models, MODFLOW-2000 uses a rectangular grid. This makes the computational scheme relatively easy. The drawback is however that the grid is not flexible to adapt to the areas of interest, which is the river in this study. Fluxes not parallel to the edges of the grid will always create some numerical diffusion, which will be a source for some inaccuracy. MODFLOW has also been further developed to use a bended mesh, which allows to follow bended structures, but still keeping the structured alignment of the cells (Jones, 1997). In this case, a 2D Galerkin finite-element solution for flow within a layer is used, whereas a finite-difference solution for vertical flow is used. Because of the quadrilateral grid of the watershed model, which serves as a boundary for the groundwater recharge from the hillslopes, and in order to keep the coupling as simple as possible, the original version of MODFLOW-2000 with all its including features has been selected.

An unlimited number of layers can be used to account for the vertical dimension. The planar view of each layer must be identical for all of them. The boundaries between the single layers need not to be parallel, but can be defined differently for each cell. Also impermeable or partly permeable layers can be defined.

4.4.4. Boundary and Initial Conditions

There is the principal distinction between no-flow boundaries and flow boundaries. No-flow boundaries do not allow any flow through its boundary. For the flow boundaries, two possibilities exist: the specification of a piezometric head or no specification. In the transient case, boundary conditions can be defined variable in time. If no connection with the surface water or other fixed head is given, at least one cell with a fixed head is needed to be specified in order to let the solver converge the equation system.

If flow should be specified at the boundary, this must be done in the form of source and sink terms. They will be accounted for in the water balance within the specified cells, but are not part of the boundary terms in the partial derivatives' equation system. Since momentum of inflowing and outflowing groundwater is small, this will not substantially effect the solution. There are different possibilities to define these flows at the boundaries: using the WELL package (§ 4.4.5) or the RECHARGE package (§ 4.4.6). The first has was for the inflow and outflow boundary in the longitudinal direction of the valley, the second to account for the lateral hillslope contribution into the aquifer.

As initial conditions, groundwater heads in any single cell have to be specified. The closer they are to the final solution, the faster the model converges. The topographic surface was used for the initial heads, since it is relatively close to the real head, if the groundwater table is not too deep. For transient simulations, it is suggested (Harbaugh et al., 2000) to use the results of a steady-state run as an initial condition. In this study, data from the digital terrain models were used as the initial condition for the steady-state run, whose head solution in itself serves as initial condition for the succeeding transient run.

4.4.5. WELL Package

The WELL package was originally developed for including wells and pumps in the aquifer. For each cell, a constant or time-variant uptake or addition of water can be defined and applied like a sink and source term, in other words: like an internal boundary condition. Just a mass flux is hereby defined, no momentum will be accounted for. The well package was used in this study for defining flow boundaries also at the edges of the modelling domain, but also as sink and source terms for the cell-by-cell river – groundwater interaction (see version (b) in § 5.5.6). Using the WELL package, it is possible to apply exchange fluxes which have been calculated by the surface water model.

4.4.6. RECHARGE Package

The recharge package specifies a water flux into the aquifer for each cell or zone of cells, constant or variable in time. It is meant to be used for recharge due to precipitation and deep percolation from the unsaturated zone, but it can also be used for the contribution from the hillslopes at the aquifer boundary. In flat areas, average deep percolation rate is basically the difference between mean precipitation and mean actual evapotranspiration. This recharge from deep percolation (as well as evapotranspiration) can be neglected in the Maggia valley, as illustrated in § 6.3.3.

There are however the hillslope fluxes, which have to be considered. The RECHARGE package was used for this purpose. These fluxes can just be estimated, be determined by calculation, or as a result of the calibration process. In our study, the spatial distribution of the recharge was the result of simulations with TOPKAPI, while the magnitude was determined by a multiplier which is constant over the whole aquifer and is the result of calibration by inverse modelling using UCODE[17] in MODFLOW (Foglia, 2006).

4.4.7. RIVER Package

The RIVER package technique plays a key role in this study, but had to be adapted to be compatible with the transient surface water model. The original version of the RIVER package can handle an arbitrary spatial distribution of (wet) river cells, but with a constant water head in these cells. River bed levels, water heads and the magnitude of the river area in each cell have to be specified. If known beforehand, they could be specified variable in time, but no feedback mechanisms between aquifer and river can be included. The decisive step in this work was to account for these feedback mechanisms.

The mathematical formulation of the exchange flux of water depends on whether the groundwater and surface water are connected to each other or not. Connection means that the aquifer in the groundwater cell is saturated at least up to the river bed; an

[17] UCODE is a universal code for automatic calibration of model parameters developed at USGS (USGS, URL_e).

unsaturated zone occurs in the unconnected case. Connection is mathematically defined as the river bed elevation being equal or lower than the groundwater head; the two systems are unconnected, if the river bed elevation is above the groundwater head. This means that the condition, whether the systems are connected or not, can vary with time during the simulation depending on the variation in the groundwater head. In two of the coupled versions being described later (versions (c) and (d)), the RIVER package technique is used. In this case however, this river water level will be variable and therefore calculated and updated continuously during the simulation by the surface water model 2dMb (see § 5.5.7 and 5.5.8).

For the mathematical description of the river – aquifer exchange, MODFLOW implements, as already mentioned in § 2.4.2, a widely used approach, where – in the connected case – the exchange rate is linearly dependent on the the difference in head between river (i.e. surface water) and groundwater. The proportionality factor is a leakage factor and accounts for the resistance of the river bed layer. It is formulated as followed:

$$Q_{exch} = \frac{K_{riv} L W}{M} (H_{riv} - h_{i,j,k}) \quad \forall \quad h_{i,j,k} > r_{bot} \quad (4.3) \text{ (river and aquifer are connected)}$$

with $C_{riv} = \frac{K_{riv} L W}{M}$ (4.4)

and $A_{riv} = L \cdot W$ (4.5),

where Q_{exch} is the flow between the stream and the aquifer, taken as positive if it is directed into the aquifer [$\mathbf{L}^3\ \mathbf{T}^{-1}$]; h_{riv} is the head in the stream [**L**]; C_{riv} is the hydraulic conductance of the stream-aquifer interconnections [$\mathbf{L}^2\ \mathbf{T}^{-1}$] where K_{riv} is the hydraulic conductivity of the riverbed material [$\mathbf{L}\ \mathbf{T}^{-1}$], L is the length of the reach [**L**], W is the width of the river [**L**], and M is the thickness of the riverbed [**L**]; hi,j,k is the piezometric groundwater head at the node in the cell underlying the steam reach [**L**], and A_{riv} is the wet area within one MODFLOW cell specified as RIVER cell [$\mathbf{L}^2$]. It should be mentioned that the streambed conductance is the same for infiltration or exfiltration conditions in the original version of the RIVER package.

In the unconnected case, only infiltration from the river into the aquifer takes place. The mathematical formulation comprises a linear dependence on the water depth in the river cell. The same proportionality factor for the resistance of the river bed layer is used as for the connected case. The equation is:

$$Q_{exch} = \frac{K_{riv} L W}{M} (h_{riv} - r_{bot}) \quad \forall \quad h_{i,j,k} \le r_{bot} \quad (4.6) \text{ (river and aquifer are unconnected),}$$

where r_{bot} is the bottom of the streambed [**L**]; all other variables are defined as in eq. (4.3).

The parameters K_{riv} (hydraulic conductivity of the riverbed material) and thickness of riverbed, M, of the resistance layer cannot be distinguished from each other, if there are no measured field data available. They are not identifiable parameters during the calibration process, since they appear always as the ratio K_{riv}/W (Foglia, 2006). They were therefore treated as one single parameter by normalizing the streambed conductance factor C_{riv} while setting the thickness equal to 1 m. Measurements are not available in the Maggia valley. The armouring layer consists of large gravel-sized particles, which make any field measurement extremely difficult. Even if local measurements were available, they would not be representative for the scale of the modelling. Values for the streambed conductance are therefore usually only determined by the calibration process.

The parameter river area A_{riv} ($= LW$) in the MODFLOW cell must also be specified as an input and known in advance, if the stand-alone application of MODFLOW-2000 is used. In our study, this parameter was used in two different ways: 1. assumed to be constant for all river cells and calibrated with $[K_{riv} A_{riv}/M]$ as a single parameter, and 2. being variable in space and time by calculating it within the coupled model. The first way was applied in a simplified coupling strategy, version (a) (see § 5.3), where MODFLOW and 2dMb were used as stand-alone programmes and run in an iterative manner. Since in this simple approach, no information was available about the wet area of the river cells, the parameters K_{riv}, A_{riv}, and M were identifiable only as the group $[K_{riv} A_{riv}/M]$. The second way was applied in the case of the full coupling of 2dMb (versions (b) to (d), see § 5.3). Here, the wet area in each MODFLOW cell was variable in time and was calculated on the basis of the number of corresponding wet cells in 2dMb (related to a given MODFLOW cell).

4.5. Surface Water Model 2dMb

4.5.1. Generalities

The model used in this work for the simulation of surface flow resp. wave propagation within the river channel and the floodplain is the 2-Dimensional Mobile Bed Model (2dMb), developed at the Laboratory of Hydraulics, Hydrology and Glaciology (VAW), ETH Zürich, Switzerland (reported in: Faeh, 1996). It is a 2D hydrodynamic open channel flow model, which solves the shallow water equations in horizontal 2D using depth-averaged variables in an unsteady mode. Quasi-steady-state results can however be obtained using boundary conditions being constant over time until no further changes occur in the simulation results.

Parts of the model were developed by *Beffa* and are also included in the HYDRO2DE model (for description see Beffa and Connell, 2001; Beffa and Connell, 2002). Many of its features are explained in more detail in Beffa (1994). 2dMb uses the finite-volume method with a cell-centred structured grid (in our case only quadrilaterals). This means that all state-variables are calculated or interpolated to the centre of the grid. The model outputs are water level and depth-averaged flow velocity components (v_x, v_y) in each grid cell, furthermore the derived variables flow depth, magnitude of velocity, shear stress and Froude number. Wetting and drying of cells is possible due to a special treatment of internal boundaries. There is further the possibility of including source and sink terms in each particular cell. This feature was used in the coupling procedure with MODFLOW. Source and sink terms can also be used to account for the flow from small tributaries.

Different friction equations can be selected (see § 4.5.4). In this study, only a modified logarithmic function was applied.

A mobile bed component could be selected in order to account for vertical erosion and deposition as well as transport of sediments in the river bed (Fäh, 1997). However, this option was not used in our study, because it was outside of the target of this work, where as a fist step, the coupling of the groundwater model with the surface water model alone was to be achieved (see also § 8.3.2).

The cell-centred finite-volume scheme has the advantage that the water balance over the edges of each single cell can be closed, as already mentioned in (§ 2.2.2), thus leading to a high numerical stability of the algorithm. The model was therefore proved to be robust even under difficult hydraulic conditions such as high Froude numbers and hydraulic jumps, which occur frequently in space and time in a natural environment such as that of the Maggia River. The model is however very computational demanding, as explained in the following paragraph.

The numerical scheme is explicit. In connection with very small mesh sizes – as it was the case in the Maggia valley (6.25 m) to allow also for very low discharge in the river –, this leads to very small time steps (here less than 1 second). This is due to the requirement of fulfilling the Courant-Friedrichs-Levy condition (CFL-condition) (Courant et al., 1928), which is a necessary numerical requirement for the convergence of an explicit mathematical solver system (eq. 4.7 and 4.8).

$$\frac{u \cdot \Delta t}{\Delta x} < C \quad (4.7) \text{ (one dimensional case)}$$

$$\frac{u_x \cdot \Delta t}{\Delta x} + \frac{u_y \cdot \Delta t}{\Delta y} < C \quad (4.8) \text{ (two dimensional case),}$$

where u is the velocity [**L T**$^{-1}$] ($u= v+c$, with v is flow velocity and c is celerity), Δt is the time step [**T**], and Δx and Δy are the length intervals [**L**], whereas the constant C is the Courant number [**-**]. For numerical reasons, a Courant-number close to one can already create numerical instabilities.

Much larger time steps as required for numerical reasons would generally be sufficient from the point of view of the physical processes. This would be possible only by using an implicit solving scheme. Nevertheless, an explicit scheme was implemented because of the variation of internal boundaries (dry cells) during the simulation. Using an implicit scheme would imply that not only the state variables are part of the solution to be solved, but also the boundary itself. This would lead to a complicated and also time-consuming implicit solving scheme.

4.5.2. Governing Equations

As noted above, the model 2dMb solves the shallow water equation in horizontal 2D, which means that all cell variables are averaged over depth such as momentum and velocity. The equation system respects the conservation of mass (4.9) and conservation of momentum (4.10), and can be written as followed (Beffa, 2004) (the equation for 1D has been shown already in § 2.2.2):

$$\frac{\partial h}{\partial t}+\frac{\partial q}{\partial x}+\frac{\partial r}{\partial y}=0 \quad (4.9)$$

$$\begin{aligned}\frac{\partial q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{q^2}{h}+\frac{gh^2}{2}-\frac{h}{\rho}\tau_{xx}\right)+\frac{\partial}{\partial y}\left(\frac{qr}{h}-\frac{h}{\rho}\tau_{xy}\right)&=-gh\frac{\partial z_b}{\partial x}-\frac{\tau_{bx}}{\rho}\\ \frac{\partial r}{\partial t}+\frac{\partial}{\partial x}\left(\frac{qr}{h}-\frac{h}{\rho}\tau_{xy}\right)+\frac{\partial}{\partial y}\left(\frac{r^2}{h}+\frac{gh^2}{2}-\frac{h}{\rho}\tau_{yy}\right)&=-gh\frac{\partial z_b}{\partial y}-\frac{\tau_{by}}{\rho}\end{aligned} \quad (4.10)$$

In these equations, the unknown variables are the flow depth h [**L**] and the x- and y-components of the specific discharge q [**L**^{2}**T**$^{-1}$]. Constants and derived parameters are: z_b = geodetic height of the river bed [**L**], τ_{bx}, τ_{by} = bed shear stress [**M L**$^{-1}$ T^{-2}], ρ = density of the fluid, [**M L**$^{-3}$], τ_{xx}, τ_{yy} = turbulent normal stresses, τ_{xy}, τ_{yx} = turbulent shear stresses.

The approximation of the solving scheme implemented in 2dMb can be first or second order. The second order solution should provide a more consistent solution, but the computational effort is much higher. Therefore, the first order approach is suggested to use for most applications, where the required accuracy of the solution is not extremely high.

4.5.3. Grid Structure

The eq. (4.9) and (4.10) must be applied over a modelling domain, which is segmented into cells, so called grids. The model 2dMb uses structured grids, whereas the spacing can be locally refined over the entire length or width of the modelling domain. In our investigations, only constant grid spacing over the whole domain was used.

The grid size has to be chosen with respect to the scale of the physical processes to be modelled. Furthermore, numerical constraints may require even finer grids. Especially, when also low flow conditions should be simulated, the grid resolution must be high. Coarser grids lead to numerical problems in drying out the domain downstream, if the flow rate is low. As a drawback of these fine grids however, this fact leads to very small time steps. Therefore, the model is computationally demanding, while computer capacity (RAM) and processor frequency still play an essential role when simulating large domains. In the case of simulating the entire main valley of the river Maggia for instance, the computational time is still in the order of real-time. There are strategies to reduce or overcome this problem, parallelization of the code is one of those. They will be described in the outlook (§ 8.3.3). In theory, a refinement of the grid by the factor of 2 prolongs the computational time by a factor of 8 because of the fourfold number of cells and the bisection of the time step due to the CFL-condition.

Geodetic data for the river bed and surface of the floodplain have to be interpolated onto the corner of the grid cells. A pre-processing tool is available to interpolate scattered data or cross-section data onto the regular grid. In this study however, external procedures using GIS have been used due to the large number of data.

All internal calculations use cell-centred variables. For this reason, 2dMb interpolates the z-values from the four corner nodes onto the centre of the cells. All output variables (results) are then also representative for the centre of the cells.

4.5.4. Friction and Turbulent Losses

External and internal forces have to be considered in the partial differential equation system. External forces stem from bed friction and wind friction. In our field of application, wind friction plays only a minimum role and can therefore be neglected. Bed friction is represented by the friction law. Internal forces are turbulence forces and laminar forces. In our case, laminar forces are also very small and are hence neglected. Attention must also be given to turbulent forces. Their consideration is essential in three-dimensional models, since they do not only provoke dissipation of energy, but also vertical exchange of momentum. Therefore, the 3D-solution is highly effected by this process. In a vertical 2D-model, the vertical momentum distribution is not considered. The only effect of turbulence forces is the horizontal exchange of momentum and the dissipation of energy. Common approaches for the description of turbulence losses are the k-ε or k-ω approach or – in the three-dimensional case – large eddy simulations (LES).

Large horizontal eddies are explicitly accounted for in the model without turbulent terms, if they are larger than the grid size. Eddies in a subgrid-scale can be handled only with eddy-viscosity terms, which account for turbulent losses. There is still the question, whether a separate consideration of turbulent losses and bed friction losses leads to better solutions, which are distinct from solutions considering only bed friction. Experience with 2dMb (Fäh, pers. comm.) were such that the differences in the solutions of flow depth and flow velocity are so small that it is not worth to consider them separately with the drawback of having more calibrating parameters, which cannot be measured in the field. Furthermore, the computational effort would increase substantially. Considering the fact that this is already a limiting factor, the possibility of using separate turbulence parameters has been abandoned, only the bed friction is considered, which accounts for the total energy losses. This means on the other hand that the roughness (bed and form roughness) derived from possible field measurements should be theoretically a bit smaller than the model roughness, which subsumes additionally implicitly the turbulent losses.

Roughness formulae describe the relationship between the friction and the depth-averaged velocity, and therefore also the flow depth. Many different empirical formulae exist for this purpose, in 2dMb different formulae can be selected. There is the wide-spread Manning-Strickler formula (eq. 4.11) implemented, and two formulae, which assume a logarithmic velocity profile (eq. 4.12). A comparison between the different implemented friction formulae is shown in (Fig. 4.2). Many theoretical and experimental investigations show that none of these formulae is universally applicable for the entire range of possible flow depths, especially with small relative submergence, i.e. if flow depth is small compared to the size of the roughness elements (e.g. Dittrich and Koll, 1997; Bezzola, 2002; Manes et al., 2007). In this case, it can be distinguished between an inner and outer layer resp. the wall region and the free-surface region, whereas there is an intermediate zone between them. The shapes of the vertical velocity profiles differ between these two layers. The logarithmic law is applicable in the outer layer, which is the case if the ratio of the flow depth y and the equivalent diameter of the roughness elements k_s is large enough. Different values for the threshold of y/k_s exist, Dittrich and Koll (1997) and Bezzola (2002) report a minimum value of 5. In any case, the formula is not appropriate for very low flow conditions, i.e. the accuracy is therefore reduced if applied there. The Manning-Strickler formula is a power law and can be seen as an approximation to the logarithmic law within a certain range of y/k_s, as it was shown by Bezzola (2002).

The accordance of the different formulae available in 2dMb is illustrated in Fig. 4.2. The curves are relatively close in the mid-range of y/k_s between 3 and 6. From the boundary layer consideration for low flow conditions from above, it is a priori not clear from the theory, which of the formulae is better suitable for very small flow depths (Beffa, 1994). As the logarithmic velocity distribution reflects however theoretically the physical behaviour for higher relative submergence such as for moderate and high flows, and since its parameter k_s has a physical meaning and can therefore be determined from field measurements, the logarithmic formula (4.12) was selected in this work.

$$\bar{u} = k_{St} \cdot r_{hy}^{2/3} \cdot S_f^{1/2} \approx k_{St} \cdot h^{2/3} \cdot S_f^{1/2} \quad \text{(4.11) (Manning-Strickler formula)}$$

$$\begin{aligned} \bar{u} &= u_* \cdot 4.5 \quad \forall \quad h \le 0.5 k_s \\ \bar{u} &= u_* \cdot 5.75 \cdot \log_{10}(12 \cdot h / k_s) \quad \forall \quad h > 0.5 k_s \end{aligned} \quad \text{(4.12) (logarithmic flow velocity law)}$$

with $u_* = \sqrt{g r_{hy} S_f}$ (4.13),

where $\bar{u}$ is depth-averaged flow velocity [**L T**$^{-1}$], u_* is shear velocity [**L T**$^{-1}$], k_{St} is Strickler coefficient (1/Manning number) [**L**$^{1/3}$ **T**$^{-1}$], r_{hy} is hydraulic radius [**L**], S_f is friction slope [–], h is flow depth [**L**], k_s is equivalent (Nikuradse) sand roughness [**L**], and g is the gravitational acceleration [**L T**$^{-2}$].

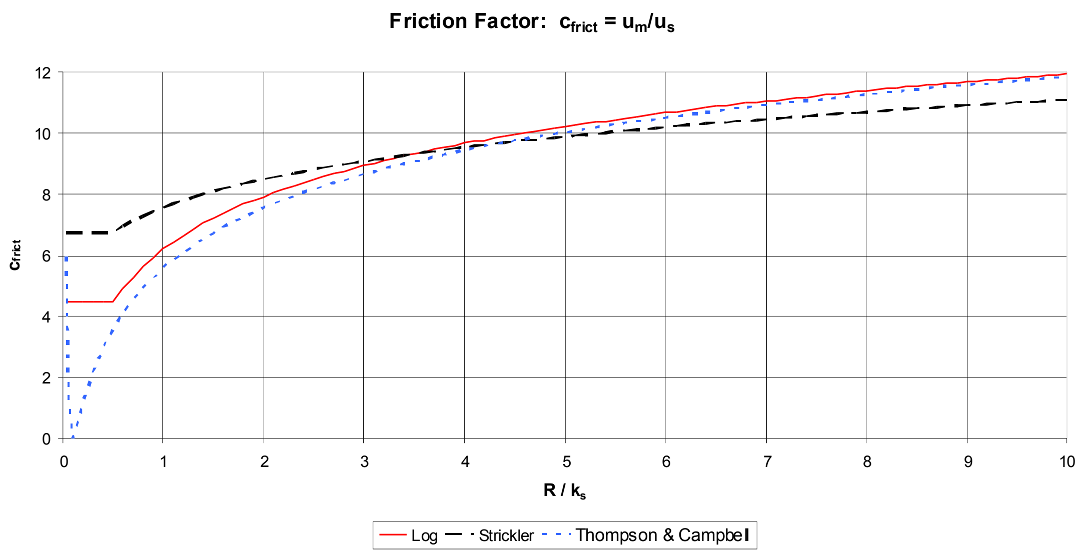


*Fig. 4.2: Comparison between the different friction formulae implemented in 2dMb. R is hydraulic radius (which corresponds to the flow depth for wide channels), k_s is the equivalent sand roughness and c_{frict} is the friction factor, from which the mean velocity u_m can be calculated as c_{frict}*u_m, with u_s as bed friction velocity.*
The correspondence between the Manning-Strickler and the logarithmic formulae is best in the middle range of flow depth, where flow depth is between three and six times the diameter of the relative sand roughness k_s elements (using the conversion between sand roughness and Manning values implemented in the model).

Bed friction is considered only along the river bed, vertical walls or obstacles are not considered. In this case, the hydraulic radius and the flow depth are equal, if the river bed is horizontal. Otherwise, it is slightly smaller than the flow depth. This fact could be corrected for by considering the real area of a grid cell instead of its horizontal projection. In reality, this deviation is very small, as long as the surface is not too steep so that the approximation $\cos\alpha \approx 1$ (where α is the angle of the slope of the river bed) still holds.

In the logarithmic velocity law being used here, the concept of the equivalent sand roughness is applied. This is also known as the Nikuradse sand roughness (Chow, 1973; Bezzola, 2002). The local equivalent sand roughness can be estimated by field measurements. Principally, volumetric and line sampling (German: "Linienproben" methods can be used to derive the median diameter meter d_m of the armouring layer, which is set equal to k_s.

However, these values would apply only for flat surfaces. Natural rivers have additional roughness$_s$ due to riffles and pools on a larger scale. This can be accounted for with a so-called form factor β, which is normally in the range between 1.5 and 4, whereas the value of 2 is suggested for a nearly plane bed and a smooth longitudinal profile (Bezzola, 2003). Total roughness k_{tot} is therefore the multiplication of $k_s \cdot \beta$. However, natural roughness and model roughness do not always need to be the same. One reason is that the form factor may depend on the grid size, because the size of the features responsible for the form roughness can be smaller or larger than the grid resolution. Instead of measuring the mean grain size of the river bed and derive the equivalent Nikuradse roughness, empirical tables can also be used. In either case, roughness values used in simulation models should be validated with field observations of water level, inundation extent or flow velocities.

An additional problem occurs with the occurrence of vegetation. As in most models, which are applied for engineering purposes, vegetated areas are treated with the same friction formula. This approach is not completely valid, since vegetation generates roughness not only at the bottom of the inundated area, but to a large extent also due to the stems and leaves. Furthermore, vegetation can bend in the flow depending on flow velocity so that its friction is variable in time during flood events.

The relatively simple modelling approaches represent natural vegetation by stiff vertical rods with a given diameter and density. The friction laws obtained by this approach are structurally different from the logarithmic laws or Manning-Stickler type of equations, and can therefore not just transformed into each other by adapting a roughness parameter. The resistance terms are more complex, and the relationship between flow velocity and flow depth can no longer be described with a simple logarithmic or power law. It is even more complex, if the flexibility of vegetation is taken into account. In this case, plants can bend into the horizontal flow therefore also reducing the resistance to flow (e.g. many *Salix* species). Often, the required parameters to describe the resistance behaviour of vegetation are not available and have to be assessed in extensive field campaigns.

Good correspondence between observed and simulated water levels respectively flow depth can be achieved using just one single Nikuradse parameter accounting both for resistance to bed friction and to vegetation. In this case however, results obtained for shear stress and therefore for flow velocity will be considerably different from reality because of the distinct friction – velocity relationship. This has been shown e.g. in a study in the Allier river (France) (Baptist, 2006). For further detailed investigations of the bed friction within inundated vegetated areas, a more detailed vegetation model (Lindner, 1982; Baptist, 2006; Kowalski et al., 2006; Baptist, in press) should be implemented. However, all those mathematical approaches, which are operationally applicable, do not consider the bending of vegetation.

Roughness parameters (Nikuradse sand roughness) can be assumed constant for the entire modelling domain, or variable for each cell. Nine different surface cover types were delineated from aerial photography (orthophoto) from the year 2000 (see Tab. 6.2 and Fig. 6.11). This allowed to identify zones with constant roughness values depending on the surface properties, resp. vegetation type.

4.5.5. Internal Boundaries: Wetting and Drying of Cells

Dynamic surface water bodies are characterized by variable wetted perimeters. Only wet cells are used in the calculation algorithm. The boundaries between wet and dry cells are treated as internal boundaries. This implies that the boundary for the internal calculation is variable in each single time step. Special algorithms had to be developed for the drying and wetting of the cells. Numerical problems occur, since there is – independent from the calculation scheme – always one point, where there is a division by the flow depth (Beffa, 1994). If a cell is dry, a division-by-zero error would occur or the solution would not converge. In order to avoid this problem, a cell is considered to be dry, if the flow depth becomes smaller than a threshold h_{dry}, which should be chosen relatively small, i.e. in the order of the mean grain size in case of rough beds. If a cell is dry, it is no longer part of the computation, and all fluxes over the cell edges are set to zero. Vice versa, a cell can be wetted again by fluxes from a neighbouring cell, if the resulting water depth is larger than h_{wet}, which is set as h_{dry} + 1 cm. Nevertheless, wetting and drying of cells is a highly non-linear process and can always give reason for numerical instabilities affecting the convergence process. This is also the reason for different values of h_{dry} and h_{wet} in order to dampen some possible oscillations and therefore to stabilize the solution.

4.5.6. Sinks and Sources

Sinks and sources can be implemented in each single cell. They contribute to the water balance in each cell for each single time step. They can be used for water abstractions or additional input, as they can also be used for additional tributaries. This is especially helpful for such rivers, which do not have a permanent discharge.

Moreover, and more central for this work is the fact that the availability of sink and source terms give the possibility to couple the 2dMb model with the MODFLOW model. The numerical considerations are described in § 5.5.4.

5. Coupling of Surface Water Model and Groundwater Model

Some general theoretical aspects about surface water – groundwater interaction as well as its coupling approaches in general have already been described an discussed in § 2 and 4. Here, the specific coupling of the surface water model 2dMb and the groundwater model MODFLOW-2000 is described. The most important issues to consider were the following:

- Choice of the mathematical formulation of the river – aquifer exchange process
- Grid-to-grid transformation of exchange rates and hydraulic heads considering different grid sizes and orientation, accounting also for the time and space variability of wet cells of the 2dMb grid
- Time scheme of communication between the two models and exchange of information between them, accounting also for different time-scheme requirements for the single models
- Numerical stability of the coupling scheme
- Reproduction realistic results of the river-aquifer interaction

5.1. Grid Size and Time Steps

Both grid size and time steps are dependent on each other by the Courant number (see § 4.5.1). This is especially true in the case of explicit numerical schemes as it is the case for the hydrodynamic model 2dMb. Numerical considerations under low flow conditions (§ 4.5.3 and 4.5.4), the scale of the physical processes and the consistency of the solutions (§ 6.1.3) determine the size of the grid of the surface water model. In connection with the maximum flow velocities to be simulated, the CFL-condition determines the time step of the 2dMb model, even if the physical processes would allow larger time steps (§ 4.5.1). Because of the much slower dynamics of the groundwater flow, much larger grid sizes and time steps are allowed. Additionally, the implicit scheme allows to expand the limits to the requirements of the physical processes.

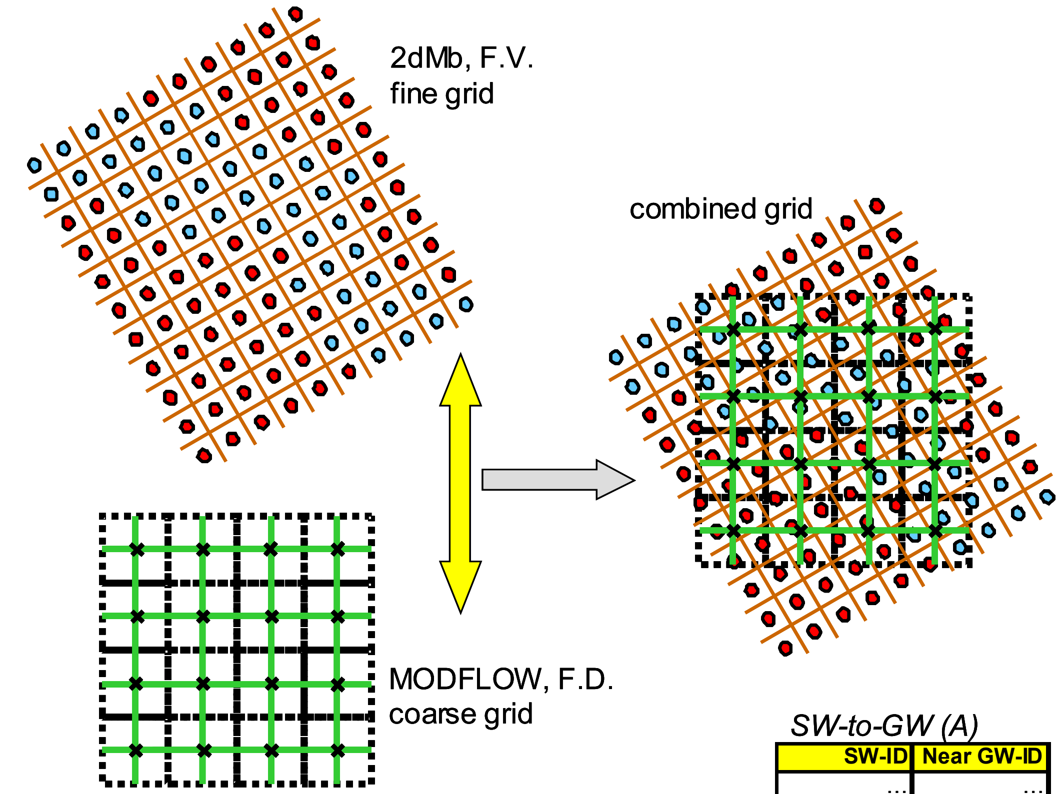
Grid orientation

Quadrilateral grids were used both in MODFLOW and in 2dMb. The coupling, as it is implemented here, allows however a different resolution as well as orientation of the two grids. This gives more freedom for optimizing each grid separately. The length of a MODFLOW cell must however be a whole-numbered multiple of the length of a 2dMb cell. The scheme is generally applicable as long as uniform grids with quadrilateral cells are used.

Only those cells of the rectangular domain, which cover the common area of interest, i.e. in the Maggia valley the potentially inundated area or the aquifer, are set as active cells, all others are excluded from the calculations. This is true both for MODFLOW and for 2dMb.

Both grids are structured with constant grid spacing. In order to communicate with each other, i.e. to assign the correct cells for river – aquifer exchange flux between the two models, two transfer matrixes were calculated beforehand. This is possible, because the size and location of the grid cells do not vary throughout the simulation. The relation is built only for the active MODFLOW and active 2dMb cells. The transfer matrixes do not consider, whether a 2dMb cell is dry or wet, this has to be taken into account during the simulation. All 2dMb cells in the whole 2dMb domain and all MODFLOW cells in the entire MODFLOW domain have their specific ID-number.

The two different transfer matrixes are illustrated in Fig. 5.1: One for transferring information from MODFLOW to 2dMb (matrix ***A***), and the other to pass information in the opposite direction, i.e. from 2dMb to MODFLOW (matrix ***B***). ***A*** contains two columns, the first with the cell ID of the active MODFLOW cell, the other with the cell ID of that 2dMb cell, whose cell centre is nearest to the given MODFLOW cell centre.



SW-to-GW (A)

SW-ID	Near GW-ID
…	…
…	0
…	…
234	1
235	1
236	…
…	…
364	m
365	m
455	m
456	m
…	…
475	3
476	3
478	1
489	3
490	3
…	…
527	3
…	…
n	…

Transfer matrixes

GW-to-SW (B)

GW-ID	SW-ID 1	SW-ID 1	SW-ID 1	SW-ID …	SW-ID k
1	234	235	478	…	0
2	0	0	0	0	0
3	475	476	489	490	527
…	…	…	…	…	…
…	…	…	…	…	…
m	364	365	455	456	0

Fig. 5.1: Schematic of the relationship between the grids and the transfer matrixes. Red dots are dry 2dMb cells, blue dots are wet cells. The crosses are the calculation points of the MODFLOW grid, the bold quadrilaterals are the related areas to these calculation points. The tables (transfer matrixes) are explained in the text.

Matrix $\boldsymbol{A}$ has the dimension $[n \times m]$, where n is the number of active 2dMb cells, and m is the maximum possible number of 2dMb cells, whose cell centres lie within the area of a corresponding MODFLOW cell. The number of active 2dMb cells, n, is defined as the number of all 2dMb cells, which are not excluded from the domain, i.e. they are allowed to be wetted during the simulation. Not all of of them will be considered during the runs, because some of them will be dry, but the number of dry and wet cells will change dynamically. m is calculated from the following relationship:

$$m = (mult + 1)^2 \quad (5.1),$$

where *mult* is the whole-numbered multiple, of which the grid spacing of the MODFLOW length is larger than the one of the 2dMb cells. The line number in the matrix corresponds to the MODFLOW cell ID, the values in the columns are the cell IDs of the corresponding 2dMb, whose centre lie in the area of the MODFLOW cell. Depending of the geometrical orientation of the two grids, the number of those 2dMb cells per given MODFLOW cell can be variable, the remaining positions in the matrix are hence filled up with zeros. The number of non zero values in each line is therefore the number of 2dMb cells, which are related to the MODFLOW cell with the ID of the line number (see Fig. 5.1).

5.2. Mathematical Description of the River – Aquifer Exchange Process

The exchange flux between river and groundwater is calculated cell by cell using the mathematical formulation already described in § 4.4.7. It is the same as being used by the original version of the MODFLOW RIVER package, also differentiated between a connected and an unconnected aquifer. Because of their central relevance within the context of surface water – groundwater interaction in this work, they are rewritten here (eq. 5.2 and eq. 5.3, see § 4.4.7, p. 63, for the meaning of the single variables) and illustrated graphically as well (Fig. 5.2). In the connected case, the flux is directly proportional to the head difference between water level in the river and piezometric head in the corresponding groundwater cell. In the unconnected case, the infiltration flux is linearly proportional to the flow depth in the river. The correspondence between the geometry in reality and the discrete river river cells in MODFLOW is shown in Fig. 5.3.

$$Q_{exch} = \frac{K_{riv} L W}{M} (H_{riv} - h_{i,j,k}) \quad \forall \quad h_{i,j,k} > r_{bot} \quad \text{(5.2) (connected case)}$$

$$Q_{exch} = \frac{K_{riv} L W}{M} (h_{riv} - r_{bot}) \quad \forall \quad h_{i,j,k} \leq r_{bot} \quad \text{(5.3) (unconnected case)}$$

This approach for describing the surface water – groundwater interaction was selected for two reasons: This approach has been widely used and its numerical stability is known (see § 2.4.2); second, the data requirements for this approach is low considering just one leakage factor, which is compatible with the target of investigating larger scales, where local effects can be neglected, as well as with the data availability in this study.

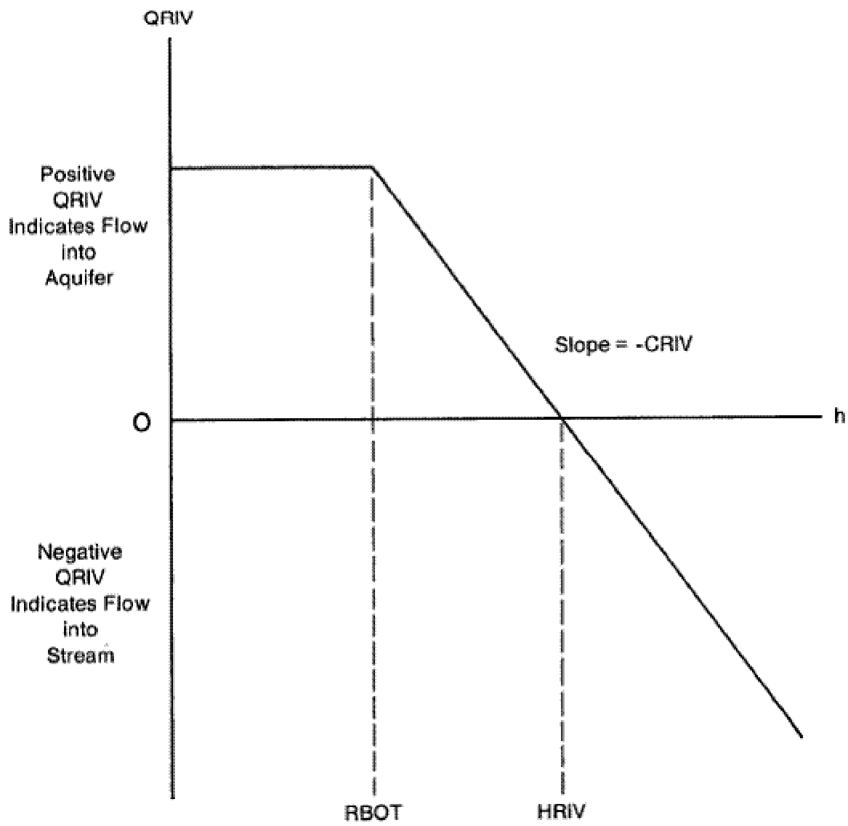


Fig. 5.2: Schematic of the river – aquifer interaction as implemented in the RIVER package (from: McDonald et Harbaugh, 1988).

Therefore, the piezometric head in the groundwater and the water level in the river must be known in each groundwater resp. surface water cell. Consequently, the exchange flux for each cell can be calculated and distributed over the corresponding cells of the complementary grid.

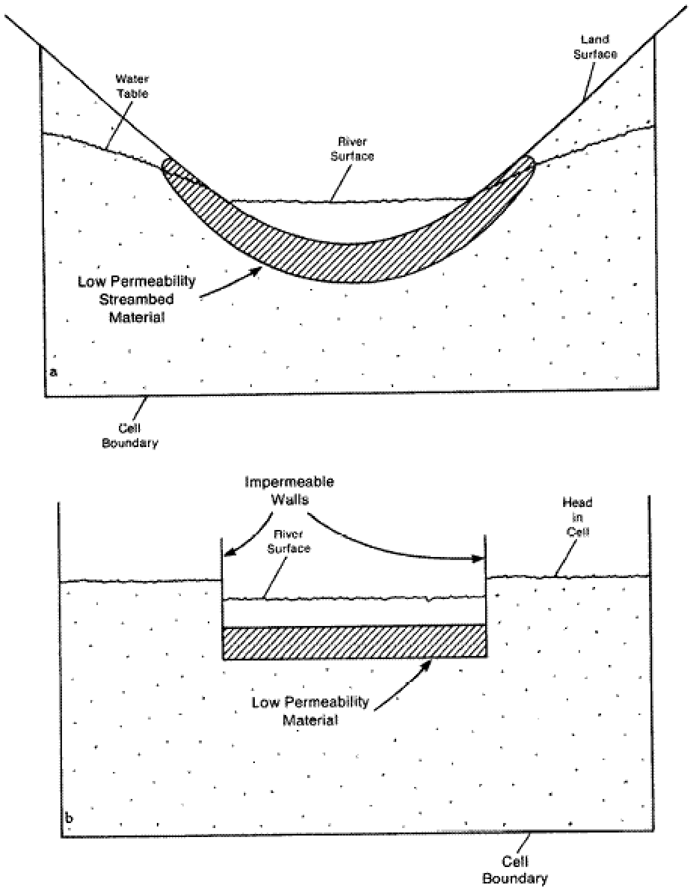


Fig. 5.3: Schematic of the river-aquifer interaction in reality and its correspondence in the discrete cells as implemented in the RIVER package (from: McDonald et Harbaugh, 1988).

5.3. Schemes of Surface Water – Groundwater Coupling (Theory)

A stepwise approach of implementing the surface water – groundwater coupling was chosen, starting from a simple approach, adding more complexity to reach the final implementation scheme. This procedure allows to gain insight into the characteristics and dynamics of the system and allows to adapt the strategy concerning the needs and relevant processes, while keeping the level of complexity of the coupling as low as possible. Furthermore, information about the numerical behaviour of the coupled system can be gained and interpreted.

Finally, four different schemes of coupling between the surface water and groundwater model have been developed, all of them using the same equations for the exchange flux (eq. (5.2) and (5.3)). They are compared in Tab. 5.1.

Tab. 5.1: Overview over the four different versions of implementation of coupling.

Version	Coupling Type	MODFLOW	2dMb	Description
a	external iterative coupling	steady-state	quasi-steady-state	MODFLOW (steady-state) and 2dMb (quasi-steady-state) are run separately, then exchange rates are calculated, and models re-run until convergence is reached. Heads are transfered from 2dMb to MODFLOW, MODFLOW calculates exchange rates using the RIVER package and transfers them to 2dMb.
b	full mass coupling	steady-state	quasi-steady-state or transient	MODFLOW (steady-state) and 2dMb coupled. Transfer of exchange rates from 2dMb to MODFLOW (using WELL package tool) and transfer of heads from MODFLOW to 2dMb
c	full mass coupling	steady-state	quasi-steady-state or transient	MODFLOW (steady-state) and 2dMb coupled. Transfer of heads from 2dMb to MODFLOW, MODFLOW calculates exchange rates using RIVER package and transfers them to 2dMb
d	full mass coupling	transient	transient	like version c , but using MODFLOW in transient way.

The simplest scheme (a) also being the first step was an external and iterative coupling between 2dMb and MODFLOW. Here, MODFLOW was run in steady-state mode, using the RIVER package, whilst 2dMb was run in quasi-steady-state conditions. After each run, information about heads and fluxes were exchanged, until a stable solution was reached (Foglia et al., 2005). This approach was intrinsically numerically stable, because no common equation of the two systems had to be solved.

The following steps (versions b and c) were the full mass coupling of 2dMb with the steady-state MODFLOW. This approach was meant to be an intermediate step before coupling both models in transient state. This approach is less complex and has the advantage of requiring only steady-state parameters for the groundwater model, and the coupled system to be calibrated without the use of those parameters, which are relevant only in the transient mode. This was considered a sufficient motivation for their implementation despite the fact that the combination of a transient model (even using quasi-steady-state conditions) and a steady-state model could give rise to some numerical problems. Although some of these effectively occurred, it is nevertheless worth to describe these two intermediate steps for the illustration of the numerical behaviour of the system.

The most complex scheme (version d), which corresponds also with the principal target of this work, was the full mass coupling between the transient 2dMb and the transient MODFLOW. This will be described in detail.

5.4. External and Iterative Coupling (a)

Here, the surface water model 2dMb and the groundwater model MODFLOW-2000 have been run separately, i.e. independently. The final result was reached using an iteration cycle until only minor changes in the results (wet cells, water level, groundwater head) occurred. This was usually the case after three iterations. Fig. 5.4 shows this procedure in detail.

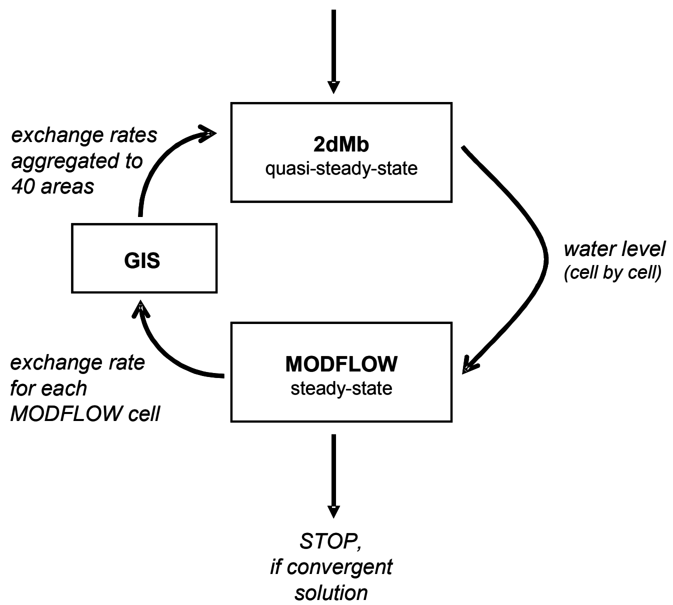


Fig. 5.4: Iteration scheme of the iterative coupling.

The iteration started with a quasi-steady run with boundary conditions of constant flow upstream and critical flow depth downstream, without any interaction between the two models. As soon as the simulations converged to a steady solution, the information of the water level of each 2dMb cell was transferred to the groundwater model. This information was used to create a RIVER file, which contains the location of any wet river cell and its corresponding river stage level. This RIVER file is the input for the RIVER package, which has already been included in MODFLOW for the interaction between surface water and groundwater (see § 4.4.7).[18] Then, the first MODFLOW simulation in steady-state mode was performed using the RIVER package. This package calculates the exchange rate in each MODFLOW cell, which is connected to a wet river cell (already described in more detail in § 4.4.7). The value depends on the difference in the piezometric heads in the river and the groundwater, and whether the aquifer is connected or disconnected from the river. These values are then used as new constant boundary conditions by the way of sink and source terms in 2dMb. Hereby, only mass transfer takes place, momentum transfer can be neglected considering the velocity of upwelling and downwelling water. Here a simplified approach was used by dividing the 2dMb domain into 40 zones of about the same size, whereas the exchange fluxes were spatially integrated and applied as sinks and sources within a single wet river cell in the centre of these zones. This is in contrast to the three other approaches, where a cell-by-cell transfer using the transfer matrixes as described above (§ 5.1) was applied in both ways, i.e. from the river to the aquifer and vice versa.[19]

Then, the second iteration cycle started. 2dMb was run with the same external boundary conditions as in the previous run, but now using the sink and source terms and therefore accounting for the interaction with the groundwater. This procedure has been repeated as long as the number and location of wet river cells became stable as well as the calculated exchange rates in MODFLOW and the heads in the surface and in the groundwater.

5.5. Full Mass Coupling Transient 2dMb with MODFLOW (Versions b, c, d)

5.5.1. Definition of Full Coupling

In this context, the term "full coupling" is used in contrast to the "external coupling" from version (a). It means that information of river heads and groundwater heads respectively exchange rates are transferred after a pre-defined time interval T_{exch}, which is also the time interval, after which MODFLOW is called resp. runs, if it is implemented in transient mode. This is performed on a grid-by-grid procedure from one model to the other. This also means that the equation system of the groundwater model and the surface water model are still solved separately. Only mass transfer using sink and source terms takes place as it will be further discussed in § 5.5.4. The exchange fluxes are

[18] In the original version of MODFLOW, the RIVER-file (therefore also the water level and the extent of the river) must be specified for the entire simulation beforehand. This is not possible if there are feedback mechanisms with the groundwater system. For this reason, this had to be adapted in this work. Here, this file has to be updated iteratively after each run of 2dMb.

[19] This simplified approach was used following the line adding stepwise complexity. The procedure using the transfer matrixes for passing the information from MODFLOW to 2dMb would methodologically be possible, but in this work, this step was omitted in order to directly implement version (b) next.

implemented in such a way that the conservation of mass is fulfilled, which is essential for long-term simulations.

5.5.2. Technical Considerations of Full Coupling

As both MODFLOW-2000 and 2dMb are written in FORTRAN77, they were combined into a single integrated FORTRAN code with 2dMb being the main programme and calling MODFLOW as a subroutine. The exchange of the river heads is in form of an external file in order to be compatible with the MODFLOW RIVER package, which is technically used.

Since MODFLOW runs in an implicit mode, the numerical time step can be the same as the scale of changes in the physical variables, whilst 2dMb must be run in very small time steps (less than 1 second) due to its numerical explicit scheme. The time step in 2dMb is not fixed, but it is being calculated after each time step again based on the maximum occurring flow velocity in any single cell therefore accounting for the CFL-condition. Both reasons led to the decision to use 2dMb as the steering programme. MODFLOW is called after any constant time, which is the chosen time step for the groundwater process, and therefore also for MODFLOW. Some variables are to be exchanged between the two models directly before and after the call of MODFLOW. The differences between the implemented versions of the coupling approach are described later in this chapter.

5.5.3. Grid-by-Grid Transfer

It has already been described in § 5.1 that the grids of 2dMb and MODFLOW-2000 are rotated against each other. The grid sizes are distinct between the two models, but are identical within each model. During the calculation, information between head and water fluxes (river – aquifer exchange rates) must be exchanged. This happens with the above mentioned transfer matrixes. Relevant for the interrelations between the two grids are the centres of the cells. Due to the regularity of the grids the maximum number of cells of the 2dMb cells, which lie within the area of a single MODFLOW cell is limited and can be calculated as described in § 5.1.

There is however the fact that the number and location of the *wet* 2dMb cells related to a given MODFLOW cell varies during the simulations. This has to be accounted for in calculating the wet areas of the MODFLOW cells, since the exchange of water flux is linearly proportional to the wet area A_{riv} (eq. 4.3 and 4.6). In order to reduce computing time, the wet area per MODFLOW cell is calculated by the number of cell mid-points of wet 2dMb, which lie in the given MODFLOW cell, multiplied by the cell size of a single 2dMb cell.

Special attention had to be paid to the fact that only a single value for the water level and river bed elevation had to be used for the MODFLOW cells within the river package. The first question arose, which is the representative bed level. Is it the average level of all 2dMb cells, regardless, if they are wet or not; should the average of all wet 2dMb cells be used; or is it favourable for numerical reasons to use the lowest value of all river bed values in order to avoid exfiltration into unconnected 2dMb cells.

Similar considerations had to be made about the representative water level in the river to be used for the RIVER package as a basis for the calculation of the exchange flux rates. The average of the water levels of the wet 2dMb cells could be used. Another approach would be averaging the flow depths of the wet cells and add this value to the representative bed level being used for the RIVER package.

Finally the question arose, whether some threshold values had to be introduced to avoid numerical instabilities or problems in the conservation of mass. As a consequence of the difference in heads between surface and groundwater, a certain flux was calculated using eq. 5.2 resp. 5.3. In the case of exfiltration, no problem occurs, the groundwater can be distributed onto all wet 2dMb cells equally. In case of infiltration, the situation is more complex. If infiltration rates are high and some 2dMb cells have a low water level, there might not be enough water available within this given cell to be used for infiltration, if infiltration flux rates are distributed equally over the wet 2dMb cells. As a result, conservation of mass cold not be fulfilled. On the other hand, it might not be possible to concentrate the entire infiltration flux related to a given MODFLOW cell to the single corresponding 2dMb cell with the highest flow depth for similar considerations as above for the cells with very small water depths. While concentrating the whole infiltration rate onto a single cell, even the one with the deepest water depth, the amount of water available for infiltration might not be sufficient. Therefore, it might be necessary to distribute the water over more than one cell.

Another issue is the question of allowing exfiltration of groundwater also into dry surface water cells, when the groundwater levels exceed the river bed level of dry 2dMb cells. This could theoretically be the case for MODFLOW cells, which are related to some wet and some dry cells. It could likewise also happen for MODFLOW cells, where all related 2dMb cells are dry. This would be the case in river channels where exfiltration occurs, but the most upper part would move upstream while groundwater level rises.

Finally, a compromise had to be found between the following three targets: 1. the procedure must be numerical stable, 2. should not introduce large errors, neither in disregard of the conservation of mass, nor in big systematic errors in over- or underestimating exchange rates, and 3. keeping the computational effort as small as possible. After some theoretical considerations as well as numerical tests and experiments, the coupling scheme was implemented as explained in the following sections.

5.5.4. Numerical Considerations of Full Coupling

A numerical scheme had to be found, which is adequate for the coupling of models requiring different spatial and temporal resolutions. Since on a long-term perspective, also long-term simulations should be performed with this modelling framework, the approach must be computationally efficient.

Therefore, a mass-coupling between surface water and groundwater has been chosen using source and sink terms, as already mentioned. This means that the exchange of vertical momentum is neglected. This can be justified for three reasons: First, the surface water model uses only depth-averaged variables and does therefore not consider vertical momentum. Second, the exchange rates with the aquifer per 2dMb cell are in the order of 10^{-5} m^3/s, whilst the horizontal runoff is between 0.5 m^3/s and 100 m^3/s. The disturbances due to these vertical exchange fluxes are therefore negligible and do not give rise to numerical instabilities. Third, this approach of mass coupling has been implemented in many other models (e.g. Matz et al., 2006) and have been well tested since then.

This implemented approach of mass coupling using sink and source terms has the advantage that both models can run in their adequate time-step and spatial resolution without overburdening the other model. Furthermore, the overall number of grid cells can be kept smaller and the solving scheme simpler than in any approach, which solves the equations for the surface water and groundwater in a single step. The final version (d) was proved to be numerically stable so that the applied approach is an adequate solution to the target problem.

To understand the temporal scheme of transfer of information between the two models, the different temporal scales of the involved processes have to be considered. The permeability in the aquifer is low with K-values generally in the order of 10^{-2} to 10^{-9} m/s. Groundwater flow velocities are therefore small, at least for unconfined aquifers. In confined aquifers, information can be transported with higher velocity due to the pressure transmission so that piezometric heads can vary much faster in this case. But still, variations of water table and flow velocities occur much faster in open channel flow. Whilst a time step of hours or days is enough for groundwater systems to capture the decisive processes, time steps in the order of several minutes are likely required in case of changing boundary conditions like the existence of a flood wave. The value itself depends on the slope, the dynamics and the size of the river as well as the spatial scale of the hydrological catchment. In case of the Maggia valley, observations suggest a time step of 10 minutes as being appropriate. This time step conditions also the time interval, after which the information of exchange, i.e. water levels and groundwater heads respectively infiltration and exfiltration rates have to be transmitted from one model to the other. This means that always after this time interval T_{exch}, MODFLOW is called from the steering programme 2dMb, whereas T_{exch} is kept constant during the simulation. The way it is implemented differs between the different versions (b), (c) and (d).

5.5.5. Differences in the Model Versions of Full Coupling

One point to consider is, which variables should be passed from 2dMb to MODFLOW and vice versa. Whilst within MODFLOW all variables have to be constant during T_{exch}, there is the possibility in 2dMb to upgrade each variable after each small time step of 2dMb (which is some orders smaller than T_{exch}), or they can be kept constant also over the entire time interval until the next call of MODFLOW.

Therefore, at least two principal possibilities exist for the full mass coupling: 1. The surface model 2dMb calculates the flow rates – based on the available information of the groundwater heads and the water levels in the river – for the time between to points of time, where the information of river – aquifer exchange is transfered. 2dMb passes then these flow rates to the groundwater model MODFLOW, whilst MODFLOW calculates the new head passing to the model 2dMb. This was the basic idea in version (b). 2. The exchange rates are calculated within MODFLOW, based on the information about groundwater heads and water levels in the river, and passed to 2dMb. This model calculates the new head in the surface water and passes this information to the groundwater model MODFLOW. This is the basic idea of version (c). However, also the original version of the RIVER package is based on this way of transferring the information between the river and the aquifer and vice versa, but without the possibility to update information during the simulation run.

Both, version (b) and version (c) are implemented with 2dMb in transient, and MODFLOW in steady-state mode. Version (d) is the final approach of the full coupling of 2dMb and MODFLOW both running in transient mode. The exchange scheme is otherwise the same as in version (c).

5.5.6. Full Coupling Transient 2dMb with Steady-State MODFLOW: Version (b)

Motivation

The approach of this version (b) was chosen, because it allows for the integration of the exchange rates over all small time steps of the surface water model. Since the physical processes in the surface water system are faster than in the groundwater, it should theoretically provide more accurate results from the physical point of view.

Technical description

In this version, information of exchange rates were passed from 2dMb to MODFLOW just directly before MODFLOW was called. This information was used as sinks and sources in each MODFLOW grid cell, where some exchange flux occurred concerning the results from 2dMb. For this, the WELL package technique of MODFLOW was used. Then MODFLOW was run in steady-state mode and provided a steady-state solution, including a new distribution of groundwater heads cell-by-cell. These heads were then passed back to 2dMb. Then 2dMb was running in small time steps over a period of T_{exch}, which is defined as the constant time interval, after which MODFLOW is called again. After each small time step in 2dMb, the model calculated the exchange rate in each single 2dMb cell and then integrated over all cells which contribute to a corresponding MODFLOW-cell. At the end of the time interval T_{exch}, all exchange rates corresponding to each MODFLOW cell were then integrated over time. These were the values to be passed to MODFLOW when it is called after the end of T_{exch}, and the cycle starts again. This procedure is shown schematically in Fig. 5.5.

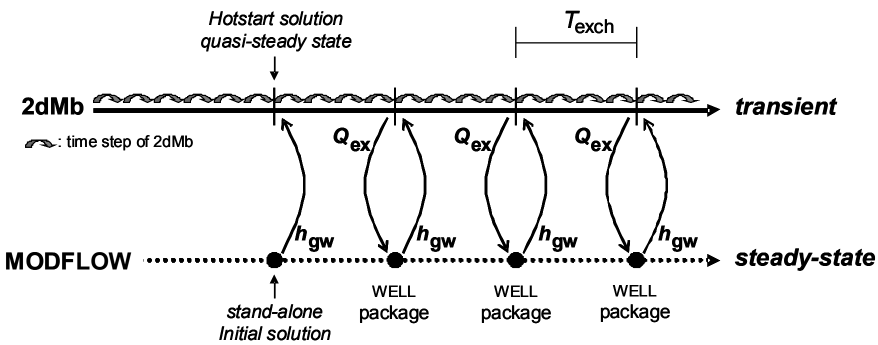


Fig. 5.5: Numerical time scheme of version (b).

The exchange rate Q_{exch} for a given MODFLOW cell [$\mathbf{L}^3$ $\mathbf{T}^{-1}$] can hereby be formulated as followed:

$$Q_{ex} = \sum_{\Omega} \sum_{T} q_{ex}(h_{gw}, h_{riv}) T_{exch} \Delta \Omega_{MODFLOW-cell} \quad (5.4),$$

where q_{ex} is the exchange rate of a single 2dMb cell corresponding to the MODFLOW cell [$\mathbf{L}^3$ $\mathbf{T}^{-1}$], h_{gw} is the groundwater head [$\mathbf{L}$], h_{riv} is the water level in the river [$\mathbf{L}$], T_{exch} is the time interval between the transfer of information between the two models [$\mathbf{T}$], Ω is the space domain and T is the time domain.

Remarks

2dMb runs numerically speaking always in a transient mode (even, when it is running in the so-called quasi-steady-state mode), but MODFLOW here in steady-state. Despite this inconsistency, it was not able to foresee a priori, whether this would lead to some numerical instabilities. It was however worth to try for two reasons: First, it is a way to stepwise increase complexity, and second, it allows a stepwise calibration of the modelling system, because transient MODFLOW parameters do not need to be considered. Additionally, it would have been a fast way to produce steady-state solutions for the groundwater table within the coupled system without waiting for the equilibrium state of the entire coupled system to occur.

A first implementation on a small area, where only the condition of a connected aquifer occurred, led to plausible results. When applying it to the entire main valley, the simulation results showed however strong oscillations of the groundwater table[20]. An analysis into the depth of the processes within a single cell showed that the reason lay in the formulation of the interaction terms. Different equations for the exchange rates are used for the connected and the unconnected case, which are continuous with respect to the groundwater table, but which are not continuously derivable. The latter fact caused these numerical instabilities.

5.5.7. Full Coupling Transient 2dMb with Steady-State MODFLOW: Version (c)

Motivation

Here in version (c), the second possibility of implementing the information of exchange variables, i.e. providing heads in 2dMb and exchange rates in MODFLOW, was tested for similar reasons as version (b). For the following reasons it was expected to run in a more stable way: 1. Less dynamics give rise to less numerical instabilities, 2. problems of partly wet cells in the surface water model can easier be handled, 3. the RIVER package in MODFLOW, which has already be proved to be numerically stable, could be used.

Technical description

Fig. 5.6 illustrates, how the coupling was implemented. It is quite comparable with the external and iterative coupling with respect to the time handling and the kind of variables, which are passed from one model to the other. 2dMb calculates the distribution of water level in the river (hydraulic river heads) at the point in time, when MODFLOW is called, and passes them cell-by-cell to the corresponding MODFLOW-cells. The information of river heads is written in an external ASCII-file, which is the input file for the MODFLOW RIVER package. MODFLOW calculates then the steady-state solution for the groundwater heads and the exchange rates using the RIVER package. The exchange rates are then distributed to all river cells, which were wet at the end of the last previous time step of 2dMb. These exchange rates are kept constant over

[20] The solution for the groundwater table alternated between a realistic solution and a depth-do-groundwater of a few hundred meters.

the whole time interval until the next call of MODFLOW – and therefore exchange of information – takes place.

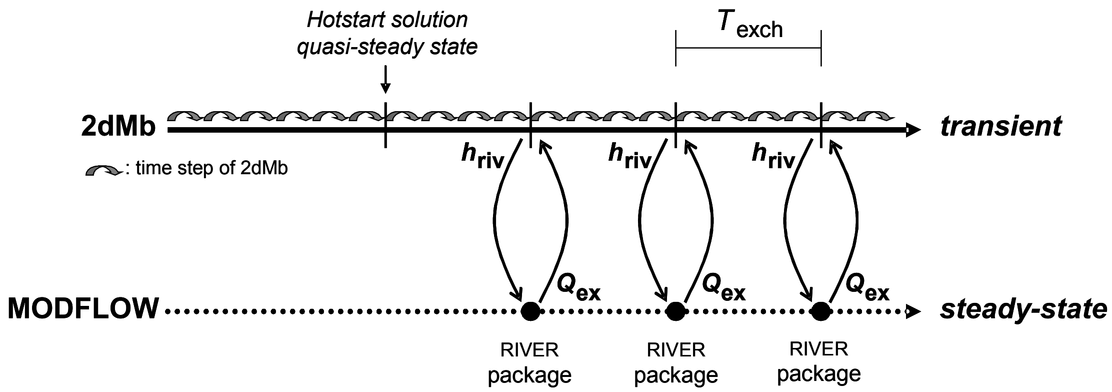


Fig. 5.6: Numerical time scheme of exchange for version (c).

Remarks

Similarly to the results of version (b), numerical instabilities were observed in version (c) as well. Here however, the oscillations were only in the range of a few meters and hence always in a plausible range, but there was still no convergent solution. In this case, no detailed investigation up to the detail of any single cell has so far been carried out, as it was the case for version (b). Nevertheless, varying exchange rates as boundary conditions were less problematic for the steady-state MODFLOW, probably because the problem of applying different equations depending on the groundwater and river heads (connected and unconnected aquifer) does not occur as it is the case in version (b). The problem however remains that small disturbances in the system effect immediately the overalls system under steady-state conditions and an implicit scheme, as it was the case for MODFLOW. This leads to the conclusion that the explicit transient scheme of the surface water model and the implicit steady-state scheme of the groundwater model are not compatible with each other leading to unstable numerical solutions. This fact led directly to implementation of version (d) as described in the next section.

5.5.8. Full Coupling Transient 2dMb with Transient MODFLOW: Version (d)

Motivation

The approach of the full coupling of 2dMb and MODFLOW both running in transient mode was the last step in the different implementations of coupling, and also the final goal of this thesis.

As already discussed, this version solves the partial differential equations of the two systems separately, using exchange of mass between the two systems. This allows to fully simulate feedback mechanisms between surface water and groundwater, which are provoked by the exchange of water between these two systems. Because of its transient mode it is able to mimic single flood events, i.e. hydrographs, which are characterized by fast changes in streamflow showing therefore a strong unsteady behaviour. All processes within this coupled model are simulated as time-dependent.

Insight into the processes of the system are necessary for the better understanding of the interaction between river and groundwater and the strong influence by the regulation of the hydropower system, in addition to the effect of hillslope fluxes into the aquifer. The results of this coupled system will act as essential information or as a boundary condition for the vegetation growth model.

Technical description

The implementation scheme is practically the same as in version (c) with one essential difference: here, also MODFLOW runs in transient, not in steady-state mode. The scheme is illustrated in Fig. 5.7 and in a flow chart in Fig. 5.8.

The time step of 2dMb is variable during its run, and adapted concerning the CFL-condition. The time step of MODFLOW can be chosen as input, but it should be done in such a way that the physical processes, which are responsible for the rate of changes of the hydraulic groundwater variables, are coherent with the time steps. The time interval T_{exch}, where exchange of information between 2dMb and MODFLOW takes place, is identical with the transient time step of MODFLOW. T_{exch} should be chosen such that numerical problems in the coupling can be avoided, that the solution is still sufficiently accurate and that it is adapted to the exchange process. In our case, the exchange processes were assumed to be as fast as the surface water processes, so that T_{exch} was chosen with respect to changes in surface water heads[21]. It turned out that this is also appropriate under numerical considerations. The value of T_{exch} is kept constant throughout the entire simulation.

As already mentioned, 2dMb is the main programme steering the entire simulation also calling MODFLOW at a certain time. 2dMb runs with irregular time steps until T_{exch} is exceeded. In the following time period, the small time difference between the small exceedance difference is accounted for and balanced so that no systematic error between the two models in defining the time intervals occurs.

---

[21] In the Maggia case study, a value of 10 minutes was assigned to T_{exch}.

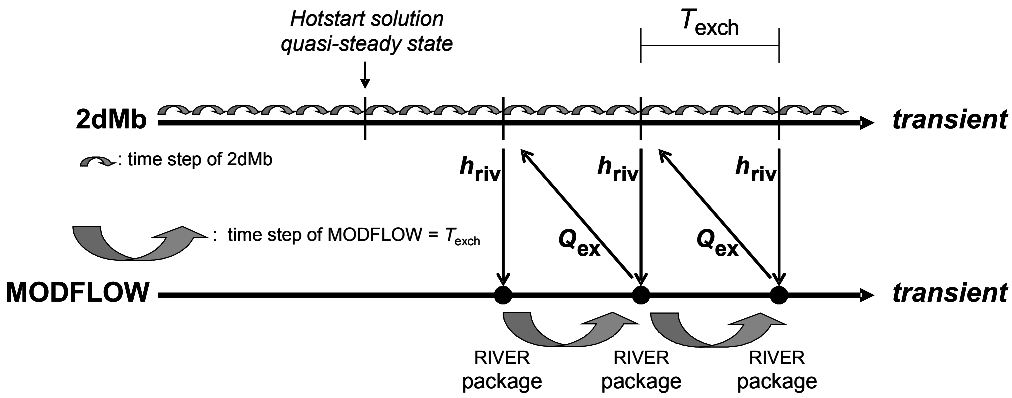


Fig. 5.7: Numerical time scheme of exchange for version (d).

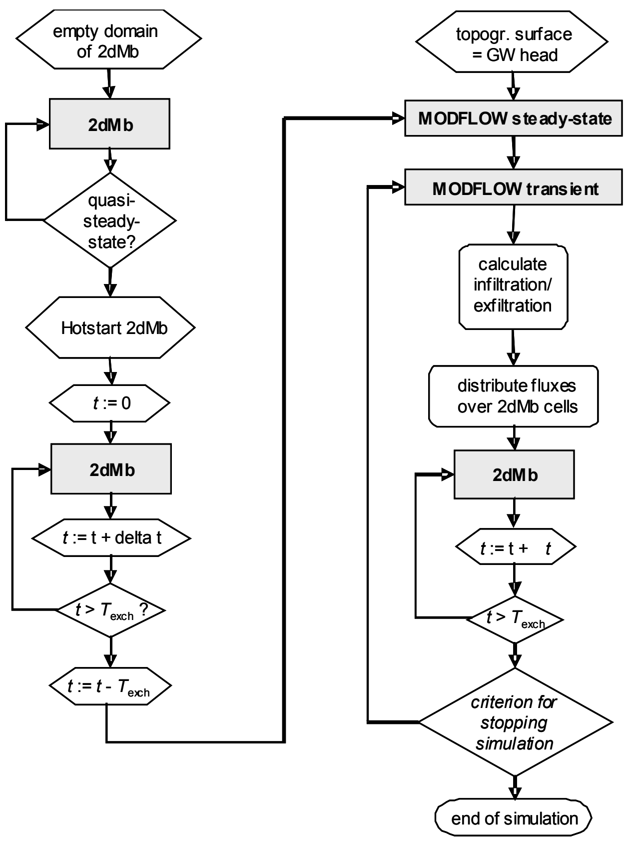


Fig. 5.8: Flowchart for the full coupling 2dMb with transient MODFLOW (version d).

Then the river heads, which have continuously been calculated in 2dMb are determined in each 2dMb cell. Using the transfer procedure from the surface water to the groundwater grid (described in § 5.1), the last river head information, i.e. at T_{exch}, is passed as input file to MODFLOW using the RIVER package interface. Then MODFLOW is called and runs for one MODFLOW time step in transient mode. The new groundwater heads are obtained as a result, as well as the exchange rates for all MODFLOW cells, which are listed in the actual RIVER package input file, i.e. all cells, which are connected to at least one wet surface water cell. The equations behind (using the RIVER package technique) are again eq. 4.3 and 4.6, which have already been explained in § 4.4.7. Therefore, the wetted area, and also the area, where exchange between the surface water and groundwater can occur, will be changed only during the 2dMb runs.

The procedure for the transfer of the information on exchange rates from the MODFLOW cells to the 2dMb cells has already been described in § 5.1. The wet 2dMb cells were kept in memory, therefore the distribution of the exchange rates is only into the wet corresponding 2dMb cells, but not into dry corresponding 2dMb cells. The exchange flux for each given MODFLOW cell, which takes part in the interaction process, is divided by the number of corresponding wet 2dMb cells and is then equally distributed over these cells.

At the end of a single transient MODFLOW run of one time step, MODFLOW is closed again and the following variables are passed to 2dMB: 1. Matrix with the exchange rates for any single 2dMb cell (zero, if 2dMb cell is dry or not active), 2. Matrix of groundwater head for any single MODFLOW cell, which has to be kept in memory, since it will be passed back again to MODFLOW at the following call as an initial condition. This is necessary for the transient simulation. In case of the steady-state simulation, the initial conditions do not affect the result as long as the values are not too far from the final solution and the numeric procedure converges (there, the surface topography has been used as an initial condition, as already described in § 4.4.4).

Then the whole cycle starts again. 2dMb proceeds exactly at the point, where it was interrupted by the call of MODFLOW. All the variables are still in the active memory, no initial conditions have hence to be defined. Only the internal boundary conditions have changed: The new matrix of the exfiltration rates has to be added to the existing sink and source terms, which are independent from the infiltration – exfiltration process. To avoid conflicts and numerical instability problems, the exfiltration rates at the boundary of the 2dMb are set to zero and therefore neglected, which can easily be justified considering the big number of cells in the entire domain.

Boundary and initial conditions

Before running the two models in the coupled mode, a hotstart file is produced to overcome the inertia effects of the system, and used as initial conditions for the simulation period of interest (see also Fig. 5.7). This file represents quasi-steady-state conditions of the surface water system. This solution is already relatively close to the one, which would be reached under quasi-steady-state conditions in the coupled model with the exception that changing discharge rates along the longitudinal profile due to infiltration and exfiltration processes are not yet considered. This file will then be used as initial condition for the coupled model run.

No such hotstart file has to be produced for MODFLOW, since it is suggested to start always with one steady-state simulation before running it in unsteady mode (Harbaugh et al., 2000). This was also implemented in this way in the coupled model. Thus, 2dMb starts with a hotstart file as initial condition, which represents the quasi-steady-state solution for a discharge rate, which is similar to the initial flow rate to be simulated at the inflow boundary. After the time interval T_{exch}, MODFLOW is called for the first time. It runs first in steady-state mode, using the surface topography as initial groundwater heads and the water level from 2dMb as boundary conditions except the constant head boundary at the upper and lower bound of the modelling domain. This procedure guarantees an immediate steady-state solution without having to wait a long time to reach the equilibrium state. Immediately afterwards, MODFLOW runs in transient mode, and its solution is stored. Then alternately 2dMb runs for a time interval of T_{exch}, MODFLOW is called again using the results from the previous transient run as initial condition and runs for one time step in transient mode. In this way, an equilibrium state will be reached under the premise that constant discharge rates are used. Then, all inertial effects of the system have been overcome, and the true simulation can start.

Time steps

The time interval between the points in time of exchanging variables is kept constant. MODFLOW runs in transient mode for one single time step, which is equal to T_{exch}, before reverting to 2dMb. The time steps in 2dMb can be specified in different ways. The standard is to calculate the maximum allowed step in each single cell respecting the CFL-condition and to multiply it with the numerical variable *cfl*, which corresponds to the constant *C* (Courant number) in eq. 4.8 and is further described in § 6.2.2. It must always be between 0 and 1. The larger *cfl*, the larger the time step for the succeeding calculation step. If it is too close to one, the risk is to overshoot, therefore not respecting the CFL-condition in the succeeding calculation step any more, and so being the source for possible numerical instabilities. For the optimization of the computation time, a dynamic adaption of this numerical parameter *cfl* has been made as described in § 6.2.2.

Threshold values for water depth in 2dMb cells

Wetting and drying is a critical point in the open-channel flow simulation, if the inundated area does not remain constant during the model run. The need of specifying the threshold parameters h_{dry} and h_{wet} has already been discussed in § 4.5.5. The value of h_{dry} is the threshold, below which a cell is considered to be dry and taken off from the active cells for the subsequent time step. Generally, this value should be small in the order of only a few centimetres or in the order of the river bed roughness elements. Experiences with the model showed that under low flow conditions, values between

0.02 and 0.04 m are suitable, but they must be higher (around 0.07–0.12 m) under high flooding conditions. This was found out by many different test runs using the domain of the entire valley. In order to avoid too strong oscillations between drying and re-wetting of cells again, the threshold of wetting, h_{wet}, is set in the model to one centimetre above h_{dry}.

When coupling the surface water model with the groundwater model, two additional water depth thresholds must be defined. One is the water depth $h_{exchange}$, above which a river cell is considered to take actively part in the river – aquifer interaction process. Physically it makes sense to define a certain water depth which is high enough to overcome the initial resistance of the colmation layer in the river bed. Besides this, this has also advantages for numerical reasons in order to stabilize the exchange process (dampening) and hence to avoid oscillations.

The other threshold concerns the minimum water depth h_{limit} in a 2dMb cell, under which no water should be taken for infiltration into the groundwater. Generally, not all 2dMb cells, which are related to a specific MODFLOW cell, have the same water depth. Infiltration rates - if any infiltration occurs – are calculated by MODFLOW on the basis of the MODFLOW cells. If divided by all wet corresponding 2dMb cells, it can happen that not all cells have enough water to release. In this case, infiltration had to be limited to the available water (above h_{dry}) in each single cell. To overcome this problem, only 2dMb cells with a water depth larger or equal than h_{limit} are considered for infiltration. The infiltration rate is divided by the number of such related 2dMb cells, whose water level is larger or equal h_{limit}, and then infiltration is applied only to these cells.

There is hence a relation between the magnitude of the different water level threshold parameters as follows:

$$h_{dry} < h_{wet} \ \wedge \ h_{dry} \leq h_{exchange} \leq h_{limit} \quad (5.5)$$

h_{dry} is the lowest of these values, h_{wet} must be slightly higher. $h_{exchange}$ must be at least equal or larger than h_{dry}, but preferably larger than h_{wet} to avoid numerical instabilities. If $h_{exchange}$ is already high enough to allow for the release of infiltration water, h_{limit} can have the same value, otherwise it must be higher. In case that h_{limit} is chosen too low, water will be taken for infiltration only until h_{dry} is reached, therefore a small inaccuracy in the water balance would occur.

6. Modelling Framework Implementation

The general performance of the individual models had to be proved before applying and implementing them in the modelling framework. They were first tested as stand-alone models. MODFLOW has been widely used worldwide and has proved its model capabilities. TOPKAPI has also been used in different studies, and it has been adapted by *Birsan, Ciarapica et Foglia*, and its results are given as an input for this work (Foglia, 2006). Since its implementation and performance is well documented there, no further testing is shown here. Since the model implementation and results from the groundwater model – already linked with the watershed model – were taken from previous work by Foglia (2006), this part is described only to such an extent as is important for the understanding of the calibration strategy and the interpretation of the results.

A few numerical adaptations were made for the optimization of the 2dMb model with respect to the high variation of discharge to be modelled (§ 6.2). Afterwards, a calibration and validation strategy had to be worked out (§ 6.3.1). As a first step, the groundwater model (with lateral recharge supplied by TOPKAPI) (§ 6.3.3) and the surface water model (§ 6.3.4) were calibrated and validated separately. Based on these results, the coupled model was implemented and calibrated resp. validated (see §).

6.1. Testing of 2dMb Model

2dMb has been used in several project related studies (Vetsch and Fäh, 2003; Fäh and Bezzola, 2004; Faeh, 2007), but it has not yet been well documented in the literature. Because of this and the likely complex conditions of natural streams the model 2dMb was tested for its robustness and behaviour under certain conditions, which resemble typical cases found in natural streams. This was done carrying out different numerical experiments mainly in a test channel, but also in a part of the natural system of the braided area.

Before applying the hydrodynamic model in the real case, the model was tested in two ways. First, numerical experiments were performed in a simple test channel in order to prove its capability and behaviour under several conditions, which occur also in natural stream beds. The aim was to test its robustness, especially for strong gradients in slope and roughness, the accuracy of its results with respect to analytical solutions for uniform flow conditions, its behaviour in case of hydraulic jumps, and its sensitivity to different boundary conditions and grid spacing, which had to be applied in the real case. Second, tests with different grid resolutions were carried out in a selected part of the braided area in order to find out the optimum between numerical stability, performance of the model and computational effort.

6.1.1. Design and Results of Numerical Experiments in the Test Channel

In order to evaluate the model behaviour, the following structures and parameters have been tested in the test channel by carrying out numerical experiments: 1. quasi-steady and unsteady flow (inundation), 2. boundary conditions, 3. roughness variation, 4. weir overflow (weir in the middle of the channel as an obstacle creating rapidly varied flow). A selection of these results are shown within this section.

Quasi-steady and unsteady flow

The model simulates unsteady flow. If the boundary conditions are kept constant, the solution approaches the steady-state solution with time. In order to show how this solution is reached, and if the quasi-steady numerical solution is in agreement with the analytical solution, the following numerical experiments were carried out in the test channel under subcritical and supercritical flow conditions: The inclined channel (4 ‰ resp. 1 %) with a length of 2000 m, a width of 100 m and a Nikuradse roughness of 0.10 m was initially dry. The parameters were chosen to mimic the conditions occurring in the real case where average slope is between 0.5 ‰ and 2 % and roughness values of 0.13 m were finally used for non-vegetated areas. The different slopes lead to subcritical and supercritical flow conditions. A constant flow boundary of 680 m^3/s was used at the upper boundary, which leads on the one hand to the change between subcritical and supercritical flow given the different slopes, and which is on the other hand in the order of the high flood events occurring in the Maggia valley. A critical flow boundary condition, i.e. a weir with a weir crest height of 1.5 m was used at the lower boundary. Fig. 6.1 and Fig. 6.2 illustrate the propagation of the wave front until it reaches the weir under subcritical and supercritical flow in the channel. Then, this has a backwater effect and increases the water level backward until a steady-state solution is reached. This is a good example of how the finite volume solving scheme works and how it accurately follows true flow conditions.

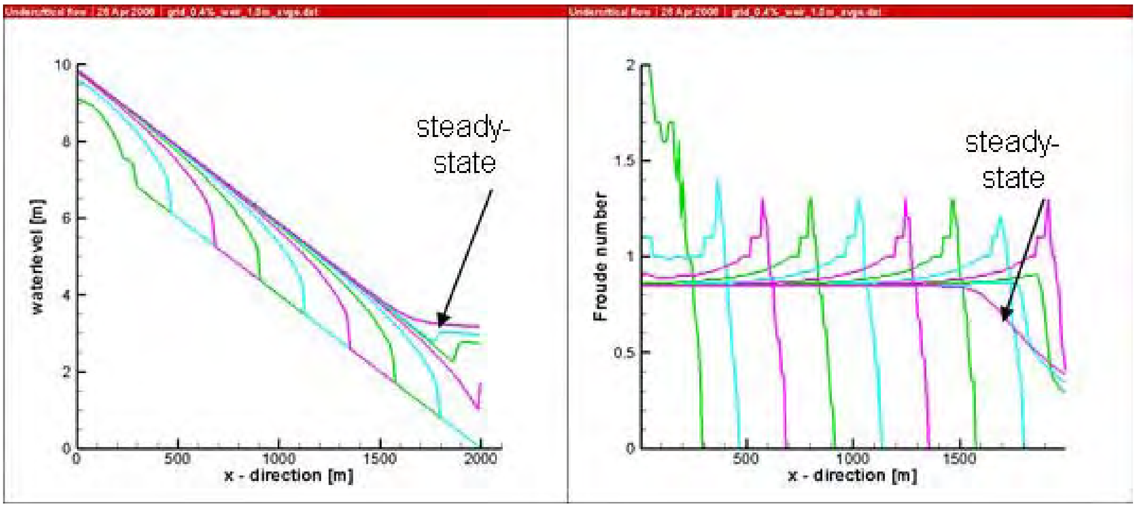


Fig. 6.1: Simulation of the propagation of a wave and the backwater effect of a weir under subcritical flow (slope = 4 ‰): water level (left) and Froude number (right) (from: Ranieri, 2006). Different colours represent the solution after different points in time.

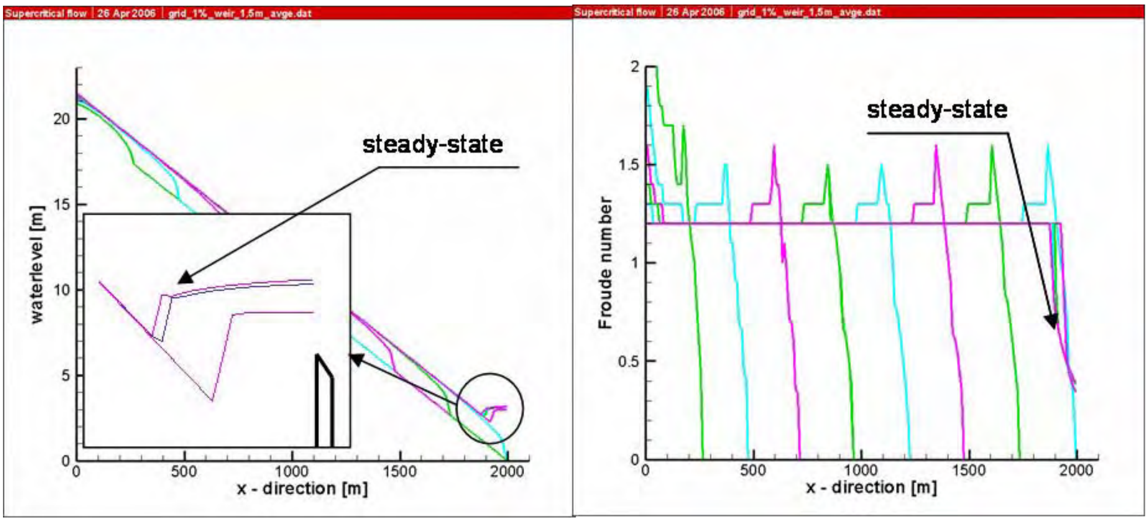


Fig. 6.2: Simulation of the propagation of a wave and the backwater effect of a weir under supercritical flow (slope = 1 %): water level (left) and Froude number (right) (from: Ranieri, 2006). Different colours represent the solution after different points in time.

Boundary conditions

In this part, the influence of the boundary condition onto the flow field is shown. It was investigated, how far upstream the water level is influenced by a weir implemented as a lower boundary condition, using different weir heights and two different flow rates.

Altogether 12 simulations were carried out, using the flow rates of 680 m^3/s and 1000 m^3/s and varying the weir crest height from 0 m to 2 m. The results are plotted in a dimensionless form, dividing both the distance from the weir and the corresponding flow depth by the uniform flow depth for the given discharge (Fig. 6.3). It is obvious that the length of the backward influence of the weir increases with its crest height. The pairs of curves with the same value for sf/Y_u (weir crest height *sf* divided by the uniform flow depth Y_u) are practically identical, which means that the length of influence does not depend on flow rate, given a specific sf/Y_u. This shows that the weir boundary condition produces no numerical artifacts, but reproduces physically plausible flow behaviour. It shows furthermore that the spatial influence of the weir boundary condition is limited to its neighbourhood and does not numerically effect the overall solution in the entire modelling domain.

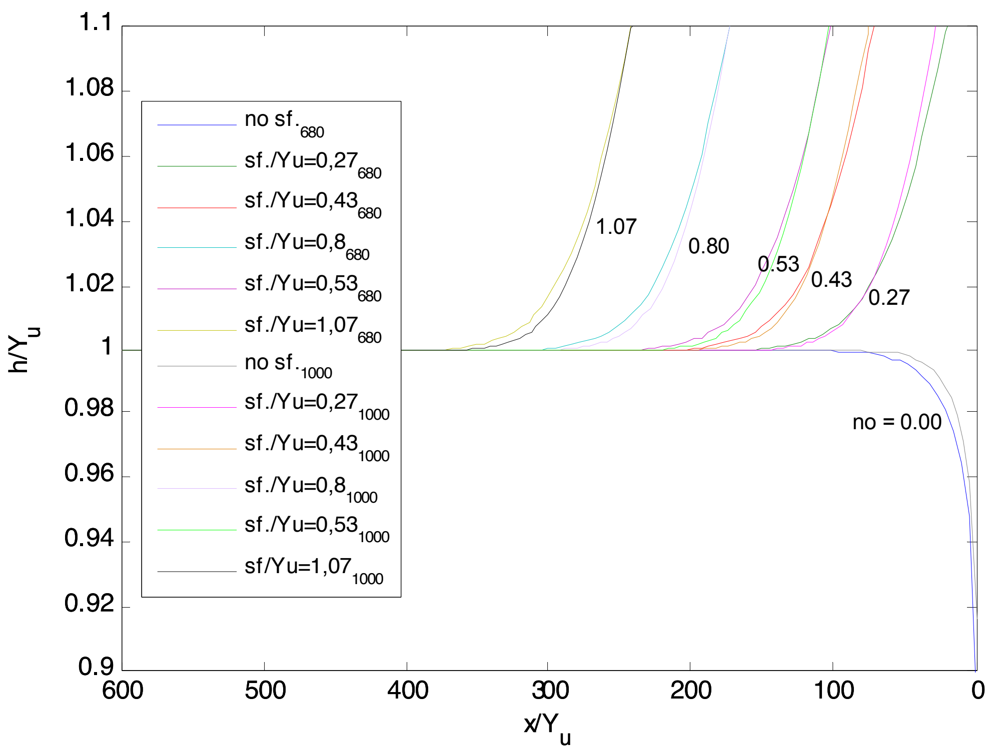


Fig. 6.3: Length of the influence of different downstream boundary conditions in a test channel under two different flow rates (from: Ranieri, 2006). h is equal to the weir crest height sf.

Roughness variation

Sudden changes in bed roughness could produce numerical instabilities because of resulting sudden changes in flow depth and velocity, but especially in case of changes between supercritical and subcritical flow conditions or vice versa.

These conditions occur also in the real case because of the delineation of the modelling area into different roughness zones with substantially distinct roughness values. The model had to be tested for its behaviour under these circumstances. For this purpose, the bed roughness in the test channel was divided into different zones with sudden changes in roughness values along the longitudinal profile. As a result, flow conditions changed from upstream subcritical over supercritical flow back again to subcritical flow conditions. As it is shown in Fig. 6.4, the model converged to numerically stable steady-state and analytically correct solutions.

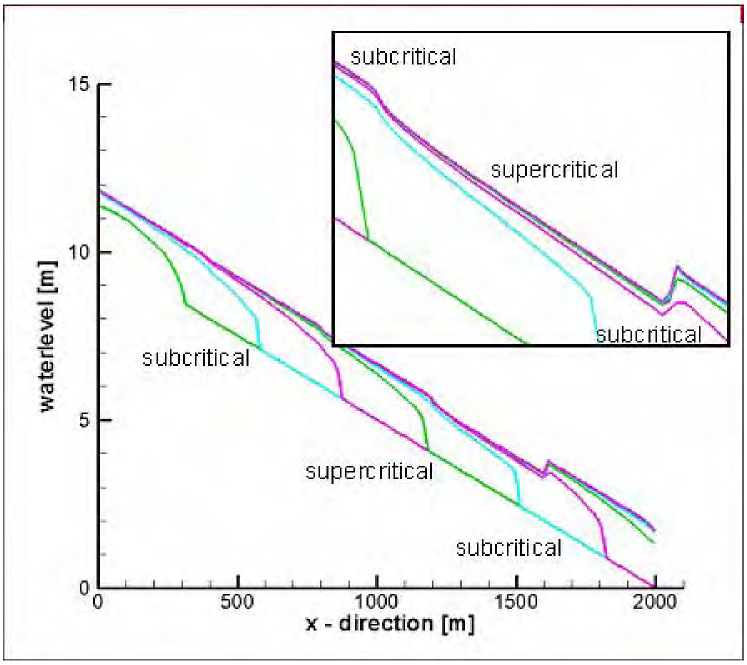


Fig. 6.4: Illustration of sudden roughness changes in a test channel with piecewise uniform flow and the transition between subcritical and supercritical flow conditions and vice versa (from: Ranieri, 2006).
Different colours represent the solution after different points in time.

Weir overflow

In order to test the capability of the model to handle sharp gradients in the river bed and hydraulic jumps as they occur in reality, an obstacle in the form of a sharp crested weir was implemented in the middle of the test channel. The model was capable to simulate this geometry without problems while simulating a hydraulic jump where it was expected from the physical point of view. This is a further proof for the numerical robustness of the model and its applicability under natural conditions with frequent hydraulic jumps possible.

6.1.2. Conclusion of the Numerical Experiments in the Test Channel

The aim of the test channel experiments was to investigate the model properties under different conditions such as found under natural conditions. This knowledge is essential when interpreting simulation results in a real case study. It can be stated that the model is robust under different geometrical and flow conditions and also in the transition from subcritical to supercritical flow and vice versa. The results agree well with the physical steady-state solutions of the shallow water equations[22]. Additionally, the model shows also the general behaviour of wave propagation as a response to sudden changes in roughness and in river bed geometry (also such as sudden flooding of forelands) in the unsteady case. All tests gave the confidence that the model fulfils all requirements to be applied in natural environments such as the Maggia valley with all its prerequisites formulated above. These are basically relatively steep slopes, partly sharp geometric gradients and an extreme range in streamflow conditions to be modelled.

6.1.3. Test for Appropriate Grid Resolution Size in Maggia Valley

There is always an optimal grid size to be used in numerical simulations. On the one hand, the grid size should be large to reduce computational time. If the grid resolution is too large however, numerical problems can occur. Even if the solution is stable, it could converge to a wrong solution. For this purpose, numerical experiments were carried out in a cutout of the braided area to find out the best grid size for the implementation in the entire Maggia valley. One limitation was found under low flow conditions. If the grid is too coarse, the river course is no longer continuous for numerical reasons (because of the h_{dry} limit), thus the river "dries out". The optimal value was found to be in the range of approx. 8-12 m, depending on the setting of numerical parameters and flow rates. Therefore, a grid size of 6.25 m was selected, since it is just a fraction ¼ of 25 m, which is the grid size of the groundwater model.

The model was implemented using different grids of approx. 3, 5, 6, 8, and 12 m in order to test, whether the choice of 6.25 m is suitable, or numerically more precisely, whether the results of the different implementations are similar. For this purpose and in addition to a visual inspection of the results, frequency distributions of the results for the different model implementations were compared for various parameters. Fig. 6.5 illustrates e.g. for the flow depth that the distribution for the grid of 6.25 m is in good conformance with the results for the finer grids, but shows some deviations from the

[22] This is not explicitly shown here. Different combinations of slope, roughness, discharge and different weir crest heights at the lower boundary were numerically tested on a long test channel with a very fine resolution grid, and the simulated results were compared with the calculated values for uniform flow conditions.

results of the coarser grid. This confirms the choice of the grid size of 6.25 m to be appropriate for the use in the main valley.

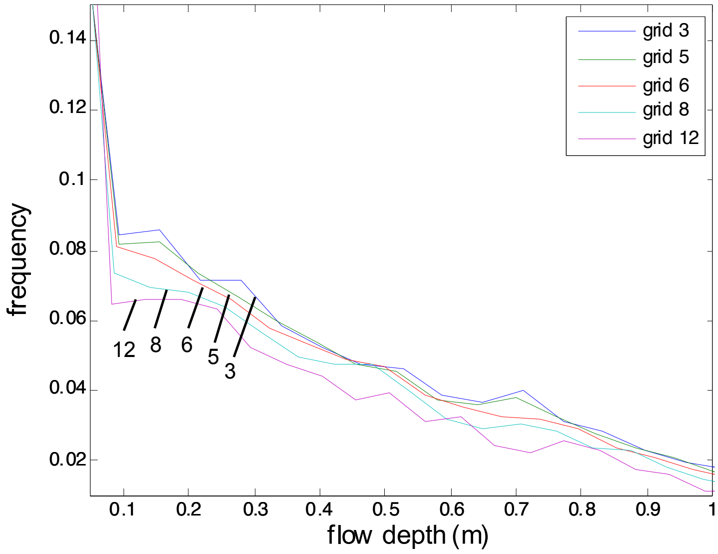


Fig. 6.5: Frequency distribution of flow depth in the braided area for different grid sizes (3 m to 12 m) using a constant Nikuradse sand roughness of 0.13 m (from: Ranieri, 2006).

6.2. 2dMb Model Adaptations

Two numerical adaptations have been performed for reasons of numerical stability under low flow conditions and of computing time: dynamic adaption of the drying limit parameter h_{dry} and of the *cfl*-parameter for the calculation of the time steps. Both were originally implemented to be constant, but were now related to streamflow.

6.2.1. Drying Limit Parameter h_{dry}

It was found out that the numerical parameter h_{dry} (see § 4.5.5) has an influence, whether the numerical drying out of the river occurs or not. If discharge and therefore flow depth becomes too small (in the order of 10 cm), some cells become inactive and the flow in the river is interrupted. As a consequence, the downstream part of the river falls dry. The experience was that very high flow rates require higher values for h_{dry} than lower ones. In order to simulate historical hydrographs ranging from 1 to 1000 m^3/s, h_{dry} is linearly adapted to the discharge and now calculated after each time step as followed:

$$h_{dry} = h_0 + \frac{h_0 \cdot h_{mult} - h_0}{1000\, m^3/s - 1.2\, m^3/s} \cdot (Q_{\text{inflow}} - 1.2\, m^3/s) \quad (6.1),$$

where h_0 and h_{mult} are parameters to be specified as input (in the Maggia valley, $h_0 = 0.4$ and $h_{mult} = 3.0$ was found to be suitable; and Q_{inflow} being the inflow discharge at the upper boundary (in the Maggia Valley: in Bignasco). The values of 1.2 m^3/s and 1000 m^3/s stem from the specific case in the Maggia Valley and represent the expected flow ranges to be simulated. They should be adapted for using the model at other sites of investigation.

6.2.2. Time Step Relevant *cfl*-Parameter

Experience acquired through many test runs showed that high values of *cfl*, which lead still to numerically robust solutions, are not optimal in the sense of computational time (see § 5.5.8). The higher the streamflow in the modelling domain, the smaller *cfl* should be chosen. A possible explanation for this phenomenon could be that in the case of high flow rates the likelihood is much higher that – as a result of the underlying numerical scheme, not for physical reasons – just in a single cell a very high velocity occurs, which strongly restricts the following time step. Using smaller values for *cfl*, this numerical risk might be reduced. Therefore, an algorithm was implemented to adapt the variable *cfl* after each computational step in such a way that it can continuously vary between given limits, but is higher for low discharge, and lower for high discharge.

$$cfl = cfl_1 + \frac{cfl_2 - cfl_1}{1000\,m^3/s - 1.2\,m^3/s} \cdot (Q_{\text{inflow}} - 1.2\,m^3/s) \quad (6.2),$$

where cfl_1 and cfl_2 are parameters to be specified as input (in the Maggia valley, $cfl_1 = 0.4$ and $cfl_2 = 0.25$ was found to be suitable; and Q_{inflow} being the inflow discharge at the upper boundary (in the Maggia Valley: in Bignasco). The values of 1.2 m^3/s and 1000 m^3/s stem from the specific case in the Maggia Valley and represent the expected flow ranges to be simulated. They should be adapted for using the model in other investigation sites.

6.3. Implementation, Calibration and Validation of the Modelling Framework

6.3.1. Overall Strategy

According to the scope of this thesis, the following basic steps had to be accomplished:

- Implementation, testing and calibration resp. validation of the surface water model alone
- Implementation of the already calibrated groundwater model, which accounts already for the link with the watershed model
- Calibration resp. validation of the coupled modelling system

A stepwise approach was chosen because of the complexity of the modelling system consisting of three independent models (Fig. 6.6). The models were first implemented as stand-alone models (2dMb resp. MODFLOW already linked with TOPKAPI) and those parameters were calibrated which are not effected by the later coupling.

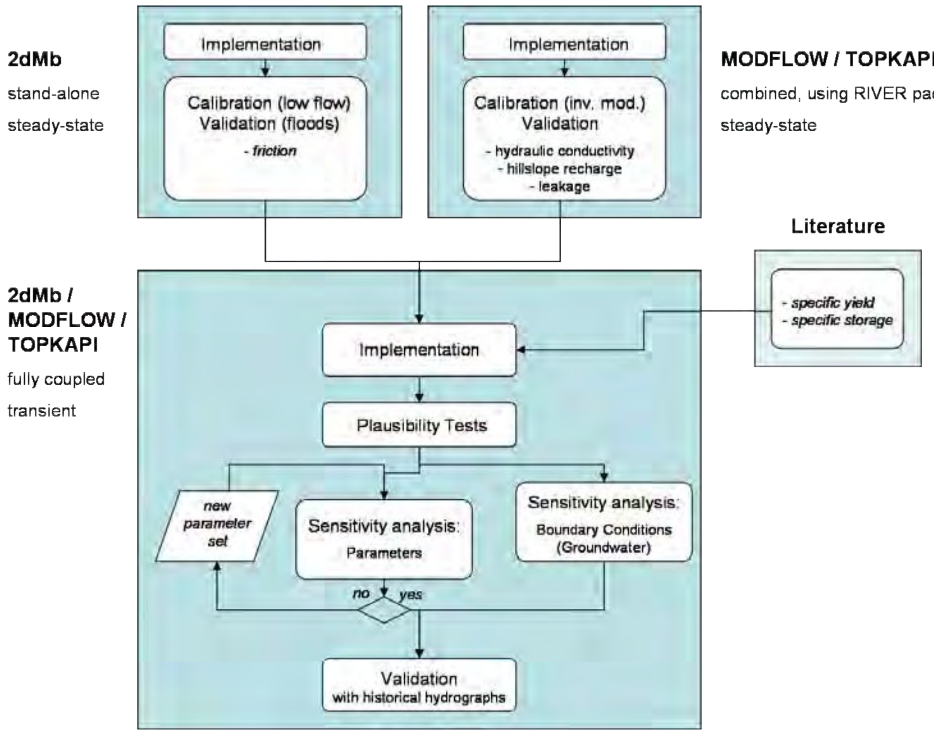


Fig. 6.6: *Schematic of the calibration and validation scheme.*

In case of coupling MODFLOW with TOPKAPI, only one parameter is common in the one-way coupling between TOPKAPI and MODFLOW. This is the fraction of the distributed TOPKAPI subsurface flow, which is taken as lateral recharge from the hillslopes into MODFLOW. This value was calibrated in a common inverse modelling procedure considering both models together.

After the calibration of the stand-alone models 2dMb and MODFLOW/TOPKAPI, the determination of the common parameters set for the coupled system under steady-state conditions as well as the estimation of the aquifer parameters, which are relevant just under transient conditions, still remained to be done. For the first point, the parameter set of hillslope recharge, hydraulic aquifer parameters, leakage factors and also the roughness values of the riverbed were taken over without a new calibration, since the MODFLOW/TOPKAPI parameter estimation already took into account the river – aquifer interaction under steady-state (and low flow) conditions using the RIVER package which is also used in the coupled system. For the wet area to be used in the RIVER package, an average value stemming from the external iterative coupling was used and applied uniformly to all river cells. This is one difference to the conditions of the full coupling under steady-state conditions. The assumption was made that this is of minor influence for the calibration. Furthermore, different model implementations of MODFLOW yielded similar results for the parameter estimation, especially for the hydraulic conductivity and the streambed conductance (leakage factor). However, to prove this assumption, a sensitivity analysis was carried out to show how large is the influence of the parameter choice for the simulation results. Additionally, the influence of different boundary conditions for MODFLOW within the coupled system on the simulation results was investigated.

The general behaviour of the modelling system was investigated using synthetic hydrographs. Finally, validation of the parameter set was performed while applying it to historic hydrographs and comparing it with observations of piezometric heads.

6.3.2. Modelling Domains

In the case of the Maggia valley, the modelling domains of MODFLOW and 2dMb are exactly the same, regardless if implemented as stand-alone models or in the coupled model. This is true both for grid size and grid orientation. The domain and the grids differ however between the two models due to the scale of the physical processes and numerical requirements. The explanations given here apply for all succeeding sections in this chapter.

Both models were applied in the same investigation area of the Maggia valley, which is the riverine corridor, also called here "main valley" (see Fig. 3.1). Concerning the tests with 2dMb in the cutout of the braided area, the grid size of 6.25 m was chosen. The physical processes and the implicit scheme of MODFLOW allowed for a grid size of 25 m.

In terms of gird orientation, the following considerations were made: Because of the fine resolution required for the surface water model, the rectangular 2dMb domain was chosen in such a way that the number of cells was minimized still covering the entire area of interest, which is basically in the NE-SW direction. MODFLOW has a higher resolution and is oriented in N-S direction, therefore being compatible with the grid of the watershed model TOPKAPI, which is identical with the available land use and soil data to be used as input for TOPKAPI. As a consequence, the two grids are rotated against each other. This leads to a number of more than 1.4 million cells for the 2dMb grid, and approx. 300'000 for the MODFLOW grid. The common area of interest, i.e. the main valley between Bignasco and Avegno/Ponte Brolla was set as active cells in both models, whilst all other cells, i.e. basically all cells covering the hillslope areas are set as inactive and hence excluded from the simulation. The number of active cells is therefore approx. 400'000 for 2dMb and 25'000 for MODFLOW. The location and orientation of the 2dMb and the MODFLOW grid are shown in Fig. 6.7.

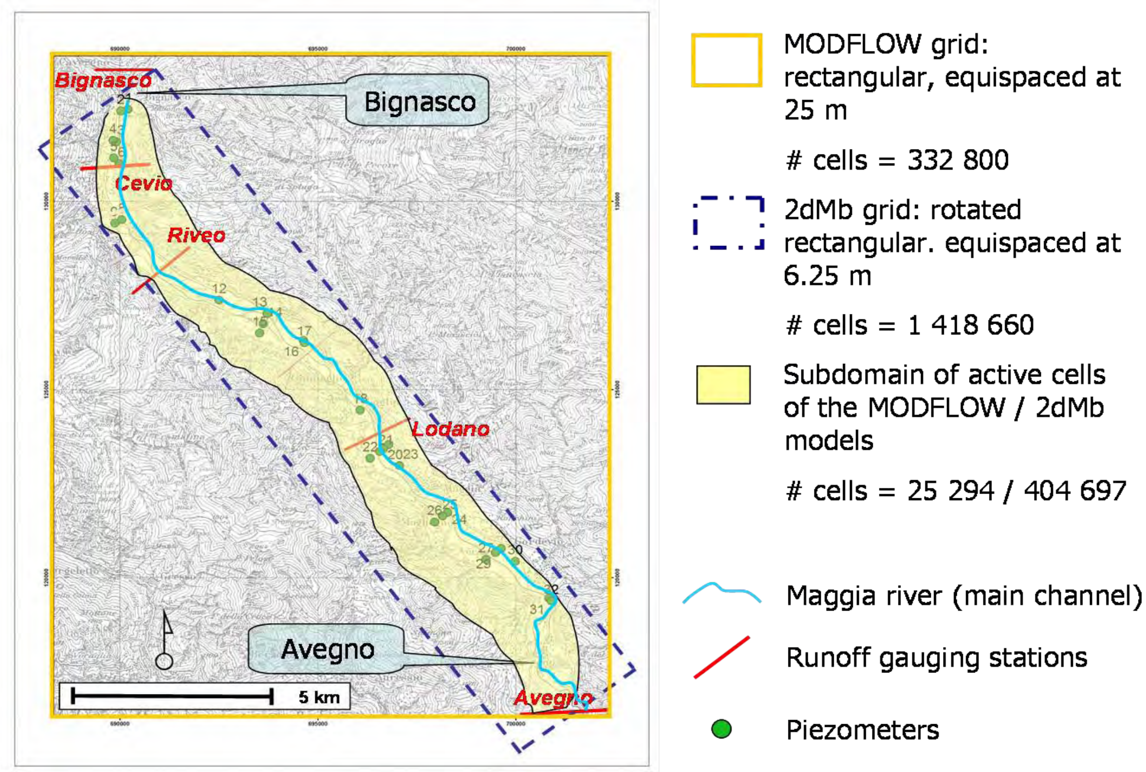


Fig. 6.7: Location and orientation of the MODFLOW and 2dMb grid as implemented in the MaVal project.

The exclusion of the hillslope area makes sense in the case of the Maggia valley, because no groundwater is present in this part of the domain, and because there is no possibility of flooding any hillslope part by the Maggia River. The water fluxes – surface and subsurfaces – are only one-way connected to the inundation area and the aquifer. They can therefore be treated just as external boundaries, both in 2dMb and in the groundwater model. (In 2dMb, only the tributaries from the side valley are considered as flow boundary conditions from the hillslopes.)

Interaction is possible between all corresponding cells of the two models, since flooding is allowed in the total area of the floodplain, which is the main valley. Only a limited number of cells at the upstream and downstream boundary of the domain of 2dMb is excluded in order to avoid possible numerical problems.

6.3.3. Calibration and Validation of the Groundwater Model Including Recharge from the Watershed Model

Watershed model TOPKAPI

The watershed model TOPKAPI was implemented with a grid size of 100 m x 100 m, using the same grid as the available land use and soil maps, which is north-south and west-east oriented. It was set up for a larger area, namely for the watershed of the Maggia River at Locarno-Solduno, which lies further downstream of Avegno/Ponte Brolla, which is the watershed considered for this study. The watershed area at Solduno is 926 km^2 compared to the 592 km^2 at Ponte Brolla. The additional part of the watershed is less influenced by hydropower. The model was calibrated and validated against the stream flow gauge in Locarno-Solduno. The implementation of the watershed model to supply the lateral recharge to the groundwater model is not described in further detail here, because it is not the target of this work, but it is documented in Foglia (2006).

Groundwater model MODFLOW-2000

The groundwater model MODFLOW was completely set up by Foglia (2006). It accounts already for the river – groundwater interaction under steady-state conditions using the RIVER package and information provided by TOPKAPI accounting for the lateral inflow from the hillslopes into the aquifer.

A regular grid of 25 m by 25 m was used, which is also north-south/west-east orientated and therefore compatible with the watershed model TOPKAPI. It was implemented by Foglia (2006) only in steady-state mode, whereas it had to be implemented for the coupling in this work in transient mode. The model was implemented with two layers to allow a higher flexibility, where the boundary between them lies 10 m below the topographic surface, hence the lower layer is always saturated. Due to restricted data availability, all parameters were finally kept the same for both layers (Foglia, pers. comm.), so that the implementation corresponds practically to a 1D representation in the vertical direction. It has been proved that the solutions using one or two layers with identical parameters produce the same results (Foglia, pers. comm.).

Zones for defining the hydraulic conductivity were taken from a map providing the information of different hydrogeological units seen at the surface (Pfammatter and Zanetta, 2003). Different model implementations with different ways of combining them led to a model implementation using four zones, which was considered to be the best model implementation (Foglia, 2006) as illustrated in Fig. 6.8. Zones 1, 3, 4 as well as 5, 7 were combined because of similar a priori information (Foglia, 2006).

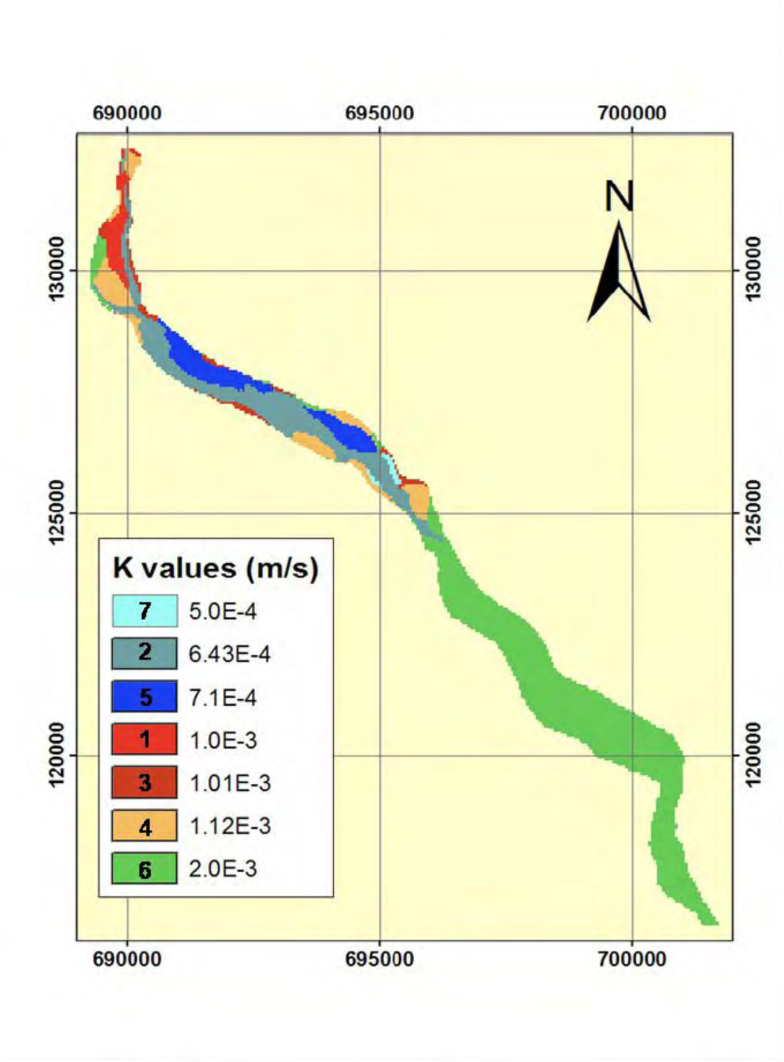


Fig. 6.8: Different zones (1 to 7) of hydraulic conductivity in the aquifer used in MODFLOW (from: Foglia, 2006 resp. Foglia, 2007). Finally, zones 5 and 7 were combined because of insensitivities problems, zones 1, 3, and 4 because of similar magnitudes and based on geological information.

The streambed conductance C_{riv} (eq. 4.4) consists of three factors, namely the riverbed conductance K_{riv}, the wet area A_{wet} and the thickness of the streambed resp. conductance layer M. Only A_{wet} in each MODFLOW cell can be determined using 2dMb in case of full coupling, K_{riv} and M are unknown since no field data are available. They are hence totally subject to calibration. As the parameters K_{riv} and M are not separately identifiable, they must be treated as one single parameter, and were normalized in our case by setting the thickness of the conductance layer to 1 m (see § 4.4.7). Only one lumped value for the streambed conductance was used, since no information was available about any spatial variability.

Recharge from the hillslope was taken from TOPKAPI as already mentioned above and which is described in further detail in Foglia (2006). It depends on the streamflow in Bignasco and is higher under wet conditions. The spatial distribution pattern was assumed to be constant under dry and wet conditions. A single multiplier was however found to multiply the recharge matrix under wet conditions, which corresponded to a discharge in Bignasco of 35.4 m^3/s. For the use in the transient coupled model, an interpolation and extrapolation rule had to be defined for the adaptation of the hillslope recharge dynamically, which is described in § 6.3.5.

Evaporation and transpiration as well as recharge from precipitation and deep percolation over the entire area of the aquifer were not considered. As the main valley is relatively flat, the average deep percolation is estimated to be 1325 mm as the difference between mean precipitation (ca. 1850 mm/a, Kirchhofer and Sevruk, 2001) and mean actual evapotranspiration (ca. 525 mm/a, Menzel et al., 2001). Considering the exchange rates which have been measured (infiltration of approx. 0.5 m^3/s), estimated evapotranspiration in the main valley (area: 25 x 0.5 km^2 x 500 mm/a) is two orders of magnitude (factor 70) smaller than the river – aquifer interaction and can hence be neglected. Similar considerations are true for the deep percolation. In study areas where these processes become important, the evapotranspiration rate could be specified for each cell using the evaporation or the recharge package already included in MODFLOW-2000.

The level of surface topography was used as initial condition. After having tested different boundary conditions, finally the heads in the river using the RIVER package together with additional inflow (as a source term) at the upper boundary in Bignasco were chosen by Foglia (2006). This inflow was proportional to the streamflow. Thus, the upper and lower boundary of the aquifer were flow boundaries with variable flow, which was not specified as a condition. In the transient coupled model, further boundary conditions were tested, and a sensitivity analysis was performed.

Given the geometry of the aquifer, the most important parameters in MODFLOW to be calibrated under steady-state conditions are the saturated hydraulic conductivity K_{sat} of the aquifer transmissivity and the leakage factors of the streambed. Additionally, the inflow at the upper boundary was subject to calibration as well. Different model implementations were tested accounting for different zonal subtypes for the hydraulic conductivity as well as applying both the river package and the streamflow routing package, which accounts only for one-dimensional streamflow. As a result, the parameter values for the different model implementations did not differ too much. Finally, the version accounting just for four conductivity zones was taken as the best model implementation. Parameter optimization was carried out using UCODE_2005 (Poeter et al. 2005; USGS, URL_e), which is an automatic inverse calibration procedure.

In a second step, the recharge factor was included into the parameter optimization procedure so that the calibration of the models was undertaken simultaneously.

All investigations were carried out under both "dry" and "wet" conditions (Foglia, 2006). Since the model was run only under steady-state conditions, only one single value of groundwater heads in each observation point (piezometer) had to be used, i.e. the average value over a selected period. Criteria for the selection of dry conditions were long time intervals with constant runoff in the Maggia River and only little rainfall during this period. The period of February to April 2002 was used for the calibration of MODFLOW alone under dry conditions (the period refers to a condition of constant minimum flow release in Bignasco of 1.2 m^3/s).

For the determination of the recharge, an inverse parameter estimation procedure was carried out using MODFLOW and TOPKAPI together, where streamflow data in Bignasco, Lodano and Avegno were used under dry conditions (April 30th, 2004, end of an extended low flow period with a streamflow of 1.2 m^3/s in Bignasco). For wet conditions, the period from November 13th to December 1st, 2002, was selected as a time interval representing a relatively long period of runoff in the Maggia River above the minimum flow release (average flow in Bignasco of 35 m^3/s with a maximum daily dis–charge of 236 m^3/s within this period) (Fig. 6.9). Under wet periods, only validation was possible because of the lack of streamflow data greater than 10 m^3/s at the outlet in Avegno, which were needed for calibration (Foglia, 2006).

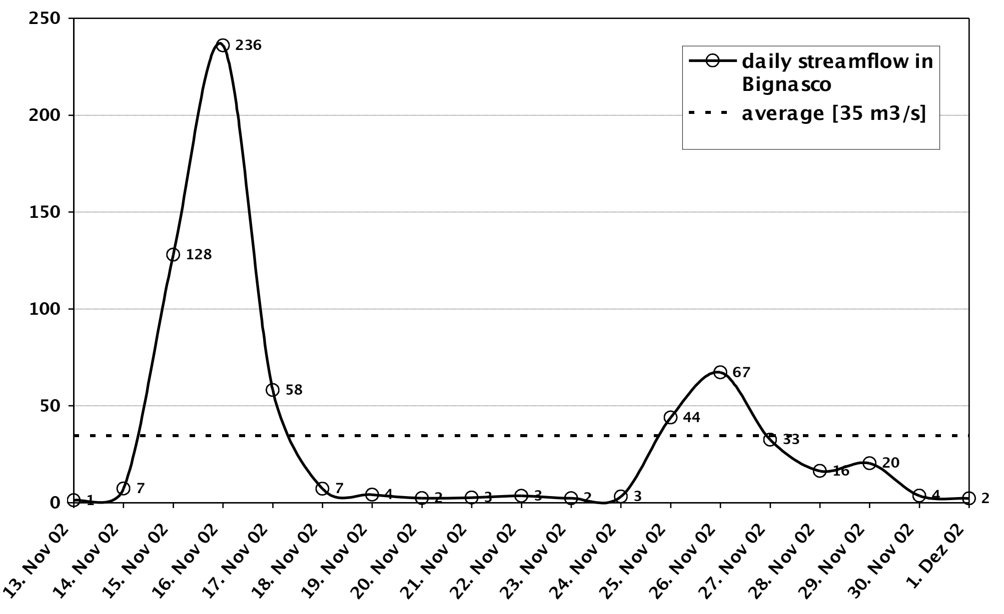


Fig. 6.9: Hydrograph of the Maggia River in Bignasco during the time interval of "wet" conditions used for the validation of the steady-state MODFLOW.

The results of the overall calibration is shown in Tab. 6.1 and in Fig. 6.10. Larger deviations occurred at the piezometers in Gordevio, which are dominated by infiltration from a tributary within an alluvial fan together with hillslope fluxes, and in the piezometers in the braided area in Someo (Foglia, 2006). There, changes in the river bed may play an important factor, as it is also illustrated in Fig. 6.12.

Tab. 6.1: Parameters relevant for the coupling, stemming from the common calibration of MODFLOW with TOPKAPI (Foglia, 2006).

Parameter Name	k zones 1,3,4 [m^2/s]	k zone 2 [m^2/s]	k zones 5,7 [m^2/s]	k zone 6 [m^2/s]	leakage[23] (K_{riv}/M) [m/s]	recharge multiplier dry[24]	recharge multiplier wet
Parameter Value	4.6 E-4	1.5 E-3	2.6 E-3	1.5 E-3	3 E-6	1.0	2.562

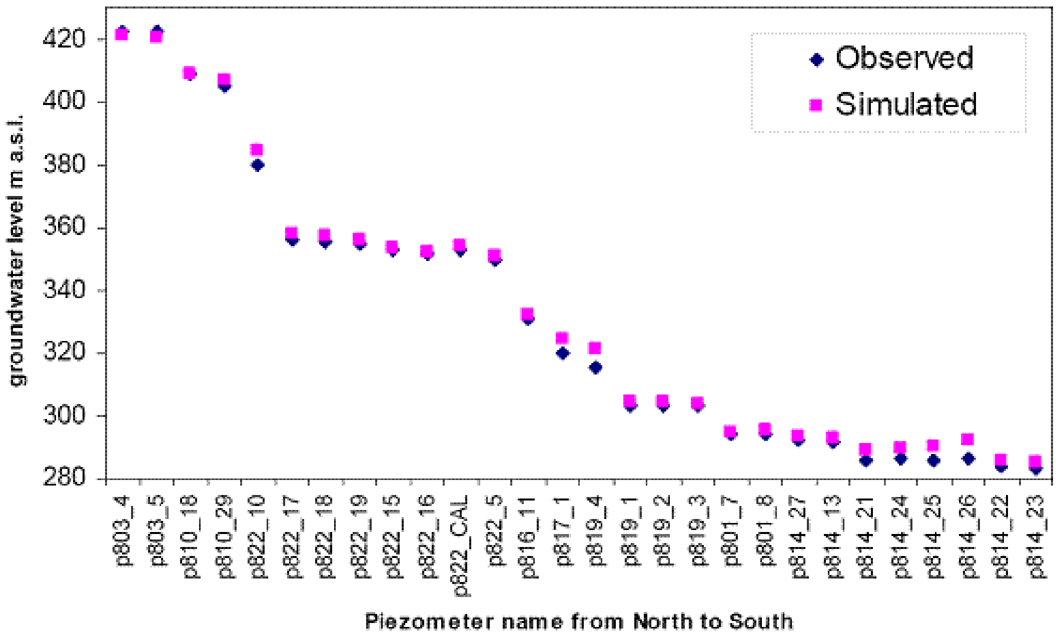


Fig. 6.10: Comparison between observed and simulated piezometric heads (validation of MODFLOW coupled with TOPKAPI), under low flow conditions (1.2 m^3/s) (from: Foglia, 2006).

[23] This leakage factor had to be multiplied by the wet area in each cell, which was set to be constant for each cell by Foglia, but which changes dynamically when coupling later with the surface water model.

[24] The recharge multiplier (dry or wet) has to be multiplied with each recharge cell along the hillslope. Their values are spatially variable, but are in the order of 10^{-6} m^3/s per 25 m grid cell length.

6.3.4. Calibration and Validation of the Surface Water Model

Strategy

Two factors dominate the performance of the surface water model 2dMb: the river geometry and the roughness values. The first is given by the DTM and is related to the model implementation. The second is represented by the value of the equivalent sand roughness k_s and the form factor β. Total roughness k_{tot} is obtained as the product of k_s and ($k_s = k_{tot}*\beta$). The sand roughness was determined using field measurements of line samples (§ 3.3.4) for those areas which are free from vegetation. For vegetated areas, roughness values were taken from the literature, as it was also done for the form factor. Therefore, no calibration was necessary, however the model was validated for the flow depth resp. the inundated area under low flow and flood conditions. With the available friction laws implemented in the model, namely the Manning-Strickler formula and the logarithmic friction law, the sand roughness parameters and the form factor could theoretically also be calibrated and consequently the k_s-values of the open sediment area be compared with those derived from the measurements. This was however not possible due to a restricted data set during flood events so that only a validation could be performed. Instead, a sensitivity analysis was additionally carried out.

Boundary and initial conditions

Boundary conditions have to be applied at the boundaries of the area of the active 2dMb cells. Main fluxes occur at the upper boundary, where the Maggia River enters the modelling domain, and the lower boundary, which is the outflow of the Maggia River out of the modelling domain. Additional fluxes are tributaries from the side valleys. When considered, they were implemented using source and sink terms, as it was mentioned previously.

An upstream flow boundary was chosen in Bignasco, where also streamflow data are available from the federal gauging station. The flow rates must be specified, and are then equally distributed over a specified cross-section. This cross-section is kept constant during the entire simulation being related to river cells that are wet regardless, if the flow rate is high or low. The angle of flow was chosen parallel to the thalweg of the river bed.

At the lower boundary in Avegno/Ponte Brolla, a weir condition was applied. The weir crest was set to the minimum geodetic head of the river bed cross-section at the boundary, the condition applied over the total width of the alluvial plane of the valley. This is equivalent to a boundary condition of critical flow depth in each cell of the outflow boundary, independent of the flow rate. The model 2dMb allows also for the specification of many other boundary conditions, but these ones were experienced to be the numerically most stable ones, whose effects are restricted just to the surrounding area next to the boundaries. All other boundary cells not mentioned here were set to no-flow boundary cells.

Confluences of the tributaries were not used during the validation under quasi-steady state conditions or sensitivity analysis of the surface water model. This is possible, because most of the tributaries are dry, or have a negligible discharge under low flow conditions. Under flooding conditions, the contribution of by far the most important tributary Rovana could be accounted for without explicitly including it as a boundary condition. Peak flow values at the confluence in Visletto were estimated considering the translation time and a possible attenuation of the flood hydrograph between Bignasco and Visletto[25]. For the validation of historical events, i.e. single hydrographs, the most important tributaries were included as source and sink terms.

The quasi-steady-state solution of 2dMb was used as the initial condition for the run with the coupled model (version d). Quasi-steady-state means that 2dMb had been run with constant boundary conditions until the variables of the models did no longer vary with time, or just to a very small extent. In order to launch such a stand-alone simulation, i.e. without coupling and hence no consideration of any exchange rates, the stand-alone model itself had to be initialized. The entire domain was set empty to be dry at the beginning of the run, except for a few cells in the vicinity of the upper boundary, where a certain water depth was specified[26].

Using the approach described above, the result of the quasi-steady state run provides river heads and flow velocities in both horizontal directions in each active surface water model cell, whereas the exchange rates are all zeros, as initial condition.

Friction law and roughness parameters

The friction law based on a vertical logarithmic profile (eq. 4.12) was chosen, because it was seen as the most appropriate choice out of the available formulae in the model to account for open-channel flow in the conditions encountered in the Maggia River. It is well known (Beffa, 1994; Dittrich and Koll, 1997; Bezzola, 2002; Manes et al., 2007) that this formulation has its limitation under low flow conditions as discussed already in § 4.5.4, but it is well established for flow conditions where the flow depth is larger than 5 times the dimension of the roughness elements k_s (Dittrich and Koll, 1997).

Nikuradse sand roughness k_s and the form parameter β can be defined in the model separately for each grid cell. For this reason, nine distinct land surface classes were

[25] This was possible by comparing different historical events where measurements were available in Cevio, Riveo and Lodano.

[26] This approach is widely used in surface water modelling, since a dry domain is a realistic condition from the numerical point of view. If the domain is already flooded, the problem occurs of assigning realistic flow velocities to a given water table. The drawback is that simulation time, until steady conditions are reached, becomes relatively long, if the extent of the domain is large.

delineated from the aerial orthophoto from 2000 (see Fig. 6.11), and a specific sand roughness value was assigned to each class (Tab. 6.2).

Tab. 6.2: Land surface classes delineated from aerial photography and assigned Nikuradse sand roughness values.

Land Surface Class[27]	**k_s**	**$k_{tot} = k_s{*}\beta$**
Water	~ 0.13	0.25
Water with sediment	~ 0.13	0.25
Sediment	~ 0.13	0.25
River bed (not further specified)	~ 0.13	0.25
Grass and bushes		2.5
Shrubs		5.0
Wooded area		10.0
No data		0.2
Outside the delineated area (likely not to be flooded)		0.2

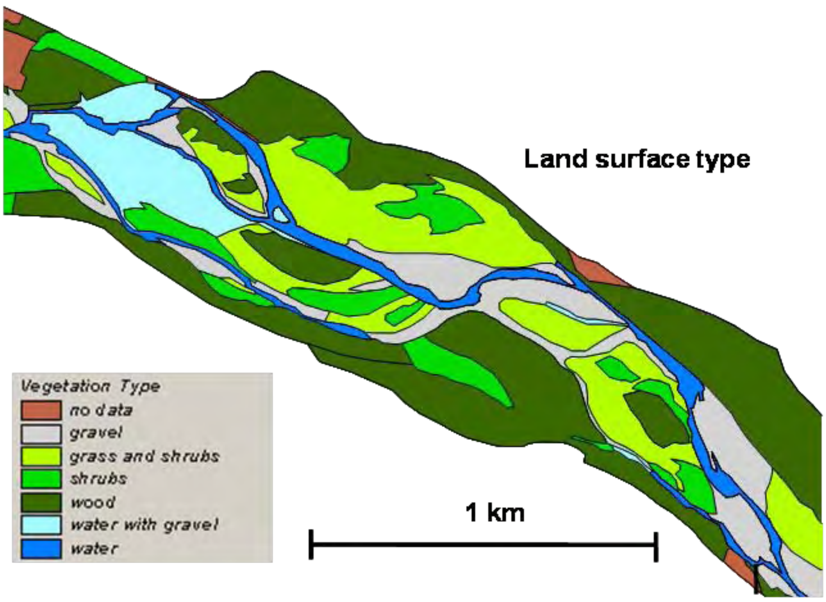


Fig. 6.11: Segment of the map of land surface cover classes delineated from aerial photographs and used for determining sand roughness.

For the surface classes "water" and "open sediment", data of the median grain size d_m were available from the line samples. Despite some variations along the entire

[27] The classes refer to the different types of land surface cover distinguishable from the photographs. The pictures were taken under low flow conditions. "Water" means therefore the main channel being wet under low flow, "Open sediment" the non-vegetated part of the river bed, and "Water with sediment" the transition zone between these two. "River bed" is the combination of the upper three classes in areas of the main valley, where they were not further classified. The same roughness values were assigned to each of the four classes above mentioned because of their similarity.

longitudinal profile of the valley a constant value for the entire value was chosen, because the roughness only slightly varies within the braided area of interest. Moreover, the information about the more detailed spatial variability for the upstream and downstream part of the valley stems from a data source (Anastasi, 1990) which cannot directly be compared to the measurements carried out by Sturzenegger (2005) and by own measurements. The form factor β was assumed to have the value of 2 as it is suggested for nearly plane beds by Bezzola (2003) (see 4.5.4). For the other surface classes, i.e. the vegetated area, the total roughness k_{tot} was assigned on the basis of literature values. (Baptist, pers. comm.; standard values used in the Netherlands; see Tab. 6.2).

Validation and sensitivity analysis of 2dMb as a stand-alone model

Due to the physical nature of the shallow water equations, the necessity resp. freedom for calibration is small. In our case, it is limited to the chose of the friction law and the friction parameters themselves. All other parameters are numerical ones, which may have a certain influence on the solution to some extent, but are not subject to any calibration procedure. Moreover, they must be kept within a certain range for reasons of numerical stability. These parameters include for instance the different threshold values for drying and wetting cells, the choice of the order of the solving scheme and the time steps. Finally, the following values of the numerical parameters were selected (Tab. 6.3):

Tab. 6.3: Numerical parameters used in the simulations (the first order approximation solving scheme was used in order to keep the computational effort as low as possible while accepting some possible inaccuracy of the solution).

h_{dry} (Q=1.2 m3/s)	h_{dry} (Q=1000 m3/s)	h_{wet}	h_{limit}	$h_{exchange}$	cfl (Q=1.2 m3/s)	cfl (Q=1000 m3/s)	T_{exch}
0.04 m	0.08 m	h_{dry}+ 0.01 m	0.08 m	0.09 m	0.4	0.25	10 min

After having chosen the equation for the vertical velocity profile and assigned all roughness parameters, validation of the surface water model was carried out both under low flow and flood conditions.

Low flow

Despite the limited applicability of the friction low under extreme low flow conditions, a validation and a sensitivity analysis were undertaken under conditions with a flow rate of 1.2 m^3/s in Bignasco at eight cross-sections along the longitudinal profile of the river. These were the only streamflow conditions, where numerous measurements were available along the longitudinal profile. The runoff varies substantially along the longitudinal profile because of infiltration and exfiltration processes (see Fig. 3.9).

Piecewise different reaches were used for this purpose. In each reach, uniform flow conditions were assumed. Steady-state simulations were carried out for each flow rate measured in these reaches and sand roughness parameter were varied. With this procedure, measured water levels, flow depth and extension of the water course could be compared in each reach with the results of the simulations. The results are shown in Tab_A 1 in the appendix.

The geodetic measurements pointed however out that the observed and simulated values could not have been directly compared because of changes in the river bed. Deviations were found in all cases between the measured topography and the DTM. In some areas, the horizontal location of the river bed was correct, but the vertical elevation of the river bed differed up to two metres. This is due to old DTM data and local deposition and erosion processes over the years. In the other cases, basically in the braided area, a relatively new and precise DTM (2002) could be used. However, some medium sized floods in the meantime led here to significant horizontal changes of the river course, as it can be seen in the example in Fig. 6.12. This means that a comparison was not possible in these cases either.

Fig. 6.12: Example for horizontal (left) and vertical (right) deviations between DTM and field measurements. Orthophoto is from 2000, river bed topography left from 2002, and 1996 on the right. Yellow (big) dots: geodetic field measurements in 2005, blue (small) dots: simulations under low flow conditions using the underlying river bed topography.
On the right sight, the horizontal coherence is good, but there are vertical differences between the DTM and field measurements of approx. 1.5–2 m.

Despite the vertical and horizontal deviations of the river beds over the entire valley, flow depth could be generally compared between simulations and observations. A tendency of underestimation the flow depths by simulations throughout the entire valley was recognized (with simulated flow depth in most cases not higher than 0.2 m).

Even by varying the sand roughness parameter k_s for the water and sediment area between 0.05 m and 0.35 cm, which are the limits of the range still being feasible from the physical point of view, simulated water level and therefore also flow depth varied only within a few cm (in most cases only 1 to 3 cm, see Tab_A 1 in the appendix). This means that the topography of the DTM and its representation given by the grid structure and resolution totally dominated the flow simulation. Under the given circumstances of low flow, there is practically no possibility of influencing the modelling results by choosing a different parameter set.

Fig. 6.13: Typical river bed reach of the Maggia River under low flow conditions. Transition between wet and dry area (shoreline) is smooth, and representative flow depth is difficult to define.

The observed relatively small flow depths can be explained by the limitation of applying the logarithmic friction law for very low flow depths and large roughness elements on the river bed. It is obvious that the model comes to its limit when flow depth and sand roughness are of similar size. Also in nature, it is difficult to specify a representative flow depth under low flow conditions, because of the structure of the river bed with boulders and big gravel. The border between wet and dry area must be seen in reality under low flow conditions as a transition zone as illustrated in (Fig. 6.13) Nevertheless, even if the relative errors are relatively large, the absolute errors in the water table are still small compared to the accuracy which can be reached by groundwater simulation. While in one-dimensional models, much of the irregularity of the river bed is subsumed in the form factor β and therefore in the total roughness coefficient k_{tot}, it is different in two-dimensional models. There, topography can really dominate the simulation of the flow processes.

Flood events

As flow depth measurements are extremely difficult to perform during flood events, only the extent of the inundated area could be used for validation. This extent had to be determined after the flood events either by the evaluation of the aerial photography taken just after the extreme flood event in August 1978, or by GPS measurements of some flooding marks after the floods in May 2002 and November 2002, though only for restricted areas. The correspondence between the observed and simulated flooded area within the braided area is shown for the event from 1978 in Fig. 6.14 and Fig. 6.15.

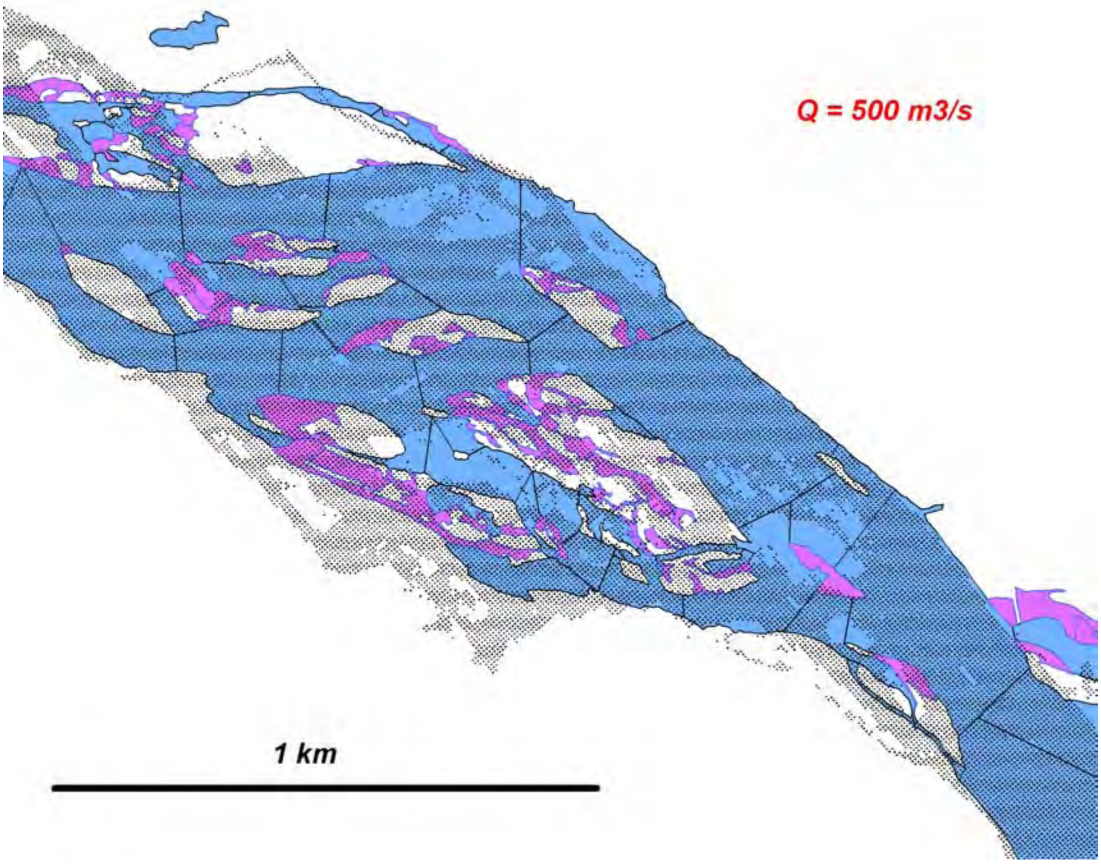


Fig. 6.14: Comparison between simulated and observed (aerial photography) inundated area during the flood event in 1978 with a discharge of approx. 500 m³/s. Observations are in colour, namely in blue: flooded area; in violet: probably flooded, white: non flooded. Simulated flooded area is in grey.

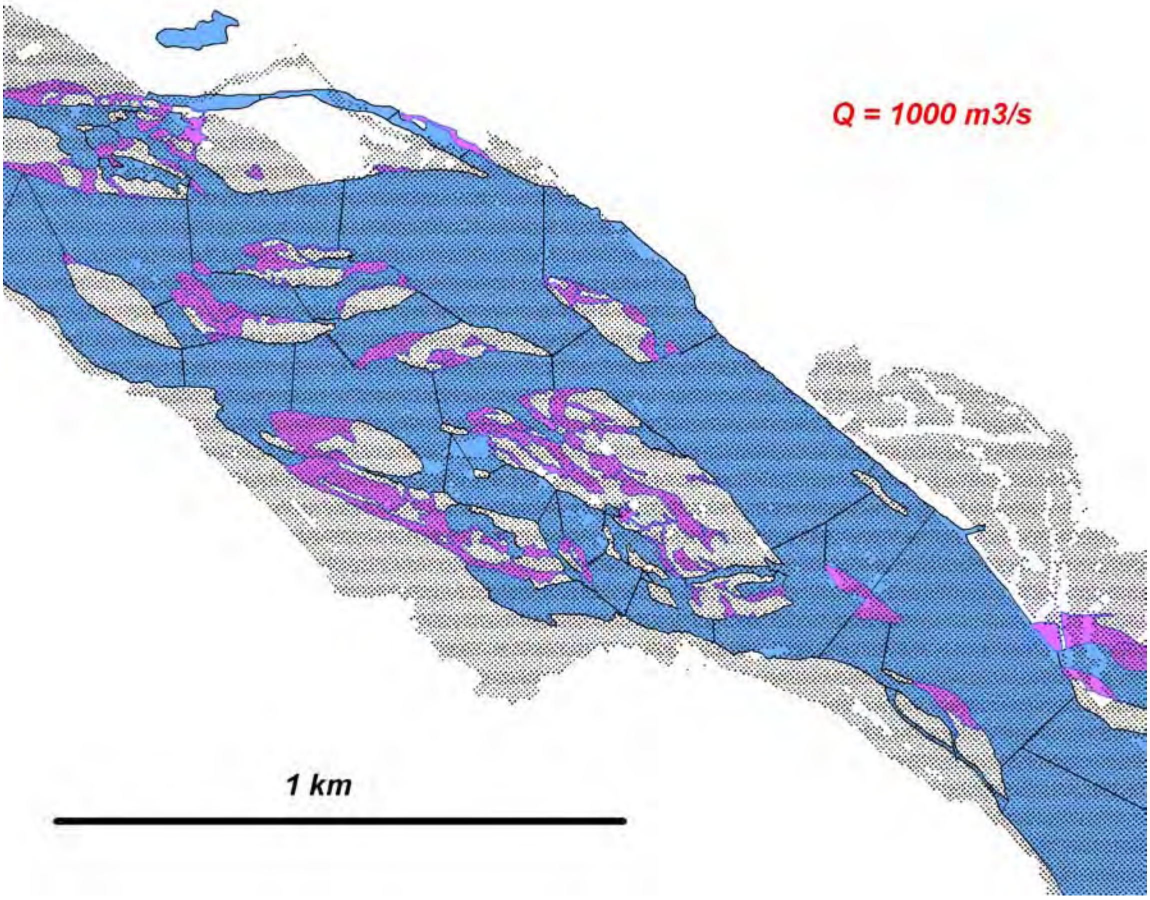


Fig. 6.15: Comparison between simulated and observed (aerial photography) inundated area during the flood event in 1978 with a discharge of approx. 1000 m³/s.
Observations are in colour, namely in blue: flooded area; in violet: probably flooded, white: non flooded. Simulated flooded area is in grey.

Rough estimates assume a peak discharge of approx. 1000 m^3/s. Due to the large inundation area and the relatively short time of peak discharge, some retention effect was likely to occur so that the flood wave might have been flattened. Furthermore, it can be assumed that after the flood only those areas can be identified as flooded in the aerial photograph, which were affected by relatively high inundation depth or velocity hence related to high shear stresses. Only in this way, flooding marks can be observed. This means that these areas have most likely already been flooded below the peak flow. The simulated flooded area is likely expected to be larger than the area identified as being flooded. For these two reasons, the simulated inundated area is shown for the peak discharge of 500 m^3/s and for 1000 m^3/s.

The comparison shows that the inundated area of the simulation with 500 m^3/s corresponds more or less in size with the flooded area detected from the aerial photographs, but the location is considerably different. Only one large island in the northern part of the cutout is unflooded in both simulations and observations. This illustrates the dislocation of the river channels and alterations in the river bed morphology between 1978 and 2002. Therefore, comparison of the simulated and observed results is restricted. The maximum inundation extent was however reproduced quite well by the simulations – here, local morphological differences did not play such a considerable role. Considering all these facts, the validation result is satisfying.

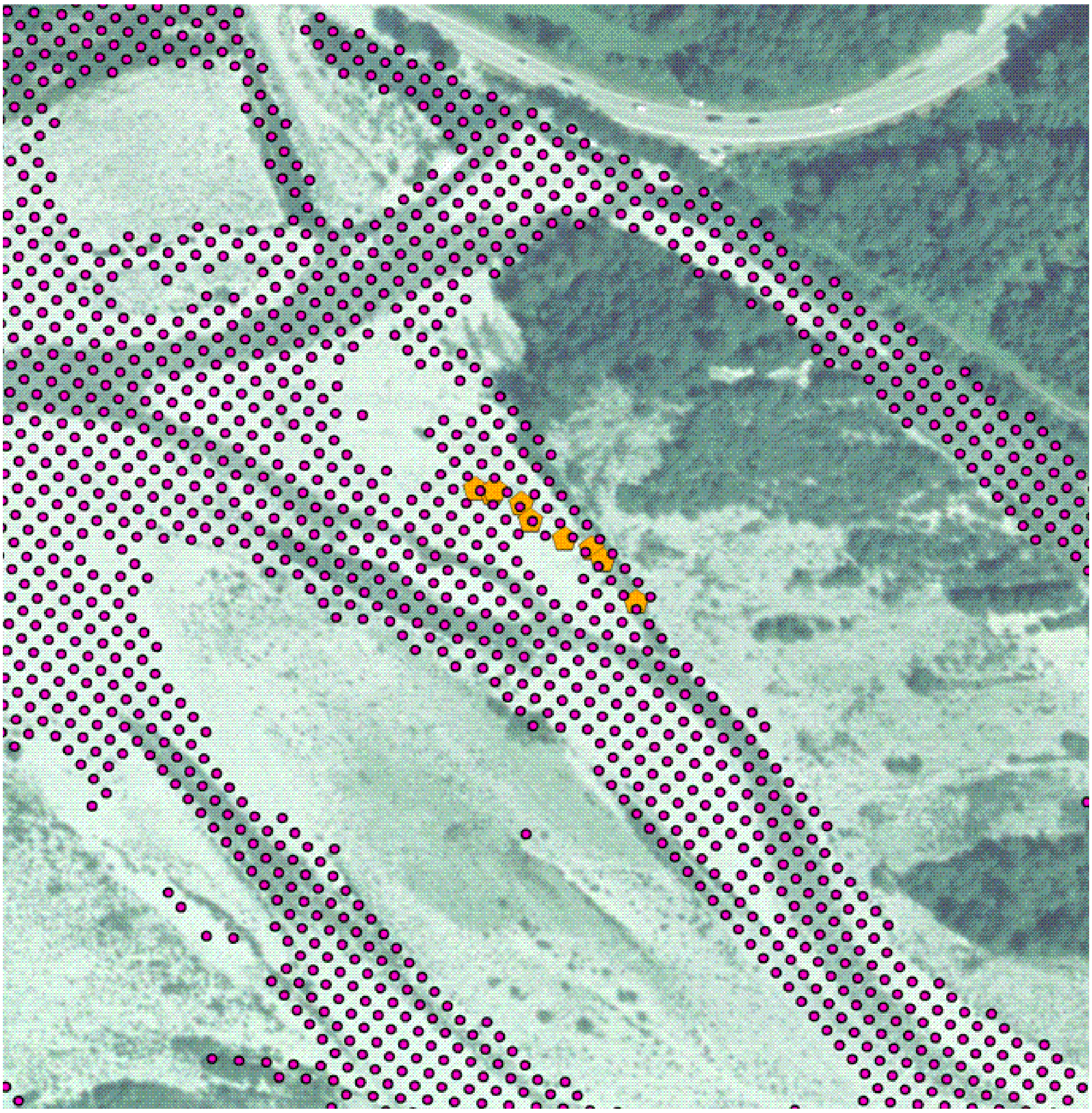


Fig. 6.16: Comparison between simulated flooded area (small purple dots) and points identified as flooded on the basis of field inspections after the flood. Location: Riveo. Estimated peak discharge: 180 m³/s. Field measurements were carried out using GPS. Orange pentagons are locations which were clearly flooded during this event.

Local validation of the inundated area during the events from May 2002 with a peak discharge of 180 m^3/s (Fig. 6.16, Riveo), and from November 2002 with a peak discharge of 560 m^3/s (Fig. 6.17 and Fig. 6.18, Riveo and Someo) was performed. In some areas, the correspondence between observation and simulation of inundation is considerably well, while a dislocation of the river bed becomes obvious in pother areas.

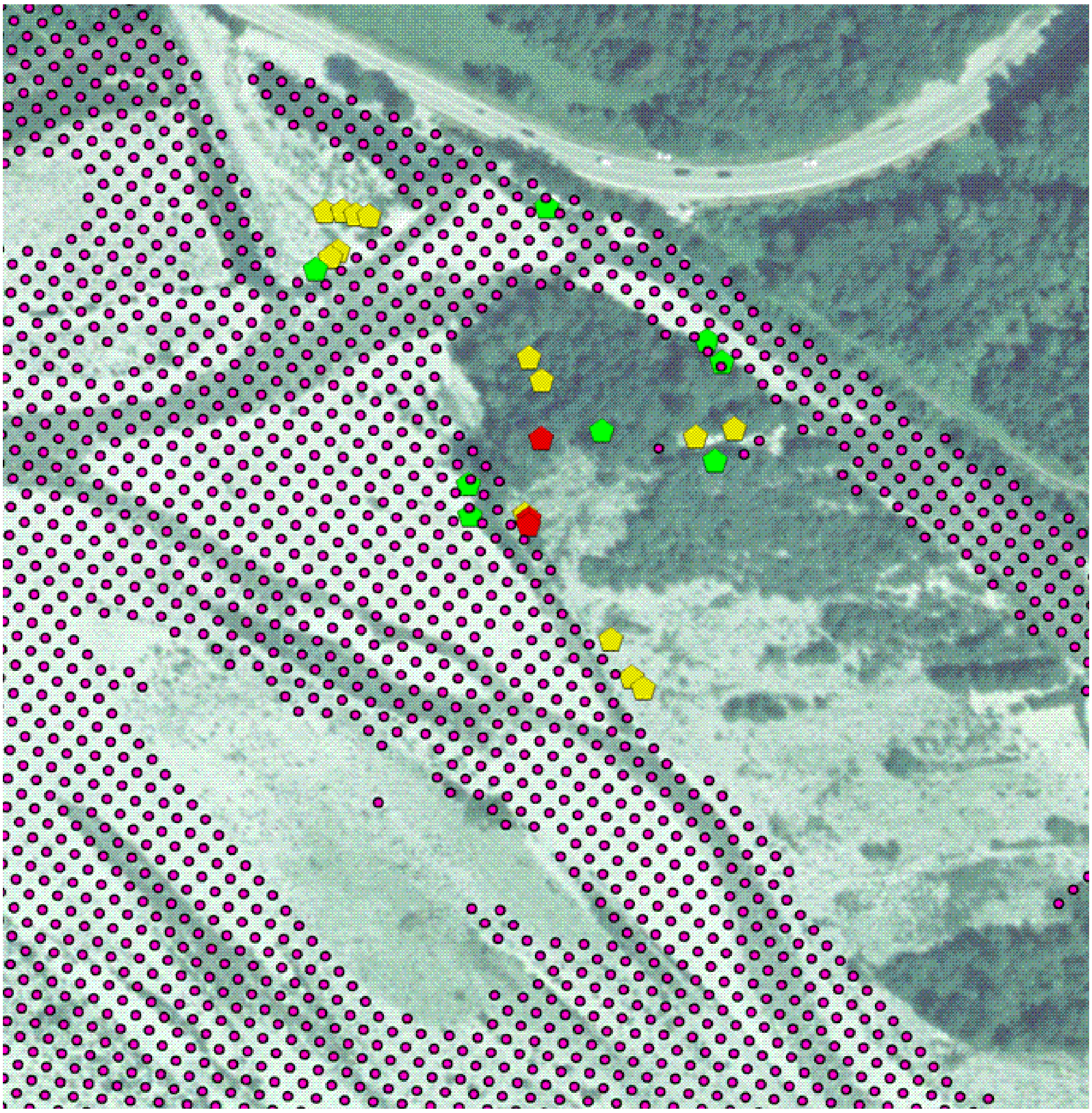


Fig. 6.17: Comparison between simulated flooded area (small purple dots) and points identified as flooded on the basis of field inspections after the flood. Location: Riveo. Estimated peak discharge: 560 m^3/s. Field measurements were carried out using GPS. Red pentagons lie in an non-flooded area yellow; probable border line between flooded and non-flooded area; green: lie in flooded area.

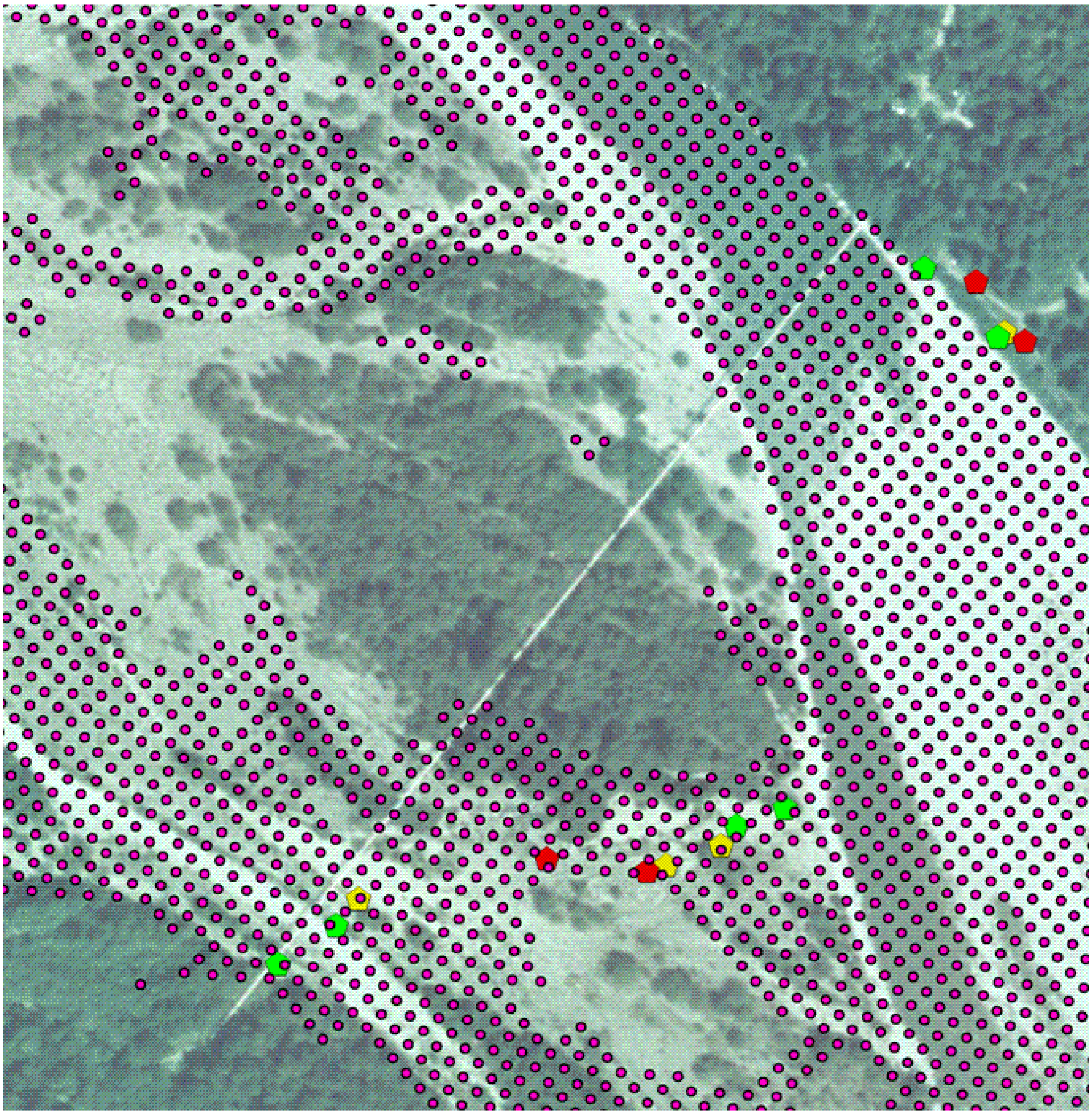


Fig. 6.18: Comparison between simulated flooded area (small purple dots) and points identified as flooded on the basis of field inspections after the flood. Location: Someo. Estimated peak discharge: 560 m^3/s. Field measurements were carried out using GPS. Red pentagons lie in an non-flooded area yellow: probable border line between flooded and non-flooded area; green: lie in flooded area.

In summary, the model reproduces therefore high flood events reasonably well. Considering the available data, the model has shown a behaviour in the validation, which is consistent with the observations. However, the number and quality of validation data could be improved, if more data were available, thus allowing a more systematic evaluation. Furthermore, validation was undertaken only considering the inundated area, therefore indirectly also for the flow depth, but no conclusion can be drawn about the flow velocities, and therefore also not for the shear stress which is strongly related to depth and velocity. However, it must be pointed out once more that the model was not yet implemented primarily with the goal to simulate these values within the vegetated area, as it has already been mentioned in § 4.5.4. Other friction laws, but also more detailed data about vegetation properties would be needed for this approach.

A sensitivity analysis was conducted in the limited area of the braided river system also under flood conditions in order to gain more insight in the model behaviour. For this purpose, the total roughness parameter k_{tot} was varied, and the results are shown in Fig. 6.19 and Fig. 6.20. The roughness factor is defined here as a factor, with which the roughness k_{tot} is multiplied, compared to the a priori parameter set. The results show some sensitivity, if the roughness is considerably reduced (comparison between a) and b)). The simulated flow depth varies within several decimetres, and the braiding is much weaker. An increase of the roughness over the a priori values does however affect the solution only to a minor extent. This means that the model can be assumed to produce robust results in case of high flood events for the following reason: There is always some uncertainty in the determination of the roughness values. Higher values than assumed would have only a very small effect to the simulation results, as previously described. Lower values are however not likely to occur in reality, because the sand roughness parameters were determined especially with respect to the braided area, where roughness tends to be lower as in the remaining, i.e. downstream part of the river, and because the selected value of the form factor β is more toward the lower range of possible literature values.

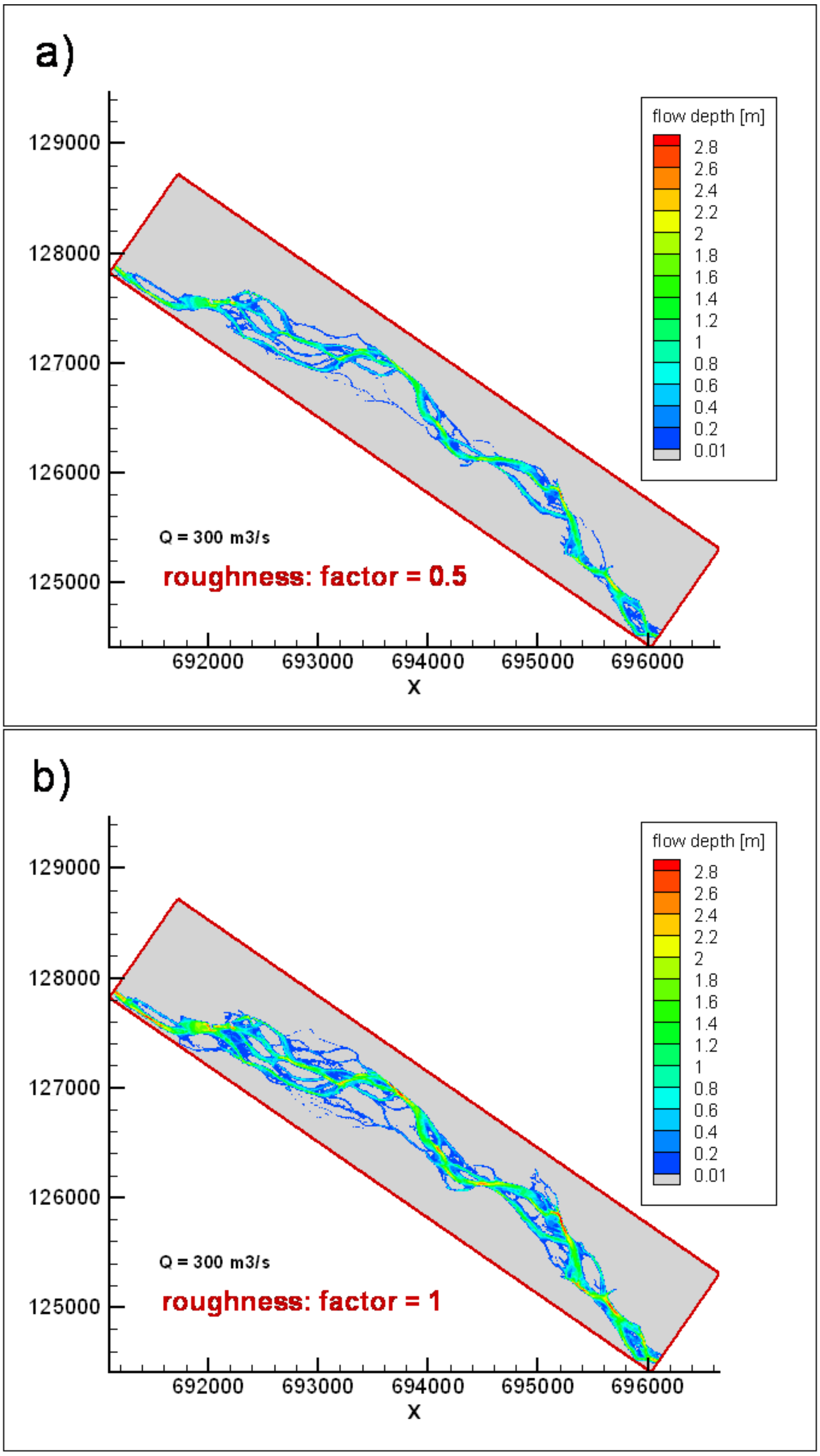


Fig. 6.19: Sensitivity of inundated area and flow depth to roughness in the braided area between Riveo and Coglio under flow conditions of 300 m³/s. Part a) and b)

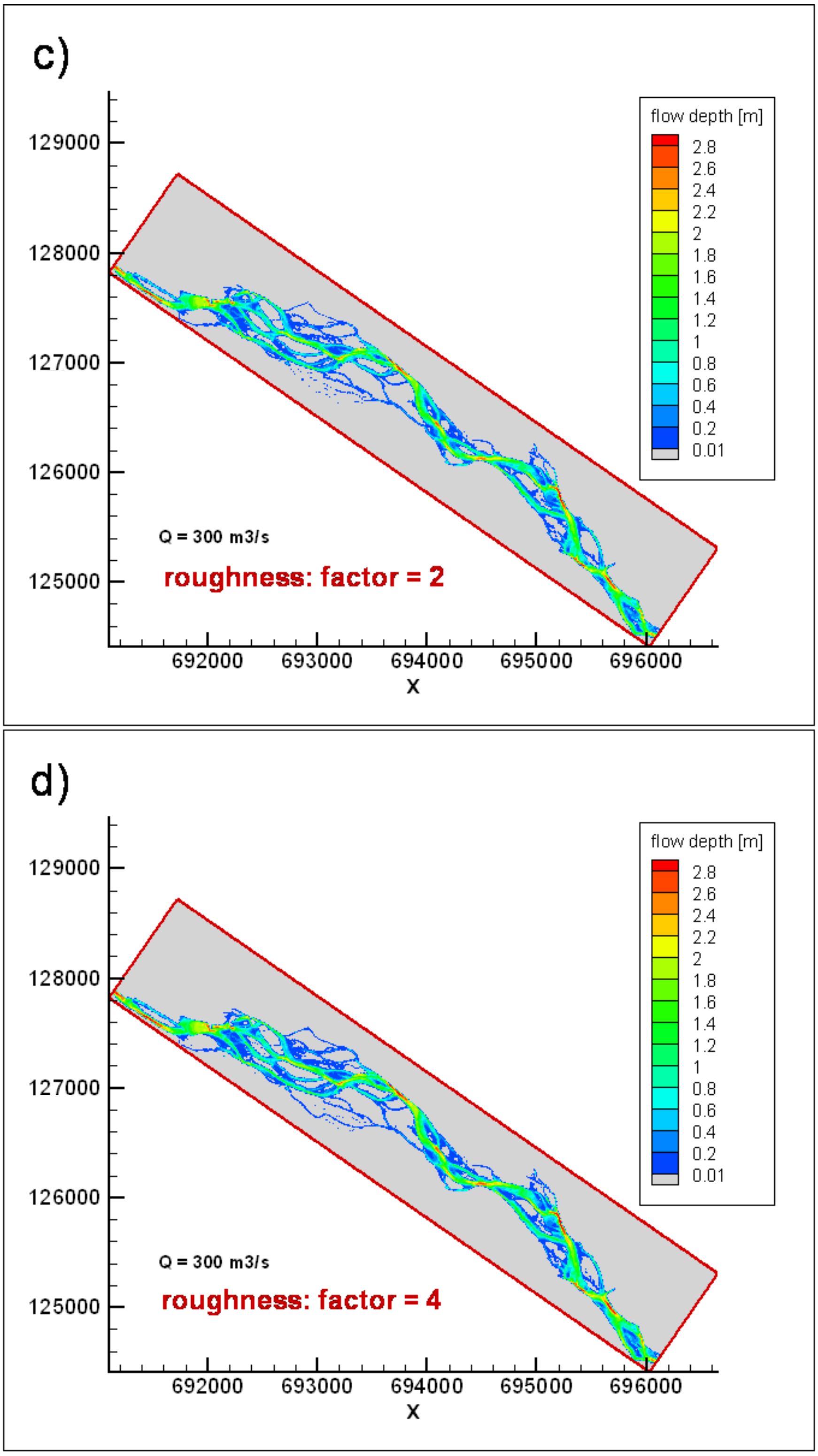


Fig. 6.20: Sensitivity of inundated area and flow depth to roughness in the braided area between Riveo and Coglio under flow conditions of 300 m^3/s. Part c) and d)

6.3.5. Calibration and Validation of Coupled 2dMb and MODFLOW-2000

Adaptations of the MODFLOW implementation for the use in the coupling

MODFLOW as a stand-alone model described in § 6.3.3 was implemented under the background of a possible coupling. Nevertheless, some minor features had to be adapted, basically in relation to the transient mode of MODFLOW in the coupled model: the dynamic adaptation of the hillslope recharge multiplier, specifications in the layer properties, and boundary conditions. All of them are described in the following paragraphs.

The hillslope recharge multiplier (see § 6.3.3) had to be dynamically adapted to the hydrological conditions. Since high streamflow is usually also related to high subsurface flow from the hillslope due to the shallow soils and fast response of the system to rainfall, this variable can be used as a surrogate to describe wet conditions. Furthermore, streamflow in Bignasco was also used by Foglia (2006) to relate the multiplier to the wet conditions. Values were available only for dry conditions (1.2 m^3/s) and a wet period with an average of 35.4 m^3/s (§ 6.3.3). For the transient dynamic coupled simulations, this multiplier had to be both interpolated and extrapolated up to a streamflow of 1000 m^3/s. The model was adapted to use a linear relationship through these two points, or a power law using the origin as a third point to enable to determine the two parameters of the power function. Finally, the decision was made for the power function because of the following consideration: If rainfall intensity and duration increases, small channels, which are usually dry, become activated and convey water directly into the Maggia River, as it can be shown by field observations. Therefore, the subsurface flow does not increase linearly, furthermore, the capacity of the subsurface drainage is also limited due to thin soil layers. This leads to the assumption that hill-slope runoff increases at a slower ratewith increasing streamflow.

Some small changes to the groundwater model implemented by Foglia (2006) had to be made for the transient model, but these do not effect the parametrization. The first concerns the layer properties. In order to save computational time, *Foglia* set the properties to confined layers, although no confining layer is known or can be assumed to be present in the aquifer. Simulations in MODFLOW perform faster using this option, but the results in steady-state are however practically identical. Under transient conditions, this is no longer possible, because only the specific storage could be applied accounting for the compressibility of the water and the porous matrix, but no specific yield could be defined. In order to be consistent with the layer properties, the second horizontal layer has been removed, therefore accounting now only for one single layer of the aquifer, which has a variable groundwater table.

The boundary conditions of MODFLOW had to be adapted. This should however affect the transient model only in the neighbourhood of the boundary, while it could be essential in the steady-state case. For in the latter case, the entire domain will instantaneously adapt to these conditions, while in the transient case, these effects are overlain by the more dominant processes of surface – water groundwater interaction. How significantly the boundary conditions affect the results, was investigated in the sensitivity analysis (see below).

In case of coupling, the choice of suitable boundary conditions became much more important. Depending on the parameter combinations of riverbed conductivity K_{riv} and hydraulic conductivities in the aquifer, some combinations were numerically unfeasible. After several tests for the influence of the boundary conditions in the coupled system (see below), constant head boundaries both at the upstream and downstream end of the aquifer without an additional flow were found to be the best solution.

Parameters

The most technical and general aspects of the coupling have already been described above. The focus here lies on the parameter choice and the boundary conditions of the coupled system.

The roughness parameters of the surface water model were taken from the 2dMb stand-alone implementation. The values are independent from the coupling, because they depend only on the river bed and surface characteristics.

The parameters from the groundwater model including the hillslope recharge (Foglia, 2006) were taken from the stand-alone calibration from MODFLOW including TOPKAPI and accounting for the river – groundwater interaction using the RIVER package under steady-state conditions. It has already been discussed, why this parameter set was supposed to be suitable also for the coupled model. There are however some differences in the model implementation, especially in the determination of the wet part of the river cells A_{wet} and in using the model in a transient mode in the fully coupled version.

The streambed conductance factor is the most essential one with respect to the coupling. If this factor was allowed to change with the new coupling scheme, then it would affect also the other parameters of the groundwater model, which were subject to a joint optimization strategy for calibration. This would lead to a iterative loop of the overall calibration process, which is not feasible regarding the long computational time of the surface water model given the large size of the grid to be used in the Maggia valley. Therefore, a sensitivity analysis of the given parameter set was carried out, and a validation of historical events was performed to test the performance of the coupled model.

For the transient simulation, two additional values had to be defined: specific yield (Sy) and specific storage (Ss). The specific yield is the percentage of an aquifer volume, which is used for storage of water, when this volume was dry and becomes completely wet. This parameter applies only in unconfined aquifers, and is equivalent to the usable porosity. The specific storage is much smaller and accounts for the additional water, which can be stored in an aquifer volume due to the compressibility of the water and the porous matrix. It can usually be neglected in unconfined aquifers, but must be specified in the model in any case. Since no field information was available, literature values were taken (Hölting, 1989), and they were included into the sensitivity analysis. One lumped value was used for the entire valley.

Sensitivity analysis

A sensitivity analysis for the stand-alone model MODFLOW in steady-state mode has already been carried out, and is extensively described and discussed in Foglia (2006). Similarly, a sensitivity analysis for the 2dMb as stand-alone model has been reported in § 6.3.4. Additionally, a sensitivity analysis for the fully coupled transient model was performed and is described here. It is presented in two parts: the sensitivity analysis of parameters, which are closely related to the coupling respectively the unsteady nature of the flow, and the sensitivity of the model to different boundary conditions.

Sensitivity analysis of physical parameters

The sensitivity of the following parameters was tested: river bed conductivity K_{riv}, saturated conductivity of the aquifer, specific storage Ss, and specific yield Sy, and the recharge multiplier. Due to the complexity of the model it is not possible to derive sensitivity functions analytically. Because of the extremely long computational time, it was also not feasible to perform a multivariate sensitivity analysis, which would be principally desirable. This restriction led to the approach of starting from the best-fit parameter set and changing only one parameter at a time while keeping the others fixed. This approach allows us to get insight into the model behaviour.

The choice of these parameters was driven by the consideration that especially the effects of the transient and the "coupling" character of the model should have been investigated. This was the reason for choosing the leakage (K_{riv}/M) and the two "transient-relevant" parameters specific yield (Sy) and specific storage (Ss). The velocity, how fast disturbance (e.g. changing head or water level) travels through the system, mainly depends on the hydraulic conductivity of the aquifer. This is an additional aspect of the inherent property of this parameter, which has not been accounted for in the steady-state calibration. Finally, the recharge parameter was also investigated, since it is an essential parameter for understanding the interplay between lateral inflow from the hillslopes and river – aquifer exchange, because the groundwater heads in the aquifer are the result of these two overlapping processes.

The sensitivity analysis was based on the a priori parameter set from the joint calibration of MODFLOW with TOPKAPI using the original version of the river package under steady-state conditions. Starting from this parameter set, 20 sensitivity runs were performed using a step function for the river inflow in Bignasco by changing only one parameter at a time and keeping the others fixed. Hereby, the parameters k-factor[28], leakage (K_{riv}/M)[29], and specific storage were varied with the factors of 1/100, 1/10, 10, and 100, the parameter recharge multiplier wet[30] with the factors 1/25, 1/5, 5, and 25, whereas the parameter specific yield was varied as 0.13, 0.20, 0.34 and 0.41. The results are illustrated in appendix 3 (Tab_A 3, Fig_A 2 and Fig_A 3).

This analysis led the the following general conclusion: the modelling results are very sensitive to some parameter combinations. Most of the modelling results showed a groundwater table which was far too deep in the upper part of the valley, and too high in the lower part except for some selected parameter combinations (Fig_A 3). The key point was the combination of the river bed conductivity K_{riv} and the hydraulic conductivity k of the aquifer. The root mean square error RMSE (eq. 6.3) was calculated for the begin of the simulation runs which corresponds still to the steady-state solution of the system:

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n} (h_{obs,i} - h_{sim,i})^2}{n}} \quad (6.3),$$

where $h_{obs,i}$ is observed groundwater head, $h_{sim,i}$ is simulated groundwater head, and n is number of observed piezometers.

Three parameter sets were acceptable with a similar RMSE of 1.5 resp. 1.6 m. When comparing with the observations from MODFLOW/TOPKAPI as stand-alone in steady-state conditions (see Fig. 6.10), they were probably in the same order of magnitude.[31] The a priori parameter set was not among those three. They differed from this by either a river conductance, which was 10-fold higher, or a hydraulic conductivity of the aquifer, which was 10-fold or 100-fold lower than a priori assumed.

For all further investigations, especially for the simulation of historical events which are shown later in this chapter, the parameter set with the 10-fold higher river bed conductance (leakage) was chosen. The values are shown in Tab. 6.4, simulation results in Fig_A 2 in the appendix. The reason for the problem with the original parameter set is

[28] The k-factor is the factor, with which all hydraulic conductivity values (K-values) from Tab. 6.1 are multiplied.

[29] This parameter is identical with the one in Tab. 6.1

[30] This parameter is identical with the one in Tab. 6.1

[31] The exact RMSE for the results showing in Fig. 6.10 could not have been calculated, because the original data were not accessible to the author.

seen in the differences between the distinct modelling approaches to obtain or verify these parameter values.

Tab. 6.4: Best parameter set adapted after the sensitivity analysis and used for further investigations with the coupled model.

Parameter Name	k-factor [–]	leakage (K_{riv}/M) [m^2/s]	recharge multiplier wet	specific yield Sy [–]	specific storage SS [m^{-1}]
Parameter Value	1	$3 * 10^{-5}$	2.562	0.27	$3 * 10^{-4}$

The leakage and the hydraulic conductivity of the aquifer have by far the largest influence. When leakage is high, already the initial steady-state solution of the coupled system show only very small deviations between the water level in the river and the piezometric head in the underlying groundwater cell. When leakage is low, the opposite can be observed. Somehow opposite is the influence of the hydraulic conductivity of the aquifer. Higher values lead to large deviations between the observed and simulated groundwater table, while very low conductivities result in practically identical river and groundwater heads directly along the river. The findings both for the leakage and the hydraulic conductivity are qualitatively what should be expected.

The influence of the recharge parameter is more indifferent. Very high values change the overall pattern of the groundwater table without influencing the overall performance of the model. This is true for the given combination of the other parameter values. The behaviour might be different, if more than one parameter would be changed at a time.

The parameters specific yield and specific storage are relevant only under transient conditions, they have therefore no impact on the initial steady-state solutions. The specific storage was expected to be insensitive, since theoretically it is relevant only in a confined aquifer. This was confirmed by the sensitivity analyses. The specific storage is important for the transport of water volumes through the aquifer. If it is small, only little water is used to generate relatively large changes in the piezometric head. Groundwater levels will therefore respond relatively fast to changes in the water level in the river. This is true both for infiltration and for exfiltration. If the specific storage is large, the opposite behaviour is expected. The simulated reaction of the groundwater table as a result of the rising water table in the river and the use of different values for Ss was in correspondence with this theory.

To summarize, the coupled model is very sensitive to the choice of the parameter combination of the leakage and the hydraulic conductivity. Starting with a plausible combination, all sensitivity runs led to physically plausible results. This proves that the model behaves physically correctly. Out of those parameters which are relevant for the coupling, the leakage parameter and the hydraulic conductivity of the aquifer are most sensitive. The chosen parameter leads to plausible results. Fine tuning of these parameters was not possible within the time framework of this study, because the computational time still restricts the number of possible model runs. This problem can be overcome by a substantial reduction in computing time as proposed in the outlook.

Influence of the boundary conditions on modelling results

In addition to the investigation of the parameter sensitivity, some sensitivity runs were performed to investigate the influence of the boundary conditions on the modelling results. For this, the same parameter set from Tab. 6.4 was used. The different sensitivity runs are listed in Tab_A 2 in the appendix. The results are shown in Fig_A 1.

The comparison between the modelling results of the water levels and groundwater heads along a longitudinal profile shows that the influence of the boundary conditions can be seen only in the vicinity of the boundary. This proves that the boundary conditions of MODFLOW are not critical, if heads are specified at the inflow and outflow boundary and the overall parameter sets are feasible.

Validation of historical events

Historical events of floods or intermediate flows have been selected after a long period of constant low flow. This gives the opportunity to start the simulations with steady-state conditions of the entire system, as they occur at a flow rate of 1.2 m^3/s in winter or 1.8 m^3/s in summer. The results for the validation run for the highest flood event during the study period were also hourly piezometric data. The flood event started November, 14^{th}, 2002, and lasted approx. 3 days, and is illustrated in Fig. 6.21. The peak discharge in Bignasco was 370 m^3/s, the one of the tributary Rovana approx. 200 m^3/s, whereas the flood peaks were almost superimposing. The results for the groundwater response of selected piezometers are presented in Fig. 6.22.

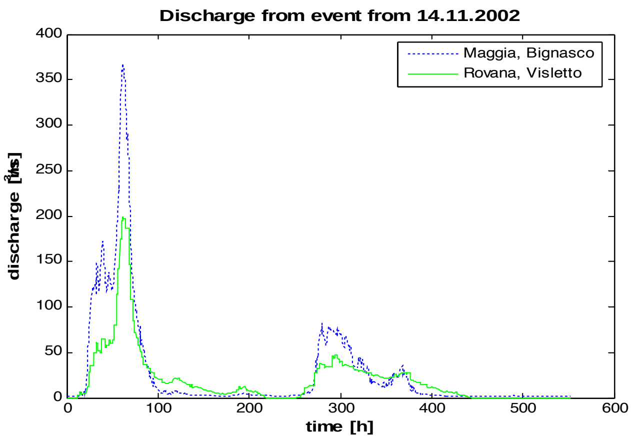


Fig. 6.21: Streamflow of the Maggia River in Bignasco and of the tributary Rovana before its confluence in Visletto, used for the validation run. Only the first flood peak with a duration of approx. 3 days starting from November 14^{th}, 2002, was simulated.

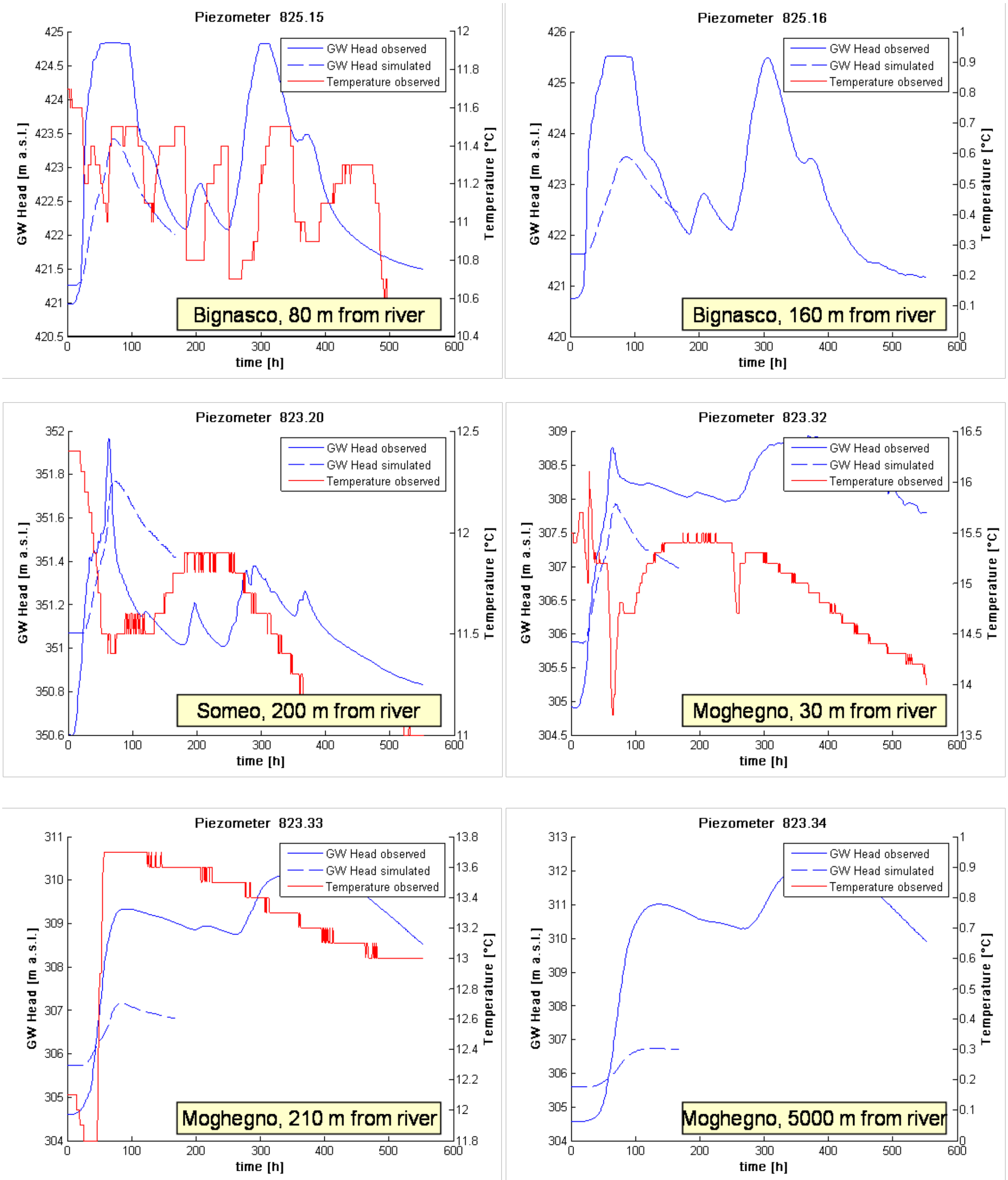


Fig. 6.22: Comparison of simulated and observed groundwater heads as a response to a large flood event starting from November, 14th, 2002. Groundwater temperature is also shown where data were available.

The two piezometers 825.15 and 825.16 in Bignasco are the most upstream ones in the main valley. The river is always infiltrating in this reach. The two piezometers are perpendicular to the thalweg, and the piezometer 825.16 which is more remote from the river, is located very close to the hillslope. Piezometer 823.20 is situated in the braided area in the part behind the embankments. The piezometers 823.32, 823.33 and 823.34 are located in a straight line also perpendicular to the thalweg, as shown in Fig. 1.7.

Looking at all piezometers, it can be stated, that for this flood event the general characteristics of the groundwater table response, i.e. the general shape of the curves, are simulated by the model for most of the piezometers in accordance with th physical processes. The reaction of the simulated groundwater table is however still too slow or weak in most cases. Additionally, the parametrization of the coupled model still suffers from the implementation of the groundwater model which was given as an input for this work. There, differences of one to several meters in the groundwater levels were observed (Fig. 6.10) in steady-state, so that it is expected that these errors propagate also into the the coupled transient model. Indeed, in most cases already the initial groundwater heads in the coupled model show these differences between observed and simulated values.

The accordance was generally better for high flood events than for other simulation runs with moderate flood waves during the study period. This can be explained by the fact that the problem of simulating the correct groundwater tables before the flood becomes here more important, while in the other case, the flood wave in itself influences the model to a larger extent.

Additionally, simulations were also performed with different leakage factors for the connected and the unconnected aquifer. The simulated results showed similar *RMSE* values, and are therefore not discussed in more detail here.

Besides of the information gained by the analysis of the water heads, additional information is provided by the temperature data from the piezometers as illustrated in Fig. 6.22, too. This information can also be used for validation. As an example, the piezometers 823.20 and 823.32 are discussed here. It can be seen that the groundwater level and temperature curves are opposite in phase with each other. This confirms the high interaction between the Maggia River and these two piezometers by the infiltration process for the following reason: in November, surface water is colder that groundwater, which leads to a drop in temperature, if infiltration occurs, but temperature in the groundwater would not change in case of exfiltration. The response, i.e .the temperature difference, to the infiltration process is much stronger in piezometer 823.32. This can easily be explained by its close neighbourhood to the river, while piezometer 823.20 is located 7-fold farer away from the river. Moreover, the fast response of temperature due to changes in the water level in the river confirms the high connection between the river and the groundwater system.

To conclude, improvement of the coupled model is still possible by optimizing the parameter set. This requires however numerous calibration runs, which is only realizable after the parallelization of the programming code. Nevertheless, what is most important, the model shows qualitatively the correct behaviour and is robust for simulating even high flood events. This is the premise, under which this parallelization work should start. After its completion, one should return to the issue of parameter calibration in this special case of the Maggia valley.

7. Results

7.1. Discussion of the Modelling Tool

The final goal of this work was to provide a modelling tool, which is capable to simulate surface water flow (open channel flow in a river including inundation) and groundwater flow in the aquifer in a transient manner and simultaneously in order to account for feedback mechanisms. This tool had to be suitable to simulate two-dimensional phenomena in the floodplain such as activating and deactivating river branches, but also to deal with continuously rising water tables and therefore continuously increasing inundated areas, since natural braided rivers do not flow in a well-defined river bed. The model must be able to account for wetting and drying and must be robust also for relatively steep slopes and to handle hydraulic jumps. The modelling system must reproduce correctly the physical processes it is simulating and give reliable results.

It was shown that the modelling system developed and implemented in this work, fulfils all these requirements. The implemented coupling system is numerically robust and simulates natural behaviour. Feedback mechanisms between the river system and the aquifer can be taken into account. The model is suitable for transient simulations.

Calibration and validation of the modelling system was not a single straightforward procedure due to the complexity of the system and due to strong limitations in computational power. Extremely long model runs do not allow for inverse techniques or more sophisticated calibration procedures. A stepwise procedure was therefore chosen, which allowed the calibration and validation for the single models, followed by targeted combined calibration. A first calibration combining groundwater flow and surface water flow was possible for steady-state conditions using the already implemented river package in MODFLOW. Calibration under transient conditions was not possible within the limited time frame of this work, but validation runs were performed.

The groundwater model shows some parameter sensitivity, at least under steady-state conditions. The surface water model is almost insensitive to roughness parameters under low flow conditions due to the high influence of the roughness layer while using the logarithmic friction law. The sensitivity to flood events has been illustrated.

In general, the results of the validation are satisfactory. The modelling system shows in all cases principally a plausible behaviour with some deviations from the available observations. Because of morphological changes within the river system it is intrinsically not easy to discriminate model or parameter errors from observation errors. If the river bed has changed, this has also an effect on the piezometric heads in the groundwater, and the values are no longer comparable. Thus, some local changes in the observation period did not allow a one-to-one validation of the model, but lead to a mixed validation based on matching of patterns, temporal dynamics and variable ranges.

In general, it can be stated that any modelling system of this kind is very data demanding if the results have to be verified by a rigorous point-to-point calibration and validation procedure. Furthermore, the computing time is still a limiting factor, both in terms of calibration and validation, especially for long-term simulations. For this reason, it will be necessary in the future to find ways to drastically reduce computational time. Possible strategies are outlined in the outlook in § 8.3.3.

Limitations are given for the flow depths in the river under very low flow conditions, because flow depth is always underestimated by the model in this case. Accuracy of groundwater levels are very different depending on the area. This can be the case because of incorrect river topography or by the bulk representation of the river bed conductivity or hydraulic conductivity of the aquifer. Higher numbers of conductivity zones would not lead to an overall better fit, as was shown by Foglia (2006). In this case, the problem of overparameterization may, however, become significant. On the other hand, it cannot be expected to have a very good fit in all piezometers if the aquifer is inhomogeneous and if local aquifer and river bed properties become important such as in the case of the piezometers close to the river. Local refinement of the parameter zones could overcome this problem, but the effort required by such a strategy both in terms of data availability and computational requirements would have exceeded the time framework of this study.

With respect to the ecological issues within this project, certain requirements for the accuracy are given. Inundated area is represented principally well, and flow depth and duration of inundation are plausible. This information can be used for the investigation of riparian vegetation evolution and for the development of a vegetation model. More critical is the accuracy of the groundwater model. The behaviour of the depth-to-groundwater is essential in this point. Furthermore, sedimentation and deposition of the river bed has an additional impact on the model accuracy. First, a higher precision can be reached using the detailed DTM. Furthermore, the depth-to-groundwater is decisive for the plants to access water through the root system, since the floodplain alluvium has a very low water storage capacity. On the other hand, groundwater levels must be deep enough to provide sufficient oxygen throughout the root system. The estimation of the root depth for different species is very difficult, because first, it strongly depends on the habitat, and second, there is still a lack of scientific knowledge in this respect. A good comprehensive literature research has been carried out by Polomski (1998). He argued that different plant species have a quite different range in root depths. Some develop their roots in a band of just a few decimetres, others of several metres (e.g. Common oak [= Quercus robur]). For the first class of plants, the accuracy of groundwater heads obtained by the modelling system may not be sufficient, but for the second group it would be.

A higher number of piezometers in the braided area would also be beneficial, if higher precision of the modelling output is desired. For this study, a set of piezometers was available only on the embanked area. It turned out that many of them were substantially influenced by the lateral recharge fluxes from the hillslope. Within islands and bars of the braided area or further upstream or downstream, no piezometers were available. In the 1970s, some more piezometers were installed in this area, but they are no longer available. The problem is that due to limited accessibility and the coarse river bed material, it would be a very big effort and therefore very costly to install further piezometers in this area.

Another problem is the temporal evolution of the floodplain. In principle, after each flood with a sediment mobilization and channel forming capacity, the geometry should be measured or modelled again and a new DTM should be developed for a new model implementation. So far, only one representation of the surface topography could have been used, but as it has been described in § 6.3.4, already during the time span of this project, some changes in the morphology occurred therefore limiting the interpretation of the modelling results with respect to the observed reality.

7.2. Simulation Results

After some general consideration about the overall model features, carried out in the previous section, the results of the simulations of the coupled model for the main valley are presented here. They are mainly shown as examples to illustrate the capability, flexibility and potential use of the model. Output such as the spatial representations of inundated area, flow depths and flow velocities and bed shear stress are first presented, and infiltration – exfiltration patterns are analyzed in more detail. Further, the spatial patterns of flow depth, vegetation type and depth-to-groundwater were able to be compared with each other. Finally, statistical analysis in form of histograms of flow depth and flow velocity is presented. The following chapter (7.3) will then present further evaluations of modelling results in relation to ecological aspects.

7.2.1. Spatial Representation of Modelling Results

Inundated area: Flow depth and flow velocity - shear stress

First, the development of the simulated inundated area including flow depth and flow velocity is shown in Fig. 7.1 for a section of the braided area. It can be seen how additional river channels emerge with increasing flow. It is shown, how the braiding of the river system develops, and how flow velocities increase with increasing flow depth.

Using the shields equation for a rough calculation for the Maggia valley (critical shields parameter of 0.4, average slope of 1 % total roughness of 0.25 m), a critical shear stress for the begin of motion of the river bed of approximately 160 N/m^2 is obtained. This value can be compared in Fig. 7.1 for runoff conditions of 35 and 350 m^3/s. For the lower discharge, no areas show shear stresses above this critical shear stress in the simulated result, whereas a significant area should be in motion for the higher discharge value. This can serve as additional information for the validation of the roughness. It shows that the selected roughness tends to be at the lower edge of the range of feasible values.

Infiltration / Exfiltration

Spatial representation of infiltration and exfiltration rates can also be extracted from the modelling system. Two results are shown and discussed in this section: 1. the evolution of infiltration and exfiltration areas under transient conditions of a rising hydrograph (in Bignasco: from 1.2 m^3/s to 1000 m^3/s within 2 h; see Fig. 7.3), using the full coupling (version d); 2. a comparison between the external iterative coupling (version a) under low flow conditions (1.2 m^3/s in Bignasco, i.e. the present environmental flow (EF) for the winter season) and the full coupling (version d).

Rising hydrograph under transient conditions

Fig. 7.3 illustrates the spatial distribution of the exchange fluxes after 2 h, 4 h, 6 h and 8 h. The figure after 2 h corresponds still to the quasi-steady-state conditions for a runoff of 1.2 m^3/s. Both infiltration and exfiltration occurs, while infiltration takes place especially in the upstream part of the valley[32], and exfiltration occurs mainly in the downstream part, but with a significantly higher rate.

[32] Since the infiltration rate has a relatively small magnitude due to the low water depth in the area of the unconnected aquifer, it falls into the mid range (yellow dots), and can therefore not be discriminated in this figure.

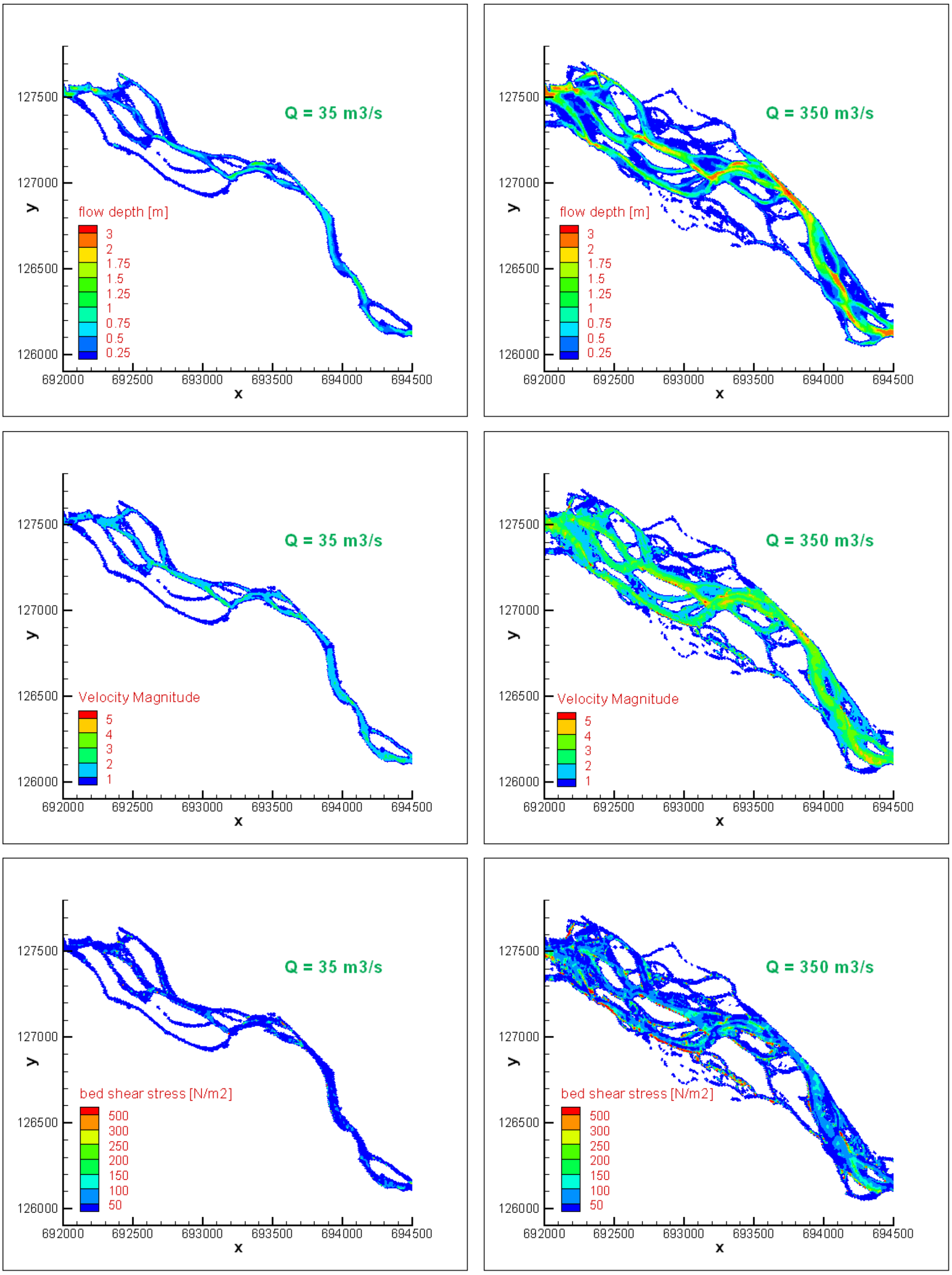


Fig. 7.1: *Flow depth, flow velocity and shear stress for steady-state discharge conditions of 35 resp. 350 m^3/s for a section in the braided area.*

This general behaviour coincides on the one hand with the theory of upwelling and downwelling under the existence of geological controls (see Fig. 2.1), and with the measurements under low flow conditions (see Fig. 3.9), which show that streamflow increases toward the downstream boundary in Avegno to a rate which is higher than the inflow. This can be explained by the hillslope fluxes into the aquifer, the contribution of the hydropower plant in Giumaglio in the middle of the valley and the small tributary Salto Maggia, which contributes also even under low flow conditions and which is affected by the operation in Giumaglio.

The integration over all exchange fluxes leads to a total exfiltration of 0.41 m^3/s. This is in a good agreement with the water balance, as it is shown in Tab. 7.1. The energy production there depends mainly on the demand, less on the actual streamflow conditions. The average values of the release in Giumaglio and of the creek Salto Maggia is therefore taken as an indicator for the contribution under steady-state low flow conditions. Using these values only a difference of 0.18 m^3/s respectively 6 % is obtained. This deviation includes errors in the simulation as well as measuring errors, the fact that the measurements have been performed only once after a long lasting dry period, and the fact that simulated inflow was not exactly the measured one.

Tab. 7.1: Water balance for the quasi-steady-state simulation with the coupled model.

	Estimated flow [m^3/s]	**Minimum flow** [m^3/s]	**Maximum flow** [m^3/s]
Inflow Bignasco (measured)[33]	1.27	---	---
Spatially integrated exchange fluxes (simulated)	0.41	---	---
Hydropower release in Giumaglio	1.00	0.00	3.20
Tributary Salto Maggia	0.22	0.03	2.15
Sum	2.90	---	---
Outflow Avegno (measured)	3.09		
Difference	*0.19*		
Ratio	*6.2 %*		

The frequency distribution for the quasi-steady-state conditions is little skewed to the left side, but still relatively symmetric as shown in Fig. 7.2. The hydrograph increases between 2 h and 4 h, as it can be seen in a clear increase of infiltration in the upper part up to the braided area. This reach corresponds to the travel distance of the increasing flood wave. The frequency distribution is already clearly skewed to the right, i.e. toward increasing infiltration. After 6 hours, the flood has already reached the lower boundary in Avegno. The flood propagation was much faster in this time interval due to the high flow velocities related to flooding. At this point in time, infiltrating conditions occur

[33] The simulation was performed under slightly different flow conditions of 1.2 m^3/s.

throughout the entire valley. The integrated exchange fluxes have no the opposite sign, i.e. there is a total of 16.0 m^3/s of infiltration. Between 6 h and 8 h, only minor changes can be seen. In the downstream part of the valley, there are two additional reaches being inundated, and very small changes in the infiltration – exfiltration pattern in the braided area occur. At both points in time (6 h and 8 h), the frequency distribution is strongly skewed with almost no more exfiltration, and the total exchange fluxes are practically the same (Fig. 7.2).

This experiment shows that the general behaviour of the coupled model is in good accordance with the expectations of the physical processes of flood propagation and river – aquifer interaction.

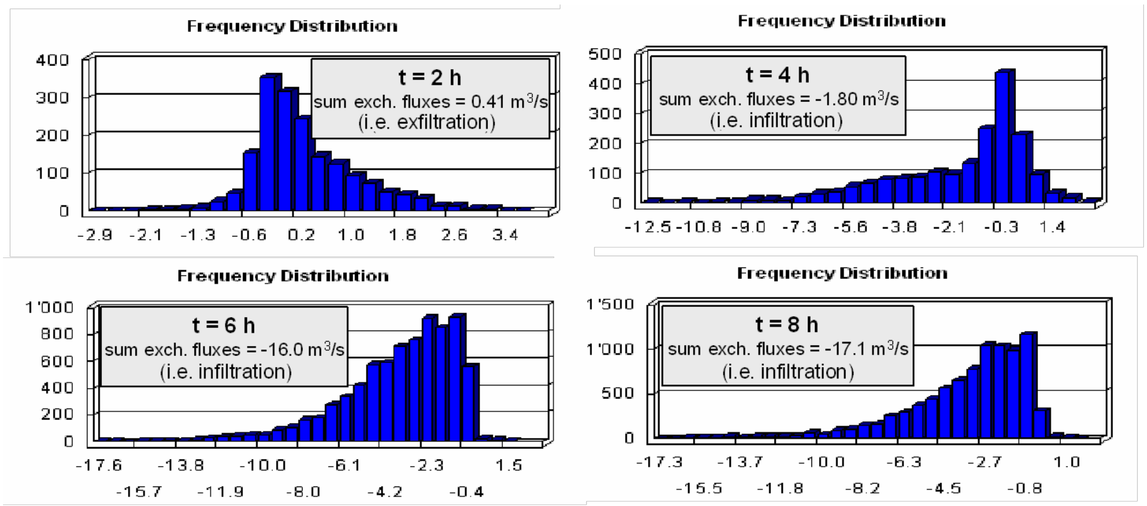


Fig. 7.2: Absolute frequency distribution of the number of cells of exchange flux at different points in time and the spatially integrated exchange fluxes simulated for the synthetic flood hydrograph illustrated in Fig. 7.3. The units of the bins are 10^{-6} ms^{-1}.

Comparison between iterative and full coupling

The results for the river–aquifer exchange for low flow conditions (1.2 m^3/s in Bignasco) from the iterative coupling (version a, see § 5.4) are shown in Fig. 7.4. The infiltration and exfiltration rates are integrated over the areas of the red polygons, and the discharge at the border of each polygon was calculated. These values are compared with streamflow measurements under these low flow conditions. The general accordance is quite good and reflects the general pattern of infiltration and exfiltration reaches along the longitudinal profile of the river. The absolute magnitude of exfiltration in the lower part of the valley is however underestimated by the model. The reason is probably due to the negligence of the small tributary Salto Maggia and the release from the hydropower station in Giumaglio (see precious section).

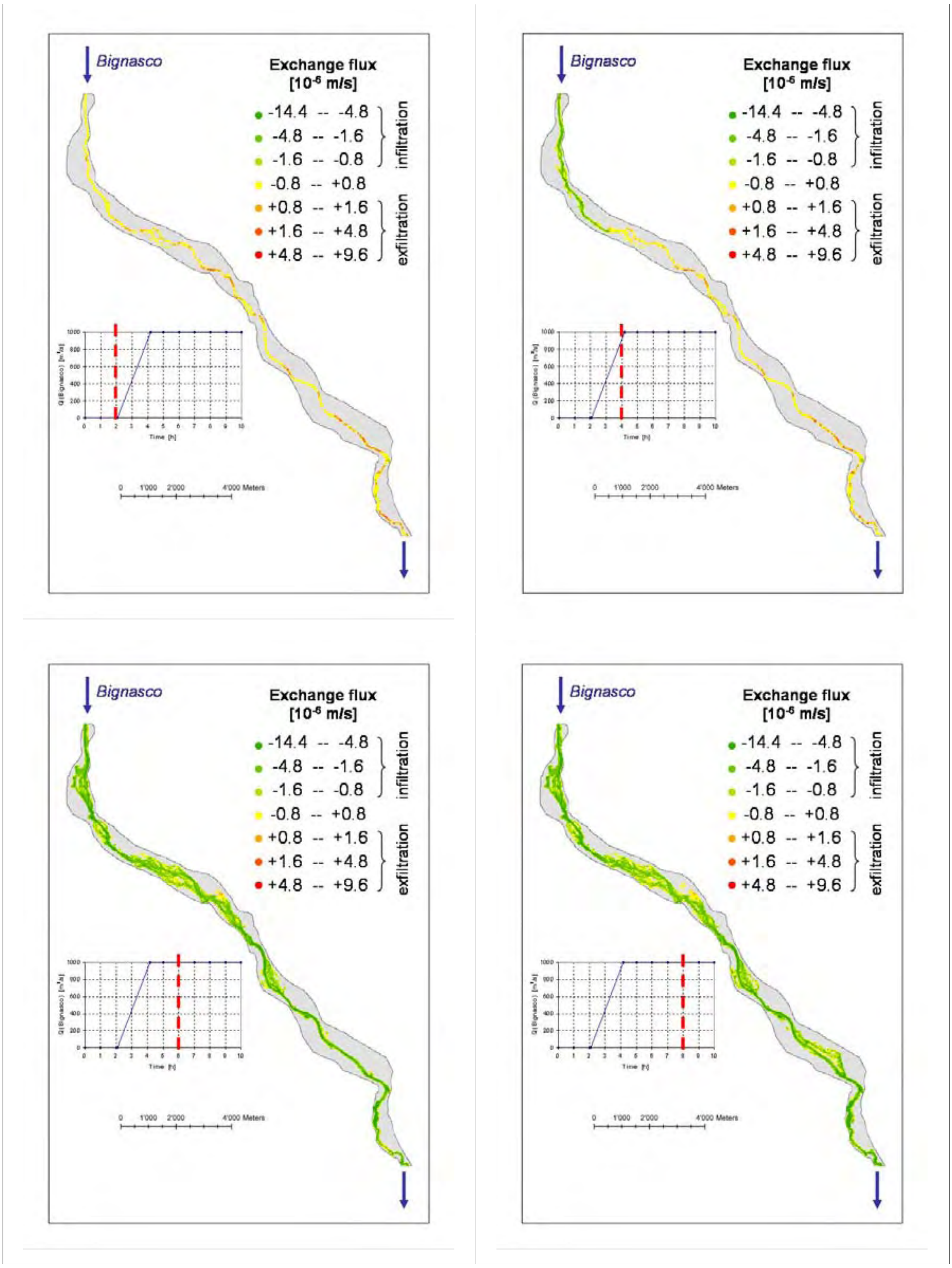


Fig. 7.3: Evolution of the pattern of river – aquifer exchange during a hypothetical flood. Starting under steady-state conditions with a streamflow of 200 m³/s, the flow was relatively instantaneously increased to a maximum of 1000 m³/s (linearly between time=2 h and time=4 h, as shown in the small hydrographs). Positive values are exfiltration, negative values infiltration.

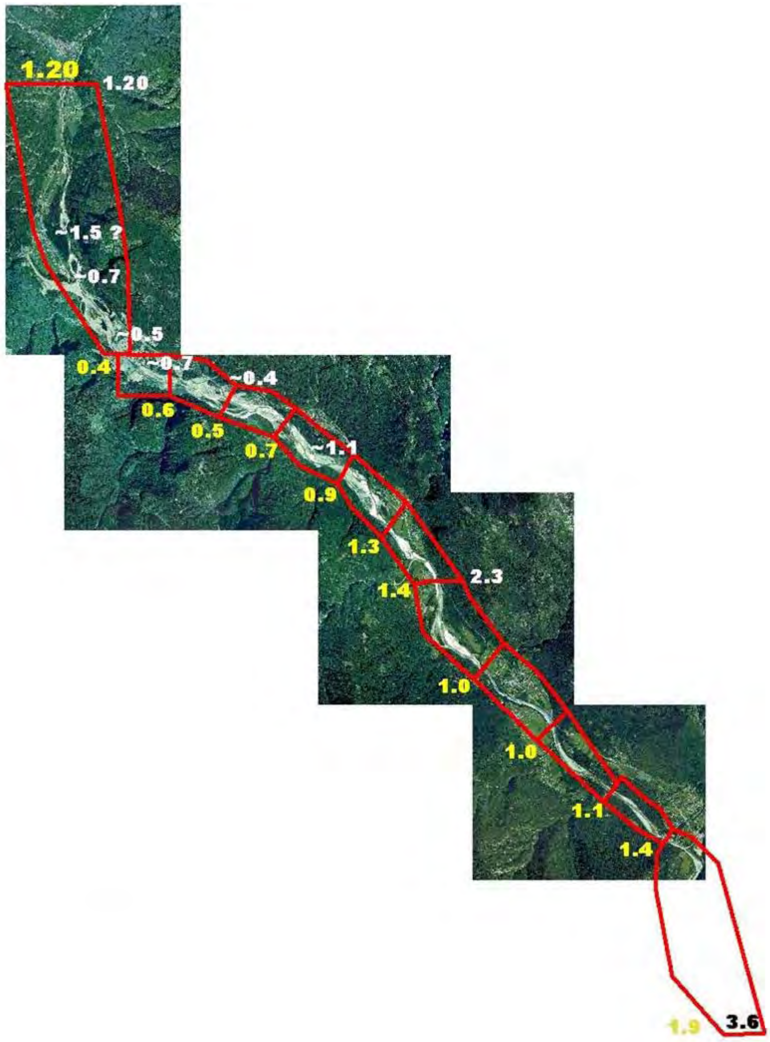


Fig. 7.4: Comparison of simulated (yellow) and measured (white) runoff based on simulated exchange rates using the iterative coupling approach (version a). For the analysis, the exchange rates were integrated over the areas of the red polygons.

More interesting in this context is the comparison of the simulated results between the external and iterative coupling (version a) and the initial steady-state solution of the full coupling (version d). It can be seen as a test for the reliability of the scheme of the full coupling. Indeed, the results are supposed to be similar. Smaller differences may however occur especially due to the different way of the treatment and location of sink and source terms and the different consideration of the wet portion of the wet area in the MODFLOW river cells.

Fig. 7.3 illustrates for the time after 2 hours still the results of the steady-state simulation for version d. These results can be compared with those from Fig. 7.4 from version a. Two reaches of relatively strong exfiltration (red) can be found in the central and in the downstream part of the main valley. These areas are consistent with the exfiltration reaches from the iterative coupling. Furthermore, the integration of infiltration and exfiltration over the entire valley leads to a net exfiltration of 0.4 m^3/s for the full coupling (Fig. 7.2) and 0.7 m^3/s for the iterative coupling (Fig. 7.4). Considering the absolute values of infiltration and exfiltration over the whole valley, this difference in the net exchange value is not very large and can be explained with the differences in the two approaches.

7.2.2. Comparison of Spatial Patterns

Besides quantitative evaluation of important flow quantities at the pixel scale, the flow and groundwater patterns can be analyzed by matching them with the patterns of the river corridor vegetation system, which is controlled by flow dynamics. For this reason, the aerial photographs, vegetation types, flow depth (and therefore illustrating inundated area) under 300 m^3/s discharge (steady-state) and depth-to-groundwater under low flow steady-state conditions (1.2 m^3/s) were qualitatively compared as shown in Fig. 7.5. The discharge of 300 m^3/s was chosen because it is characterized by a return period of little more than two years, thus occurring quite frequently and having a magnitude large enough to have a possible impact on the floodplain vegetation. The discharge of 1.2 m^3/s represents the long-lasting low flow periods, where groundwater table is assumed to be lowest therefore having the largest impact for the riparian vegetation in terms of possible water stress (drought), and also corresponds to the present EF for winter conditions.

The most striking observation is the good agreement of the spatial patterns, as it was elaborated statistically in more detail in Ruf et al. (in press). The congruence between the channel system visible in the orthophoto and the flow depth pattern indicates that the 2dMb model produces plausible results. Flow depth at a runoff of 300 m^3/s is highest where the channel system is evolved and present in the aerial photograph. Smaller deviations due to some changes in the river bed are however observed. Additionally, the inundated area corresponds to a large extent with the open sediment area. The same pattern is reflected by the vegetation. Correspondingly, the depth-to groundwater is largest in these areas. All these observations are seen as an additional validation and proof of the suitability of the model to be used for studying flow-vegetation interactions in braided streams. Moreover, these spatial patterns can also serve as a starting point to investigate the relative influence of the two water-related factors on the vegetation development: inundation and the depth-to-groundwater, e.g. by spatial correlations analysis.

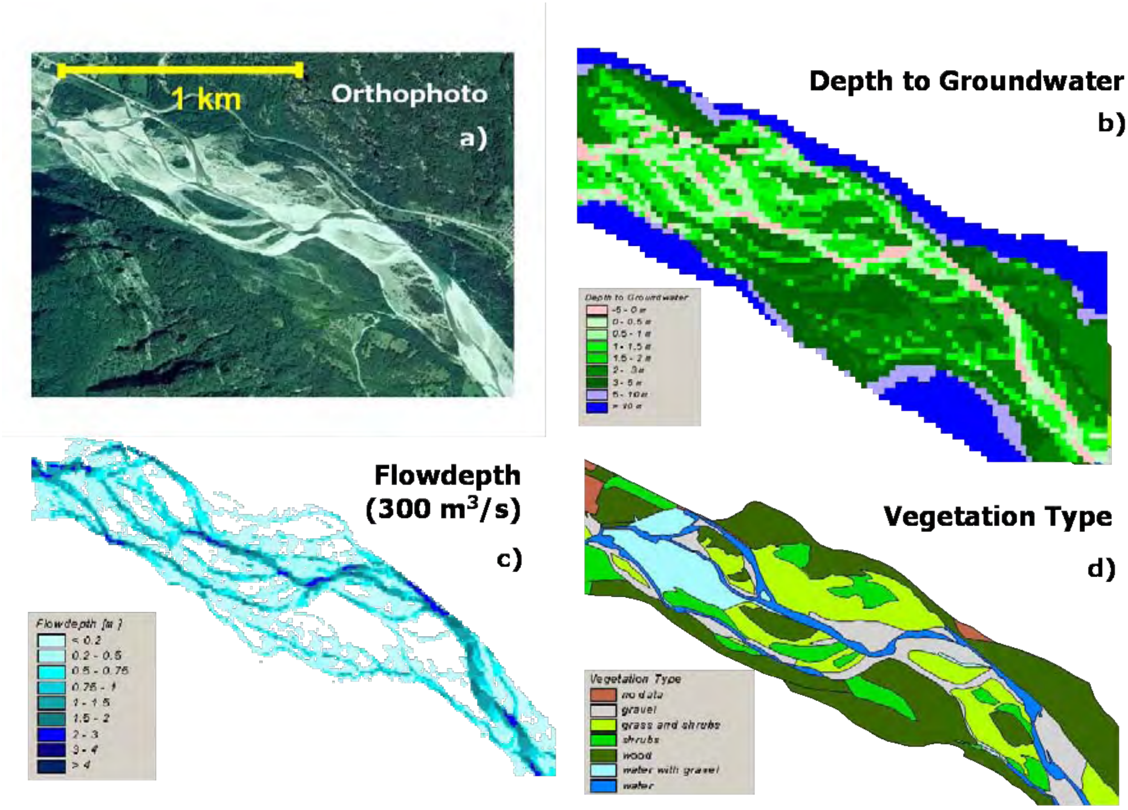


Fig. 7.5: *Comparison of patterns in the braided area: a) aerial photograph, b) depth-to-groundwater, c) inundated area (flow depth) under a discharge of 300 m^3/s, d) vegetation type (corresponds to land surface class).*

7.2.3. Histograms of Flow Depth and Flow Velocity

Especially under the premise of environmental flow requirements it is interesting to see how the flow depth and flow velocities would change in case of an increase in the environmental flow release. These two variables are important not only for the vegetation, but especially for all aquatic habitats in the river. For this purpose, the frequency distributions observed as modelling results of the flow depth and flow velocities of the wet river cells were investigated and are illustrated in histograms (Fig. 7.6). The relative frequency is always related to the same total number of active cells, i.e. the entire valley. Due to the problem of accurately simulating very small flow depths, all surface water cells with a simulated flow depth below 20 cm were not considered and treated as if they were dry cells.

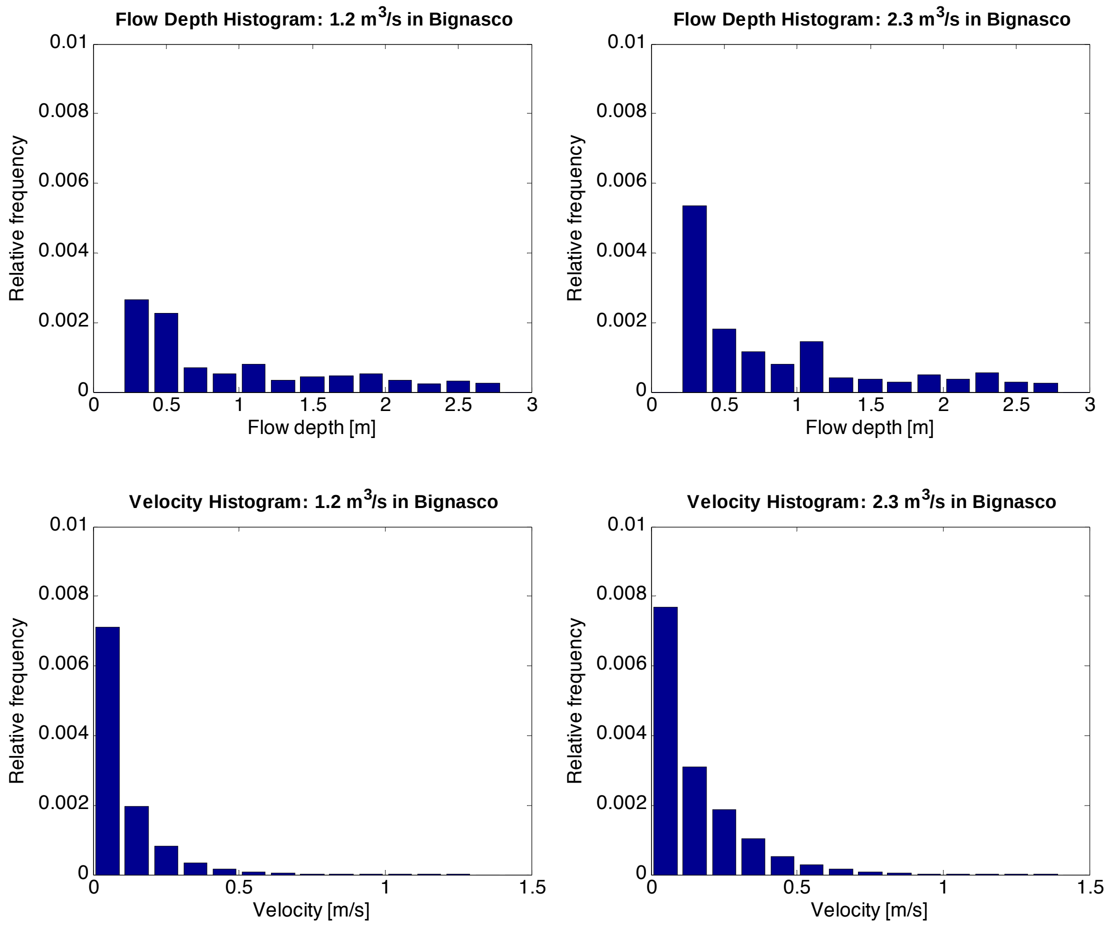


Fig. 7.6 Histograms of flow depth and flow velocity derived from the simulation of different discharges under steady-state conditions. On the left: flow depth, on right: flow velocity.

Besides the fact that the total number of wet cells increases with a higher flow rate, it is illustrated that also the distribution changes. Especially the flow velocity distribution becomes wider in the sense that also also higher velocities occur, therefore a higher variability of aquatic habitats is possible. This effect is different for the flow depth where especially the number of cells with small flow depths show a remarkable increase. This interpretation must be considered cautiously, because of the limitations of the model under low flow conditions. Nevertheless, a tendency is shown. Principally this kind of analysis would be possible also with higher accuracy if the model were adapted especially for low flow conditions as suggested in the outlook.

7.3. Model's Ability to Simulate Vegetation Relevant River – Aquifer Response

In this paragraph, the benefit of the integrated modelling system for the ecological evaluation will be shown. Three aspects will be highlighted: First, the changes in the dynamics of flow, inundated area and shoreline length will be shown. Second, the spatial patterns of inundation, depth-to-groundwater and land surface type will be investigated. Finally, the modelling system will be presented as a tool using transient simulations for investigating the water conditions within the floodplain under given vegetation species.

7.3.1. Changes in Flow, Inundated Area and Shoreline Length (Duration Curves)

Changes in the flow

Streamflow has been altered due to the hydropower operation, as already described (§ 3.2.1). For the analysis, daily stream flow data were used and split into three-periods: I. pre-dam period between 1929 and 1953, where the hydrological regime was still under natural conditions, II. post-dam period, from 1954 until 1977, where no environmental flow was imposed and low flows were smallest, and III. present (1980-2003), which represents the runoff regime under present environmental flow regulations. The annual mean of daily flows has already been illustrated in Fig. 1.2.

Especially the duration of low flow increased due to hydropower operation, but also the dynamics of the runoff system changed. Since the hydropower system is operational, long-lasting low-flow periods are only interrupted by relatively high flows. This means for the present vegetation that only two basic situations occur: absence of inundation with a stable, but relatively deep groundwater table and sudden flooding events. In the pre-dam period, much more differentiated hydrological conditions occurred with more continuous transitions between them, too; it is likely the case that also the sediment supply, especially the fine sediment regime was different. Moreover, frequent bank storage effects could also feed the aquifer and bring the water to the root system of those plants which are situated further away from the river and whose root system does not reach the groundwater table (see also below).

Concept of inundated area and shoreline length for the riverine habitats

As it has been reported in van der Naat et al. (2002), both the temporal evolution of the inundated area as well as the shoreline length play an important role for the establishment and development of morphologically active riverine habitats. Shoreline length was reported as a good indicator for the value of a riverine ecosystem, since the shoreline is the important transition zone between terrestrial and aquatic habitats.

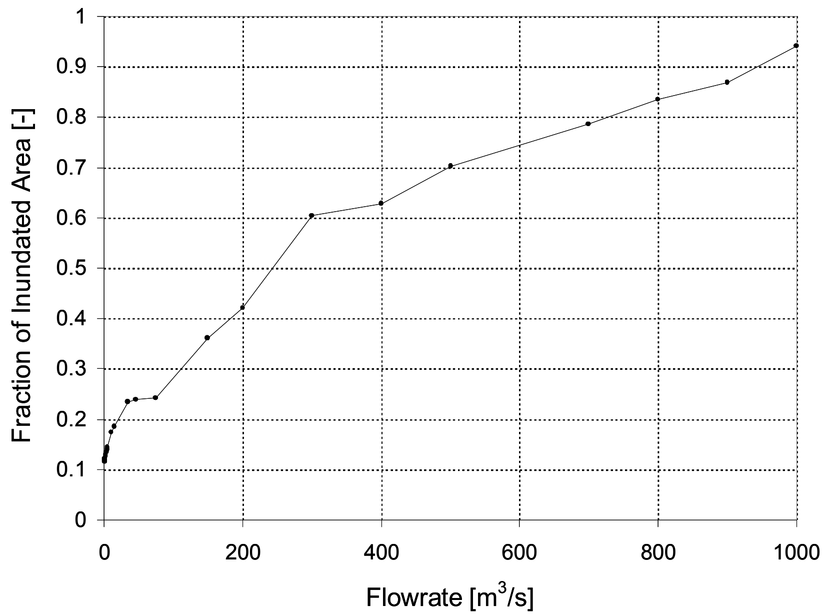
There is a relatively clear relationship between runoff and both inundated area and shoreline length (Fig. 7.7). The hydrological model as it was set up e.g. in the Maggia valley allows for building up these two relationships between flow and inundated area and shoreline length using simulation results instead of requiring field measurement of flooded area under different high streamflow conditions. The spatial distribution of area of inundation was obtained from the simulations using different flow conditions from 1.2 to 1000 m^3/s and was evaluated in the braided area. As a result, inundated area was calculated as a fraction of the entire braided area of investigation, whereas the shoreline length was calculated and normalized on the one hand to the area, on the other to the length of the braided area (two axes in Fig. 7.7).

Inundated area monotonously increases from a minimum value for lowest discharge to the maximum of 1, in case the entire area is flooded under extreme floods. Shoreline length will however show a maximum for intermediate flow conditions. If under low flow only a single channel is active, the relative shoreline length will have a value of two, if there are no meanders in the river. At the other extreme, if the entire floodplain is flooded, a value of two is expected as well, since then the river has again only one margin on each side. For flow conditions in between, braiding and isolated areas will occur. For each additional braiding, relative shoreline length will increase up to a point, where some river channels will be connected again due to additional flooding, because these isolated areas will disappear due to inundation. The maximum in the braided area of the Maggia was found to be at around 300 m^3/s. The dependency of the inundated area and the shoreline length on the flow rate is illustrated in Fig. 7.7.

The results here are consistent with the observations in the Tagliamento river by van der Naat et al. (2002). They obtained quite similar relationships by field measurements in their study. This is also seen as a kind of model validation that our model produces results, which are consistent with the reality. The advantage of modelling these relationships is that the difficult and time-consuming measurements are not necessary. A comprehensive comparison with other river systems is also given by van der Naat et al. (2002).

Duration curves

Flow duration curves are an established tool to assess the non-exceedance frequency of a given flow rate, expressed as number of days per year characterized by such flow rate or lower. Fig. 7.8 shows the changes in the duration curve as elaborated from the historical observation for three different periods, respectively 1929-1953, representing pre-dam conditions, 1954-1977, representing the first hydropower operation period without EFs, and finally 1980-2003, representing the period after the introduction of EFs.



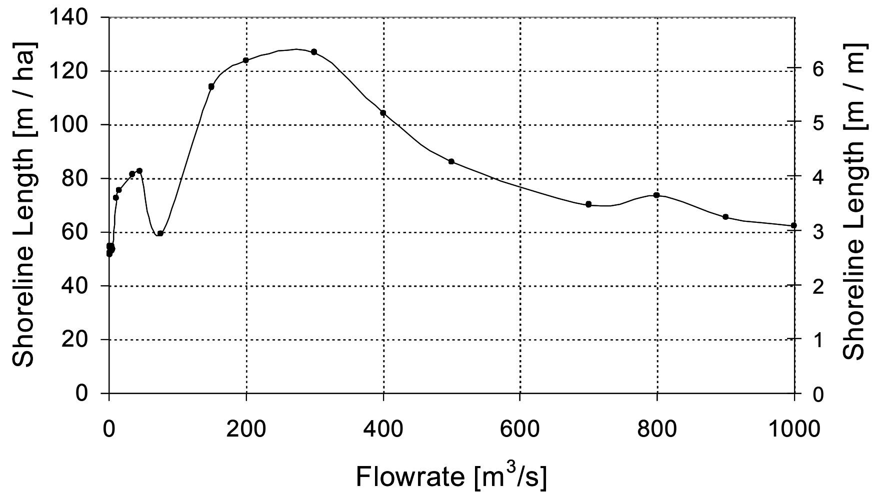


Fig. 7.7: *Relationship between relative inundated area in the braided area (top) resp. relative shoreline length (bottom) from the discharge in the Maggia River.*

Similar curves can be derived on the basis of the model simulations also for the relative inundated area or shoreline length using the relationships between these variables and flow from Fig. 7.7. After the start of the hydropower operation, a significant reduction of the number of days per year, where the fraction of the inundated area is between 0.13 and 0.2, can bee seen in Fig. 7.8b. A similar behaviour is illustrated in Fig. 7.8c for the shoreline length in the range of the 55 to 75 m/ha in the braided area of the Maggia valley.

The information about the temporal variation in runoff, inundated area and shoreline length is probably even more ecologically important, because alpine floodplains are predominantly characterized by the dynamics of the occurring processes. If the system is undisturbed, temporal variability is expected to be high. Fig. 7.9 and Fig. 7.10 illustrate, how these dynamics changed from the pre-dam period to the present post-dam period. For both periods, a typical year (1951 and 2002) with a daily hydrograph in Bignasco was selected (7.8.-a1, 7.9.-a1). Using the relationships from Fig. 7.7 the daily values for the inundated area and shoreline length were derived (a2 and a3). It is shown that in the pre-dam period, mid-range variations of runoff, inundated area and shoreline length (in the temporal scale of approx. 10 days) are interrupted by the variation due to large flood events. In the present period, no variation is observed in the flood intermittent periods.

In order to quantify these variations, the figures in the lower lines (b) of Fig. 7.9 and Fig. 7.10 must be interpreted. They show the empirical relative frequencies of the temporal variation of these three parameters runoff, inundated area and shoreline length. The variation was determined as the difference between the values of the daily time series and the identical time series of a time lag of ten days. The lag of 10 days was chosen because it is characterizes the variations of the natural system better than a time lag of one day. Comparing the years 1951 and 2002, the largest differences in the frequency distribution can be seen for the shoreline length. Whilst in the year 2000 only negligible changes occur (values around zero) throughout most time of the year, large changes occur only with a very low frequency. This is different in the year 1951, where also mid-range changes in shoreline length (values of 0.5 to 1 m/m) occur with a considerably higher frequency. This means that in the pre-dam period there was a number of channels, which was activated with a medium frequency, whilst under present flow conditions, only the main channel is active during most days of the year. The activation and deactivation of the mid-range channels influences not only the aquatic habitats, but also the existence and magnitude of the bank storage. Activated channels are an additional contact zone for river – groundwater interaction, which is thus considerably reduced in space in the present period.

It can be seen that the flow dynamics in the annual hydrograph were much higher in the pre-dam period. Also the duration of moderate flow events were higher. At present, there is only a peaky runoff regime consisting of long periods of constant low flows interrupted by short flood peaks. The reduced temporal variability of streamflow is also reflected in reduced variability of different inundated areas and shoreline length. But furthermore, also dynamics of the changes in these parameters have altered considerably. Whilst in 1951 the differences in shoreline length between ten days was relatively high (Fig. 7.9: bottom right), there are hardly any changes nowadays. This means that in the past, activation and deactivation of river channels was frequent. This also means that the potential areas of bank storage were much larger in the past. This may also affect the groundwater conditions for the vegetation.

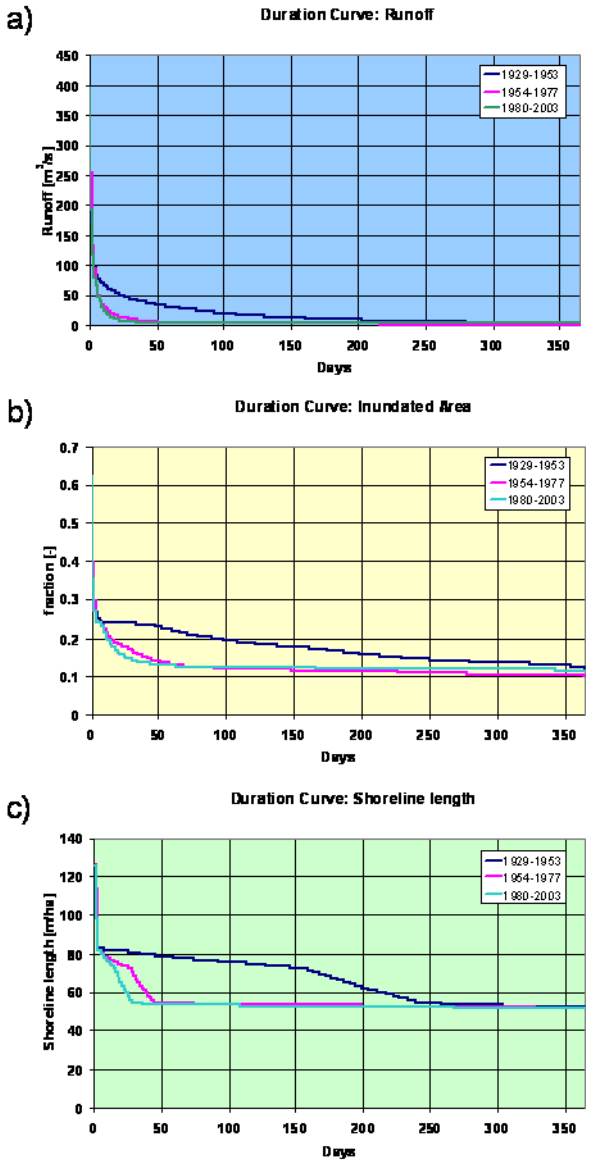


Fig. 7.8: Duration curves for three different periods for the difference in flow (a), inundated area (b) and shoreline length (c) for the braided area. Period 1929-1953 corresponds to the pre-dam period, 1954-1977 to the first post-dam period without environmental flow regulation, 1980-2003 to the second post-dam period with the recent environmental flow requirements.

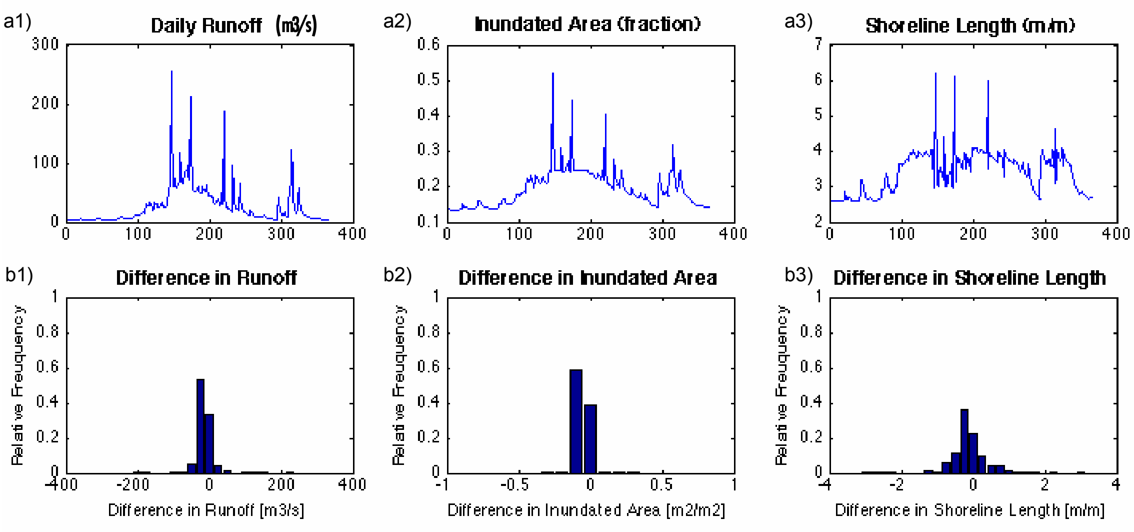


Fig. 7.9: Annual time series for three different periods for the difference in flow (a), inundated area (b) and shoreline length (c) for a time-lag of 10 days, for the braided area. Curves are based on model simulations, and data are from the year 1951 (pre-dam period).

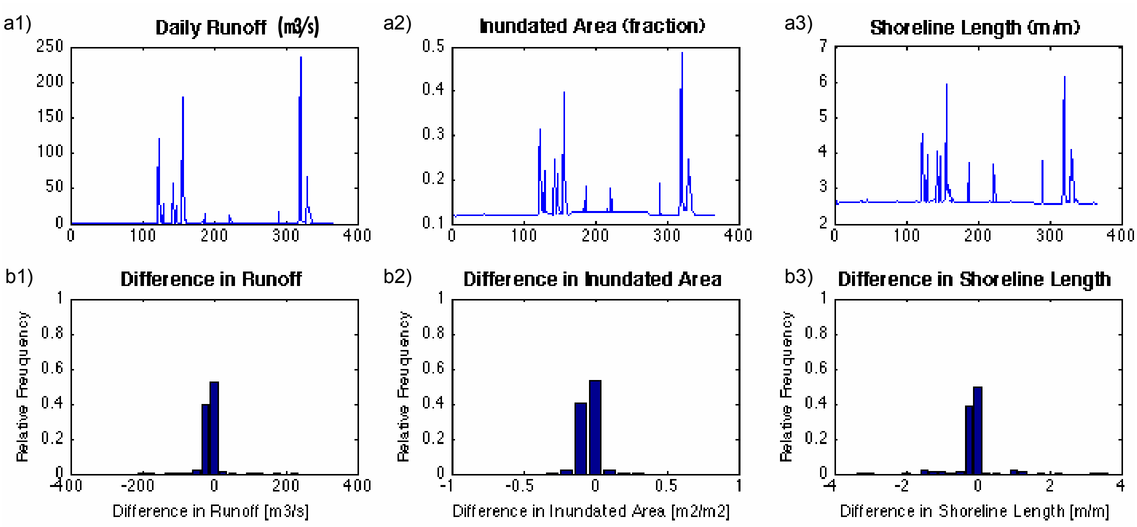


Fig. 7.10: Annual time series for three different periods for the difference in flow (a), inundated area (b) and shoreline length (c) for a time-lag of 10 days, for the braided area. Curves are based on model simulations, and data are from the year 2002 (post-dam period)

7.3.2. Surface Water and Groundwater Level Fluctuations in Relation to Existing Vegetation

In this section, it is shown how the coupled modelling system can be used to identify the oscillations of the surface water and groundwater levels as a consequence of the dynamics of the infiltration – exfiltration process following the changes due to a changing hydrograph. Such an analysis is useful to identify the exposure of the vegetation within the braided area to groundwater stress and flooding conditions. The analysis, here illustrated for an artificial hydrograph, can be extended to a multi-year hydrograph in order to assess the long-term exposure of vegetation to water dynamics.

This vegetation analysis is illustrated for a selected transect within the braided area. Under transient simulations, a rising hydrograph (Fig. 7.11) is used illustrating the steady increase in the inundated areas and the sudden activation of new channels, but also the infiltration of river water into the aquifer and its lateral propagation. Finally, the temporal evolution of the surface water level and the groundwater level together with the ground surface are simultaneously illustrated in Fig. 7.12.

Before examining briefly the potential of this analysis for the Maggia case study, it is necessary to introduce shortly the vegetation characteristics in the braided area, as they can be observed from available aerial photographs and ground data. The most recent aerial photographs (from the year 2001) were taken and the following classes were distinguished: no vegetation, herbaceous vegetation, shrubs, young forest, mature forest. Certain dominating species for all classes were assigned by Sturzenegger (2005), based on earlier studies on vegetation in the Maggia valley (Bayard, 1990; Favre, 2004) (Tab. 7.2).

Tab. 7.2: Vegetation types delineated from aerial photographs and its dominating species (from: Sturzenegger, 2005).

Class	Characteristics	Typical Species
Water	open water surface	
Sediment	exposed sediment	
Herbaceous Plants	pioneer vegetation, grass	Epilobium dodonaei, Myricaria germanica
Shrubs	shrubs, young tress (divided into 4 subclasses depending on the cover density)	Salix eleagnos, Salix pupurea, Fraxinus excelsior
Up-coming Forest	mature shrubs, trees	Alnus incana, Quercus robur, Robinia pseudoaccacia
Mature Forest	mature shrubs, trees	Alnus incana, Quercus robur, Robinia pseudoaccacia
Non-alluvial areas	streets, fields, buildings	

Important for the evaluation of groundwater availability for the plant growth is the depth, up to which water can get into the effective root zone by capillary rise. This "limiting depth-to-groundwater" (in German called: "Grenzflurabstand") can be calculated as the sum of the root depth and the rate of capillary rise of the groundwater under the given soil texture (Arbeitsgruppe Bodenkunde, 1982). The maximum capillary rise in the Maggia valley was estimated to be 50 cm based on literature values under the assumption of coarse sand (Arbeitsgruppe Bodenkunde, 1982; Renger and Strebel quoted in (Hölting, 1989)). As far as it was available, information about the range of root depths for the dominating species was taken from Polomski and Kuhn (1998), as a first approximation. Finally, the sum of maximum capillary rise and upper resp. lower bound of the root depth were summed up in order to obtain maximum and minimum values for the "limiting depth-to-groundwater" (Tab. 7.3). If the groundwater table always fluctuates within this range, the likelihood of water stress is relatively small (under the assumption that values for the root zones are correct).

Tab. 7.3: Root depth and "limiting depth-to-groundwater" for different vegetation classes (with an assumed capillary rise of 0.50 m)

	water, sediment, others	**herbaceous vegetation**	**shrubs**	**young forest**	**mature forest**
min. root depth [m]	0.00	0.70	0.70	0.80	1.10
max. root depth [m]	0.00	1.00	2.50	3.00	6.00
min. "limiting depth-to-groundwater" [m]	0.50	1.20	1.20	1.30	1.60
max. "limiting depth-to-groundwater" [m]	0.50	1.50	3.00	3.50	6.50

The transient simulation was performed with an artificial hydrograph rising from 1.2 m^3/s to 300 m^3/s as shown in Fig. 7.11. Analyzing the results of the transient simulations (Fig. 7.12), one could preliminarily argue that the groundwater table might not be a limiting factor for the existence of present vegetation, since in most cases the plants have probably access to groundwater under the entire range between low flow and extreme flood conditions. This is the case even under consideration of some uncertainties in the modelling results of the groundwater table. The existing vegetation however is the result of a process over decades of years so that the natural system had enough time to adapt to the current water conditions. Whether the vegetation system would have had adapted differently under different groundwater dynamics must be still subject for further studies. The occurrence of specific dominant species (within one vegetation type class) is most likely part of the adaptation process, too. Future investigations could also show, whether a shift in the distribution of particular species has occurred in the past or is likely to occur in the future.

Sensitivity to the groundwater table is expected to be highest for species with a relatively small root depth and during the time of succession, when the root system of the plants is not yet fully developed. In this phase, the role of bank storage effects may be essential, therefore the altered flow system with the reduction in its flow dynamics may become an important factor.

Another point to be evaluated is the sensitivity of the hillslope fluxes since they can change the hydraulic gradient between groundwater head and water level in the river significantly. Due to the high uncertainties in this respect and due to the lack of measurements especially in the braided area, further investigation or at least sensitivity analysis is needed. Understanding of the interaction between sediment transport and vegetation is also required to account for effects such as uprooting due to erosion, seed trapping by deposited sediments and other similar complex mechanisms.

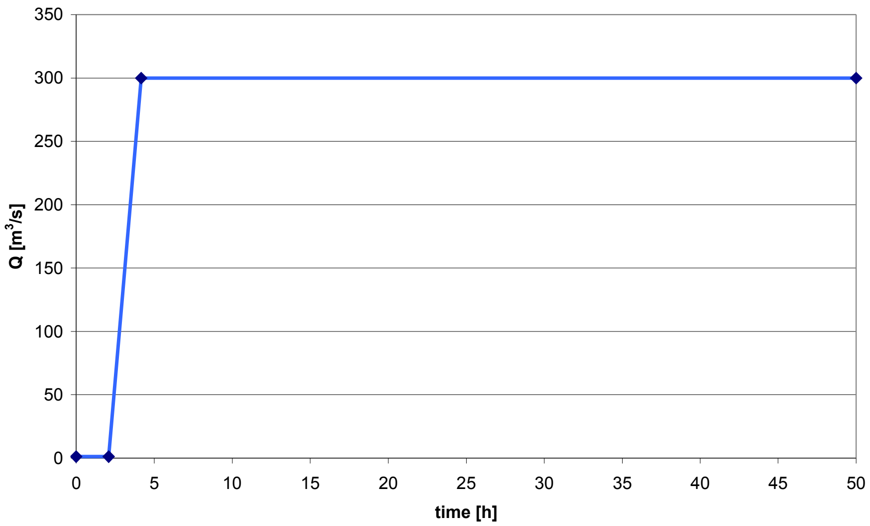


Fig. 7.11: Hydrograph used for the transient simulation to illustrate the relationship between discharge, groundwater level and existing vegetation.

Nevertheless, the coupled modelling system gives already insight into the interplay between surface water and groundwater dynamics and vegetation. The application of this tool has the potential to bring a further understanding of the mechanisms being responsible for the evolution or maintenance of the riverine vegetation. Furthermore, the most important information and essential variables can be taken from this to include as driving factors or boundary conditions for the vegetation growth models currently under development

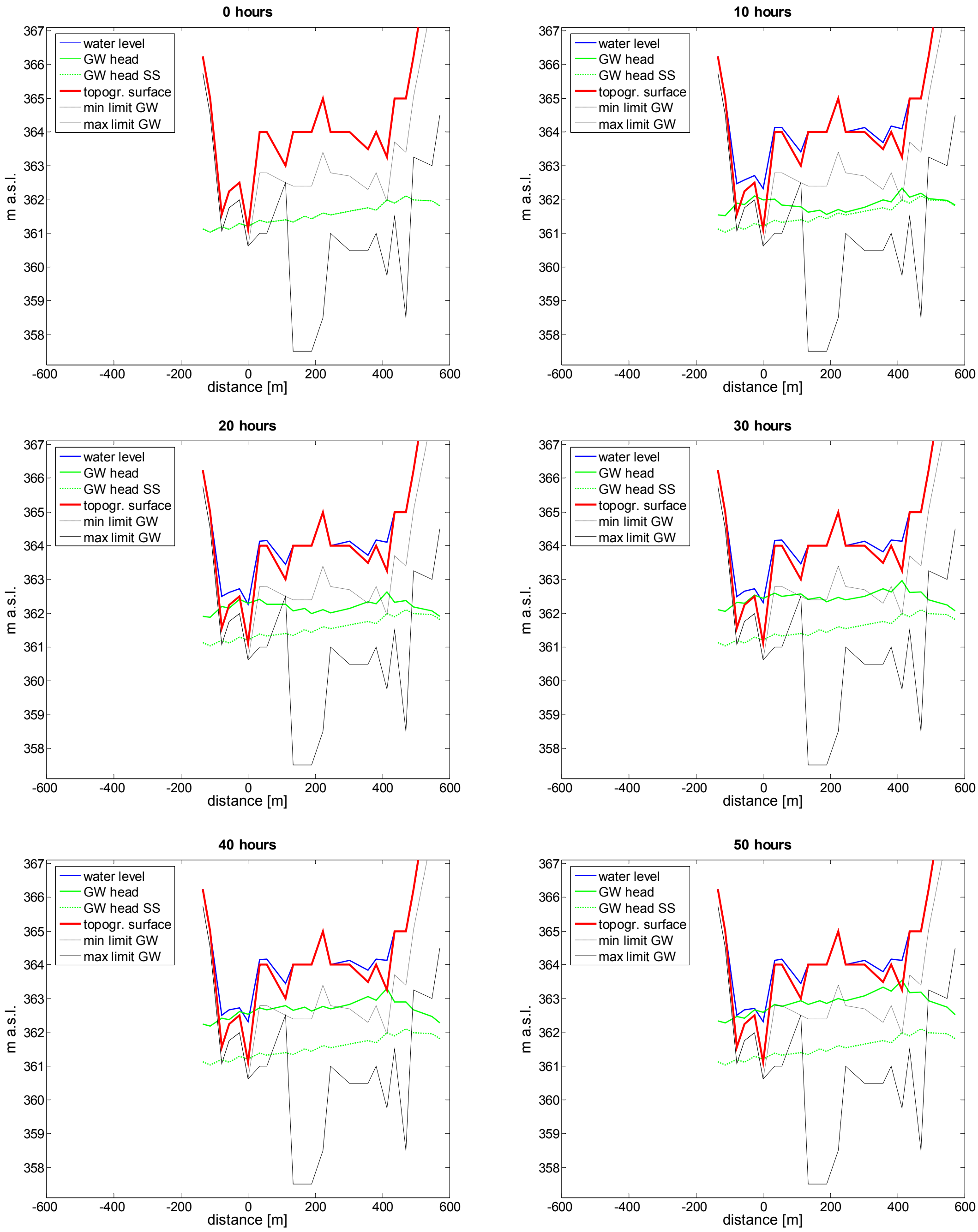


Fig. 7.12: Results of a transient simulations of a rising hydrograph (1.2 to 300 m³/s) in the braided area. Illustrated are the topographic surface, water level of inundated area, groundwater heads (GW head SS is at the beginning of the simulations, i.e. the result of the steady-state simulation; GW head is the actual groundwater level at the specified time), and upper and lower bound of the "limiting depth-to-groundwater" (min limit GW and max limit GW).
The ratio of exaggeration is approx. 100.

8. Concluding Remarks

8.1. Summary

This work was developed in the frame of the MaVal project[34]. The overall aim of the project was to develop a modelling tool for simulating the long-term water flow, i.e. inundation and groundwater table, in an environmentally active floodplain, which is characterized by strong and fast feedback mechanisms between the surface water and groundwater system. The modelling system should provide information about environmentally important hydrological and hydraulic variables such as inundation depth and duration, shear stress and depth-to-groundwater. These parameters are all relevant for the development and growth of riparian vegetation. The Maggia valley is characterized by strong alterations due to hydropower. In the same time, the riparian vegetation appears to have undergone strong alteration. It was expected that the project should also give some indications how environmental flow requirements and their effect on vegetation may be analyzed.

Within this overall framework, the rationale of this work was focused on modelling the interactions between river and groundwater in an environmentally active alpine floodplain, which is characterized by a relatively pristine river morphology with a braided river system and changing shorelines depending on the discharge in the river, and by high dynamics in the aquifer and in the river bed. The groundwater model and the watershed model, both being part of the overall modelling framework, were have already been developed by another study (Foglia, 2006) and could therefore be used as an input. The task was therefore to couple a transient two-dimensional open-channel flow model with a transient quasi-three-dimensional groundwater model, which is

[34] A detailed description can be found under: http://www.maggia.ethz.ch.

universally applicable, but fulfils the requirements posed by the complex conditions of an alpine river.

For this purpose, the open-channel model 2dMb and the groundwater model MODFLOW were used. A special focus had to be drawn to the different grid sizes and temporal resolution of the surface and groundwater model, the drying and wetting of cells of the surface water, and to the application of different equations for the exchange of water in case of a connected or an unconnected aquifer. Additional attention had to be focused on the numerical stability of the mass coupling scheme.

In order to have sufficient data for the calibration and validation, some new monitoring stations were installed in addition to the already existing network, and field campaigns were carried out. An extensive testing of the capability of the surface water model was conducted. Then, calibrations under steady-state conditions were performed for the stand-alone models. Afterwards, the parameters were taken over as far as possible for the coupled model. A sensitivity analysis for the parameters and different boundary conditions was executed, and finally, validation runs with historical hydrographs under transient conditions were performed. With the developed tool, it was possible to show the variations of the hydrological and hydraulic variables in space and time, especially with respect to the patterns of vegetation cover.

8.2. Modelling System Critical Appraisal

The main goal of coupling a transient two-dimensional surface water model with a transient quasi-three-dimensional groundwater model was achieved by this work. Mass coupling was shown to be suitable for this kind of problem, and a good compromise between the applicability of the model and model complexity. On the other hand, it was also shown that a simplified approach, namely the coupling of only the *steady-state* groundwater model with the *transient* surface water model led to some numerical stability problems.

The transient coupled model provides information about hydraulic and hydrological variables in space and time, which are: inundation duration and depth, flow depth and flow velocity, shear stress, Froude number, infiltration and exfiltration rates, groundwater table and depth-to-groundwater. Therefore, a better understanding of complex water systems including feedback mechanism can be gained. The coupled model is universally applicable and therefore also transferable to other study sites, where enough data are available. It can be used, wherever surface water – groundwater interaction is important and surface topography is complex, under the assumption that sediment transport is not the dominating process and does not largely effect the modelling results due to major changes of the river bed. In this case, the sediment transport routine accounting for a mobile bed (§ 8.3.2) has to be included.

Applications are seen especially in river restoration projects. Also for the investigation of riverine habitats, aquatic or terrestrial, the common knowledge about flooding and groundwater table as well as upwelling and downwelling zones is essential. Furthermore, this modelling tool can provide useful information about optimal environmental flow requirements when being applied in an ecological context and an integrated investigation – such as it is on the way also in the Maggia valley within the MaVal project.

As illustrated in the previous chapter, the coupled model is numerically robust under a very large range of flow conditions, namely between 1 m^3/s and 1000 m^3/s. All tests proved the qualitatively proper behaviour in reproducing patterns and time series of flow depth, flow velocity, groundwater heads, and exchange rates with respect to the applied boundary conditions. It is capable to activate and deactivate river channels with altering streamflow and to inundate the floodplain. The simulated exchange rates agree well with measurements under low flow conditions. Quantitatively less accurate is the rate of rise and falling of the groundwater table. Initial groundwater heads under steady-state conditions have already shown substantially differences between observations and simulations, which affect also the transient simulation. One reason could be that the stand-alone implementation of MODFLOW did not consider microscale or mesoscale heterogeneity of the aquifer. Another problem can be seen in the calibration of the coupling relevant parameters, which was strongly limited because of the extreme computational demand of the model.

There are however still some limitations, which are mostly related to the computational effort not being the focus of the study. They restrict so far the simulation to single events or time intervals of some weeks. Also the calibration strategy had to be adapted to this time limitation, because multi-parameter optimization procedures are not feasible. There are however possible ways to overcome these problems, especially in the parallelization of the programme code, as it is discussed in the outlook.

8.3. Outlook

8.3.1. Improvement of Model Capabilities and Process Description

Friction due to vegetation or low flow conditions

Friction is so far only considered as surface roughness, i.e. grain and form roughness. In case of inundation in a floodplain, vegetated areas are often flooded. Here, the equivalent Nikuradse sand roughness is used as a bulk roughness. If only flow depth is of interest, satisfying results can be obtained, but shear stress values are so far reliably reproduced only under the absence of vegetation. This is also the case for flow velocities. In literature, different approaches can be found to separate the friction due to surface roughness and that due to vegetation. Principally, these equations could be implemented into the model, if much more data about the density and properties of the vegetation present in each grid were available. If in further studies, the resistance to flow of vegetation will be investigated, the shear stress at the bed and the resistance due to vegetation are the most important factors. In this case, it is indispensable to include a better formulation of the vegetation roughness (e.g. Baptist, 2006).

As most open-channel flow models, 2dMb is designed mainly for moderate to high flow conditions and has therefore the choice of friction laws which have only a limited applicability under low flow conditions, as already discussed. If only the water level is of interest, the absolute error is relatively small compared to the inaccuracy of the simulated groundwater heads. In case of flow depth, flow velocity or shear stress, the resulting errors may be not negligible. Then a more sophisticated approach for the mathematical description has to be chosen as it can e.g. found in Dittrich and Koll (1997), Bezzola (2002) or Manes et al. (2007).

Mathematical formulation of the surface water – groundwater exchange process

In the present coupled model, a simple Darcy-like equation for the infiltration and exfiltration process is used, assuming the same resistance factor for the river bed both for infiltration and exfiltration. Although this approach is widely used, it is not proved to be correct. Its popularity comes most likely from its easy formulation and the small number of parameters related to it. Sophocleous (2002) suggests also different formulae to describe the river – aquifer exchange process. All of them would however increase the complexity of the coupling structure with the drawback of the risk of numerical complications and the need for a higher data requirement for validation. Detailed measurements and investigations, probably combining different hydrogeological and geophysical methods would therefore be requested in order to decide, which of these formulae might be the best representation under the given circumstances.

Implementation of unsaturated zone

While coupling surface and groundwater, the unsaturated zone has been neglected so far. Different approaches can be found in the literature to couple models for the unsaturated zone with MODFLOW. If necessary, it is principally possible to combine one of these approaches with the coupled model presented here, but it would increase again the level of complexity and would be an additional source for potential numerical instabilities. The largest benefits are seen in case of the groundwater unconnected to the river. So far, it is assumed that the infiltrated water reaches the saturated zone immediately. In a steady-state simulation, this assumption can hold, since in nature, there will be a constant flux from the river through the unsaturated zone below the river toward the aquifer. In a transient case however, this assumption can only be an approximation, the infiltrating water will reach the river in nature always with some delay. The infiltration rates will therefore be correct average over a long time period (probably approx. one year), but the real temporal distribution might have larger errors. But also in this case, detailed investigation would be necessary to verify any more sophisticated approach. In addition to this, the process of bank storage could be described in a more physically based fashion accounting for a correct formulation in the unsaturated zone. Finally, the knowledge about the water content in the unsaturated zone would be beneficial for vegetation related investigations, since the uptake of water by roots could be treated in more detail.

Modelling recharge from hillslopes

In this study, the recharge from hillslopes was provided by the work of *Foglia* (Foglia, 2006) as a result of an inverse modelling procedure under more or less steady-state conditions coupling the watershed model TOPKAPI with the groundwater model MODFLOW using additional runoff measurement data. The contribution from the hillslopes into the aquifer was calibrated as a small proportion of the subsurface flow provided by TOPKAPI. Only the spatial distribution of infiltrating water was directly taken from TOPKAPI. Assuming that no water in the watershed gets lost into the bedrock or by anthropogenic diversion out of the watershed or evapotranspiration, the surface and the subsurface runoff must finally end up in the aquifer. This is due to the special topographic conditions in the Maggia valley with very steep hillslopes of more than 45° and a practically flat valley bottom with very high infiltration capacity, but is also consistent with field observations. This is in contrast to the assumption made during the combined (TOPKAPI and MODFLOW) calibration procedure that only a small part of the subsurface flow should contribute. It was beyond the target of this study to evaluate these hillslope fluxes, but it would be worthwhile to investigate better the values of the hillslope fluxes. Using correct values of this kind of groundwater recharge could probably improve the overall performance of the modelling system. The problem illustrated in this paragraph is to a large extent case specific to the Maggia valley, the question of the determination of lateral hillslope fluxes in general is however still a topic of research, where no standard procedures can so far be applied.

8.3.2. Sediment Transport and Inclusion of Mobile Bed into the Model

So far, sediment transport has not been considered in this study. There were different reasons: The context of environmental flow directs the focus on the changed flow regime. It was therefore a priority to investigate if the dynamics of the surface water and groundwater regime alone can explain the observed changes in the vegetation dynamics. The consideration of the sediment transport and the mobile bed as well as the interaction between sediment and vegetation was therefore seen as an important, but second step. Nevertheless, the validation of the coupled model turned out to be difficult because of local, sometimes very significant changes in the river bed morphology, which could not be accounted for. This supports the idea to include the mobile bed into the modelling system, although the complexity and therefore the number of parameters would increase.

Numerical considerations were another reason for so far neglecting the sediment transport. The model 2dMb has already the possibility of vertical erosion and deposition on the river bed and therefore of simulating mobile beds and accounting for sediment transport, but the computational effort is even manifold higher. Furthermore, a step-wise approach in coupling the models was chosen. Only as soon as the coupled model runs numerically stable it is advisable to include more complexity using a mobile bed.

As a first advancing step, the mobile bed module should be used for event-based simulations, but without coupling with the groundwater. In the special case of the Maggia River, the time intervals of sediment transport are limited to short, but intense flood events. This approach brings already a great potential to overcome some of the shortcomings mentioned above. In a long-term perspective, the mobile bed module could also be implemented into the coupled model. Careful attention has to be paid to numerical issues, since additional sources of complications will probably occur.

Any approach to include a mobile bed requires also additional data. Besides the existing cross-section data, whose measurements could be repeated in the future, there is still a great potential in evaluating the aerial photography. For most of the pictures in the past, overlapping photos are available, which allow a stereoscopic evaluation. This includes also the generation of a digital surface model[35]. A digital terrain model (i.e. without buildings and vegetation), can be derived as a succeeding step from the digital surface model. Despite it is not a straightforward procedure in this environment, it is probably worth to do it. The open sediment areas, which are probably the most active and therefore the most important areas with respect to sediment transport issues, are anyhow not affected by this problem because of the absence of vegetation.

8.3.3. Computational Requirements

So far, the modelling system as it is presented here is computationally very demanding. As an example, under flooding conditions, computational time was about threefold the length of the real world simulation period on an Intel-base personal computer with a 2.3 GHz processor and 2 GB RAM. This is especially due to the surface water model 2dMb, which is the limiting factor with respect to long-term simulations. So far, the application is limited to either selected time periods, relatively small domains or to conditions, under which a relatively coarse 2dMb grid can be used in order to reduce the number of cells and to prolong the possible time steps. None of the three factors were given in the Maggia valley. The measures to overcome the problem of long

[35] A digital surface model represents the visible earth surface, i.e. - were present - the upper bound of the canopy of vegetation and the top of buildings.

computing times can be divided into the following groups: hardware, software, changes in the modelling framework structure, and simplification of the modelling processes.

Hardware

The computer technology is still under big development. As in the past, the power of new computer generations can still be increased by means of higher processor frequencies, introduction of 64-bit technology and others. This would be, of course, beneficial for the application of the modelling system presented here. Faster simulation may also be possible using computers with larger RAM (1 GB or 2GB used in this study).

Software

Parallelizing the computer code of 2dMb

Up to now, there is no advantage of using computer clusters for saving computation time, since the code of 2dMb is not build for the use on parallel nodes. All calculations have to be performed serially. The basic structure of the code would, however, principally allow its parallelization. Herein lies probably by far the highest potential in accelerating the computing time. This step should therefore be undertaken with highest priority.

Unstructured grid

Very small grid sizes are required only under low flow conditions. Under low flow, only a small proportion of the modelling domain is wet and hence active in the simulation of 2dMb. These relatively few cells do nevertheless determine the number of cells for the entire domain leading to a very fine grid resolution everywhere. If the refinement of cells could be restricted to the wet river zone under low flow conditions, the total number of cells could be largely reduced. In the general case, as it is also in the Maggia Valley, refinement just in certain rows or columns is not feasible because of bending river beds. One possibility could be the use of a block-centred grid. This would however require substantial changes in the code. Another way to refine the grid is the use of unstructured grids, as it had originally been developed for finite element methods. This is also possible for the finite volume method. The internal structure of the model is however more complex, and in this case, a complete rebuilding of the code is necessary. This has recently be done at the Laboratory of Hydraulic, Hydrology and Glaciology (VAW) at ETH Zurich, using C++ as programming language and having transferred the model 2dMb into the newly developed model BASEMENT (VAW, URL). Despite the reduced number of cells necessary to cover the modelling domain, the calculation time is still significantly higher than with 2dMb. The reason for this are not yet clear. (Fäh, pers. comm.).

One point has anyhow to be considered when refining the grid locally. The small cells will determine the time step for the overall simulation in the entire domain because of the CFL-criterion. Only in the case of the block-structured grid, small time-steps could be used just for single blocks, the result then integrated over space and time to provide the boundary conditions for the coarse overall grid.

Modelling framework structure

So far, the entire modelling domain for the surface water has been modelled using a 2D approach. In many cases, this would not be necessary, so also in the Maggia Valley, where a large part of the river upstream and downstream of the braided area consists of a single channel, which would allow a 1D-simulation. Therefore, a longitudinal coupling of a 1D and 2D surface water model could be used in case of the Maggia Valley. This would require a full coupling between MODFLOW with both the 1D surface water model and the 2dMb. Additionally, the outflow of the upper 1D model would be the upper boundary condition for the 2D model in the central part of the valley, and similarly, the outflow of the 2D model would be the input or upper boundary of the lower 1D model.

LITERATURE

Abbott, M.; J.C. Bathurst, J.A. Cunge; P.E. O'Connell; J. Rasmussen (1986a): An introduction to the European Hydrological System - Système Hydrologique Européen, "SHE", 1: History and philosophy of a physically based distributed modeling system, *Journal of Hydrology*, 87(1-2), 45-59.

Abbott, M.; J.C. Bathurst, J.A. Cunge; P.E. O'Connell; J Rasmussen (1986b): An introduction to the European Hydrological System - Système Hydrologique Européen, "SHE", 2: (1986b): Structure of a physically-based, distributed modelling system, *Journal of Hydrology*, 87(1-2), 61-77.

Anastasi, G. (1990): *Studio per la sistemazione del fondovalle Vallemaggia*, Locarno, report for Cantone Ticino, *unpublished*.

Anderson, E.I. (2003): An analytical solution representing groundwater-surface water interaction, *Water Resources Research*, 39(3); doi:10.1029.2002WR001536, Art. No. 1071.

Arbeitsgruppe Bodenkunde der geologischen Landesämter und der Bundesanstalt für Geowissenschaften und Rohstoffe in der Bundesrepublik Deutschland (1982): *Bodenkundliche Kartieranleitung*, Hannover, 3rd edition, 331 p.

Arnold, J.G.P.; M. Allen; G. Bernhardt (1993): A comprehensive surface groundwater flow model, *Journal of Hydrology*, 142, 47-69.

Baptist, M.J. (2006): Modelling the influence of vegetation on the morphodynamics of a flood event in the Allier, France, *Geophysical Research Abstracts*, SRef-ID: 1607-7962/gra/EGU06-A-02516, 02516.

Baptist, J.; V. Babovic; J. Rodriguez Uthurburu; M. Keijzer; R.E. Uittenbogaard; A. Mynett; A. Verwey: On inducing equations for vegetation resistance, *Journal of Hydraulic Research*, *in press*.

Bayard, M.; F.H. Schweingruber (1991): Trees at their limit on the riverbed of the Maggia in the Canton of Ticino, Switzerland: A dendroecological study, *Botanica Helvetica*, 101, 9-28.

Bayard, M. (1990): *Auswirkungen von Hochwassern auf Holzpflanzen und ihre Standorte in der Maggiaaue*, Botanisches Institut, Universität Basel, Diploma Thesis.

Bear, J. (1979): *Hydraulics of Groundwater*, McGraw-Hill, New York et al., 569 p.

Beatrizotti, G. (1973): *Valle Maggia: modello di simulazione del deflusso delle acque sotterranee*, Ufficio Geologico Cantonale, Bellinzona, report for Cantone Ticino, *unpublished*.

Beffa, C. (1994): Praktische Lösung der tiefengemittelten Flachwassergleichungen, *Mitteilungen der Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie der Eidgenössischen Technischen Hochschule Zürich*, 133, 128 p.

Beffa, C. (2004): *Flachwassergleichungen: Grundlagen - Lösungen - Anwendungen*; Vorlesung Hydraulik II, Institut für Hydromechanik und Wasserwirtschaft, ETH Zürich, Schwyz, Zürich, lecture notes, *unveröffentlicht*.

Beffa, C.; R.J. Connell (2001): Two-dimensional flood plain flow. I. Model description, *Journal of Hydrological Engineering*, 6(5), 397-405.

Beffa, C.; R.J. Connell (2002): Two-dimensional flood plain flow. I. Model description (vol 6, pg 397, 2001: Correction), *Journal of Hydraulic Engineering*, 7(2), 194-194.

Beven, K.J. (2001): *Rainfall-runoff modelling: the primer*, John Wiley, New York, 360 p.

Bezzola, G.R. (2002): Fliesswiderstand und Sohlenstabilität natürlicher Gerinne: unter besonderer Berücksichtigung des Einflusses der relativen Überdeckung, *Mitteilungen der Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie der Eidgenössischen Technischen Hochschule Zürich*, 173, 258 p.

Bezzola, G.R. (2003): Vorlesungsmanuskript *Flussbau*, Fassung WS 03/04, Professur für Wasserbau, ETH Zürich, Stand 18. Oktober 2003, lecture notes, *unpublished*.

Brunke, M.; T. Gonser (1997): The ecological significance of exchange processes between rivers and groundwater, *Freshwater Biology*, 37, 1-33.

Burgherr, P; J.V. Ward (2001): Longitudinal and seasonal distribution patterns of the benthic fauna of an alpine glacial stream (Val Roseg, Swiss Alps), *Freshwater Biology*, 46(12), 1705-1721.

Cerini, U. (2003): *Le fond de vallée du Valmaggia entre Bignasco et Giumaglio*, Faculté des Lettres, Institut de Géeographie, Université de Lausanne, Diploma Thesis, 164 p.

Chaudhry, M.H. (1993): *Open-Channel Flow*, Prentice Hall, Eaglewood Cliffs.

Chiang, W.-H.; W. Kinzelbach (2001): *3D-groundwater modeling with PMWIN: a simulation system for modeling groundwater flow and pollution*, Springer, Berlin, 346 p.

Chow, V.T. (1973): *Open-Channel Hydraulics*, McGraw-Hill Book Company, Auckland, International Student Edition, 680 p.

Chung, I.M.; N.W. Kim; J. Lee; Y.S. Won (2006): *Proc. HydrEco'2006 International Conference on Hydrology and Ecology*: the Groundwater/Ecology Connection, 11-14 September 2006, Karlovy Vary, Czech Republic; Integrated modelling of surface water and groundwater by using combined SWAT-MODFLOW model, 199-202.

Ciarapica, L.; E. Todini (2002): TOPKAPI: a model for the representation of the rainfall-runoff process at different scales, *Hydrological Processes*, 16(29), 207-229.

Courant, R.; K. Friedrichs; H. Lewy (1928): Über die partiellen Differentialgleichungen der mathematischen Physik, *Mathematische Annalen*, 100(1), 32-74.

Crowder, D.W.; P. Diplas (2000): Using two-dimensional hydrodynamic models at scales of ecological importance, *Journal of Hydrology*, 230, 172-191.

Cunge, J.A; F.M. Holly; A. Verwey (1980): *Practical Aspects of Computational Hydraulics*, Pitman Advanced Publishing Program, Boston, London et al., 420 p.

Cunge, J. (1969): On the subject of a flood propagation computation method (Muskingum method), *Journal of Hydraulic Research*, 21, 1132-1140.

DHI (ed.) (2004): MIKE 11: *A Modelling System for Rivers and Channels*; Reference Manual; DHI Software.

Disse, M. (1995): Modellierung der Verdunstung und der Grundwasserneubildung in ebenen Einzugsgebieten, Institut für Hydromechanik und Wasserwirtschaft, Universität Karlsruhe (TH), *IHW*, 53, 180 p.

Dittrich, A.; K. Koll (1997): Velocity field and resistance of flow over rough surfaces with large and small relative submergence, *International Journal of Sediment Research*, 12(3), 21-33.

Du, Ch. (1998): Ein Finite-Punkt-Verfahren für stationäre zweidimensionale Strömungen mit freier Oberfläche, *Mitteilungen des Institutes für Wasserbau und Kulturtechnik der Universität Karlsruhe* (TH), "Theodor-Rehbock-Wasserbaulaboratorium", 197, 145 p.

Faeh, R. (1996): Erosion-based dambreak simulation, *Proc. Second Int. Conference on Hydroinformatics '96*, vol. 2, 681-688.

Fäh, R. (1997): Numerische Simulation der Strömung in offenen Gerinnen mit beweglicher Sohle, *Mitteilungen der Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie der Eidgenössischen Technischen Hochschule Zürich*, 153, 161 p.

Faeh, R. (2007): Numerical modeling of breach erosion of river embankments, *Journal of Hydraulic Engineering*, 133(9), 1000-1009.

Fäh, R.; G.R. Bezzola (2004): Hochwasservorsorge - Erkenntnisse aus vier Fallbeispielen; Teilbericht Gefahrensituation, *Berichte des BWG*, Serie Wasser, Bern.

Favre, V. (2004): *Evolution of the Maggia floodplain: Analysis of an aerial photographs time series from 1962 to 2001*, Laboratoire de Biologie de la Conservation, Université de Lausanne, Diploma Thesis, 76 p.

Fehr, R. (1987): Geschiebeanalysen in Gebirgsflüssen, *Mitteilungen der Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie der Eidgenössischen Technischen Hochschule Zürich*, 92, 139 p.

Fehr, R. (1987): Einfache Bestimmung der Korngrössenverteilung von Geschiebematerial mit Hilfe der Linienzahlanalyse, *Schweizer Ingenieur und Architekt*, 38, 1104-1109.

Fette, M.W. (2005): *Tracer studies of river-groundwater interaction under hydropeaking conditions*, ETH Zürich, Doctoral Thesis, 117 p; http://e-collection.ethbib.ethz.ch/show?type=diss&nr=16103.

Foglia, L. (2006): *Alternative groundwater models to investigate river-aquifer interactions in an environmentally active alpine floodplain*, ETH Zürich, Doctoral Thesis; http://e-collection.ethbib.ethz.ch/show?type=diss&nr=16103.

Foglia, L.; W. Ruf; P. Perona; R. Faeh; P. Molnar; P. Burlando (2005): Improved hydrodynamic modelling tools for river-groundwater interaction, *Geophysical Research Abstracts*, 7, 06582, SRef-ID: 1607-7962/gra/EGU05-A-06582.

Foglia, L.; S.W. Mehl; M.C: Hill; P. Perona; P. Burlando (2007): Testing alternative ground water models using cross-validation and other methods, *Ground Water*, 5, 627-641; doi:10.1111/j.1745-6584.2007.00341.x

Freeze, R.A.; R.L. Harlan (1969): Blueprint for a physically-based, digitally-simulated hydrologic response model.

Graham, D.; O. Larsen (2002): MIKE-SHE, ein dynamisches Modellsystem für den vertikalen Austausch zwischen Grundwasser und Oberflächenwasser, Forum für Hydrologie und und Wasserbewirtschaftung, Wechselwirkung zwischen Grundwasserleitern und Oberflächengewässern, *Beiträge zum Tag der Hydrologie 2002*, 20.-22. März in Suderburg, Lüneburger Heide, 184-189.

Gunduz, O.; M.M. Aral (2005): River networks and groundwater flow: a simultaneous solution of a coupled system, *Journal of Hydrology*, 301, 216-234.

Harbough, A.W.; E.R. Banta; M.C. Hill; M.G. McDonald (2000): MODFLOW-2000, The U.S. Geological Survey Modular Ground-Water Model - User guide to modularization concepts and the ground-water flow process, *U.S. Geological Survey Open-File Report* 00-92, 119 p.

Hill, M.C.; E.R: Banta; A.W. Harbaugh; E.R. Anderman (2000): MODFLOW-2000, The U.S. Geological Survey modular ground-water model - User's guide to the observation, sensitivity, and parameter-estimation processes, *U.S. Geological Survey Open-File Report* 00-184, 209 p.

Holocher, J.; V. Matta; W. Aeschbach-Hertig; U. Beyerle; M. Hofer; F. Peeters; R. Kipfer (2001): Noble gas and major element constraints on the water dynamics in an alpine floodplain, *Ground Water*, 39(6), 841-852.

Hölting, B. (1989): *Hydrogeologie*: Einführung in die Allgemeine und Angewandte Hydrogeologie, Enke, Stuttgart, 396 p.

HydroGeoLogic, Inc.(URL): Software MODHMS, <http://www.modhms.com/software/modhms.html>, download: 13 Oct, 2007.

Jasper, K.; J. Schulla (URL): WaSim: Model Homepage, <http://homepage.hispeed.ch/wasim/index.html; download: July 29, 2007, 17:00>.

Johnson, W.C.; M.D. Dixon; R. Simons; S. Jenson; K. Larson (1995): Mapping the response of riparian vegetation to possible flow reductions in the Snake river, Idaho, *Geomorphology*, 13, 159-173.

Jones, M. (1997): NCF: A finite-element computer program to simulate ground-water flow within the U.S. Geological Survey Modular Ground-Water Flow Model (MODFLOW), *Ground Water*, Computer Notes, 35(4), 721-723.

King, I.P. (1990): *Program Documentation RMA-V2*. Two dimensional finite element model for flow in estuaries and streams, Version 4.3, Resource Management Associates, Lafayette, CA.

Kinzelbach, W. (1986): *Groundwater modelling*: an introduction with sample programs in Basic, Elsevier, Amsterdam, 343 p.

Kirchhofer, W.; B. Sevruk (1992, 1995, 1997, 1999, 2001): Mittlere jährliche korrigierte Niederschlagshöhen 1951-1980, Landeshydrologie, Bundesamt für Wasser und Geologie, Hydrologischer Atlas der Schweiz, Bern.

Kowalski, R.L.; P.M. Schröder; T. Kaluza (2006): Pflanzen in der 2D Simulation von Flüssen, Strömungssimulation im Wasserbau - Flow Simulation in Hydraulic Engineering; *Dresdner Wasserbaukolloquium am 09. und 10. März 2006*, Fakultät für Bauingenieurwesen, Institut für Wasserbau und Technische Hydromechanik, Technische Universität Dresden, 32, 399-406.

Krause, S.; A. Bronstert (2002): Modellierung des Wasserhaushaltes im Gebiet der Unteren Havel: Möglichkeiten der Darstellung der Grundwasser-/Oberflächenwasser - Interaktionen mittels Modellkopplung, Forum für Hydrologie und Wasserbewirtschaftung, Wechselwirkung zwischen Grundwasserleitern und Oberflächengewässern, *Beiträge zum Tag der Hydrologie 2002*, 20.-22. März in Suderburg, Lüneburger Heide, 83-88.

Lindner, K. (1982): *Der Strömungswiderstand von Pflanzenbeständen*, Fachbereich für Bauingenieur- und Vermessungswesen, TU Braunschweig, Doctoral Thesis, 234 p.

Liu, Z; E. Todini (2002): Towards a comprehensive physically-based rainfall-runoff model, *Hydrology and Earth System Science*, 6, 859-881.

Loage, K.; C.S. Heppner; R.H. Abrams; A.E. Carr; J.E. VanderKwaak; B.A. Ebel (2005): Further testing of the Integrated Hydrology Model (InHM): event-based simulations for a small rangeland catchment located near Chickasha, Oklahoma, *Hydrological Processes*, 19(7), 1373-1398.

Malard, F.; K. Tockner; J.V. Ward (1999): Shifting dominance of subcatchment water sources and flow paths in a glacial floodplain, Val Roseg, Switzerland, *Arctic Antarctic and Alpine Research*, 31(2), 135-150.

Manes, C.; D. Pokrajac; I.McEwan (2007): Double-averaged open-channel flows with small relative submergence, *Journal of Hydraulic Engineering*, August, 896-904.

Matthess, G.; K. Ubell (2003): *Allgemeine Hydrogeologie - Grundwasserhaushalt*, Lehrbuch der Hydrogeologie, 2nd edition, Borntraeger, Berlin, 438 p.

Matz, S.; M. Pätsch; O. Larsen; H. Weis (2006): Integrierte Modellierung eines Flusseinzugsgebietes am Beispiel der Oberen Iller, Strömungssimulation im Wasserbau - Flow Simulation in Hydraulic Engineering; *Dresdner Wasserbaukolloquium am 09. und 10. März 2006*, Fakultät für Bauingenieurwesen, Institut für Wasserbau und Technische Hydromechanik, Technische Universität Dresden, 32, 67-74.

McDonald, M.G.; A.W. Harbaugh (1986): A modular three-dimensional finite-difference ground-water flow model, *U.S. Geological Survey Techniques of Water-Resources Investigations* Book 6, Chapter A1, 586 p.

Menzel, L.; H. Lang; M. Rohmann (1992, 1995, 1997, 1999, 2001): Mittlere jährliche Verdunstungshöhen 1973-1975, Landeshydrologie, Bundamt für Wasser und Geologie, *Hydrologischer Atlas der Schweiz*, Bern.

Nieuwenhuis, G.J.A. (1986): Integration of remote sensing with a soil water balance simulation model (SWATRE), ICW Technical Bulletins. Institute for Land and Water Management Research, Wageningen, ITC Technical Bulletins, New Series, 59, 119-128.

Panday, S.; P.S. Huyakorn (2004): A fully coupled physically-based spatially-distributed model for evaluating surface/subsurface flow, *Advances in Water Resources*, 27, 361-382.

Peyrard, D.; S. Sauvage; M. Quintard; J.M. Sanchez-Perez; Ph. Vervier (2006): *Proc. HydroEco'2006*, International Conference on Hydrology and Ecology: The Groundwater/Ecology Connection, 11-14 September 2006, Karlovy Vary, Czech Republic; Modelling surface/subsurface flow interactions using a coupled Saint Venant/Depuit model: application to a large river, 217-220.

Pfammatter, S.; P. Zanetta (2003): *Hydrogéologie de la plaine alluviale du Valmaggia entre Bignaso et Giumaglio (Tessin)*, Institut de Géeophysique, Institut de Minéralogie et Géochimie, Université de Lausanne, Diploma Thesis, 208 p.

Pinder, G.F.; S.P. Sauer (1971): Numerical simulation of flood wave modification due to bank storage effects, *Water Resources Research*, 7(1), 63-70.

Poeter, E.P.; M.C. Hill; E.R. Banta; S. Mehl; S. Christensen (2005): *UCODE_2005 and six other Computer codes for universal sensitivity analysis, calibration, and uncertainty evaluation*, U.S. Geological Survey Techniques and Methods 6-A11, 283 p.

Poeter, E.P.; M.C. Hill; E.R. Benta: MMA: a computer code for multimodel analysis; *U.S. Geological Survey Techniques and Methods* 6-E2, *in press*.

Poff, N.L.; J.D. Allan; M.B. Bain; J.R. Karr; K.L. Prestegaard; B.D. Richter; R.E. Sparks; J.C. Stromberg (1997): The natural flow regime: A paradigm for river conservation and restoration, *BioScience*, 47, 769-784.

Polomski, J.; N. Kuhn (1998): *Wurzelsysteme*, Paul Haupt, Bern, 290 p.

Prudic, D.E.; L.F. Konikov; E.R. Banta (2004): A new streamflow-routing (SFR1) package to simulate stream aquifer interaction with MODFLOW-2000, *U.S. Geological Survey Open-File Report* 88-729, 113 p.

Rampazzi, F.; G. Carraro; P. Gianoni; A. Focarile; B. Jann; N. Patocchi (1993): *Società Ticinise di Scienze naturali*, Studio naturalistice del fondovalle vallemaggese, Memoire, vol. III, Lugano, 348 p.

Ranieri, M. (2006): *Simulazione di correnti a superficie libera mediante modelli bidimensionali ai volumi finiti: il caso della Valle Maggia*, 1. Facoltà di Ingegneria, Politecnico di Torino, Diploma Thesis, 121 p.

Rembe, M.; D. Wenske (1998): Die Seerandbedingung - eine zusätzliche Randbedingung für das modulare dreidimensionale Grundwasserströmungsmodell MODFLOW, in: Thiergärtner, H.: *Mathematische Grundwassermodellierung - unkonventionelle Lösungen und Randbedingungen*, Band 2, CPRESS-Verlag, Dresden.

Renger, M.; O. Strebel (1982): Einfluss der Grundwasserabsenkung auf landwirtschaftliche Erträge; *4. DVWK-Fortbildungskurs Grundwasser*, 1.-14.10.82, Bonn.

Ruf, W.; L. Foglia; P. Perona; P. Molnar; R. Faeh; P. Burlando: Modelling the interaction between groundwater and river flow in an active alpine floodplain ecosystem, *Peckiana*, *in press*.

Rushton, K.R.; S.C. Redshaw (1979): *Seepage and groundwater flow-numerical analysis by analog and digital methods*, John Wiley and Sons, New York, 339 p.

Schaffranek, R.W.; R.A. Baltzer; D.E. Goldberg (1981): A model for simulation of flow in singular and interconnected channels; *U.S. Geological Survey Techniques of Water Resources investigations*, book 7, chapter C3, 110 p.

Schulla J.; K. Jasper (URL): Model Description WaSim-ETH, <http://homepage.hispeed.ch/wasim/download.html; download: July 29, 2007, 16:50>.

Schulla, J. (1997): Hydrologische Modellierung von Flussgebieten zur Abschätzung der Folgen von Klimaänderungen, *Zürcher Geographische Schriften*, Geographisches Institut, ETH Zürich, 69, 161 p.

Singh, V.P. (ed.) (2006): *Watershed models*, Taylor&Francis, Boca Raton, FL, 653 p.

Sommer, Th.; C. Schmidt; B. Klöcking; B. Pfützner (2002): Kopplung eines Grundwassermodells auf der Basis MODFLOW mit dem Einzugsgebietsmodell ArcEGMO, Forum für Hydrologie und Wasserbewirtschaftung, Wechselwirkung zwischen Grundwasserleitern und Oberflächengewässern, *Beiträge zum Tag der Hydrologie 2002*, 20.-22 März in Suderburg, Lüneburger Heide, 199.

Sophocleous, M. (2002): Interactions between ground water and surface water: the state of the science, *Hydrogeology Journal*, 10, 52-67.

Sturzenegger, M. (2005): *Changes in river morphology and vegetation caused by streamflow regulation: A case study in the Maggia valley, TI, Switzerland*, Institut für Hydromechanik und Wasserwirtschaft, ETH Zürich, Diploma Thesis, 39 p.

Sudicky, E.A. (URL): HydroGeoSphere: A 3D model for simulating flow and contaminant transport in integrated surface-subsurface flow systems, <http://cimh.edu.bb/Projects/Sudicky%20Barbados%20HGS-1.pdf>, download: 13 Oct, 2007.

Swain, E.D. (1996): Implementation and use of direct-flow connections in a coupled ground-water and surface-water model, *Ground Water*, 32(1), 139-144.

Swain, E.D.; E.J. Wexeler (1996): A coupled surface-water and ground-water flow model (MODBRNCH) for simulation of stream-aquifer interaction; *U.S. Geological Survey Techniques of water-resources investigations*, book 6, chapter A6, 125 p.

Thorens, A.; C. Mauch (2002): *Case Study 1: Maggia Valley*, EUWARENESS research project on European Water Regimes and the Notion of a Sustainable Status, Institut de Hautes Études en Administration publique (IDHEAP), Chavannes-près-Renens, 30 p; www.unil.ch/idhea/index.htm.

Tockner, K.; F. Malard; P. Burgherr; C.T. Robinson; U. Uehlinger; R. Zah; J.V. Ward (1997): Physico-chemical characterization of channel types in a glacial floodplain ecosystem (Val Roseg, Switzerland), *Archiv für Hydrobiologie*, 140(4), 433-463.

Todini, E.; L. Ciarapica (2001): The TOPKAPI model, in: V.P. Singh et al. (eds.): *Mathematical models of large watershed hydrology*, chapter 12, Littleton, Colorado.

Uehlinger, U.; F. Malard; J.V. Ward (2003): Thermal patterns in the surface waters of a glacial river corridor (Val Roseg, Switzerland), *Freshwater Biology*, 48(2), 284-300.

US Army Corps of Engineers (2001): *HEC-HMS, Hydrologic Modeling System*, Version 2.1.3, Davis, California.

U.S. Geological Survey (URL_a): Abstract for model MODBRNCH, <www.water.usgs.gov/cgi-bin/man_wrdapp?modbrnch>, visited: July 1st, 2007, 15:35 UTC+2.

U.S. Geological Survey (URL_b): Method for model MODBRNCH, <www.water.usgs.gov/cgi-bin/man_wrdapp?modbrnch>,downloaded: July 1st, 2007, 15:35 UTC+2.

U.S. Geological Survey (URL_c): Water Resources Applications Software: MODBRNCH: Ground-Water/Surface-Water Coupled Flow Model using USGS MODFLOW and BRANCH Models; Information, Software and Documentation, <http://water.usgs.gov./software/modbrnch.html>, downloaded: August 8th, 2007, 1:30.

U.S. Geological Survey (URL_d): MODFLOW and related programs, <http://water.usgs.gov/nrp/gwsoftware/modflow.html>, downloaded: August 8th, 1:50.

U.S. Geological Survey (URL_e): Water resources applications software: UCODE_2005 and six other computer codes for universal sensitivity analysis, calibration, and uncertainty evaluation, <http://water.usgs.gov/software/ucode.html>, downloaded: July 31st, 2007 - 10:45.

van der Nat, D.; A.P. Schmidt; K. Tockner; P.J. Edwards; J.V. Ward(2002): Inundation dynamics in braided floodplains: Tagliamento River, Northeast Italy, Ecosystems, 5(7), 636-647.

VanderKwaak, J.E.; K. Loague (2001): Hydrologic-response simulations for the R-5 catchment with a comprehensive physics-based model, *Water Resources Research*, 37(4), 999-1013.

VAW [Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie]: Basement: basic simulation environment for computation of environmental flow and natural hazard simulation: Homepage (program and documentation), <http://www.basement.ethz.ch>; download. August 7th, 2007 - 17:00.

Vetsch, D.; R. Fäh (2003): Rheinvorstreckung - Hydraulische Modellversuche und numerische Simulationen zur Entwicklung der Rheinmündung in den Bodensee, Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie, ETH Zürich, Bericht Nr. 4149-2, Band 2, *unveröffentlicht*.

Wald, J; G. R. Schiffler (2002): Einsatz eines gekoppelten Grundwasser-Oberflächengewässer-Modellsystems in der Ingenieurpraxis, Forum für Hydrologie und Wasserbewirtschaftung, Wechselwirkung zwischen Grundwasserleitern und Oberflächengewässern, *Beiträge zum Tag der Hydrologie 2002*, 20.-22. März in Suderburg, Lüneburger Heide, 67-76.

Werner, A.D.; M.R. Gallagher; S.W. Weeks (2006): Regional-scale, fully coupled modelling of stream-aquifer interaction in a tropical catchment, *Journal of Hydrology*, 328(3-4), 497-510.

Wesseling, J.G.; P. Kabat; B.J Van den Broek; R.A: Fedders (1989): *The program SWATRE/SWACROP - user manual*, The Winand Staring Centre for Integrated Land, Soil and Water Research (SC), Wageningen, the Netherlands.

White, F.M. (2003): *Fluid Mechanics*, McGraw-Hill, New York, 866 p.

Wright, K.K.; C.V. Baxter; J.L. Li (2005): Restricted hyporheic exchange in an alluvial river system: implications for theory and management, *Journal of the North American Benthological Society*, 24(3), 447-460.

APPENDIX

1. Project Partners

The MaVal project is being carried out under the direction of:

- Institute of Environmental Engineering, ETH Zurich, ETH-Hönggerberg, CH-8093 Zürich, Switzerland.

The following institutions have been taken part in the MaVal project with scientific, technical or administrative support and human resources:

- Institute for Earth Science (IST-SUPSI), University of Applied Science of Southern Switzerland, Mailbox 72, CH-6952 Canobbio, Switzerland.
- Ufficio dei Corsi d'Acqua, Dipartimento del Territorio, Viale S. Franscini 17, CH-6501 Bellinzona, Switzerland.
- Centre d'Analyse Minerale, Sciences de la Terre, Université de Lausanne, BFSH 2, CH-1015 Lausanne, Switzerland.
- U.S. Geological Survey, Water Resources Division, 3215 Marine Street, Boulder, Colorado, 80303, U.S.A.
- Institut d'Écologie, Bâtiment de Biologie, Université de Lausanne, CH-1015 Lausanne, Switzerland.
- Department of Ecology and Evolution, Laboratory for Conservation Biology, Université de Lausanne, Biology Building, CH-1015 Lausanne, Switzerland.
- DITIC - Dipartimento di Idraulica, Trasporti e Infrastrutture Civili, II Facoltà di Ingegneria, Politecnico di Torino, Vercelli, Italy.
- Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie, Gloriastr. 37/39, ETH-Zentrum, VAW D 26, CH-8092 Zurich, Switzerland.

2. Sensitivity to Roughness under Low Flow Conditions

Tab_A 1: Sensitivity to roughness under low flow conditions for different reaches of the Maggia River. The Δ values refer to the difference between simulated and observed values.

Bignasco, straight channel, 1.2 m^3/s, 13/13 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	---	---	---	---	---	---
0.075	---	---	---	---	---	---
0.10	---	---	---	---	---	---
0.13	1.27	1.37	0.41	0.41	1.68	1.85
0.16	---	---	---	---	---	---
0.195	1.30	1.48	0.43	0.45	1.73	1.89
0.35	1.30	1.48	0.43	0.45	1.74	1.76

Cevio, straight channel, 0.9 m^3/s, 18/20 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	1.14	1.21	0.56	0.36	1.70	1.65
0.075	1.14	1.21	0.56	0.36	1.70	1.65
0.10	1.14	1.21	0.56	0.36	1.70	1.65
0.13	1.14	1.21	0.56	0.36	1.70	1.66
0.16	1.14	1.21	0.56	0.36	1.71	1.67
0.195	1.14	1.21	0.56	0.36	1.71	1.67
0.35	1.14	1.21	0.57	0.36	1.71	1.68

Riveo, braided area, 0.9 m³/s, 3/10 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	0.70	0.70	0.25	0.25	0.95	0.95
0.075	0.70	0.70	0.24	0.24	0.95	0.95
0.10	0.70	0.70	0.25	0.25	0.95	0.95
0.13	0.70	0.70	0.25	0.25	0.95	0.95
0.16	0.70	0.70	0.25	0.25	0.95	0.95
0.195	0.70	0.70	0.25	0.25	0.95	0.95
0.35	0.70	0.70	0.26	0.26	0.96	0.96

Someo, braided area, 0.9 m³/s, 9/35 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	1.75	2.00	0.37	0.39	2.12	2.38
0.075	1.75	2.00	0.37	0.39	2.12	2.38
0.10	1.75	2.00	0.37	0.39	2.13	2.38
0.13	1.75	2.00	0.38	0.39	2.13	2.39
0.16	1.75	2.00	0.38	0.40	2.13	2.39
0.195	1.75	2.00	0.38	0.40	2.14	2.39
0.35	1.75	2.00	0.39	0.41	2.14	2.41

Giumaglio – upstream, braided area, 1.67 m³/s, 2/20 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	---	---	---	---	---	---
0.075	1.64	1.64	0.30	0.30	1.94	1.94
0.10	---	---	---	---	---	---
0.13	1.64	1.64	0.31	0.31	1.95	1.95
0.16	---	---	---	---	---	---
0.195	1.64	1.64	0.31	0.31	1.96	1.96
0.35	1.64	1.64	0.32	0.32	1.96	1.96

Giumaglio – downstream, braided area, 2.08 m³/s, 3/3 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	0.63	0.63	0.19	0.19	0.82	0.82
0.075	0.62	0.62	0.19	0.19	0.82	0.82
0.10	0.62	0.62	0.19	0.19	0.82	0.82
0.13	0.62	0.62	0.18	0.18	0.81	0.81
0.16	0.63	0.63	0.19	0.19	0.81	0.81
0.195	0.62	0.62	0.19	0.19	0.82	0.82
0.35	0.63	0.63	0.20	0.20	0.82	0.82

Moghegno, straight channel, 2.31 m³/s, 11/12 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	0.30	0.31	0.40	0.36	0.70	0.66
0.075	0.30	0.31	0.39	0.36	0.69	0.64
0.10	0.30	0.31	0.39	0.36	0.69	0.64
0.13	0.30	0.31	0.39	0.36	0.69	0.64
0.16	0.30	0.30	0.40	0.37	0.69	0.63
0.195	0.30	0.30	0.40	0.37	0.69	0.64
0.35	0.30	0.30	0.42	0.38	0.72	0.67

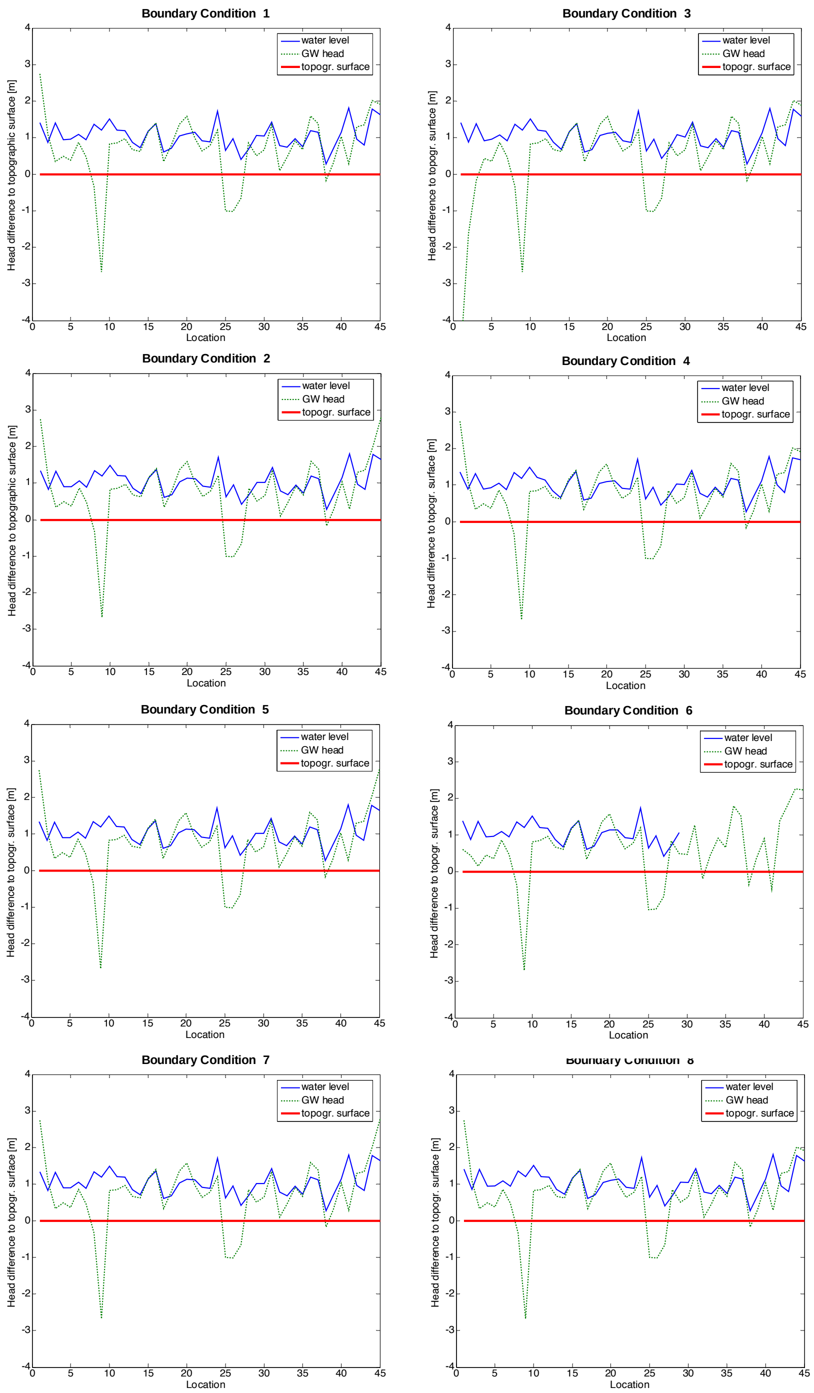
Gordevio, straight channel, 3.09 m³/s, 15/16 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	0.82	0.89	0.31	0.32	1.13	1.15
0.075	0.82	0.89	0.30	0.31	1.12	1.15
0.10	0.82	0.89	0.30	0.30	1.12	1.14
0.13	0.82	0.89	0.49	0.49	1.12	1.14
0.16	0.82	0.89	0.31	0.31	1.13	1.15
0.195	0.82	0.89	0.31	0.31	1.13	1.15
0.35	0.82	0.89	0.34	0.35	1.15	1.18

Avegno, straight channel, 3.09 m³/s, 1/39 measurement points						
k_s [m]	Δ *bed level* [m]		Δ *flow depth* [m]		Δ *water level* [m]	
	mean	median	mean	median	mean	median
0.05	1.02	1.02	-0.10	-0.10	0.92	0.92
0.075	1.02	1.02	-0.10	-0.10	0.91	0.91
0.10	1.02	1.02	-0.11	-0.11	0.91	0.91
0.13	1.02	1.02	---	---	0.90	0.90
0.16	1.02	1.02	-0.10	-0.10	0.91	0.91
0.195	1.02	1.02	-0.11	-0.11	0.91	0.91
0.35	1.02	1.02	-0.09	-0.09	0.92	0.92

3. Sensitivity to Boundary Conditions

Tab_A 2: Overview over the simulations with different boundary conditions in Fig_A 1. Simulation time was approx. 50 h, starting with steady-state conditions with a streamflow in Bignasco of 1.2 m^3/s, a linearly increase to 100 m^3/s between 7500 s and 15000 s, and a constant inflow of 100 m^3/s afterwards.

Boundary Condition Number	**head upstream**	**flow upstream**	**head downstream**	**flow downstream**
1	yes	no	yes	no
2	yes	no	no	no
3	no	no	no	no
4	yes	yes	yes	no
5	yes	yes	no	no
6	no	yes	no	no
7	yes	yes, 5-fold	no	no
8	no	yes, 5-fold	no	no

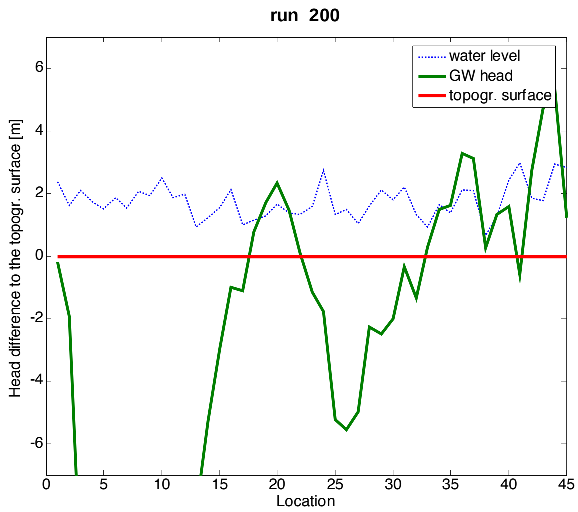


Fig_A 1: Longitudinal profile of the difference of the simulated water level and groundwater heads to the topographic surface within MODFLOW river cells for different boundary conditions (see Tab_A 2). The distance of the neighbouring location is approx. 500 m. Groundwater level above the topographic surface means exfiltration, otherwise infiltration. There is practically no influence of the boundary condition on the solution except for the very near neighbourhood of the boundaries itself.

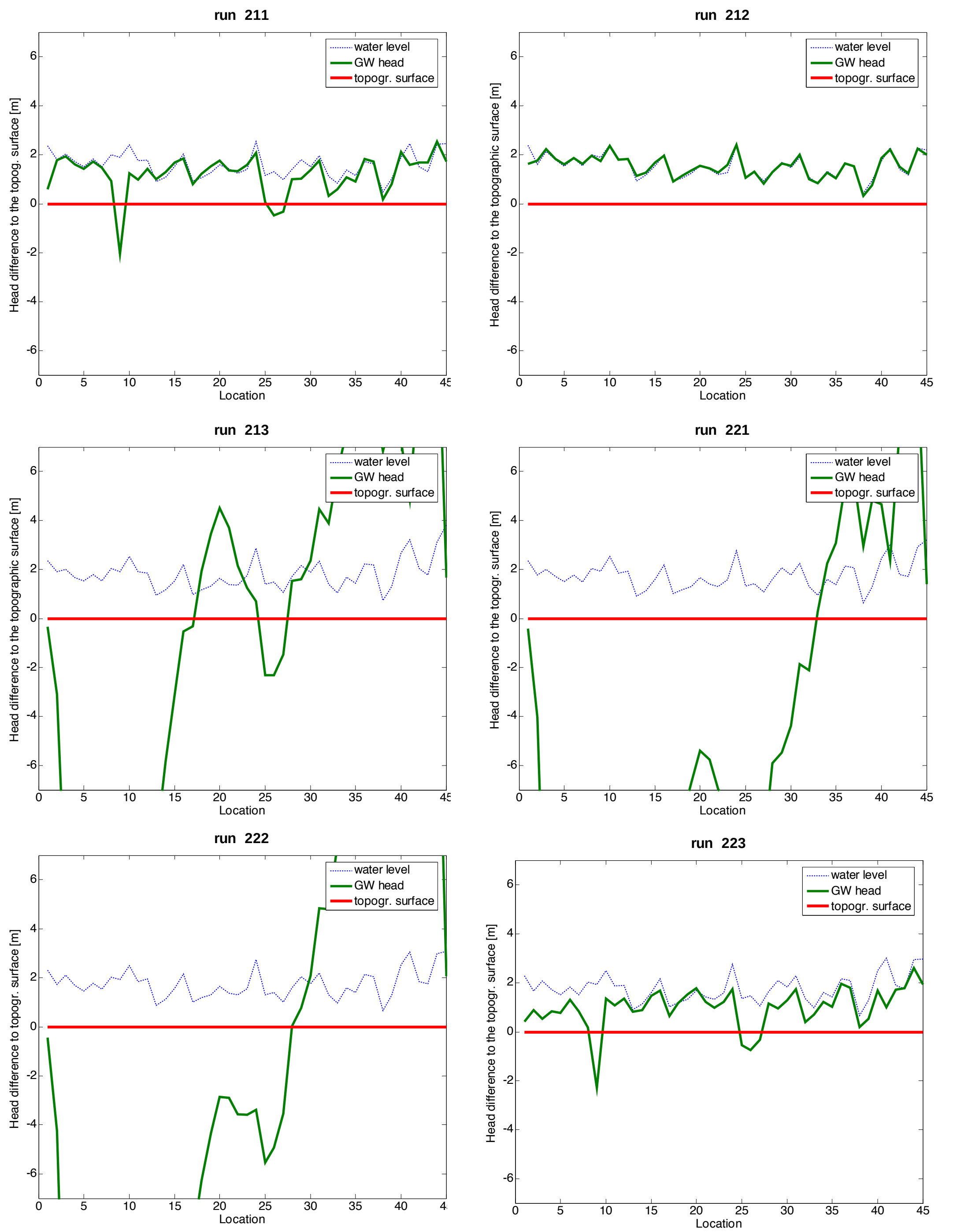
4. Sensitivity to Model Parameters

Tab_A 3: Overview over the sensitivity runs which are shown in Fig_A 2 and Fig_A 3. Run number 200 corresponds to the the best-parameter set that is reported in Tab. 6.4. Only one parameter was varied in time while the others were kept fix. All parameters were changed with the factor shown in this table except the specific yield, where the absolute values are shown. The RMSE is calculated based on the steady-state solution for the same piezometers used also by Foglia (2006) for the automatic calibration of the stand-alone MODFLOW model.

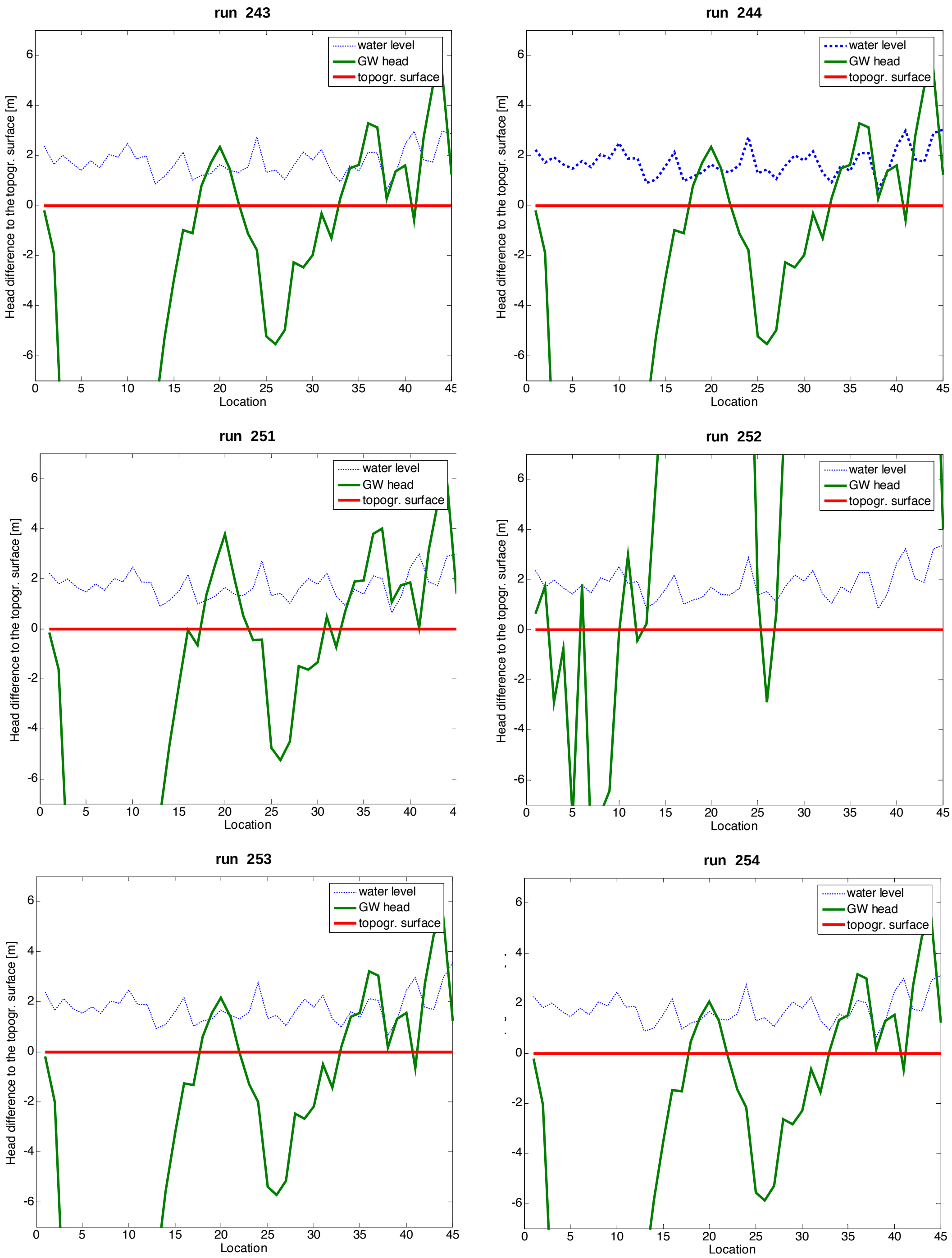
run number	**leakage (K_{riv}/M)** [*factor*]	**K-factor** [*factor*]	**specific yield** [*absolute value*]	**specific storage** [*factor*]	**recharge multiplier wet** [*factor*]	**RMSE** [m]
200	1	1	0.27	1	1	5.48
211	10	1	0.27	1	1	1.55
212	100	1	0.27	1	1	1.58
213	1/10	1	0.27	1	1	7.63
214	1/100	1	0.27	1	1	---
221	1	10	0.27	1	1	10.76
222	1	100	0.27	1	1	11.91
223	1	1/10	0.27	1	1	1.49
224	1	1/100	0.27	1	1	3.42
231	1	1	0.34	1	1	5.43
232	1	1	0.41	1	1	5.43
233	1	1	0.20	1	1	5.43
234	1	1	0.13	1	1	5.43
241	1	1	0.27	10	1	5.43
242	1	1	0.27	100	1	5.43
243	1	1	0.27	1/10	1	5.43
244	1	1	0.27	1/100	1	5.43
251	1	1	0.27	1	10	5.30
252	1	1	0.27	1	100	5.16
253	1	1	0.27	1	1/10	5.55
254	1	1	0.27	1	1/100	5.67



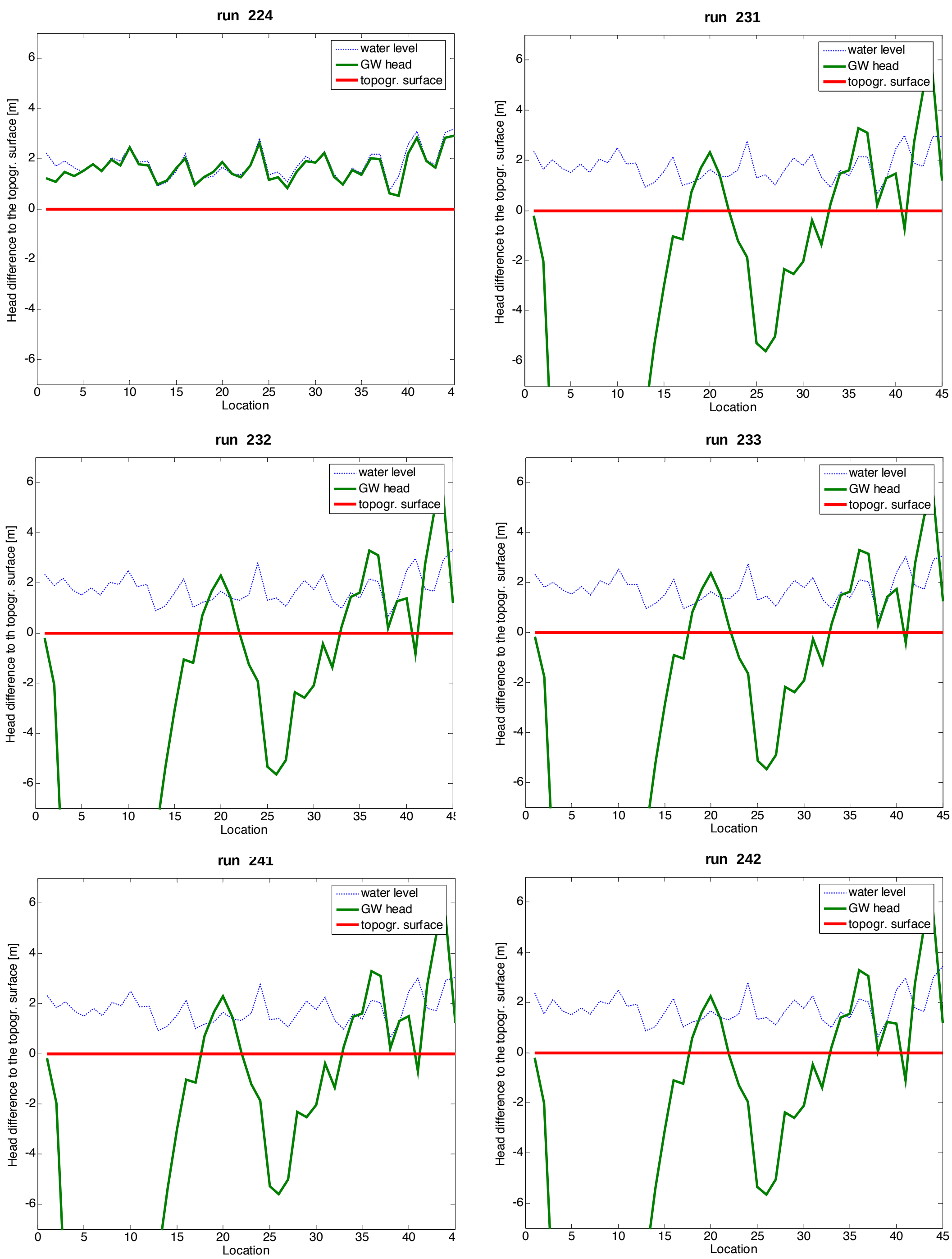
Fig_A 2: Head differences between river head resp. groundwater head and the topographic surface along the longitudinal profile of the Maggia River as a result of the simulation with the "best-parameter" set after approx. 30 h with a discharge of 300 m^3/s. This result is simultaneously part of the sensitivity runs shown in Fig_A 3.



Fig_A 3: Head differences between river head resp. groundwater head and the topographic surface along the longitudinal profile of the Maggia River as a result of the parameter sensitivity analysis after approx. 30 h with a discharge of 300 m³/s.


run 243
run 244
run 251
run 252
run 253
run 254
water level
GW head
topogr. surface
Head difference to the topogr. surface [m]
Location

Fig_A3 (cont.)


run 224
run 231
run 232
run 233
run 241
run 242
water level
GW head
topogr. surface
Head difference to the topogr. surface [m]
Head difference to th topogr. surface [m]
Location

Fig_A3 (cont.)