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SIMULATION OF IWRM ADAPTIVE MEASURES TO REDUCE THE IMPACT OF URBANIZATION AND CLIMATE CHANGE ON URBAN FLOOD RISK: THE CASE OF TURNHOUT

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«Un giorno sei nato.
Nessuno ti ha chiesto se volevi vivere.
Ma ora vivi. Talvolta è bello. Talvolta invece sei triste.
Molte cose ancora non le comprendi.
Vivi, ma perché?
Con le tue mani devi aiutare a riordinare il mondo.
Col tuo intelletto devi cercare di distinguere il bene dal male.
Col tuo cuore devi amare gli uomini e aiutarli quando puoi.
Sono tanti i compiti che ti attendono.
Che attendono le tue mani, il tuo intelletto, e il tuo cuore»

Chiara Luce Badano

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ABSTRACT

In the last years, growing attention has been drawn on the study of the combined effect of urbanization and climate change on catchment runoff. In particular, assessing the impact of climate change on urbanizing catchments have gained great importance for the water management of several cities. There has been extensive development of paved areas within the city of Turnhout in Belgium in combination with straightening of the neighboring rivers, Visbeek and Aa. Moreover, the city authority has decided to encourage more densification within or next to the cores, which would lead to frequent overflows out of the existing combined sewer system. This combination determines a faster flow of larger quantities of water in a valley that cannot physically absorb it, causing increasingly frequent and harmful floods events affecting agricultural lands. The situation could be indisputably exaggerated under climate change scenarios. For these reasons the city authority has planned adaptive measures together with new rainwater collector system. This study focused on assessing the effectiveness of buffering water from the rainwater collector system to reduce the flood problem downstream the city. For this study, a lumped conceptual hydrological model NAM has been developed for generating runoff from the catchment. A perturbation tool, developed by Katholieke Universiteit of Leuven, was applied to generate time series of future rainfall and ETo. The time series of urban runoff from the city was obtained from the correlation between rainfall and urban runoff under current and climate change scenarios. The urban runoff was taken as short time series of output from the sewer system model that was used in existing river model. An hydrodynamic MIKE11 1D river modeling was set up, including floodplain model. Rainfall-runoff was then uniformly distributed along the river reaches and urban runoff was applied as point source boundary conditions in the calibrated and validated MIKE 11 river flood model. Adaptive IWRM were applied to overcome the problem of flooding considering buffer areas at the end of the new rainwater collector system and introducing storage element located under green space. The river flood model results showed that there is an intensification of peak runoff and flooded areas resulting from combined effect of urbanization and climate change, when compared with urbanization or climate change individually. Urban runoff seemed to characterize the hydrograph by an increase in the peak with an indication of quick response to the rainfall event, which can be considered as a fourth component of total rainfall-runoff discharge with further investigations. Our results, after implementation of buffers, showed that there is some decrease in the flooded areas in the lower valley of the city. The presence of storage element has allowed to reduce critical condition in the sewer system and has also contributed to further reduction in flooding areas downstream the city. Assessing the combined effectiveness of all the adaptive measures would be the best to come up with a sustainable flood management option.

This thesis is divided into six sections. The first chapter is a general introduction of the main aspects of this thesis research. The existing problem in Turnhout and all the proposed water resource management strategies to reduce the risk of flooding in that study area are also introduced in the first chapter. The second chapter deals with a complete description of the lumped conceptual hydrological NAM model. The description includes the calibration and the validation of the model, and also the model performance evaluation considering the extreme analysis of the peak flow maxima. Finally it is illustrated the development of the composite hydrographs for different return periods from discharge-duration-frequency (QDF). The third chapter describes the river and floodplain hydraulic modelling. The chapter starts with a description of the MIKE 11 1D model and continues with a clear illustration of its implementation and calibration, based on the existing Infoworks RS model for the two rivers. The fourth chapter illustrates the development of the urbanization and climate change element as input for the MIKE 11 model. Afterwards the impacts of urbanization and climate change on flood risk are shown both separately and together, using flood maps and values. The fifth chapter explains the development of the buffer areas inside the city and at the end of the rain water collector system and their effectiveness in reducing the flooding areas in the lower part of the city. The sixth chapter reports conclusions and recommendations learnt from this work.

Appendix

In appendix A is reported in detail the existing dynamics and project in Turnhout city and for the rivers Aa and Visbeek to reduce the flood problem.

In appendix B are shown the results of the NAM model validation for the period from 1993 to 1996 in the analysis of the extreme.

In appendix C are presented some floodbranches implemented in MIKE GIS and the floodmaps derived from the analysis of the impact of urbanization and climate change on flood risk.

Key words: urbanization, climate change, river flood model, flood map, urban runoff, buffer, adaptive measure, sustainable management.

SOMMARIO

INTRODUZIONE

La gestione sostenibile delle risorse naturali non può prescindere da una analisi del problema che sia quanto più possibile ampia e che si basi su una visione integrata di tutti i fattori coinvolti ed interagenti.

Negli ultimi decenni la gestione delle risorse idriche, in particolare nelle aree urbanizzate, è divenuto argomento di grande attualità per le numerose implicazioni che questa gestione ha rispetto agli abitanti e alla qualità della loro vita. Precedenti studi hanno dimostrato come l'urbanizzazione assieme ai più importanti cambiamenti climatici, abbiano modificato nel tempo il naturale ciclo idrologico, con impatti negativi sugli eventi estremi e sul rischio alluvionale.

In questo contesto si rende necessario potersi servire di misure gestionali delle risorse idriche che possano rispondere alle nuove necessità dei centri abitati.

In generale, l'integrazione del sistema fognario con il sistema fluviale costituisce la base su cui poggiano le decisioni gestionali delle risorse idriche in aree fortemente urbanizzate e va affrontato alle diverse scale spaziali e temporali, permettendo di individuare l'alternativa migliore tra quelle disponibili e quindi di fornire delle risposte a problemi quali le piene fluviali.

In questo lavoro di tesi è stato implementato un modello integrato che ha permesso di valutare gli effetti dell'urbanizzazione e dei cambiamenti climatici sui corsi fluviali Aa e Visbeek, oggetto di frequenti alluvioni che colpiscono soprattutto l'area sottostante la città di Turnhout, situata nella provincia di Anversa in Belgio. Successivamente la valutazione di efficacia di due delle alternative proposte dalle autorità locali per la gestione integrata delle risorse idriche è stata eseguita. In quest'ottica le potenzialità di utilizzo di aree verdi come invasi naturali denominati “buffer zones” e l'introduzione di serbatoi di accumulo dell'acqua all'interno della città sono state valutate e ritenute adeguatamente efficaci per la risoluzione del problema alluvionale.

AREA DI STUDIO: CITTA' DI TURNHOUT, ANVERSA, BELGIO

L'area di studio presa in esame si colloca nella città di Turnhout, provincia di Anversa in Belgio. Il centro abitato nasce tra due fiumi, denominati Aa e Visbeek. L'area complessiva del bacino è di circa 72 km^2 e gli affluenti principali sono costituiti dai fiumi Liermansloop, Zijtak Visbeek e Rondvenloop. In figura 1 sono rappresentati la localizzazione della città e il DEM relativo al bacino oggetto di studio.

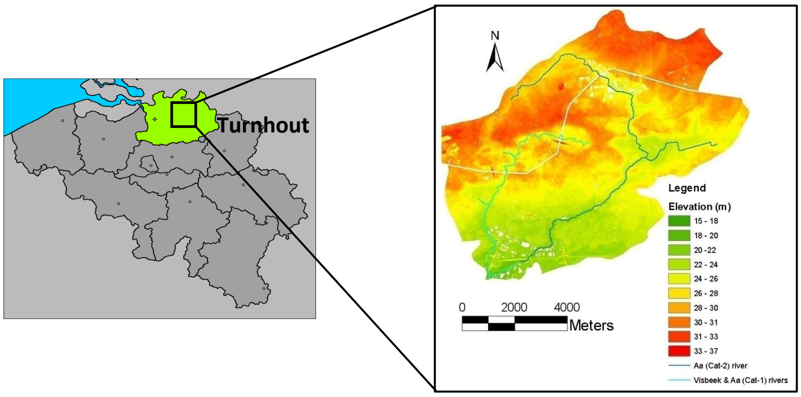


Figura 1: localizzazione della città di Turnhout e DEM relativo al bacino oggetto di studio.

Il suolo è costituito da più del 50% da sabbia limosa ed è principalmente dominato da aree urbane, terreni di pascolo e praterie.

La sempre più crescente presenza di aree impermeabilizzate nella città di Turnhout, ha portato ad una maggiore frequenza di esondazioni dovute ad un insufficienza del sistema di drenaggio urbano, attualmente di tipo combinato. Oltre a ciò i due fiumi che circondano la città hanno subito negli ultimi anni profonde modifiche del regime di moto, a causa di un raddrizzamento del loro corso fluviale che ne ha accelerato lo scorrimento. La combinazione di questi due effetti ha determinato un incremento della quantità di acqua che viene trasportata verso valle, provocando frequenti e dannose alluvioni che colpiscono ancora oggi le aree agricole, particolarmente presenti nella parte sottostante e periferica della città. Eventuali futuri cambiamenti climatici potrebbero ulteriormente aggravare una situazione già per molti aspetti non sostenibile.

Per questo motivo le autorità locali hanno studiato ed elaborato una serie di alternative che, in combinazione con la progettazione di una nuova rete fognaria che accolga esclusivamente le acque di tipo meteorico (RWA), possa ridurre il problema corrente delle inondazioni, ma anche prevenire futuri problemi legati ad una maggiore presenza di acqua che ne complicherebbe la sua gestione.

Due alternative proposte dall’autorità locale in collaborazione con l’Università Cattolica di Leuven, sono state prese in esame nello studio di questa tesi. Esse hanno riguardato l’utilizzo di aree e strutture di accumulo temporaneo delle acque meteoriche durante gli eventi estremi sia nel tratto terminale dei nuovi collettori della rete fognaria, sfruttando la presenza di estese aree verdi (buffer zone), sia a livello locale come serbatoi sotterranei presenti all’interno della città. L’obiettivo è stato quello di valutare come queste alternative possano contribuire a ridurre il rischio alluvionale considerando gli effetti dell’urbanizzazione e dei cambiamenti climatici. In particolare è stata condotta un’analisi dei valori estremi dei flussi attesi e una valutazione sulla variazione di estensione delle aree alluvionate, in particolare nell'area sottostante la città. L'analisi è stata condotta tramite l'utilizzo di un modello integrato, che simulasse il comportamento del flusso fluviale, assieme a quello del sistema fognario della città, e determinasse quali fossero le potenziali aree inondabili.

IL MODELLO INTEGRATO

Il modello integrato utilizzato in questo lavoro di tesi parte dal modello idrologico NAM, di tipo concettuale, che è stato utilizzato per determinare i deflussi dei bacini presi in esame. NAM considera il sistema idrologico come costituito da quattro serbatoi lineari in serie, che rappresentano quattro elementi fisici del bacino mutualmente correlati tra di loro. I dati di ingresso sono le serie temporali di precipitazione, di evapotranspirazione e le osservazioni di portata misurate nella stazione di Turnhout. Tramite calibrazione sono stati determinati i 9 parametri che caratterizzano il modello, mostrando una buona sovrapposizione tra la portata misurata e quella simulata. In figura 2 sono riportati i risultati della simulazione del modello NAM, confrontati con i valori di portata osservati nella stazione di misurazione di Turnhout.

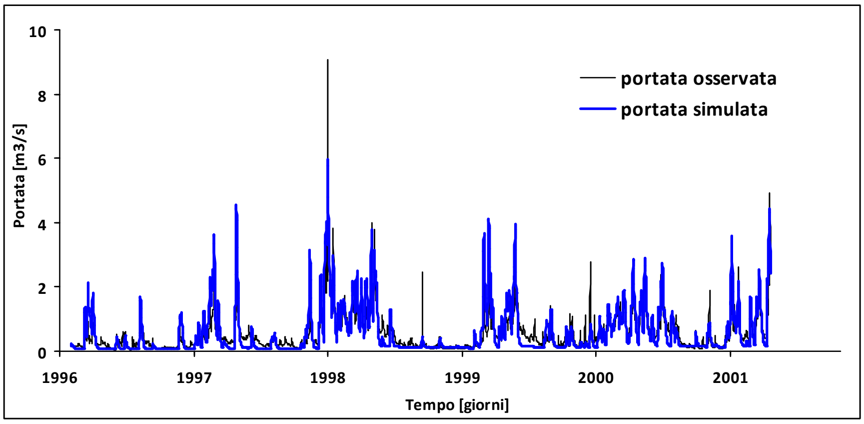


Figura 2: confronto tra i valori osservati e quelli simulati dal modello NAM dopo calibrazione.

La successiva validazione del modello ha confermato i buoni risultati ottenuti tramite calibrazione, con un valore di NSE uguale a 0.61 e un valore di discrepanza medio nel bilancio idrico tra i volumi osservati e quelli simulati uguale a -12.5%. Infine la verifica finale del modello è stata ottenuta utilizzando il programma WETSPRO (Water Engeneering Time Series PROcessing), dove sono stati messi a confronto i risultati del modello con le osservazioni, estraendo i valori estremi di deflusso tramite un criterio di indipendenza implementato nel modello. Il confronto tra gli estremi, dopo la trasformazione Box-Cox, dimostrano come il modello NAM abbia ottenuto buoni risultati nella simulazione in quanto tali valori sono concentrati intorno alla linea bisettrice. Stesse conclusioni si possono trarre confrontando i volumi cumulati del flusso totale tra i valori osservati e quelli simulati, i quali risultano quasi sovrapponibili, e valutando il legame tra gli estremi con il rispettivo periodo di ritorno, che dimostra come il modello sia in grado di prevedere condizioni estreme.

Un'analisi degli estremi è stata applicata ai risultati ottenuti dal modello NAM per determinare opportuni idrogrammi sintetici da utilizzare in fase di simulazione del modello idrodinamico. L'analisi degli estremi può essere suddivisa in varie fasi in successione:

1. si è partiti dalla serie dei valori di portata ottenuti dal modello, i quali sono stati mediati considerando diversi livelli di aggregazione (da 1 ora fino a 3 giorni);
2. per ogni livello di aggregazione sono stati estratti i valori degli estremi di deflusso al di sopra di una certa soglia (POT) tramite il programma WETSPRO. Poiché in un'analisi dei valori estremi, gli stessi devono essere indipendenti, un criterio di indipendenza è stato usato durante l'estrazione;
3. i valori estratti POT, indipendenti tra di loro, sono stati ordinati in ordine decrescente ed analizzati tramite il programma ECQ (hydrological estreme value analysis) per ogni livello di aggregazione, in modo da determinare i parametri di distribuzione di probabilità della serie degli estremi;
4. infine, utilizzando quest'ultimi, sono state ottenute le relazioni QDF (portata tempo frequenza) per differenti periodi di ritorno, e successivamente i corrispettivi idrogrammi sintetici, ridistribuendo la QDF simmetricamente intorno al centro dell'evento sintetico e partendo dalla durata più breve (1 ora) fino a quella più lunga (72 ore). In figura 3 sono riportati i risultati degli idrogrammi sintetici ottenuti per il bacino del fiume Aa.

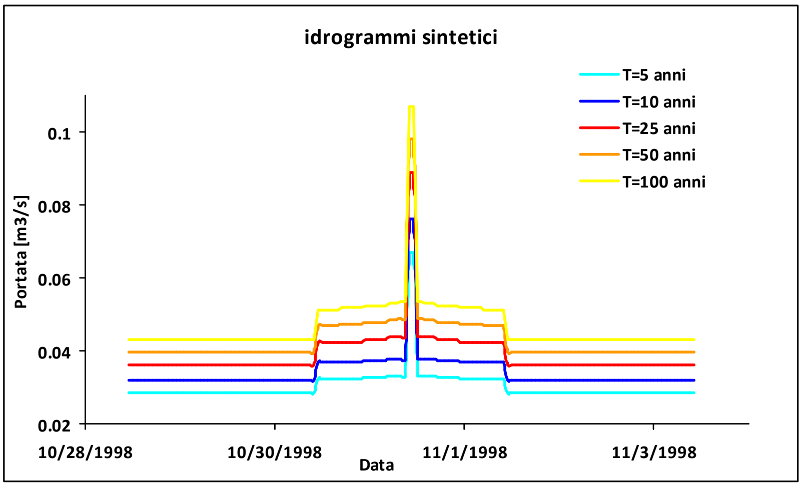


Figura 3: risultati degli idrogrammi sintetici ottenuti per il bacino del fiume Aa.

La seconda parte del modello integrato ha riguardato l'implementazione del modello idrodinamico MIKE 11 per i fiumi in esame, a partire dal modello esistente Infoworks RS. Il modello risolve le equazioni di De Saint Venant per acque basse a pelo libero usando uno schema alle differenze finite di tipo implicito. Le aree inondate sono state implementate mediante un approccio di tipo quasi 2D semplificato. Nello specifico ciascuna area inondata è stata rappresentata con un tratto di fiume, caratterizzato da una lunghezza che variava dai 10 ai 30 m, 2 fittizie ed identiche sezioni dalla forma trapezoidale ed un unico canale di collegamento che connetteva il nuovo tratto di fiume con quello esistente. Il prodotto tra le aree delle due sezioni e la distanza tra di loro rappresentava una parte del volume d’acqua totale presente nell’area inondata secondo il modello Infoworks RS. La parte addizionale di volume è stata calcolata a partire dalla relazione h-V estratta dal modello esistente e aggiunta alle informazioni relative alle sezioni fittizie in modo da essere inclusa nella computazione del modello.

Durante la calibrazione del modello MIKE 11 i valori di portata e di livello rispettivamente all’interno e al di sotto delle strutture e i valori di livello delle aree inondabili sono stati confrontati con i risultati del modello Infoworks RS. Seguendo questa procedura, sono stati modificati i coefficienti di scabrezza di tipo Manning tramite una serie di simulazioni in modo da ottenere i valori ottimali. I risultati hanno mostrato una buona correlazione tra i due modelli, con valori di NSE che variano da 0.42 a 0.95 con valore medio di 0.74, considerando i valori di portata e di livello delle strutture selezionate, e un alto valore di correlazione (tra 0.91 e 0.98) tra i due modelli per quanto riguarda le 64 aree potenzialmente inondabili ed implementate in MIKE 11.

Simulazioni di tipo storico e di tipo sintetico sono state considerate in questo lavoro di tesi e mappe 2D che indicassero le reali estensioni delle aree alluvionate sono state generate usando il programma MIKE GIS.

ANALISI DELL’IMPATTO DELL’URBANIZZAZIONE E DEI CAMBIAMENTI CLIMATICI SULLE AREE INONDABILI

L’effetto dell’urbanizzazione e l’impatto dei cambiamenti climatici sul rischio alluvionale sono stati studiati in questo lavoro di tesi sia separatamente che in combinazione per i due bacini Visbeek e Aa. In particolare lo studio ha riguardato la parte bassa del fiume Aa, laddove il rischio di inondazione è più elevato.

Gli scenari futuri in termini di pioggia ed evapotraspirazione attese sono stati ottenuti utilizzando un algoritmo realizzato all’interno del progetto CCI-HYDRO nell’Università Cattolica di Leuven per la valutazione dell’impatto dei cambiamenti climatici in Belgio. In generale nell’algoritmo si possono ottenere 3 differenti scenari, alto, medio e basso, ma solo lo scenario alto è stato preso in considerazione, laddove gli eventi meteorici sono attesi essere più elevati in termini di intensità e di volume di pioggia rispetto alle condizioni attuali.

Tramite il modello esistente del sistema fognario della città di Turnhout, implementato nel programma Infoworks CS, sono state estratte 11 serie temporali di deflusso urbano, che sono state integrate nel modello idrodinamico come sorgenti puntuali per tener conto dell'effetto dell'urbanizzazione sui due fiumi che circondano la città.

In figura 4 sono riportate le estensioni delle aree inondate per effetto dell'urbanizzazione considerando periodi di ritorno di 5, 10, 25, 50, 100 anni.

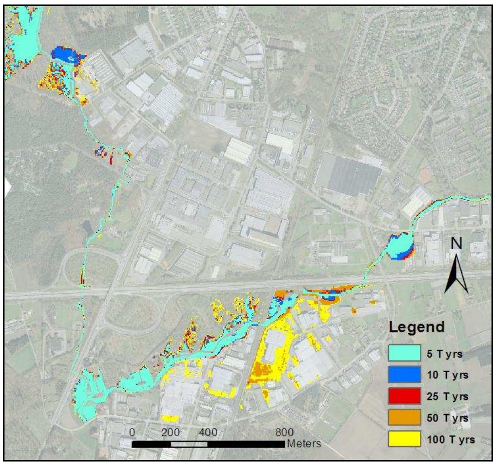


Figura 4: estensioni delle aree inondate per effetto dell'urbanizzazione considerando periodi di ritorno di 5, 10, 25, 50, 100 anni.

L'analisi dei risultati dimostra come l'effetto dell'urbanizzazione abbia un grande impatto sul rischio alluvionale: le aree inondabili presentano un sostanziale incremento dovuto all'urbanizzazione, soprattutto nella parte bassa del fiume Aa, con valori che vanno dal 36% per T=5 anni al 50.5% per T=100 anni.

Anche nell'analisi degli estremi la frequenza dei valori di picco è incrementata in media del 74%, e l'incremento dei valori estremi ha mostrato una tendenza ad aumentare al diminuire del periodo di ritorno.

Per quanto riguarda l'impatto dei cambiamenti climatici sui valori estremi di portata, considerando "l'alto" scenario, si è avuto un elevato incremento di tali valori rispetto alla situazione corrente. In aggiunta l'analisi degli estremi ha rivelato che il periodo di ritorno di un evento alluvionale diminuirà di circa 3 volte causando quindi un evento che si verificherà con una frequenza 3 volte maggiore rispetto alla situazione attuale. Le aree alluvionate subiranno anch'esse un incremento, che raggiungerà circa il 38% per periodo di ritorno centennale, considerando la parte bassa del fiume Aa. La situazione si aggraverebbe ancor di più se l'effetto dell'urbanizzazione e dei cambiamenti climatici venissero presi entrambi in considerazione. L'incremento dei valori di picco di deflusso tenderebbero in media quasi a raddoppiare se entrambi le due variabili venissero tenute in conto rispetto al caso in cui fossero considerate separatamente. Questa intensificazione si ripercuoterebbe chiaramente con un incremento delle aree alluvionali, che raggiungerebbe circa il 67% considerando la parte bassa del fiume Aa.

ANALISI DELL'IMPATTO DELLE ALTERNATIVE GESTIONALI DELLE RISORSE IDRICHE SULLE AREE INONDABILI

Una serie di strategie per la gestione integrata della risorsa idrica nella città di Turnhout sono state sviluppate per ridurre rischi alluvionali attuali e futuri. In questo lavoro di tesi la presenza di aree di accumulo delle acque meteoriche all'interno della città e nei tratti terminali delle nuove condotte (RWA) sono state considerate anche in questo caso sia separatamente che in maniera combinata come valide alternative per ridurre il rischio inondazione, in particolare per quanto riguarda la parte bassa del fiume Aa. Mappe 2D sono state generate per periodi di ritorno di 5, 10, 25, 50 e 100 anni in modo tale da valutare la riduzione delle aree inondate.

L'alternativa denominata "buffer zone" ha riguardato lo studio di ben 6 invasi naturali, situati nei tratti terminali della nuova rete fognaria, che dovranno nell'immediato futuro accogliere le acque di tipo meteorico in modo da evitare che si creino condizioni di alluvionamento nella zona bassa della città. L'eventuale straripamento di queste aree, con opportuni limiti, verrà in ogni caso riversato nel più vicino fiume. I "buffer zone" sono stati implementati nel modello Infoworks CS considerando un'altezza fittizia di 1 metro ed un'area iniziale che derivava dall'area di bacino urbano che interessava il nuovo collettore fognario. Poiché si voleva tener conto delle perdite per infiltrazione alla base di ogni buffer sono stati scelti dei valori di 5 mm/h per i 4 buffer progettati vicino al fiume, in quanto il terreno è principalmente costituito da argilla, e valori di 30 mm/h per i 2 buffer progettati all'interno della città, dove il terreno è principalmente costituito da sabbia.

L'altra alternativa ha riguardato l'implementazione, sempre all'interno del modello Infoworks CS, di serbatoi di raccolta dell'acqua sotterranei, posti al di sotto di aree verdi e destinati alla raccolta delle acque meteoriche del più vicino sottobacino urbano.

I risultati, dopo l'implementazione dei 6 buffer, hanno mostrato una diminuzione di circa il 38% nelle aree inondate nella parte bassa della città considerando un periodo di ritorno di 100 anni. La percentuale di riduzione, considerando i cambiamenti climatici, di circa 35%, è più bassa rispetto a quella rispetto alle condizioni attuali dopo l'implementazione dei 6 buffer perché “l’alto” scenario causerebbe secondo le previsioni un inevitabile incremento dei deflussi urbani che i buffer non potrebbero contenere. L’analisi dei valori estremi di portata condotta per l’opzione buffer dimostrano una diminuzione dei valori estremi di portata dei due bacini di circa 32.5% in media sotto le condizioni attuali. Discorso analogo vale per quando riguarda l’analisi degli estremi di portata considerando futuri cambiamenti climatici. In figura 5 è riportato la relazione tra i valori estremi di portata alla confluenza tra i due fiumi e la relativa frequenza di accadimento. La curva in blu mostra l’effetto dell’opzione buffer nei valori estremi di portata sotto futuri cambiamenti climatici.

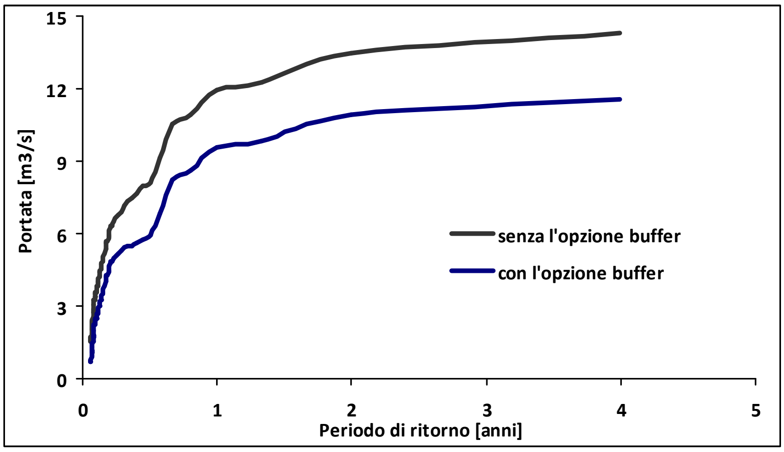


Figura 5: relazione tra i valori estremi di portata alla confluenza tra i due fiumi e relativa frequenza di accadimento.

L’utilizzo di serbatoi di raccolta all’interno della città non ha portato a grandi miglioramenti nella riduzione del problema alluvionale. La piccola riduzione è giustificata dal fatto che la potenzialità di questi serbatoi è dimostrata osservando i risultati del modello del sistema fognario della città, riportati in tabella 1.

Scenario corrente (T=5 anni)	Senza serbatoi di accumulo		Con serbatoi di accumulo		Decremento delle condizioni critiche (%)
Condotte sotto pressione	Totale	224	Totale	185	-
	Relativo (%)	77.0	Relativo (%)	63.6	17.4
Allagamento nodi	Totale	59	Totale	35	-
	Relativo (%)	20.3	Relativo (%)	12.0	40.7
Scenario futuro (T=5 anni)					
Condotte sotto pressione	Totale	245	Totale	221	-
	Relativo (%)	84.2	Relativo (%)	75.9	9.8
Allagamento nodi	Totale	159	Totale	137	-
	Relativo (%)	54.6	Relativo (%)	47.1	13.8

Tabella 1: risultati del modello del sistema fognario della città prima e dopo l'implementazione dei serbatoi di accumulo.

Le analisi dei risultati dimostrano una riduzione di condizioni critiche nelle condotte e nei nodi considerando i valori massimi. Qui di seguito sono riportati in tabella i risultati ottenuti con il programma Infoworks CS prima e dopo l'implementazione di tali serbatoi. Dai risultati emerge come i serbatoi installati nella città contribuiscano in maniera decisiva (40%) alla riduzione di allagamenti nei nodi considerando la situazione attuale. Inoltre si riducono condizioni critiche di moto in pressione in alcune condotte durante gli eventi estremi. La riduzione è di circa il 17% considerando le condizioni attuali e di circa il 10% considerando i cambiamenti climatici.

I risultati dell'effetto combinato di entrambe le opzioni studiate sul rischio alluvionale sono molto simili a quelle osservate considerando la sola opzione "buffer". Questo vuol dire che le due alternative, combinate tra di loro, non hanno un maggiore impatto nel ridurre il rischio alluvionale nella parte sottostante la città. Tuttavia entrambe le proposte potrebbero, in una visione integrata, migliorare la gestione dell'acqua per la città di Turnhout, ad esempio includendo ulteriori buffer (che sono già in fase di progetto e sono programmati per essere realizzati in futuro) oppure estendendo il numero dei serbatoi di accumulo dell'acqua in altri luoghi, come ad esempio al di sotto di parcheggi o strade.

La presa in esame di una terza alternativa, proposta dall'Università Cattolica di Leuven, che consiste nel contenere, in caso di piogge forti, grandi volumi di acqua sfruttando le caratteristiche topografiche a valle della città e utilizzando barriere infrastrutturali, potrebbe ulteriormente contribuire all'identificazione delle opportune misure per migliorare la gestione della risorsa idrica e ridurre il rischio di inondazioni nella città di Turnhout.

Chapter 1

Flood risk and Integrated Water Resources Management: the case of Turnhout

1.1 Introduction

The hydrological system within a catchment is governed by a number of factors, climate change and urbanization being the two most important ones. Significant alteration in the volume and timing of runoff may be caused by the changes in either or both of these factors (Franczyk and Chang, 2009). Many previous studies, assessing the impact of climate change on hydrology, found that there is a close association between streamflow variability and climate change. A significant impact of land use changes, especially caused by urbanization, on hydrology has been found in most of the previous studies (Tu, 2009). However, only some limited studies have been conducted to analyze the combined effects of climate change and land use changes on streamflow (Tu, 2009; Franczyk and Chang, 2009).

The Fourth Assessment Report (FAR) of Intergovernmental Panel on Climate Change (IPCC) has concluded that the global average surface air temperature has increased by 0.74 ºC during the 20th century and is projected to increase by 1.8 ºC for low scenario (B1) to 4.0 ºC for high scenario (A1FI) by 2100, relative to 1990 temperature (IPCC, 2007a). Climate change projections for Belgium state that there will be rise in temperature from 1.7 to 4.9 ºC in winter and from 2.4 to 6.6 ºC at the end of 2100 in relation to the end of 2000 (Ntegeka, 2006). The change in rainfall as projected previously shows a rise from 6% to 23% for winter and a drop of up to 50% for summer until the end of 2100 (Jean-Pascal, 2004 cited in Ntegeka, 2006). A resulting intensified global and regional hydrologic cycle is expected by these predicted temperature rises (Huntington, 2006). The consequences of this intensification are the changes in the temporal and spatial distribution of precipitation, the increased frequency of storm intensities and floodings, more frequent droughts and decreased annual snowfall (Huntington, 2006; Franczyk and Chang, 2009).

Urbanization is another important factor that influences the hydrological system. Infact it causes dramatic environmental changes as vegetation is removed, soils become covered by impervious surfaces and streams are replaced by pipes. The latter are illustrated by rapid flow responses and high peak flows following even modest rainfalls due to decreased infiltration and increased runoff (Semadeni-Davies et al., 2008; White et al., 2006). Increase in runoff occurs in proportion to the cover of impervious surface in a catchment (Arnold et al., 1996) which increases peak discharges and flood extents. Floods with shorter return periods increase more than that with long return periods (Hirsch et al., 1990 cited in Tu, 2009).

The IPCC technical paper on climate change and water, states that water and its availability and quality will be the main pressure on, and issues for, societies and the environment under climate change (Bates et al., 2008). Land cover change, especially urbanization, within a watershed is also recognized as an important factor affecting runoff (Chang, 2007), and it is possible that the transformation of land across the globe could have a greater influence on runoff than climate change (Vorosmarty et al., 2000).

Urbanization, as well as changes in land uses and vegetation, within a watershed may further exacerbate the effect of climate change on runoff from the watershed (Georgiyevsky and Shiklomanov, 2003 cited in Franczyk and Chang 2009). Until now, climate change impact assessment for urban areas has been focused on flood risk from river systems alone rather than storm- and wastewater drainage (Semadeni-Davies et al., 2008).

Therefore, conducting impact analysis is a key technique to have better understanding of what to expect out of the two changes. Such assessment of the likely impacts is vital for the integrated water resources management which is supposed to be sustainable. It is also helpful in providing scientific information to enhance political awareness and develop a sound base for political decisions to be made on the mitigation measures of the impacts of climate change (Phoon et al., 2004 cited in Taye, 2009) together with the impacts of urban development.

1.2 Integrated Water Resources Management (IWRM)

Variations in water flows and groundwater recharge, whether of climatic origin or due to land mismanagement, can add to drought and flood events, which can have catastrophic effects in terms of large scale loss of human life and damage to economic, social and environmental systems. Integrated Water Resources Management (IWRM) is a process that can assist countries in their endeavor to deal with water issues in a cost-effective and sustainable way. The concept of IWRM has attracted particular attention following the international conferences on water and environmental issues in Dublin and Rio de Janeiro held during 1992 (IWRM, 2000). The

Technical Advisory Committee of the Global Water Partnership defined Integrated Water Resources Management (IWRM) "as a process, which promotes the coordinated development and management of water, land and related resources in order to maximize the resultant economics and social welfare in an equitable manner without compromising the sustainability of vital ecosystems," and emphasized that water should be managed in a basin-wide context, under the principles of good governance and public participation.

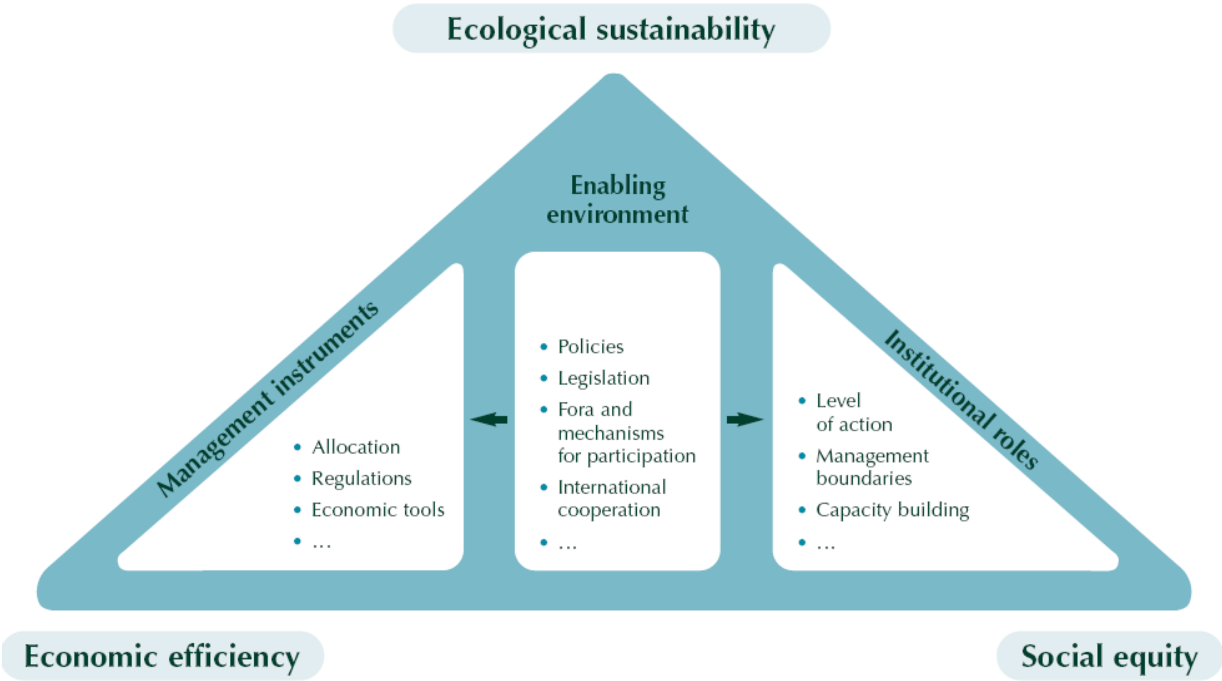


Figure 1.1: the general framework for integrated water resource management (IWRM).

One of the challenges of providing water security as indicated during the 2nd World Water Forum is to provide security from floods, droughts, pollution and other water-related hazards in order to ensure that the vulnerable are protected from the risks of water-related hazards (Savenije et al., 2008). Therefore, integrated water resources management seeks to manage the water resources in a comprehensive and holistic way. Once a range of dimensions have been considered, appropriate decisions and arrangements can be made.

1.3 The flood risk

The term “flood” is used to describe both the rapid increase of discharge in a river and the inundation of low-lying land which may result from when the hydraulic capacity is exceeded and a water overflow happens. The increase of a river discharge is usually due to high rainfall rates or to snow melting on the upstream catchment. However, it could be caused also by landslides, dam breaks, obstructions of bridges, tides, backwater, etc.

The first definition is related to the physical (hydrological) phenomenon itself and apply both to natural (rivers) and artificial (rural drainage and urban stormwater networks) drainage systems.

The second is more related to the potential (and dangerous) results of flooding and can apply also to lakes or the sea. Floods have severe consequences to economic and social damage, and also environmental problems as for example when waste water treatment plants are inundated or when factories holding large quantities of toxic chemicals are also affected. Floods may also destroy wetland areas and reduce biodiversity.

Floods are natural phenomena which cannot be prevented. However, human activity is contributing to an increase in the likelihood and adverse impacts of extreme flood events. Firstly, the scale and frequency of floods are likely to increase due to climate change - which will bring higher intensity of rainfall and rising sea levels - as well as to inappropriate river management and construction in flood plains which reduces their capacity to absorb flood waters. Secondly, the number of people and economic assets located in flood risk zones continues to grow.

Many Member States are already taking flood protection measures but concerted and co-coordinated action at the level of the Community would bring a considerable added value and improve the overall level of flood protection. Given the potential risk to human life, economic assets and the environment, we cannot afford to do nothing. Europe’s commitment to sustainable development could be severely compromised if appropriate measures are not taken.

1.4 Urbanization and climate change impact on flood risks

1.4.1 Urbanization

Urban areas might be considered as the most modified of human environments and all parts of hydrological cycle are affected by urbanization, which usually damages the local water resources. The remarkable effects of urban development on hydrological systems are caused by removal of vegetation and there by introduction of impervious surfaces, such as roofs, roads, parking spaces etc. Urban hydrographs are characterized by high flow peaks and fast response to even moderate precipitation events due to the short flow paths to drain inlets and efficient drainage; thus, urban development is held responsible for increasing runoff and as a consequence raising peak flow volumes and flood risk (Semadeni-Davies et al., 2008).

Firstly, the amount of water that is readily available for evapotranspiration is directly affected by the removal of vegetation due to urban development resulted from the reduced interception of rain and water storage in the litter zone. Secondly, the total infiltration capacity of the soil surface in the catchment is significantly reduced by the catchment's impermeable surface parts that prevent infiltration of water into soils. This obviously affects both the groundwater recharge and the overland flow in excess of infiltration which then affects runoff generation (Bronstert et al., 2002). Finally, in highly urbanized catchments, overland flow retention time will shorten and the time lag between precipitation and runoff will decrease due to a lower surface roughness of urban land than vegetated surfaces (Rose and Peters, 2001; Liu et al., 2006). The direct runoff from urban areas is dominant for a flood event compared with runoff from other land-use areas in the catchment under consideration.

Given the accelerated urbanization rate at global scale there is a strong need for modelling tools that allow assessing the possible effects of urban development on the various components of the hydrological cycle (Poelmans et al., 2010).

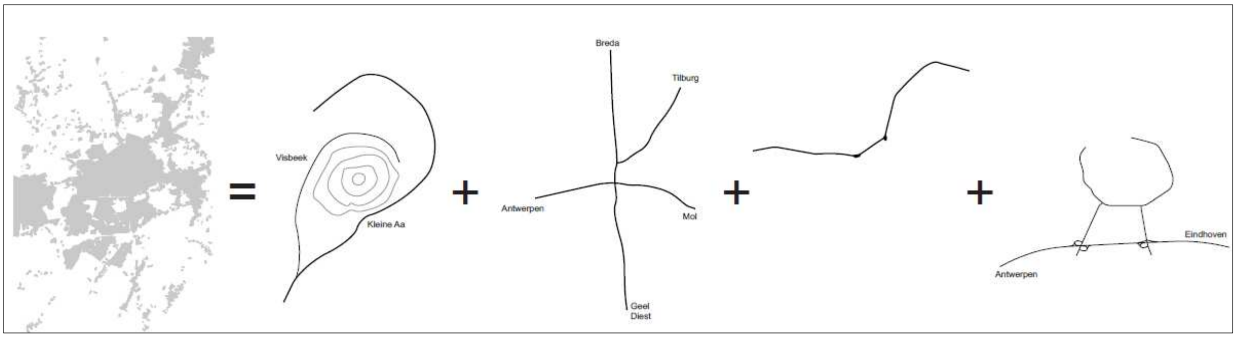
It is important to keep in mind, while thinking of an adaptive measure to reduce the effects of urbanization on flooding, that urban land on one hand consists of sealed surfaces that allow little or no infiltration, but on the other hand also contains gardens and parks which have higher infiltration capacity. Therefore the green areas in and around the city can be selected to build infiltration basins or buffer tanks in order to store water in case of a heavy rainfall in the catchment to avoid flooding downstream and to avoid possible sewer overflows. Various hydrological models coupled with hydrodynamic models can be used to assess the best possible adaptation measures that are capable of reducing the harmful effects of the urbanization with or without considering climate change impact.

1.4.2 Evolution of paved area in Turnhout city

Turnhout is a Belgian municipality located in the Flemish province of Antwerp. The city is also known as the *Capital of the Campine*. The municipality comprises only the city of Turnhout proper. On January 1, 2006 Turnhout had a total population of 39791. The total area is 56.06 km² which gives a population density of 710 inhabitants per km². Turnhout is famous as the world center of the playing card and is both legally and administratively the capital of the Kempen region and of the Arrondissement of Turnhout. It is also the economic and cultural center of its region with more than 40 schools. There is also an important services sector with 2 hospitals, a 2 stage theatre and an 8 screen cinema. Turnhout is located at about 51.3 North and 4.9 East, 42 km from the centre of Antwerp, 30 km from Breda and Tilburg, 40 km from Eindhoven. From nineteenth century the city has had an important increasing of the population due to the increasing of urbanization. This effect can be shown through a spatial analysis of Turnhout area.

Figure 1.2: location of Turnhout city in the north-east of the Flanders, Belgium.

From the spatial analysis it can be observed that the urbanization of Turnhout region can be seen as resulting from the cumulative influence of the factors as shown below in figure 1.3 (a) where the paved area is depicted as a constitutive part of urbanization. Figure 1.3 (b), (c), (d) and (e) present respectively the plateau topography surrounded by a belt of creek valleys, cross point of important connecting roads, canal Dessel-Turnhout-Schoten built during 1846-66, initially created for irrigation purposes, later used for transport and industries of clay, bricks and sand, and *E34* Antwerpen-Eindhoven-Duisburg built in 1973 and a ring road built during 1970-2000 respectively. This physical background of the region played a vital role in process of urbanization throughout the years.



(a) (b) (c) (d) (e)

Figure 1.3: schematic representation of the factors influencing urbanization in Turnhout (adapted from Nolf C., 2010).

Turnhout city belongs to a region called Flanders, that is one of the most urbanized regions in the world with a population density of more than 500 inhabitants/km^2. During the last decades, the predominant land cover change has been the conversion of open land (arable land and grassland) to urban land. The urban expansion has mainly occurred under the form of urban sprawl, a specific type of urban expansion characterized by low-density and ribbon development. This process had resulted in a present-day highly fragmented landscape, showing a mosaic of different land covers and a built-up proportion of almost 20% of the total land surface (Poelmans and Van Rompaey, 2009, 2010 cited in Poelmans et al., 2010). From a detailed study, it is noticed that almost one quarter of the area is occupied by half open and dense buildings, which constitutes a significant paved surface of the city (Province of Antwerp, 2004). Figure 1.4 presents the portion of the paved area in Turnhout in 2000 as compared to that in 1878. Figures A.2 and A.3 (Appendix A) show the urban growth and main infrastructures of Turnhout's urban area in relation to its water structure in 1775, 1878, 1930, 1970, 1990, and 2000, and the predicted areas for densification by 2010-2025 respectively.

Figure 1.4: the paved areas of Turnhout in 2000 (light grey) with comparison to that in 1878 (dark grey) (adapted from Nolf C., 2010).

1.4.3 Climate change

Climate change is a change in the statistical distribution of weather over periods of time that range from decades to millions of years. It can be a change in the average weather or a change in the distribution of weather events around an average (for example, greater or fewer extreme weather events).

Climate change in Intergovernmental Panel on Climate Change (IPCC) usage refers to a change in the state of the climate that can be identified (e.g. using statistical tests) by changes in the mean and/or the variability of its properties and that persists for an extended period, typically decades or longer. It refers to any change in climate over time, whether due to natural variability or as a result of human activity. The alterations due to climate change as observed around the world are the increase in surface temperature, sea level rise, changes in rainfall and decrease in northern hemisphere snow cover (IPCC, 2007a). These are some of the indicators that have been used to detect the existence of climate change, temperature being the most frequently used one.

There have been investigations for over past two decades on climate change associated with global warming induced by the increase in human-produced greenhouse gases such as carbon dioxide and methane in the atmosphere (Xu et al., 2005 cited in Jiang et al., 2007). Since climate system is interactive with the hydrologic cycle, global warming would immediately affect local and regional water availability (Jiang et al., 2007). These effects may include the magnitude and timing of runoff, the frequency and intensity of floods and droughts, rainfall patterns, extreme weather events and so on.

1.5 Previous studies on urbanization and/or climate change impact on flood risks

Impact of urbanization and/or climate change on flood extent and possible adaptation measures have been investigated by different researchers. A brief description of a few similar studies is presented here.

Franczyk, J. and Chang, H., (2009) have studied the effects of climate change and urbanization on the runoff of the Rock Creek basin in the Portland metropolitan area, Oregon, USA using the AVSWAT hydrological model. One of the significant findings of this study is an amplification of runoff resulting from the combination of urbanization and climate changes as compared to only climate or urban land changes.

Semadeni-Davies et al., (2008), have assessed the potential impacts of climate change and continued urbanization on waste and stormwater flows in the combined sewer of central Helsingborg, South Sweden, using a series of DHI MOUSE simulations run with present conditions as well as two climate change scenarios and three progressive urbanization storylines. They have found that urbanization together with increased rainfall or alone can worsen the current drainage problem, while system renovation and installation of sustainable urban drainage systems (SUDS) can largely allay the adverse impacts of both urbanization and climate change.

Jun Tu, (2009) has studied the combined impact of climate and land use changes on streamflow and water quality using a *GIS*-based watershed simulation model, AVGWLF. He simulated the future changes in streamflow and water quality under different climate change and land use change scenarios in watersheds of eastern Massachusetts, USA. Analyses show that there is more impact of climate change and land development on the seasonal distributions of the streamflow.

De Roo, A. et al., (2003) have investigated the causes of flooding and the influence of land use, soil characteristics and antecedent catchment moisture conditions in the Oder catchment, covering parts of the Czech Republic, Poland and Germany, using LISFLOOD. This study has revealed that land use changes between 1780 and present have increased peak discharges in the catchment due to urban growth. Moreover, the proposed flood control measures have led to a remarkable improvement and reduction of flood risk in the downstream reaches.

Semadeni-Davies et al., (2008) have also assessed the potential impacts of climate change and continued urbanization on stormwater flows to a suburban stream of central Helsingborg, South Sweden, using drainage simulations for present conditions as well as two climate (medium and high gas emission) and two water management storylines for subdivision. It was found that city growth and projected increases in heavy rainfall, both together and alone, are set to raise peak flow volumes and increase flood risk, while system renovation and installation of sustainable urban drainage systems (SUDS) can largely allay the adverse impacts of both urbanization and climate change.

Different conclusions have been drawn from various studies investigating the impact of urbanization and/or climate change on the peak flows and flooding extent. Many studies have been conducted to assess the impact of climate change on streamflow variability affecting flooded areas and some studies to investigate the impact of land use changes including urban lands on streamflow from an urban catchment. Nevertheless, the combined effects of these two variables have seldom been studied which has been coming under increased scrutiny in recent years in order to assess the regional influence of climate change on urbanizing watersheds. Moreover, integration of the urban sewer system with the neighbouring river system has never been brought under any study. This explains that a lot has to be done with appropriate methods as detailed as possible before selecting integrated water resources management adaptive measures to reduce the impact of these two variables on urban flood risks.

1.6 Problem statement

The relation of the city of Turnhout to its water system is today at the centre of a (double) controversy according to Christian Nolf., (2010):

a. The small river and the growing city

On one hand, the urban area, situated in the most upstream part of the valley, is considered as the main factor disturbing the natural rhythm of the river. More specifically, the extensive growth of the city's build up area during the last decades and consequently the proportion of paved surface has led to less capacity for natural infiltration, to more surface runoff and to overflows of the sewer system in case of heavy rainfall. As a consequence the river is downstream exposed to an increasingly volatile water regime (a very high reactivity to rainfalls), accentuated by the fact that the riverbed itself has been the object of successive series of transformations (straightening, removal of meanders, canalization and programs of rationalization of the farming parcels (in Dutch 'ruilverkaveling')). This combination leads to a faster flow of larger quantities of water in a valley that cannot physically absorb it, causing increasingly frequent and harmful flood events affecting agricultural lands downstream.

b. The densifying city and the limits of the sewer system

Next to the flood issue in the river downstream, the ongoing urban growth is also challenging the city itself. Under the pressure of the housing and the industrial sectors, the city authorities decided to encourage the densification within (for housing) or next to (for industries) the existing cores. Consequently, most empty plots and left over spaces located within the urbanized areas are today the object of new developments. This densification program optimizes the existing urban facilities and prevents further urban sprawl but will require an extra contribution from the water evacuation infrastructure. The present sewer system, consisting at 95% of a combined (rain+ waste) water system, would indisputably saturate if it gets connected to the new neighborhoods, which would cause harmful presence of water in the streets and lead to even more frequent polluting overflows in case of an important heavy rainfall.

In a nutshell, the increasingly frequent flooding in the lower part (agricultural land) of the Aa valley is caused, on the one hand, due to peak runoffs from paved area upstream (Turnhout urban area, sewer system including industrial platforms) and frequent overflows out of the mixed sewer system (rain+waste water); on the other hand due to straightening, deepening and channeling of the river bed (i.e. reduction of the sponging role of the creek valleys) in favour of housing, intensive agricultural land and industrial platforms along its course. So, there is a need to control the discharge of surface water coming out of the urban area, by means of retaining, infiltrating and containing water upstream of the valley.

1.6.1 Existing dynamics and projects in Turnhout

The city of Turnhout has undertaken an ambitious plan of water management in partnership with the river basin coordinator not only to solve the current flood issue, but also to prevent more water problems with the future densification of its urban fabrics. It consists of different measures to favour water retention within the urban area, including the decoupling of the waste and rainwater evacuation systems to relieve the sewer network and therefore to limit the risks of overflows. This will imply the development of a new parallel (piped) rainwater collector system and also require new spaces for water infiltration and buffering.

Next to the management of spatial claims for new developments and for a better control of water evacuation, the city is also active with a program for the enhancement of the Aa river valley (including a new bicycle path) and its tributaries along with the development of community facilities.

In Appendix A, figure A.4 shows the development projects undertaken in the city of Turnhout and figure A.5 shows the city's sewer system including both existing and planned projects depicting the causes of vulnerabilities in the lower valley of the rivers.

1.6.2 Proposed water management alternatives

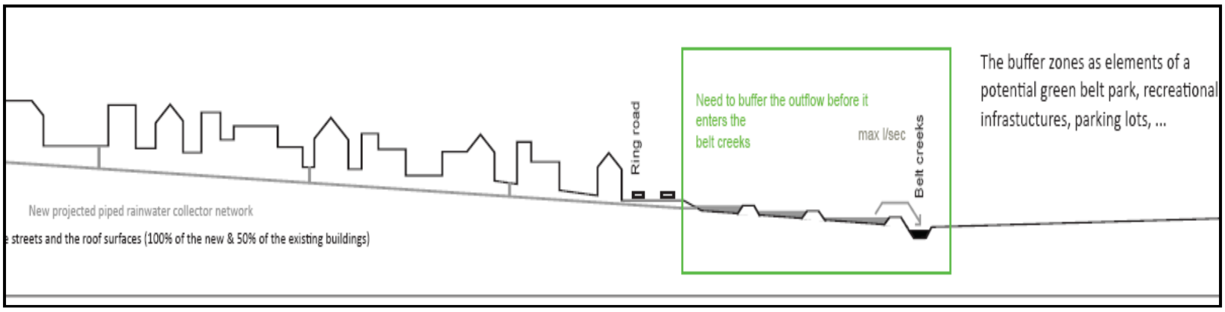
Three alternatives have been proposed and will be implemented by the city authority of Turnhout in collaboration with K.U. Leuven University. The first option is an urban vennen (Dutch word "vennen' means fens) where a ven is a small lake, usually shallow, with fluctuating water levels. The existing and potentially free spaces micro valleys (lines of lower points) and depressed areas within the city may function as potential infiltration zones to which rainwater can be conveyed through the streets (streets act as rivers) with the steepest slopes in case of heavy rainfall (figure A.6 in Appendix A).

The provision of infiltration facilities allows the water management to tackle peak flow and low water problems together. Through such facilities, not only the risk of flooding during heavy rainfall is limited but also the expected future increase in water deficit can be reduced.

This can be achieved through the construction of storm water tanks, but also through short term storage on the street or in public spaces (Figure A.7 in Appendix A). Also through the construction of local depressions in public areas (parks or other green areas, playgrounds, etc.) a large quantity of water may temporarily be stored in case of heavy rainfall (Willems, 2009b). The second alternative is a 'drempel (threshold) park' where the output from the projected new rainwater collecting network (RWA) will be buffered before entering the surrounding rivers at their threshold. These buffering elements can act as transitions between the urban centre and the open space fens (figure 1.5).

(a)

(b)



(c)

Figure 1.5: (a) and (b) representation of two example of buffer zone; (c) schematic representation of the way of the water from the new collector system to the river as IWRM alternative proposed by the Province of Antwerp.

The last alternative is a 'dammed creek' where the water, in case of heavy rainfall, will be contained along the creek valleys. In the first approximation, the topographical bottlenecks and infrastructural barriers, such as urban fens, canal, highways etc are sought to be the retention devices in the upstream region of the city (figure A.9 in Appendix A). Other alternatives have also been planned by Province of Antwerp along the rivers, such as meandering and extension of some river cross sections of the river Visbeek (Province of Antwerp, 2004 & 2006). Figure A.9 in Appendix A shows all the proposed IWRM actions by the city authority and Province of Antwerp.

Figure A.10 and A.11 in Appendix A describe the flooding vulnerability and existing context in the further lower valley of Aa river. Therefore, another river authority, Vlaamse MilieuMaatschappij (VMM), has studied the flooding problems along two main rivers within Kleine Aa and Nete catchments and proposed a number of flood management alternatives divided into two categories: non-agricultural and agricultural. The alternatives for nonagriculture related flooding problems that occur in the winter are as follows (BELGROMA, 2002):

- removal of the Aa dam downstream of Gierlebaan to Tielen;
- removal of the wooden bulkhead on the river Aa near the bridge Veedijk in Turnhout;
- increasing the left bank level of the Aa along French Seghers sanctuary near Turnhout;
- raising the dikes along the right bank of the Kleine Nete to 9.70 m TAW from Grobbendonk downward thrust to the Throne Bridge;
- moving and raising the dikes along the right bank of the Kleine Nete to 9.70 m TAW from Grobbendonk downward thrust to the Throne Bridge;
- increasing right bank and left bank of the lower Little Nete from Grobbendonk downward thrust to the Throne Bridge;
- setting a new dam level at Herentals near limnigraph 054, 12.05 m TAW (instead of 11.55 m) to create a water logging;
- establishment of the technical nature "Hellekens' in Herentals so that the water levels of Kleine Nete increases at the height of the Olen plants.

The alternatives and assessments made for agriculture related flooding problems that occur in the summer are as follows:

- establishment of the technical nature "Hellekens' in Herentals so that a the water levels of Kleine Nete increases at the height of the Olen plants objectives;
- examination of the influence of different cutting patterns between the Aa and Grobbendonk Gierle;
- assessment of the impact of a possible cull sludge in the Aa;
- assessment of the impact of the combination of an herb-and silt clearance;
- assessment of the impact on the water level of the flattening of a dam on the river Aa at the Tielen windmill arm with excessive growth of herb.

An integrated water management alternative can be selected with high effectiveness when all these management strategies, proposed by the city authority of Turnhout, Province of Antwerp and VMM, are taken into account together with water quality issue.

1.7 Thesis objectives

The overall aim of this master thesis is to develop an integrated river model in order to understand the effect of urbanization and climate change on the river flow causing severe urban flooding and to assess the effectiveness of integrated water resources management options to reduce this flooding problem. Final aim of this thesis is to support spatial planners working in 'FWO Water research in urbanized Flemish landscapes-Integrating civil engineering and urban design approaches in regional landscape development projects' to come across an appropriate alternative solution.

Specifically it can be summarized as:

1. calibration and validation of a lumped conceptual hydrological modelling for the sub-catchments using NAM (Nedbør-Afstrømnings-Model) to replace the rainfall-runoff from PDM (Probability Distributed Model);
2. development, calibration and validation of a one-dimensional hydrodynamic model for the two rivers, the Aa and the Visbeek, using MIKE 11 hydrodynamic software comparing the results with that of the existing InfoWorks RS models;
3. implementation of a river flood model using quasi-2D floodplain modelling approach to represent the spatial extension of the flooded areas and generation of flood maps using MIKE 11 GIS interface;
4. implementation of buffer elements inside the city and at the end of the new rainwater collector system in Infoworks CS model in order to retain water from the rainwater collector system for the city in case of heavy rainfall events;
5. extreme value analysis to generate composite hydrographs with different return periods both under existing and future climate change conditions of the river flow and urban runoff;
6. comparison of the flooded areas to assess the effectiveness of flood management options and to assess future vulnerability.

Chapter 2

The lumped conceptual hydrological model

2.1 Introduction

The importance of hydrological models lies in their ability to predict runoff from a catchment which usually supports improved water resources management. These models can be used to assess the likely impacts of future hydrological changes by extrapolating from observed measurements (Taye, 2009).

The input data requirement of a model, the availability of the data and the purpose of application are the determining factors for the selection of a hydrological model that should be applied for the specific case under study. In general, models can be of two types: ‘lumped’ or ‘distributed’ (Taye, 2009). A catchment is represented as a single entity or a small group of entities, such as reservoirs, in ‘lumped’ models. ‘Lumped’ models simulate state variables and fluxes into and out of the catchment as a whole. On the other hand, the catchment is divided into many entities in ‘distributed’ models, each representing small parts of the catchment, and the state variables and fluxes between the entities are determined across the catchment (Silberstein, 2006).

Physically-based distributed-parameter models are complex in terms of structure and input requirements and can be expected to provide adequate results for a wide range of applications. On the other hand, simpler models which have a smaller range of applications can yield adequate results at greatly reduced cost, provided that the objective function is suitable (Jiang et al., 2007). In general, physically based models incorporate too many processes and too complex formulations at a too detailed scale in the context of climate change and river flooding. Therefore, the conceptual models seem to be an attractive alternative because of their ability to capture the dominating hydrological processes at the appropriate scale with accompanying formulations. The conceptual models can therefore be considered as a nice compromise between the need for simplicity on one hand and the need for a firm physical basis on the other hand (Booij, 2005). A disadvantage is that it is generally impossible to determine parameters from field measurements and therefore it is necessary to use calibration technique (Refsgaard, 1996, cited in Booij, 2005).

In this study a lumped conceptual hydrological model NAM was developed to generate rainfall-runoff for the two subcatchments under studies. The model structure and its description is available in the following sections. The results of NAM model are also used to generate composite hydrographs for the hydrodynamic model simulations.

2.2 Study area description

The study focuses on the city of Turnhout in the Antwerp Kempen, North of Flanders in Belgium. The city is located on a plateau between two small rivers, the Aa and the Visbeek, belonging to the Kleine Nete basin (figure 2.1). The Aa river has a length of 19 km with a contributing area of 52.52 km^2 while the Visbeek river has a length of 11 km with a contributing area of 19.19 km^2. The main tributary of the Aa is Liermansloop and the main tributaries of the Visbeek are Zijtak-Visbeek and Rondvenloop.

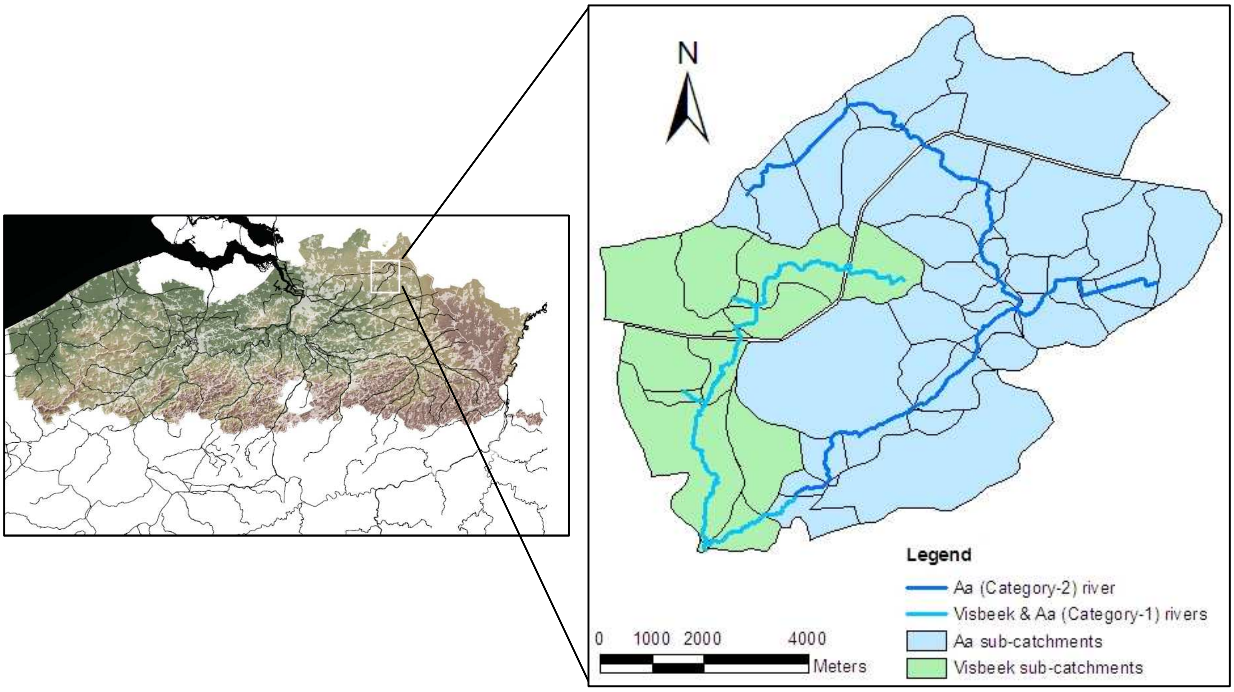


Figure 2.1: location of the city of Turnhout in Flanders, the sub-catchments and the rivers.

A Digital Elevation Model (DEM) with a resolution of 5 meters for the whole Flemish region exists. The DEM for the study area has been displayed in figure 2.2, where the upstream part has steeper slopes than the downstream part. The main slope class of the Aa sub-catchment is 0.0-

0.5% and on the other hand, the main slope class of the Visbeek sub-catchment is 0.0-0.05% which define a flat area and the elevation ranges from 15-18 m TAW to 33-37 m TAW for the entire study area.

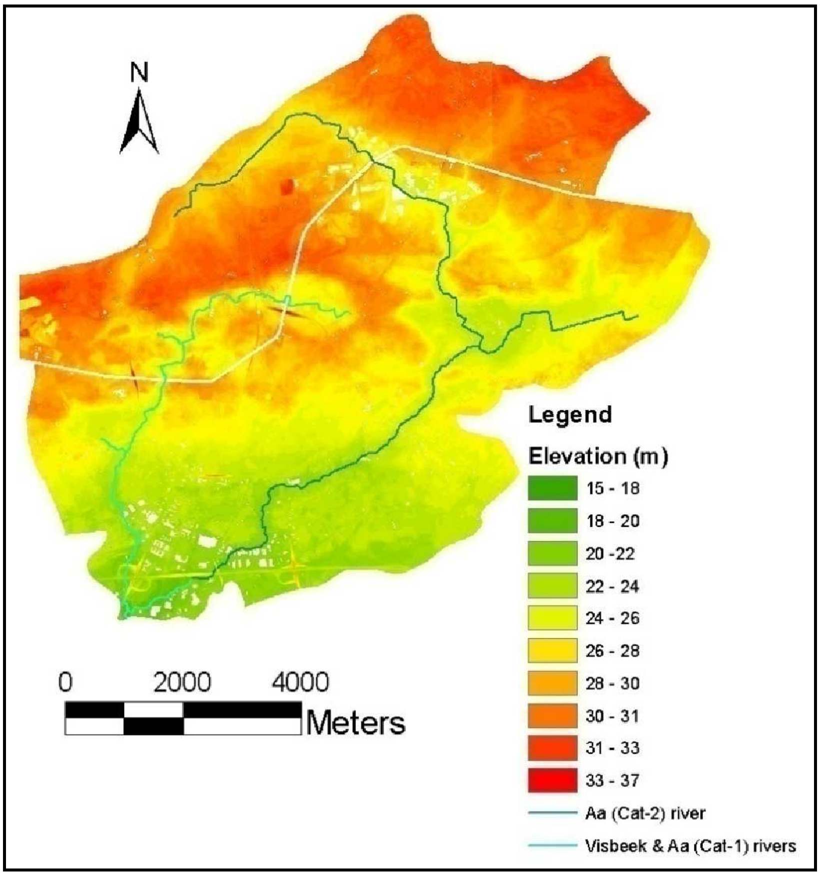


Figure 2.2: the digital elevation model (DEM) comprising the Aa and Visbeek rivers along with their tributaries.

Table 2.1 shows that the land use in both sub-catchments is dominated by the urban area (34.52%), pasture and wet meadow (32.44%), and forest (20.47%) (see also figure 2.3). In addition more than 50% of the area is covered by sandy soils. Figure 2.4 shows the soil associations for the area which are further described in table 2.2.

Land use	Area (km2)	%	Soil type	Area (km2)	%
Agriculture/Arable land	6.42	8.91	Sand	39.29	54.5
Corn/tuberous crops	2.05	2.84	Loamy sand	13.6	18.9
Forest	14.76	20.5	Light sandy loam	2.7	3.75
Pasture	23.02	31.9	Sandy loam	0.01	0.01
Wet pasture	0.37	0.51	Heavy clay	0.01	0.01
Urban	24.89	34.5	Peat	0.82	1.14
Water bodies	1.15	1.6	Dunes	1.09	1.51
Not classified	0.01	0.01	Dug area	14.57	20.2

Table 2.1: Soil type and land use in the Aa and Visbeek sub-catchments (adapted from the Province of Antwerp, 2004 & 2006)

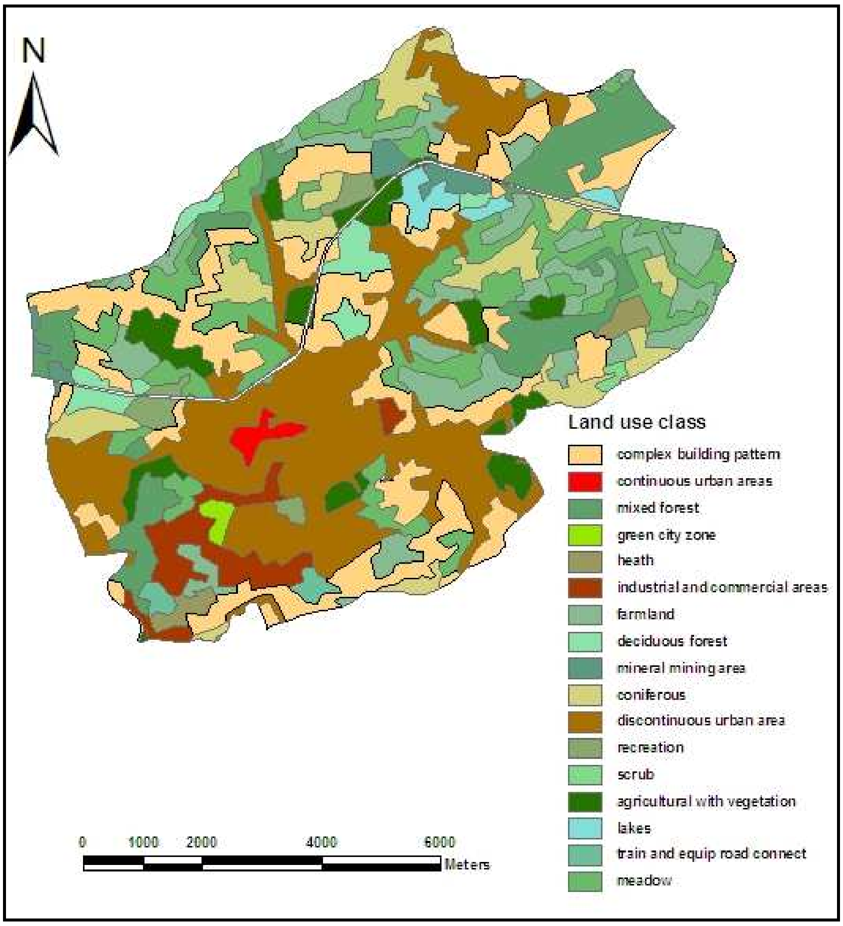


Figure 2.3: land use classification of the city of Turnhout (adapted from Province of Antwerp, 2004 &2006)

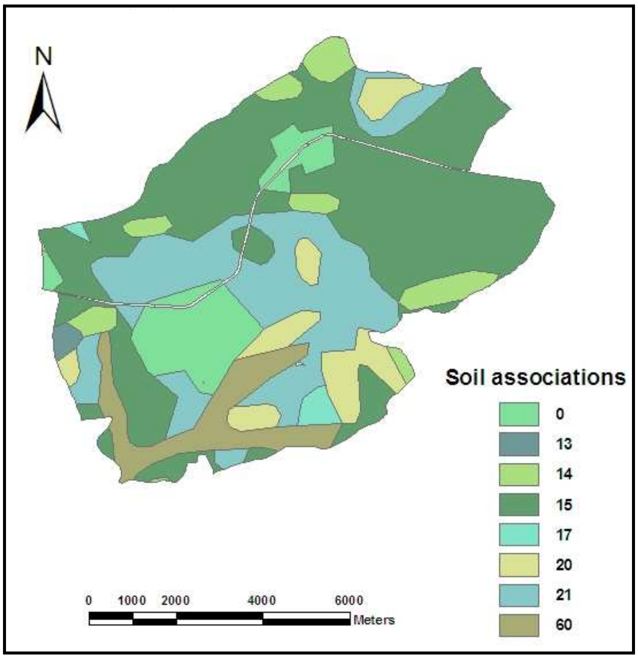


Figure 2.4: soil associations map for the city of Turnhout (adapted from Province of Antwerp, 2004 &2006).

Association	Description
13	Sandy, usually dry or very dry with no profile development appearing on many inter fluvial, not calcareous humus and giving them low agricultural value, but suitable for forestry, recreational resorts and residential neighborhoods.
14	Dry silty sand and sandy soils with humus and / or iron B horizon.
15	Silty sand and sandy soils with humus B horizon and / or iron B horizon.
17	Wet sand to light sandy loam soils with color or B horizon texture B horizon.
20	Dry silty sand and sandy soils with deep, usually grayish anthropogenic humus A horizon.
21	Wet sand and silty sands with deep anthropogenic A horizon. They occur mainly in the Northern part. They are suitable as agricultural and horticultural land, and equally suitable for grazing, depending of the drainage situation.
60	Wet alluvial soils without profile development, of varying texture ranging from sand to clay in the Kempen. Land use on a predominantly pasture with woods in the wettest areas (usually poplar). They are centrally located around the main watercourses.

Table 2.2: common soil associations in the city of Turnout and surroundings (adapted from Province of Antwerp, 2004 & 2006) (translated from Dutch).

2.3 Rainfall-runoff model (NAM)

NAM is the abbreviation of the Danish "Nedbør-Afstrømnings-Model", meaning precipitation-runoff-model. This model was originally developed by the Department of Hydrodynamics and Water Resources at the Technical University of Denmark. NAM is a lumped conceptual hydrological model, describing the hydrological system taking into account four linear reservoirs in series. The NAM hydrological model simulates the rainfall-runoff processes occurring at the catchment scale and can be characterized as a deterministic, lumped, conceptual model with moderate input data requirements. NAM forms part of the rainfall-runoff (RR) module of the MIKE 11 river modelling system (DHI, 2008).

2.3.1 NAM model structure

The NAM model represents the various components of the rainfall–runoff process by continuously accounting for the water content in four different and mutually interrelated storages, i.e. surface storage, lower zone (root zone) storage, groundwater storage and snow storage, where each storage represents different physical elements of the catchment (Madsen, 2000). Of the four storages the surface storage gives rise to overland flow, root zone storage is mainly responsible for interflow, groundwater storage is responsible for base-flow component and snow storage is considered if snow modelling is included in the catchment to be modeled. NAM can also incorporate manmade interventions in the hydrological cycle such as irrigation and groundwater pumping (DHI, 2008). The model structure is depicted in figure 2.5.

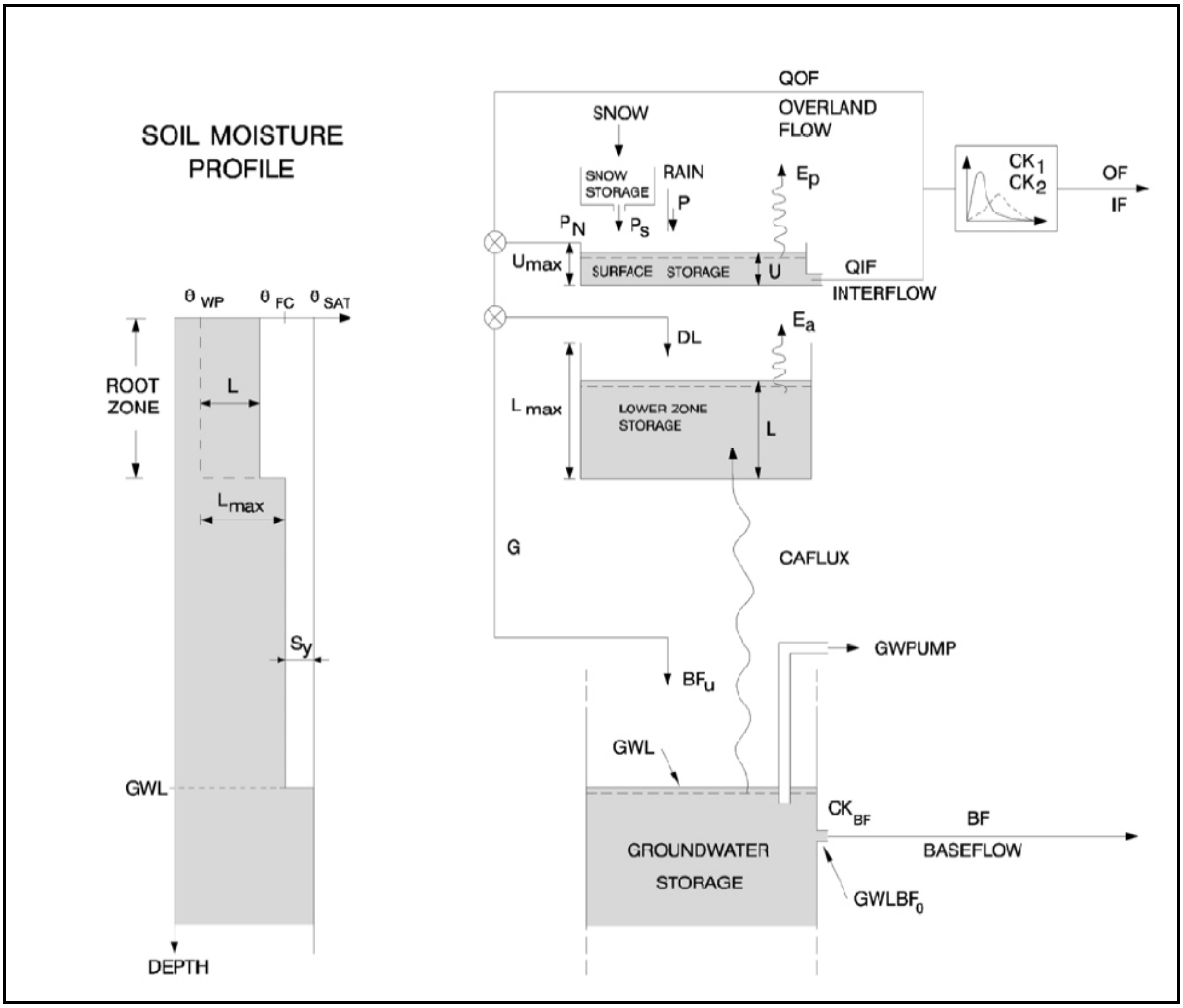


Figure 2.5: schematic representation of NAM module structure**.**

Being a lumped model, the parameters and variables associated with this model represent the average values for the entire catchment. As a result some of the model parameters can be evaluated from physical catchment data, but the final parameter estimation must be performed by calibration against time series of hydrological observations (DHI, 2008). A brief description of the nine most important parameters of the NAM rainfall-runoff model is given in table 2.3.

Parameters	Description	Lower limit	Upper limit
Umax (mm)	Maximum water content in the surface storage: This storage is interpreted as including the water content in the interception storage (on vegetation), in surface depression storages, and in the uppermost few centimeters of the ground.	10	20
Lmax (mm)	Maximum water content in the lower or root zone storage: It can be interpreted as the maximum soil moisture content in the root zone available for the vegetative transpiration.	100	300
CQOF (-)	Overland flow runoff coefficient	0.1	1
CKIF (hours)	Time constant for interflow	200	1000
CK1,2 (hours)	The time constant for routing interflow and overland flow	10	50
CKBF (hours)	The time constant for base-flow	1000	4000
TOF (-)	Threshold value for overland flow, interflow and recharge	0	0.99
TIF (-)	Threshold value for overland flow, interflow and recharge	0	0.99
TG (-)	Threshold value for overland flow, interflow and recharge	0	0.99

Table 2.3: the parameters of NAM model with their usual range (DHI, 2008).

Surface storage

In this model moisture intercepted on the vegetation as well as water trapped in depressions and in the uppermost, cultivated part of the ground is represented as surface storage. *Umax* denotes the upper limit to the amount of the water in the surface storage. The amount of water, *U*, in the surface storage is continuously diminished by evaporative consumption as well as by horizontal leakage (interflow). When there is a maximum surface storage, some of the excess water, will enter the streams as overland flow, whereas the remainder is diverted as infiltration into the lower zone and groundwater storage (DHI, 2008).

Evapotranspiration

Evapotranspiration requirements are initially fulfilled from the surface storage at potential rate. If the moisture content *U*, in the surface storage, is less that this requirement, the root activity plays an important role in fulfilling the remaining fraction of requirement from the root zone storage at an actual rate. The actual evapotranspiration (E_a), proportional to the potential evapotranspiration (E_p), varies linearly with the relative soil moisture content, L/L_{max}, described mathematically by equation 2.1. The amount of moisture content in the lower root zone is a determining factor for the amount of ground water recharge, the interflow and overland flow components.

$$E_a = \left(E_p - U\right) \cdot \frac{L}{L_{\max}} \qquad \text{Eq.2.1}$$

Overland flow

Water moves to the overland flow as well as to infiltration after the surface storage have attained full of its maximum capacity ($U>Umax$). The overland flow is assumed to be proportional to the excess water and vary linearly with relative soil moisture content. The equation governing this flow component is as follows.

$$QOF = \begin{cases} CQOF \cdot \dfrac{L/L_{\max} - TOF}{1 - TOF} \cdot P_N & for\ L/L_{\max} > TOF \\ 0 & for\ L/L_{\max} \leq TOF \end{cases} \qquad \text{Eq.2.2}$$

where:

- QOF denotes overland flow;
- $CQOF$ is the overland flow runoff coefficient ($0 \leq CQOF \leq 1$);
- TOF is the threshold value for overland flow ($0 \leq TOF \leq 1$);
- P_N is the proportion of excess water;
- L/L_{max} the relative soil moisture content.

Interflow

The contribution of interflow is assumed to be proportional to the soil moisture *U* and to vary linearly to relative soil moisture content of the lower zone storage. This relation is given mathematically in the following equation.

$$QIF = \begin{cases} (CKIF)^{-1} \cdot \dfrac{L/L_{\max} - TIF}{1 - TIF} \cdot U & for\ L/L_{\max} > TIF \\ 0 & for\ L/L_{\max} \le TIF \end{cases} \qquad \text{Eq.2.3}$$

where:

- QIF denotes interflow;
- $CKIF$ is the time constant for interflow;
- TIF is the root zone threshold value for interflow ($0 \le TIF \le 1$);
- U is the moisture content.

Ground water recharge

A fraction of the rainfall contributes to overland flow and to interflow and then the remaining part is used to increase the soil moisture content. The amount of water infiltrating to the groundwater for recharge depends on the soil moisture content in the root zone. The groundwater recharge is mathematically described by Equation 2.4.

$$G = \begin{cases} (P_N - QOF) \cdot \dfrac{L/L_{\max} - TG}{1 - TG} & for\ L/L_{\max} > TG \\ 0 & for\ L/L_{\max} \le TG \end{cases} \qquad \text{Eq.2.4}$$

where:

- G denotes groundwater recharge;
- TG is the root zone threshold value for groundwater recharge ($0 \le TG \le 1$).

The routing of interflow is performed through two linear reservoirs in series with the same time constant *CK1,2*. On the other hand the routing of overland flow is also based on the linear reservoir concept but with a variable time constant. The base-flow from the groundwater storage is calculated as the outflow from a linear reservoir with time constant $CKBF$ (DHI, 2008).

2.3.2 Available data and pre-processing

The basic input requirements for the NAM model consist of meteorological data, hydrological data, model parameters and initial conditions. The basic meteorological inputs are rainfall and potential evapotranspiration averaged over the whole catchment. The required hydrological data is the discharge at the outlet of the catchment required for model calibration and validation. The necessary input data, point rainfall and discharge, were originally obtained from HYDRONET database operated by Vlaamse MilieuMaatschappij (VMM), while evapotranspiration data were obtained from Royal Meteorological Institute (KMI) for the Uccle station. The rainfall and discharge data are for the period from 1986 to 2001, while the evapotranspiration data is the period from 1900 to 2003. Figure 2.6 shows the measurement's station of Turnhout where the rainfall and discharge data were recorded, while the position of Uccle station, where the evapotraspiration data was recorded, is located outside of the study area. The time series of discharge is observed at Turnhout flow gauging station on the Aa river (station ID L10_064), downstream of the Aa catchment.

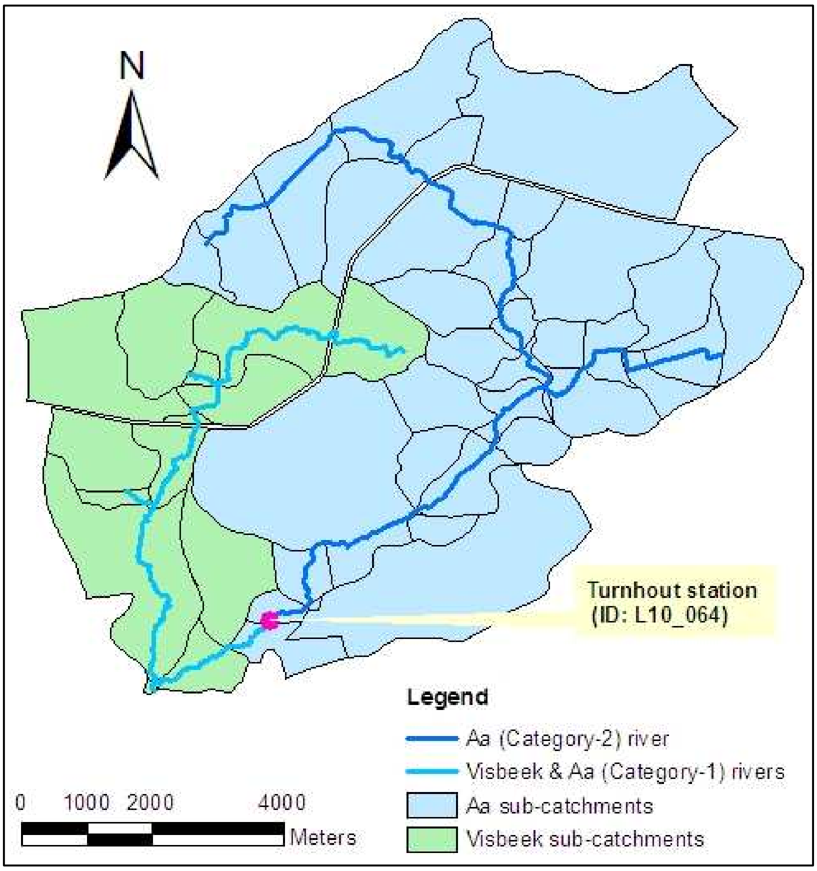


Figure 2.6: representation of Aa and Visbeek sub-catchment and position of Turnhout's rainfall and discharge measurements station.

2.4 Calibration methodology

2.4.1 General description

NAM facilitates two types of model calibration procedures, the manual calibration and automatic calibration. During the manual calibration the parameters are adjusted by trial and error until the simulated river discharges match as close as possible to the observed river discharges. The auto calibration is based on a multi-objective optimization strategy in which different calibration objectives are optimized simultaneously.

Generally four objectives should be considered in the process of calibration taking into account the trade-offs between these objectives (DHI, 2008). These objectives are:

1. a good agreement between the average simulated and observed catchment runoff (i.e. a good water balance);
2. a good overall agreement of the shape of the hydrograph;
3. a good agreement of the peak flows with respect to timing, rate and volume;
4. a good agreement for low flows.

In manual calibration, it is recommended to change one parameter at a time to understand the effect of the parameter on the different flows. At first the maximum water contents in the surface storage *Umax* and the root zone storage *Lmax* should be adjusted to have good water balance.

In the next stage, the peak flows can be dealt with by changing *CQOF* and *CK12*. The former is used to adjust the volume of the peaks and the latter is responsible for the shape of the peaks. In case of the shape of the base-flow recession, the base-flow time constant *CKBF* plays the major role. The volume of the base-flow is determined by the other flow components. The more the overland and interflow are, the less the base-flow will be.

The root zone thresholds related to the three flow components (*TOF*, *TIF and TG*) need to be adjusted to improve the simulation results. These parameters are threshold values where generation of overland flow, interflow or recharge will not be successful if the relative moisture content of the lower zone storage, *L/Lmax*, is less than their values. In auto calibration, four objective functions are to be optimized (DHI, 2008). These are:

1. agreement between the average simulated and observed catchment runoff: overall volume error;
2. overall agreement of the shape of the hydrograph: overall root mean square error (RMSE);

3. agreement of peak flows: average RMSE of peak flow events;
4. agreement of low flows: average RMSE of low flow events.

There might be a problem to find a set of parameters that can satisfy all objective functions simultaneously. A set of parameters that gave good result for the peak flows might not give similar result for the low flows. Therefore, emphasis can be given to one or more of the criteria according to the objective of the study. In case of this climate change impact study, emphasis is given to the extreme flows.

2.4.2 Calibration of NAM parameter

In the case of Aa catchment the basic three components are considered since there is negligible occurrence of snow in the area. The input data for the modelling process was meteorological daily data of precipitation and potential evapotranspiration. The period from 1st January 1997 to 31st December 2001 was considered for calibration.

Using the provided input data the NAM model simulates catchment runoff. The optimum values of parameters that represent the entire catchment were determined by calibrating the model against the observations. While conducting manual calibration the nine basic model parameters were adjusted by trial and error and then the optimum values were obtained through auto-calibration.

At first the maximum amount of water in the surface storage, U_{max}, and the root zone storage, L_{max}, were adjusted to arrive at a smallest water balance error possible. Values of 19 mm and 116 mm for U_{max} and L_{max} respectively gave better result for this catchment.

The parameter *CQOF* was used to adjust the volume of the peaks, where this parameter indicates the amount of water that contributes to the overland flow. A higher value is expected as the catchment is a highly urbanized area and the paved area contributes more to the overland flow although the principal soil type is sandy and loamy sand. The amount of the interflow is determined by *CKIF* parameter. This parameter was also adjusted to have reasonable amount of interflow in the catchment.

Afterwards the shape of the hydrograph for the peaks was adjusted by the time constant for routing parameter (*CK1,2*). As the size of the catchment is relatively small, the response to the rainfall is expected to be fast. The shape of the recession was then adjusted using base-flow time constant (*CKBF*). The threshold values *TOF*, *TIF* and *TG* were calibrated to further fine tune the total amount of the flow components. The calibration of the ungauged Visbeek sub-catchment was done based on the relationships between the model parameters of the gauged sub-catchment characteristics, which are important for the runoff response such as shape, slope, land use and soil type (Timbe, 2007). The characteristics of gauged and ungauged sub-catchments were compared and finally the same NAM model parameters were used for the ungauged sub-catchment.

2.5 Simulation results of NAM model

The final simulation result of Aa catchment has the following optimum values for the nine NAM parameters (table 2.4).

Parameters	**Umax**	**Lmax**	**CQOF**	**CKIF**	**CK1,2**	**TOF**	**TIF**	**TG**	**CKBF**
(mm)	(mm)	(-)	(hours)	(hours)	(-)	(-)	(-)	(hours)	
Values	19	116	0.9	681	44	0.006	0.92	0.15	3610

Table 2.4: summary of calibrated NAM model parameters for Aa catchment

The optimum values of these parameters lie between the range of values as recommended by the NAM manual (DHI, 2008). *CQOF* and *CK1,2* values are reasonable according to the characteristics of the catchment. Since the size of the catchment is small, the fast response to the rainfall input is as expected. Besides, since the area is highly urbanized leading to more impervious area, the infiltration capacity of the area is expected to be lower and in turn higher proportion of the excess rainfall goes to the overland flow component.

The graphical comparison of simulated and observed flow for the whole calibration period is presented in figure 2.7. The less accurate results might be related to the input data uncertainty. Further evaluation of the models has been performed with graphical methods in the next section.

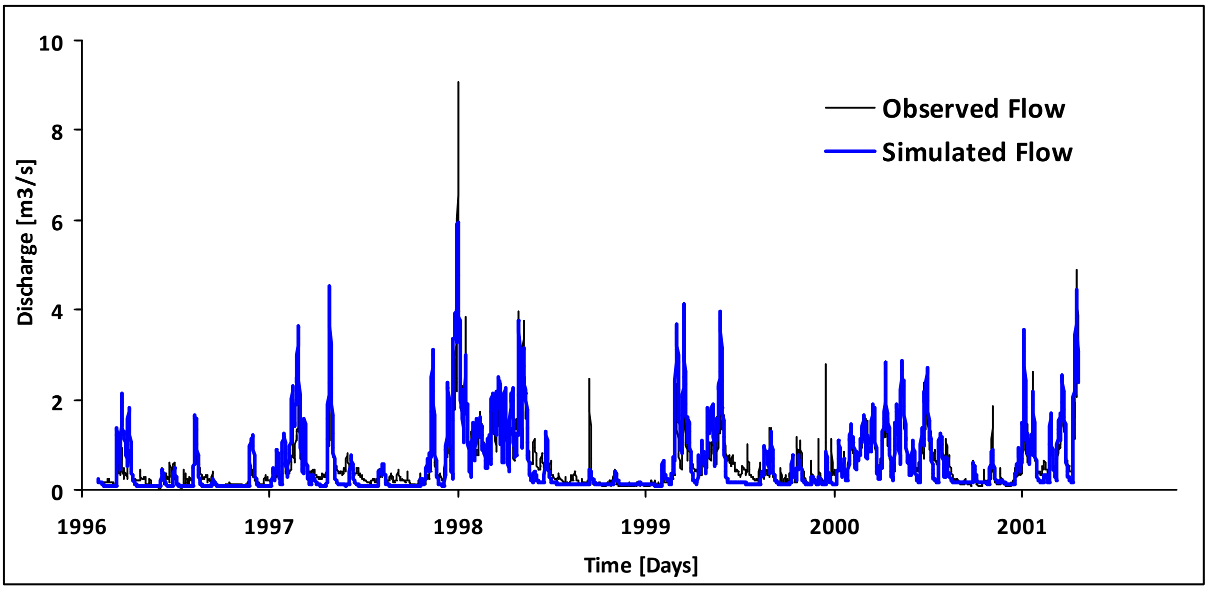


Figure 2.7: calibration result of NAM for the period 1997-2001.

2.6 Model validation

Model validation was checked using data that was not used for the calibration purpose. In this case the period 1st January 1993 to 31st December 1996 was used. The model gives a good match between the modeled and the measured discharge for most of the period with NSE equal to 0.61 and water balance discrepancy equal to -12.5%. The performance evaluation of NAM model, using multi-objective set of statistics with supporting graphical criteria, was also carried out for the validation period. The figures are shown in Appendix B.

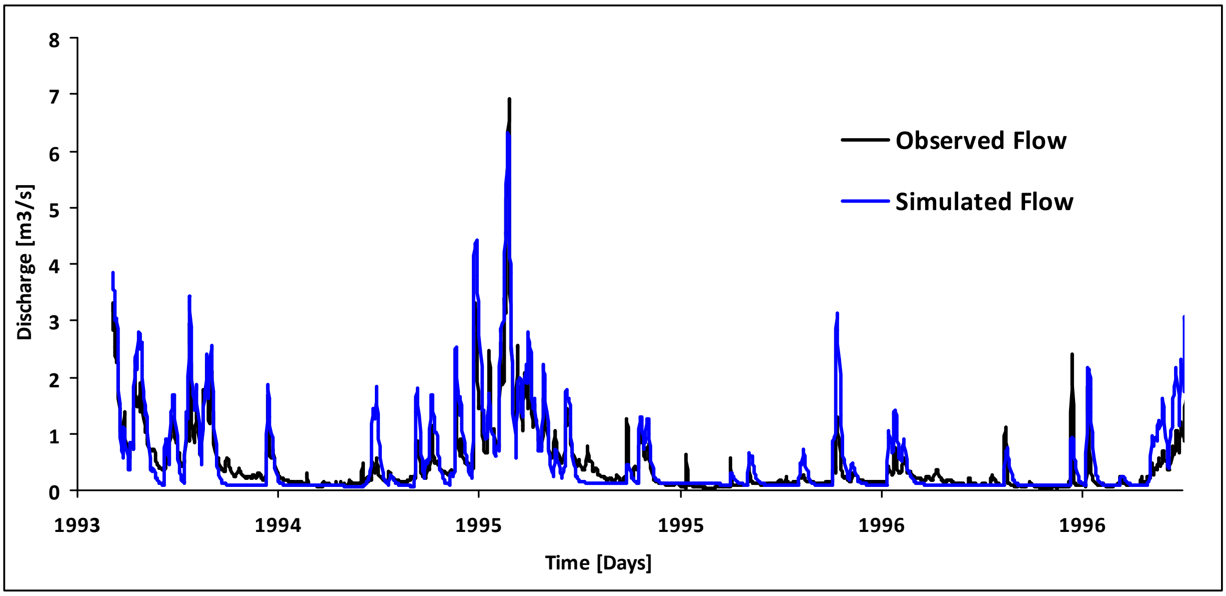


Figure 2.8: validation results for NAM model for the period between 1st January 1993 to 31st December 1996.

2.7 Model performance evaluation

During model calibration, as well as during model evaluation/validation, evaluation of model performance is needed. This is primarily done based on goodness-of-fit statistics. These statistics have the advantage, but also the disadvantage that they largely summarize the goodness-of-fit information only in a few numbers and values. The statistics can be complemented with graphical goodness-of-fit plots. In general, these plots compare observed and modelled values of a flow series (as also used on the basis of the statistics) in a graphical way, and provide the modeller with complete information about the goodness-of-fit. For instance, when a poor goodness-of-fit is concluded, the plots can make clear to the modeller whether this poor performance is caused by the high flow events, the low flow events, the quick flow components and/or the slow flow components, etc. Physical insight of the modeller in the hydrology and related model code then can make clear where focus needs to be given for improving model calibration (WETSPRO manual version 2, Willems, 2004).

The performance evaluation of NAM model has been performed by using multi-objective set of statistics with supporting graphical criteria. The WETSPRO tool (Willems, 2004a) is used for this assessment. Separate evaluation of peak flows, low flows and cumulative volume has been performed for the model results.

Model performance evaluation for the peak flow and low flow magnitudes is carried out after Box-Cox transformation of the values. Box-Cox (BC) transformation can be applied to give similar weight to errors at high and low flow conditions (Box and Cox, 1964). The BC transformation is commonly applied in the scope of regression and correlation analyses in an attempt to reach homoscedasticity in the residuals (model errors), and thus for a more unbiased evaluation of the model performance. The Box-Cox transformation (BC), when apply to a variable "x" is thus defined:

$$BC(x) = \frac{\left(x^{\lambda} - 1\right)}{\lambda} \qquad \text{Eq.2.5}$$

where:

- x is the value of the peak or low flow;
- λ is the transformation parameter.

λ ranges from 0 to 1 and is dependent on the data/model in question. A value of λ=0 corresponds to a logarithmic transformation and a value of λ=1 corresponds to no transformation.

In the model performance evaluation the values are selected from the time series using hydrograph separation method. Nearly independent values of high flows are extracted from quick flow periods and nearly independent low flows are extracted from base-flow recession periods. Then the BC transformation is applied with λ=0.25 and the results are depicted in the figure 2.9 and 2.10, confirming a good performance of the hydrological model.

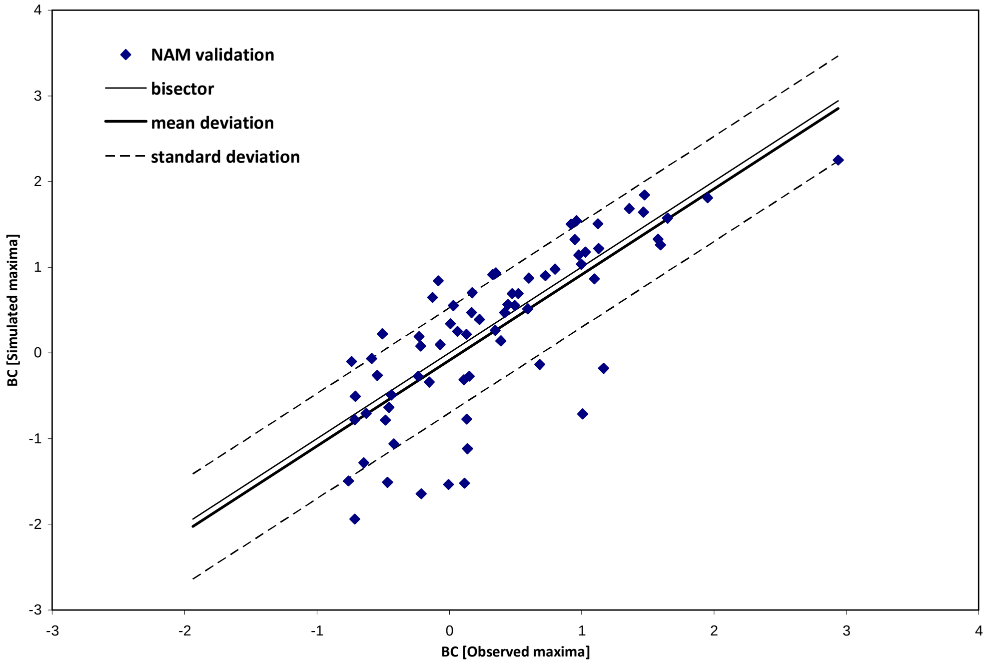


Figure 2.9: comparison between measured and modeled values for the peak flow extremes after BC transformation.

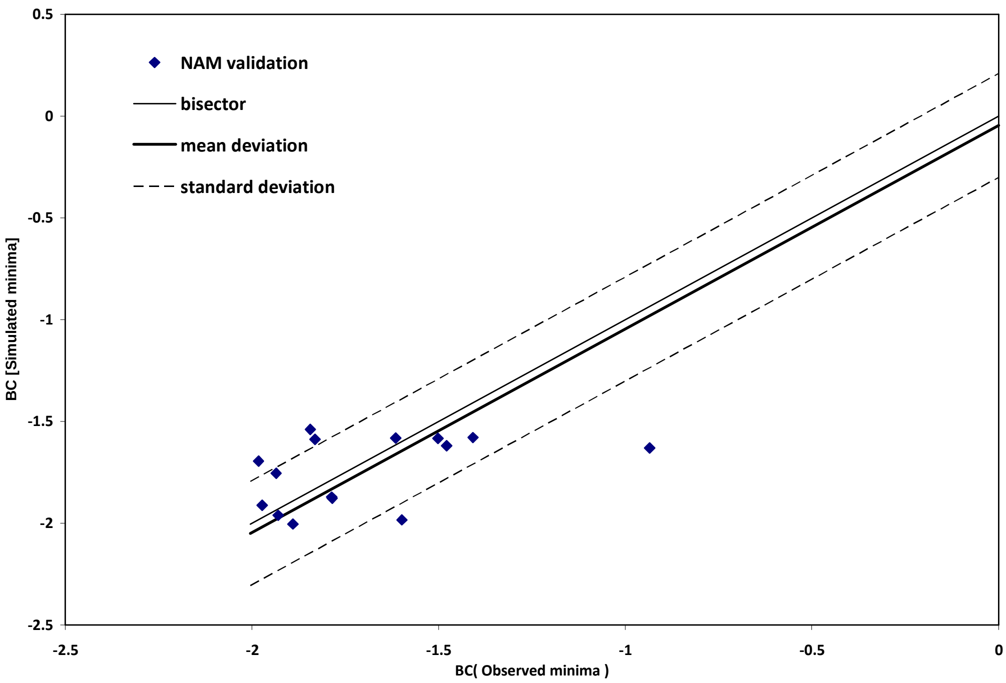


Figure 2.10: comparison between measured and modeled values for the low flow extremes after BC transformation.

The scatter plots for peak flow maxima and low flow minima demonstrates that NAM model performs well in simulating the high flows and low flows as most of the points are concentrated around the bisector line.

Comparison of cumulative volumes for total flow as shown in figure 2.11 is then used to evaluate the overall water balance of the model. The result displays that NAM estimates the total flow for most of the calibration period quite perfectly.

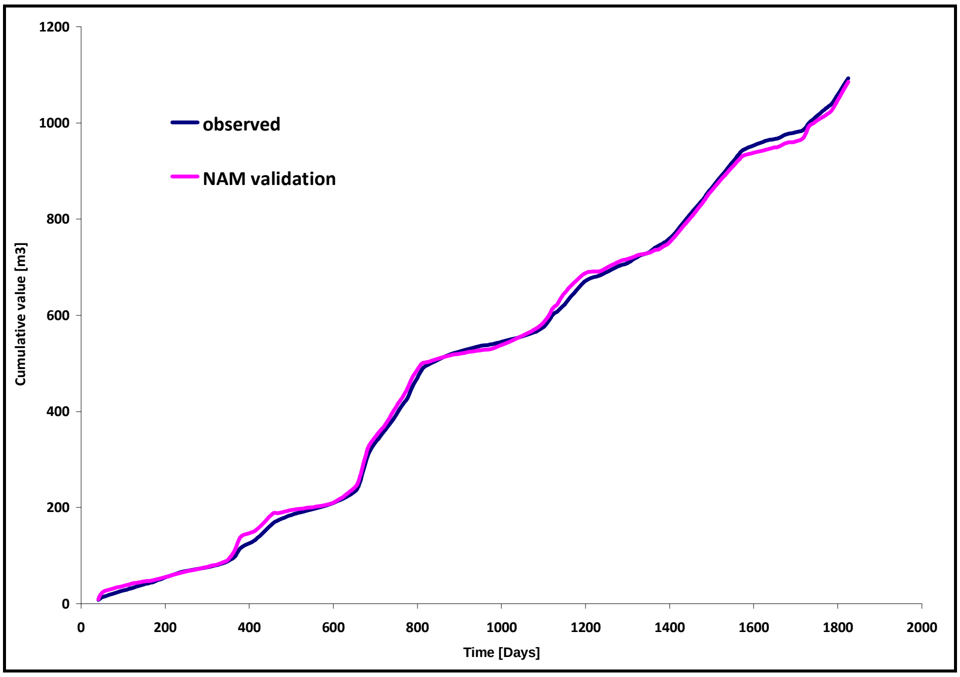


Figure 2.11: evaluation of cumulative volume.

The indirect indicator, flow probability distribution, from the observed historical series is used to evaluate model's prediction ability of more extreme conditions. This distribution behavior is analyzed towards the tail, if the historical observations and the modeled results have similar shape towards their tails.

The analysis of the distribution for the extreme conditions is done by plotting the extreme values in the peak and low flow with the empirical return period, comparing measured and simulated data. The results are illustrated in figure 2.12 and 2.13.

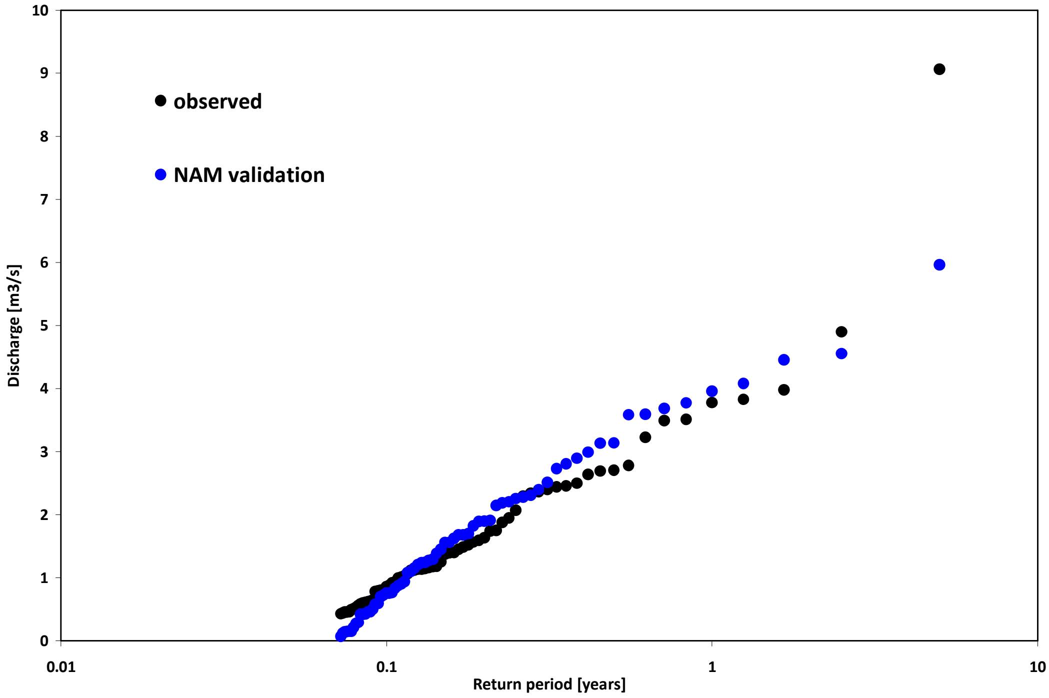


Figure 2.12: graphical evaluation of peak flow extremes value distribution.

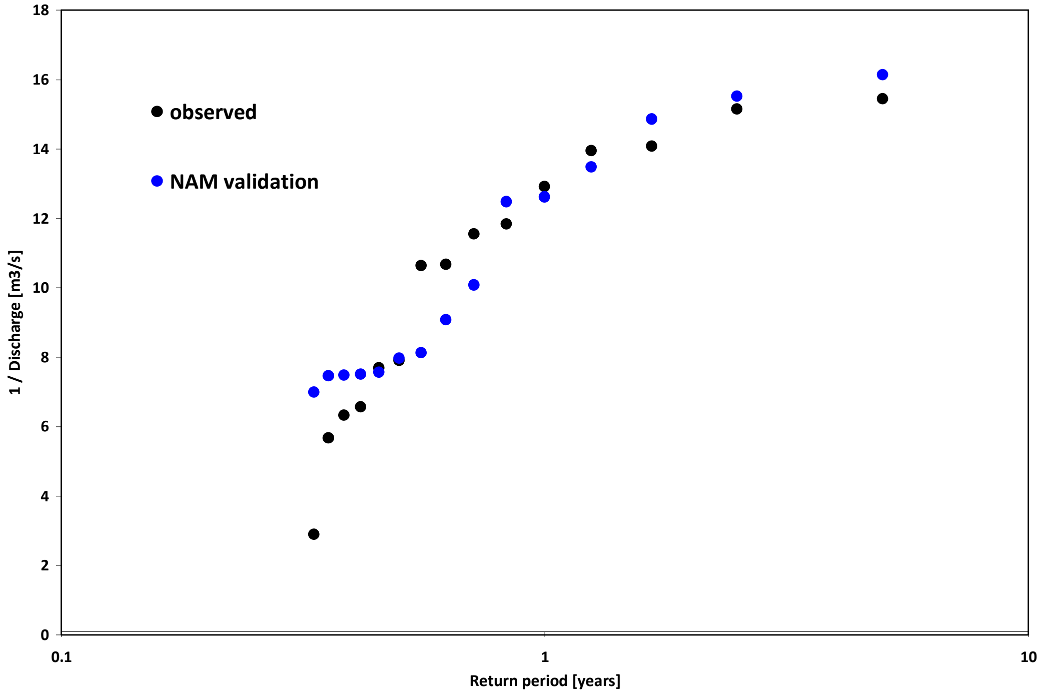


Figure 2.13: graphical evaluation of low flow extremes value distribution.

Comparison between NAM model and the historical observations shows similar shape towards the tail. Hence, the model's performance is good because of its ability to predict more extreme conditions both peak and low.

2.8 Composite hydrographs

The selected historical representative hydrographs do not lead to the considered effect with the same return period at all locations along the river reach. Therefore, composite hydrographs are composed which have the important feature that one single short-term simulation can originate river states with the same safety level at all locations along the river (Vaes et al., 2000). Composite hydrographs can be composed based on the 'discharge/duration/frequency (QDF)' curves where these QDF curves include the appropriate discharges for all the aggregation levels (time scales over which the rainfall-runoff discharges are averaged) starting from the rainfall-runoff time step to the maximum concentration time for the required return period. An extreme value analysis of the NAM rainfall-runoff discharges is made for each aggregation level in order to generate discharges as a function of the aggregation level for every return period, which finally composes a composite hydrograph (Vaes, 1999).

2.8.1 POT selection

The selection of peak over threshold (POT) values is carried out by the WETSPRO tool (Willems, 2004a). The extremes have to be independent in the extreme value analysis and the extraction of independent values is done using one of the 'independency criteria' available in the tool. The number of extremes to be selected is determined by one of these criteria. According to Willems, 2009 two subsequent peak events are considered nearly independent when the following three conditions are fulfilled:

- the time length τ of the decreasing flank of the first event exceeds a time k_p:

$$\tau > k_p \qquad \text{Eq.2.6}$$

- the discharge drops down – in between the two events – to a fraction lower than *f* of the peak flow:

$$\frac{q_{\min}}{q_{\max}} < f \qquad \text{Eq.2.7}$$

or close to the base-flow q_{base}:

$$\frac{q_{\min} - q_{base}}{q_{\max}} < f \qquad \text{Eq.2.8}$$

- The discharge increment $q_{max} - q_{min}$ has a minimum height q_{lim}:

$$q_{\max} - q_{\min} > q_{\lim} \qquad \text{Eq.2.9}$$

This procedure for peak flow selection has three parameters: k_p, f and q_{lim}. It is based on the concept that a peak flow event can be considered largely independent from the next one, when the inter event discharge drops down to a low flow condition or almost to the base-flow level.

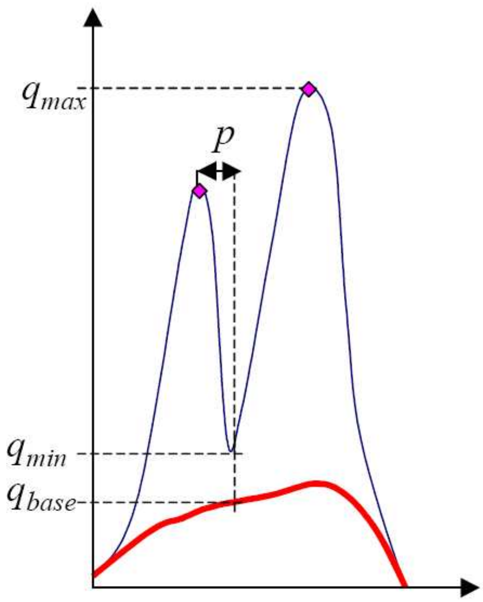


Figure 2.14: parameters for selecting nearly independent POT values (adapted by Willems, 2009).

Since independent extreme values are necessary for further analysis, extreme values from the rainfall-runoff time series have been extracted as peak-over-thresholds (POT) in 'nearly independent' quick runoff events. The moving average approach has been used for five different aggregation levels of 1, 6, 12, 24 and 72 hours. The method and the parameters used for the rainfall-runoff of Aa sub-catchment are presented in table 2.5.

Aggregation level (hr)	1	6	12	24	72
Parameter, f (-)	0.8	0.8	0.8	0.9	0.9
Independency period, k (days)	6	6	6	6	6
Min peak height, q_{lim} (m3/s)	0.25	0.25	0.2	0.15	0.1

Table 2.5: the POT selection criteria for the rainfall-runoff discharges from the Aa sub-catchment for different aggregation levels.

2.8.2 Extreme value analysis

The next step after selecting the nearly independent POT values for the different aggregation level is the analysis of these data separately for distribution plots.

In many water engineering applications, without any long-term time series of observations (for rainfall intensities, discharge, water level, etc…) the description of extreme in the state of water system (such as floods, droughts, or low flow), is of primary importance and can be done with long term simulation results from mathematical model. In extreme value analysis the tail of the distribution describing the probability of occurrence of extreme events is analyzed and modeled by a separate distribution.

As it is known from literature, a limited number of possible distributions, called Generalized Extreme Value (GEV) distributions *H(x)* (Jenkinson, 1955), describes the probability distribution of the maximum observation. *H(x)* is defined as:

$$H(x) = \exp\left(-\left(1+\gamma\cdot\frac{x-x_t}{\beta}\right)^{-\frac{1}{\gamma}}\right) \qquad \text{Eq.2.10}$$

Pickands (1975) has shown moreover that, if only value of *X* above a sufficiently high threshold are taken into consideration, the distribution of excess value over the threshold converges to the Generalized Pareto distribution (GPD) when the threshold increases towards infinity:

$$\begin{aligned} G(x) &= 1-\left(1+\gamma\cdot\frac{x-x_t}{\beta}\right)^{-\frac{1}{\gamma}} & \quad if \quad \gamma\neq 0 \\ G(x) &= 1-\exp\left(-\frac{x-x_t}{\beta}\right) & \quad if \quad \gamma = 0 \end{aligned} \qquad \text{Eq.2.11}$$

If *G(x)* represents the probability distribution of the extremes above a threshold x_t, calibrated to t observations in n period, the return period T, also called recurrence interval of the exceedance level x_t is equal to:

$$T[number\,of\;years] = \frac{n}{t} \cdot \frac{1}{1 - G(x_t)} \qquad \text{Eq.2.12}$$

An important condition that underlines the extreme value theory is the independence criterion for the extreme, that is carried out by WETSPRO, described previously. This influences the number of extreme and also the interpretation of the return period of an extreme event.

In the analysis of the extreme the parameters for the extreme value distributions are calibrated using the hydrological extreme value analysis tool ECQ (Willems, 2004b). For each aggregation level, the independent peak flow events selected are ranked from highest to lowest and extreme value distribution is obtained. The type of the distribution (heavy tail, normal tail or light tail) is judged based on the behavior of the tails and the different slopes in exponential quantile, Pareto quantile and UH-plot distribution plots. For the type of distribution determined, the following parameters are obtained:

- the extreme threshold value (x_t): the point of maximum deviation of the extreme tail of the distribution from the main distribution. It is the threshold that minimizes the *MSE* of the regression, above which the weighted regression has to be performed in the optimal estimation of γ. It is determined by subjective judgment based on the least *MSE* of the extreme value Quantile-Quantile plot;
- slope of the distribution (β);
- the extreme value index (γ) shapes the tail of the distribution of the two preceding cases;
- number of events above the threshold (t).

In this analysis the exponential distribution is considered and the QDF are obtain for different return period.

2.8.3 QDF relationship

Design flows for different return periods for the various aggregation levels were calculated using the equation below:

$$Q = \beta \cdot \ln\left(\frac{n}{t \cdot T}\right) + x_t \qquad \text{Eq.2.13}$$

where:

- Q is the design flow [m3/s];
- β, t and x_t are as defined above;
- n is the number of years of the time series.

The equation is obtained from the Generalized Pareto Distribution (GPD) eq.2.11 with γ=0. QDF relationships are thus derived based on Eq.2.13 and the QDF relationships for the Turnhout station is shown in figure 2.15.

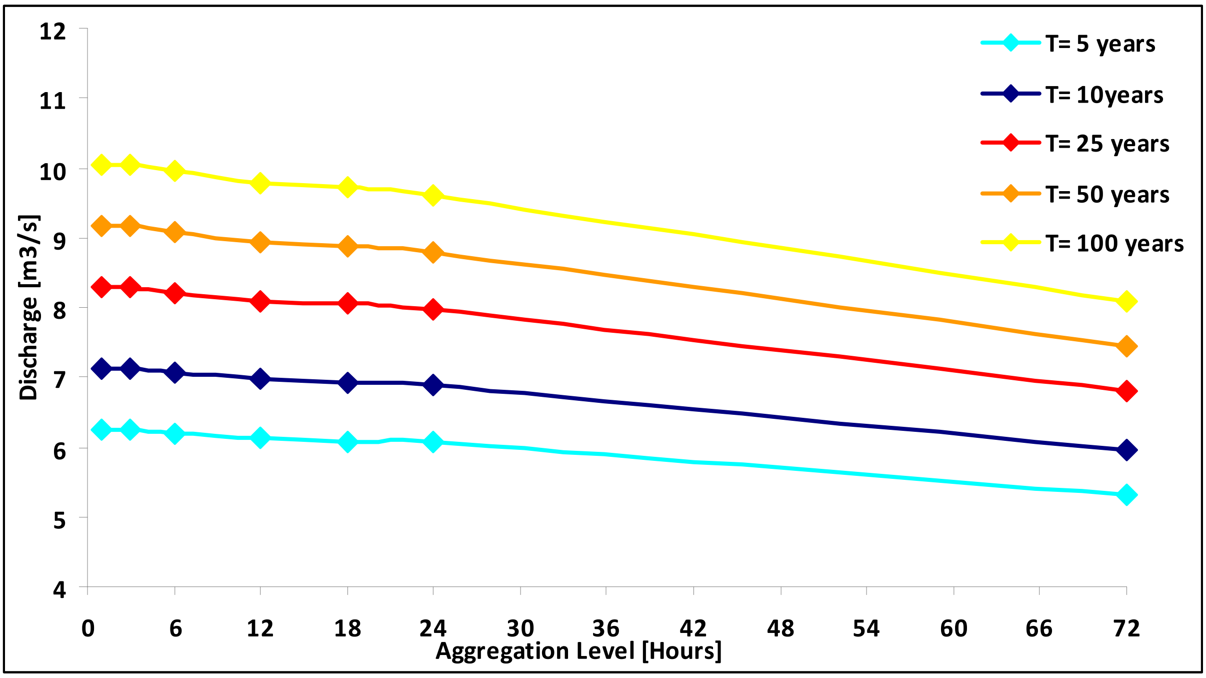


Figure 2.15: generated QDF relationship for the Turnhout station.

2.8.5 Generation of composite hydrographs

Finally the composite storms are determined by setting out the discharge volumes from the QDF-relationship symmetrically around the centre of the storm starting from the shortest storm duration (1 hour) till the longest storm duration t_{max} (72 hours).

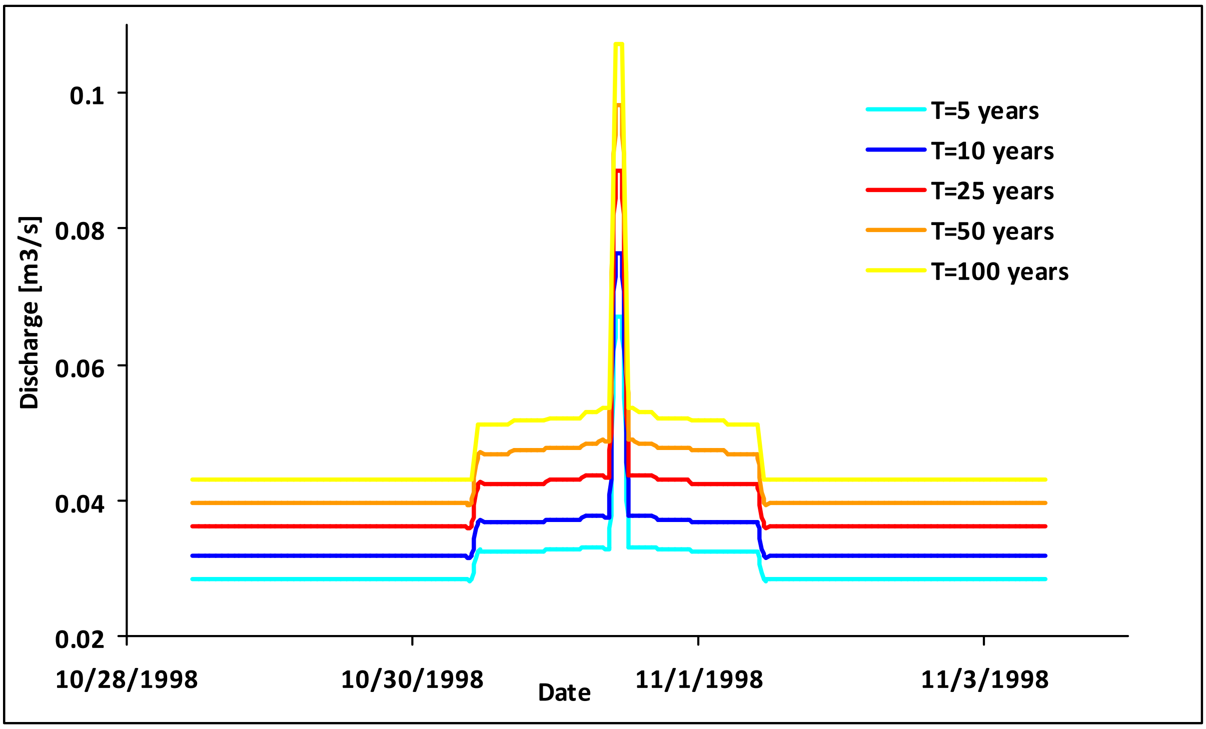


Figure 2.16: composite hydrographs for the upstream boundary of Aa river with different return periods.

These constructed composite hydrographs are synthetic hydrographs which refer to an average discharge corresponding to a specific return period for all the durations that are considered centrally in the hydrograph (Vaes et al., 2000). For both the rivers, the composite hydrographs have first been generated for the rainfall-runoff of the two sub-catchments, for 5 upstream boundary conditions and for the 11 point source boundaries due to urban runoff and used as input in the MIKE 11 hydrodynamic river model. An assumption has been made in this study: the probability of occurrence of the extreme values for the two sub-catchments, due to the 11 urban runoff, are considered statistically correlated. Hence the return period is assumed to be the same in all the confluence between the urban runoff and the rivers, and between Visbeek and Aa rivers. With further investigations in future studies, this hypothesis might be verified.

2.9 Conclusion

The lumped conceptual rainfall modelling system NAM has been implemented for the gauge subcatchment. The discharge time series were generated by the hydrological model using the long rainfall and evapotranspiration series available in and around the catchment.

The calibration was done manually, adjusting the parameter of the model by trial and error until the simulated river discharges matched as close as possible to the observed river discharges. With this approach a unique set of model parameter was found and the results have shown a good correlation between NAM model and the historical observations for the calibration and validation period.

The evaluation of the model is performed by WETSPRO tool, where an extreme value analysis was applied to a long-term time series of rainfall-runoff discharges using multi-objective set of statistics with supporting graphical criteria and separating evaluation of peak flows, low flows and cumulative volume. The distribution of the extreme value, comparing simulated and observed POT, revealed that the model is able to predict more extreme conditions for both peak and low flow. Despite this, more attention may be needed for the calibration of low flows when low flow analysis is to be done using this NAM model.

Finally, the rainfall-runoff discharges from the hydrological model were analyzed for the flood frequency distributions with different aggregation-levels which vary from 1 hour to 3 days. These distributions are summarized in the form of ‘discharge/duration/frequency’ (QDF) relationships and more advanced, in the form of ‘composite hydrographs’ for each sub-catchment and boundary. The use of composite hydrographs for each return period were then used as input for the hydrodynamic model to simulates different scenarios.

Chapter 3

River and floodplain hydrodynamic modelling

3.1 Introduction

River flood modelling plays a vital role in the assessment of flood risk evaluation of floodplains, evaluation of flood damage for project planning, hydraulic structures designing for flood control forecasting of flood-runoff for project operations (US-Army Corps of Engineers, 1994). In order to evaluate the possible adaptive measures according to integrated water resources management, within the framework of flood management, computer models have become very efficient tools to minimize flooding. Generally, the whole process of river flood modelling can be simulated using two different models: a hydrological model and a hydrodynamic model. Modelling starts with a hydrological model (described in the chapter 2) which simulates the relationship between rainfall and rainfall-runoff discharges or thus the quantity of rainfall reaching the watercourses. As such, the hydrographs generated by the hydrological model can then be used as input for the hydrodynamic model.

The hydrodynamic model determines the discharges in the watercourses, resulting from the rainfall reaching the watercourses. The water levels, next to the discharges in the river, are simulated. Therefore, the verification of river overflows over its banks can be performed and flood maps can be generated, usually by means of Geographic Information System. Thus the combination of both the hydrological and the hydrodynamic models establishes the relationship between rainfall and flooding (Timbe, 2007). Figure 3.1 represents general methodology of river flood modelling:

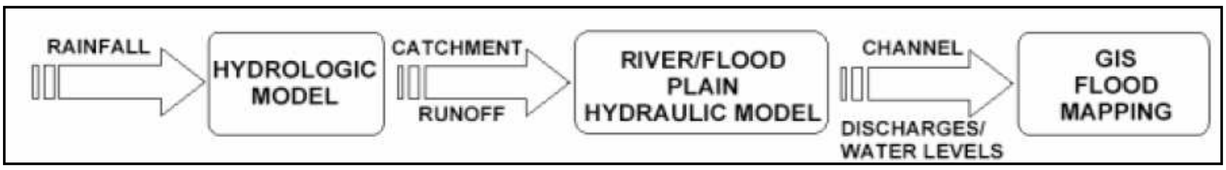


Figure 3.1: schematic representation of river flood modelling (adapted by Timbe, 2007).

In this study 1D hydrodynamic MIKE 11 (DHI, 2008) model was implemented for Aa and Visbeek rivers and the calibration of the model was done by using the results from the existing InfoWorks RS model (Wallingford, 2009). Finally 2D flood maps were generated from the model considering historical and synthetic events for different return periods.

3.1.1 Hydrodynamic river modelling

For hydrodynamic modelling of rivers, models with varying degrees of complexity can be used. Surface water physics are often so complex that they cannot be modelled in full detail. Simplified models are required for predictions, management and long term simulations. The choice of the hydrodynamic model detail depends on the nature of the problem to be solved, the amount of the available data and the available software/hardware. The model choice must be aimed at gaining the most adequate model results given the problem and the limiting factors mentioned above (Radwan, 2002).

The essence of river flow modelling is the description and forecasting of maximum stages in rivers subject to phenomena such as precipitation runoff, tidal influences, hydraulic structure operations and possible structure or dike failure. Moreover, discharge and stage hydrographs, velocities of anticipated currents and duration of flooding are of interest. MIKE 11 (DHI, 2008) and InfoWorks RS (Wallingford, 2009) are some of the examples of one dimensional hydrodynamic software models. These models solve the Saint-Venant equations for shallow water waves in open channel using a finite-difference scheme. The models require data on river geometry (cross-sections perpendicular to the flow direction), the stream bed resistance factors (Manning coefficient) and the time series of upstream and downstream flow discharge or stage height boundary conditions. So, for each grid point (cross-section) average water depth and velocity is then calculated using the finite-difference approximation (Timbe, 2007).

3.1.2 Floodplain modelling

In the last years, the more accurate flood maps have been created through the development of new technologies, such as GIS, GPS and remote sensing at a reasonable cost with improved efficiency and speed. As a result, better floodplain management has become easier for the floodplain managers. A one-dimensional or two-dimensional approach or a coupling system integrating both techniques can be used to perform floodplain modelling.

There are three different one-dimensional approaches that can be differentiated. The first approach (figure 3.2a) is a simplified one-dimensional method where the floodplain is considered as a part of the river cross-section. The assumption in this case is that the water level is considered to be the same in the river and floodplain. It is recommended that this approach should be used when the floodplain and the river are not separated with dikes or embankments, and the floodplains cover a limited transverse dimension (Timbe, 2007).

The second floodplain modelling approach considers floodplain as a storage reservoir (figure 3.2b) where the relationship elevation-volume (or area) can be derived using topographical data (DEM) for each floodplain.

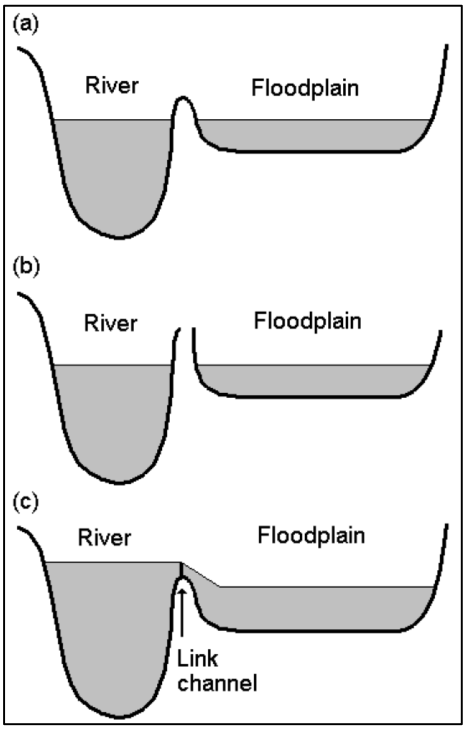


Figure 3.2: one-dimensional floodplain modelling (adapted by Timbe, 2007).

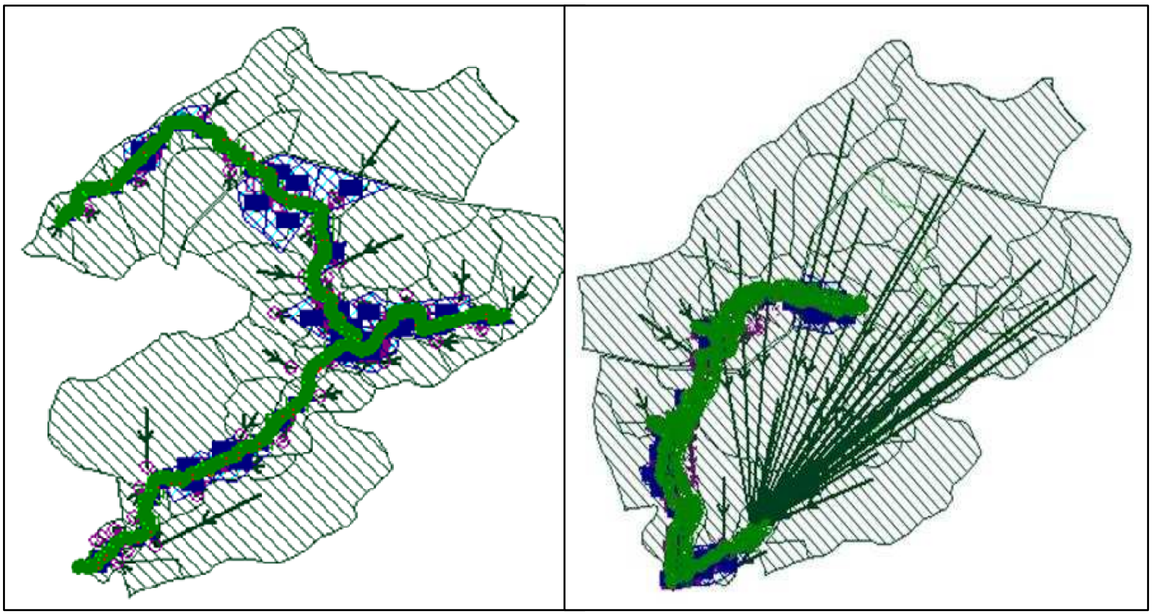
All these storage elements can be connected to the one-dimensional model of the main river by lateral weirs or link channels. The water surface profile in the floodplain is not modelled in this method. The advantage of this approach over the 1D is that the over-bank flow and the mass conservation in the floodplains are taking into account (Willems et al., 2002). This approach was applied in two separate InfoWorks RS models for the Aa and Visbeek rivers, of which the information has been used to combine two separate models in MIKE 11.

The last approach is the quasi or pseudo two-dimensional one for floodplain modelling using a one-dimensional hydrodynamic model. In this latter case, the floodplains are modelled as a network of river branches and spills (figure 3.2c). In MIKE 11 (DHI, 2008) the floodplains are connected to the main river by lateral spills mimicked by link channels and the discharge through these spills are calculated using a weir equation (Timbe, 2007).

3.2 Existing model: Infoworks RS

InfoWorks RS (IW-RS) is a hydrodynamic software model which computes one-dimensional flow. Bringing together source data and hydraulic modelling into a unique product, IW-RS combines the advanced ISIS flow simulation engine, GIS functionality and database storage within a single environment. Data can be brought together from a wide range of sources within the software in a rapid and flexible way. The software includes full solution modelling of open channels, floodplains, embankments and hydraulic structures. The input of the IW-RS river model, rainfall-runoff, can be simulated using both event based and conceptual hydrological methods. The lumped, conceptual model for continuous rainfall-runoff simulation developed and maintained by the UK Centre for Ecology and Hydrology is known as PDM (Probability Distributed Model) (Wallingford, 2009).

The detailed InfoWorks RS models two rivers: the Aa and the Visbeek (figure 3.3 (a) & (b)). The main tributaries, the Liermansloop of the Aa and the Zijtak and Rondveloop of the Visbeek, are also included in the IW-RS model. The PDM hydrological model that generates rainfall-runoff for the river model is calibrated and validated by the Province of Antwerp using the observed flow from the Turnhout station on the Aa river. The input data for the PDM hydrological model are the areal rainfall and daily evapotranspiration. The areal rainfall was measured during the measurement campaign while the daily evapotranspiration were derived from the Uccle station generated by the Royal Meteorological Institute of Belgium (Province of Antwerp, 2004 & 2006). The rainfall-runoff discharges generated by the PDM model are used as input at the boundary nodes in the hydrodynamic river model (figure 3.3 (a) and (b)).



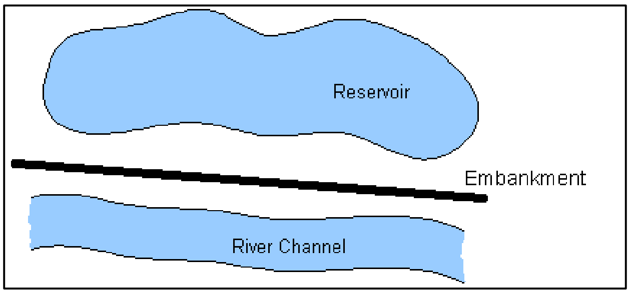
(a) (b)

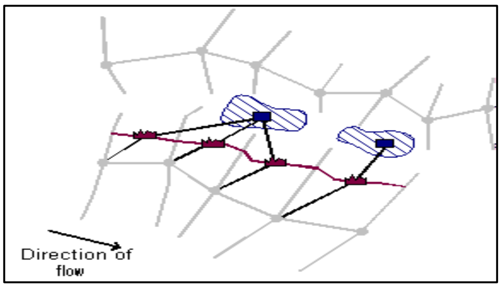
Figure 3.3: infoworks RS model for (a) the Aa and the Visbeek river (adapted from Province of Antwerp, 2004 & 2006).

The computation of flow depths and discharges at each river node by the IW-RS hydrodynamic river model is based on the equations for shallow water waves in open channels, i.e. the Saint-Venant equations (Wallingford, 2009).

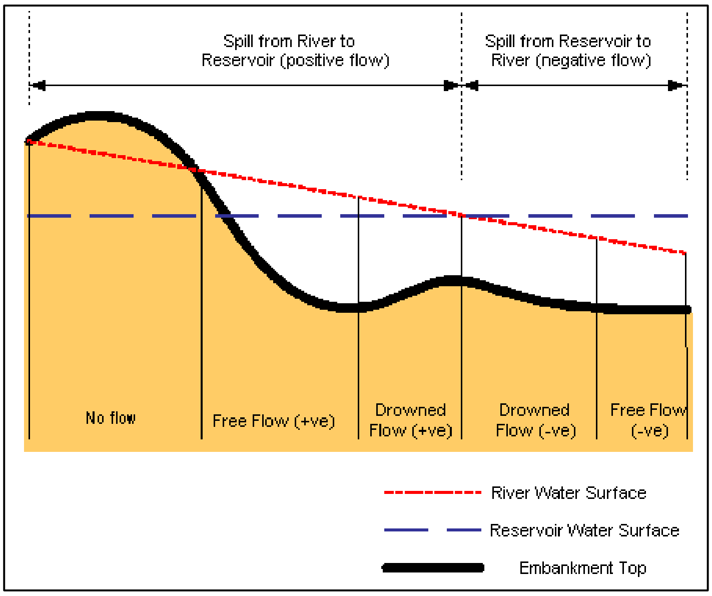
The floodplain modelling in the IW-RS model was performed by incorporating the floodplains as storage areas. For each storage area, an elevation-volume relationship was derived using the topographical data from a Digital Elevation Model (DEM). Floodplains included in the IW-RS model were 64 in total.

The spill units connect the floodplains to the main river where the spill units model the flow over the embankment existing between the river and the flooded area. The Spill calculates flow over an irregular weir by splitting the calculation into determining flows over segments using an integrated form of the weir equation for dry, free and drowned, forward and reverse modes. The sum of these flows then gives the values over the entire spill. Based on the water level in the river and in the floodplain, five different modes are defined. Figure 3.4 (a) shows the overhead view of a common spill arrangement, figure 3.4 (b) shows the connection of spills between river and storage area and figure 3.4 (c) represents different modes. The relevant equations can be found in the manual of the InfoWorks RS software (Wallingford, 2009).





(a) (b)



(c)

Figure 3.4: common spill arrangement (a) overhead view, (b) connection of spills between river and storage area and (c) side on view (source: Wallingford, 2009).

Simulation results from the existing model, developed by the Province of Antwerp, are then used to calibrate and validate the MIKE 11 river flood model.

3.3 MIKE 11 hydrodynamic model

The MIKE 11 hydrodynamic module is a modelling system for the one dimensional hydrodynamic computation of unsteady flows in rivers by solving the basic hydrodynamic equations in a implicit finite difference scheme (DHI, 1993). The system has been used in numerous engineering studies all over the world. MIKE 11 HD applied with hydrodynamic wave description solves the vertical integrated equations of conservation of continuity and momentum (the Saint Venant equations), based on the following assumptions:

- the water is incompressible and homogeneous, i.e. negligible variation in density;
- the bottom slope is small, thus the cosine of the angle it makes with the horizontal may be taken as 1;
- the wave length is large compared to the water depth. This ensures that the flow everywhere can be regulated as having a direction parallel to the bottom, i.e. vertical acceleration can be neglected and the hydrostatic pressure variation along the vertical can be assumed;
- the flow is sub-critical (super-critical flow is modelled in MIKE 11, using more restrictive conditions, however).

The basic equations can be formulated by using the principles of conservation of mass and momentum within a control volume.

- $\frac{\delta Q}{\delta x} + \frac{\delta A}{\delta t} = q$ conservation of mass Eq.3.1
- $\frac{\delta Q}{\delta t} + \frac{\delta\left[\alpha \cdot \frac{Q^2}{A}\right]}{\delta x} + g \cdot A \cdot \frac{\delta h}{\delta x} + n^2 \cdot \frac{g \cdot Q \cdot |Q|}{A \cdot R^{\frac{4}{3}}} = 0$ conservation of momentum Eq.3.2

where:

- Q: discharge [m3/s];
- A: flow area [m2];
- Q: lateral inflow [m2/s];
- H: stage above datum [m];
- n: Manning resistance coefficient [m1/2/s];
- R: hydraulic or resistance radius [m];
- α momentum distribution coefficient.

The Saint Venant equations are solved in finite difference form. This means that the “derivatives” in the equations are approximated by finite differences (differences in water levels or discharges at two succeeding locations). The locations in the river at which the water level or discharge are calculated are called “nodes”. In MIKE 11, the Q-nodes (the locations at which the discharge are calculated) and the h-nodes (the locations at which water levels are calculated) are alternating.

Computational grid

A computational grid with alternating Q (discharge) and h (water level) points is applied for the transformation of the Saint Venant equations to a set of implicit finite difference equations. The automatic generation of Q points between two h points is performed by placing h points at the location of cross sections. The combined system of equations is solved according to a 6-point Abbott-scheme (DHI, 2008a). The accompanying equations are available in ‘MIKE 11 Reference Manual, 2008’.

Cross section

The topographical description of the area to be modelled is achieved through the specification of cross sections for the model branches. They have to be considered approximately perpendicular to the direction of flow.

Cross sections are specified by a number of x-z co-ordinates where x is the transverse distance from a fixed point (often left bank top) and z is the corresponding bed elevation. The x-z coordinates are entered as raw data. The raw data are then automatically processed into a form used in the hydrodynamic calculations, i.e. the hydraulic parameters, cross-sectional area, hydraulic radius and width are calculated for a number of elevations between a minimum and maximum level. A relative resistance can be also specified for each pair of co-ordinated.

Hydraulic structure

MIKE 11 includes descriptions for a wide range of structures, which act as a control point. The structures are modelled in MIKE 11 model at Q-points in the computational grid. Depending on the structure category, a relationship between the discharge and the upstream and downstream water level is determined based on the flow condition, entrance and exit losses, and a critical correction factor.

Contraction and expansion losses at the structure are considered in MIKE 11 as loss coefficient. It is also necessary to describe a river cross-section immediately upstream and downstream of the structure since the loss formulations are based on the relationships between the available cross-sectional area at the structure and that of the immediately adjacent river.

Boundary conditions

External boundary conditions are required at all upstream and downstream ends of model branches. The relationship applied at these limits may consist of:

- constant values of h or Q;
- time varying values of h or Q;
- a relationship between h and Q (e.g. rating curve).

If the resolution of the time varying boundaries is greater than the time step used in the simulation, intermediate values in the boundary conditions are determined by linear interpolation.

Rainfall-runoff discharge

The catchments rainfall-runoff discharge, as modelled by NAM, are used as lateral inflow input to the hydrodynamic module. The runoff from each subcatchment discharges were uniformly distributed along a stretch of the two rivers by specifying the upstream and the downstream chainages and define the area of each subcatchment.

3.4 River flood model implementation

3.4.1 Mike 11 1D hydrodynamic model for Aa and Visbeek rivers

The combined MIKE 11, the one-dimensional hydrodynamic model, has been developed on the basis of two separate InfoWorks RS models optimized and built up for two rivers: the Aa and the Visbeek (Province of Antwerp, 2004 & 2006). Hereafter, a general description and schematization and implementation of the above mentioned MIKE 11 model are presented in the next sections.

3.4.1.1 Model domain

MIKE 11 HD model considers the main rivers Aa and Visbeek and their main tributaries (figure 3.5). Table 3.1 shows the watercourses included in the model domain with their corresponding length.

River	Tributary	Length (m)
Aa (category 2)		17220
	Liermansloop (category 2)	3535
Aa (category 1)		1545
Visbeek (category 2)		10877
	Rondvenloop (category 3)	517
	Zijtak Visbeek (category 2)	570

Table 3.1: the modelled rivers and their tributaries.

3.4.1.2 Model schematization

The river network comprising the main rivers and their tributaries is shown in figure 3.5 according to the river centerlines extracted from two IW-RS models separately developed for two main rivers in the area (Province of Antwerp, 2004 & 2006).

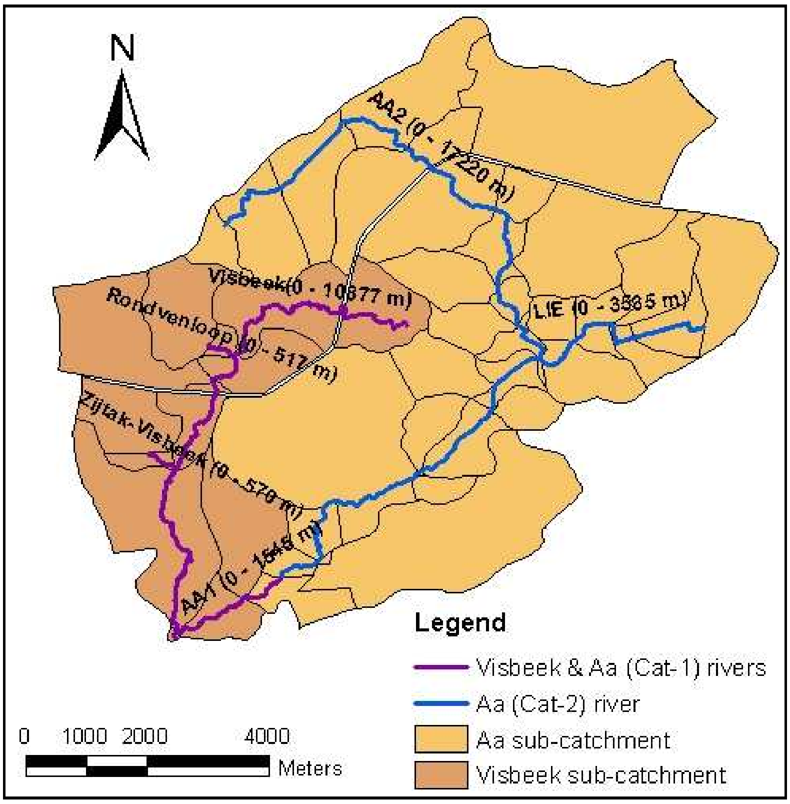


Figure 3.5: model set up for the river network.

Measured discharges are available at Turnhout station on the Aa river (chainage AA2: 17220 m). Initially these discharges are used to calibrate and validate NAM, the rainfall-runoff model, for the gauged and ungauged catchments which generates runoff discharges. The calibrated NAM runoff results are uniformly distributed along the river reaches in the MIKE 11 hydraulic model (table C.1, C.2 and C.3 in Appendix C) together with the upstream and downstream boundary conditions. The downstream boundary is located at the end of the Aa river of category 1 (chainage AA1: 1545 m). The water levels used in the simulation of IW-RS models have been considered as the downstream boundary condition according to Province of Antwerp, 2004 & 2006. The urban runoffs from the city of Turnhout have been integrated into the MIKE 11 river flood model as point source boundaries (table C.4 in Appendix C).

3.4.1.3 River cross-sections and hydraulic structures

The entire cross sectional data were derived from the IW-RS models developed separately for two rivers. These RAW data describing the physical shape of a particular cross section are then entered in the MIKE 11 model using the x and z coordinates in the river cross section editor. Each hydraulic structure (culvert, bridge and weirs) is accompanied with a cross section both at the upstream and at the downstream point.

In total 103 hydraulic structures have been implemented in the river network including weirs, culverts and bridges. Table 3.2 summarizes the distribution of cross sections and hydraulic structures along the river branches.

Branch	Cross section	Culverts	Bridge	Weirs
Aa (category 2): AA2	380	35	1	-
Aa (category 1): AA1	38	2	-	-
Visbeek (category 2)	281	45	-	-
Liermansloop (category 2): LIE	91	10	-	2
Rondvenloop (category 3)	15	-	-	-
Zijtak Visbeek (category 2)	21	8	-	-

Table 3.2: distribution of cross sections and hydraulic structures along the river branches.

These hydraulic structures are always placed at Q points where the momentum equation is normally solved. The reason for including a structure in a model is always to replace the momentum equation with something more suitable for the structure under consideration (DHI, 2008b). Figure 3.6 shows the location of all hydraulic structures along the river branches in the model domain.

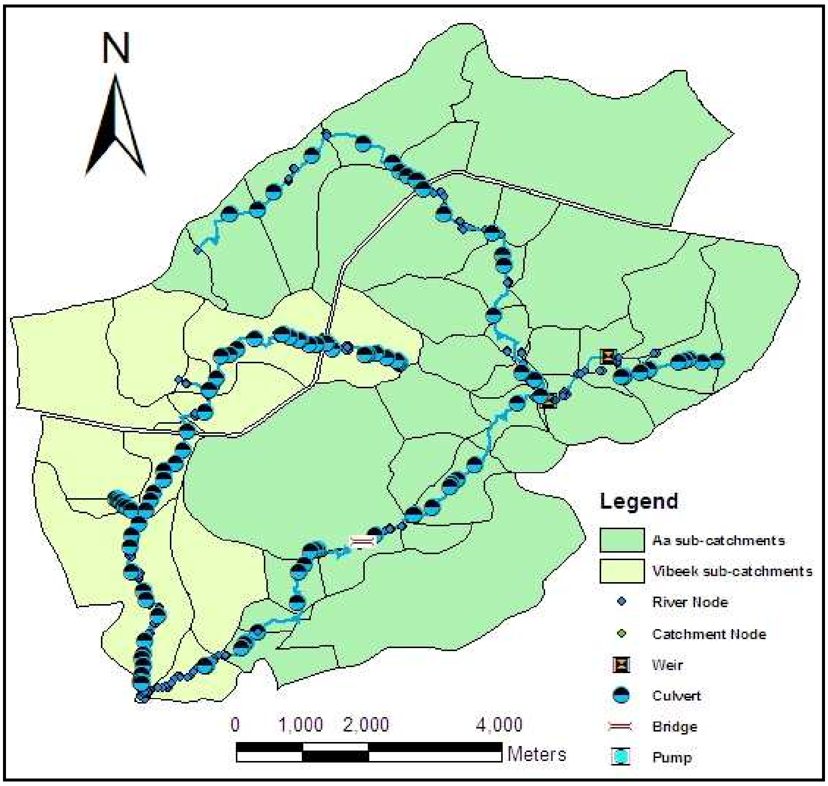


Figure 3.6: overlay of the MIKE 11 HD model domain on the Aa and Visbeek river system.

Two types of culverts, circular and rectangular, have been implemented in the model network specifying diameter for the first type while depth and width for the latter type. After specifying all the parameters for both types of culverts the Q-h relationships can be calculated.

The sprung arch bridges from the IW-RS model are defined by the rectangular culverts in MIKE 11 model where the irregular level-width table has been calculated from the spring height, crown height and invert level of the bridges following the basic equation of a parabola (figures C.1 and C.2 in Appendix C). While the US Bureau of Public Roads (USBPR) method is used for another bridge located on Aa (category-2).

In the first method, one of the pure free flow methods, an orifice type of equation is used to describe the discharge through the bridge. The equation is derived under the assumption of a rectangular channel and is based on a single span arch opening. Multiple arch openings are handled by a simple multiplication factor. In the second method, free-surface flow is estimated assuming normal depth conditions and this method can be combined with both submergence and overflow methods (DHI, 2008a). Two weirs on the Liermansloop are defined by the Weir Formula 2 (Honma) specifying weir coefficient (C1), weir width and weir crest level extracted from IW-RS model.

3.4.1.4 Manning's coefficients

The values of resistance numbers (Manning's *n*) should be determined through model calibration or based on other calibrated models with similar topographic characteristics. Initial estimated values can also be found in the literature. In general, a constant Manning's *n*, independent of the water depth, is applied to the river cross section in MIKE 11. However, it can also be defined as a function of the water depth (DHI, 2008).

3.4.1.5 Hydraulic data

The stage/discharge relationships (rating curve) using polynomial or exponential relations are available at the Turnhout station (ID: L10_064) (Province of Antwerp, 2004). In the detailed study by the Province of Antwerp, using IW-RS model, the water levels were derived from these rating curves and the same were used as the downstream boundary conditions for the simulation of the river flood model. Figure C.3 in Appendix C shows the rating curve at the Turnhout station on the Aa river.

3.4.2 Floodplain model implementation

3.4.2.1 Identification of flooded areas

The initial delineation of floodplains and storage areas in MIKE 11 river flood model has been done on the basis of the MIKE 11 (DHI, 2008) one-dimensional HD modelling system using all the information from the InfoWorks RS (Wallingford, 2009) models (Province of Antwerp, 2004 & 2006). The flood maps generated by IW-RS models is then used as a reference to make further modifications in defining floodbranches in order to identify the potential flood risk zones.

3.4.2.2 Floodbranches

Topographical information required to define cross sections for the MIKE 11 quasi-2D hydrodynamic model has been extracted from the IW-RS models developed separately for Aa and Visbeek rivers. In addition to that, the preferential flow direction of the floodbranches is derived from the DEM. A set of 64 storage areas have been implemented in the quasi-2D hydrodynamic model that covers all the floodplains on the basis of the IW-RS models. ArcGIS 9.2 has an extension MIKE 11 GIS which facilitates the processing of DEM and defining flow direction based on a raster operation. This flow direction of a single floodbranch, located at the deepest portion of the storage area, is derived for each storage area. After identification of the preferential flow direction, the positions of the cross sections along floodbranches are obtained from the DEM along lines perpendicular to the flow direction. A floodbranch of a short length (10-30 m) has been chosen with two cross sections at distance of 15 to 25 m for each storage area in MIKE 11 river flood model.

Two identical imaginary cross sections of trapezoidal shape (two example are shown in figure C.4 and C.5 in Appendix C) are generated for each storage area, based on the DEM taking into account the maximum extension of the cross sections is limited to the boundary of the storage area. The cross product of two cross sectional areas and distance between them should share a part of the storage volume in the respective storage area. Then the additional storage volume is computed on the basis of the information from the IW-RS models. A storage-elevation relationship is computed from the DEM and the additional flooded area is included in the cross sectional data as an additional storage area in the processed cross section data editor.

Finally, the real storage volume in the floodplains must be represented by these storage areas. Figure 3.7 shows the overlay of the DEM with storage areas along the rivers in both the sub-catchments according to the information and the flood maps obtained from the simulation of two IW-RS models developed separately for two rivers. In figure C.6, C.7 and C.8 are presented three examples of the floodbranch implementation and the link channels connecting to the main river.

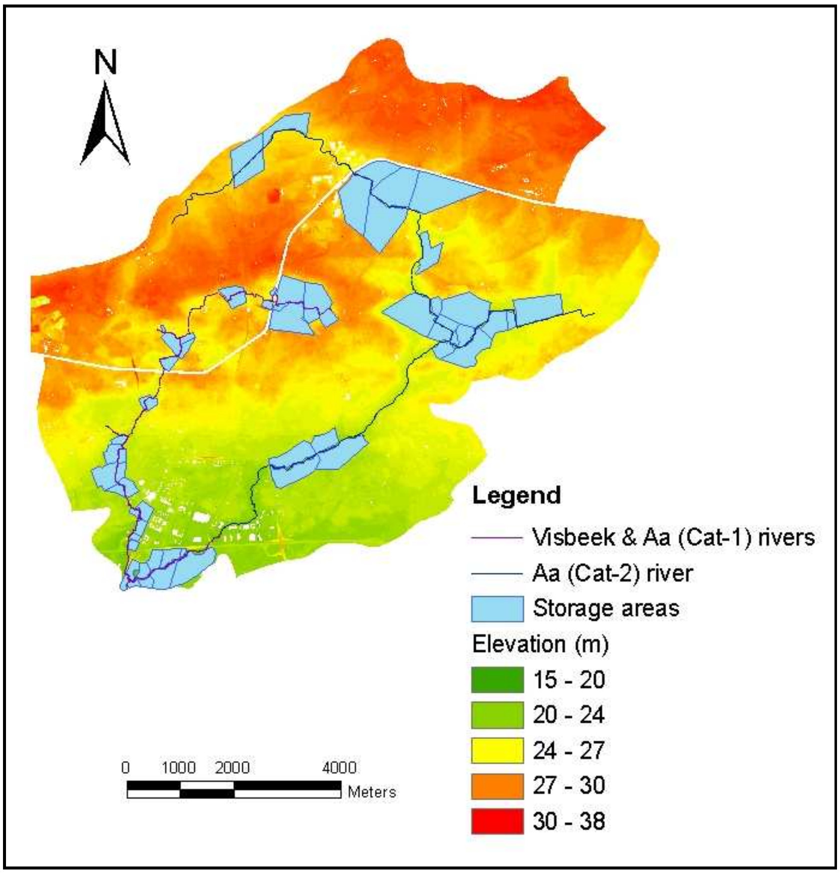


Figure 3.7: overlay of the storage areas along the rivers.

3.4.2.3 Lateral spills

In the quasi-2D approach of MIKE 11 river flood modelling a lateral spill is incorporated to allow water flow from the main river into the floodplain and vice-versa (figure 3.8). In MIKE 11 the lateral spills are represented by link channels in the river network. Link channels are a special and simpler type of river branches where cross sections are not required to be specified. A link channel is implemented as a short branch which connects the floodplain to the main river branch.

The link channels are modelled as a single weir branch in MIKE 11 using the longitudinal geometry along the embankments as main input. The cross section geometry of a link channel is defined as a depth-width relationship derived from the geometry of the embankments (DHI, 2008). Other parameters required are bed resistance along the link channel using Manning's *M* or n (an average Manning's *n*=0.05 was used in this study) and head loss coefficients for inflow and outflow (default values of 0.5 and 1 were used respectively). With this input data *Q/h* relationships are computed as the calculated flow area, hydraulic radius and conveyance etc are presented in figure 3.9.

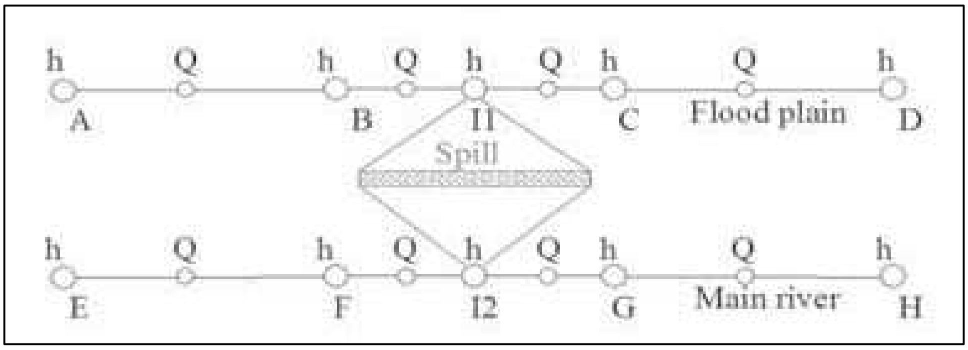


Figure 3.8: schematic representation of the lateral spill (adapted from Villazòn & Willems, 2009).

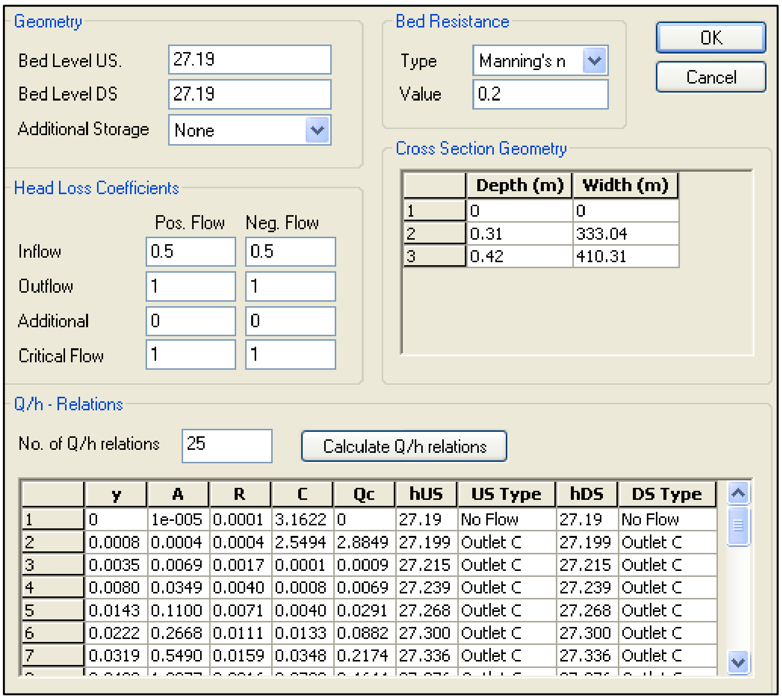


Figure 3.9: the link channel property sheet (Link Channel: LC-AA-L1).

The spill level of the lateral spills can be derived from the DEM or also more accurately from the cross section survey. The accuracy of the dike crest levels or embankment levels is very important for the flood model as it determines the return period of flooding and other flooding parameters (Timbe, 2007). Embankment levels based on the cross section survey are taken as they describe the crest level more accurately. A set of 64 link channels, one designated for each floodbranch, has been defined in the model domain and their distribution in the floodplain model is presented in the model configuration.

3.5 Calibration and validation of the river flood model

For the calibration and validation of MIKE 11 river flood model, simulation results of the existing physically-based, fully dynamic, two separate one-dimensional InfoWorks RS (IW-RS) (Wallingford, 2009) models for two sub-catchments, developed by the Province of Antwerp, were taken into consideration. The simulation results of the IW-RS model for the storm event of December 1960 have been used to calibrate the MIKE 11 river flood model because this was the only data available at the beginning of the study, while the results of September 1998 have been used as an independent dataset for the validation of the calibrated MIKE 11 river flood model combined for the two rivers. The water levels and discharges downstream of some selected structures along the rivers and water levels in the floodplains are used to calibrate and validate the model results. Following this procedure, Manning's *n* coefficients are calibrated along the river channels. In addition to that, the maximum extent of inundation, as observed in the flood maps, is compared with the flood maps generated from IW-RS models separately for two rivers. Trial-and-error calibration method has been applied to find out optimal parameter values involving a number of simulations to be performed.

3.5.1 Calibration Manning's coefficient

The Manning's *n* coefficient has been calibrated based on water levels at different locations along the river (different cross-sections). Finally a global value of 0.035 $s/m^{1/3}$ producing good match between MIKE 11 and IW-RS results has been considered as the calibrated Manning's *n* for all the river reaches. However, different resistance factors are used for the upper and lower parts of the river cross-sections in order to have higher roughness in the upper part than in the corresponding lower part.

3.5.2 Calibration water level downstream of structures

The calibration of water level was done considering locations downstream of different structures and changing the Manning coefficient in order to have a good match between the results of MIKE

11 and IW-RS model. Figure 3.10 shows comparison of water levels between MIKE 11 and InfoWorks RS models downstream of different structures for the period from 02/12/1960 04:00:00 to 17/12/1960 09:00:00. From these results it appears that there is a good match between the water levels being compared with some systematic under- and overestimations of some structures, which can be overcome with further calibration efforts.

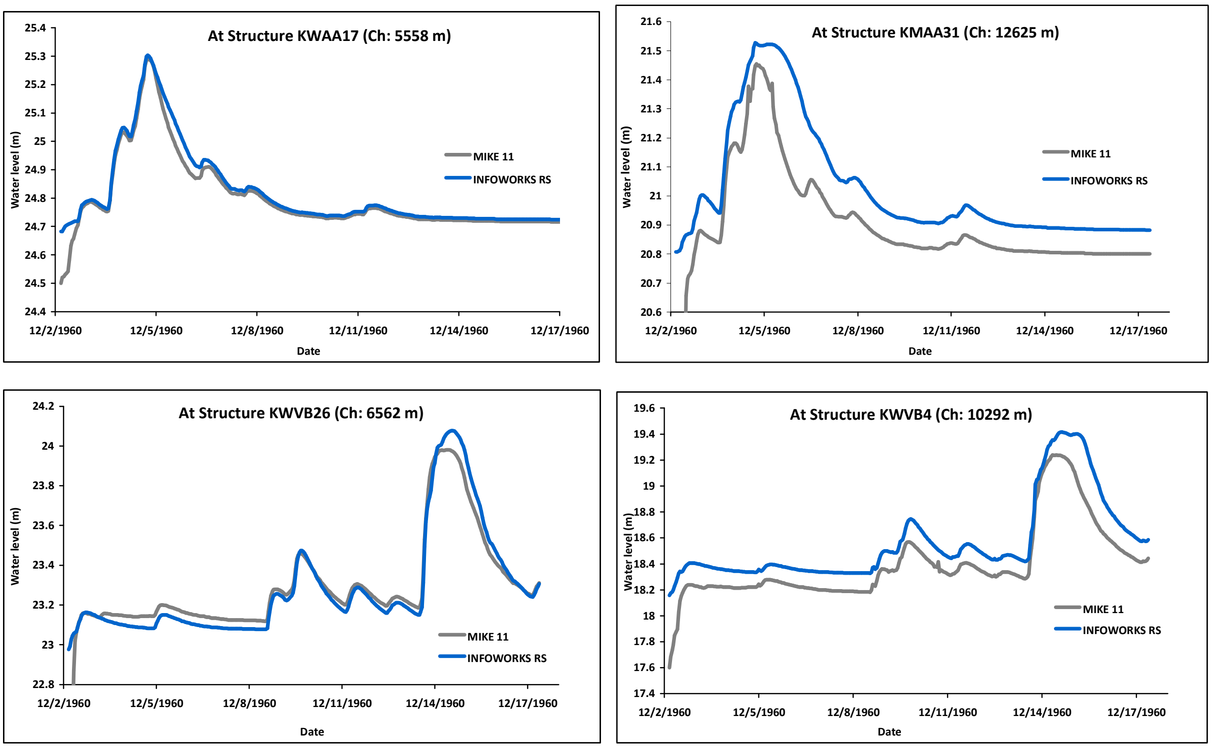


Figure 3.10: comparison of water level between MIKE 11 and InfoWorks RS models at different structures along the rivers for the period from 02/12/1960 4:00 AM to 17/12/1960 9:00 AM.

3.5.3 Calibration of the discharges through structures

Figure 3.11 reports the comparison of discharges between MIKE 11 and InfoWorks RS models at the downstream of different structures for December 1960 (from 02/12/1960 04:00:00 to 17/12/1960 09:00:00) flood event. From these results it is clear that there is a good match between the discharges being compared with some discrepancies in the peak which can be improved by further calibration efforts (setting different Manning's *n* for different river reaches).

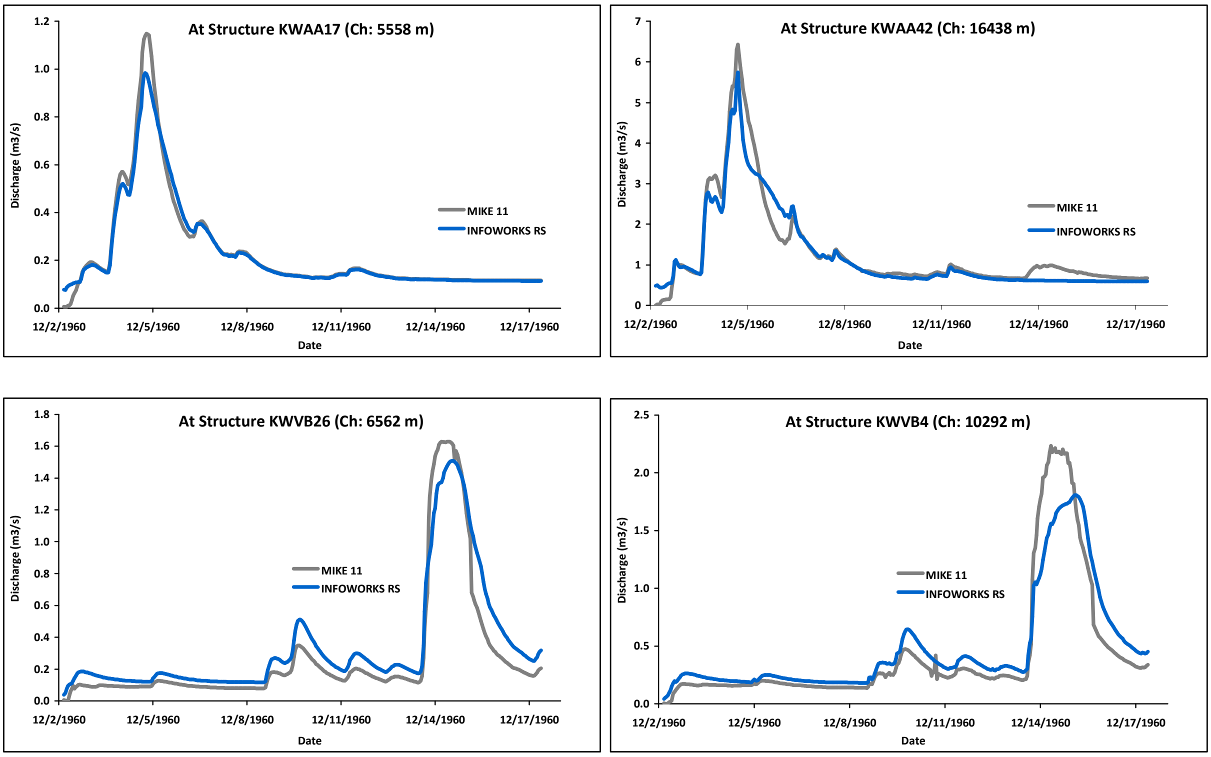


Figure 3.11: comparison of discharge between MIKE 11 and InfoWorks RS models at different structures along the rivers for the period from 02/12/1960 4:00 AM to 17/12/1960 9:00 AM.

3.5.4 Calibrated water levels in the floodplains

The Manning's *n* for the floodplains has been calibrated by achieving a good agreement between the maximum and minimum water levels in the floodplains during the flooding conditions. The simulated average maximum water level in the floodplains from MIKE 11 is 22.80 m as compared to average value of 23.05 m from IW-RS with an average variation in maximum water levels by 1.16% and by 0.98% in minimum water levels.

The flood maps generated after calibration of MIKE 11 river flood model for Manning's *n* show good implementation of the floodbranches, although some modifications could yield better results.

3.5.5 Model validation

The model validation was performed using the hourly time series of water levels in the floodplains for the period 1 September 1998 to 30 September 1998. The simulated average maximum water level in the floodplains from MIKE 11 is 22.54 m as compared to 22.88 m from

IW-RS with an average variation in the maximum water levels by 1.46% and by 0.02% in the minimum water levels. The model performance is also satisfactory for water levels in all the floodplains with an R value of 0.99 and a NSE value of 0.96 for the validation period.

Later the simulated rainfall-runoff from NAM was used to validate the MIKE 11 model for the period from the 1st September 1998 to the 30th November 1998. The simulated average maximum water level in the floodplains from MIKE 11 is 22.71 m as compared to 22.89 m from IW-RS with an average variation in the maximum water levels by 0.71% and by 2.47% in the minimum water levels. The model performance is also satisfactory for water levels in all the floodplains with an R value of 0.99 and a NSE value of 0.97 for the validation period with NAM results.

Figure 3.12 presents the simulated discharge by MIKE 11 and observed discharge at the Turnhout station for the validation period.

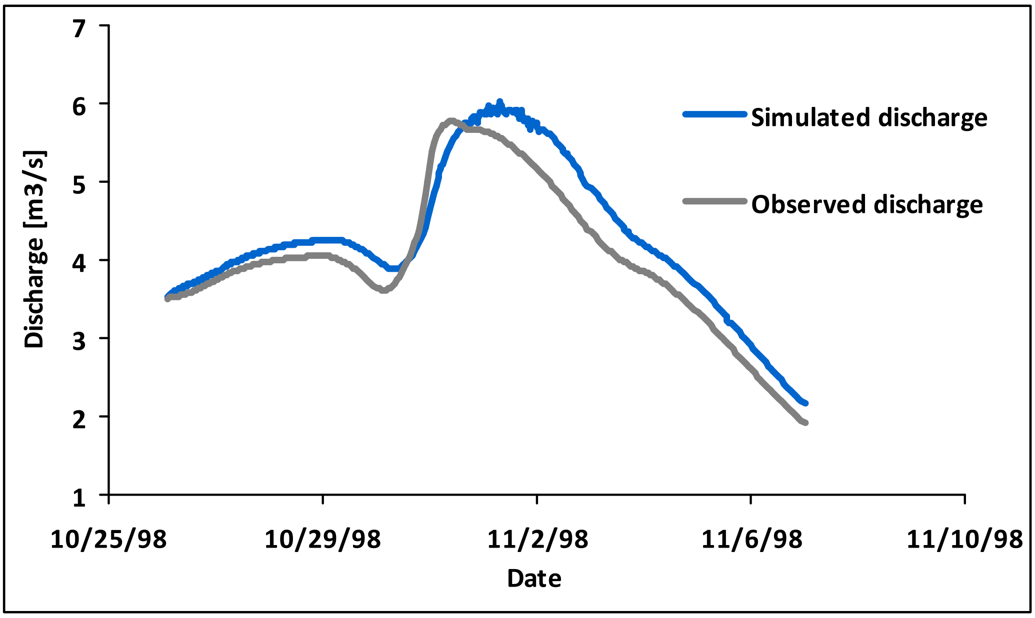


Figure 3.12: simulated discharge by MIKE 11 and observed discharge at the Turnhout station for the period from 1 September 1998 to 30 November 1998.

3.6 Model performance evaluation

Model performance has been evaluated for water levels downstream of the selected structures and water levels in the floodplains. For the comparison of the MIKE 11 model results with IW-RS model results, the Linear Correlation Coefficient (R) and the Nash-Sutcliffe coefficient (NSE) (Nash and Sutcliffe, 1970) were used.

$$R = \frac{\sum_{i=1}^{n}\left(O_i - \overline{O}\right)\cdot\left(S_i - \overline{S}\right)}{\sqrt{\sum_{i=1}^{n}\left(O_i - \overline{O}\right)^2}\cdot\sqrt{\sum_{i=1}^{n}\left(S_i - \overline{S}\right)^2}} \qquad \text{Eq. 3.3}$$

$$NSE = 1 - \frac{\sum_{i=1}^{n}\left(O_i - S_i\right)^2}{\sum_{i=1}^{n}\left(O_i - \overline{O}\right)^2} \qquad \text{Eq. 3.4}$$

where:

- O_i is the i-th observed value within the time series;
- O_a is the average of the observed time series values;
- S_i is the i-th simulated value within the time series;
- S_a is the average of the simulated time series values.

Table 3.3 reports the list of all the statistical parameters for the water levels downstream of different structures and also in all the floodplains.

Location	R	NSE
Structure KWAA17 (Ch: 5558 m)	0.98	0.95
Structure KMAA31 (Ch: 12625 m)	0.91	0.42
Structure KWVB26 (Ch: 6562 m)	0.96	0.92
Structure KWVB4 (Ch: 10292 m)	0.97	0.68
Average at the structures	0.96	0.74
In all the floodplains	0.93	0.83

Table 3.3: model performance evaluation of water levels downstream of different structures and water levels in the floodplains for the period from 02/12/1960 at 4:00 AM to 17/12/1960 at 9:00 AM.

In general, the model has a satisfactory performance for all the hydraulic structures and for all the floodplains with very high values of R ranging from 0.91 to 0.98. The model performance is also quite satisfactory with respect to the NSE values ranging from 0.42 to 0.95 with average value of 0.74 for the hydraulic structures selected.

3.7 Flood mapping

MIKE 11 (DHI, 2008) produces 2D flood maps based on 1D simulation. The maps are constructed through interpolation in space of the grid point results. Thus, such maps should be viewed as 2D interpretations of results from a 1D model (DHI, 2008). Construction of grid flood maps is based on triangular interpolation routines which make use of the water levels stored at the most extreme points of consecutive cross sections. This option enables flood map generation based only on the cross sectional survey. When a suitable DEM is available it can be used to improve mapping performance by introducing DEM values in between adjacent cross sections, in this way, linear interpolation between consecutive cross sections is improved when DEM elevations are accounted. 2D flood maps generated by the latter procedure are restricted only to areas bounded by the cross sectional width, thus, over estimations resulting from point-grid-based extrapolation routines, as those observed in former versions i.e. MIKE 11 GIS (DHI, 2005), are now restricted to the modelled area.

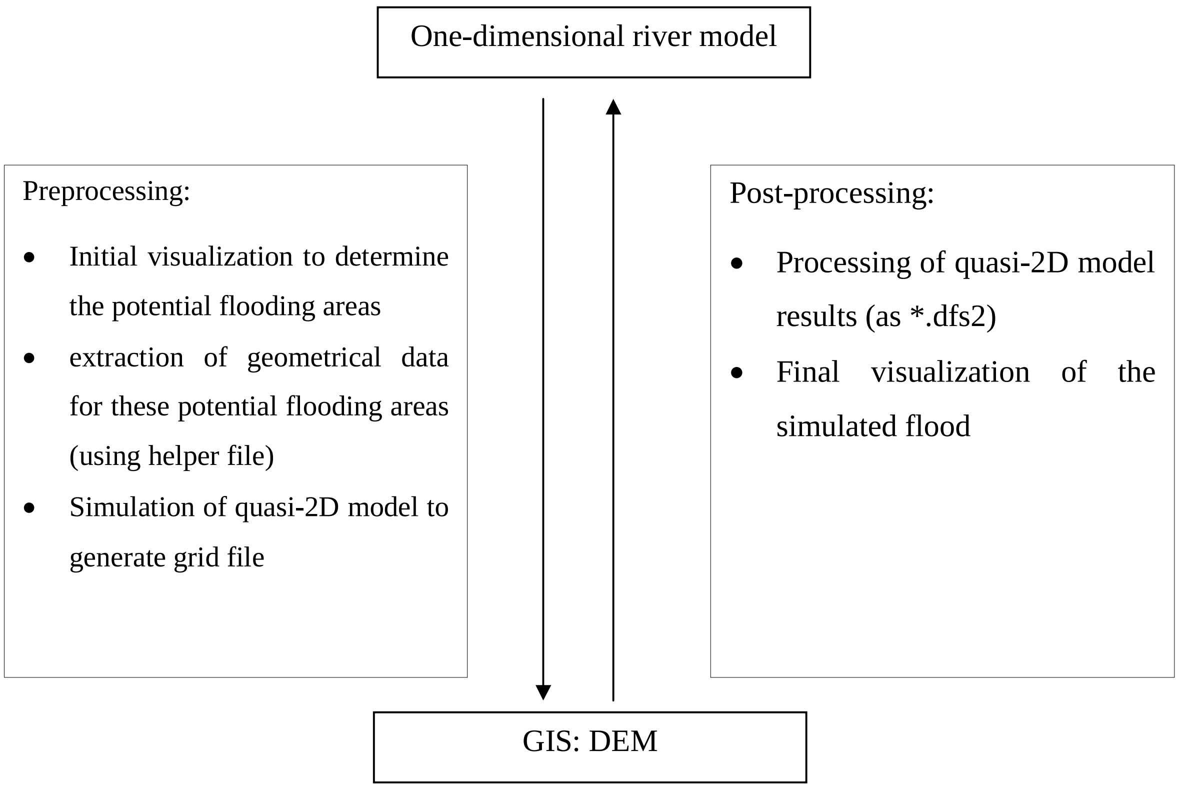


Figure 3.13: schematic representation of MIKE 11 GIS fro river flood modeling (adapted from Willems, 2009).

However, estimation of additional flooded areas is possible by user-defined options which enable to control the generation of flood maps on areas which are supposed to be inundated. In this procedure the interpolation routine is not a triangular operation as described previously, but it makes use of former interpolation routines as the distance weighted interpolation of MIKE 11 GIS (DHI, 2005), where the flooded-grid is constructed through extrapolation starting from a known z-value and comparing it with the surrounding topographic information (DHI, 2005). It is clear that this flood map technique is only a self grid construction, where, a user defined polygon (shape) assumes the water level from a preselected cross section (h-point) in order to be operated with the topographic grid and produce new-flooded grid-cells. This routine is an easy way to visualize the flood extent especially in flood prone areas, despite its versatility; its application implies two requirements: the DEM has to incorporate both land depression (ponds, natural storage elements, etc) and important elevations. On the other hand the modeller has to know the areas where this mapping approach can be applied. Otherwise overestimation in the resulting maps is highly probable.

3.7.1 Integration of GIS with flood modelling

GIS provides a consistent visualization environment for displaying model data and results. On the other hand, complex models take the advantage of GIS to overcome their limitation of data management and preparation; and concentrate only on the modelling effort (Karimi and Houston, 1996 cited in Timbe, 2007). The integration of the model with GIS means representation of both the spatial variation and temporal change of the processes and variables modelled (Timbe, 2007). In a broad sense integration can be implemented for preprocessing data into a specific format for analysis, support for modelling directly within the GIS package, and as a post-processing tool for reformatting, reporting and mapping (Zerger and Wealands, 2004 cited in Timbe, 2007).

The integration between the model and GIS is carried out via data exchange using either ASCII or binary data format. MIKE 11 GIS (DHI, 2008) is implemented in ArcGIS 9.2 as they have a common interface to link model results and the GIS software using the specific data format. Initial visualization to determine the potential flooding areas and extraction of geometrical data for these potential flooding areas are performed in GIS with the creation of a helper file. In MIKE 11 the flood maps are determined by the interpolation of the water levels with the DEM for specific time moments as selected. MIKE 11 GIS uses gridded elevation data for the analysis and the flood maps are derived in grid format which allows the calculation of the area and volume flooded in ArcGIS 9.2.

3.7.2 Historical flood maps

Historical flood maps, so-called ROG maps (Recent Overstroomde Gebieden – maps of recent floods, according to the Dutch abbreviation), present maximal spatial extent of all the observed floods in Flanders since 1988. The flood maps are based on different sources of information available at regional and local water authorities, such as field reports, aerial photographs and videos, television footage etc. The ROG maps shows some inaccuracies in their delineation and sometimes not all the inundated areas are included (Timbe, 2007). Some remarks regarding the quality of the ROG maps include (MGV, 2003 cited in Timbe, 2007):

- the ROG maps of floods from the most recent years have more trusted data;
- the ROG maps present the zones that were really inundated;
- the ROG maps should not be used to calculate return periods, because of the limited period of data (1988-2003);
- they should not correspond to the maximum moment of inundation, but for some maps on the time moment of aerial photos and videos taken;
- some of the floods could be caused by failure of infrastructure (dikes).

The flood maps were generated both for the Aa and Visbeek rivers from the simulation of IW-RS river flood models during the detailed study (Province of Antwerp, 2004 & 2006). These maps have been compared with the ROG maps and many disparities are identified. As such, floodplain modelling using MIKE 11 is based on the generated flood maps after observing the topography from the DEM. DEM shows the local topography which is consistent with the flood maps generated from IW-RS models.

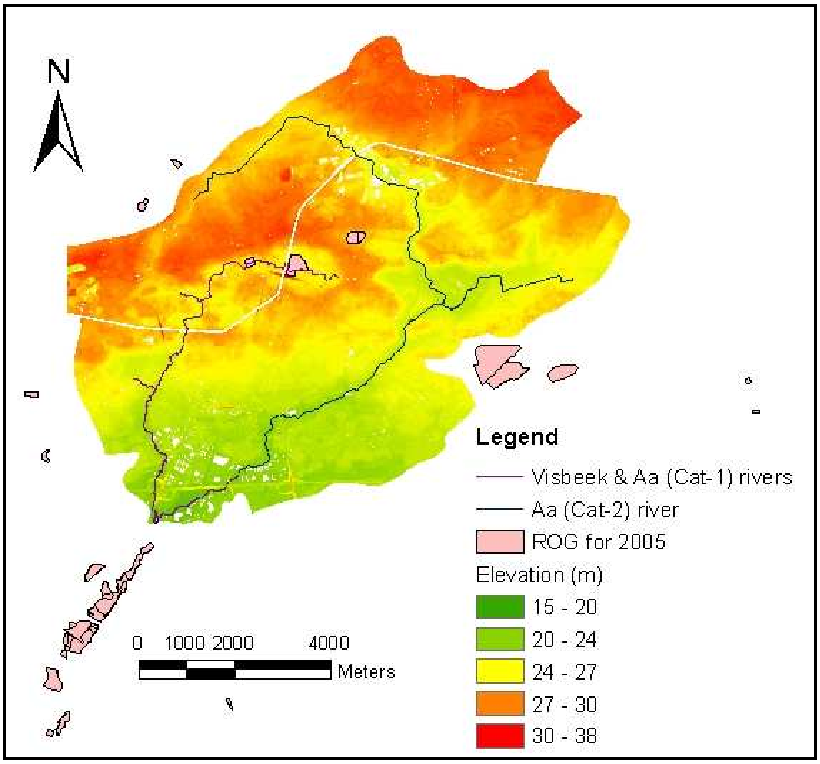


Figure 3.14: overlay of historical flood map ROG for 2005 on DEM for the study area and surroundings.

3.8 Simulation results

Simulations of combined Visbeek and Aa rivers modelling were performed. In the calculation, an approximately distance between computational grid points at least every 30 m is used and hourly averaged rainfall-runoff input discharge were considered. The stability conditions of the model was guaranteed by the ratio between the time step and the grid spacing (Courant number).

3.8.1 Historical events

Historical events of 1960, 1998 and 2000 were selected as input for the MIKE 11 simulations and the generation of flood maps. One of these simulations is shown in this section. Figure 3.15 shows result given by MIKE 11 model for the period from 2/12/1960 to 17/12/1960 while the figure 3.16 shows the result given by the existing model for the same period.

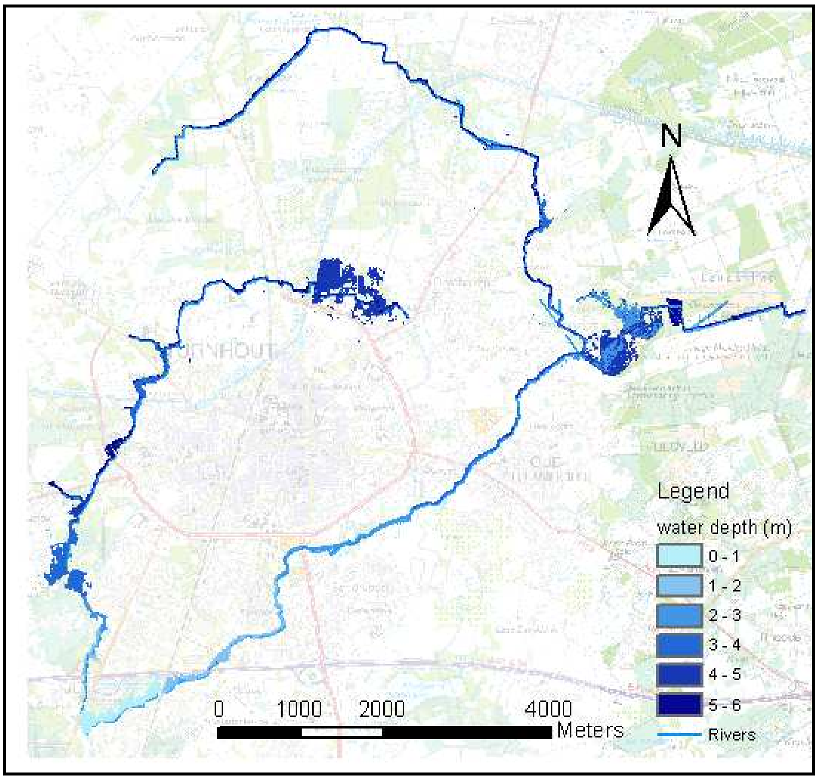


Figure 3.15: MIKE 11 simulation results for the period from 2/12/1960 to 17/12/1960.

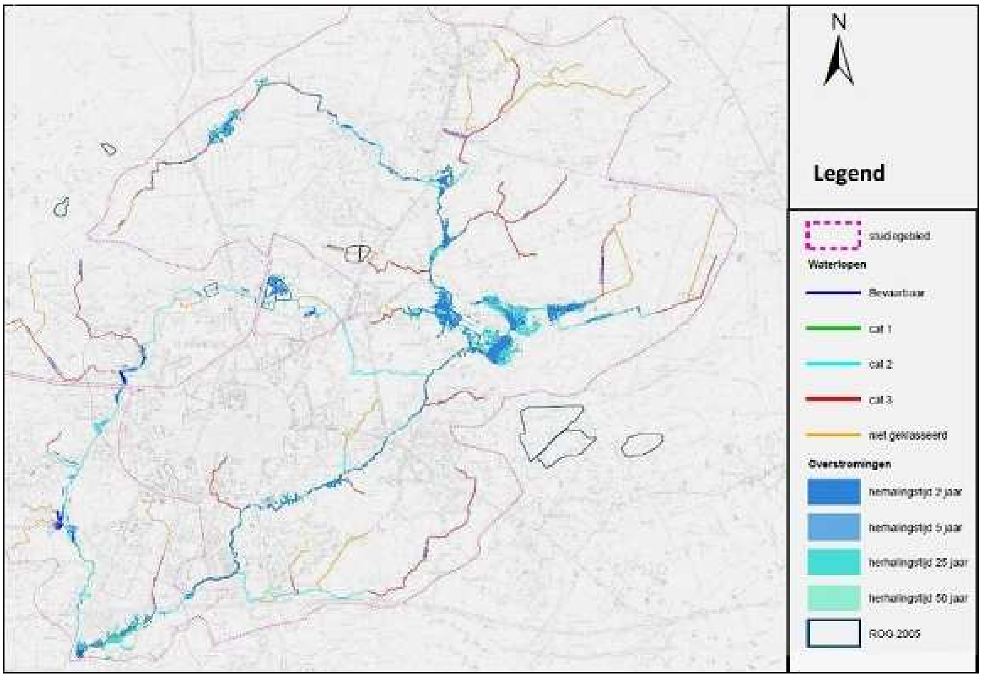


Figure 3.16: Infoworks RS simulation results for the period from 2/12/1960 to 17/12/1960.

3.8.2 Synthetic events

Flood maps have been derived for return periods of 5, 10, 25, 50 and 100 years from the simulation of MIKE 11 GIS model using the composite hydrographs in the hydraulic model as lateral inflows and up and downstream boundary conditions. Figure 3.17 shows the flood map for the region near the confluence of Aa and Visbeek rivers with return periods of 5, 10, 25, 50 and 100 years.

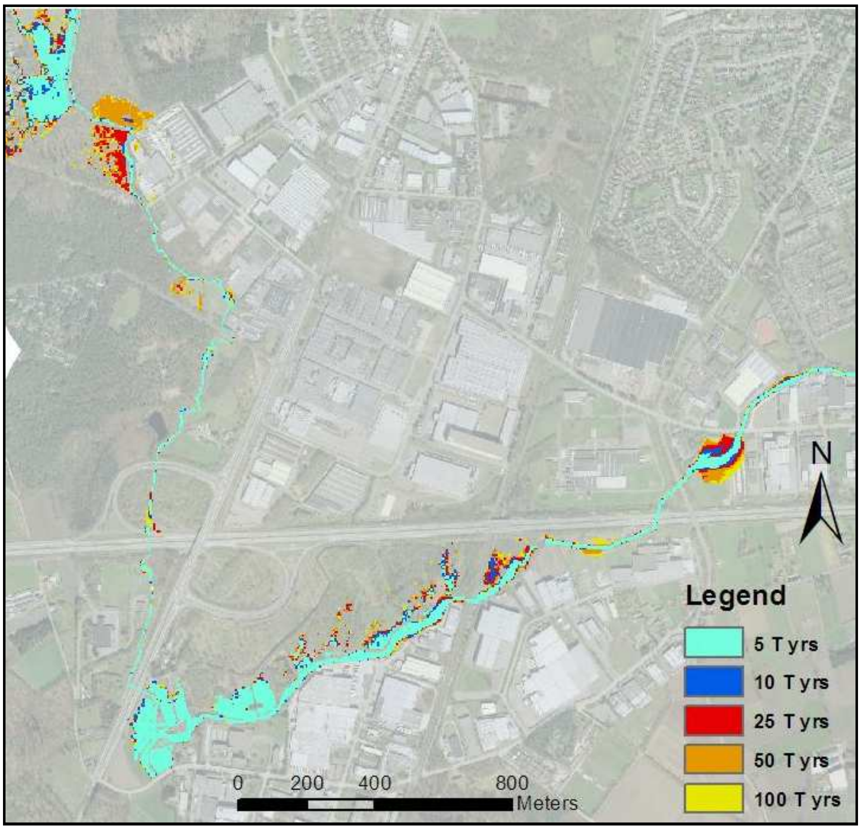


Figure 3.17: flood maps showing the present scenario without any influence near the confluence of Aa and Visbeek rivers for return periods of 5, 10, 25, 50 and 100 years.

This kind of flood maps illustrates how the increase in spatial extent of flooded areas with the increase in returns periods. The flood related variables such as water depth and inundation areas can be determined from these maps.

3.9 Conclusion

In this chapter, the hydrodynamic river model for Visbeek and Aa river was implemented based on the existing IW-RS model. In the model the implementation of the flood plain areas was included, which was based on the information and the flood maps obtained from the simulation of two IW-RS models developed separately for the two rivers. The flooding areas were implemented in a quasi 2D approach, where the floodplains were represented as a new branches, characterized by two fictitious cross-sections of trapezoidal shape for each flood plain and connected to one dimensional model of the main river by one link channel. The cross product of two cross sectional areas and distance between them is part of the total storage volume of the floodplain. The additional storage volume was then computed based on the h-V relation of each storage area and was included in the cross sectional data as an additional storage area in the processed cross section data editor.

The simulation results of the IW-RS model for the storm event of December 1960 have been used to calibrate the MIKE 11 river flood model while the results of September 1998 have been used as an independent dataset for the validation of the calibrated MIKE 11 river flood model combined for two rivers.

In the calibration the water levels and discharges downstream of some selected structures along the rivers and water levels in the floodplains were checked. Following this procedure, Manning's *n* coefficients are calibrated along the river channels. Trial-and-error calibration method has been applied to find out optimal parameter values involving a number of simulations to be performed.

The results have shown a good correlation between the two models. In particular, with the use of a simplified method for the implementation of the floodplain, the model was able to achieve good results in the water level of each storage element, as the model performance evaluation suggested. The results have also shown a satisfactory performance for all the hydraulic structures and for all the floodplains with a very high values of R ranging from 0.91 to 0.98. The model performance is also quite satisfactory with respect to the NSE values ranging from 0.42 to 0.95 with average value of 0.74 for the hydraulic structures selected.

Finally 2D flood maps were generated based on the model considering historical and synthetic events.

Chapter 4

Impacts of urbanization and climate change on flood risk

4.1 Introduction

The effect of urbanization, due to climate change, have adverse impacts of extreme flood events. Assessment of the potential impact of climate change on water system has been an essential part of hydrological research over the last couple of decades. An increase in surface temperature, sea level rise, changes in rainfall and decrease in northern hemisphere snow cover are expected in the future.

The impact of climate change on the hydrological cycle have the potential to affect not only the natural environment but also the human built environment. Towns are arguably the most dramatic examples of change as soil become covered by impervious surface and streams are replaced by pipes leading to rapid flow responses and high peak flows.

In this chapter the relative impacts of climate change and urbanization are assessed both separately and together for the Visbeek and Aa catchment. Of most of interest is how this two element influence the flood risk, particularly in the lower part of Aa river. The study has required the creation of both climate and urbanization scenarios.

The climate change scenarios were obtained using the perturbation algorithm developed in the CCI-HYDR project at Hydraulics Laboratory of K.U. Leuven for facilitating the climate change impact assessment in Belgium. A combined dynamical–statistical downscaling method based on perturbations method was used and the high scenario, where the risk associated with flooding is expected higher, was considered. Instead the urbanization scenario was built considering the existing correlation between the urban runoffs from the city and the rainfall.

4.2 Climate change development

Climate change is a complex problem involving interactions and feedbacks between atmosphere, oceans and land surface (Jiang et al., 2007). Hydrological models can be used as tools for climate change impact analysis on the basis of results from the climate models.

The link between climate change and hydrology resembles the interaction between the climate system and the hydrological cycle. Hence, the representation of climate system is accomplished by climate models while the illustration of the hydrological system is made by hydrological models. The difference of these models lies in their concept, natural processes included and resolutions. In order to link the common variables (temperature, precipitation etc) of the two models, an alternative method, downscaling, is made available (Ntegeka, 2006). This method is generally divided into two types, dynamic and statistical downscaling.

4.2.1 Combined downscaling approach (Frequency perturbation method)

This downscaling method combines the advantages of dynamic with statistical downscaling methods. In this approach, only differences in the most relevant climatic variables to hydrology, typically rainfall and evapotranspiration (ETo), are extracted from the control simulations (simulations of the past and present climate) and the scenario simulations (simulations of the future climate) of the climate model. Therefore the derived perturbation factors will be used to perturb input series of hydrological models and to make offline simulations to assess a response to the future climate conditions (Boukhris et al., 2008).

The combined dynamical–statistical downscaling method based on perturbations method was selected and used for a project called CCI-HYDR in Belgium. The method was applied by correcting few drawbacks of the original method and using the advantages of perturbation approach. The advantage of this approach is that it uses observed climate as a baseline, it is a stable method and always gives results that can be related to present conditions.

As stated in the CCI-HYDR project's report, the disadvantage of the perturbation approach which is related to application of the perturbation methods in a static way was tackled by using perturbation factors that depend on the time scale and the intensity level or return period. Specifically for rainfall variable the perturbation factor was separately derived for the number or frequency of rainfall events (i.e. storm events) and the mean intensity per event (Boukhris et al., 2008). Due to its advantage over the separate dynamic and statistical downscaling method the combined approach with the modifications mentioned was also applied in this research to find the future time series inputs for the hydrological model used.

4.2.2 Climate scenario

A climate scenario refers to a plausible future climate that has been constructed for explicit use in investigating the potential consequences of anthropogenic climate change. Such climate scenarios should represent future conditions that account for both human-induced climate change and natural climate variability (IPCC, 2001). A scenario is not a forecast. Rather, each scenario is an alternative image of how the future can unfold. A projection may serve as the raw material for a scenario, but scenarios often require additional information (e.g., about baseline conditions) (Carter, 2007). The studies on climate change impact assessment can be done by using different categories of climate scenarios. Scenarios based on different climate model outputs are most commonly used for the impact assessment studies.

The IPCC Special Report on Emissions Scenarios (SRES) team defined four narrative storylines where each of them represents different demographic, social, economic, technological, and environmental developments that diverge in increasingly irreversible ways. In simple terms, the four storylines combine two sets of divergent tendencies: one set varying between strong economic values and strong environmental values, the other set between increasing globalization and increasing regionalization. These four storylines are described as follows (Nakicenovic et al., 2000):

1. A1 storyline and scenario family: a future world of very rapid economic growth, global population that peaks in mid-century and declines thereafter, and rapid introduction of new and more efficient technologies;

2. A2 storyline and scenario family: a very heterogeneous world with continuously increasing global population and regionally oriented economic growth that is more fragmented and slower than in other storylines;

3. B1 storyline and scenario family: a convergent world with the same global population as in the A1 storyline but with rapid changes in economic structures toward a service and information economy, with reductions in materials intensity, and the introduction of clean and resource efficient technologies;

4. B2 storyline and scenario family: a world in which the emphasis is on local solutions to economic, social, and environmental sustainability, with continuously increasing population (lower than A2) and intermediate economic development.

For ease of interpretation in the CCI-HYDR Perturbation Tool, three scenarios are defined as high, mean and low based on the expected hydrological impacts. The definition is not dependent on the projected rainfalls alone, rather it is based on the combined effect of the rainfall and evapotranspiration. In other words, the variables are combined to generate an impact which can then be classified as high, mean and low. The high scenario projects a future with wet winters and dry summers while the low scenario projects a future with dry winters and dry summers. Thus, it is expected that the risk associated with flooding is higher in the high scenario than in the low scenario which is critical for low flows. In essence, the high, mean and low scenarios may be referred to as wet, mild and dry scenarios respectively. For the present study only the high scenarios are considered as flooding is associated with them.

4.2.3 Derivation of potential climate change scenario

The climate change scenarios were obtained using the perturbation algorithm developed in the CCI-HYDR project at Hydraulics Laboratory of K.U. Leuven for facilitating the climate change impact assessment in Belgium.

The algorithm imparts a perturbation to an observed series to generate future time series. In this tool the observed time series are perturbed on the basis of four SRES scenarios (A1, A2, B1 and B2) (Ntegeka and Willems, 2008). So, the perturbation of rainfall from the Turnhout station and evapotranspiration from the Uccle station are performed by using this CCI-HYDR Perturbation Tool to generate the future time series of the same for 2100s. These time series were then used as the input in the calibrated hydrological model NAM to generate the rainfall-runoff from the sub-catchments under future climate. The simulations results from NAM are statistically processed (extreme value analysis) and compared with the current values to assess the change in only the high peak values for the analysis of flood frequency towards the end. This comparison is performed as follows:

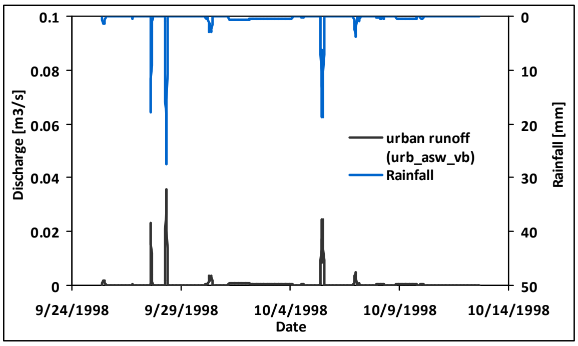
1. the current and scenario period results are ranked in descending order giving rank one to the highest value in the series;

2. the empirical return period for the ranked runoffs is calculated (using formula n/m) where n is the number of years for the time series and m is the rank number.

The time series of the rainfall-runoff under current and future climate conditions are then statistically analyzed to construct composite hydrographs with different return periods.

4.3 Urbanization development

The short time series of urban runoff for different storm events in both sub-catchments are made available from the InfoWorks-CS model for the simulation of InfoWorks RS models during the detailed study by the Province of Antwerp. These time series of urban runoff are not long enough for the extreme value analysis to be carried out in order to construct composite hydrographs for flood analysis. With a view to generating long time series of urban runoff, several correlations have been checked for a simple conceptualization of the runoffs, such as correlation between rainfall and urban runoff etc. A high correlation was found between rainfall at Turnhout station and urban runoff from the Aa sub-catchment, while a high correlation was observed between rainfall at Uccle station and urban runoff from the Visbeek sub-catchment. Figure 4.1 shows the correlation between rainfall and two urban runoffs from two different areas in the city, for example, that were used in the simulation of IW-RS models.



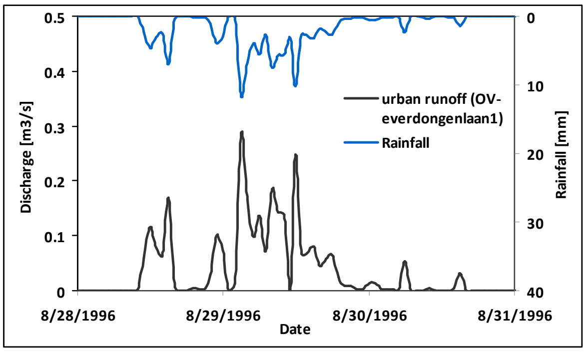


Figure 4.1: two examples (urb_asw_vb and OV- everdongenlaan1) of correlations between rainfall and urban runoff.

Figure 4.2 shows the generated long time series of two urban runoffs, for example, on the basis of the correlation found out as depicted in the figure 4.1. The relation between rainfall and urban runoff is that a certain percentage of rainfall contributes to the urban runoff which has been checked and calculated for all the runoff coming from different sources in the city of Turnhout (table 4.1). Finally, the correlation of an urban runoff with rainfall was expressed as an average percentage of rainfall to generate urban runoff for long time period.

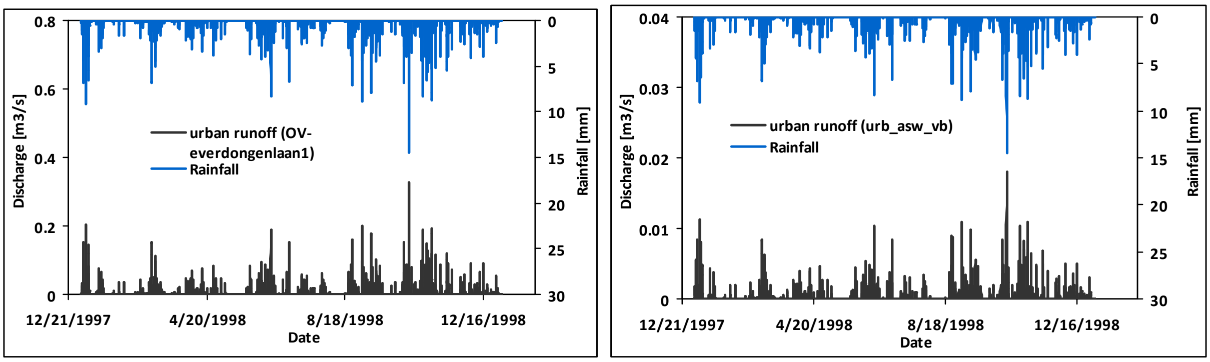


Figure 4.2: generated urban runoff using their correlations with rainfall.

Name urban runoff	Rainfall contributes to urban runoff (%) (average)
urb_sb (UB1)	1.76
urb_rh (UB2)	2.8
urb_asw_zv (UB3)	0.16
urb_swn (UB4)	0.18
urb_E34o (UB5)	0.21
urb_E34 (UB6)	0.06
urb_asw_vb (UB7)	0.12
OV_everdongenlaan1 (OV1)	2.25
OV_Everdongenlaan2 (OV2)	1.84
OV_Everdongenlaan3 (OV3)	1.07
OV_parking (OVP)	1.63

Table 4.1: the correlations between rainfall and different urban runoffs.

The correlation is then verified using the output from the existing IW-CS model. Figure 4.3 shows an example of a comparison between the urban runoff derived from IW-CS model and the urban runoff generated using the correlation between rainfall and urban runoff. The comparison demonstrates that there's a correlation between rainfall and urban runoff during the peak event because the response of the urban drainage system is very fast but the same correlation is not so good as expected comparing to the correlation find out between rainfall and urban runoff in the results taken from Infoworks RS model. Further analysis of the relation between rainfall and runoff are recommended for future studies in order to find out the real correlation between input and output such as the calibration of conceptual models that replace the detailed physically based IW-CS in a simplified way.

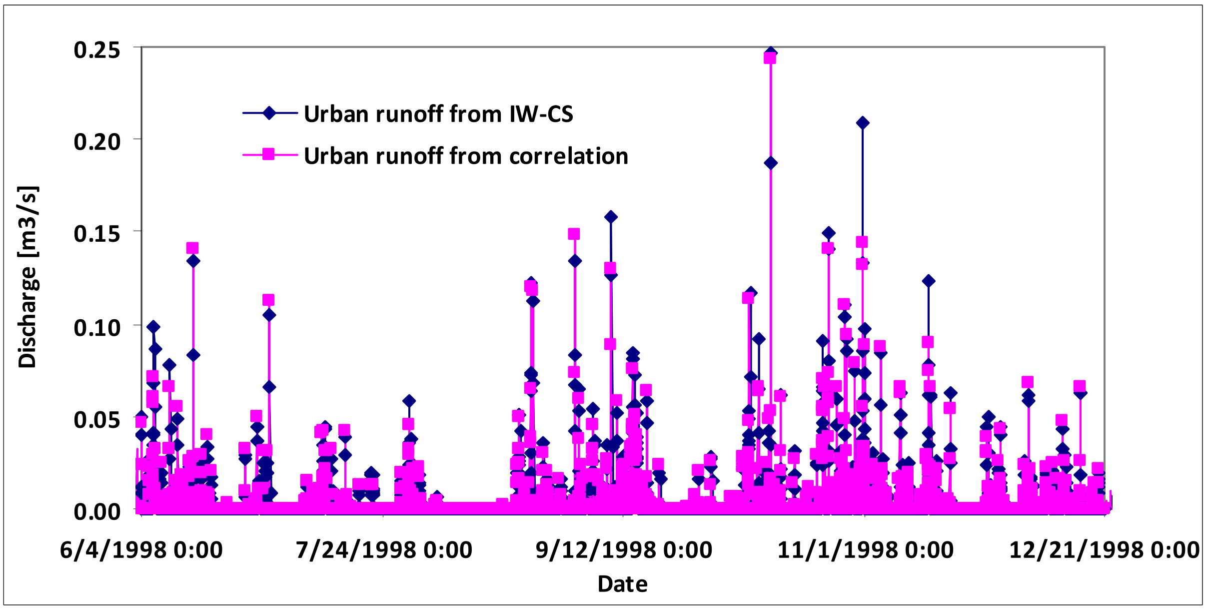


Figure 4.3: comparison between urban runoff generated from correlation and urban runoff simulated from IW-CS model for an urban runoff source from the period from 6/4/1998 to 12/21/1998.

The future urbanization scenarios, based on the correlation found out between the urban runoffs and the rainfall, have represented current and climate condition. The perturbed rainfall is used to generate the urban runoffs from different sources in the city using different correlations established previously and defined in the table 4.1. As the rainfall-runoff, the time series of the urban runoff under current and future climate conditions are then statistically analyzed to construct composite hydrographs with different return periods.

4.4 Urban runoff as flow component

In the numerical filter technique, a time series of total rainfall-runoff discharges can be splitted into three subflows, such as the overland flow, the subsurface flow or interflow and groundwater flow or baseflow (Willems, 2009). Urbanization does not belong to the hydrologic domain, but it has an impact on the hydrologic cycle (Bauwens, 2009). Urbanization causes increase in the paved area removing the vegetated surface, which reduces infiltration through the impervious surfaces. The increasing proportion of paved surface within the urban fabric transforms the hydrographs to ones that are characterized by high flow peaks and fast response to even moderate rainfall events (Semadeni-Davies et al., 2008). Figure 4.4 shows the effect of urban runoff (as a consequence of urbanization) on simulated river discharges which is characterizing the hydrograph by an increase in the peak with an indication of quick response to the rainfall event. With further investigations in other study areas, this urban runoff can be considered as a fourth component of total rainfall-runoff discharge to confirm the results of the present study.

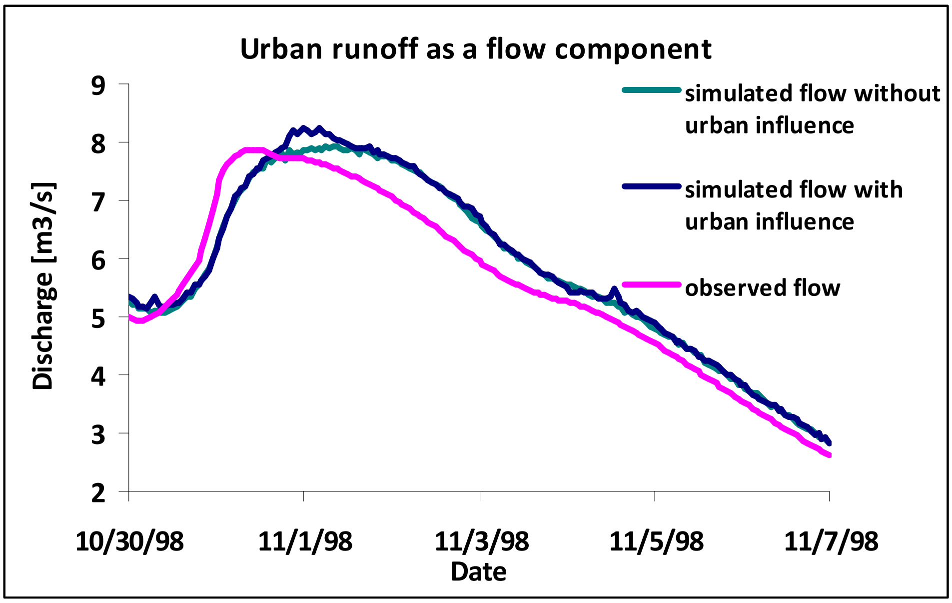


Figure 4.4: influence of urban runoff on the simulated river flow by MIKE 11 river flood model at the boundary downstream.

4.5 Impact of urban runoff on flooding

Total eleven urban runoffs were considered to assess their impact on peak runoff from two sub-catchments. Out of these urban runoffs, four are planned to be buffered in future by the city authority. It is planned that runoff collected through the new rainwater collector system would be stored during the high flow periods in the green spaces at the transition of the city and the river and it is expected to have positive implication on flooding in the downstream of the city. Firstly it is imperative to ascertain the impact on the peak urban runoff from the sub-catchments. In order to assess this impact, the peak over threshold (POT) runoff values, the simulated runoff results of NAM, were selected for subsequent extreme value analysis. Then extreme value analysis reveals that there is an average increase of about 74% in the peak runoff from two sub-catchments due to these urban runoffs. Figure 4.5 indicates that the frequency of an extreme high event will be increased (i.e. decreasing the return period) if these urban runoffs continue to be added to the total runoff.

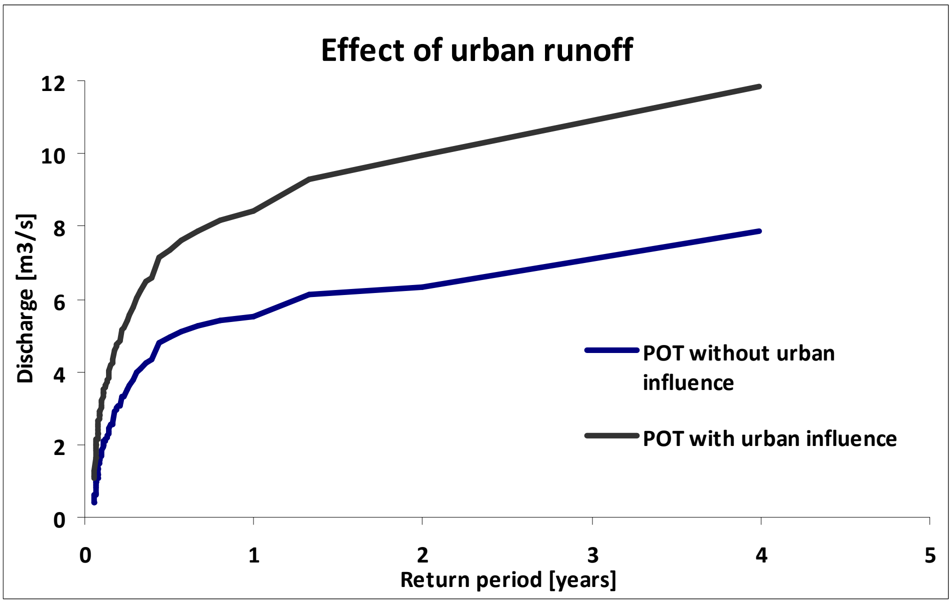


Figure 4.5: effect of urban runoff from the city on peak runoff from the two sub-catchments.

The flood maps generated from the simulation of composite hydrographs illustrate the impact of urban runoff on flooding areas more precisely. Figure 4.6 illustrates the extent of flooded areas near the confluence due to urban runoff for return periods of 5, 10, 25, 50 and 100 years.

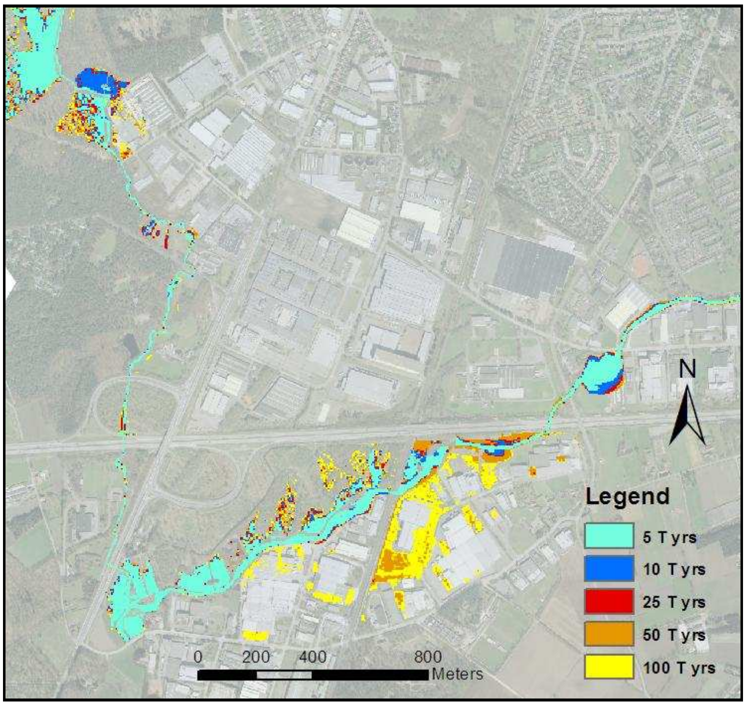


Figure 4.6: flood maps for the region near the confluence of Aa and Visbeek rivers for return periods of 5, 10, 25, 50 and 100 years considering influence of urban runoff.

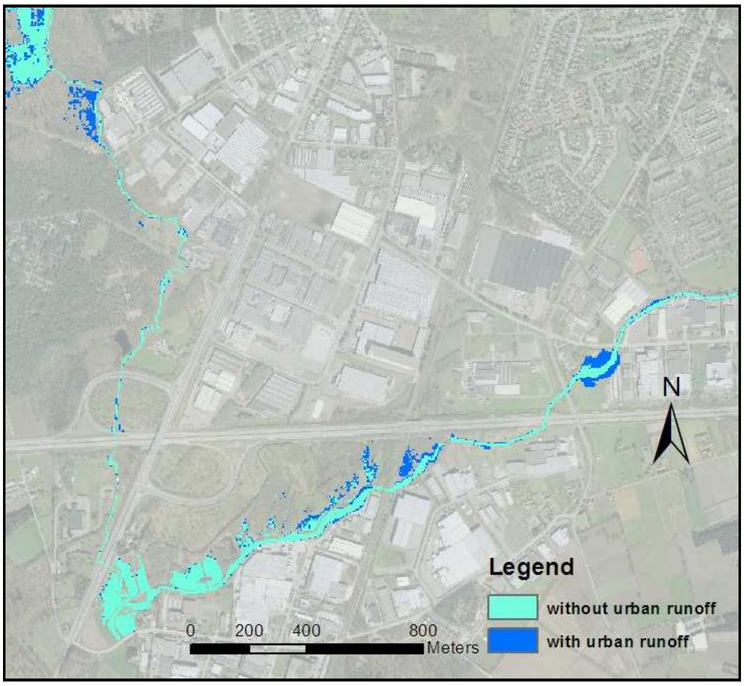


Figure 4.7: flood maps for the region near the confluence of Aa and Visbeek rivers for return periods of 5 years with and without urban runoff.

The relationships between flooded area and return period are important to understand the floodplain behaviour in response to flood events which provide very useful information for flood control measurements and planning of the floodplains. The relationships between the volume of flooding water and return periods are also useful for the design of buffer zones to store urban runoff coming from the city to avoid flooding in the downstream of the city. These relationships allow water managers to decide what adaptive measures are to be implemented for the vulnerable areas to overcome the adverse situation of flooding. Specifically, these relationships are very useful for the prevailing flooding situation in Turnhout to search for different water management alternatives, such as infiltration ponds within the urban fabric, buffering the urban runoff in the green zones at the threshold of the rivers, retention of water using the natural bottlenecks in the upstream region etc.

Table 4.2 shows the changes in flooded area and volume of flooding water with different return periods of urban runoffs as compared to the condition without considering any impact.

Return period	**Change in flooded area (%)**		**Change in volume of water (%)**	
(years)	All floodplains	Lower valley	All floodplains	Lower valley
5	13.6	36	22.6	45.5
10	12.5	37.5	21	45.7
25	12.8	40.5	21.3	46.4
50	13.6	41.9	21.8	47.8
100	18	50.5	22.8	48.5

Table 4.2: change in flooded area and volume of flooding water with the return periods of urban runoffs compared to the situation without any influence.

In case of floods with lower return periods, urbanization increases peak discharges and flood magnitudes (Hollis, 1975; Hirsch et al., 1990; Leopold, 1994; Schueler, 1994; Paul and Meyer, 2001; Konrad and Booth, 2002 cited in White and Greer, 2006). The results for all the floodplains also show the consistency in the city of Turnhout as the increase in flooded area due to urban runoff is the maximum (i.e. 13.6%) for the flood event with return period of 5 years which could better understood with lower return periods. Urban runoffs might aggravate the situation of flooding in the downstream of the valley as they cause further extension of flooding areas, increasing from 18% in all the floodplains to 50.5% in the lower valley for a return period of 100 years. Other flood maps showing the impact of urban runoff on flooded areas for return periods of 10 and 100 years are presented in Appendix C in figure C.9 and C.10.

4.6 Impact of climate change on flooding

The impact of climate change alone on peak runoff is presented in figure 4.8 which also shows an indication of significant influence on the regional extreme runoff with an increase of peak runoff by 30% on an average as compared to the current condition. The extreme value analysis reveals that the return period of a flood event will be decreased by about 3 times causing the event to be about 3 times more frequent in future with comparison to the current frequency of occurrence. This trend might be useful for the city water managers to search for the options which must be designed to be functional to combat this exaggerating impact that has not yet been taken into consideration in any of the alternatives proposed hitherto. Figure 4.9 shows the extent of flooded areas for climate change scenario in 2100s with comparison to current flooded areas for a flood event with return period of 100 years. Other flood maps showing the comparison between current and future climate conditions are presented in Appendix C.

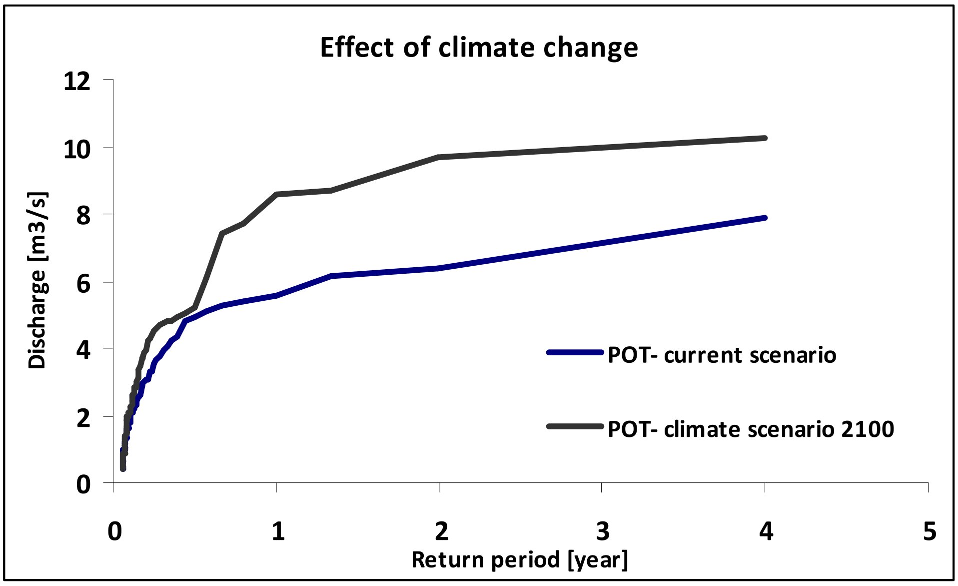


Figure 4.8: effect of climate change on peak runoff from the two sub-catchment.

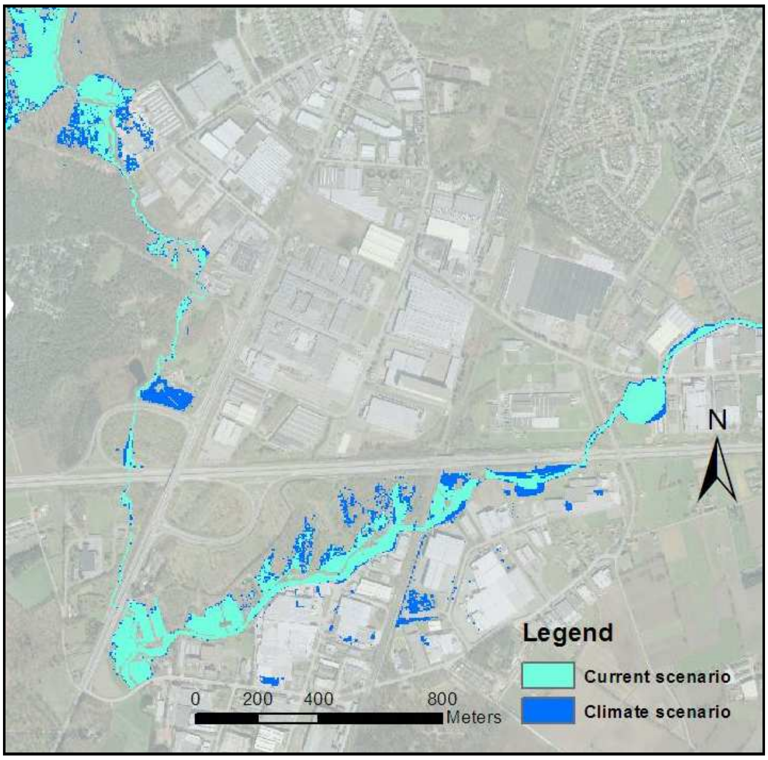


Figure 4.9: the extent of flooded areas near the confluence of Aa and Visbeek rivers due to climate change for return period of 100 years without considering urban runoff.

Return period	**Change in flooded area (%)**		**Change in volume of water (%)**	
(years)	All floodplains	Lower valley	All floodplains	Lower valley
50	32	35	33	39
100	34	38	34	40

Table 4.3: change in flooded area and volume of flooding water between current condition and climate scenario for 2100s without considering urban effect.

The results show that the further extension of flooded area could be quite frequent under future climatic conditions. The climate change alone may aggravate the situation although this issue has not been considered by the water managers in the city. The flooded area in all floodplains increases by 32% and 34% due to climate change in comparison to the current scenario for return period of 50 years and 100 years respectively. The situation might be even worse in the lower valley of the rivers as the flooded areas are further extended due to climate change by higher percentages. The results above provide important information of inundation area and volume of water in the flooded areas which could be utilized to design appropriate management alternative to overcome any future water hazard in a holistic way.

4.7 Combine impact of urban runoff and climate change on flooding

Since the runoff is more sensitive to urban runoff (i.e. urbanization) than to climate change, the situation might deteriorate further if the two aspects are combined together. The increase in peak runoff becomes higher when the impact of both the variables is taken into account concurrently than they are considered individually. The average peak runoff increasing is expected to be double with comparison to the current trend (figure 4.10). This intensification might cause more frequently and higher floods than the one prevailing currently and being considered in selecting an appropriate management alternative to overcome the problem. The relative impact of urbanization and climate change both individually and combined is presented in figure 4.11.

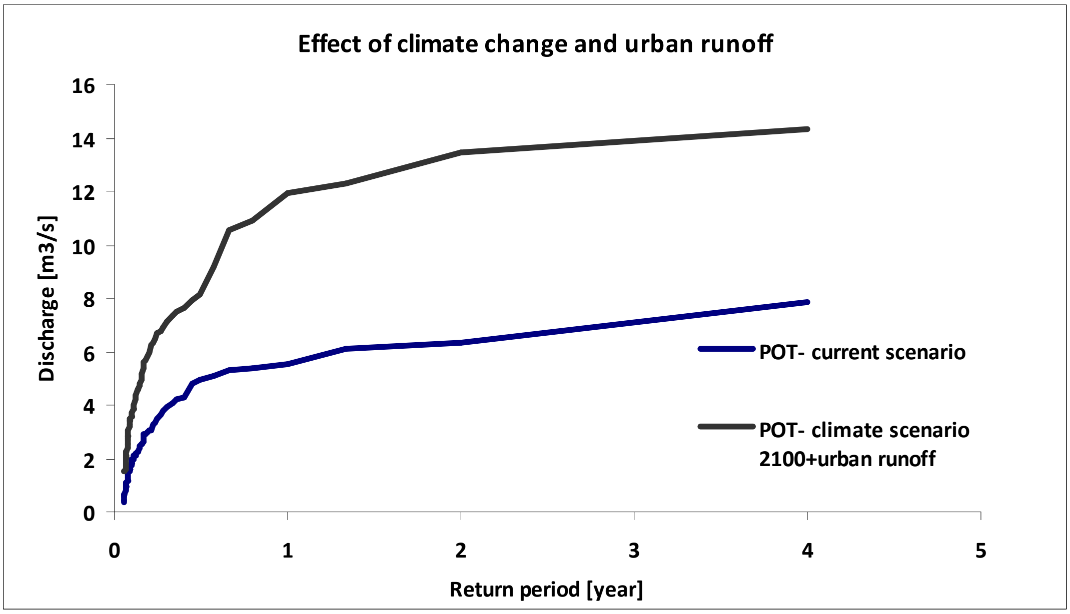


Figure 4.10: effect of both the urban runoff and climate change on peak runoff from the two sub-catchments.

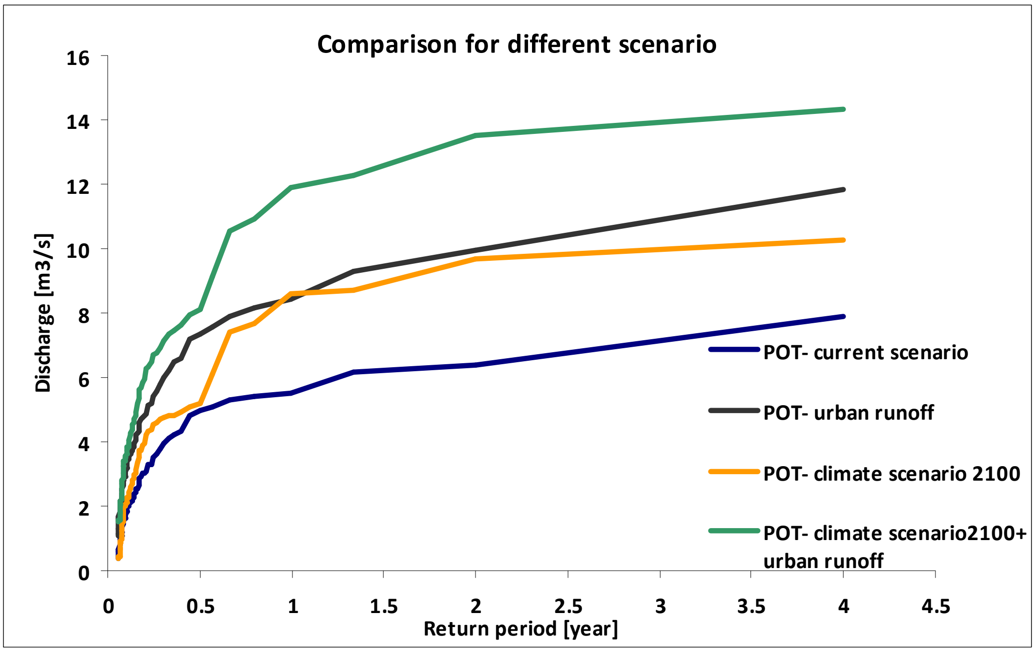


Figure 4.11: relative effect of urban runoff and climate change both alone and together on peak runoff from the two sub-catchments.

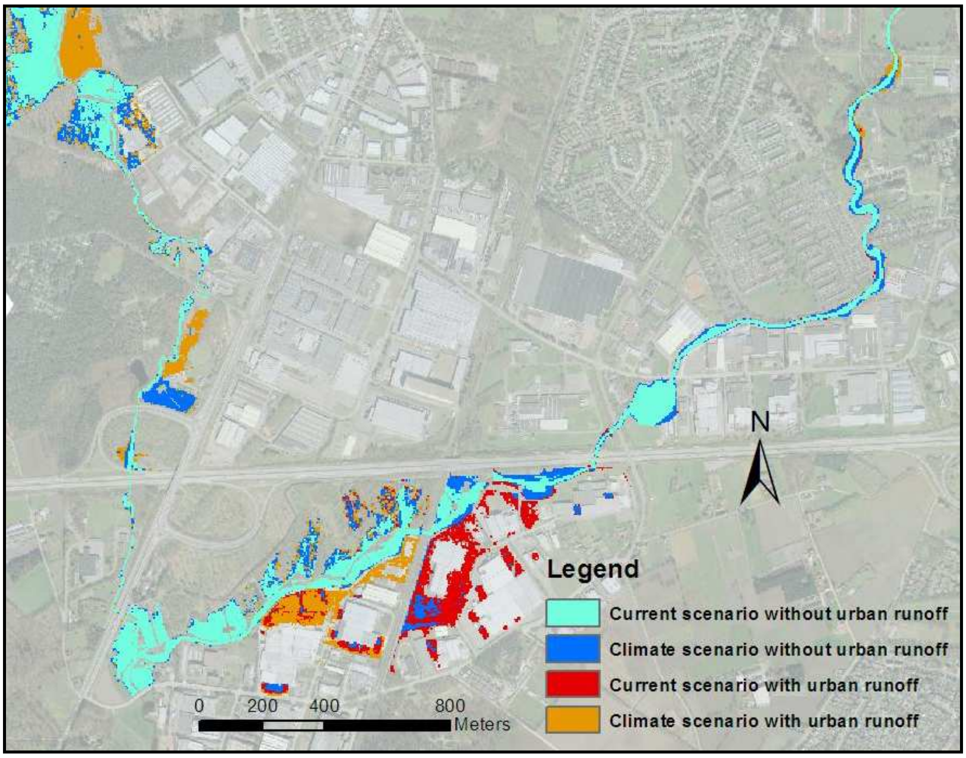


Figure 4.12: flood maps of climate change scenario with and without urban runoff in comparison to current conditions with and without urban runoff for return period of 100 years.

Return period	**Change in flooded area (%)**		**Change in volume of water (%)**	
(years)	All floodplains	Lower valley	All floodplains	Lower valley
50	42	66	55	70
100	45	67	59	74

Table 4.3: change in flooded area and volume of flooding water between current condition without urban impact and climate scenario for 2100s combined with urban runoff.

An amplification of both inundation area and volume of flooding water resulting from intensified runoff from the sub-catchments is caused from the combined effect of urbanization and climate change, when compared with only climate change or only urbanization. For return period of 100 years, the inundation area in all the floodplains increases from the current condition by 45% due to climate change with comparison to 18% due to only urbanization and 34% due to only climate change. The flooding situation is exaggerated in the downstream of the city when the combined impact of climate change and urbanization is taken into account. It was observed during the detailed study by the Province of Antwerp that the agricultural lands in the lower valley are mainly affected by the frequent flooding events and the flood maps generated in the present study are consistent with the situation with all the scenarios analyzed.

4.8 Conclusions

The impact of climate change and urbanization on flood risk were studied both separately and in combination. Perturbated rainfalls and evapotranspirations were obtained for the climate change scenario using the perturbation algorithm developed in the CCI-HYDR project at Hydraulics Laboratory of K.U. Leuven for facilitating the climate change impact assessment in Belgium.

In the evaluation of the urbanization effects on the flood risk a total of eleven urban runoffs are taken into account. The analysis of the results have shown that flood events with lower return period (i.g. T=5 years) have revealed an higher increasing in flooded area due to urban runoff comparing to the results for higher return period. However, in the extreme analysis the frequency of an extreme high event is increased in the peak runoff due to this urban runoff, in particular for lower return period.

The impact of climate change alone on peak runoff has also shown a significant influence on the regional extreme runoff with an increase of peak runoff in average as compared to the current condition. Furthermore, the extreme value analysis reveals that the return period of a flood event will be decreased by about 3 times causing the event to be about 3 times more frequent in future with comparison to the current frequency of occurrence.

The situation was extremely deteriorated when including both urbanization and climate change impact. The increase in peak runoff becomes much higher when the impact of both the variables is taken into account concurrently than when they are considered individually. This intensification is clearly reflected in a increasing of the flooding areas.

To reduce the existing and future flood risk adaptive measured of integrated water resource management have to be considered to overcome the problem, such as the design of buffer zones to store urban runoff coming from the city and to avoid flooding in the downstream of the city.

Chapter 5

IWRS adaptive measures to reduce the flood risk

5.1 Introduction

Urban development is an useful illustration of some of the most obvious effects of land use change on water management. Vegetation soils are replaced with impermeable surfaces, increasing overland flow and reducing infiltration, bypassing the natural storage and attenuation of the subsurface. In addition, the conveyance of runoff to stream is modified. Overland runoff is conventionally collected by pipe storm-water drainage systems and conveyed rapidly to the nearest stream. The result is a greater volume of runoff, discharging in a shorter time, potentially leading to dramatically increased flood peaks, but also reduce low flow and less groundwater recharge (Wheater, Evans 2009).

Not only to solve the current flood problems, but also to prevent more water problems with the future densification of its urban fabric, an ambitious plan of water management has been developed for the city of Turnhout. Different measures have been proposed to favour water retention within the urban area, which permit a temporal redistribution and the attenuation of high floodwaters. One of the option proposed by the city authority of Turnhout is the decoupling of the waste and rainwater evacuation systems to relieve the sewer network and therefore to limit the risks of overflows. This will imply the development of a new parallel (piped) rainwater collector system and also require new spaces for water infiltration and buffering.

Within the scope of this study, the potentiality of the buffers and the introduction of storage element inside the city will be assessed as effective management options to have positive repercussion on the existing flooding situation deeming that the city urban runoffs are exaggerating the crisis.

5.2 Existing model for the city of Turnhout

InfoWorks CS is a comprehensive, easy-to-use and flexible system for the management of urban drainage network models, including stormwater or wastewater drainage systems (or a combined stormwater and wastewater system, as in the present study). A network contains all the information needed to describe the drainage system. Each network is modelled as a collection of sub-catchment areas that drain to nodes (manholes or grade breaks) which are joined by links (conduits, pumps etc.) (Wallingford, 2009).

The software incorporates the sophisticated HydroWorks modelling engine for simulating the behaviour of the network under many different conditions. When a network has been set up, event data (real or synthetic) representing the volume of water entering different points can be supplied on the network over a period of time. The modelling engine then runs a simulation to demonstrate the effect of the water on the network, so that weak points in the network can be identified.

Many different models can be created for the same network, based on different event data and various periods of time, from a few hours to a number of years. In Turnhout case, an existing model for the collector system, InfoWorks CS, has been generated by Province of Antwerp using composite storms, based on the IDF relationships for Flemish design applications determined in 1996 of the rainfall data at Uccle station operated the Royal Meteorological Institute of Belgium (*KMI*) for the period 1967-1993 with a time step of 10 minutes.

The model, which is in accordance with the plan of water management in partnership with the river basin coordinator, includes the present sewer system, consisting of a combine (rain+waste) water system and the new parallel (piped) rainwater collector system, and was set up in order to study new solutions to prevent flooding events. This new pipe collector, called RWA, required also the presence of new space for water infiltration and buffering. Figure 5.1 shows the location of some of the planned buffers for the city of Turnhout.

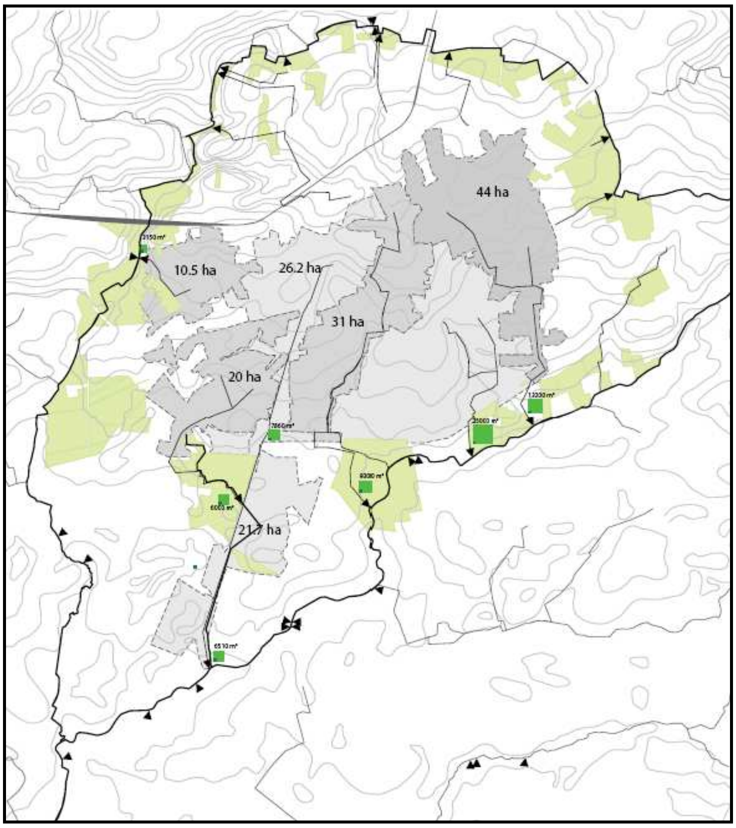


Figure 5.1: representation of the planned water collector pipes, the corresponding covered area and a representation of the water buffer needed (green squares representing the m³ in m² at scale (source: Nolf, C., 2010).

5.3 Implementation of the buffer

The buffer zone or 'green storage areas' are natural systems in which the runoff from the new collector system is temporarily stored and destined to infiltration and/or evaporation or, if it reaches the maximum volume, is released to the river in a second moment. The buffers are going to be positioned at the end of this new collector system.

Naturally this storage process attenuates the flood peak and delays the flood timing so that its volume is discharged over a longer time. An initial volume of this buffer has been defined based on the covered area of the new collector pipe and considering the limitation of 280 m3/ha (normative). Another limitation is that the water that overflows the buffer and reaches the river must not exceed 5 l/s/ha, according to the code of good practice (code van geode practijk). Table 5.1 lists the location of the buffers and the corresponding covered area, volume and limit of the discharge that is allowed to be released into the rivers.

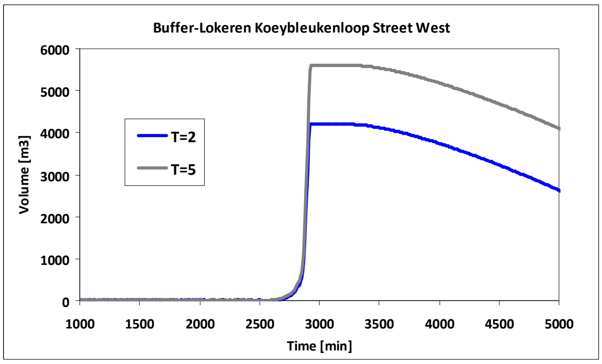
Location	covered area [km2]	Volume [m3]	Allowable limit of discharge [m3/s]
Lokeren Koeybleukenloop Street West	0.2	5600	0.1
Koeybleukenloop Nord	0.26	7330	0.13
Koeybleukenloop zuid	0.22	6070	0.11
Turnhout Oost	0.44	12320	0.22
Kasteelloop	0.31	8680	0.16
Galgebeek	0.11	2940	0.05

Table 5.1: location of the buffers and the corresponding covered area, volume and allowable limit of the discharges (adapted from Province of Antwerp 2004 & 2006).

The buffer are implemented in the model as storage node at the end of each RWA collector under study and shown in figure 5.1. The buffers were inserted with a square base and a height of 1 meter. A weir links the buffer to the outfall. The presence of the weir has allowed to control the maximum quantity of the water that overflows which is limited by the normative.

In order to take into account the loss of infiltration at its base, SUDS parameters were inserted in the storage element. The term SUDS describes a variety of structures designed to lessen the impact of urban development on the environment. These structures are also frequently referred to as BMPs Best Management Practices (BMPs).

One important parameter was the infiltration loss coefficient, which has been determined by previous studies about soil’s characteristic in the city and the design values. Indicative values that were chosen in this studies were 30 mm/h for the two internal buffers, where the soil is prevalently sand, and a value of 5 mm/h for the other 4 buffers close to the rivers, where the soil is almost clay. The different values of the infiltration reflect the behavior of the buffer during extreme events, as shown in figure 5.2.



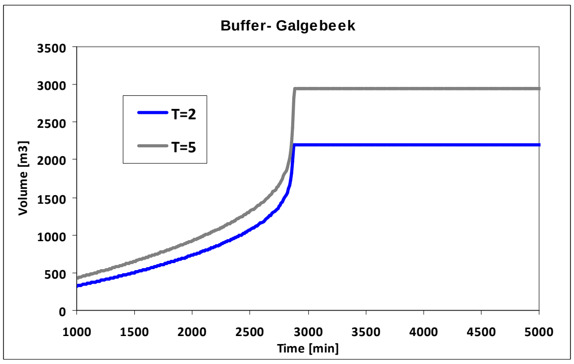


Figure 5.2: variation in volume for return period of 2 and 5 years for the buffer located in Lokeren Koeybleukenloop Street West (inside the city) and Galgebeek (close to the river).

These two examples depict the diverse variation in volume that occurs in two distinct points of the city due to the different type of soil and, as a consequence, of the infiltration rate. In Lokeren Koeybleukenloop Street West the volume is approximately zero at the beginning of the simulation and tends to decrease after the peak event because of the high value of the infiltration. In Galgbeek, that is close to the river where the infiltration rate is very low, the behaviour is different: the volume increases and remains constant after reaching the maximum value.

In engineering practice, however, simpler approaches can be followed in order to reduce complexity of analysis and calculation efforts and simplifications can be made in the storage process simulation. A simple conceptual model was built in order to simulate the behavior of the buffers, calculating the water that overflows and verifying the limits of the volume and the overflows.

5.4 Simple conceptual modelling for the buffers

The simple conceptual model is based on the equation of continuity and the buffer has been considered as a “green” storage with a square base and a height of 1 meter, and considering loss of water as infiltration. A value of 5 mm/h was taken for the buffers located close to the river, where the soil is prevalent sandy, and 30 mm/h for the two buffers inside the city, where the soil is prevalently clay, as it was done in the implementation of the buffer in the CS model. In the simulation process the water that enters into the buffer can be stored until it reaches the maximum volume. Above the maximum volume, the water that overflows is determined by a weir equation with a width of 5 meter, respecting the limit from normative. A schematic diagram of a buffer is shown in figure 5.3.

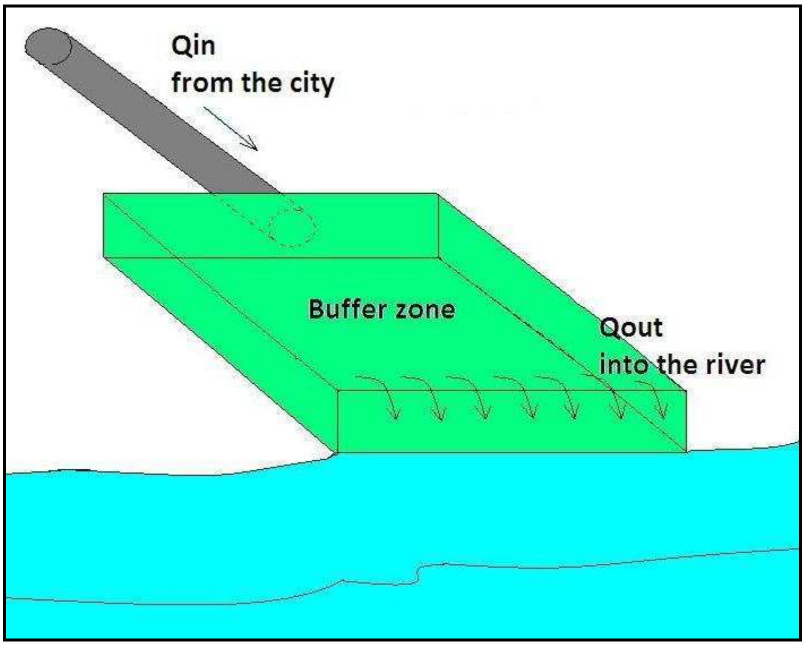


Figure 5.3: a schematic diagram of a buffer.

Results of the latest pipe in the new collector system have been extracted from the existing IW-CS model of the city and have been used as input of the simple conceptual model. The output from these buffers as a result of overflows above the threshold volume is used as point source boundaries have been considered in the model's simulations. Finally flood maps are generated to assess the effectiveness of these buffers on the extent of frequent flooding downstream of the valley (table 5.2).

Name	Location	Connecting River	Chainage (m)
OV_Everdongenlaan1 (OV1)	Koeybleukenloop zuid	Aa (category-1)	764.85
OV_Everdongenlaan2 (OV2)	Turnhout Oost	Aa (category-2)	13390
OV_Everdongenlaan3 (OV3)	Kasteelloop	Aa (category-2)	15791.94
Urb_rh (UB2)	Galgebeek	Visbeek	5871.32

Table 5.2: name and location of the buffers used as point sources in MIKE 11 river flood model with the chainages at the connection to the rivers.

5.5 Assessing the effectiveness of the buffers to reduce flooding

One of the three alternatives proposed by the city water service to overcome Turnhout's water management problems is a 'drempel (threshold) park' where the output from the projected new rainwater collecting network will be buffered before entering the surrounding rivers at their threshold. These buffering elements can act as transitions between the urban centre and the open space fens (Province Antwerp, 2004 & 2006). In the present study different scenarios are attempted to be analyzed to see the effectiveness of this flood management measure called 'buffer'. Firstly, the proposed buffers are integrated into the MIKE 11 river flood model as point sources with the overflow discharges of the buffers within the allowable limit. The effectiveness is then verified against different synthetic flood events with different return periods. The buffers are planned to reduce the flooding areas in the lower valley of the rivers. Hence, attempt has been made to analyze their efficiency to minimize the hazards in the lower valley.

The ranked POT values of runoff discharges before implementing the buffers were compared with the ranked POT value of runoff discharge after implementing the buffers (figure 5.4). Table 5.3 shows the decrease in flooding areas and volume of flooding water after implementation of these buffers.

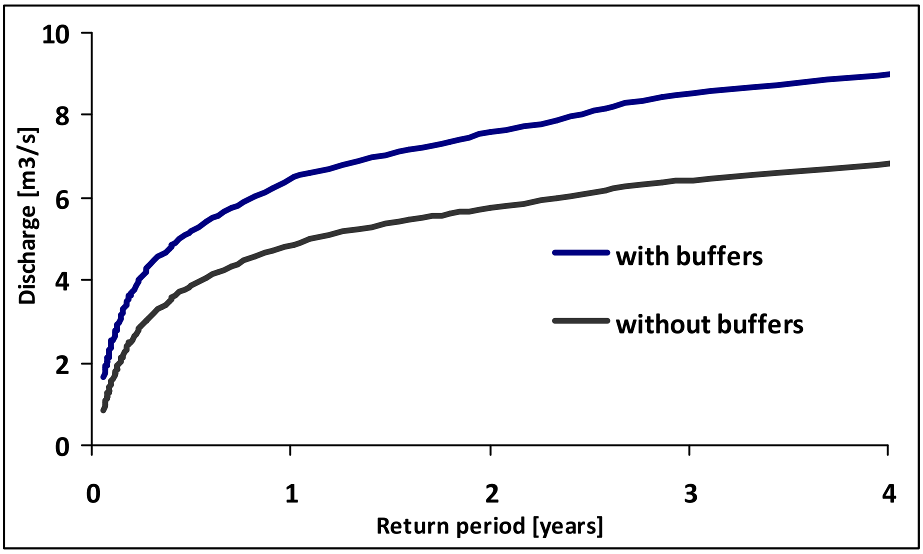


Figure 5.4: effect of buffers on runoff discharge from the two sub-catchments.

Return period	Change in flooded area (%)		Change in volume of water (%)	
(years)	All floodplains	Lower valley	All floodplains	Lower valley
5	8.6	22.8	12.8	25.9
50	9.2	27.7	13.6	27
100	15.1	38.19	13.7	27.7

Table 5.3: decrease in flooding areas and volume of flooding water after the buffers implementation.

Results show a reduction of peak runoff discharge by an average value of 32.5% after the implementation of buffers under present condition. Furthermore, there is some degree of change in the flooding areas in the downstream of the city which could lead to the partial removal of the existing problem in the region. Figure 5.5 and figure 5.6 depict the effectiveness of the buffers by mean of comparing the flooding situation before and after the buffers were implemented.

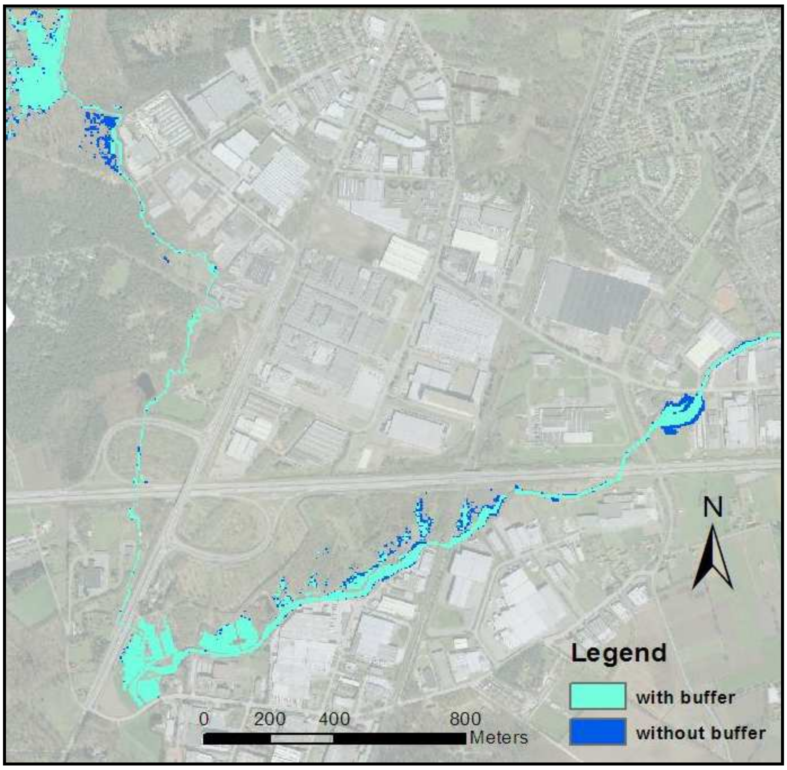


Figure 5.5: flooding situation in the lower valley of the rivers before and after the buffers are installed against the flood event with return period of 5 years.

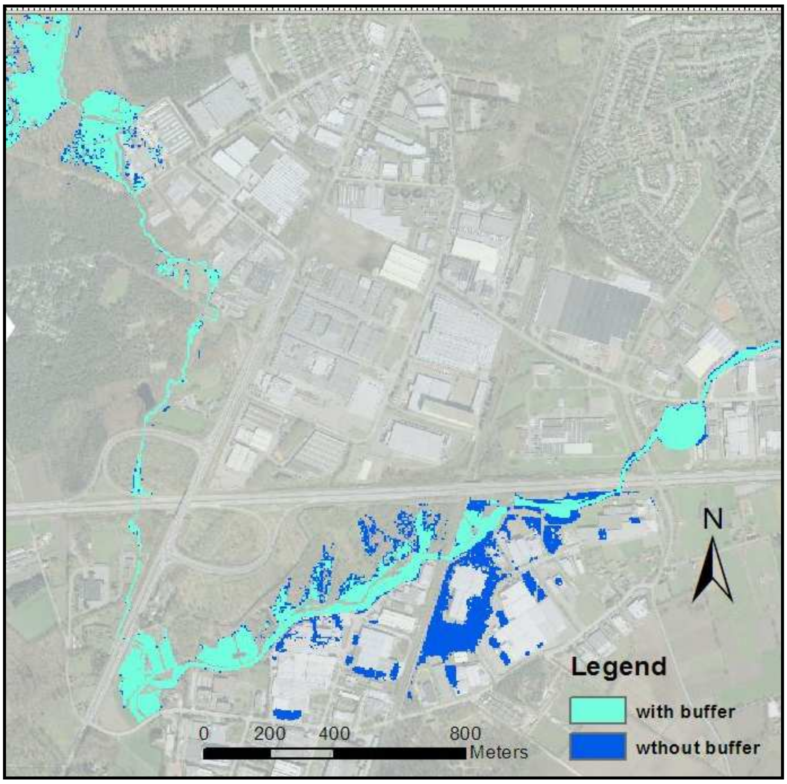


Figure 5.6: flooding situation in the lower valley of the rivers before and after the buffers are installed against a flood event with return period of 100 years.

The buffers contributes to reduce the flooding down the valley with a decreasing of flooding area along the Aa river. Further analysis with the consideration of all the proposed buffers in long term might reveal the real picture of their effectiveness more confidently.

The issue of climate change is another variable which might challenge the proposed options of reducing flood down the valley and more importantly, it has not been taken into consideration in the planning of these buffers hitherto. Figure 5.6 shows the reduction of peak runoff discharge under climate change scenario with the implementation of buffers while figure 5.7 shows the effectiveness of the buffers to counteract the impact of climate change on a flood event with return period of 100 years.

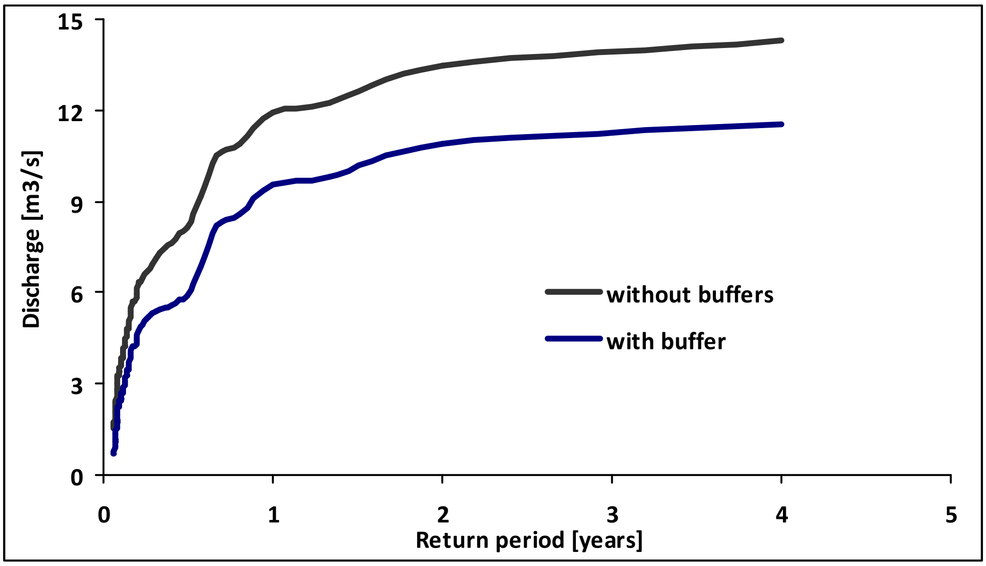


Figure 5.7: effect of buffers on runoff discharge under climate change scenarios.

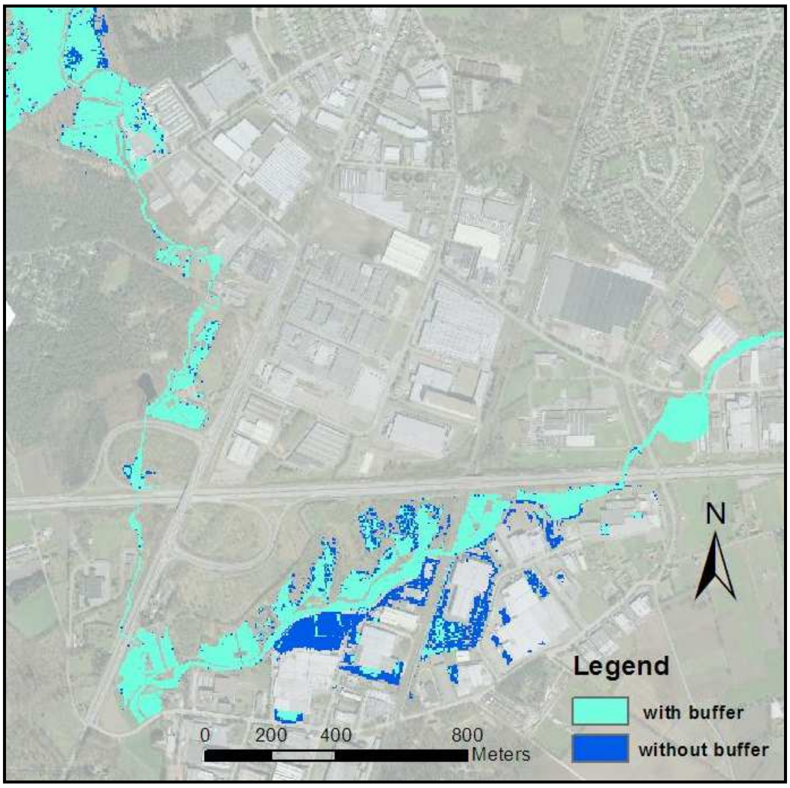


Figure 5.8: flood map showing the effectiveness of buffers on the flooding areas in the lower valley under future climate change scenarios for the 2100s.

Table 5.4 shows the decrease in flooding areas and volume of flooding water after implementing these buffers under future condition.

Return period	**Change in flooded area (%)**		**Change in volume of water (%)**	
(years)	All floodplains	Lower valley	All floodplains	Lower valley
50	9.3	32.5	11.4	9.2
100	10.2	35.5	19.9	10.1

Table 5.4: decrease in flooding areas and volume of flooding water after the buffers have been implemented under climate change scenario.

Under climate change scenarios, the buffers are able to reduce peak runoff discharge by an average value of 31.8%. The percentage reduction under climate change scenarios is lower than that under present condition after implementation of the buffers, because the combined impact of urbanization and climate change is more amplified than their individual impact and in addition the high scenario causes an increasing of the runoff discharges that the buffers could not contain. However the final determination of their effectiveness depends on further investigations including new planned buffers and improving the conceptual model by considering an appropriate initial volume of the buffers, a real coefficient of infiltration rate and the introduction of evaporation loss.

However the contribution of buffers on reducing flooded areas in the lower valley should be integrated to the other adaptive measures as an IWRM. Considering the high climate scenario the increasing of the volume of water that could not be contained by the buffer cause inevitably an increasing of overflow and of course an increasing of flood risk. Moreover, other water management alternatives proposed by VMM for further downstream of the city must be taken into account to come up with sustainable flood management options along the river valley.

5.6 Storage element inside the city

The effect of climate change due to urbanization should be taken into account by water managers in the next decades of this century in the planning of resizing the dimensions of the pipes in the sewer system and associating buffering (rainwater tanks, infiltration basins, etc.). Considering the latter aspect, accurate studies reveals that it is expected to have an increasing of the buffer volume between 15 to 35% under the high-climate scenario (Willems, 2009).

Not only increasing the size of buffer, but also having additional facilities which store rainwater and/or infiltrate into the ground should be taken into consideration to tackle the problems in low flow and flooding risk. Through such arrangements the risk of flooding during heavy rainfall is limited and the future expected increase in water stress is reduced (through the water storage, or infiltration to groundwater supplement). This can be reach through the construction of storm water facilities, but also through short-term storage on the streets or in public spaces (parks or other green areas, playgrounds, etc.) where a large quantity of water may temporarily be stored in case of heavy rainfall (Willems, 2009b).

In this study storage elements have been implemented in the sewer system model in order to assess the effectiveness in reducing the flood problem inside the city and in the lower valley, limiting the capacity of the sewer and also of the buffer zone.

This storage element are located in existing and potentially free spaces: micro valleys (lines of lower points) and depressed areas within the city may function as potential infiltration zones to which rainwater can be conveyed through the streets (streets act as rivers) with the steepest slopes in case of heavy rainfall. In figure 5.9 it is shown the potential infiltration areas in the city of Turnhout, where the storage element have been inserted in the CS model.

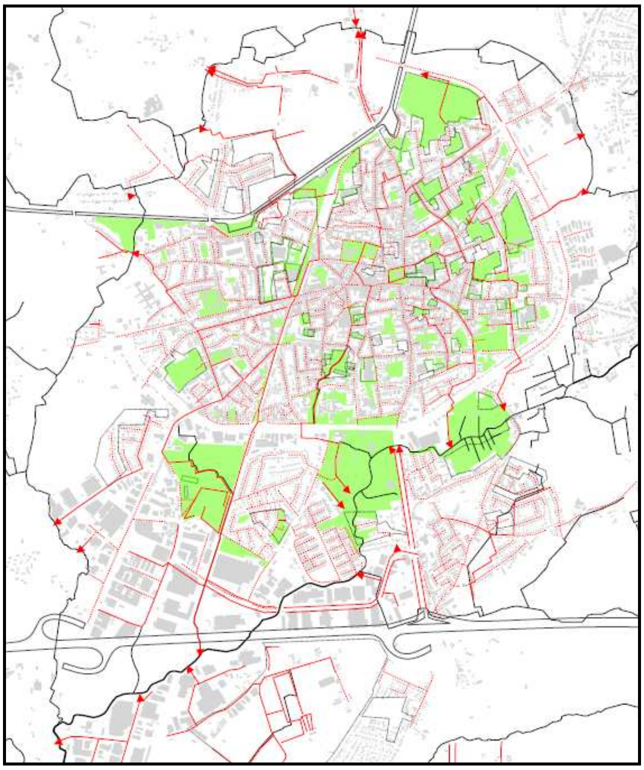


Figure 5.9: planned RWA (red lines) and open green space.

The storage elements have been implemented in Infoworks CS as storage nodes with a square base and a volume based on the limitation of 280 m3/s from normative. Each storage element is located underground the green space, and collects the water from the closest sub-catchment. The presence of infiltration is also included at the base of each storage element, with a value of 50 mm/h. A total of 28 storage elements have been inserted below the green areas, where the water from the closest catchment conveys into this element before going to the RWA system.

5.7 Effect of the storage elements in the city and in the lower valley

Simulation's results reveal that the presence of the storage elements in the city produce a clear reduction of the water coming through the pipes into the river. Investigation is made on the influence of the sewer linking on the amount of sewer pipes under pressure. This is done by analysing, for each of the 291 nodes under studies in the new RWA, the pressure height along the pipes considering the maximum values in the simulation results. When the pressure height is higher than the pipe crest level at any location along the pipe during the peak period, then this pipe is classified as a sewer pipe under pressure. When also the ground level is exceeded there is obviously sewer flooding on the nodes of the sewer system. In the table 5.5 are reported the results of the simulation for a return period of 5 years considering current and climate scenario.

Current scenario (T=5 yrs)	without storage elements		with storage elements		decrease of critical condition (%)
Under pressure	Total	224	Total	185	-
	Relatively (%)	77.0	Relatively (%)	63.6	17.4
Flood	Total	59	Total	35	-
	Relatively (%)	20.3	Relatively (%)	12.0	40.7
Climate scenario (T=5 yrs)					
Under pressure	Total	245	Total	221	-
	Relatively (%)	84.2	Relatively (%)	75.9	9.8
Flood	Total	159	Total	137	-
	Relatively (%)	54.6	Relatively (%)	47.1	13.8

Table 5.5: comparison between current and climate scenario of the total and relative amount of sewer pipes under pressure and the number of nodes that flood considering the return period of 5 years.

Under current scenario the number of pipes under pressure decrease from 224 to 185 during the peak event. With climate scenario the number of pipes increase up to 245 and the decreasing is less important than the current scenario, with a number of 221 pipes under pressure.

For both current and climate scenario the number of the pipes under pressure decrease respectively of 17.4% and 9.8%. Here is reported an example of one of the pipes under study, located near Boomgaardstraat.

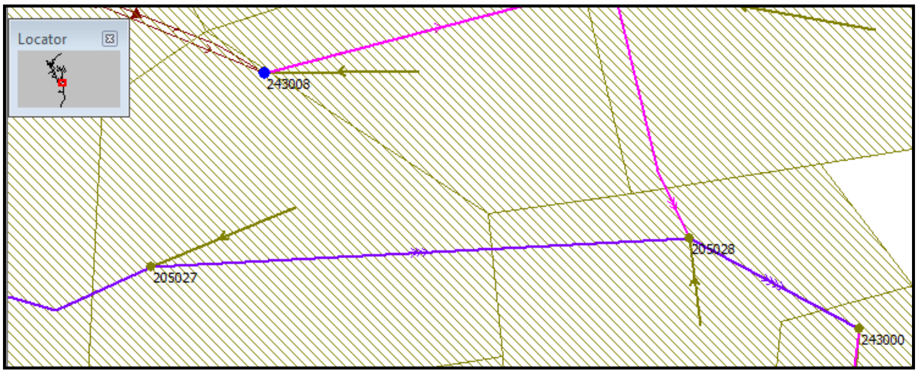


Figure 5.10: results of the simulation for the pipes between the nodes 205027 and 243000 before implementing the storage elements.

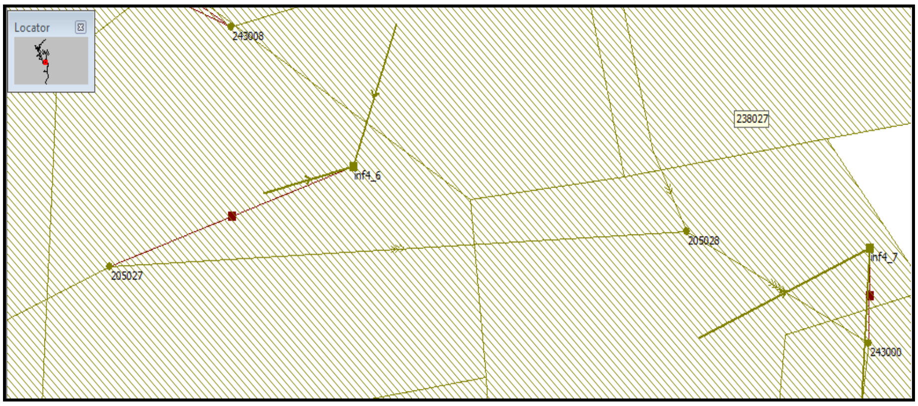


Figure 5.11: results of the simulation for the pipes between the nodes 205027 and 243000 after implementing the storage elements.

The simulation's result before implementing the storage element (figure 5.10) shows critical condition during the peak (violet colour) but, in the same part of the sewer system, this condition disappear after implementing the storage element (figure 5.11).

Also the amount of node that flooded during the peak event decrease with the presence of storage element, with a consistent reduction of 40% considering the current scenario. An example comes out from the storage element implemented in the green park near Albert Van Dyckstraat. The storage element is connected to the node 214007 (ID number). Figure 5.12 and 5.13 shows the simulation's result in water level of the node 214007 before and after implementing the storage element. The red line refers to the ground level.

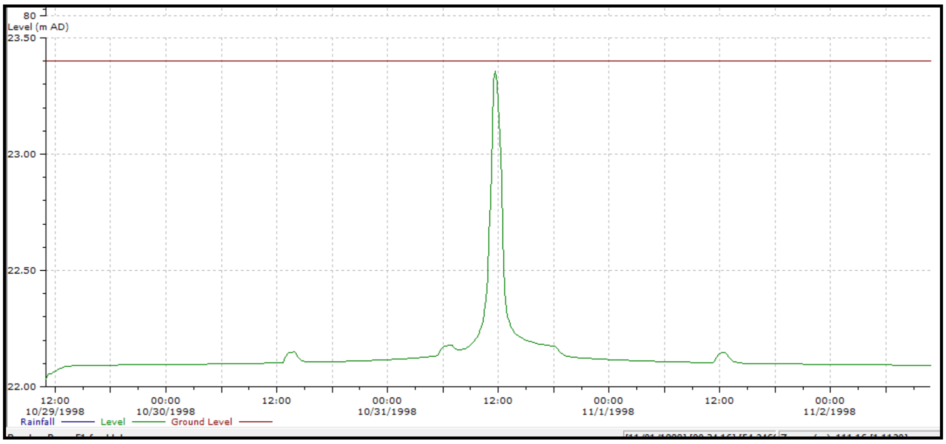


Figure 5.12: results of the simulation in water level for the node 214007 before implementing the storage element.

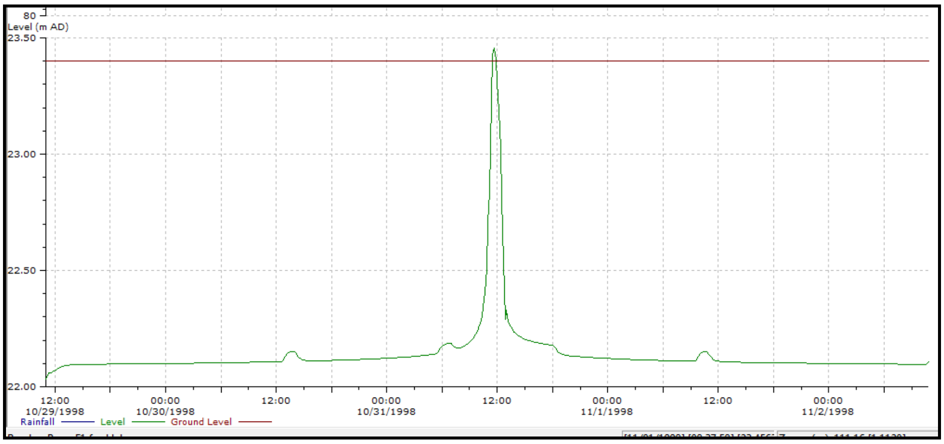


Figure 5.13: results of the simulation in water level for the node 214007 after implementing the storage element.

The node were the storage element is connected has a decreasing in the water level that changes from 23.46 m to 23.35 m, under the level of the nodes. It means that in this node the flood in the city is eliminated because the storage element stores a consistent quantity of water during the peak event. Under climate scenario, the results show a decreasing of the number of flooded nodes, which is less evident comparing the current scenario, about 14%.

The benefit of the storage elements is also relevant for the flooding problem in the rivers, as shown in figure 5.14 and in table 5.6.

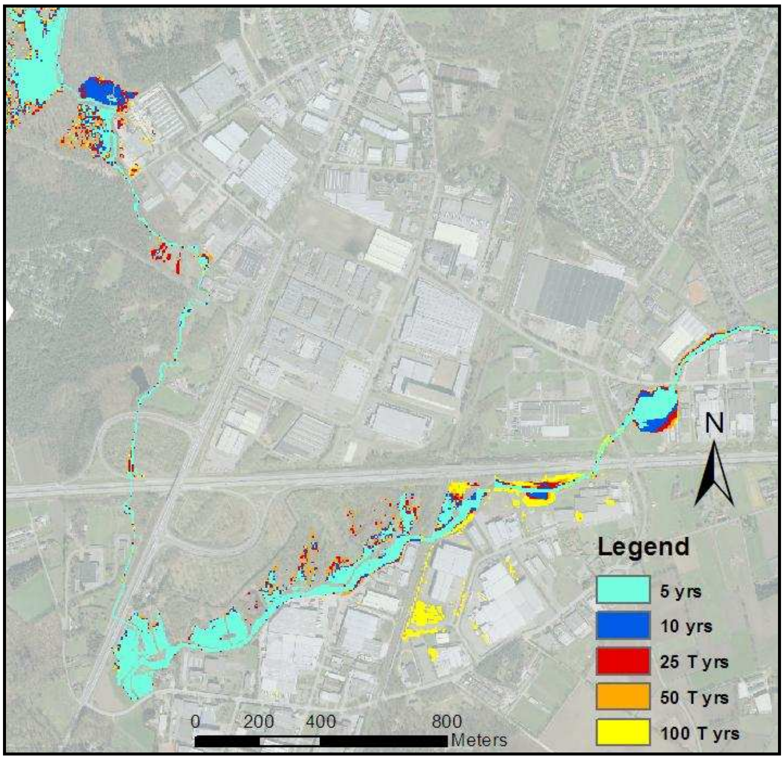


Figure 5.14: flood maps for the region near the confluence of Aa and Visbeek rivers for return periods of 5, 10, 25, 50 and 100 years considering influence of only storage elements inside the city.

Flood event/Return period	Decrease in flooded area (%)	Decrease in volume of water (%)
T=5 years	1.4	2.4
T=50 years	3.1	3.5
T=100 years	4.5	4.2

Table 5.6: decrease in flooding areas and volume of flooding water after only the storage elements have been implemented inside the city under current scenario.

The effectiveness of reducing the flood problem is not so evident in the lower valley comparing to the buffer option but the results are quite satisfying. The combine effect of the storage elements with the buffer reveals an additional reduction in the flooding area of the lower valley that varies, in percentage, between 1.1 and 1.5 from T=5 years to T=100 years but the combine effect doesn't introduce great effects in solving the flooding problem as the buffer's option. Even thought the effectiveness is not satisfying, the potentially of this storage elements in the city is proved by results from the sewer system model that consist in reducing critical condition in the pipes, reducing the flooding in some nodes and store water that should reach the buffer zone.

5.8 Conclusion

In this chapter potentials of the buffers and introduction of storage element inside the city were assessed as effective integrated water resource management options to reduce the impact on flood risk in the lower part of Turnhout city.

The buffers were implemented in the CS model as a storage element with a defined volume and considering infiltration loss. In addition a simple conceptual model was built in order to simulate the behaviour of the buffers, reducing the complexity of the analysis and process of the simulations. The water that overflows from the buffer is then used as input in MIKE 11 hydrodynamic model.

Extra storage elements were then implemented under green areas in order to assess the effectiveness in reducing the flood problem inside the city and in the lower valley, limiting the capacity of the sewer and also of the buffers zone.

The results have shown an expected reduction of peak runoff discharge after the implementation of buffers under current scenario. Compared to the scenario that consider the absence of this option, there is of course a reduction in the flooding areas in the downstream of the city but not so effective as expected if it refers to all the flood plain. A major reduction it was obtained in the lower valley, where the buffers option showed a consistent influence in reducing peak runoff discharge under current and climate change scenarios, which increased when considering the combined impact of urbanization and climate change.

The presence of the storage elements, combined with the buffer option, contribute to have an additional reduction of the flooding area downstream the city but this reduction doesn’t exceed 1.5% considering the most extreme event. Nevertheless the results of this thesis have shown important benefits inside the city eliminating potential critical condition of RWA in some areas and limiting not only the initial volume of the buffer but also the water overflowing into the river.

Besides that, in order to have a clear determination of their effectiveness, further investigations must be conducted, including future planned buffers and adding new facilities which will store rainwater and / or infiltrate into the ground for a better water management in the Turnhout area.

Chapter 6

Conclusions and Recommendations

6.1 Foreword

The sustainable water resources management has many factors to be considered when it is implemented in a holistic way. Integration of the sewer system in a highly urbanized area with the neighboring river system is an emerging issue to be scrutinized under the impacts of further urban development and climate change. An integrated river flood model has been developed in order to understand the effect of urbanization and climate change on the river flow causing frequent urban flooding in the downstream of the city of Turnhout and to assess the effectiveness of Integrated Water Resource Management options to reduce this flooding problem. Within the scope of this study the potentials of the buffers and the introduction of storage element inside the city were selected from many other options proposed by the authorities in the region. To achieve these goals, the study started with the development of a hydrological model NAM for generating all the rainfall-runoff from both the gauged and ungauged subcatchments which were then used for urbanization and climate change impact investigation. Then a river flood model has been implemented combining two existing river models into one for the city located in the plateau of two rivers, Aa and Visbeek. The combination of CCI-HYDR Perturbation Tool (Ntegeka and Willems, 2008) and NAM was utilized to generate the time series of rainfall-runoff under future climate change scenarios for 2100s using the perturbed time series of rainfall and evapotranspiration. The urbanization scenarios are based on the correlations between rainfall and urban runoff as observed in the data used from the existing river flood models developed by Province of Antwerp.

6.2 Impact analysis on hydrological extremes

At the beginning of this study, all the hydrologic and hydraulic data are collected for the calibration and validation of the lumped conceptual hydrological model NAM. The same parameters for the gauged sub-catchment have been used to generate rainfall-runoff for the ungauged sub-catchment assuming the similar characteristics. Results of calibration and validation show that the model is capable of generating river flow from rainfall-runoff distribution to a quite acceptable level. The model's performance might also be considered valid because of its ability to predict more extreme conditions both peak and low flows. However, more attention may be needed for the calibration of low flows when low flow analysis is to be done using this NAM model.

Extreme value analysis has been conducted on hourly river peak flows generated by NAM both for the gauged and ungauged stations in order to construct a set of probabilistic flood events in the form of 'composite hydrographs'. A set of composite hydrographs was derived with different return periods from the QDF relationships for the runoffs. The results of the extreme value analysis for the current rainfall-runoff has been summarized to assess the impacts of urbanization and climate change both individually and combined.

Under current conditions, the average increase in peak runoff from the two subcatchments is 74% due to the impact of urbanization in the region. On the other hand, the individual impact of climate change on peak runoff from the same sub-catchment shows also a quite significant influence with an average increase in peak runoff by 23% and an average reduction of relatively lower peak runoff by 19% as compared to the current condition without considering the impact of urbanization. Finally, the combined impact of urbanization and climate change shows a very significant intensification of the peak runoff with the peak runoff that is expected to be doubled as compared to the current condition when no impacts are considered.

6.3 Impact analysis on flood events

The second part of this study focused on the implementation of a river flood model using MIKE 11 river modelling software to transform two existing IW-RS river models into one. This has been done to integrate the urban runoff from the city derived from the existing sewer system model (e.g. IW-CS) simulation which has already been used in the IW-RS models. A 'quasi-2D approach' was used for the implementation of the river flood model based on a 1D hydrodynamic model (MIKE 11). The floodplains were implemented as additional storage areas, which were represented by a small fictitious river called 'floodbranch' connected to the main river by one link channel and two identical cross sections with a trapezoidal shape. The calibrated and validated MIKE 11 river flood model was capable of producing flood with the similar water depth in all the floodplains along the river reaches as it can be observed in the results from IW-RS models. The model's performance appears also good with an R value of 0.93 and an NSE value of 0.83 for calibration period, while the values of 0.99 and 0.96 are obtained respectively for the validation period. The calibrated and validated MIKE 11 river flood model was then used for the scenario analysis.

Different composite hydrographs with return periods of 5, 10, 25, 50 and 100 years have been simulated for the current conditions while return periods of 50 and 100 years have been chosen for the climate change scenarios. The synthetic events have an advantage that these can be used for different scenario analysis using short simulation runs. In the simulations the water level results of the hydrodynamic model in the floodbranches were used to generate flood mapping using the pre-processing and post-processing procedures in MIKE 11 GIS. The visualization of floodplain water levels in a full 2D way was facilitated using a high resolution DEM in a GIS.

The results from the flood maps are then summarized to assess the impacts of urbanization and climate change both individually and combined on flood risks in Turnhout, more specifically in the lower valley. The increase in flooded area and volume of flooding water was the highest in the case of lower return periods when all the floodplains are taken into account.

This increase could be better understood if few other lower return periods are selected for the analysis. The impact analysis on the flooding in the lower valley showed that the flooded area increases by 50.5% due to urbanization for a flood with return period of 100 years in comparison to the scenario without any urban influence. The situation is also aggravated when the impact of climate change is considered, since the inundation area in the problematic areas increased by 38% for a flood with return period of 100 years when compared with the current condition under no impact. Moreover, it flooding areas could be even bigger if taking in account that the warmest scenario A1FI was not considered in the analysis. Actual trends in fact suggest that it is an increasingly plausible scenario (Raupach et al., 2007).

The inundation area is further extended by 67% for a flood with return period of 100 years when compared with the current condition under no impact. An amplification of the flooded areas and volume of flooding water was found, in all floodplains and also in the lower valley, from the combination of urbanization and climate changes, when compared with individual effect of urbanization or climate change, which is one of the most significant findings of this study.

The increasing proportion of paved surface within the urban fabric transforms the hydrographs so that they are characterized by high flow peaks and fast response to even moderate rainfall events (Semadeni-Davies et al., 2008). With further investigations in other study areas, this urban runoff can be considered as a fourth component of total rainfall-runoff discharge to confirm the results of present study.

6.4 Impact analysis of IWRM options

An attempt has been made to assess the effectiveness of integrated water resources management options to reduce the flooding in the lower valley. The potentiality of buffer zone and the introduction of storage element inside the city were considered in this study both separately and combined and flood maps were generated for return period of 5, 10, 25, 50 and 100 years in order to determine the reducing in flooding areas. The buffer, represented as storage element, were planned to be positioned at the end of the new rain water collector system. The overflows from these buffers, within the allowable limit, were planned to be discharged into the rivers. A decrease of 38% in the flooded area of the lower valley for return period of 100 years was observed after the implementation of the six buffer planned in the immediate future. The percentage reduction under climate change scenarios (35%) is lower than that under present condition after implementation of the buffers, because the high scenario causes an inevitably increasing of the runoff discharges that the buffers could not contain. Extreme value analysis has been conducted also for the buffer option. Results demonstrated a decreasing of the peak runoff from the two subcatchments of about 32.5% on average under current condition.

The introduction of storage element inside the city has not lead to great improvements in reducing the flooding problems. The small reduction is justified by the fact that the potentially of this storage elements is principally demonstrated observing results from the sewer system model of the city. Infact the analysis of the results reveal that the critical condition of the pipes in some areas and also the flooding in some nodes are reduced during peak events.

Results of combined effects of both the mentioned options were very similar to the ones observed for the buffers option alone. The latter indicates that means the two alternatives together have not a bigger impact on reducing in the flooding areas downstream of the city. Nevertheless the two options might serve, in a holistic way, to improve water management for the city of Turnhout, i.e. by including further investigations to ulterior buffers, which will be planned in long term and by extending the number of storage elements to other places such as car parking spaces or streets. Besides that, the contribution on reducing flooded areas in the lower valley must be investigated together with other adaptive alternatives in order to come up with a sustainable flood management option for the city.

6.5 Recommendation

A number of limitations have been identified during this study on the assessment of urbanization and climate change impacts on the urban flood risks. There are three rainfall measuring stations in Turnhout region, but data for only Turnhout station is made available in the databank of VMM, i.e. HYDRONET. Rainfall data from other stations could better represent the rainfall-runoff over the whole sub-catchment.

The floodplain model has been developed based on the information on storage areas from the existing two river flood 'IW-RS' models. The existing models were developed for two rivers covering two different sub-catchments. The model for Aa sub-catchment was calibrated and validated with the observed river flow at the Turnhout station. On the other hand, the model for the Visbeek sub-catchment was calibrated and validated with the measured flow at a point downstream of the confluence of Aa and Visbeek rivers during the measurement campaign. Hence, the combination of the two models into one has been a difficult task during the present study, which could have been overcome if the observed river flow data at Poederlee (Station ID: K10_061) and other catchment information were available. These information are also very necessary for any further extension of the NAM and MIKE 11 river flood model to the most vulnerable areas according to the ROG for 2005 (Figure 3.14). Extended study must be carried out to overcome these limitations.

However, implementation and calibration of the MIKE 11 river model was done using short time series for few events because of the long computational time required to simulate events with long time series using InfoWorks RS & CS models. It is recommended to improve the model considering other events with long time series. The latter may be achieved by making use of semi-lumped or lumped conceptual models which have been found to perform well for land use studies (Breuer et al., 2009).

Combined assessment of other adaptive measures planned by the city water resource managers must be performed to understand the relative effectiveness in solving the flooding problem.

More importantly, water quantity problems should be studied in an integrated way together with water quality problems, as new principles of integrated river basin management requirement.

River quality deterioration is affected by different sources of water and pollution from the city. To overcome these problems, an investigation could be made considering the impact of these sources of pollution on the receiving surface water. The integration of all the proposed adaptive measures combined with water quality and coordination between upper and lower valleys are required in the IWRM in order to choose an efficient adaptive measure leading to a sustainable water resources management.

APPENDIX-A

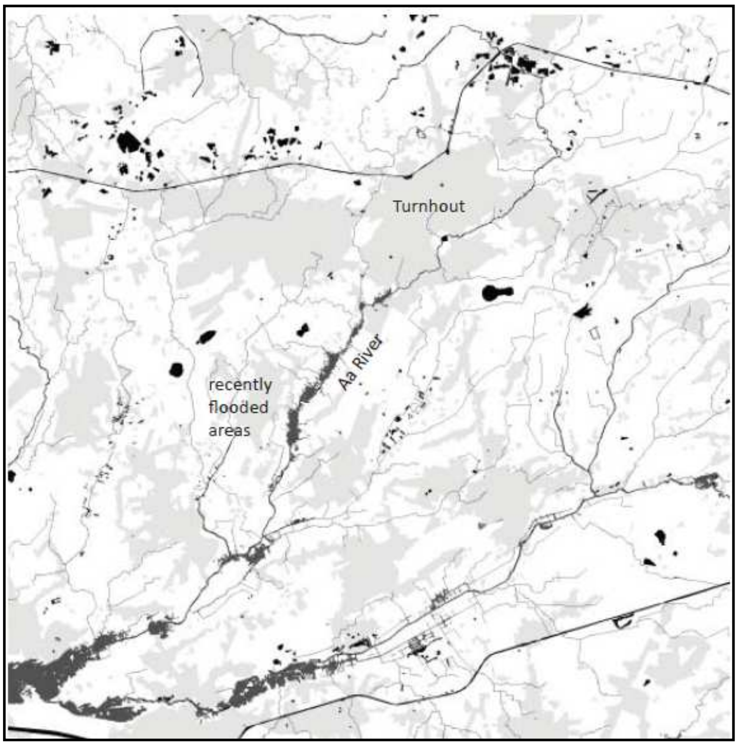
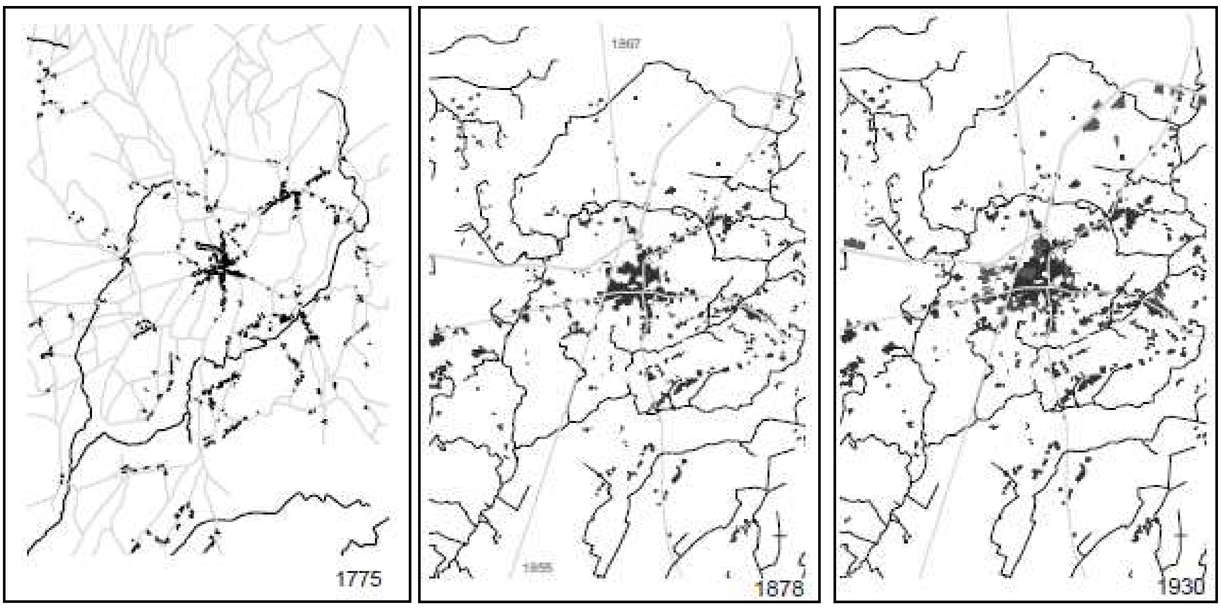


Figure A.1: problematic situation in the downstream region of Turnhout with rivers and lakes (black), the urban areas (light grey) and the recently flooded areas (dark grey).



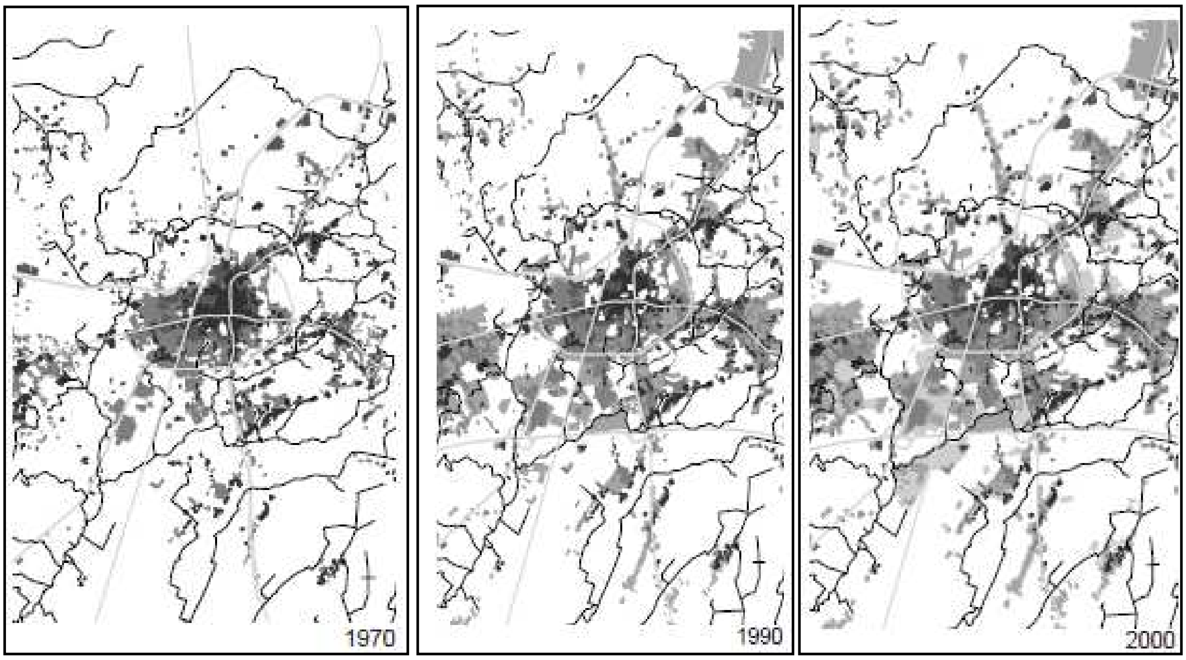


Figure A.2: evolution of paved areas in the city of Turnhout throught the years (adapted from Nolf, 2010).

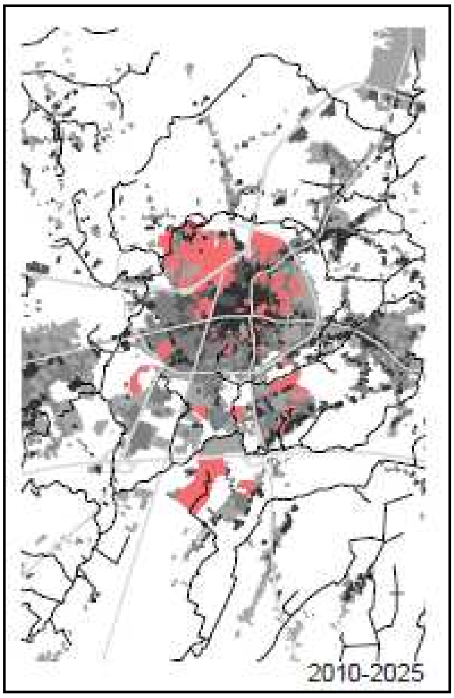


Figure A.3: the predicted areas of densification (red areas) in Turnhout by 2010-2025.

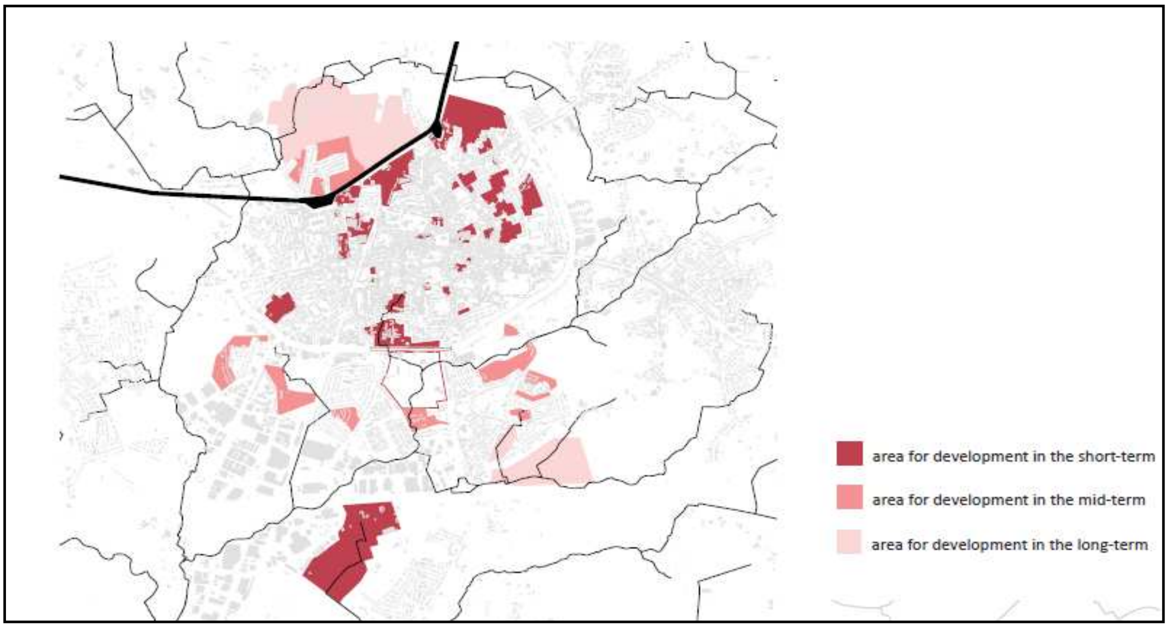


Figure A.4: existing development projects in the city of Turnhout.

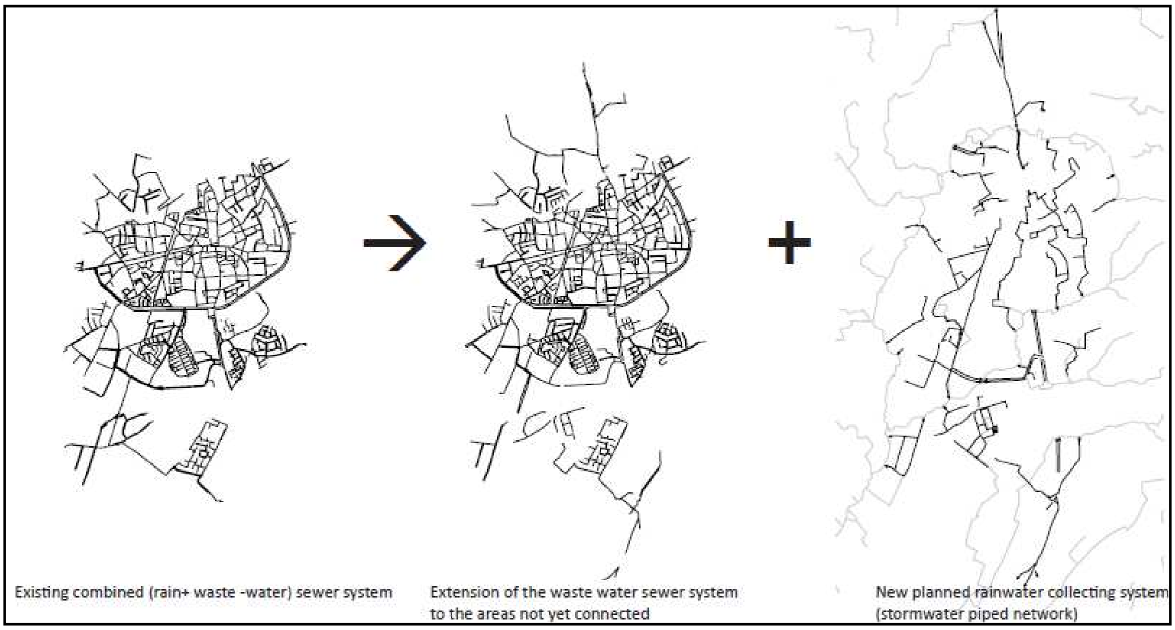


Figure A.5: existing combined (rain+wastewater) sewer system, extension of the system and the planned rainwater collecting system for the city (adapted from Nolf, 2010).

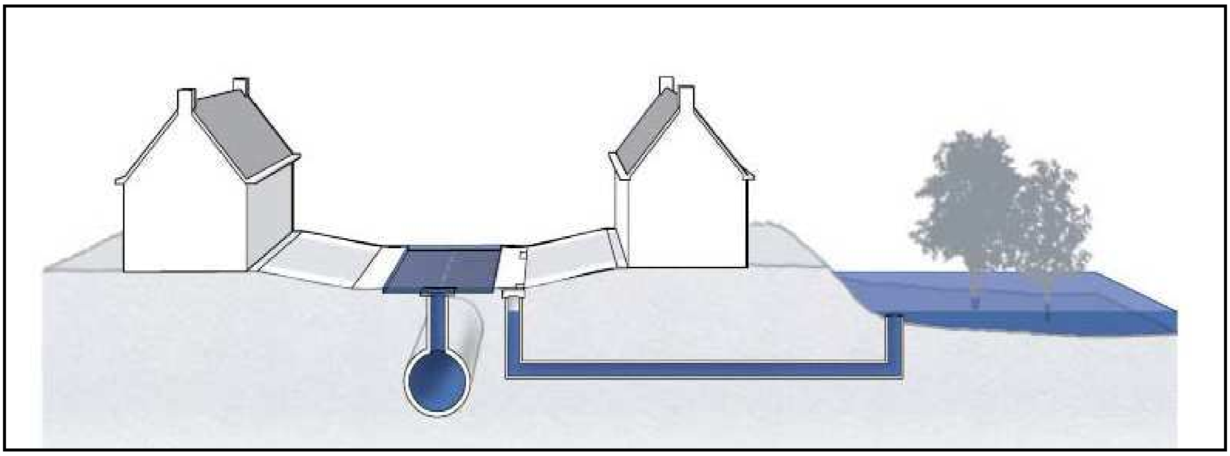


Figure A.6: schematic illustration of water retention system within the urban fabric (source: Willems, 2009b).

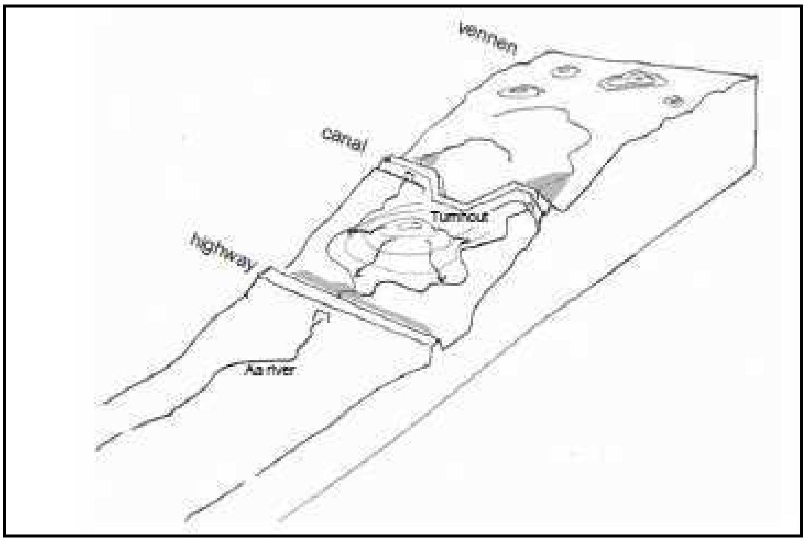


Figure A.7: topographical bottlenecks and infrastructural barriers as retention devices ‘dammed creeks’ (adapted from Nolf, 2010).

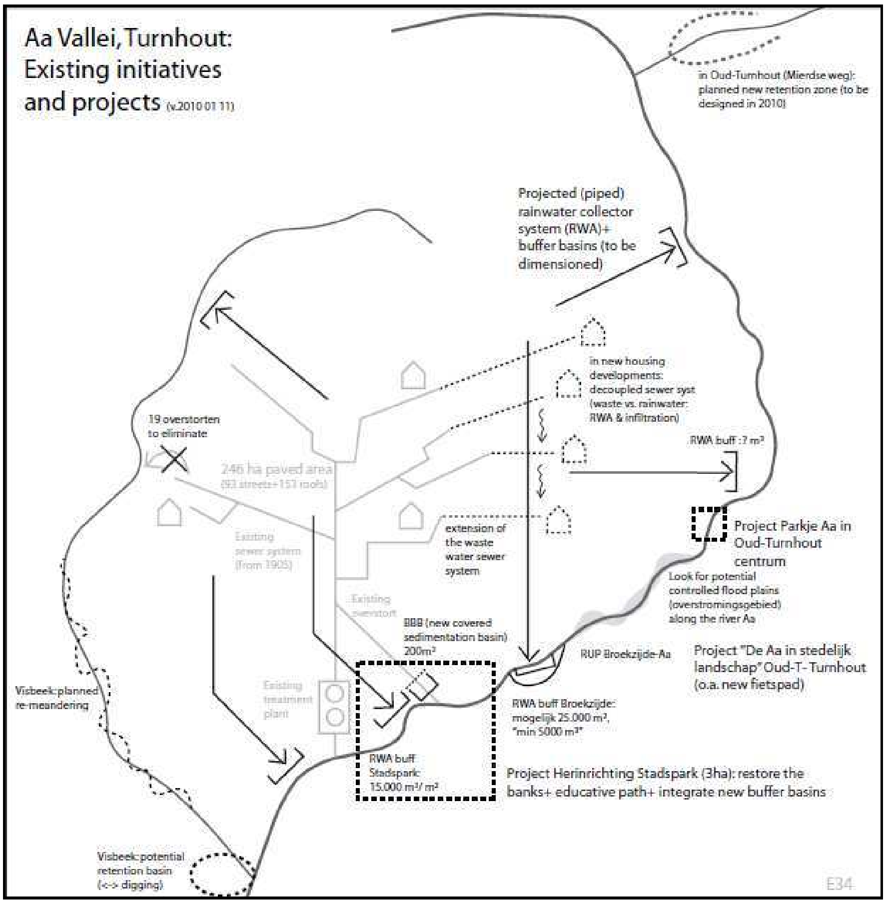


Figure A.8: the proposed IWRM actions by the city and river authorities (adapted from Nolf, 2010).

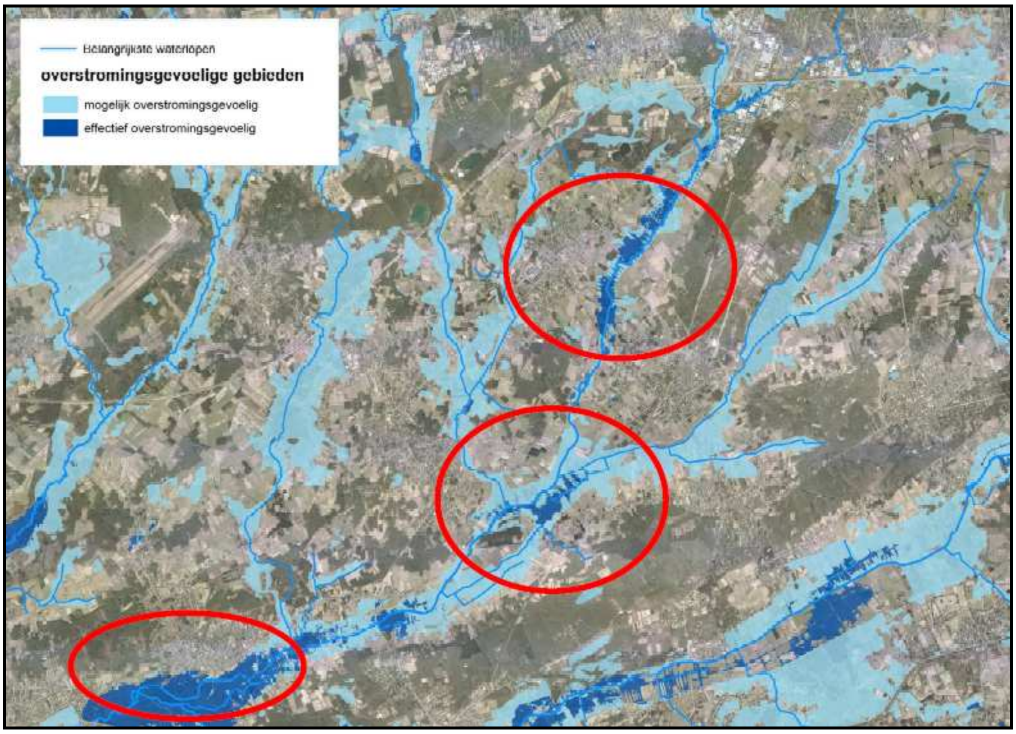


Figure A.9: the potential (light blue) and effective (dark blue) flood prone areas (red circle) along the lower Aa valley (adapted from Abrams, 2009).

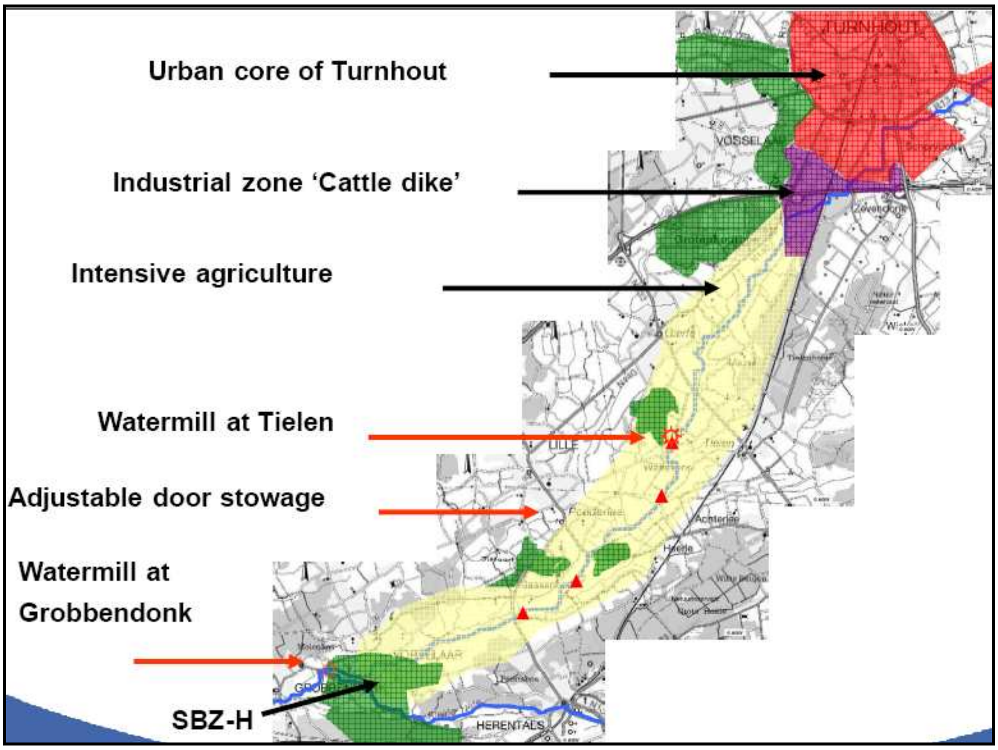


Figure A.10: the location of existing context along the lower Aa valley (adapted from Abrams, 2009).

APPENDIX-B

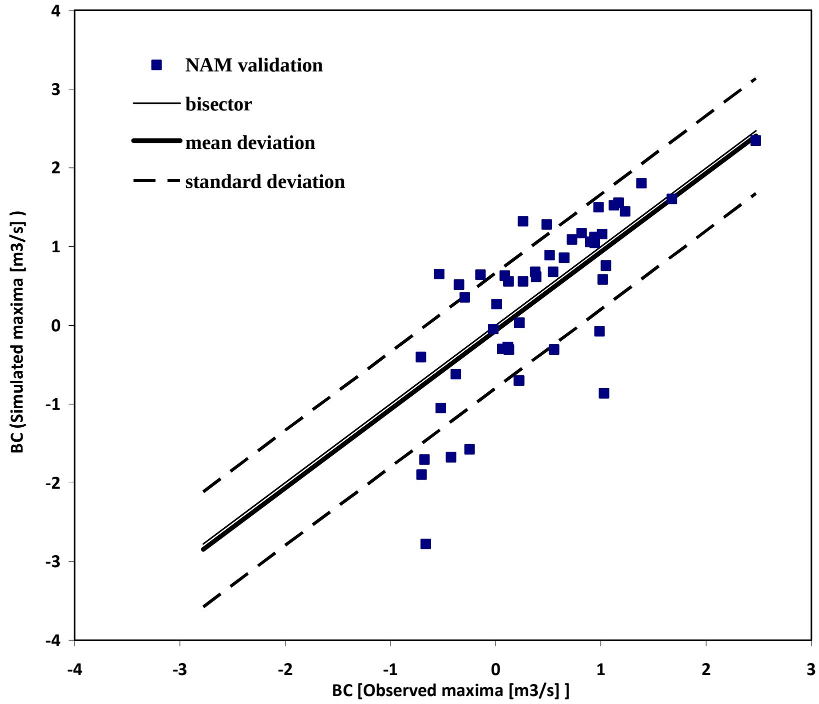


Figure B.1: graphical evaluation of peak flow maxima for validation period.

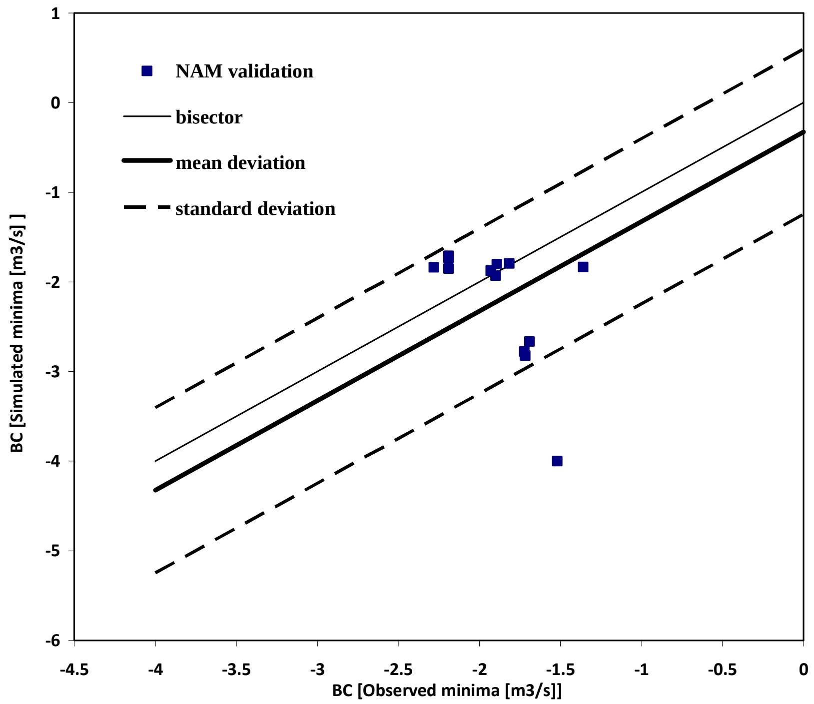


Figure B.2: graphical evaluation of peak flow minima for validation period.

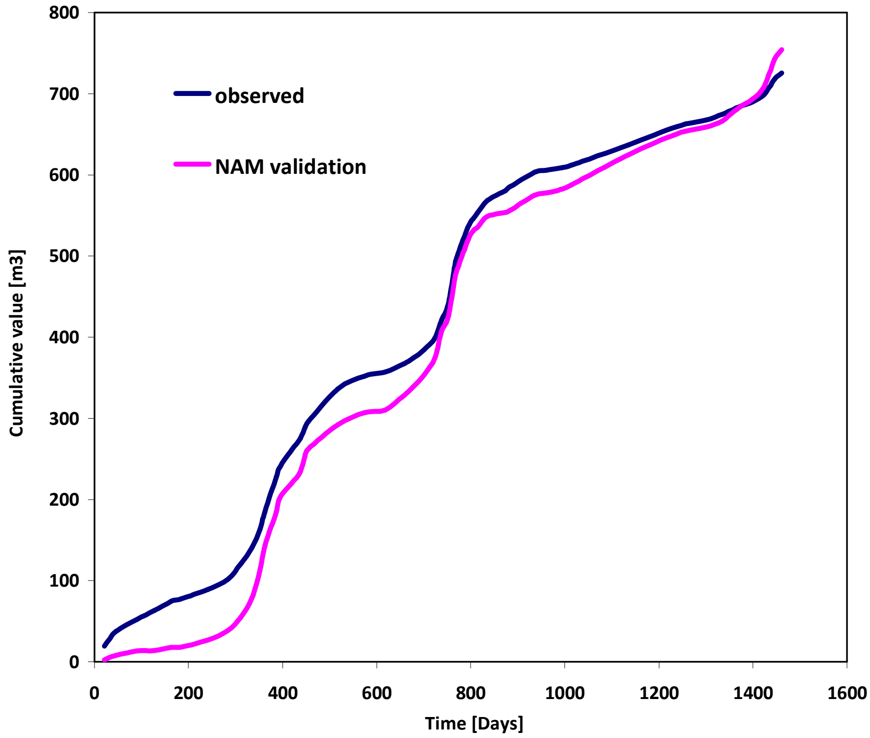


Figure B.3: evaluation of cumulative volume for validation period.

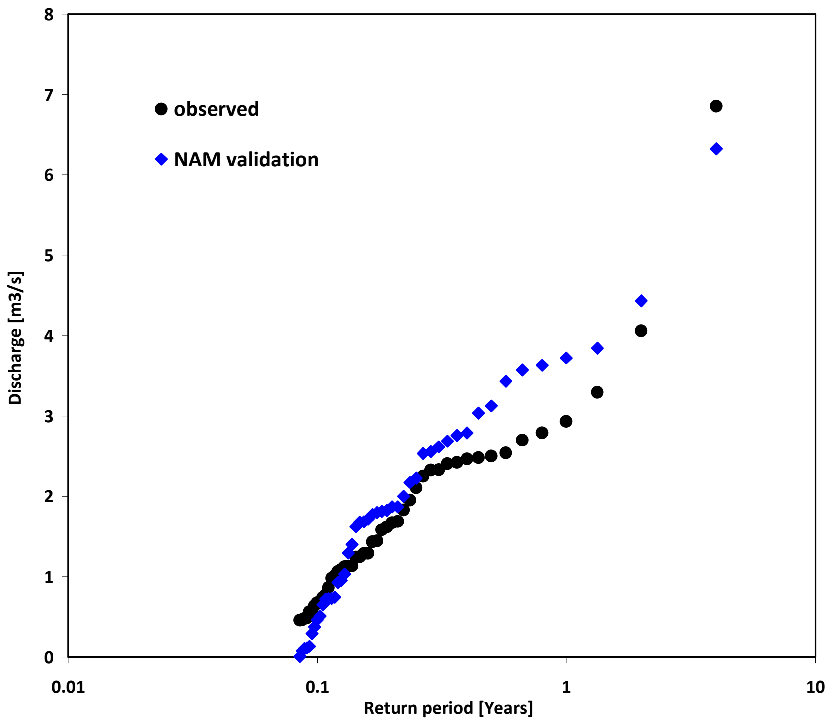


Figure B.4: graphical evaluation of peak flow extreme value distribution for validation period.

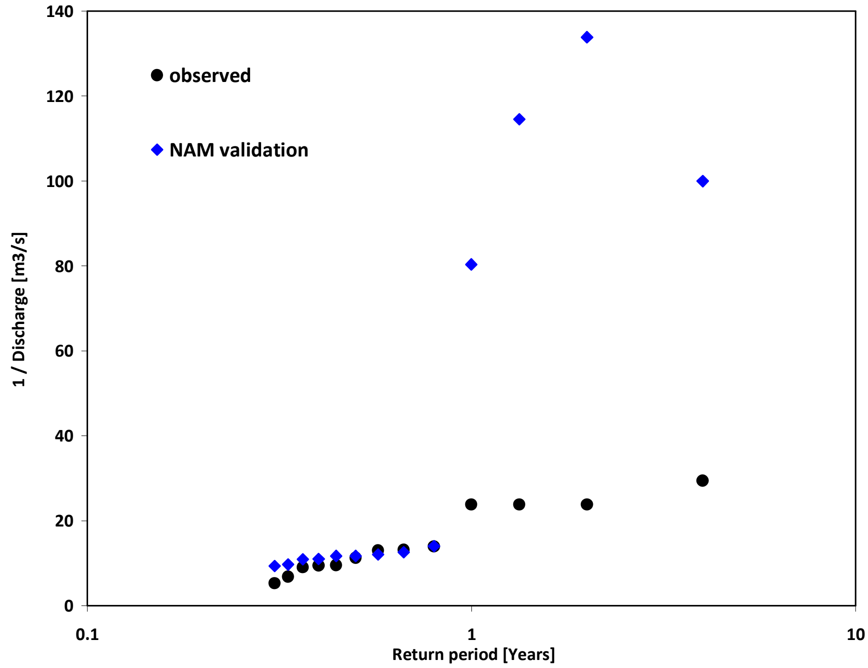


Figure B.5: graphical evaluation of low flow extreme value distribution for validation period.

APPENDIX-C

Branch Name	Catchment	Area [km2]	Upstream chainage [m]	Downstream chainage [m]
AA2	AA-In2	1.8	350	1021
AA2	AA-In3	1.91	1586	2286
AA2	AA-In4	2.51	3148	4331
AA2	AA-In5	2.34	4196	4784
AA2	AA-In6	2.42	5140	6100
AA2	AA-In7	0.81	6551	7403
AA2	AA-In8	7.21	6659	6659
AA2	AA-In9	3.09	7541	7541
AA2	AA-In10	1.24	8069	8069
AA2	AA-In11	0.38	8029	8768
AA2	AA-In12	0.81	8384	9064
AA2	AA-In13	0.38	9019	9627
AA2	AA-In14	0.6	9850	10518
AA2	AA-In15	1.2	10616	10616
AA2	AA-In16	0.61	11070	11070
AA2	AA-In17	1.04	11258	12412
AA2	AA-In18	0.78	12730	12730
AA2	AA-In19	0.37	12928	12928
AA2	AA-In20	0.58	12975	14330
AA2	AA-In21	6.43	14572	14572
AA2	AA-In23	0.65	14684	15557
AA2	AA-In24	5.97	15688	15688
AA2	AA-In25	0.35	15798	16326
AA2	AA-In26	0.84	16527	17145
AA2	Bossenloop	1.93	16445	16445
AA2	AA-In1	0.56	0	0

Table C.1: the NAM rainfall-runoff distribution along the Aa river.

Branch Name	Catchment	Area [km2]	Upstream chainage [m]	Downstream chainage [m]
LIE	LIE-In1	2.08	0	0
LIE	LIE-In2	0.86	101	1217
LIE	LIE-In3	0.57	1313	1664
LIE	LIE-In4	1.6	1946	1946
LIE	LIE-In5	0.7	2398	2398
LIE	LIE-In6	0.52	2698	2698
LIE	LIE-In7	1.31	3139	3139

Table C.2: the NAM rainfall-runoff distribution along Liermansloop.

Branch Name	Catchment	Area [km2]	Upstream chainage [m]	Downstream chainage [m]
Visbeek	VB-lat1	4.02	5986	10677
Visbeek	VB-lat2	3.4	1736	5488
Visbeek	VB-lat3	1.57	155	1485
Zijtak-Visbeek	Zijtak-Visbeek2	0.26	139	355
Rondvenloop	Rondvenloop2	0.21	189	418
Visbeek	achterstoktloop	2.77	5302	5302
Zijtak-Visbeek	Zijtak-Visbeek	1.52	0	0
Rondvenloop	Rondvenloop	1.82	0	0
AA1	AA-lat1	1.02	0	1113

Table C.3: the NAM rainfall-runoff distribution along the Visbeek river and its tributaries.

Name of the urban point source	Connected to the Branch	Chainage at connection [m]
OV_everdongenlaan1 (OV1)	AA2	764.85
OV_Everdongenlaan2 (OV2)	AA2	13390
OV_Everdongenlaan3 (OV3)	AA2	15791.94
OV_parkring (OVP)	AA2	15899.79
urb_sb (UB1)	Visbeek	5302.5
urb_rh (UB2)	Visbeek	5871.32
urb_asw_zv (UB3)	Zijtak-Visbeek	223.04
urb_swm (UB4)	Visbeek	4972.66
urb_E34o (UB5)	Visbeek	9918.13
urb_E34 (UB6)	Visbeek	10143.86
urb_asw_vb (UB7)	Visbeek	6958.64

Table C.4: distribution of urban runoffs from the city along the rivers.

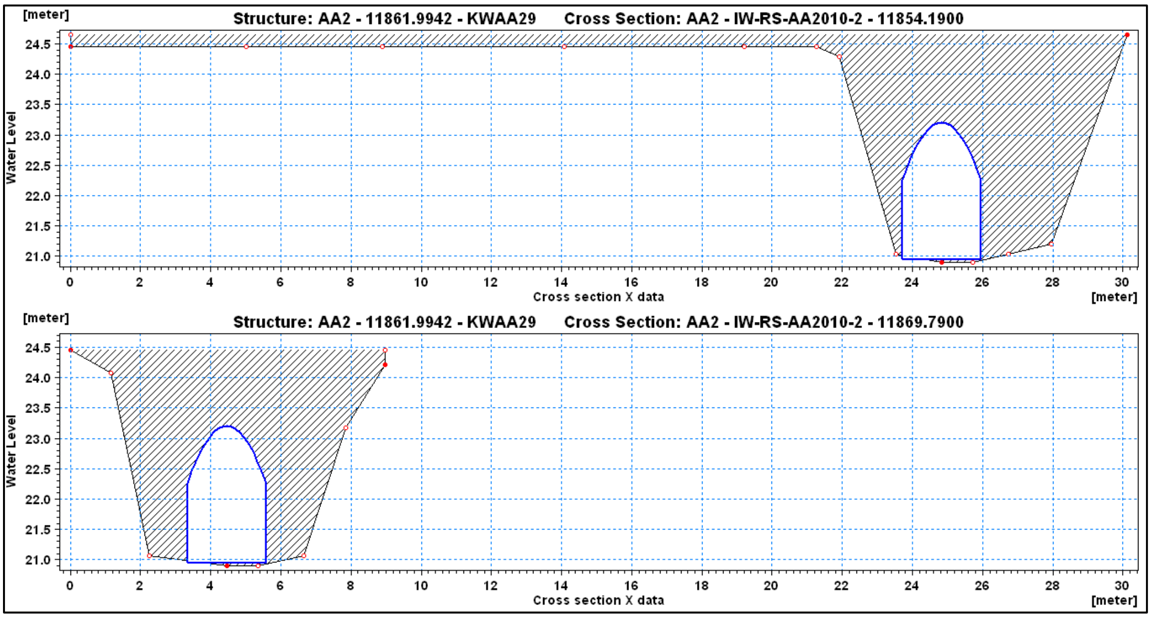


Figure C.1: sprung arch bridge in IW-RS defined as rectangular culvert in MIKE 11 model with the irregular level-width table at chainage 11862 m of Aa (Cat-2).

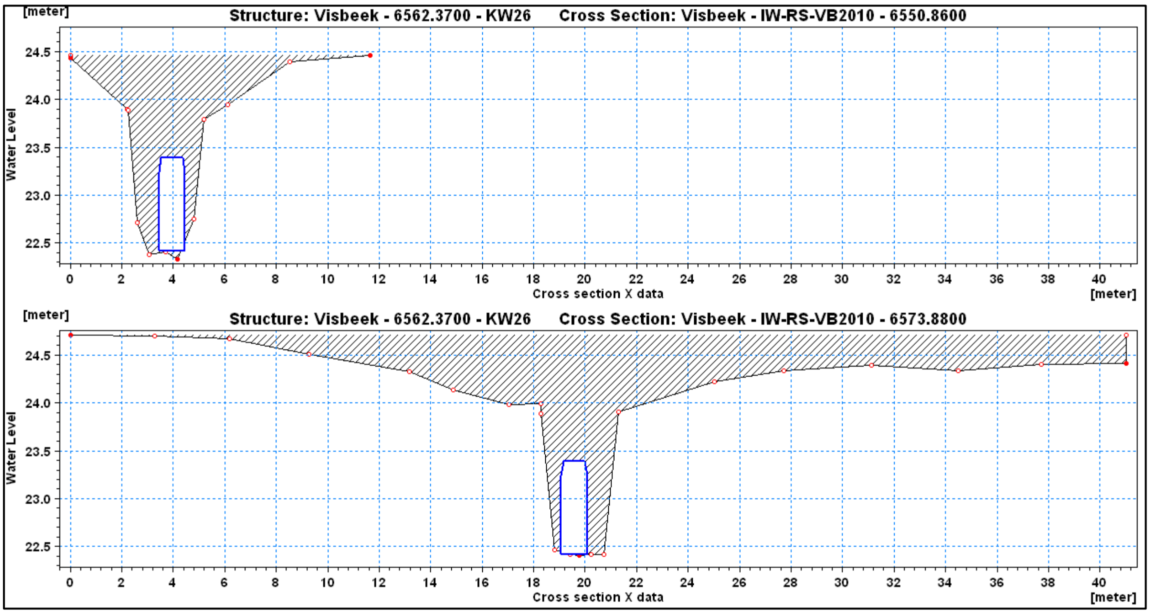


Figure C.2: sprung arch bridge in IW-RS defined as rectangular culvert in MIKE 11 model with the irregular level-width table at chainage 6562.37 m of Visbeek.

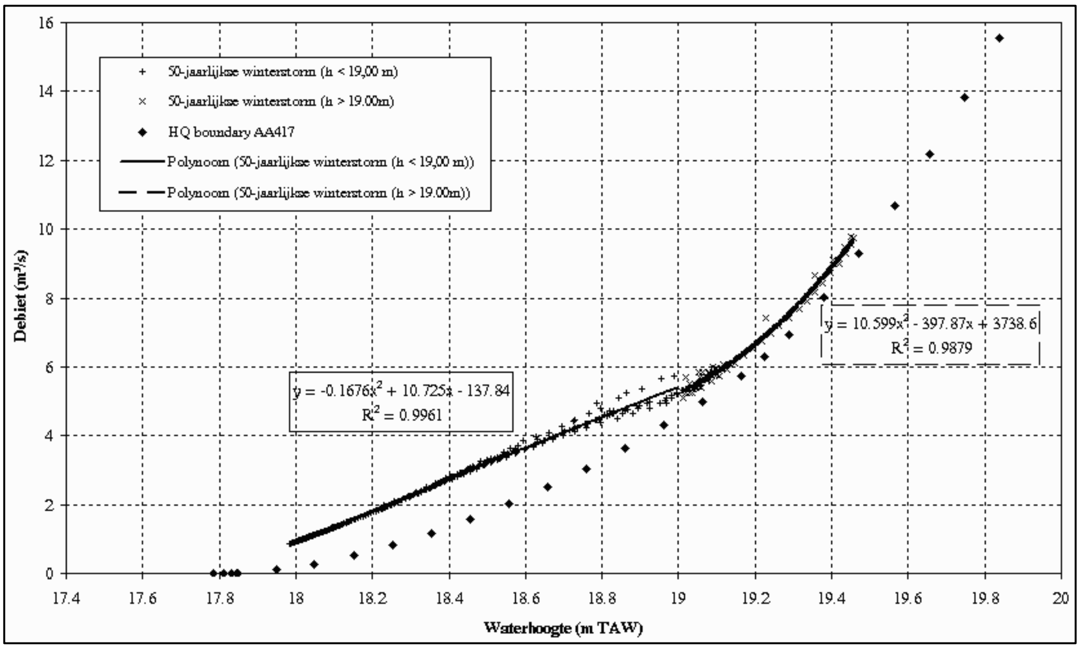


Figure C.3: rating curve for the Turnhout station on the Aa river (source: Province of Antwerp, 2004).

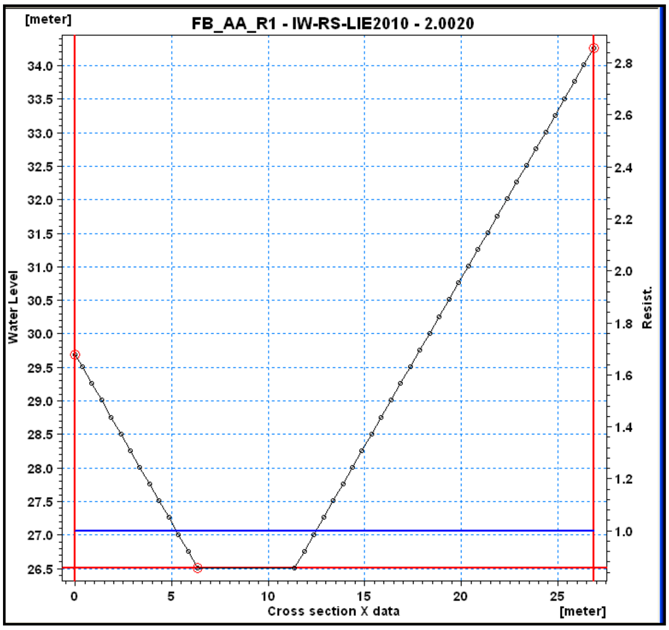


Figure C.4: an imaginary cross-section created for the floodbranch FB_AA_R1 on the right floodplain of AA2 river.

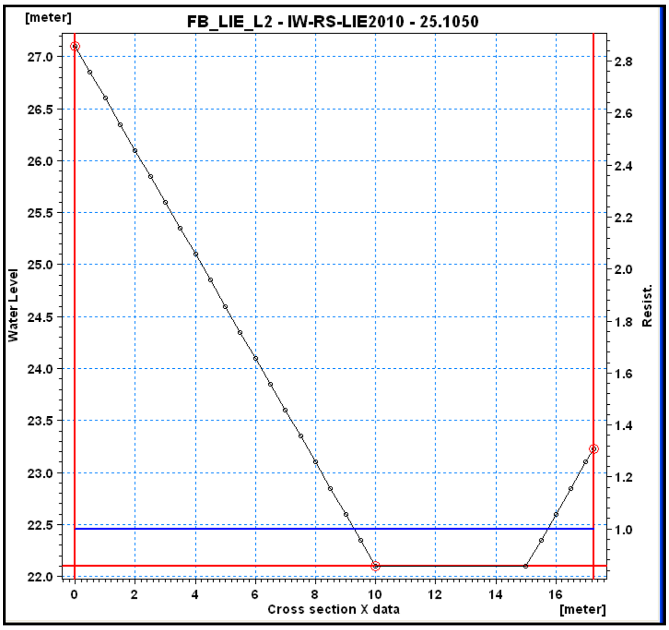


Figure C.5: an imaginary cross-section for the floodbranch FB_LIE_L2 on the left floodplain of Liermansloop.

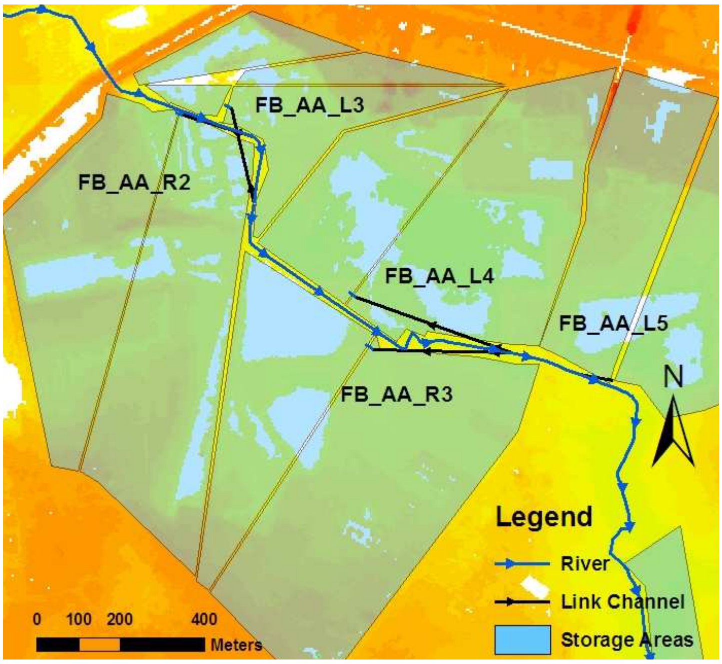


Figure C.6: configuration of the floodbranches in MIKE 11 GIS between chainages 5000- 7500 m along AA2 river.

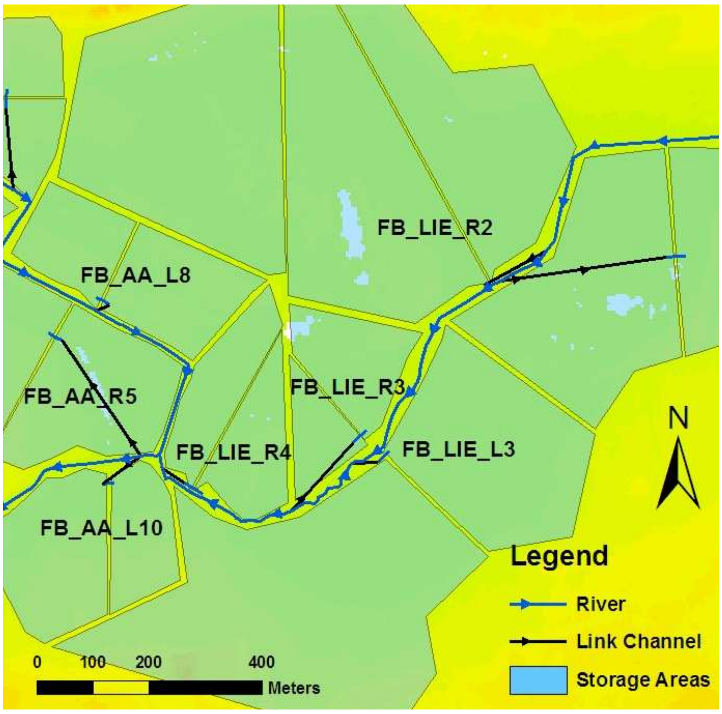


Figure C.7: configuration of the floodbranches near the confluence of Aa (category-2) river and Liermansloop.

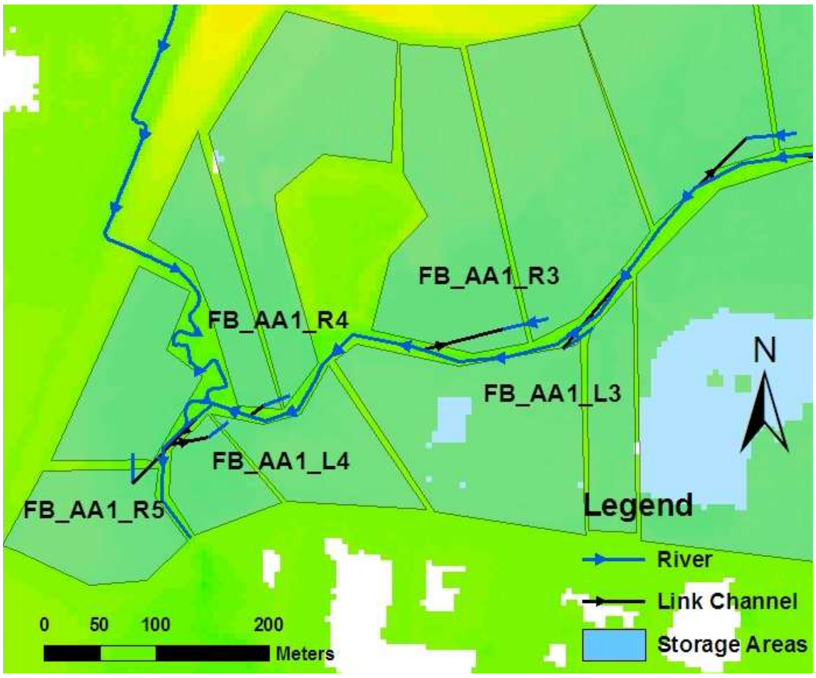


Figure C.8: configuration of the floodbranches near the confluence of Aa (category-1) and Visbeek rivers.

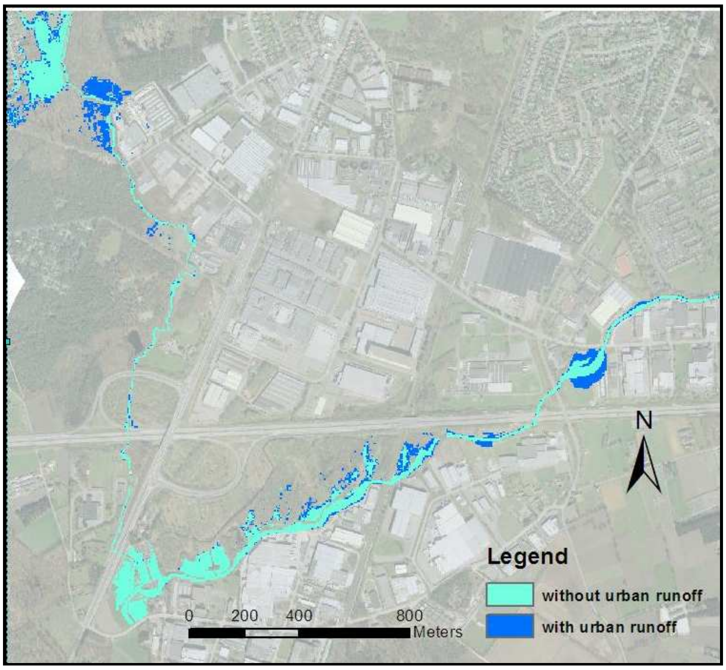


Figure C.9: flood maps for the region near the confluence of Aa and Visbeek rivers for return period of 10 years with and without urban runoff.

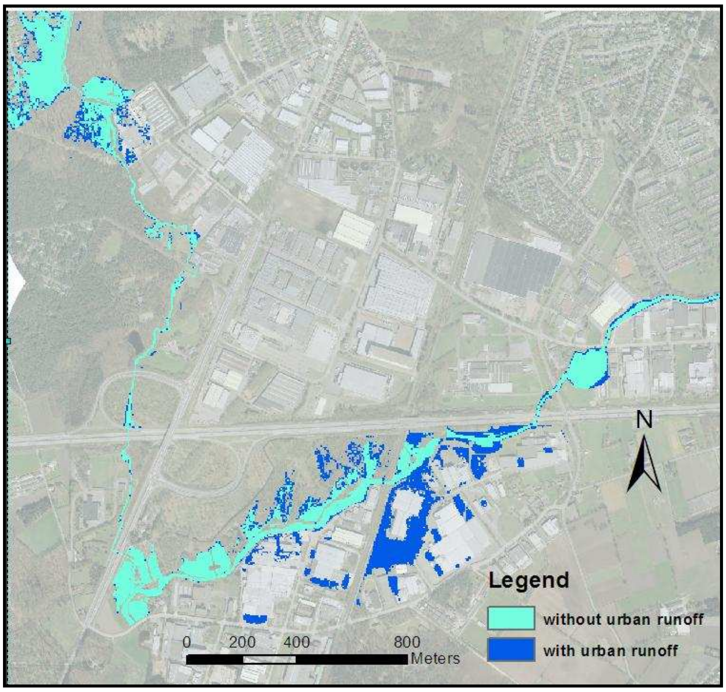


Figure C.10: flood maps for the region near the confluence of Aa and Visbeek rivers for return period of 100 years with and without urban runoff.

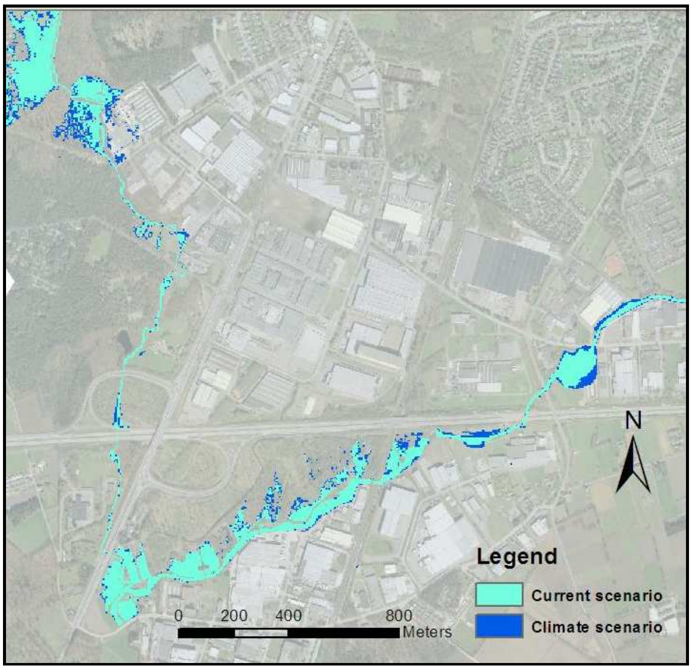


Figure C.11: the extent of flooded areas near the confluence of Aa and Visbeek rivers due to climate change for return period of 50 years without considering urban runoff.

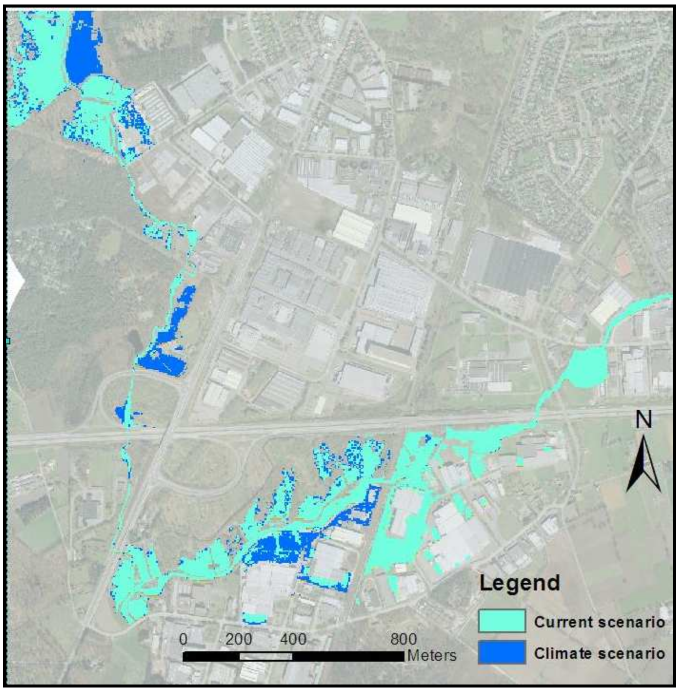


Figure C.12: the extent of flooded areas near the confluence of Aa and Visbeek rivers due to climate change for return period of 100 years considering urban runoff.

BIBLIOGRAPHY

Arnold, C. L. and Gibbons C.J., 1996. Impervious surface coverage - The emergence of a key environmental indicator. Journal of the American Planning Association 62(2): 243-258.

Abrams, B., 2009. Integrated Water Management: Nete basin - Valley of the Aa. SUDEM - November 20, 2009. (In Dutch)

Bates, B.C., Kundzewicz, Z.W., Wu, S. and Palutikof, J.P., Eds., 2008. Climate Change and Water. Technical Paper of the Intergovernmental Panel on Climate Change, IPCC Secretariat, Geneva, : 210 pp.

Bauwens, W., 2009. Surface Water Hydrology, 1st year lecture notes, Interuniversity Programme in Water Resources Engineering, K.U.Leuven and V.U.B, Belgium.

Belgroma, 2002. Final Report, Surface Water Quantity Modeling of the Kleine Nete and Aa. Ministry of the Flemish Community, AMINAL, Water Division.

Booij, M. J. 2005. "Impact of climate change on river flooding assessed with different spatial model resolutions." Journal of Hydrology 303(1-4): 176-198.

Boukhris, O., Baguis, P., Ntegeka, V., Roulin, E., Willems, P., Demarée, G., 2008. Climate change impact on hydrological extremes along rivers and urban drainage systems. I. Literature review, Final report, K.U. Leuven – Hydraulics Section & Royal Meteorological Institute of Belgium, May 2008.

Bronstert, A., Niehoff, D. and Burger, G., 2002. Effects of climate and land-use change on storm runoff generation: present knowledge and modelling capabilities. Hydrological Processes 16(2): 509-529.

Carter, T. R., 2007. General guidelines on the use of scenario data for climate impact and adaptation assessment. Task Group on Data and Scenario Support for Impact and Climate Assessment (TGICA), Intergovernmental Panel on Climate Change.

Chang, H. J., 2007. Comparative streamflow characteristics in urbanizing basins in the Portland Metropolitan Area, Oregon, USA. Hydrological Processes 21(2): 211-222.

De Roo, A., Odijk, M. et al., 2001. Assessing the effects of land use changes on floods in the Meuse and Oder catchment. Physics and Chemistry of the Earth Part B-Hydrology Oceans and Atmosphere 26(7-8): 593-599.

DHI, 2005, MIKE 11 GIS, Floodplain mapping and analysis, User Guide, DHI, Hørsholm, Denmark.

DHI, 2008, MIKE 11 A modelling system for rivers and channels, Reference and User Manual, DHI Water & Environment, Hørsholm, Denmark.

Franczyk, J. and Chang, H. 2009. The effects of climate change and urbanization on the runoff of the Rock Creek basin in the Portland metropolitan area, Oregon, USA. Hydrological Processes. 23: 805-815.

Georgiyevsky, V. and Shiklomanov I., 2003. In World Water Resources at the Beginning of the 21st Century. Cambridge University Press: Cambridge.

Hirsch, R. M., Walker, J.F., Day, J.C., Kallio, R., 1990. The influence of man on hydrologic systems. In: Wolman, W.G., Riggs, H.C. (Eds.), Surface Water Hydrology: The Geology of America O-1(Geological Society of America, Boulder, CO, USA).

Hollis, G.G., 1975. The effects of urbanization on floods of different recurrence intervals. Water Resour. Res. 11, 431–435.

Huntington, T. G., 2006. Evidence for intensification of the global water cycle: Review and synthesis. Journal of Hydrology 319(1-4): 83-95.

IPCC, 2001. The scientific basis. Contribution of Working Group I to the Third Assessment Report of the Intergovernmental Panel on Climate Change. GRID-Arendal.

IPCC, 2007a. Intergovernmental Panel on Climate Change (IPCC), 2007. Fourth Assessment Report (FAR) 2007.

Agarwal, A., Angeles, M.S. et al., 2000. Integrated water resources management. Global water partnership technical advisory committee background papers, No. 4.

Jean-Pascal, V. Y., 2004. Impacts of climate change in Belgium. Report commissioned by Greenpeace.

Jiang, T., Chen, Y. et al., 2007. Comparison of hydrological impacts of climate change simulated by six hydrological models in the Dongjiang Basin, South China. Journal of hydrology 336(3-4): 316-333.

Jonch-Clausen, T., 2004. Integrated Water Resources Management (IWRM) water efficiency plans by 2005: why, what and how? Global water partnership, Sweden.

Karimi, H.A. and Houston, B.H., 1996. Evaluating strategies for integrating environmental model with GIS: Current trends and future needs, Computers, Environment and Urban Systems, 20, 413-425.

Konrad, C.P. & Booth, D.B., 2002. Hydrologic trends associated with urban development for selected streams in the Puget Sound Basin, Western Washington. U.S. Geological Survey, Water-Resources Investigations Report 02-4040. Tacoma, WA, USA.

Leopold, L.B., 1994. A View of the River. Harvard University Press, Cambridge, MA, USA.

Liu, Y. B., Gebremeskel, S. et al., 2006. Predicting storm runoff from different land-use classes using a geographical information system-based distributed model. Hydrological Processes 20(3): 533-548.

Madsen, H., 2000. Automatic calibration of a conceptual rainfall-runoff model using multiple objectives. Journal of Hydrology 235(3-4): 276-288.

Taye, M. T., 2009. Hydrological modelling of climate change impact on selected catchments of the Nile River basin. Master thesis, K. U. Leuven and V.U.B., Belgium.

MGV, 2003. Risk zones for flooding (in Dutch). Note by Ministry for the Flemish community, Brussels, 8pp.

Nakićenović N., Alcamo J., Davis G., de Vries B., et al., 2000. Emissions scenarios. A Special Report of Working Group III of the Intergovernmental Panel on Climate Change. Cambridge University Press.

Nash J. E. and Sutcliffe J.V., 1970. River flow forecasting through conceptual models. Part I: a discussion of principles. Journal of Hydrology 10:282 - 290.

Nolf, C., 2010. Case study I: Problematic of water retention in the urban area of Turnhout. Water Research in Flemish Urbanized Areas. A synthesis report, ASRO/OSA, K.U.Leuven, Belgium.

Ntegeka, V., 2006. An empirical statistical approach to climate change impact on hydroclimatic extremes. Master thesis, K. U. Leuven and V.U.B., Belgium.

Ntegeka, V. and Willems, P., 2008. CCI-HYDR Perturbation Tool: a climate change tool for generating perturbed time series for the Belgian climate. K.U.Leuven, & Royal Meteorological Institute of Belgium, Manual, version December 2008, 1-7 pp.

Paul, M.J., Meyer, J.L., 2001. Streams in the urban landscape. Ann. Rev. Ecol. Syst. 32, 333–365.

Phoon, S.Y., Shamseldin, A.Y., Vairavamoorthy, K., 2004. Assessing impacts of climate change on Lake Victoria Basin, Africa. 30th WEDC International Conference, Vientiane, Lao PDR, UK.

Poelmans, L., Van Rompaey, A., 2009. Detecting and modelling spatial patterns of urban sprawl in the Flanders–Brussels region (Belgium). Landsc. Urban Plann. 93, 10–19.

Poelmans, L., Van Rompaey, A., 2010. Complexity and performance of urban expansion models. Comput. Environ. Urban Syst. 34, 17–27.

Poelmans, L., Van Rompaey, A., Batelaan, O., 2010. Coupling urban expansion models and hydrological models: How important are spatial patterns? Land use policy, 27, 965-975.

Province of Antwerp, 2004. Hydrological and hydraulic study of the basin of the Aa No 9 in Turnhout and Oud-Turnhout, Final Report Part 1, Part 2 & Part 3 (in Dutch). Water Service, Province of Antwerp, Koningin Elisabethlei 22, 2018, Antwerp, Belgium.

Province of Antwerp, 2006. Hydrologic and hydraulic study of the basin Visbeek No 9.13, Final Report Part 1, Part 2 & part 3 (in Dutch). Water Service, Province of Antwerp, Koningin Elisabethlei 22, 2018, Antwerp, Belgium.

Refsgaard, J.C., 1996. Terminology, modelling protocol and classification of hydrological models. In: Abbott, M.B., Refsgaard, J.C. (Eds.), Distributed Hydrological Modelling. Kluwer, Dordrecht, pp. 17–39.

Rose, S. and Peters, N. E., 2001. Effects of urbanization on streamflow in the Atlanta area (Georgia, USA): a comparative hydrological approach. Hydrological Processes 15(8): 1441-1457.

Savenije, H. H. G., & Zaag, P.V.D., 2008. Integrated water resources management: Concepts and issues. Physics and Chemistry of the Earth 33: 290-297.

Schueler, T., 1994. The importance of imperviousness. Watershed Protect. Tech. 1 (3), 100–111.

Semadeni-Davies, A., Hernebring, C. et al., 2008. The impacts of climate change and urbanisation on drainage in Helsingborg, Sweden: Combined sewer system. Journal of Hydrology 350(1-2): 100-113.

Semadeni-Davies, A., Hernebring, C. et al., 2008. The impacts of climate change and urbanisation on drainage in Helsingborg, Sweden: Suburban stormwater. Journal of Hydrology350(1-2): 114-125.

Silberstein, R. P., 2006. Hydrological models are so good, do we still need data? Environmental modelling & software 21(9): 1340 - 1352.

Timbe, L., 2007. River flooding analysis using quasi-2D hydraulic modelling and geospatial data. PhD thesis, K.U.Leuven, Faculty of Engineering, Leuven, Belgium.

Tu, J., 2009. Combined impact of climate and land use changes on streamflow and water quality in eastern Massachusetts, USA. Journal of Hydrology 379(3-4): 268-283.

Vaes, G., 1999 . The influence of rainfall and model simplification on combined sewer system design. PhD thesis, Katholieke University of Leuven, Belgium.

Vaes, G., Willems, P. and Berlamont, J., 2000. Selecetion and composition of representative hydrographs for the river design calculations. In: ERB 2000, Int. Coference on 'Monitoring and modelling catchment water quantity and quality' (ERB), 27-29 September 2000 Gent: 53-55.

Villazón, M.F. & Willems, P., 2009. The importance of spill conceptualizations and head loss coefficients in a quasi two-dimensional approach for river inundation modelling. Taylor & Francis Group, London, ISBN 978-0-415-48507-4, 305-315.

Vorosmarty, C. J., P. Green, et al., 2000. Global water resources: Vulnerability from climate change acid population growth." Science 289(5477): 284-288.

Wallingford, 2009. InfoWorks RS Help Documaentation, Version 10.5, Wallingford Software, United Kingdom.

White, M.D., Greer, K.A., 2006. The effects of watershed urbanization on the stream hydrology and riparian vegetation of Los Peñasquitos Creek, California. Landscape and Urban Planning 74, 125–138.

Willems, P., Vaes, G., Popa, D., Timbe, L. & Berlamont, J., 2002. Quasi 2D River Flood Modelling. River Flow 2002. Proceedings of the International Conference on Fluvial Hydraulics, D. Bousmar & Y. Zech (Editors), Swets & Zeitlinger, Lisse, Netherlands.

Willems, P., 2004. Hydrological extreme value analysis tool ECQ, Reference Manual. K.U.Leuven – Hydraulics Laboratory, Kasteelpark Arenberg 40 – B-3001, Heverlee, Belgium.

Willems P., 2009. A time series tool to support the multi-criteria performance evaluation of rainfall-runoff models. Environmental Modelling & Software 24 (3): 311 – 321.

Willems P., 2009b. Update and extrapolation of hydrological parameters in the new Code of Good Practice for the Design of Sewerage Systems. Study undertaken for Flemish Environment - Water Operations Section Final Report, Leuven. (In Dutch)

Xu, C-Y., Wide´n, E., Halldin, S., 2005. Modelling hydrological consequences of climate change - progress and challenges. Advances in Atmospheric Sciences 22 (6), 789–797.

Zerger, A. and Wealands, S., 2004. Beyond modelling: Linking models with GIS for flood risk management, Natural Hazards, 33, 191-208.

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