Green–blue water in the city: quantification of impact of source control versus end-of-pipe solutions on sewer and river floods

K. De Vleeschauwer, J. Weustenraad, C. Nolf, V. Wolfs, B. De Meulder, K. Shannon and P. Willems

ABSTRACT

Urbanization and climate change trends put strong pressures on urban water systems. Temporal variations in rainfall, runoff and water availability increase, and need to be compensated for by innovative adaptation strategies. One of these is stormwater retention and infiltration in open and/or green spaces in the city (blue–green water integration). This study evaluated the efficiency of three adaptation strategies for the city of Turnhout in Belgium, namely source control as a result of blue–green water integration, retention basins located downstream of the stormwater sewers, and end-of-pipe solutions based on river flood control reservoirs. The efficiency of these options is quantified by the reduction in sewer and river flood frequencies and volumes, and sewer overflow volumes. This is done by means of long-term simulations (100-year rainfall simulations) using an integrated conceptual sewer–river model calibrated to full hydrodynamic sewer and river models. Results show that combining open, green zones in the city with stormwater retention and infiltration for only 1% of the total city runoff area would lead to a 30 to 50% reduction in sewer flood volumes for return periods in the range 10–100 years. This is due to the additional surface storage and infiltration and consequent reduction in urban runoff. However, the impact of this source control option on downstream river floods is limited. Stormwater retention downstream of the sewer system gives a strong reduction in peak discharges to the receiving river. However due to the difference in response time between the sewer and river systems, this does not lead to a strong reduction in river flood frequency. The paper shows the importance of improving the interface between urban design and water management, and between sewer and river flood management.

Key words | floods, river, sewer, source control, stormwater retention, urban drainage

K. De Vleeschauwer
J. Weustenraad
V. Wolfs
P. Willems (corresponding author)
Hydraulics Division,
Department of Civil Engineering,
KU Leuven,
Kasteelpark Arenberg 40, BE-3001 Leuven,
Belgium
E-mail: *Patrick.Willems@bwk.kuleuven.be*

C. Nolf
B. De Meulder
K. Shannon
Department of Architecture,
KU Leuven,
Kasteelpark Arenberg 51, BE-3001 Leuven,
Belgium

P. Willems
Department of Hydrology and Hydraulic Engineering,
Vrije Universiteit Brussel,
Boulevard de la Plaine 2, 1050 Ixelles,
Belgium

INTRODUCTION

It is increasingly being accepted that trends in urbanization and climate change are impacting on urban flood risks. Increased areas of impervious surface due to urbanization result in reduced infiltration of rainwater, and increased and more rapid surface runoff which poses an increased burden for both sewer systems and rivers (e.g. O'Loughlin *et al.* 1995; Semadeni-Davies *et al.* 2008). During heavy rain storms, the sewers cannot cope with this large amount of water and spill water to the receiving rivers via combined sewer overflows. At the same time, reduced infiltration decreases the recharge of groundwater and diminishes the volume of water in the soil and in the aquifers. Climate trends also have the potential to increase rainfall and surface runoff and sewer overflow frequencies and volumes in wet periods, and decrease water availability during dry seasons (IPCC 2012; Willems *et al.* 2012; Arnbjerg-Nielsen *et al.* 2013). These trends increase the need for adaptation strategies that can respond to this trend towards more temporal variability and extremes. Given the strong interactions that exist between the subsystems involved (sewers, rivers, groundwater, land surface and runoff, etc.), an integrated approach is required that takes these interactions into account. The interactions that exist between land management, including urban design and spatial planning in

doi: 10.2166/wst.2014.306

urban environments, and water management should also be considered. Different adaptation options exist, ranging from small-scale stormwater storage and infiltration facilities to medium-scale retention basins up to large-scale flood control reservoirs, etc. There is a growing consensus in the literature that source control measures are more cost-effective than end-of-pipe solutions (O'Loughlin *et al.* 1995; Stahre 2006; Chocat *et al.* 2007; Arnbjerg-Nielsen 2011; Willems *et al.* 2012; Arnbjerg-Nielsen *et al.* 2013; Zhou 2014). Although there exist some studies that quantify and compare the impacts of different alternative adaptation strategies (O'Loughlin *et al.* 1995; Coombes *et al.* 2002; Waters *et al.* 2003; Watt *et al.* 2003; Rankin & Ball 2004; Mailhot & Duchesne 2010; Beecham & Chowdhury 2012), their number is still rather limited.

In light of this need for adaptation and integrated impact studies, this paper presents the evaluation of three different types of adaptation options for the Belgian city of Turnhout. This city suffers from both river and sewer floods, due to – among other reasons – the increasing urbanization. The adaptation options studied range from source control to mid-catchment interventions to downstream end-of-pipe solutions as follows:

1. rainwater storage and infiltration in open, green areas in the city centre, also referred to as source control by means of 'blue–green water integration';
2. retention basins downstream of the stormwater sewers but just upstream of the outfalls or overflows to the receiving rivers;
3. flood control reservoirs downstream on the river, which can be categorized as end-of-pipe solutions.

The efficiency of these options is evaluated by quantifying the reduction in sewer and river flood frequencies and volumes, and the volumes of sewer overflows. This is done by means of long-term simulations (100-year rainfall simulations) using an integrated conceptual sewer–river model calibrated to full hydrodynamic sewer and river models. The methodology and its different steps discussed next are schematically summarized in Figure 1.

CASE STUDY

The population of the city of Turnhout is approximately 40,000 inhabitants and is located in an upstream region of the Nete basin in the north-east of Belgium. Its combined sewer system is currently being replaced by a separate system, with 80 rainwater overflows and outfalls into the receiving rivers, namely the River Aa and River Visbeek that surround the city on the eastern and western sides (Figure 2). Downstream of the city, the River Aa is frequently flooded causing considerable damage. This has

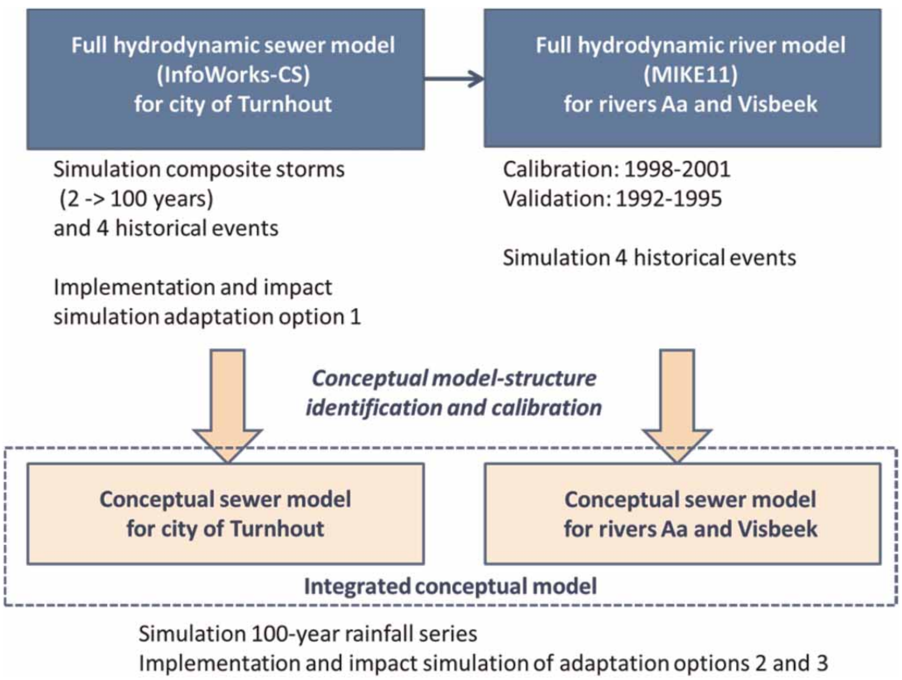


Figure 1 | Schematic overview of the methodology.

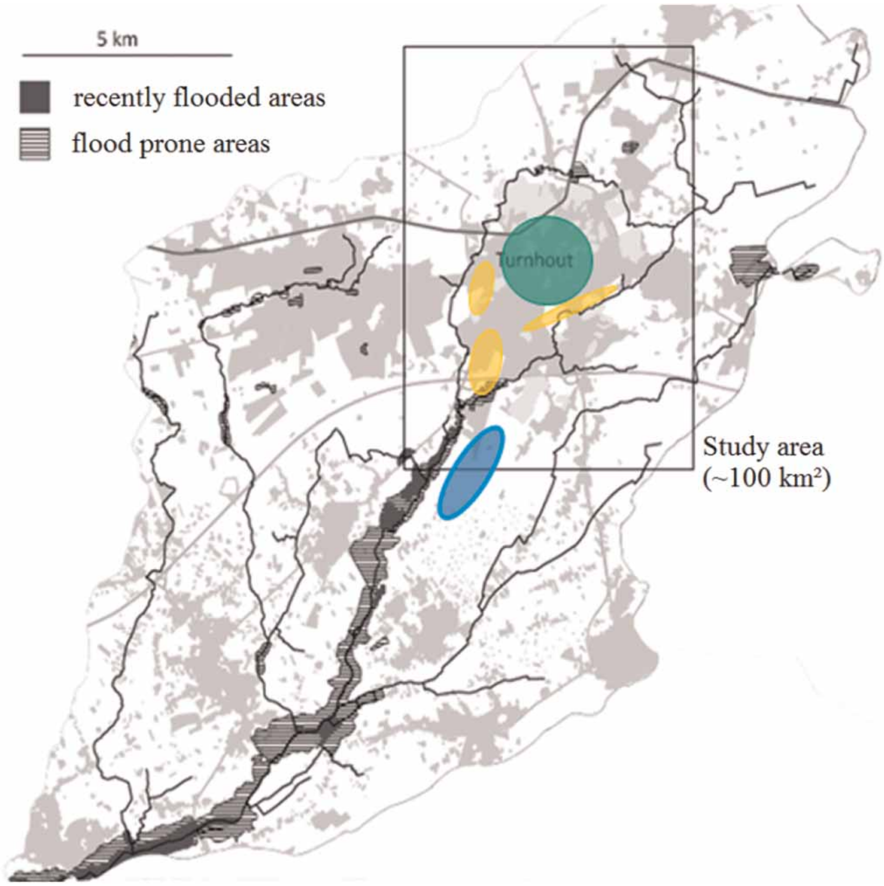


Figure 2 | Location of the city of Turnhout in the catchment of the River Aa in relation to many recently flooded areas. The three studied adaptation options are schematically shown in green (source control by blue–green water integration in the city centre), yellow (storage reservoirs downstream of the stormwater sewers), and blue (river flood control reservoirs). The grey areas are built-up surface (from Corine Land Cover) (Nolf 2013).

several causes, ranging from the natural valley characteristics to historical straightening of the river in the 1970s, which reduced the total river length by about 30%. Also the increased urbanization, with increased impervious surfaces and increased amounts of water that spills into the rivers, has the potential to be an important factor. Replacement of the combined sewer system by a separate sewer system was planned by the city at the start of this study. This planned separate system was considered as a starting basis for this research.

INTEGRATED CONCEPTUAL SEWER–RIVER MODEL DEVELOPMENT

To simulate the impact of the three studied adaptation options on sewer overflows and river flows and flood conditions, an integrated model was built of the separate stormwater system of the city and the River Aa and River Visbeek within the 'study area' shown in Figure 2. This integrated model was based on existing full hydrodynamic models of the sewer (InfoWorks-CS) and river systems (MIKE11). These models were built based on detailed topographical survey data (e.g. cross-sections approximately every 50 m along the river) and other hydraulic system properties. All hydraulic structures including pumps and weirs were implemented based on their dimensional properties and regulation rules. Conceptual rainfall runoff models estimate runoff into the sewer and river hydrodynamic models. The hydrological–hydrodynamic river model was calibrated and validated based on a state-of-the-art comparison of discharge and water level simulation results at a downstream flow gauging station for the period 1992–2001 (1998–2001 for calibration, 1992–1995 for validation). Given the focus of this study on floods, special attention was paid to the accuracy of the peak discharges and water levels also in relation to the return period (Figure 3). The unbiased

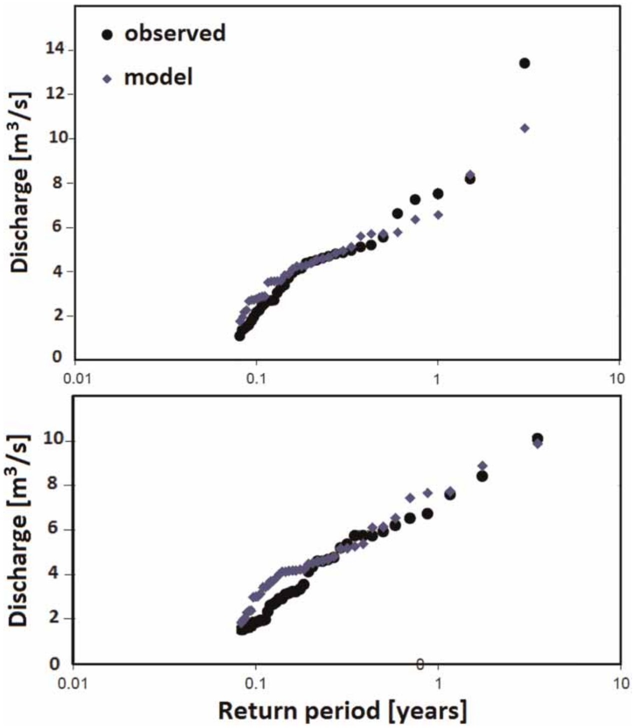


Figure 3 | Comparison of observed and model-based river peak discharge versus return period at the river gauging station for calibration (top, 1998–2001) and validation (bottom, 1992–1995) periods, after simulation of the full hydrodynamic integrated sewer–river model.

results for the river peak flows and for the increase in peak flow versus increase in return period show that the model is applicable for extreme event simulations.

To allow bi-directional interactions between both systems to be taken into account, but without excessive calculation times, a conceptual model of the integrated sewer–river system was developed. The structure of this model was identified and the parameters calibrated to simulation results generated by the full hydrodynamic models. This was using the methodology presented and evaluated by Wolfs *et al.* (2013). The entire sewer and river system is separated by subsystems, each represented by a reservoir-type model, with inflow, throughflow and potential overflow. Water continuity is considered when computing the time-varying storage in the system, separated in static and dynamic storages. Throughflow discharges are modelled by means of relationships between the static storage and the throughflow discharge, or by applying transfer functions to transfer upstream and/or lateral flows to downstream flows (depending on the subsystem characteristics). Overflow discharges are determined applying relationships with the system storage. More details of the methodology can be found in Wolfs *et al.* (2013).

Three types of model simulations were considered in this study: (i) simulation of synthetic design storms for return periods of 2, 5, 10, 20, 50 and 100 years (called composite storms; see Willems (2013) for details); (ii) simulation of selected historical events based on local rain gauge data (two summer events on 18/08/1992 and 24/06/1997, and two winter events on 10/12/1993 and 19/01/1995, were selected and were extended with two extreme rain showers on 15/07/1962 and 23/06/1969); (iii) long-term simulations using a 100-year series (1901–2000) of 10-minute rainfall intensities recorded at Uccle, Belgium. The long-term simulations were included to consider all types of rain events and sewer–river interaction effects (other rain storms may lead to extreme conditions in river catchment runoff, urban drainage and sewer impact).

The conceptual model simulation time step is 10 minutes. Goodness-of-fit statistics applied to evaluate the accuracy of the conceptual model for the model calibration and validation events are the percentage model bias (PBIAS) and the Nash–Sutcliffe model efficiency (NSE) for the conceptual model results $Y_F(i)$ versus the full hydrodynamic model results $Y_C(i)$ at the different 10-minute time steps i

$$\text{PBIAS} = 100\frac{\sum_{i=1}^{n}(Y_C(i) - Y_F(i))}{\sum_{i=1}^{n} Y_F(i)}$$

$$\text{NSE} = 1 - \frac{\sum_{i=1}^{n}(Y_C(i) - Y_F(i))^2}{\sum_{i=1}^{n}\left(Y_F(i) - \overline{Y_F}\right)^2}$$

where n is the number of 10-minute time steps per event, and $\overline{Y_F}$ the mean of the full hydrodynamic model results for each event.

DESIGN OF THE ADAPTATION OPTIONS

Design of blue–green water integration

As a result of an intensive collaboration between urban designers/planners and urban water engineers, suitable locations for blue–green water integration measures (Adaptation Option 1) have been identified (Figure 4). These are open, mainly green, areas in the city centre that can serve multiple functions including stormwater retention and infiltration. They were identified taking the following factors into account (Nolf 2013):

- sewer flood locations (as obtained from the sewer model simulations);

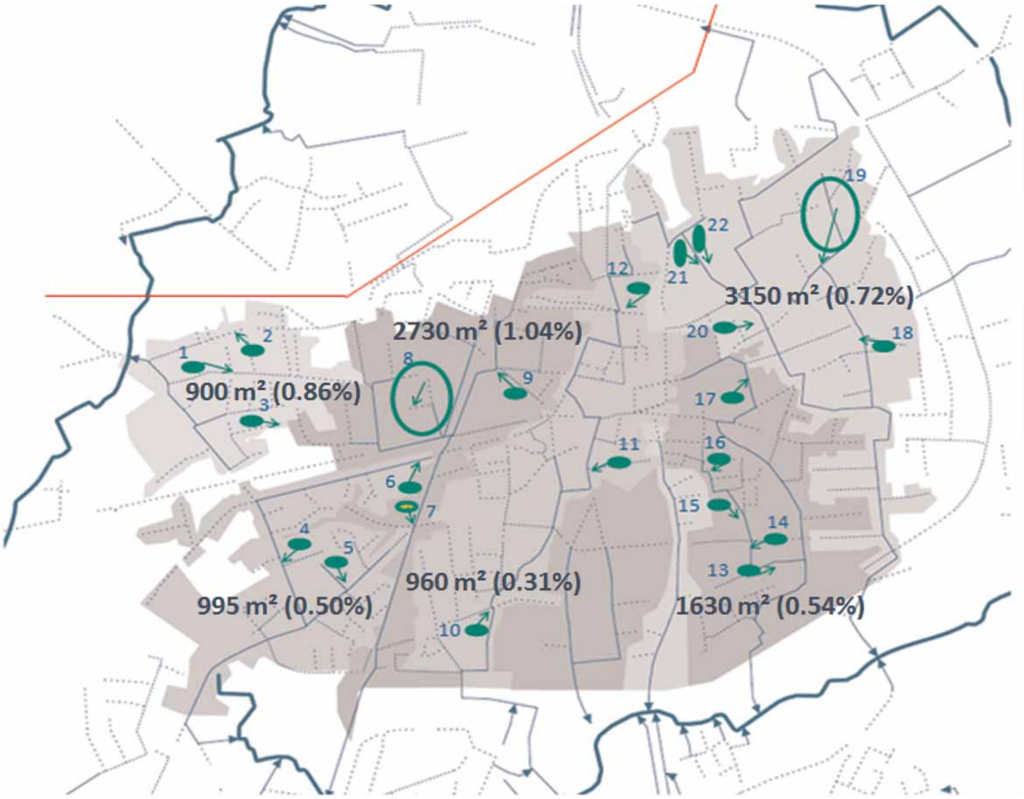


Figure 4 | Locations of the 22 open, green areas identified for green–blue water integration measures (Adaptation Option 1). The background map shows the separate sewer system (lines) and drainage areas (grey areas).

- the open, green zones in the city such as parks, city gardens, playing gardens, areas around schools and hospitals (see Figure 5(a));
- distinction between private and public ownership (city-owned places, schools, hospitals, churches, social services) (see Figure 5(b) for the public owned spaces);
- zones that are in transition or for which redevelopment is planned on the short, medium or long term were also mapped as potential additional opportunities for blue–green water integration in the future (see Figure 5(c)).

As a choice for this study, only (but all) existing green spaces belonging to the public domain and upstream of the sewer flood locations for a return period of 20 years were selected. See Figure 6 for an example of such space, converting an existing playground into a multi-use water infiltration/retention facility.

From these selection criteria, the identified areas for green–blue water integration in the city centre have a total area of 10,365 m^2. When the area for green–blue water integration is computed for each of the separate sewer subsystems (these are the subsystems that drain to one of the downstream stormwater outfalls), they cover between 0.31% and 1.04% of the total surface areas connected to these subsystems. The fraction varies according to the more or less open character of the urban fabric that covers each subsystem. The presence of two public green squares in the central part of the city explains the higher coefficient (1.04%) in the sub-watershed at points 8 and 9. For each of the individual areas, the feasible storage capacity by lowering the surface was identified as part of the urban design process, but was aimed to achieve overflow return periods of 20 years or higher. To ensure the safety of the residents, the maximum water depth in the areas was limited to 40 cm.

Storm sewer retention basins

The best locations for the stormwater retention basins downstream of the storm sewer network (Adaptation Option 2) were carefully selected. This was achieved by simulating the different composite storms in the InfoWorks-CS model to quantify the impacts on the discharges at all 80 installed overflows or outfalls. Six overflows were selected that are responsible for 80% of the total volume of overflows (for a

Figure 5 | Identification of the open, green areas for green–blue water integration (Adaptation Option 1) after overlaying the sewer flood locations (not shown), the green zones (a), the public owned parcels (b), and the places in transformation (c) (after Nolf 2013).

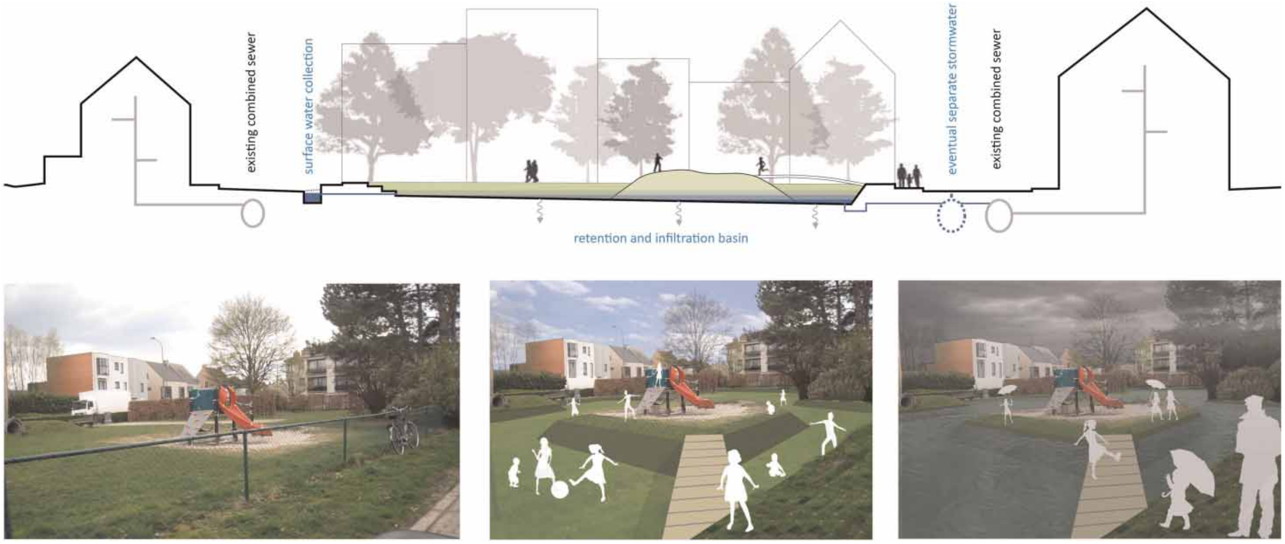


Figure 6 | Example of a designed open, green area for green–blue water integration – converting an existing playground into a multi-use water infiltration/retention facility.

20-year return period composite storm). These overflows also have the highest peak discharges. Stormwater retention basins were designed for installation at these six overflows, such that the return period of overflow is equal to 20 years. This resulted in a combined storage capacity of 58,000 m^3 for the six retention basins. The actual distribution of this total capacity over the six basins is shown in Figure 7.

River flood control reservoirs

River flood control reservoirs downstream of the sewer system were implemented as storage reservoirs, filled by overflowing of a weir, representing the (reduced) dike crest or gate level of the hydraulic structure which would control the reservoir inflow. The weir crest level was taken equal to the river water level above which downstream flooding starts. This level corresponds to a river flow of 11 m^3/s.

MODEL IMPLEMENTATION

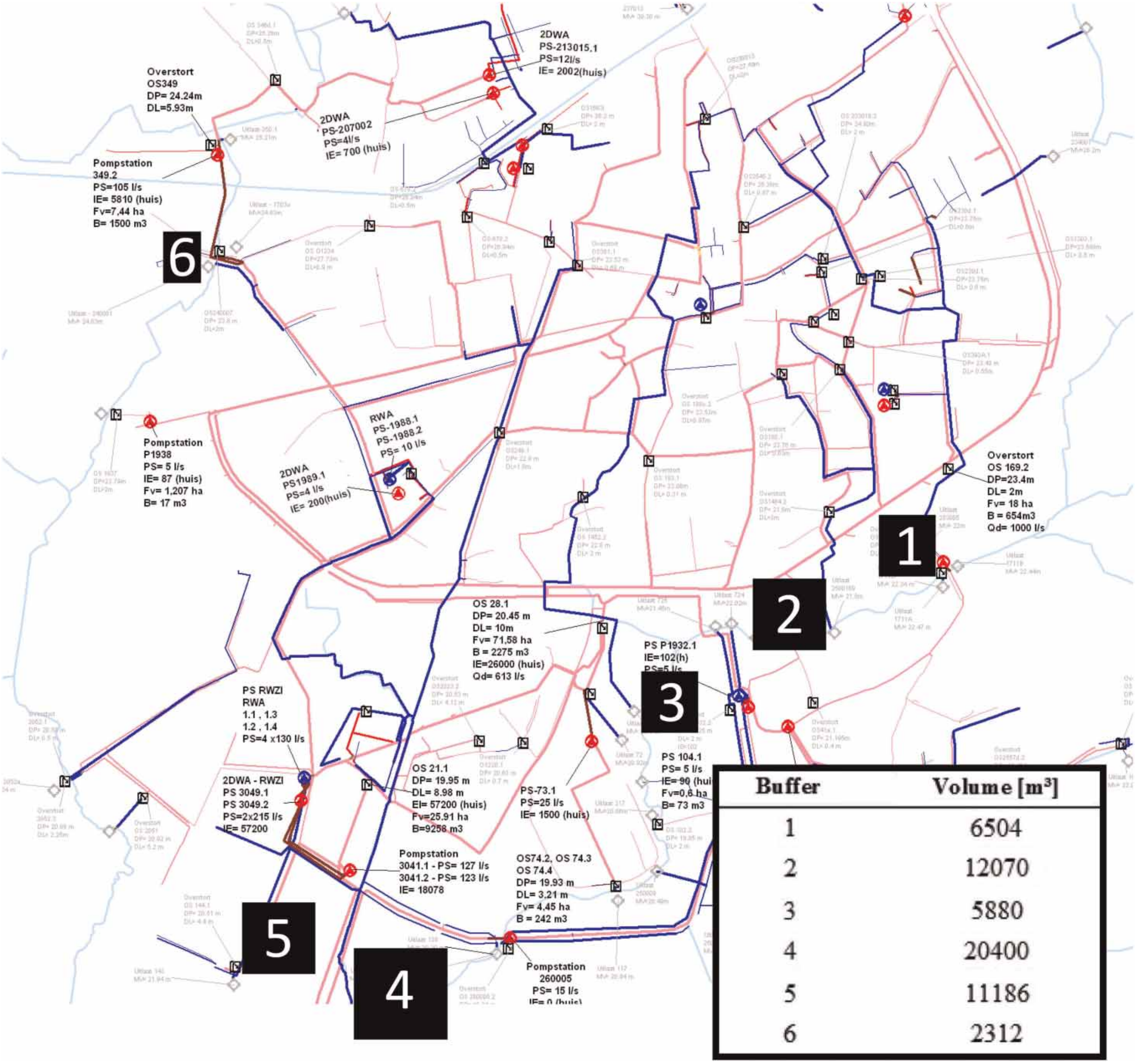
Calibration of the conceptual model

Given that the six selected storm sewer overflows cover about 80% of the total overflow volume, only these overflows were explicitly included in the conceptual sewer model. A factor was applied to the simulated overflowing discharges to account for the 20% of overflows not explicitly modelled. For each of the six overflows, the upstream storm sewer networks were represented by a piece-wise linear relation between the downstream sewer throughflow discharge and the sewer system storage, a piecewise linear relation for the overflow discharge and the system storage, and a water continuity equation.

See Figure 8 for an example of the identified relation for the storm sewer subsystem upstream of overflow 6 based on the composite storm simulation results for return periods of 20 and 50 years in the full hydrodynamic sewer model.

The river system, consisting of the River Aa and River Visbeek, is divided into five interconnected segments. Each segment is represented by a reservoir, characterized by a transfer function. This transfer function calculates the outflow based on the inflows and outflows of current and previous time steps. The inflows consist of the throughflow and overflow discharges from the sewer outfalls and overflows, the upstream catchment runoff, the flow from upstream river segments or boundary conditions (e.g. point sources, inflowing tributary rivers). The flows are then converted to water levels by means of rating curves. Both the transfer functions and the rating curves were calibrated to simulation results from the full hydrodynamic model. In parallel to the river reservoirs, the river floodplains are implemented as storage reservoirs filled by overflowing water from the river to the floodplains. Given that the rainfall runoff models applied for the full hydrodynamic model were already conceptual, they were linked to the conceptual river model without further simplifications.

The calibration of the conceptual sewer sub-models to the full hydrodynamic models was based on the simulation results for the composite storms and the four historical



Buffer	Volume [m³]
1	6504
2	12070
3	5880
4	20400
5	11186
6	2312

Figure 7 | Locations of the six stormwater reservoirs downstream of the storm sewer sub-networks (Adaptation Option 2), together with their storage capacity.

events; whereas the calibration of the river sub-model was based on the four historical events. The two extreme rain storms were subsequently used to validate the integrated conceptual sewer–river model. An example of such validation is shown in Figure 9. The PBIAS values for the four historical events ranged between −6.6% and 5.3% for the discharge time-series results at the outlets of the different sub-models. These are for the river locations upstream, at and downstream of the sewer overflows. The NSE values range between 0.91 and 0.99. For the two extreme validation events, the PBIAS ranges between −0.1% and 1.8%, and the NSE between 0.98 and 0.99. This shows close agreement of the conceptual model to the full hydrodynamic model results in terms of both systematic and random error: the PBIAS values are close to 0, and the NSE values close to 1. Obviously, the model results will differ more from the real values, as was presented before.

Implementation of the adaptation options

After set up, calibration and validation of the models, the three adaptation options were implemented. The

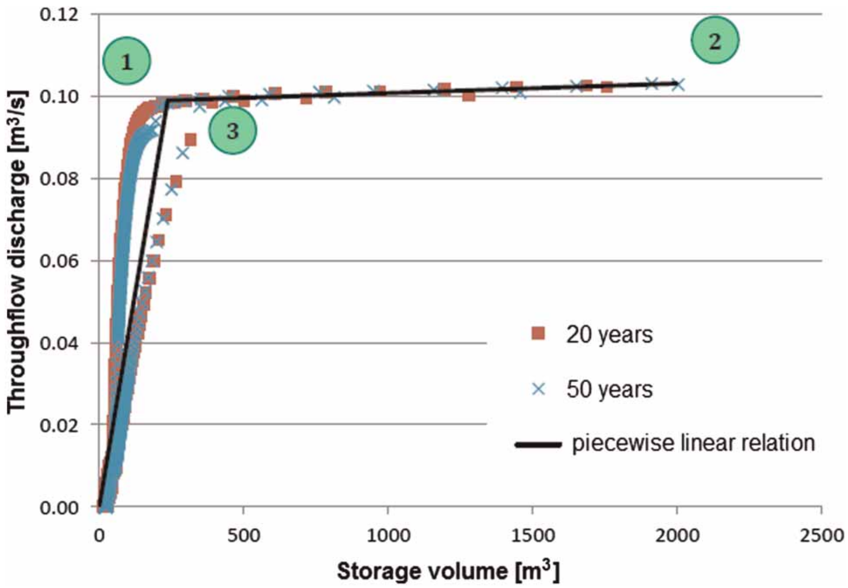


Figure 8 | Identification and calibration of the piecewise linear throughflow–storage relation for the storm sewer subsystem upstream of Overflow No. 6 based on the composite storm simulation results for return periods of 20 and 50 years in the full hydrodynamic sewer model. Point 1 represents the system storage at which an internal system overflow starts; point 2 the maximum system storage at which the internal overflow discharge reaches a maximum value; point 1 is at the increasing flank of the hydrograph event, and point 3 at the decreasing flank, showing hysteresis in the throughflow–storage relation. This hysteresis was, however, not taken into account; the piecewise linear relation represents average conditions for the increasing and decreasing flanks.

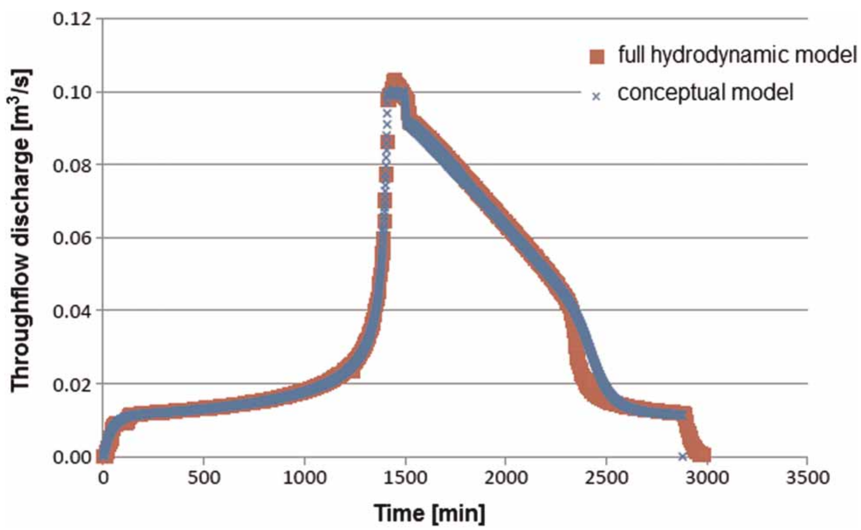


Figure 9 | Comparison of throughflow simulation results for the full hydrodynamic and conceptual sewer models for Retention Basin 6.

green–blue water integration option involved representing the identified open, green areas as storage reservoirs along the sewer network, filled by surface runoff from the connected areas, with an overflow to the storm sewer network, and emptied by infiltration. A constant infiltration rate of 20 mm/h was adopted given that the region predominantly consists of fine sandy soils.

The six retention basins downstream of the storm sewer system were implemented as off-line storage reservoirs. The stormwater runoff leads to throughflow to the receiving river; only during heavy rainfall events will the retention basin be filled via the internal overflow (Figure 10). The retention basin is emptied by means of a flap valve. Each retention basin has an external

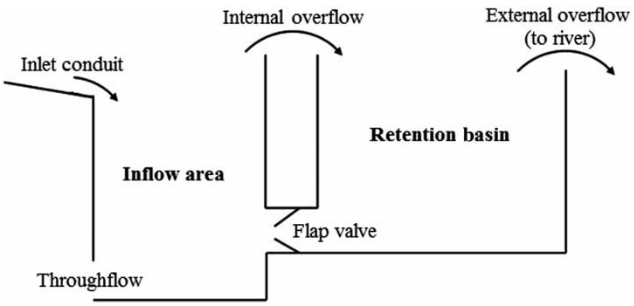


Figure 10 | Schematic representation of the retention basin as implemented in the conceptual model.

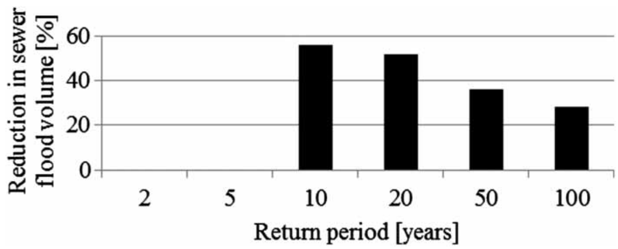


Figure 11 | Reduction in total sewer flood volume for the entire sewer system of the city of Turnhout as a function of the return period of the composite storm due to the implementation of Adaptation Option 1 in the full hydrodynamic sewer model.

emergency overflow to the receiving river in case the storage capacity of the retention basin is exceeded. Similar off-line storage was also considered for the flood control reservoirs downstream on the River Aa, this time filled based on overflow from the river when the river water levels exceed a given level.

The different adaptation options were implemented both in the full hydrodynamic and conceptual (sewer, river and integrated) models. In the same way as the conceptual sewer and river models were validated based on comparison of simulation results with those of the full hydrodynamic models, the same comparisons were undertaken after the implementation of each of the adaptation options, for the composite and four historical events.

RESULTS

Results before and after implementation of each of the adaptation options were obtained from simulation of the composite storms in the full hydrodynamic sewer model, and after simulation of the 100-year Uccle rainfall series in the integrated conceptual sewer–river model. The results of the time series simulation were statistically analysed for changes in the overflow and flood frequencies and volumes, and frequency distributions (peak flows versus empirical return period).

Figure 11 shows the reduction in sewer flood volumes after implementation of Adaptation Option 1, as a function of return period. No reduction in sewer flood volume is noted for storms with return periods less than 5 years, since the storm sewer system was designed to cope with storms of these magnitudes. Although the open, green zones available for stormwater storage and infiltration are limited (only about 1% of the total catchment area in the city), the reduction in sewer flood volumes is very large: (50–57%) for return periods of 10–20 years. This is due to the highly non-linear response of the flood variables to the runoff change, because sewer floods (as is the case also for overflows) are due to exceedance of runoff or sewer flow thresholds. Due to mathematical reasons, when the exceedance probabilities are lower or the threshold higher – this means for systems with a higher safety level – the relative change often becomes higher. There is no need to explain that the impact ranges can even be wider when studying environmental or socio-economic impacts.

The impact on the storm sewer overflow peak discharges to the receiving rivers is, however, limited, as shown in Figure 12, where the sum of the peak discharges from the six selected overflows is shown versus the return period. This sum can be seen as the maximum potential impact these overflows have on the receiving river. The actual impact obviously will be lower because of time shifts between the flow peaks and the spatial distribution of the overflows along the river. Figure 12 shows that the retention basins lead to a significant reduction in peak overflow discharges. This is mainly the case for return periods smaller than 20 years, given that the retention basins were designed for that return period. For larger

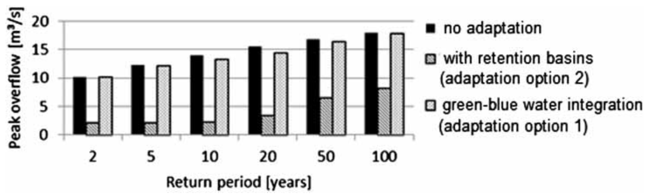


Figure 12 | Sum of the peak overflow discharges for the six selected overflows as a function of the return period of the composite storm, before and after Adaptation Options 1 and 2 as simulated by the full hydrodynamic sewer model.

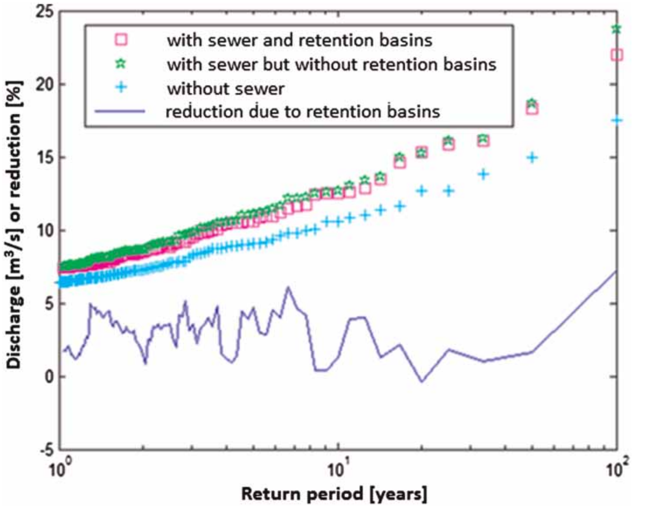


Figure 13 | River peak discharge versus return period with and without the storm sewer system contribution downstream of the confluence of the Rivers Aa and Visbeek, and before and after Adaptation Option 2 based on simulation of the 100-year rainfall series in the integrated conceptual sewer–river model.

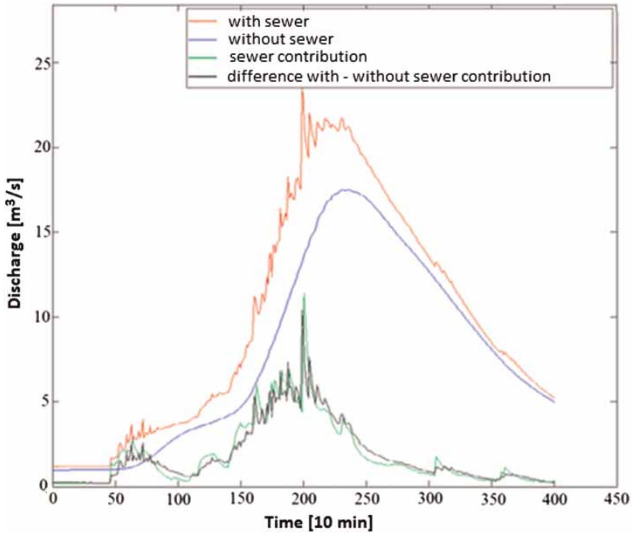


Figure 14 | Temporal river discharge variations downstream of the confluence of the rivers Aa and Visbeek for the event leading to the highest river peak flow in the 100-year simulation in the integrated conceptual sewer–river model, with and without the storm sewer system contribution, and before and after Adaptation Option 2.

return periods, the effects are smaller but still more than 50%.

When the impact on the receiving river is quantified, it is shown in Figure 13 that the sewer system has a significant impact on the river peak flows. To allow correct comparisons between river flows before and after the sewer contributions, the catchment area of the sewer system was added to the river catchment area for the simulation without sewer contribution. This adjustment in catchment area, however, had a minor effect on the river flows. The installation of the retention basins reduces the river peak flows by a maximum of 5%. This is a disappointing result, considering the considerable investment costs of the installation of these basins. To explain these unsatisfactory results, several single rainfall events were examined. Figure 14 displays the result of the event leading to the highest peak flow in the 100-year simulation period. It is shown that the peak flow from the sewer overflow is shifted by about 6 hours from the river peak flow. This temporal shift explains why changes in the sewer flows do not have a significant impact on the river peak flows. This means that the retention basins, which aim to reduce the sewer overflow peaks by delaying the urban runoff flows in time, do not achieve their goal. One can imagine that in some other cases the installation of retention basins can have a negative impact rather than a positive or negligible effect on the river peak flows. This happens when the delay in urban runoff peaks is such that these peaks coincide with the catchment runoff or flow peaks in the river. Without the installation of the retention basins, the urban runoff peaks typically occur earlier in time due to the shorter response time of the urban drainage system to rainfall.

Finally, when evaluating the river flood control reservoir, Figure 15 shows that the reservoir leads to a larger reduction in river peak flows than the retention basins.

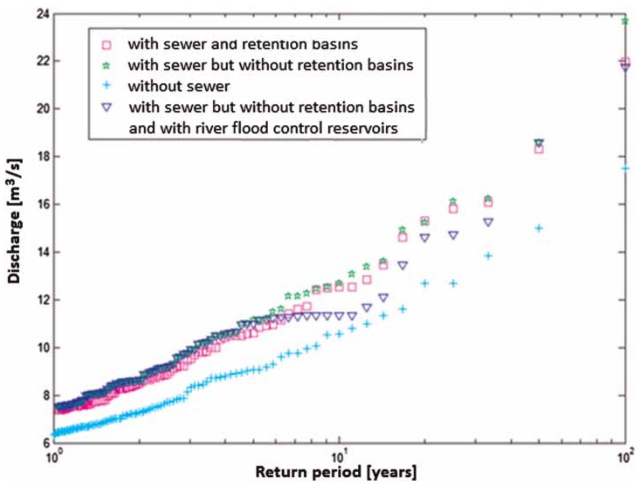


Figure 15 | River peak discharge versus return period downstream of the confluence of the rivers Aa and Visbeek with and without the storm sewer system contribution and before and after Adaptation Options 2 and 3 based on simulation of the 100-year rainfall series in the integrated conceptual sewer–river model.

Peak flow reductions are of course only noticed for flows exceeding the flood control threshold of 11 m^3/s.

DISCUSSION AND CONCLUSIONS

Adaption Option 1 can be considered very efficient given that the stormwater storage and infiltration in the open, green zones, which occupy merely 1% of the total catchment area in the city centre, result in a strong reduction of the sewer flood volumes (between 30 and 50%, depending on the return period). The results indeed show that even with a very limited amount of surface storage and infiltration, a significant reduction of the sewer flood volume can be achieved. This, however, requires intensive collaboration between urban water management and urban planning in an early phase of the design of a stormwater drainage system. The collaboration experience from this study has demonstrated how the knowledge of the urban fabric and its capacity for transformation provides alternative solutions for (more) open, green zones (e.g. include private gardens, or future areas to be transformed).

The stormwater storage and infiltration by improved interfacing between urban design and water management does not, however, reduce storm sewer overflows or river flows. While retention basins reduce peak sewer overflow, these reductions do not decrease the river peak flows much. After installation of six retention basins at the main stormwater overflows, the sum of the peak flows from the six overflows is reduced by 80% for return periods smaller than 20 years. For larger return periods, the reduction is smaller due to the external overflow of the retention basin to the river. For a return period of 100 years, the reduction is still around 50%.

Despite the great decrease in the peak overflow discharges, the retention basins do not have a significant impact on the river peak flows. The reduction in these peak flows is less than 5% for all return periods. Given the considerable costs involved in the construction of such retention basins, this result is very disappointing. The reason is the strong difference in response time between the sewer and river systems, and the consequent time shifts between the peak flows from/in both systems. The concentration time of the sewer system at the sewer overflows is 1 to 2 hours, whereas the catchment runoff has a concentration time of 6 hours. Related to this difference in concentration time, different rain storms lead to extreme flow conditions in both systems. These are extreme convective summer storms for the sewer system, and longer duration storms after long wet periods (leading to high soil saturation) for river catchment runoff.

The use of open, green zones in the city for stormwater storage and infiltration has a negligible effect on the reduction of the river peak flows, since this has barely an effect on the sewer overflow discharges. Further investigation on this (not shown) indicated that, in order to obtain a comparable impact on the river peak flow reduction as the installation of the retention basins, 5 to 20% (depending on the return period) of the rainfall should be retained in open, green zones in the city centre. In reality, such retention is not achievable in highly urbanized areas. This may be different for towns located in more rural areas, where adaptation based on green–blue water integration at large scale may be achievable, leading to significant reductions in both sewer floods and river floods.

The most effective way to reduce river peak flows is not by the installation of retention basins downstream of the storm sewer systems, but by flood control reservoirs along the river downstream. Measures more upstream in the river catchment may be efficient as well, but were not analysed given the focus of this study on the city stormwater related problems and adaptation needs.

Note that these impacts could be evaluated based on long-term simulations, in this study based on 100-year rainfall series simulations, thanks to the application of a conceptual sewer–river model, identified and calibrated based on shorter simulation runs with full hydrodynamic sewer and river models. The use of the conceptual integrated model kept simulation times manageable: the calculation time of the 100-year rainfall series simulation in the integrated model applied in this study was about 90 minutes on a single core computer. One limitation of this study is the ignoring of the spatial variability of rainfall; the same rainfall input was applied to the river and sewer catchments. Due to the limited size of these catchments, it is expected that this simplification did not bias the conclusions obtained from the study. It is, however, advised to take the spatial rainfall variability over catchments into account. Also other sources of uncertainty in the modelling chain may affect the impact results of the studied alternative adaptation measures. As is the case for any model-based impact study, it would be very useful to extend the analysis with a detailed uncertainty analysis.

It should be noted that the results obtained in this research are only valid for the case-study area. This means that these results cannot be transferred directly to other sites. The question arises whether a more general theory can be postulated whereby, based on the main sewer and river characteristics, one can determine whether retention basins between sewers and a river can result in a cost-effective positive impact. The difference in response time between the sewer and river systems appears to be the most important factor here, but other factors may play a role as well, such as the specific locations and spatial configuration of the different sewer overflows, and the relative ratio of the sewer overflow discharges to the river peak discharges.

ACKNOWLEDGEMENTS

The 100-year rainfall time series was made available by the Royal Meteorological Institute of Belgium. The authors would like to thank Innovyze for the InfoWorks CS license and DHI Water and Environment for the MIKE11 license. The detailed InfoWorks CS model was set up in a project for the city of Turnhout, and the detailed river survey data that enabled implementation of the MIKE11 model were provided by the Province of Antwerp. Adaptation Option 1 was designed in cooperation with the KU Leuven Department of Architecture, through the PhD study by Christian Nolf, supported by the Fund for Scientific Research (FWO) – Flanders. The contributions by Vincent Wolfs were supported by the Agency for Innovation by Science and Technology in Flanders (IWT).

REFERENCES

Arnbjerg-Nielsen, K. 2011 Past, present, and future design of urban drainage systems with focus on Danish experiences. *Water Science and Technology* **63** (3), 527–535.

Arnbjerg-Nielsen, K., Willems, P., Olsson, J., Beecham, S., Pathirana, A., Bülow Gregersen, I., Madsen, H. & Nguyen, V.-T.-V. 2013 Impacts of climate change on rainfall extremes and urban drainage systems: a review. *Water Science and Technology* **68** (1), 16–28.

Beecham, S. & Chowdhury, R. 2012 Effects of changing rainfall patterns on WSUD in Australia. *Journal of Water Management* **165** (5), 285–298.

Chocat, B., Ashley, R., Marsalek, J., Matos, M. R., Rauch, W., Schilling, W. & Urbonas, B. 2007 Toward the sustainable management of urban storm-water. *Indoor and Built Environment* **16** (3), 273–285.

Coombes, P. J., Kuczera, G., Kalma, J. D. & Argue, J. R. 2002 An evaluation of the benefits of source control measures at the regional scale. *Urban Water* **4** (4), 307–320.

IPCC 2012 Managing the risks of extreme events and disasters to advance climate change adaptation. In: *A Special Report of Working Groups I and II of the Intergovernmental Panel on Climate Change* (C. B. Field, V. Barros, T. F. Stocker, D. Qin, D. J. Dokken, K. L. Ebi, M.D. Mastrandrea, K. J. Mach, G.-K. Plattner, S. K. Allen, M. Tignor & P.M. Midgley, eds). Cambridge University Press, Cambridge, UK.

Mailhot, A. & Duchesne, S. 2010 Design criteria of urban drainage infrastructures under climate change. *Journal of Water Resources Planning and Management* **136**, 201–208.

Nolf, C. 2013 *Sections of Flanders. Challenges of Upstream Water Management and the Spatial Structuring of the Nebulous City*. PhD thesis, KU Leuven – Faculty of Engineering, Leuven, Belgium.

O'Loughlin, G., Beecham, S., Lees, S., Rose, L. & Nicholas, D. 1995 On-site stormwater detention systems in Sydney. *Water Science and Technology* **32** (1), 69–175.

Rankin, K. & Ball, J. E. 2004 A review of the performance of permeable pavers. In: *Proceedings of the 2004 International Conference on Water Sensitive Urban Design (WSUD2004): Cities as Catchments, Adelaide, South Australia*.

Semadeni-Davies, A., Hernebring, C., Svensson, G. & Gustafsson, L.-G. 2008 The impacts of climate change and urbanisation on drainage in Helsingborg, Sweden: combined sewer system. *Journal of Hydrology* **350**, 100–113.

Stahre, P. 2006 *Sustainability in Urban Storm Drainage. Planning and Examples*. Svenskt Vatten, Stockholm.

Waters, D., Watt, W. E., Marsalek, J. & Andreson, B. C. 2003 Adaptation of a storm drainage system to accommodate increased rainfall resulting from climate change. *Journal of Environmental Planning and Management* **46**, 755–770.

Watt, W. E., Waters, D. & McLean, R. 2003 *Climate Variability and Urban Stormwater Infrastructure in Canada: Context and Case Studies*. Toronto-Niagara Region Study Report and Working Paper Series, Report 2003-1. Meteorological Service of Canada, Waterloo, Ontario, Canada.

Willems, P. 2013 Revision of urban drainage design rules after assessment of climate change impacts on precipitation extremes at Uccle, Belgium. *Journal of Hydrology* **496**, 166–177.

Willems, P., Olsson, J., Arnbjerg-Nielsen, K., Beecham, S., Pathirana, A., Bülow Gregersen, I., Madsen, H. & Nguyen, V.-T.-V. 2012 *Impacts of Climate Change on Rainfall Extremes and Urban Drainage*. IWA Publishing, London.

Wolfs, V., Villazon Gomez, M. F. & Willems, P. 2013 Development of a semi-automated model identification and calibration tool for conceptual modelling of sewer systems. *Water Science and Technology* **68** (1), 67–75.

Zhou, Q. 2014 A review of sustainable urban drainage systems considering the climate change impact and urbanization impacts. *Water* **6** (4), 976–992.

First received 28 February 2014; accepted in revised form 25 June 2014. Available online 24 July 2014