Model-based evaluation of stormwater management alternatives for a new development

Modélisation de solutions alternatives de gestion des eaux pluviales pour un nouveau développement urbain

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RÉSUMÉ

Cette étude de cas analyse et compare la performance de plusieurs méthodes de gestion des eaux pluviales pour un futur développement résidentiel situé en République Tchèque. Les solutions alternatives en matière de drainage sont mises au point à partir de la conception originale proposée précédemment par une équipe de consultants et utilisant des approches conventionnelles de la conception des systèmes de drainage. Les lacunes potentielles de la conception d'origine ainsi que la performance des solutions alternatives sont analysées en utilisant un modèle de simulation de pluie et d'écoulement conçu spécifiquement à cet effet. Quatre scénarios de modélisation ont été envisagés : conception fournie par le consultant avec un égout surdimensionné et des mesures de contrôle à la source, conception modifiée avec branchement sur une cuve de rétention et des mesures de contrôle à la source, drainage conventionnel avec branchement sur une cuve de rétention, drainage conventionnel sans aucun dispositif de rétention. Tous les scénarios de conception sont comparés et évalués en termes de performance hydraulique et de coût de construction. Les résultats indiquent que même lorsque la disposition de chaque système est ajustée pour le même niveau de service pendant la période de retour des pluies, ils peuvent avoir des performances différentes lors de pluies particulièrement fortes. En outre, les conclusions actuelles suggèrent qu'une distribution décentralisée de la capacité de stockage dans le bassin versant pourrait permettre des économies, par rapport à un contrôle centralisé du débit en aval.

ABSTRACT

This case study analyses and compares the performance of different stormwater management options for a future residential development located in the Czech Republic. The drainage alternatives are developed from the original design proposed earlier by a team of consultants using conventional drainage design approaches. The potential shortcomings of the original design as well as the performance of its alternatives are analysed with the use of a rainfall-runoff simulation model built specifically for this purpose. Four design scenarios were considered: design provided by the consultancy with oversized sewer and source control measures; modified design with on-line detention tank and source control measures; conventional drainage with on-line detention tank; conventional drainage without any detention facilities. All design scenarios are compared and evaluated in terms of their hydraulic performance and construction costs. The results indicate that even when the system layouts are adjusted for the same level of service given by the return period of the rainfall, they can perform differently under extreme rainfall conditions. Moreover, present findings suggest that a decentralized distribution of storage capacity within the catchment could contribute to cost savings when compared to centralized downstream flow control.

KEYWORDS

Hydraulic performance, modelling, stormwater management, sustainable urban drainage

1 INTRODUCTION

It is generally acknowledged that the planning and design phase is a key part of any land development, redevelopment, or retrofit project. The importance of the design grows with the project size as well as with the seriousness of potential consequences of the design failure. The conventional approach towards the design of stormwater management in urban areas has for a long time been a single-objective task that has been mainly focused on water quantity control. Traditionally, engineers aim to minimize construction costs whilst ensuring good system performance under specified design criteria. It is clear that such design procedure with its single-oriented focus on hydraulic performance of pipework cannot longer be a sufficient option for future developments (Zhou, 2014).

In urban drainage, as in many other disciplines, there is a profound difference between theory and practice. In order to bridge the gap between more or less abstract values and concrete engineering decisions there are performance standards in place (Kolsky & Butler, 2002). But even with them the design of a new drainage system does not need to be a straightforward task as it very often is driven by experience and attitude of a particular planner or designer.

The problem with this is that the engineer´s judgement might be biased in favour of traditional and proven techniques. The final design then might comply with all standards and criteria required by authorities but it can still be far from its optimum. Furthermore, the institutions responsible for the public administration of urban stormwater are very often set up in favour of traditional drainage approaches and methods (Brown, 2005). That makes it even more difficult for the engineer to escape from vicious circle of conventional drainage, and assess the situation without the bias.

2 METHODS

This case study aims to evaluate and compare the performance of several stormwater management options for a future development. The task was to set up a simulation model based on the preliminary design provided by the consultancy and subsequently develop alternative system layouts that satisfy water quality and hydraulic requirements with minimal material and construction costs. The main objective was to analyse proposed design with the use of modelling tool and compare it with the cost-effectiveness of other drainage alternatives.

2.1 Study site

The study catchment is located in the south-eastern part of Brno, which is the second largest municipality in the Czech Republic. The city of Brno is the administrative and cultural centre of the South Moravian Region and has about 400,000 inhabitants. The city lies at the confluence of the Svitava and Svratka rivers and belongs to the temperate climate zone with a mean annual precipitation of 505 mm and a mean air temperature of 9.4°C.

The total size of the study catchment is 13.8 ha. The highest part of the area is at the east part of the catchment with heights up to 238 m.a.s.l. steeply sloping down towards the west (202 m.a.s.l.). The average slope of the catchment is about 8.45%, which is relatively steep in comparison to average slopes of surrounding residential and commercial areas. The site is undeveloped (agricultural land) at the present; however, it is planned to be built up with residential development with an expected population of 2,750 inhabitants.

2.2 Boundary conditions

A common requirement when developing a particular site is to compare what the estimated runoff from the catchment, usually the peak flow, was before and what is after the development. An alternative method is to set a universal outflow threshold that is equitable for all developers. In Brno such an approach is applied by the local authorities. According to the General Master plan of drainage system, the peak runoff from new developments should not be higher than 10 l/s/ha for any design storm with the return period up to 5 years.

Apart from the outflow threshold, the drainage design of the study area is limited by two other boundary criteria arising from local conditions of wider catchment. The first condition being: wastewaters from the existing urbanized areas are collected by a combined sewer network with a limited capacity. Because of this, it is necessary to construct a new storm sewer that would convey stormwater runoff from new developments to the nearby watercourse. The receiving water body is located approximately 700 m to the west from the development site (Figure 1).

The second boundary condition is determined by the presence of three other neighbouring catchments that are reserved for future development. As shown in Figure 1, two planned land developments (8 and 3.7 ha) lie between the study area (red hatch) and the watercourse, with the other one (19.6 ha) being located at a higher ground to the east. For this reason, the drainage system had to be designed to convey stormwater flows from the wider catchment with a total contributing area more than 45 ha.

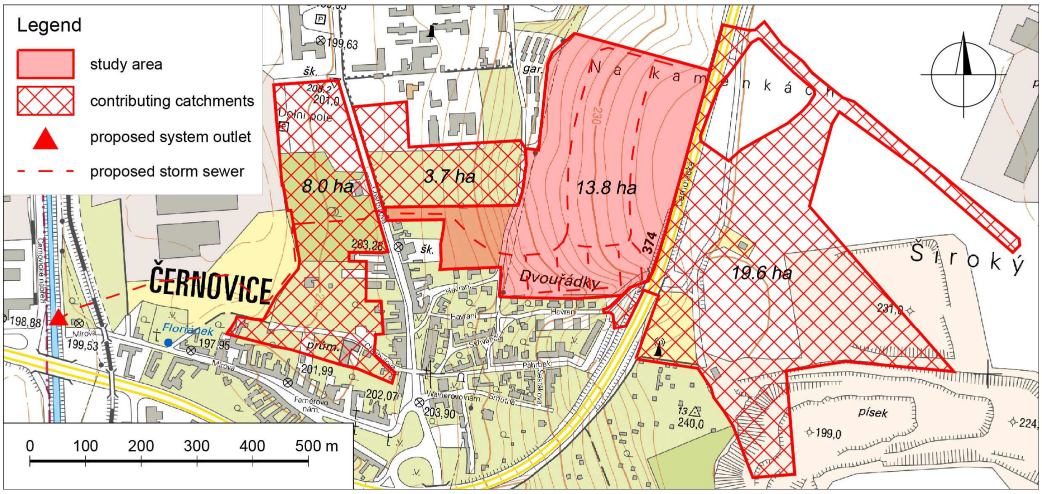

Figure 1. Study area and contributing catchments

2.3 Preliminary (original) design

A typical abatement plan applied by most practitioners in the Czech Republic is to design a detention facility at the outlet of the catchment in order to provide storage for reducing the peak flow on required value. The design provided by the consultancy is a combination of conventional drainage with a central detention facility and decentralized source control measures distributed in the study catchment.

Any public areas consisting of roads, pavements, or lawns are planned to be drained directly through the conventional piped system that creates the backbone of the storm sewer network for the study area and other contributing catchments. In order to comply with the outflow threshold limit, the conventional drainage system has been amended by the oversized sewer (DN2400, length 61 m, storage 244 m^3) located at the outlet of the study catchment. The flow from this on-line detention facility is controlled by a vortex valve. An emergency overflow is provided by a weir that operates at higher flow-rates resulted from extreme rainfalls as well as from the stormwater inflow from the upper catchment.

Pervious and impervious surfaces such as buildings, gardens, or car parks belonging to private properties are connected to the drainage network via source control measures (bio-retention cells and geocellular units). These decentralized devices were designed with respect to the outflow limit set for the whole catchment, i.e. the maximum allowed discharge from a single unit is 10 l/s per hectare of drained surface (min. 0.5 l/s), while the storage capacity should not be exceeded for any design storm with the return period up to 5 years.

2.4 Model setup

The simulation software package selected for this study was the EPA Storm Water Management Model (SWMM). Due to the fact that the design represents a drainage system of a new development with no calibration (measured) data available, all the model parameters had to be estimated either from information included in the consultancy design, the SWMM manual, existing literature or similar case studies (Tsihrintzis & Hamid 1998; Rossman 2010; City of Edmonton 2011; Rossman 2014).

The very first step in the model setup was building a conceptual model divided into sub-catchments that represent the study area itself as well as other contributing catchments. Because the input data had not contained any specific information about the imperviousness, land-use, or drainage network of contributing catchments, these were in the model simplified as nodal inflows. It was assumed that the runoff from these catchments would be limited to the threshold value of 10 l/s per hectare. A constant baseline inflow was hence determined as multiple of their areas and the discharge threshold value.

The next step in the data pre-processing for the runoff modelling in SWMM was the segmentation of the study catchment into sub-catchments. The objective of catchment delineation was to identify sub-catchments with more or less homogenous characteristics based on land-use, slope, and the different runoff conditions. The data about the future project had been provided by the consultancy. These inputs were analysed and processed with the use of AutoCAD and AutoCAD Civil 3D respectively. The study catchment was delineated into 81 sub-catchments. The segmented areas range from 0.014 to 0.725 ha. Their characteristic widths, which in the model represent the overland flow paths for sheet flow runoff, were estimated according to the manual's recommendation.

The percentage of impervious surfaces in each sub-catchment (i.e. roofs, pavements, roads, and car parks) was derived from the land-use analysis. The average impervious rate of the developed study area expected to be 44%. Based on the topology analysis of the proposed design it was identified that the average slope would be reduced to 4.46%. Apart from information about the area, width, imperviousness, and slope, each modelled sub-catchment contains data about hydrologic response parameters such as the Manning's coefficients for overland flow, depression storage, and infiltration (Table 1).

Table 1. Hydraulic response parameters

Parameter	Selected Value
Manning's coefficients	
Impervious surfaces	0.013
Pervious surfaces	0.150 (0.250)
Depression storage	
Impervious surfaces (mm)	1
Pervious surfaces (mm)	2.75 (4.25)
Infiltration	
Suction Head (mm)	219.96
Conductivity (mm/hr)	3.6
Initial Deficit (fraction)	0.181

The pervious surfaces in the model are divided into two categories – lawns and extensive lawns. Pervious area infiltration is based on the Green-Ampt method. This infiltration method was selected after a practical consideration as it requires minimum parameters. The input data for this method were obtained from a hydrogeological survey report provided by the consultancy. Soil saturated hydraulic conductivity was derived directly from the survey report, whereas soil capillary suction and initial moisture content were identified from the soil characteristic table in the SWMM manual.

One-factor-at-a-time sensitivity analysis was performed to screen pre-selected model parameters and identify those with significant effects on the peak flow. The results suggest that the soil infiltration parameters were the most sensitive. The parameters include the saturated hydraulic conductivity, the initial moisture content, and the capillary suction head.

The spatial distribution of drainage network and the physical parameters of conduits in the model were both based on the design proposed by the consultancy. The conventional stormwater conveyance system is made up of storm sewers (concrete), oversized sewer (glassfiber reinforced plastic), and manholes (precast concrete rings). The pipe drainage system was designed with the use of the Rational Method and to convey the surface runoff resulting from a design storm with a two-year return period and duration of 15 minutes (i.e. 161 l/s/ha). Pipe geometries were sized to meet the criteria defined by the local standards for sewerage and drainage systems.

As described earlier, original design combine a conventional drainage system with source control measures (Low Impact Development practices). Although controlled soakage of stormwater into the ground is not possible due to the site hydrogeological conditions, LID facilities were implemented into the design to collect, treat, store, and then slowly release stormwater to the drainage system. LID components include 34 bio-retention cells with the total area of 830 m^2 and the total storage volume of 249 m^3; 80 geocellular units with the total storage capacity of 959 m^3. The total combined storage provided by these decentralized facilities is hence larger than 1,200 m^3.

In the model the proposed LID devices are presented as bio-retention cells and rain barrels respectively. While bio-retention cells are made of surface, soil, storage and underdrain layer, rain barrels only have storage and underdrain layer. Because LID controls in the proposed design are not linked in a "train treatment" configuration, it had been decided to place them into the existing sub-catchments where they can act in a parallel. The limiting aspect of this approach is the need for the adjustment of sub-catchments properties in terms of their relative imperviousness and width.

2.5 Design alternatives

The first alternative (Design A) was derived from the original design (Design 0). The system layout remained the same apart from the central detention unit and pipe diameters. The main purpose was to identify an appropriate storage volume of detention unit as well as the pipe diameters with the use of model-based approach. The oversized sewer was hence replaced by an underground concrete tank, size of which could have been adjusted more easily. The maximum design flow-rate of vortex valve was increased in order to fully utilize the storage volume provided by the detention tank. All other parameters of the design, including the distribution and the volume of source control measures, have remained the same.

The second alternative (Design B) reflects the most common approach utilized in different variations by many practitioners. It consists of a conventional separate drainage system and a detention facility located at the outlet of the study catchment. The outflow from the central storage unit is controlled by a vortex valve of the same setting as presented in Design A. However, the design does not contain any source control measures.

In the last scenario (Design C), the study catchment is drained by a conventional separate sewer system discharging stormwater directly to the watercourse. The maximum discharge from the development is not restricted to reduce the risk of flooding in the stream. Although Design C does not comply with the outflow threshold set by local authorities, this scenario has been included in the study for purely academic purposes, comparing the current practice with already built drainage systems.

Table 2. The overview of all designs in terms of their representation in the SWMM model

	Design 0	Design A	Design B	Design C
	Design provided by the consultancy with oversized sewer and source control measures	Adjusted original design with on-line detention tank and source control measures	Conventional drainage with on-line detention tank	Conventional drainage without any detention facilities
Junction Nodes	68	66	66	67
Storage Facility	Oversized sewer	Detention tank	Detention tank	-
Conduit Links	67	66	66	67
Storage Facility	Oversized sewer	Detention tank	Detention tank	-
LID Units	34 bio-retention cells and 80 geocellular units	34 bio-retention cells and 80 geocellular units	-	-

Because the Design scenarios A, B, and C had been developed directly from the Design 0, it was necessary to adjust their hydraulic capacity for the required level of service. The design basis set by the local standards provides a general guidance on the type of rainfall event to use for assessing the hydrologic performance of individual system elements (i.e. storm sewers, storage facilities). Table 3 presents the design basis applicable to all stormwater management elements in Designs A, B, and C.

Table 3. Design basis for individual system elements

System element	Performance standard	Design scenario
Storm sewers upstream of the detention tank (conventional drainage system)	2-year return period	A, B, C
Central detention tank	5-year return period	A, B
Storm sewers downstream of the detention tank (conveyance of transformed flows)	5-year return period	A, B

2.6 Rainfall data

In this study the effects of different stormwater schemes on the hydraulic capacity of drainage systems were investigated under various types of storm events. The storms considered in the study can be classified into three groups:

Group I – The first group consists of 2-year return period storm events with durations from 10 to 120 minutes and the rainfall depths range from 12 to 23 mm. The storm events in Group I were used for the adjustment of pipe sizes in Design scenarios A, B, and C.

Design storms based on standard IDF (intensity – duration – frequency) curves are commonly used for various purposes in most parts of the world. However, as pointed out by Berggren et al. (2014), block design storms with a uniform distribution are more suitable for evaluating the hydraulic capacity of existing systems, whereas a varying intensity storm profile is more often recommended for designing new systems. Because the rainfall data in Group I were intended for the pipe adjustment in order to determine the diameters appropriate for the required level of service (i.e. for design purposes), their distribution was transformed from a uniform to a varying intensity storm profile.

There are several methods (e.g. CDS, the Chicago Design Storm) of creating design rainfall from IDF curves or measured rainfall data. The most common approach, applied by many practitioners in the Czech Republic, probably is the method proposed by Sifalda (1973). Sifalda's design rainfall has a "triangular" shape and consists of three parts – the prior-rainfall (14%), the main rainfall (56%), and the after-rainfall (30%). Its construction is relatively simple, as input data for the procedure can be obtained directly from IDF curves or from any other block rainfall with given intensity and duration.

Group II – This group of rainfall data consists of storm events with a 5-year return period and durations varying from 30 to 360 minutes. The block rainfalls in Group II were used for the adjustment of storage of central detention facilities (Design A and B) as well as for the LID performance testing (Design 0 and A).

Group III – The design rainfalls in the third group were employed for the evaluation and assessment of hydraulic capacity of drainage systems (Designs 0, A, B, and C). The storm events have different return periods (2, 5, 10, 20, 50, and 100 years), and durations varying from 5 to 60 minutes. The corresponding total rainfall depths range from 8 to 56 mm.

2.7 Model simulation and evaluation criteria

Once the system elements of design alternatives had been adjusted for the required level of service, each model was analysed in terms of the peak flow-rates, filling and emptying of detention facilities, and the overall system behaviour. In following simulations, it was studied how the different design scenarios perform under extreme or onerous rainfall conditions. The models were loaded with various design storms as described in previous chapter (see Group III), and their hydraulic capacity was assessed with a wider range of performance criteria. The main purpose of this analysis was the comparison of different system designs. Therefore the performance indicators were aggregated together for further assessments.

When comparing different competing alternatives which are otherwise equally appropriate to be implemented on technical grounds, it is a common procedure to determine the most cost-effective option with a life cycle cost analysis. Due to a lack of representative data (mainly related to LID controls) this case study only includes the estimation of construction and material costs. The cost estimate presented in the study is based on the bottom-up approach that relies on unit cost data. Typical unit costs were derived from the official price lists and from the costs estimates that were included in the project proposal provided by the consultancy.

3 RESULTS

3.1 LID performance

As mentioned earlier, LID controls in Designs 0 and A treat runoff from nearly 75% of the catchment impervious surfaces. Owing to this, their correct setup and performance was crucial for accurate simulations and subsequent comparisons. The behaviour of modelled LID facilities was studied in terms of their filling, emptying, and adequate storage capacity determined during the design stage. The initial tests were run for randomly chosen LID units with an unalike storage capacity representing both types of applied source control measures.

During the preliminary testing it was identified that the geocellular units affected sub-catchment runoff more than it had been expected. Because geocellular units (underground plastic boxes) are in the SWMM model represented as rain barrels that treat inflow from the connected impervious surfaces as well as direct rainfall, it was necessary to adjust their surface areas and design depths so that they replicated their behaviour in the real world. In other words, the surface area of individual LID controls was reduced to one square meter and corresponding depths were increased.

Selected geocellular units were tested in terms of the percentage of filled storage volume for different durations of design rainfall events with 5-year return period. The maximum filling reached around 95% of overall units' storage capacity. These results indicate that the storage volumes of analysed units were designed appropriately as they are not excessively oversized and do not overflow during any design rainfall event. Simulations also confirm that the design assumptions regarding relevant critical durations were correct. In contrast, the storage volumes provided by bio-retention cells are not utilized as effectively as in case of geocellular units. Their storage capacity is on average filled just from 80% during the design rainfall with critical durations of 120 minutes, and it decreases down to 60% for other analysed durations. It is very likely due to the safety infiltration coefficient applied when designing bio-retention cells.

3.2 Inflow hydrograph

During the initial testing of sewer network it was found that the constant nodal inflow of 196 l/s simulating the runoff from the upper catchment (19.6 ha) was causing high peak flows downstream of the central detention unit. This problem occurs when the on-line detention facility is nearly full and the runoff from the study area still at its peak value. Constant baseline inflow then causes that the remaining storage capacity of the central detention unit, which would otherwise be sufficient, is filled much earlier. In consequence, the peak runoffs from the study area and the upper catchment are combined, which results in high peak flows downstream of the detention unit.

Hence, the simplified baseline inflow could only be used for the contributing catchments that were connected to the drainage system downstream of the on-line unit. The constant inflow from the upper catchment had to be replaced with a flow hydrograph replicating the situation more realistically. Due to the lack of data about the future development at the upper catchment it had been decided to derive the inflow hydrograph from the outflow hydrograph of detention tank in the study area. This hydrograph was obtained from Design A and it consequently was adjusted to simulate the inflow from the upper catchment. Even though the derived hydrograph is not ideal or completely realistic, it still is a better approximation of a real behaviour than the original constant baseline inflow.

3.3 Analysis of the original design

The conduits upstream of the oversized sewer indicated no surcharge for the design block rainfalls with return period of 2-years and performed acceptably well even after the model was loaded with synthetic rainfalls of the same return period, only with varying intensity. This suggests that the original estimation of the pipe sizes based on the Ration Method was sufficient and reasonably accurate. Upstream nodes in the system in general were more affected by rainfall of high intensity, whereas for downstream nodes closer to the main outlet the rainfall volume was more critical.

The storage capacity of the oversized pipe (244 m^3), on the other hand, did not prove to be sufficient for any rainfall event with a 5-year return period, and its design thus does not meet requirements set by the local standards. Alarmingly, the sewer capacity was exceeded even during 2-year return period storm events. In order to test whether the problem only lied in the flow control device installed at the outlet, the maximum design flow-rate of the vortex valve was increased to the value used in Designs A and B, i.e. to 138 l/s. As expected, the new vortex regulator substantially improved the performance of the oversized sewer. Nevertheless, its storage capacity was still too small for 5-year return period events. In comparison to the adjusted storage unit in Design A that has an effective storage volume of 340 m^3, the capacity of the oversized sewer is about 100 m^3 smaller.

To sum up, the hydraulic behaviour of the drainage system is negatively influenced by two major factors. One of them clearly is the underestimated design of the detention unit in terms of its capacity and outflow control; the other aspect is the inflow from the upper catchment. It was found that the high peak flows in the system occurred even after the inflow hydrograph with a varying flow-rate had been adopted. It is a question whether such a situation would appear in the real system since it is induced by various elements and specific conditions.

It is apparent that without the application of additional measures it is very difficult to comply with the peak flow criterion at the main outfall even with the increased storage volume of the central detention tank. Therefore, this problem cannot simply be addressed by one-off design of the drainage system, or, more precisely, it can but only with enormous expenditures. An alternative option would be to approach the problem in two separate stages. Firstly, it would be advisable to adopt the proposed changes of the central detention unit. By the time the upper catchment is about to be developed, the maximum controlled flow-rate at the outlet of the storage tank can be increased from original 138 to 334 l/s (i.e., 138 plus 196 l/s).

3.4 Design comparisons

During the aforementioned analyses it was established that the peak flow limit at the main system outlet could only be accomplished in a cost-effective way with the multi-stage approach. Because of that, the rest of the evaluations presented below were carried out without the inflow from the upper catchment. Besides the peak flows, which traditionally are considered the main indicators of hydraulic performance, this study explored a wider range of response criteria. The designs were evaluated in terms of their overall hydraulic capacity, which might in certain aspects be a better indicator of possible impacts on the urbanized catchment than the peak flows. The performance criteria included the numbers of surcharged conduits and flooded nodes, the total flood loss, and the maximum water level in nodes related to the ground level. The obtained results were aggregated for individual return periods that varied from 2 to 100-year.

The first analysed performance indicator is related to the numbers of surcharged conduits in the system. Even though this criterion is not absolutely predicative in terms of the system capacity, it is the most sensitive parameter describing the occurrence of pressurized flows. From the Figure 2 (a) it can be seen that Designs B and C (i.e. designs in a sense conventional) response very similarly to different return periods, and compared to Designs 0 and A they perform substantially better during higher return periods.

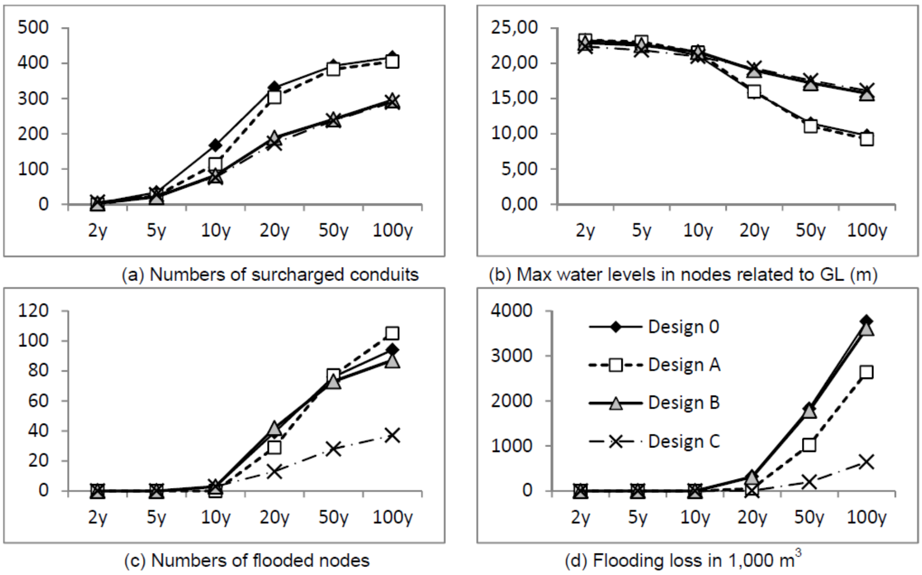

Figure 2. Hydraulic performance of analysed designs

A more indicative response criterion considering the capacity is the maximum water level across all system nodes. As shown by the data in Figure 2 (b), the free available capacity in the system decreases with higher return periods. Surprisingly, the overall safety margin in the drainage networks with LID controls (Designs 0 and A) on average is depleted more than in case of the conventional drainage systems. However, it does not completely reveal whether the system performs better or not, as far as the potential impacts on drained properties or surrounding area go.

A more reliable indicator would probably be the number of nodes in the system where maximum water level exceeds certain thresholds such as the ground level or any other critical level. Figure 2 (c) displays the numbers of flooded nodes in the system, i.e. the number of nodes where the maximum water level reached the surface. Apart from Design C, the numbers of affected nodes are very similar for each of the return periods. Moreover, it appears that rainfall events with return period of 20 years or higher cause a sudden growth in the quantity of flooded nodes.

This fact is visible even more in Figure 2 (d) that demonstrates the total flooding loss. From this perspective, Design C represents the best protection against flooding during rainfall events with a 50 and 100-year return period, but only at the cost of shifting the problem to different parts of the catchment. In terms of flood control, Design A performs reasonably well up to a 20-year return period, and even then it handles potential flood volumes better than Design 0 or B.

In conclusion, conducted evaluations demonstrate an obvious increase in hydraulic response with increasing return period. The differences in performance between analysed designs of a maximum of

10-year events are minimal, but then they increase rapidly along with the rainfall return period. Additionally, the findings presented above imply that when evaluating the system's hydraulic capacity with various criteria we can get more comprehensive information on the real behaviour and performance. For instance, the results provided in Figure 2 (a) and (b) suggest that Design B performs substantially better than Design A during extreme rainfall conditions; however, in terms of the flood volume (Figure 2 (d)) it is other way around, even though both designs originally were adjusted for the same level of service.

Besides hydraulic performance, the design alternatives were evaluated from an economic point of view. Figure 3 provides an overview of estimated material and construction costs that are classified according to particular design items and the main system components. The results indicate that the expenses related to Designs 0 and C are comparable. It confirms a growing body of evidence in the literature that suggests that source control measures, such as LID controls applied in this study, can lead to the reduction of overall stormwater management costs while ensuring better protection than traditional drainage (e.g. Braden & Ando, 2012; Duffy et al., 2008). In comparison to the original design provided by the consultancy, the adjusted Design A has slightly higher costs which can be mainly attributed to the central detention facility that is represented in the model as an underground storage tank. An oversized sewer of the same storage capacity in a place of the concrete storage tank would reduce reported costs by 10%, i.e. from 71.8 to 64.2 million CZK.

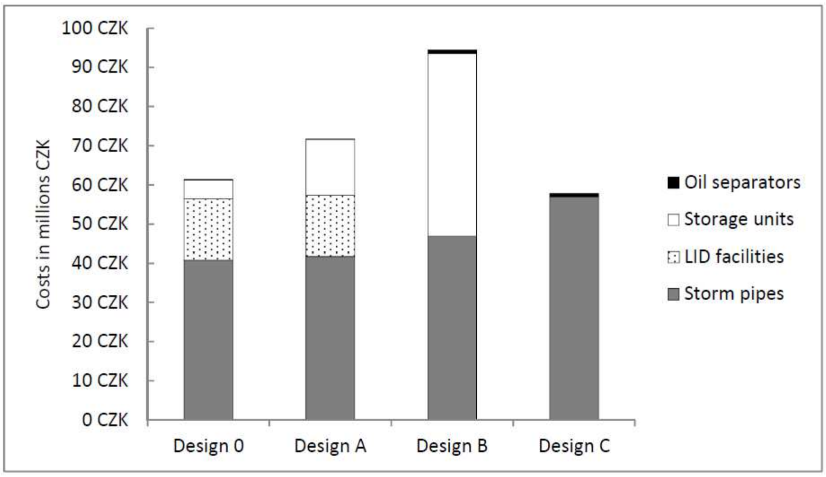

Figure 3. Overview of material and construction costs

Design B can be considered the most expensive alternative out of all. This finding is worrying, given the fact that such a system layout is commonly adopted in practice by many urban drainage engineers in the Czech Republic. Nevertheless, it should be noted that estimations presented above do not include planning, design, operation, or maintenance costs. These costs could possibly be expected higher for alternatives that include LID controls. However, the difference should not be that significant as LID operation and maintenance costs would be evenly divided between several subjects (i.e. the city council, the sewer undertaker, and property owners).

From the comparison of Designs A and B it can be concluded that the application of source control devices can lead to cost savings which could be significant, at least in terms of material and construction costs. The literature contains several studies showing a similar trend in cost reduction where a distributed storage in the catchment is applied (see for example Andoh & Declerck, 1997). Therefore, it can be assumed that a drainage system based on a completely decentralized upstream control would be even more cost-effective than the designs analysed in this study.

3.5 Conclusion

The main purpose of this study was to analyse and compare the performance of different stormwater management options for a future development based on the original design proposal provided by the consultancy. In order to investigate the potential measures a rainfall-runoff simulation model was built and alternatives were compared in terms of their hydraulic performance and construction costs.

The analysis of the original design found that the dimensions of the piped drainage system had been identified with a reasonable accuracy, and apart from a couple segments downstream of the central detention unit they complied with the required level of service. Similarly, the model simulations proved the applied LID controls (i.e. bio-retention cells and geocellular units) to have been designed correctly as far as their filling, emptying, and storage capacity are concerned.

The main shortcoming of the original design could be seen in the inadequate storage capacity of the oversized sewer, which serves as a central detention facility for stormwater runoff from public spaces. Moreover, the study revealed that the original design had fundamentally underestimated the maximum hydraulic capacity of a flow control device that regulates the outflow from this detention unit. Additionally, it was established that the peak flow limit at the main system outlet could only be accomplished in a cost-effective way with the multi-stage approach. The proposed procedure includes the adoption of other design measures that would be applied within the drainage system once one of the contributing catchments in the area had been built up.

Alternative stormwater management scenarios developed from the original proposal were adjusted in order to meet the design basis for conveyance systems and storage facilities set by the local standards. Consecutive assessments confirmed that the impact of a model-based adjustment procedure on the hydraulic performance was significant which was especially apparent when compared with the peak-flow rates at the main system outlet.

Besides the peak flows, the study also explored other response criteria related to the overall hydraulic capacity of drainage systems. The performance criteria included the numbers of surcharged conduits and flooded nodes, the total flood loss, and the maximum water level in nodes related to the ground level. The differences in the performance between the analysed designs were minimal for storms with return periods up to 10 years. Rainfall events with higher return periods (20, 50, and 100-year) did not only cause a dramatic increase in the hydraulic response, but they also more clearly pointed out differences between individual designs. The results indicate that the different drainage layouts can perform differently under extreme rainfall conditions, even if the system components are adjusted for the same level of service.

More general findings and problems encountered during the model-based design or the assessment of hydraulic capacity of urban drainage systems can be summarized as follows: (1) Drainage systems with source control measures are viable alternative to the conventional approaches towards stormwater management; (2) In comparison to centralized downstream control, decentralized distribution of storage capacity within the catchment can lead to material and construction cost savings; (3) Evaluating a drainage system from different perspectives with various performance indicators may result in a better understanding of its overall hydraulic performance; (4) There is no clear guidance on model-based design or evaluation of new urban drainage systems.

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