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# Urban Inundation Simulation Incorporating Sewerage System without Structure Effect

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**Synopsis**

The main difference between rural and urban area inundation is the existence of an impermeable layer and the complexity of the road and building configuration. Therefore, although surface overland flow can be appropriately simulated by the two-dimensional diffusive model in a rural area, there also is a need for a more accurate and elaborate simulation model. In this study, we simulated homogeneous and non-homogeneous mesh which involves road network to estimate the effect of road networks in urban inundation analysis. Two-dimensional run-off flow model and one-dimensional slot-model simulated ground surface run-off flow and sewer pipe flow. To connect both models, we used a newly suggested bi-directional model and its coefficients (LEE et al., 2013). The simulated results showed a non-homogeneous case could calculate more reasonable results; the results further reveal the importance of using storm drains as exchange spots in the model.

**Keywords**: urban inundation, flooding, sewerage system, storm drain

## 1. Introduction

Urban inundation due to climate change and torrential rainfall are an inevitable problem for many cities around the world. For the case of rural areas such as agricultural fields, surface overland flow can be appropriately simulated by the two-dimensional diffusive model based on non-inertial surface flow dynamics (Hromadka and Lai, 1985; Wasantha Lal, 1998) because the surface overland flow processes are primarily determined by the topography, land cover, and soil type. On the other hand, the surface overland flow process in urbanized areas is determined by artificial structures such as building configuration, road networks, and drainage systems (Hsu et al., 2000).

The dual-drainage concept (Smith, 1993; Djordjević et al., 1999; Mark et al., 2004; Nasello and Tuciarelli, 2005) describes the bidirectional flow interactions between the ground surface and sewerage system. Runoff flow on the ground surface and drainage flow could be calculated simultaneously based on the dual-drainage concept. Much of the attention of flood inundation researchers has concentrated on benchmarking model codes against two-dimensional hydraulic models, including DIVAST, DIVASTTVD, TUFLOW, SIPSON and SWMM (Hunter et al., 2008).

There are two main reasons for urban inundation. The first is a lack of sewer system capacity, and the second is an insufficiency of capturing capacity of storm drains (Lee et al., 2013). For instance, if the flow rate in a sewer pipe exceeds the maximum capacity of the pipe, the surface runoff flow will not drain into the sewer

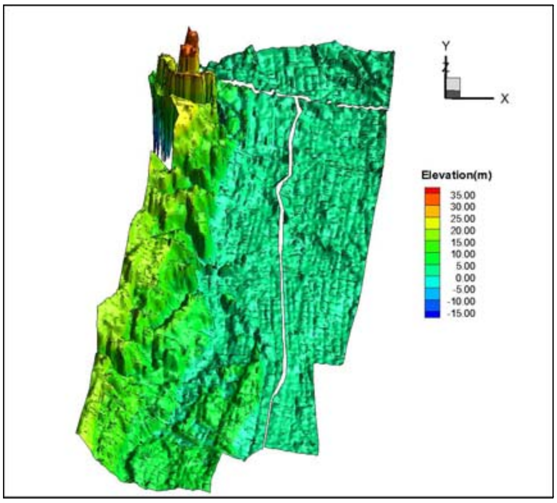

Fig. 1 3D view of study area

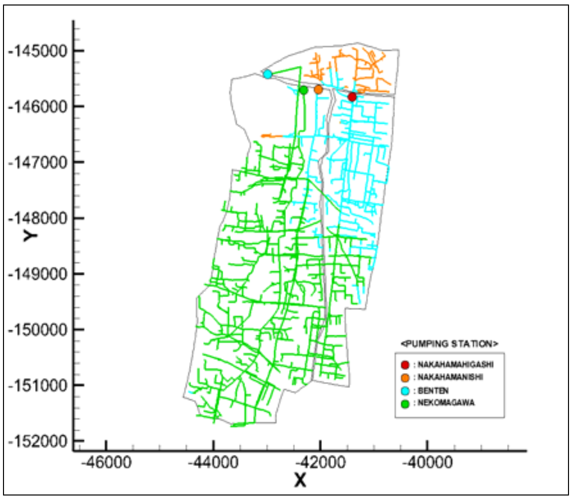

Fig. 2 Pipe connectivity analysis result

system through the storm drains and overflow may occur. On the other hand, if the flow rate does not exceed the maximum capacity, the surface flow can be captured and drained through the storm drain. It is clear that lack of sewer system capacity causes urban inundation. However, it is somewhat difficult to imagine there may be an inundation in the latter, caused by lack of capturing capacity of storm drains, but it is possible. Although the designed capacity of a sewer system is enough to carry flooding discharge, if the discharge on the ground surface cannot be captured by the storm drains, ground surface inundation may occur (Hsu et al., 2000).

Generally, overland flow direction is decided by gravity's direction along the road, but it can be changed by anthropogenic structures such as houses, buildings, curb, and levees. Urban inundation can limit or completely obstruct the functioning of traffic systems and has indirect consequences such as loss of business and economic opportunities in addition to possible social consequences, the expected total direct and indirect losses are related to the physical properties of the inundation: depth, area, and duration.

There are several techniques to consider the blockage effect of buildings in urban areas by using; (1) local friction-based representation of buildings; (2) the bottom elevation technique; and (3) vertical walls (Chen et al., 2008). McVillan and Basington (2007) have also tested the capabilities of building resistance, building blocks, building holes, and building porosity methods in urban inundation simulation and found the sub-grid treatment is needed for the parameterization of coarse grids in certain cases. However, road networks are predominant factor to decide flow direction before the runoff flow depth can exceed the height of curb.

Our research group has been making an effort to develop an elaborate urban inundation model that can simulation run-off flow on the ground surface, the drainage process in a sewerage system and bi-directional interactions between the sewer system and the ground surface simultaneously. A number of models (SWMM, SIPSON, UIM etc.) have been proposed for urban inundation using the manhole as an exchange spot based on dual-drainage system because of numerical convenience. Generally, exchange discharges between ground surface and sewerage system are calculated at the manhole based on dual drainage concept for convenience of numerical treatment. However, Lee et al., (2012; 2013) have emphasized the importance of storm drains as exchange spots of bi-directional interactions to approximate reality. The discharge exchanges are occurred at storm drains in real situation except some special cases such as manhole cover blow-up, clogged storm drain cover, burst of sewer pipes etc. Therefore, it is necessary to consider the storm drains are interaction spots for developing elaborate urban inundation simulation.

This study aims to develop and estimate integrated urban inundation model for considering the effect of storm drains as an exchange spot for calculating bi-directional discharge at road networks.

In this study, we divided meshes into two

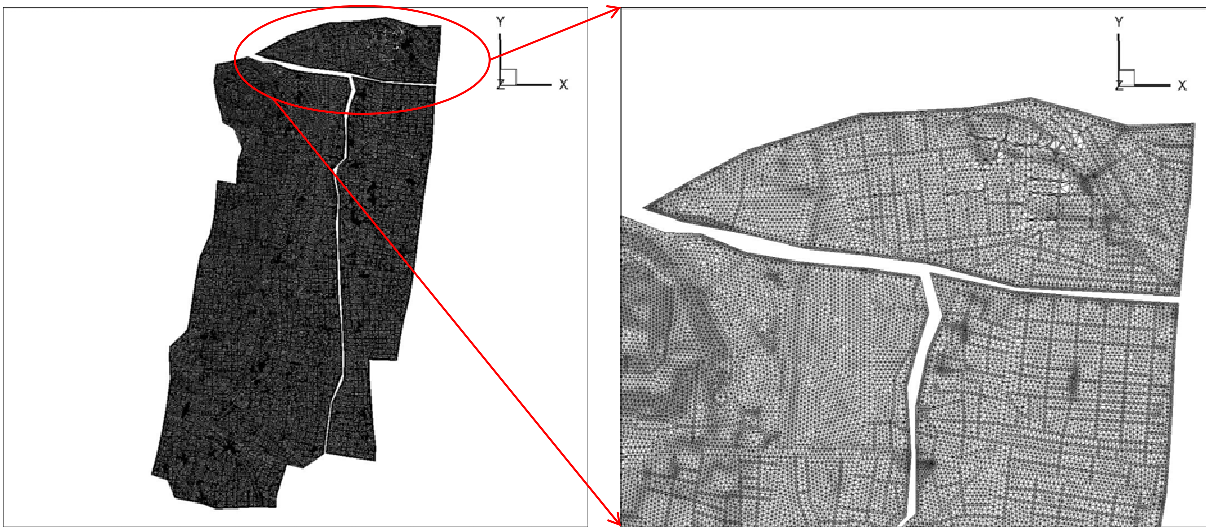


Fig. 3 Generated mesh of study area

categories, road networks (non-homogeneous) and normal meshes (homogeneous) which reflect only elevation data; we artificially adjusted the elevation (-20cm) of the road meshes to induce flows from normal mesh to road networks and the simulation results are compared with a non-adjusted mesh case. We used a two-dimensional (2D) run-off flow model (Kawaike et al., 2002) and a one-dimensional (1D) slot-model to simulate surface run-off and sewer pipe flow, respectively.

To evaluate the model applicability, we selected the Nakahama area of Osaka, Japan and used

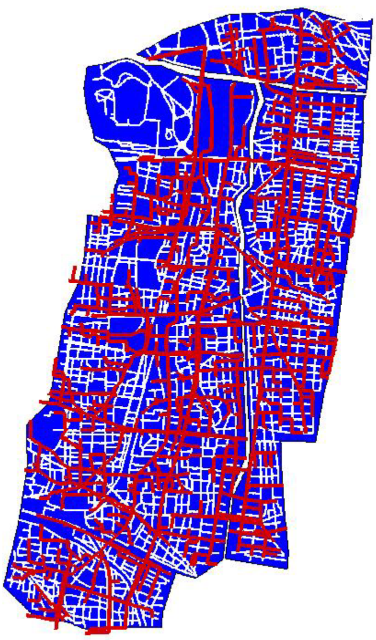

Fig. 4 Arrangement of sewer pipes and storm drains

rainfall data from 27 August, 2011 as a study case. The precipitation data and pumping station maximum capacities were the boundary conditions.

## 2. Delineation of Study Area

We carried out simulations for the recent severe urban flooding in the Nakahama area of Osaka, Japan on 27 August 2011. The study area is approximately 18.1km$^2$. The Nakahama area is highly urbanized and geographically east part is higher than west. Fig. 1 shows the 3D view of study area. As stated above, the elevation of edge of left side is about 20m while right side is averagely lower than 5m. The interesting thing about topography of study area is that it can be divided as three segments by the rivers so that each segment is blocked by levees. The levees can safeguard each inland against overflow from the river. Therefore, the study area is relatively less dangerous from overflow flooding. However, the levees paradoxically make the drainage process difficult to be naturally drained.

There are four pumping stations in order to artificially drain the storm water through the sewer pipes. Basically, the sewerage system is composed of sewer pipes, manhole, storm drains and pumping stations. Properties of the data such as position of each components, slop, shape and diameter of pipes, were obtained from the Osaka city. There are 4,445 pipes and 4,317 manholes. It is necessary to confirm the connectivity analysis of manholes and pipes because the data are involving uncertainty. To conduct the connectivity analysis, 'flow direction trace' work was carried out. All pipes have upstream and downstream manholes so that it is possible to trace flow direction from the pumping station. The numbers of sorted data, which can flow the storm water to pumping station are 3,026 pipes 2,903 manholes, respectively. Fig. 2 shows the result of the analysis. It looks like the each pipe has one outlet pumping station, but most pipes can flow the storm water to all pumping station, not specified pumping station.

## 3. Mesh Generation

We obtained the GIS data of the road network was obtained from the Geospatial Information Authority of Japan (http://www.gsi.go.jp/index.html). We used 5m×5m Digital Elevation Model (DEM) data as the mesh elevation data. To generate unstructured triangular mesh, we employed the GID (11.0.5 version) software. The aim of this study is to develop and estimate integrated urban inundation model for considering the effect of storm drains at road networks. Therefore, two kinds of meshes are needed. The first one is a homogeneous mesh as comparison group. This mesh has 20m mesh size without any artificial structure. The second mesh involves road network configuration. Fig. 3 shows the mesh of second case. In addition, storm drains are generated on the road network meshes as exchange spot to calculate bidirectional discharge between ground surface and sewer pipes. Fig. 4 shows the arrangement of sewer pipes and storm drains. In this figure, red line means sewer pipes and white square means generated storm drain on the road networks meshes.

## 4. Numerical Model

We used two kinds of flow models (2D run-off flow and 1D pipe flow) and interaction models (an exchange discharge model between the ground surface and sewer pipes, and an exchange discharge model between the manholes and sewer pipes) to simulate the study area. The run-off flow model used in this study consisted of a horizontal 2D inundation flow model (Kawaike et al., 2002) and a 1D slot model of sewer pipe flow (Chaudhry, 1979), while the interaction models were composed of the weir and orifice formula to calculate exchange discharge at the storm drains and the adjoining part between manholes and pipes (LEE et al., 2013).

### 4.1 Two-Dimensional Inundation Model for Surface flow

The governing equations used for the 2D inundation flow model are as follows:

$$\frac{\partial h}{\partial t}+\frac{\partial M}{\partial x}+\frac{\partial N}{\partial y}=r_e-q_{ex} \tag{1}$$

$$\frac{\partial M}{\partial t}+\frac{\partial (uM)}{\partial x}+\frac{\partial (vM)}{\partial y}=-gh\frac{\partial H}{\partial x}-\frac{gn^2M\sqrt{u^2+v^2}}{h^{4/3}} \tag{2}$$

$$\frac{\partial N}{\partial t}+\frac{\partial (uN)}{\partial x}+\frac{\partial (vN)}{\partial y}=-gh\frac{\partial H}{\partial y}-\frac{gn^2N\sqrt{u^2+v^2}}{h^{4/3}} \tag{3}$$

where $h$ is water depth; $H$ is water elevation; $u$ is x-direction water velocity; $v$ is y-direction water velocity; $M$ is $uh$ (x-direction flux); $N$ is $vh$ (y-direction flux); $r_e$ is effective rainfall; $q_{ex}$ is the interaction discharge between the ground surface and sewer pipe; $g$ is gravity acceleration; $n$ is Manning's roughness coefficient; and $t$ is time. Computational meshes are unstructured and triangular in shape; we adopted the Finite Difference Method.

### 4.2 One-dimensional Sewer Sewerage system Model

The 1D flow simulation with slot model simulated the flow within a sewer pipe. The governing equations are as follows:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q_{ex} \tag{4}$$

$$\frac{\partial Q}{\partial t}+\alpha\frac{\partial (uQ)}{\partial x}=-gA\frac{\partial H_p}{\partial x}-gn^2\frac{|Q|Q}{R^{4/3}A} \tag{5}$$

where $A$ is the wet area of the cross section; $Q$ is flow discharge; $q_{ex}$ is the lateral inflow discharge per unit of pipe length; $\alpha$ is a reduction factor of the convective acceleration term (Djordjević et al., 2004); $u$ is velocity; $R$ is hydraulic radius; and $n$ is the roughness coefficient (we used $n$=0.012 for the sewer pipe in this study). $H_p$ is the piezometric head

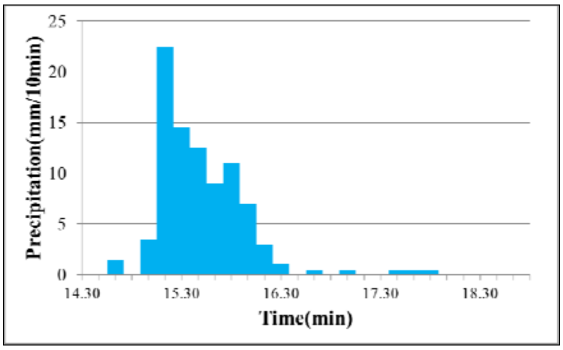


Fig. 5 Precipitation data

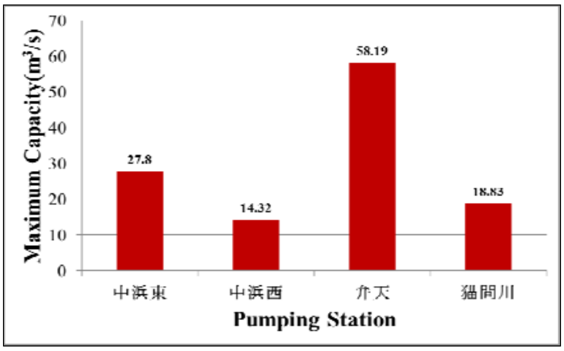


Fig. 6 Pumping station capacities

($H_p$ = $Z_p$ + $h$); $Z_p$ is the bottom elevation of the sewer pipe; and $h$ is water depth determined as follows:

$$h = \begin{cases} f(A) & : A \le A_s \\ D + (A - A_s)/B_s & : A > A_s \end{cases} \tag{6}$$

where $f$ is a function of the relationship between water depth and wet area of the cross section of a circular pipe; $A_s$ is the cross-sectional area of the sewer pipe; $D$ is the pipe diameter; and $B_s$ is slot width, determined as follows:

$$B_s = \frac{gA_s}{a^2} \tag{7}$$

10.0$m/s$ represented the pressure wave speed $a$ of the sewer pipe.

The manhole continuity equation can be written as:

$$\frac{\partial h_m}{\partial t} = \frac{\sum_{i=1}^{M} Q_i}{A_m} \tag{8}$$

where $h_m$ is the water depth of the manhole; $Q_i$ is the exchange discharge between the manhole and connected pipes and manhole; and $M$ is number of the connected pipes.

The water depth of pumping station is calculated as:

$$\frac{\partial h_{pm}}{\partial t} = \frac{Q_{pm}}{A_{pm}} \tag{9}$$

where $h_{pm}$ is the pumping station water depth; $Q_{pm}$ is the drainage capacity of the pumping station; and $A_{pm}$ is the bottom area of pumping station.

**4.3 Interaction Model**

There are two interaction models: between the manhole and sewer pipe, and between ground surface and sewer pipe, to calculate exchange discharges.

(1) Manhole and sewer pipe

(a) In the case of $h_s \ge h_m$

$$Q = \begin{cases} 0.35 \times A_s \sqrt{2gh_s} & : h_m / h_s \ge 2/3 \\ 0.91 \times A_m \sqrt{2g(h_s - h_m)} & : h_m / h_s < 2/3 \end{cases} \tag{10}$$

(b) In the case of $h_m \ge h_s$

$$Q = \begin{cases} -0.35 \times A_m \sqrt{2gh_m} & : h_s / h_m \ge 2/3 \\ -0.91 \times A_s \sqrt{2g(h_m - h_s)} & : h_s / h_m < 2/3 \end{cases} \tag{11}$$

where $A_s$ is the wetted cross-section area of the sewer pipe; and $A_m$ is the virtual cross-section of the manhole.

(2) Ground surface – sewer pipe

(a) Inlet flow

$$Q = \begin{cases} C_{do} \times A_{sd} \sqrt{2g(h_{hs} - h_{hg})} & : (h_{hs} - h_{hg})/B_0 \ge 0.5 \\ C_{dw} \times L \times \frac{2}{3} \sqrt{2g}(h_{hs} - h_{hg})^{3/2} & : (h_{hs} - h_{hg})/B_0 < 0.5 \end{cases} \tag{12}$$

(b) Overflow

$$Q = \begin{cases} -C_{do} \times A_{sd} \sqrt{2g(h_{hg} - h_{hs})} & : (h_{hg} - h_{hs})/B_0 \ge 0.5 \\ -C_{dw} \times L \times \frac{2}{3} \sqrt{2g}(h_{hg} - h_{hs})^{3/2} & : (h_{hg} - h_{hs})/B_0 < 0.5 \end{cases} \tag{13}$$

where $C_{do}$ is the orifice coefficient; $C_{dw}$ is the weir coefficient; $h_{hs}$ is the piezometric head of the sewer pipe; $h_{hg}$ is the water depth on the ground surface; $A_{sd}$ is the storm drain area; $L$ is the

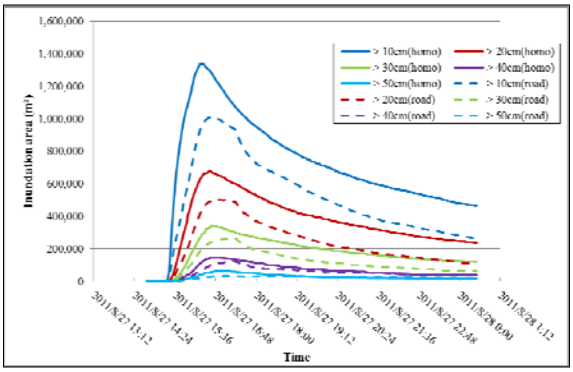


Fig. 7 Comparison of time variation of inundation depth

perimeter of the storm drain; and $B_0$ is smallest width of the storm drain.

## 5. Model Application

We carried out simulations for the recent severe urban flooding in the Nakahama area of Osaka, Japan on 27 August 2011. The study area was approximately 18.1km$^2$. We compared two study cases to estimate the effect of road networks. Fig. 5 shows precipitation data from the Osaka observatory station records. Fig. 6 shows the maximum capacities of the outlet pumping stations. The rainfall duration was approximately 3 hours (the rainfall intensity was 76.5mm during 1 hour). The simulation started at 14:30, 27 August, 2011 in real time, and ended at 00:30, 28 August, 2011, encompassing 10 hours to confirm the drainage process. This study did not include infiltration and river flow effects.

Both cases show similar inundation propagation tendency from the left side to the right side because the elevation of left side is higher than right side. Inundation started at 15:36 27 August, and the maximum inundation depth was recorded at 16:43. However the inundation areas were different in both cases. Fig. 7 shows the time variation in the inundation depth. In both cases, the peak time of inundation was similar, but the inundation areas of homogeneous case were larger than the non-homogeneous case.

Table 1 summarizes the time variation of the inundation area according to the inundation depth. The mesh size of the homogeneous case was larger than the non-homogeneous case. Therefore, the homogeneous case may have a wider inundation area under the same conditions because the surface run-off flow can distribute on a mesh without disturbance of road network configuration. Also there is no tipping effect of the run-off flow caused by road and curb.

Similarly, the total volume of stored water should be the same if the sewerage system, including storm drains, has the same capacity. We can easily check the amount of stored water on the ground by depth multiplied by area. However, stored water volume also showed the homogeneous case was higher than the non-homogeneous case. This phenomenon can be explained by the effective distribution of storm drains.

Storm drains are distributed on the ground mesh vertically above the sewer pipe grid in the homogeneous case, while storm drains are distributed along the road networks in the non-homogeneous case. Also, we reduced the elevation of road network meshes 20cm to represent the curb that divides the road from other areas in an urban area. Generally, road networks are appropriately designed to carry storm water. Run-off flow should accumulate on the road and is captured by storm drain. In the homogeneous case,

Table 1 Comparison of peak inundation depth

| Index(peak depth) | | > 10cm | > 20cm | > 30cm | > 40cm | > 50cm |
|---|---|---|---|---|---|---|
| Area(m$^2$) | homo | 1,340,371 | 673,572 | 328,353 | 140,212 | 52,847 |
| | road | 1,010,646 | 502,491 | 263,657 | 122,710 | 33,511 |
| Total volume (m$^3$) | homo | 134,037.1 | 134,714.4 | 98,505.9 | 56,084.0 | 26,423.5 |
| | road | 101,064.6 | 100,498.0 | 79,098.1 | 49,084.0 | 16,755.5 |
| Ratio(homo/non-homo) | | 1.33 | 1.34 | 1.25 | 1.14 | 1.58 |

the storm drains are not distributed on the road.

## 6. Conclusion

In this study, we used two different kinds of to estimate the importance of considering the road network and storm drains in an urban inundation simulation. The first mesh was a homogeneous type. The second mesh considered road. Both meshes used the same 5m×5m DEM data. In the homogeneous case, we only used average DEM elevation values for the mesh elevation. However in the second case, we reflected the road network and artificially adjusted the road network mesh (-0.2m) to represent their properties.

We selected the Nakahama area of Osaka, Japan as study area. The precipitation event of 27 August, 2011 was the selected study case. The duration of the rainfall was about 1 hour, but the simulation covered 10 hours to confirm the drainage process.

A 2D run-off flow model and a 1D slot model simulated the surface run-off and sewer pipe flow, respectively. We set the storm drains as the exchange discharge spot between the ground surface and sewerage system. Although both simulation results showed a similar propagation tendency of the inundation area, the total volume of stored water and inundation areas on the ground calculated indicated the homogeneous case was larger and wider than the non-homogeneous case.

The drainage process was not well simulated in the homogeneous case. Therefore, the selection of the exchange discharge spot is very important in urban inundation simulation. Distributing the storm drains on the road could be a good choice.

Unfortunately, we could not obtain the inundation survey data. It was thus impossible to compare our results with realistic data. Therefore, the model's applicability must be validated by survey data in the next study.

## Acknowledgements

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