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Calibration of stormwater management model using flood extent data

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The Seogu (western) portion of Daegu, Korea experiences chronic urban flooding and there is a need to increase flood detention and storage to reduce flood impacts. Since the site is densely developed, use of an underground car park as a cistern has been proposed. The stormwater management model (SWMM) is applied to study alternative hydraulic designs and overall performance, and it is shown that by linking SWMM to a two-dimensional flood inundation model, SWMM parameters can be calibrated from observations of flood extent. Calibration would otherwise not be possible because storm sewer flows are not internally monitored. This study reveals a significant sensitivity in SWMM relative to surcharge prediction, demonstrates a creative repurposing of urban infrastructure to manage extreme flood events and shows the importance of flood documentation relative to drainage infrastructure analysis and design.

1. Introduction and background

The major urban centres of the world have developed aside coastal embayments and major rivers owing to ready access to trade routes. Governments promoted major hydraulic works in these waterways to facilitate navigation and protect developed land from flooding. Today, many major cities of the world are defended from river and coastal flooding threats by a combination of dams, reservoirs, levees and offshore barriers such as the Thames Barrier in London, UK, the Delta Works in the Netherlands and the Mose (modulo sperimentale elettromeccanico (experimental electromechanical module)) system in Venice, Italy. However, local intense precipitation is another driver of urban flooding (e.g. Marsalek *et al.*, 2006). This leads to so-called pluvial flooding (i.e. saturation of the drainage infrastructure, which can result in overland flow along streets and inundation of low-lying infrastructure such as car parks, subways and utility vaults for power and telecommunication systems (e.g. Maksimovic *et al.*, 2009). Flooding of wastewater treatment infrastructure introduces human health risks from the transport of pathogenic organisms, and the mobilisation/transport of hazardous chemicals represents both a human and environmental health concern (Who, 2008). Damage to infrastructure or even the interruption of power, transportation and telecommunication systems can be very costly to cities, their businesses and residents. In England and Wales, for example, pluvial flooding from rainfall is estimated to cost £270 million a year (Post, 2007). With continued urbanisation projected for the decades ahead along with intensification of the hydrologic cycle (WMO, 2008), there is a need to focus more attention on urban flood risks (Hanson *et al.*, 2011; Jha *et al.*, 2012; McGranahan *et al.*, 2007). Particular attention is warranted in Asia, where the birth and growth of megacities has magnified urban flood risks (e.g. Hochrainer and Mechler, 2011). Innovative urban drainage systems in Asia include the G-Cans project in Saitama, Japan, which involves a massive underground cistern, and the stormwater management and road tunnel (Smart) in Kuala Lumpur, which involves a dual-purpose tunnel for either stormwater or vehicular traffic.

Rainfall in urban areas leads quickly to runoff due to the extensive coverage of impervious surfaces. Runoff is diverted into street gutters, underground sewer systems and drainage channels that have traditionally been designed to move water as quickly as possible into neighbouring waterways. The importance of flood water detention and storage in urban areas has now been recognised relative to ecological, water quality and flood risk reduction benefits, leading to sustainable urban drainage systems (Suds) in the UK and low impact development (Lid) in the US. However, to date, Suds and Lid practices are only beginning to be implemented. Another common feature of urban flood control systems are pump stations that overcome elevation differences between urban terrain and receiving waters such as rivers or coastal embayments. Indeed, the elevation deficit can be substantial when localised precipitation events coincide with regional flooding events or storm surge events that create high water levels

in adjacent river and embayments, and pumps generally suffer a reduction in flow capacity with an increase in head.

This scenario played out in Daegu, South Korea in 2003 (Figure 1) when typhoon Maemi brought intense rainfall exceeding 37 mm/h and high flows to the Gumho River. Pluvial flooding exceeding 1 m was experienced in the 41·89 ha Bisan catchment located in the Seo-gu district of Daegu. This densely developed catchment includes storm sewers that direct runoff to a pump station, which in turn lifts water to the nearby Dalseo Stream. However, drainage was inhibited by high water levels in the Dalseo Stream caused by backwater of the Gumho River, as well as by operational errors made at the pump station. The event motivated the local authorities to research the possibility of constructing a cistern to collect runoff during episodes of intense precipitation, not only as a means of promoting gravity-driven drainage within the catchment, which is less vulnerable to human error, but also to avoid the need for further expansion of the existing pump station. With an expanded forebay, the pump station can be operated in a more uniform manner over longer periods of time to drain the low-lying Bisan catchment. Attention turned to the necessary size of the cistern and where it could be placed.

Urban areas are densely developed and it is rarely possible to site new reservoirs but, with inspiration from the Smart project that involved the dual support of stormwater and transportation needs, a creative idea was explored in the Bisan catchment. As shown in Figure 1, a bus terminal with an underground car park was identified as having the capacity for significant storage, the basic features required for adequate performance (concrete bottom and walls) and the flexibility to serve transportation needs for the vast majority of time while occasionally serving flood control needs. Interestingly, this would also re-introduce human decision-making into the flood control scheme. Nevertheless, consistent with flood risk management strategies now being embraced globally (Merz *et al.*, 2010a; WMO, 2008), a modelling study was completed to study the potential redesign of the bus terminal, looking specifically at the ability to limit flood impacts. This paper presents the results of this analysis, which was conducting by coupling several models to account for urban hydrology, drainage and overland flow along streets.

For analysis of sewer flow in urbanised areas, the stormwater management model (SWMM) is considered a landmark hydrologic model (Leandro *et al.*, 2009). The basic SWMM formulation serves as the engine for several commercial software packages for drainage design, including PC-SWMM (Computational Hydraulics International, 2012), XP-SWMM (XP Solutions Inc, Portland, USA), Mike-SWMM (DHI Software, Hørsholm, Denmark) and InfoSWMM (Innovyze, Broomfield, USA). These models link to a geographic information system (GIS) to access and archive data and include graphical user interfaces (GUIs) for ease of use. However, the basic SWMM formulation is limited in the depiction of urban inundation because, as a basic link-node model formulation, it predicts water levels at junctions of the sewer network irrespective of the above-ground spreading and storage of flood water. Hence, software developers and researchers have focused attention on models that couple the basic SWMM capabilities with an overland flow and ponding capability similar to that offered by two-dimensional (2D) shallow-water models. A review of so-called dual-drainage modelling is presented by Smith (2006). Examples of modern commercial packages include Mike-Urban (DHI Software, Hørsholm, Denmark), Infoworks CS (Innovyze, Broomfield, USA), Mouse-Mike 21 and Sobek Urban (Leandro *et al.*, 2009). Research applications include Sipson (Djordjević *et al.*, 2005) and Sipson/UM (Chen *et al.*, 2007). 1D/1D or 1D/2D coupled urban inundation models have also been developed to evaluate the hydraulic performance of sewer flow and overland flow simultaneously. Hsu *et al.* (2000) and Chen *et al.* (2005) developed an urban inundation model combining a storm sewer model SWMM with a 2D diffusive

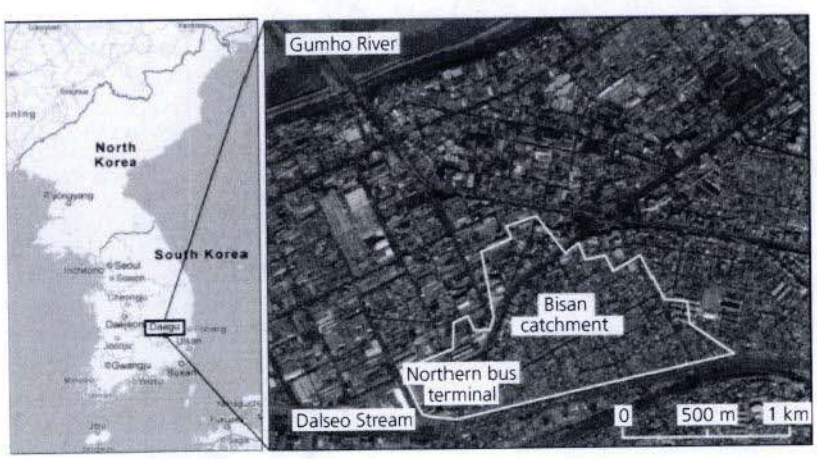

Figure 1. Study area (Source: Google Maps)

overland flow model for simulating flood processes in Taiwan. Furthermore, Mark *et al.* (2004) and Leandro *et al.* (2009) showed extensive comparisons of 1D/1D and 1D/2D models, including their potentials and limitations.

With the application of coupled drainage models in practice, model calibration represents an important first step in the model-assisted redesign of drainage infrastructure (e.g. Barco *et al.*, 2008; Merz *et al.*, 2010b; Takahashi *et al.*, 2010). When a model proves capable of reproducing the response of a drainage system to a specific storm event (or several storm events), then modellers are in a good position to characterise the effects of proposed changes to infrastructure such as new sewer lines, detention basins and pump stations. In contrast, without calibration, modelling of proposed infrastructure changes is far less reliable.

Stormwater monitoring at the outlets of catchments is common and such data are widely used for model calibration. However, outlet monitoring alone is insufficient for learning about the internal dynamics of watersheds (Beven, 2000). In cases of urban flooding, outlet monitoring will not yield insight into the conditions giving rise to localised sewer surcharging. Internal data are required, but not available. However, sewer surcharging leads to localised inundation, so by documenting flood zones with photographs and wrack and water mark measurements, the resulting data can be used to further constrain parameter values leading to a better understanding of internal dynamics. This demands a coupled 1D/2D model in order to link sewer line surcharging to spatial patterns of inundation.

In this study, the SWMM is coupled with the Brezo flood inundation model (Sanders, 2008; Sanders *et al.*, 2008) to demonstrate how flood extent data can be used to constrain the SWMM parameters. The coupling itself is not innovative – in fact it is rather simple – and this makes the study results applicable to 1D/2D drainage models in general. The calibrated model is then applied to analyse alternative bus terminal designs for the Bisan catchment. Calibration of the SWMM would otherwise be impossible because flow rates through the sewer system were not monitored during typhoon Maemi. This study demonstrates an important benefit of coupled 1D/2D models over 1D models (SWMM) relative to model calibration, provides insight into the most sensitive SWMM parameters relative to surcharge flow predictions and presents an innovative example of how existing infrastructure can be re-tooled to manage the risk of urban flooding with the aid of hydraulic models.

2. Urban inundation model

SWMM and Brezo were coupled to account for runoff, sewer flow, pumping and overland flow leading to flood inundation. The SWMM accounts for rainfall–runoff, flow through the sewer network and sewer surcharging, which represents an input to Brezo. In turn, Brezo resolves overland flow and return flow to the sewer where such capacity exists.

2.1 SWMM

SWMM is a dynamic rainfall–runoff simulation model, developed by the United States Environmental Protection Agency, which computes the quantity and quality of urban runoff in storm and combined sewer systems (Huber and Dickinson, 1988). SWMM consists of multiple functional computational blocks as follows.

- The runoff block, which calculates the surface runoff and water quality constituents from rainfall.
- The transport block, which calculates the flows and water quality of the drainage system with no surcharge through dynamic routing.
- The storage/treatment block, which traces flows and water quality through a storage control device.
- The extran block, which calculates hydraulic flows by steady flow, kinematic wave and dynamic wave tracing (Delleur, 2003).

The blocks can be overlaid and run sequentially or can be run separately with linking or interfacing data being transferred between the blocks (Peterson and Wicks, 2006). In this study, runoff and extran blocks were used. The runoff block performs hydrologic simulation and its outputs are taken as input to the extran block, which routes the conduit (or pipe) flow in a storm sewer system using the complete dynamic flow routing equations (Saint-Venant equations) (Hsu *et al.*, 2000).

2.2 Brezo model

Brezo is employed to calculate inundation zones and depths due to the surcharge flow on the land surface. Brezo is a hydrodynamic model based on the 2D shallow-water equations, consisting of a fluid continuity equation and two momentum equations (Begnudelli and Sanders, 2008; Sanders, 2008). This model uses an unstructured grid Godunov-type finite-volume scheme based on Roe's approximate Riemann solver and Euler time stepping, and includes a number of features to enhance performance in urban flooding applications. These include a robust wetting and drying treatment, an adaptive method of variable reconstruction to minimise numerical dissipation, a local time stepping scheme to reduce run times without loss of accuracy and an implicit friction treatment to avoid stability restrictions arising from flow resistance (Begnudelli and Sanders, 2008; Sanders, 2008). Application of the model at field scale has shown that topographic data are essential to accurately predict flood extent and that the model performs well using physically reasonable parameter values (Gallegos *et al.*, 2009; Schubert *et al.*, 2008). The ability to prescribe resistance parameters based on land cover is exploited here so that model calibration can focus on SWMM parameters, which represent a greater source of uncertainty in the coupled system.

Coupled drainage models (1D/2D) are typically linked at junctions of the storm sewer network. When the 1D model predicts surcharging, the surcharged flow rate becomes a point source in the 2D model to predict the lateral spreading of flood water. In

addition, water is returned from the 2D domain to the 1D sewer network after the period of surcharging ends. Coordinated updating of 1D and 2D continuity equations is therefore required to conserve fluid, and the coupling is dynamic in the sense that the flow rate is scaled by differences in head across junction points. In practice, this requires source code that exchanges 1D and 2D model data on the fly but, to demonstrate 1D model calibration with 2D data (which is the focus of this paper), it is sufficient in this particular case of flooding to pass data through files that are written to disk. That is, in this case, street flooding was localised within the vicinity of one manhole so it was possible to first run the SWMM to predict the surcharge time series and to subsequently run Brezo to predict ponding. It is stressed, however, that this is not a recommended approach for a general-purpose model.

3. Typhoon Maemi flooding

3.1 Study area

Floods are the most frequent natural hazards in Korea: typhoon Rusa took 213 lives and caused US5 billion worth of damage in 2002 and typhoon Maemi claimed 117 lives and caused US4·5 billion worth of damage in 2003 (www.kma.go.kr). As shown in Figure 1, the urban Bisan catchment is located in Seo-gu, Daegu, South Korea. Typhoon Maemi delivered intense precipitation to the city and much of South Korea for roughly 24 h on 12 September 2003. As shown in Figure 2, precipitation in Daegu reached a maximum intensity of approximately 37 mm/h and flood stage in the Dalseo Stream adjacent to the Gumho River rose 5·5 m, from a baseline elevation of 19·62 m to a peak elevation of 25·12 m. Rainwater drains from the Bisan catchment through the storm sewer network shown in Figure 3(a), which includes the representative main pipelines and the Bisan rainwater pump station designed to pump stormwater to the neighbouring Dalseo Stream, shown in Figure 1. However, high water levels in the Dalseo Stream (Figure 2) reduced the pump station flow rate. This, in combination with errors made by pump station operators, led to surcharging of the sewer network at manhole 102 (shown in Figure 3(a)) and inundation of adjacent terrain (DMC, 2006; Kim *et al.*, 2010). After this flood, a field investigation was conducted by city officials to observe flood marks and collect first-hand reports on flooding (DMC, 2006). Similar investigations have been presented previously (Kang, 2009). The Daegu investigation resulted in a georeferenced and digitised polygon.

3.2 Model parameterisation, calibration and application

Data characterising the topography, land cover and storm sewer network were obtained from the local authorities and used to parameterise the SWMM and Brezo models (DMC, 2006). Topographic data were supplied as a digital contour map derived from a 1:1000 scale topographic map and containing 20 cm contours and spot heights reported with centimetric precision. Absolute accuracies were not reported but, in Korea, maps such as this typically achieve absolute accuracies less than 20 cm and centimetric relative errors as measured by neighbouring points (Lee *et al.*, 1998). Digital contour lines were subsequently converted to points and combined with spot heights to create a triangulated irregular network (TIN) digital terrain model (DTM). Figure 3(b) shows the 40 sub-catchments of the study area and Tables 1 and 2 present properties of the pipes and sub-catchments. Figure 3(c) shows a land cover map of the study area. The pump station shown in Figure 3(a) was modelled based on a set of operational rules as follows (DMC, 2006): the volume of Bisan drainage pump station is 1400 m^3; pumps activate when depths in the forebay exceed 1·0 m; flow linearly increases with depth beyond 1·0 m, reaches a maximum of 7·76 m^3/s in a depth of 1·7 m and maintains the maximum flow for larger depths.

To apply Brezo, a 2D unstructured mesh of the Bisan catchment was created using Triangle (Shewchuk, 1996). The mesh has a resolution of approximately 4 m in roads and 6 m in remaining areas, based on the square root of the average cell area, and

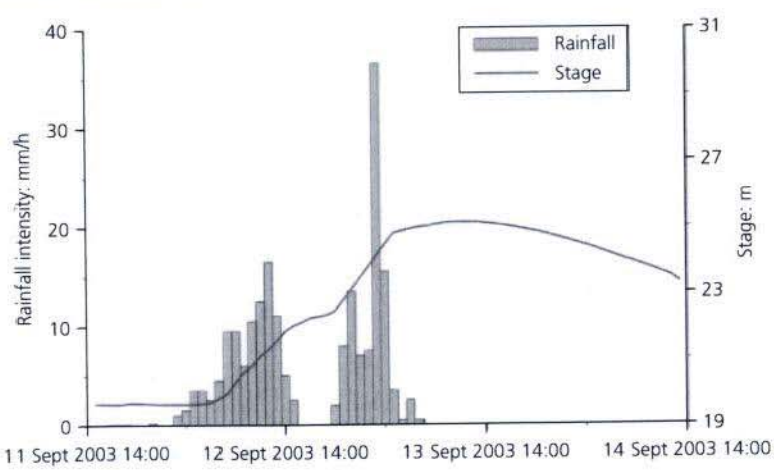

Figure 2. Rainfall hyetograph and river stage for typhoon Maemi

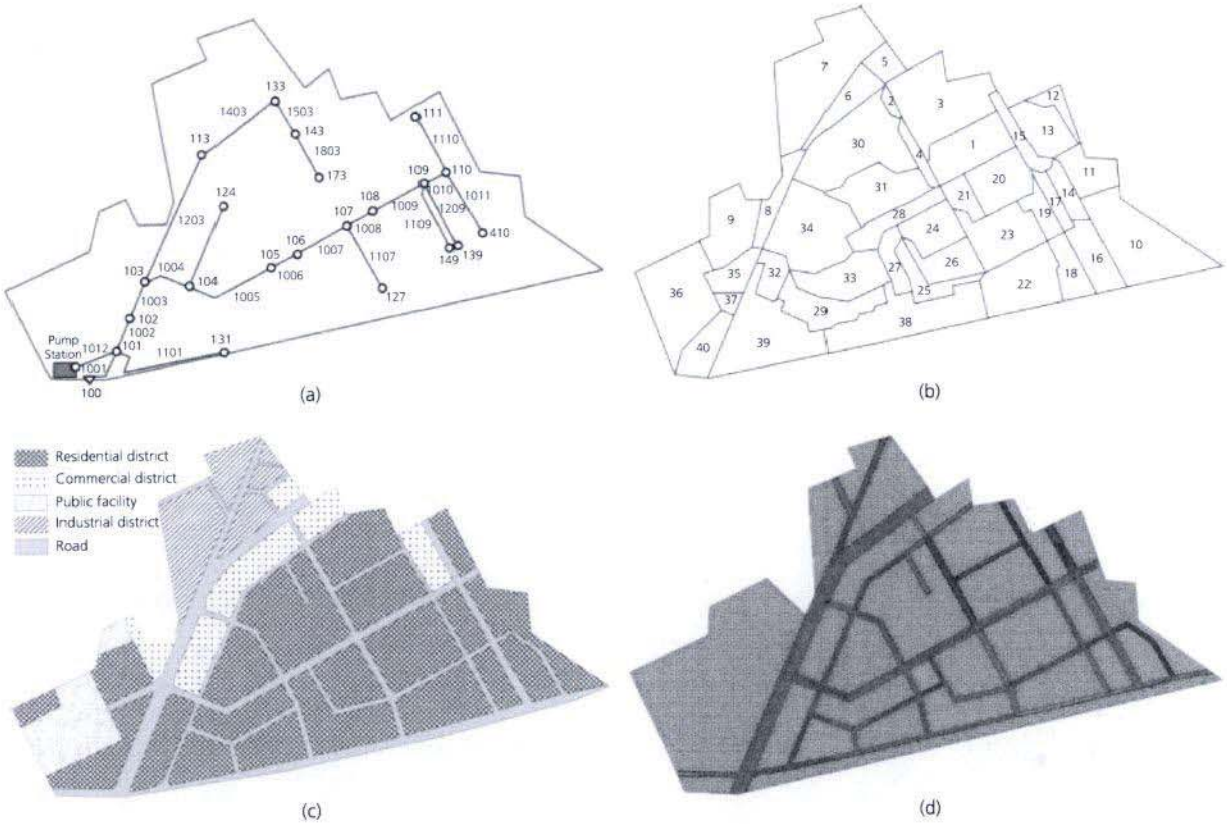

Figure 3. Properties of the Bisan catchment: (a) drainage system; (b) sub-catchments; (c) land cover map; (d) unstructured grids of triangular cells

includes 46 299 cells shown in Figure 3(d). Elevation was assigned to vertices of the mesh from the TIN (linear interpolation). A spatially distributed Manning resistance parameter was used, with values of 0·014 $m^{-1/3}s$ for roads and 0·30 $m^{-1/3}s$ for remaining areas which are residential, industrial and commercial (Schubert and Sanders, 2012; USACE, 1981).

With a good understanding of the urban drainage features of the basin (Figure 3) and the hydrometeorology that caused the flooding event (Figure 2), there were sufficient data to prescribe most of the SWMM parameters (e.g. geometrical features of sub-catchments and pipes) and all of the Brezo parameters, and to subsequently simulate the hydrologic response of the basin to typhoon Maemi. However, several of the SWMM parameters could not be prescribed from watershed data. These were calibrated to minimise the difference between observed and predicted flood extents. This mandates the SWMM–Brezo coupling presented here. That is, Brezo translates SWMM predictions of surcharging into flood extent predictions and, by adjusting SWMM parameters, the mismatch between predicted and observed flood extent is minimised.

Eight parameters were identified as candidates for calibration of SWMM, as shown in Table 3. These include parameters that affect the modelling of depression storage, infiltration, overland flow and sewer (conduit) flows. According to SWMM documentation for urban drainage modelling, these parameter values are expected to fall within prescribed ranges as shown in Table 3. However, a sensitivity study showed that only the Manning resistance parameter for conduits had a significant effect on the predicted rate of surcharging. Modelling with group A and B parameters, which scale the maximum range of the land surface parameters, revealed only a very small impact on the predicted rate of surcharging at manhole 102 (Figure 4) and flood extent (Table 4). However, when all of the land surface parameters were assigned values recommended by Daegu Metropolitan City (DMC, 2006) based on previous modelling studies and only the conduit Manning parameter was varied (group C), a significant change in surcharging (Figure 4) and flood inundation (Table 4)

Conduit no.	Shape	Diameter (bottom width): m	Height: m	Length: m
1803	Rect_Closed	0·70	0·6	186
1503	Rect_Closed	0·70	0·5	115
1403	Rect_Closed	2·00	2·0	220
1203	Rect_Closed	1·70	1·7	400
1011	Rect_Closed	0·60	0·6	142
1110	Rect_Closed	0·70	0·4	139
1010	Rect_Closed	1·00	1·0	65
1209	Circular	0·45	0·4	152
1109	Rect_Closed	0·60	0·4	145
1009	Rect_Closed	1·00	1·0	120
1008	Rect_Closed	1·00	1·0	66
1107	Circular	0·70	0·4	154
1007	Rect_Closed	1·60	1·6	133
1006	Rect_Closed	1·60	1·6	59
1005	Rect_Closed	1·60	1·6	210
1104	Circular	0·70	0·4	179
1004	Rect_Closed	1·60	1·6	94
1003	Rect_Closed	1·70	2·5	100
1002	Rect_Closed	1·70	2·5	80
1101	Circular	0·60	0·4	347
1001	Rect_Closed	1·70	2·5	175
1012	Rect_Closed	2·00	1·5	120

Table 1. Properties of conduits (DMC, 2006)

was observed. This result showed that the Manning parameter for sewer conduits is the greatest source of sensitivity among unknown parameters. Consequently, this parameter was calibrated so that the Brezo prediction of flood extent best matched the observed flood extent as shown in Figure 5. The calibration was done by incrementally adjusting the Manning parameter within the allowed range shown in Table 3 until a maximum in the flood prediction fitness was identified. Fitness of the flood extent prediction was measured as

$$1. \quad \text{Fitness}(\%) = [A_M \cap A_P]/[A_M \cup A_P] \times 100$$

where A_M is the measured inundation area and A_P is the predicted inundation area. A maximum in fitness was identified near $0{\cdot}013\ m^{-1/3}s$ and Table 4 shows several trials that were tested to identify the optimal parameter value. These results show a high level of sensitivity: 1% changes in the Manning parameter yielded changes in fitness exceeding 10% and therefore the optimal Manning parameter was prescribed down to four significant figures (i.e. $0{\cdot}01308\ m^{-1/3}s$). This achieved a flood extent prediction fitness of 78%. Hence, the calibrated SWMM adopts the group D parameter set shown in Table 3. Table 5 and Figure 5 show the flood volumes and flood extents across all of the parameter sets considered.

Sub-catchment no.	Outlet node no.	Width: m	Area: ha	Percent impervious: %	Slope
1	173	115·08	1·45	80·0	0·025
2	143	16·87	0·27	50·0	0·025
3	143	164·52	3·06	80·0	0·034
4	143	13·58	0·33	80·0	0·018
5	113	6·67	0·07	80·0	0·033
6	133	36·52	0·42	80·0	0·033
7	113	60·61	2·00	48·1	0·034
8	103	16·25	0·65	80·0	0·040
9	103	86·47	1·47	30·6	0·034
10	410	194·57	3·58	80·0	0·040
11	110	76·47	0·78	80·0	0·003
12	111	13·19	0·19	66·4	0·040
13	110	66·19	0·92	80·0	0·007
14	110	20·26	0·31	80·0	0·042
15	109	49·23	0·32	80·0	0·007
16	139	70·94	0·83	80·0	0·068
17	109	12·50	0·19	80·0	0·075
18	149	45·00	0·54	80·0	0·054
19	109	15·862	0·23	80·0	0·010
20	108	98·33	1·18	80·0	0·068
21	107	48·48	0·32	80·0	0·058
22	127	240·00	1·56	80·0	0·075
23	107	97·40	1·50	80·0	0·054
24	106	66·92	0·89	80·0	0·037
25	106	35·53	0·81	80·0	0·025
26	106	21·63	0·53	80·0	0·025
27	105	61·02	0·36	80·0	0·034
28	105	29·83	0·71	80·0	0·018
29	104	47·62	1·00	80·0	0·033
30	124	89·92	2·23	48·1	0·033
31	124	129·56	1·49	80·0	0·034
32	103	44·68	0·42	80·0	0·040
33	104	106·38	1·00	80·0	0·034
34	104	125·70	2·25	30·6	0·040
35	102	53·00	0·53	80·0	0·003
36	102	136·29	1·69	80·0	0·040
37	101	4·17	0·01	80·0	0·007
38	131	85·98	2·33	80·0	0·042
39	101	79·83	2·77	80·0	0·007
40	100	40·00	0·70	70·0	0·068

Table 2. Properties of sub-catchments (DMC, 2006)

Figure 5 shows that the area of flooding is slightly over-predicted at 62 127 m^2 compared to the surveyed flood extent computed as 56 790 m^2. In addition, the calibration suggestions that the surcharge persisted for over 15·0 h, attained a peak rate of $0{\cdot}76\ m^3/s$ and the maximum inundation depth was 1·22 m. To illustrate how the flood evolved over time, Figure 6 shows Brezo predictions of the flood depth and extent at four different times

Parameter	Allowed range	Group A	Group B	Group C	Group D
Depression storage					
Roughness height of impervious area: mm	1·27–2·54[a]	1·3	2·5	1·5	1·5
Roughness height of pervious area: mm	2·54–7·62[a]	3·0	6·0	4·0	4·0
Horton's infiltration method					
Maximum infiltration: mm/h	25·4–127·0[b]	120·0	25·0	50·0	50·0
Minimum infiltration: mm/h	0·03–11·9[b]	8·0	2·0	5·0	5·0
Decay coefficient: h^{-1}	2–7[b]	2	5	4	4
Manning's coefficient					
Impervious area	0·02–0·05[a]	0·02	0·04	0·024	0·024
Pervious area	0·04–0·50[a]	0·04	0·40	0·12	0·12
Conduits	0·011–0·015[a]	0·012	0·012	0·014	0·01308

[a] Mays (2001)
[b] ASCE (1982)

Table 3. Parameters for calibration of SWMM

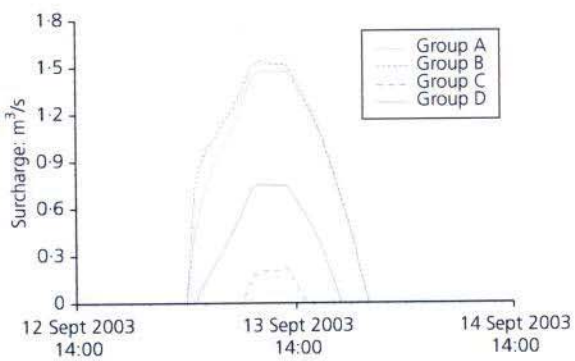

Figure 4. Surcharged flows at manhole 102

Manning's n	Surcharge		Flood inundation
	Peak flow: m^3/s	Volume: m^3	Fitness: %
0·0128	0·96	38 045	52·0
0·0129	0·87	34 260	61·0
0·0130	0·84	30 877	73·0
0·01308	0·76	27 852	78·0
0·0131	0·75	27 336	77·0
0·0132	0·69	23 595	66·0

Table 4. Sensitivity of surcharge and flood extent predictions to Manning's n

after surcharging started at manhole 102 (04:00 on 13 September 2003). Note that the flood zone expands for roughly 10 h, consistent with the duration of the surcharging shown in Figure 4. With the calibrated model, attention turned to assessing the potential benefits of integrating the underground car park into the flood control infrastructure.

3.3 Bus terminal analysis

As shown in Figure 7, the northern bus terminal is located in close proximity to manhole (MH) 102 where surcharging occurred, and in an underground configuration that is suitable for use as a detention basin. To examine system performance with the underground storage facility (USF), three alternative designs were considered, as shown in Figure 7 and Table 6. Option A includes a single connection between MH 102 and the USF, link 1021. This can be viewed as a low-cost alternative as only a single connecting pipe would be constructed along the roadway, an established right-of-way. In this configuration, the USF would fill as a result of head gradients between MH 102 and the USF, and the USF would drain when the head gradient reversed, most likely with the assistance of a sump pump. Option B includes link 1021 and a second link to connect MH 101 to the USF, as shown in Figure 7. This would allow the USF to connect more directly to the pump station (link 1012) and the gravity drain (link 1001), although a right-of-way would need to be established because the pipe passes under private property. Option C would include link 1021 and a second link directly to the pump station. This would allow the USF to fill through link 1021 and improve the potential for drainage vis-à-vis the direct pump station connection. Again, a right-of-way would need to be established.

With all the model parameters and forcing data, the same as described previously, the SWMM–Brezo modelling system was applied for options A, B and C and three possible sizes of the USF that scaled its available storage. The USF footprint is 120 m × 150 m and heights of 3·4, 3·7 and 4·0 m were considered. A height of 3·4 m corresponds to the existing car park under the bus terminal, while the other heights would require renovation of the facility with a deeper first-floor car park. This represents a far costlier approach, but is a realistic one because

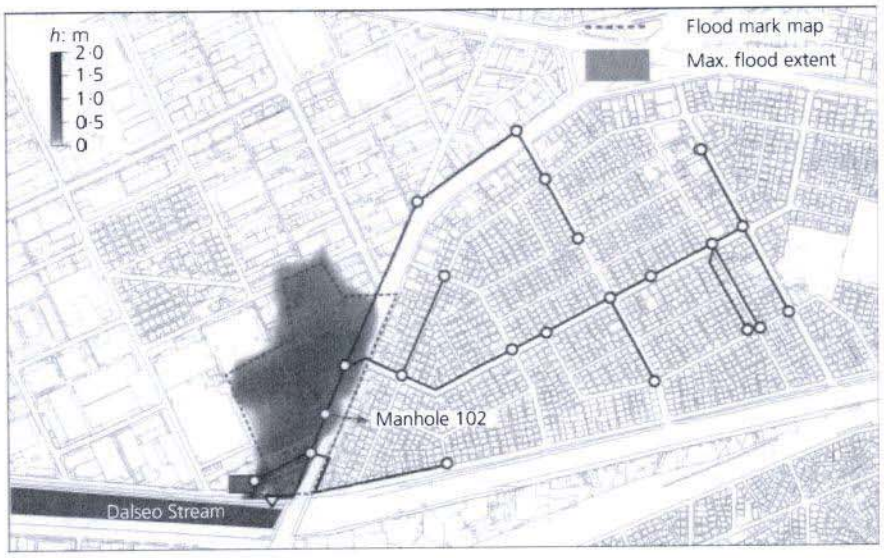

Figure 5. Comparison of observed and predicted flood extent

Group	Surcharge		Flood inundation
	Peak flow: m^3/s	Volume: m^3	Fitness: %
A	1·48	71 875	37·0
B	1·53	77 061	34·0
C	0·22	4165	46·0
D	0·76	27 852	78·0

Table 5. Predicted results based on group of calibration parameters

the existing bus terminal is reaching the end of its design life and needs to be replaced.

Figure 8 shows the time history of surcharging (combined flow at the detention basin and manhole 102) for each configuration and USF height. These results show that the detention basin helps to significantly reduce flooding, but that the sewer line configuration is very important. In particular, option B leads to the most surcharging and option C the least, and the largest detention basin achieves the greatest reduction in flooding as expected. The superior performance of option C is attributed to its design against back-flooding from high river stage. By comparison, the poor performance of option B is attributed to the potential for backwater effects to penetrate through the gravity drain and into the USF.

Table 7 shows how the peak surcharge flow, duration and total surcharge volume vary according to the detention basin size and the sewer line configuration. In addition, Figure 9 shows predicted flood extent according to the option C design. As shown in Table 8, these results show that flood extent is reduced by 59, 66 and 98% and flood depth is reduced by 42, 60 and 90% when the height of the underground detention facility is 3·4, 3·7 and 4·0 m respectively and configured according to option C.

A peculiar trend in the above results is that, under the option A configuration, the amount of surcharging is not very sensitive to the capacity of the USF compared with options B and C. Option A involves only a single connecting pipe between the detention basin and the sewer network, while options B and C include two connecting pipes. The implication is less transport capacity in option A, and the ability to only fill or drain the basin at any given time; that is, no through-flow. This suggests that surcharging is primarily constrained by flow capacity in option A and by storage in options B and C. The broader implication is that the benefits of detention basins in urban flood control systems can be limited by the both the volume of the reservoir and the capacity of the sewer network to fill and drain it.

4. Discussion

The presented results highlight an unexpectedly high level of surcharging sensitivity to Manning resistance parameters used for sewer pipes. It is well known that resistance parameters are a source of uncertainty in predictive flow modelling and therefore the high degree of precision required to achieve accurate SWMM surcharging predictions is a concern. In other words, in practice, it is generally not possible to estimate resistance parameters beyond one or two significant figures. The reader is cautioned that these findings are drawn from a single case study and more

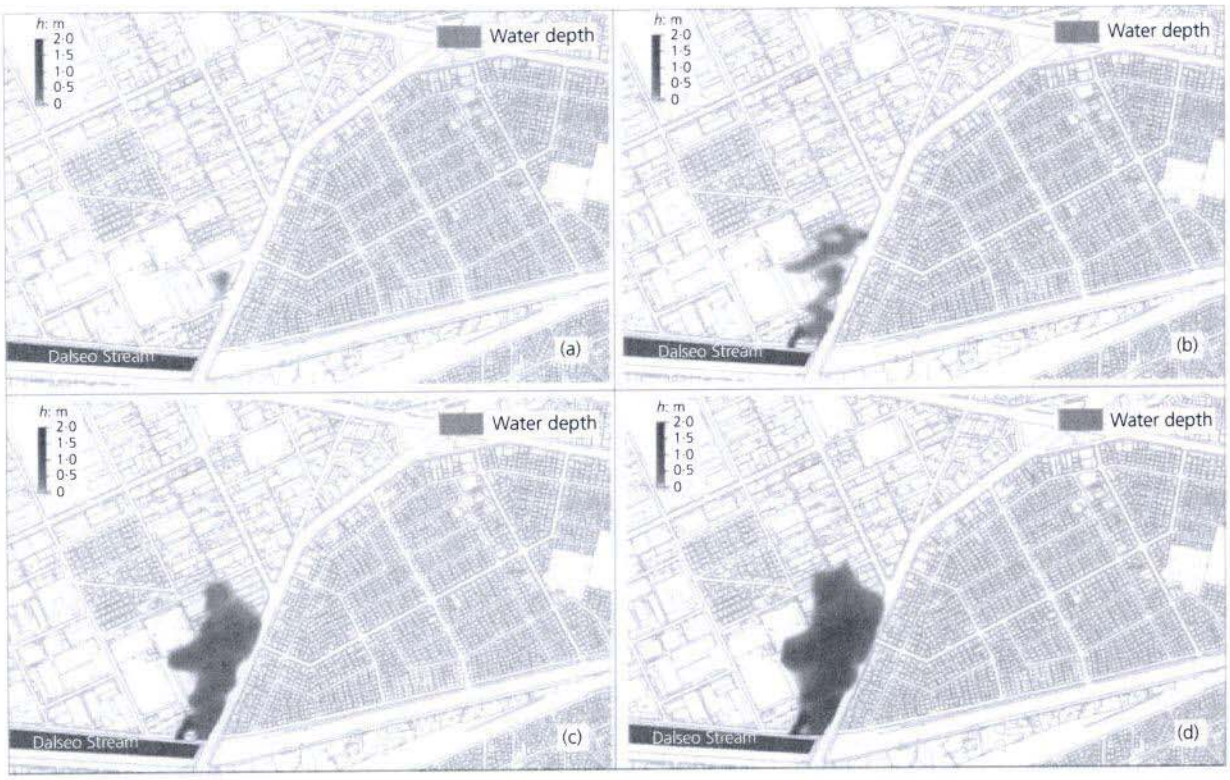

Figure 6. Evolution of flood zone on 13 September 2003: (a) 04:30; (b) 07:00; (c) 11:00; (d) 16:00

studies are needed to better understand whether this is a site-specific or general result.

The results show that the typhoon Maemi flood zone in Daegu, Korea could have been reduced by 59% were an existing bus terminal repurposed for flood control, but that the performance of the system would be sensitive to the configuration of sewer lines, including connections to existing pump stations. To avoid pluvial flooding completely, a major renovation of the bus terminal and its underground car park would be required. This is presently under consideration because the existing facility is nearing the end of its design life. These results highlight synergies that can be achieved with urban infrastructure that is designed to support more than one critical lifeline. Identifying the most cost-effective alternative is beyond the scope of this paper, but would involve analysis of several storm scenarios corresponding to a range of exceedance probabilities, modelling to analyse the flood zones associated with each option and economic analysis focused on flood damage and business interruption. By weighing the economic risk of flooding against various investment alternatives, the best scenarios for the basin can be identified. It should be stressed that the most cost-effective alternatives may not necessarily be those of a traditional 'flood control' nature such as the detention basin concept proposed here. What could prove more cost effective are distributed watershed management practices that act to reduce runoff and the resulting stress on the collection network (Brown and Hunt, 2012; Duffy *et al.*, 2008; Li *et al.*, 2009; Xiao and McPherson, 2011).

5. Conclusions

Coupling of a 1D drainage model (SWMM) with a 2D inundation model (Brezo) enables mapping of flood extent resulting from predicted surcharging. After parameterising SWMM with measurable properties of the system for a drainage system in Daegu, Korea, including pipe sizes, lengths, catchment sizes and land cover attributes such as the percentage of impervious cover, several unknown parameters remain to be estimated and the Manning resistance parameter for sewer pipes was found to be the greatest source of uncertainty relative to surcharge prediction and thus flood extent prediction by Brezo. Near the optimal parameter value, a 1% change in the Manning parameter yields a 10% change in the fitness of flood extent predictions (a measure of accuracy). Consequently, the Manning parameter was refined down to four significant figures to maximise fitness at 78%. This represents a very high level of precision considering the typical uncertainties associated with resistance parameters and raises questions about the ability of SWMM to accurately predict surcharging without calibration. An important benefit of

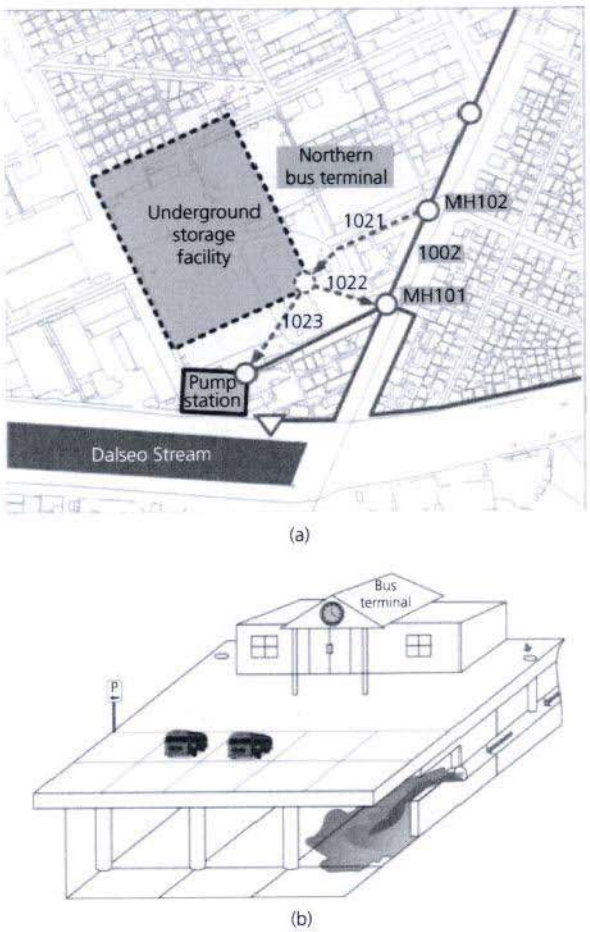

Figure 7. Sketch of underground storage facility: (a) assumed flow routes; (b) schematic outline

the SWMM–Brezo coupling shown here is that flood extent observations can be used to calibrate SWMM, whereas in the absence of a 2D inundation model (Brezo or some other tool) it is not possible to link surcharging flows to flood extent. This highlights an important benefit of post-flood reconnaissance programmes beyond established benefits such as damage estimation.

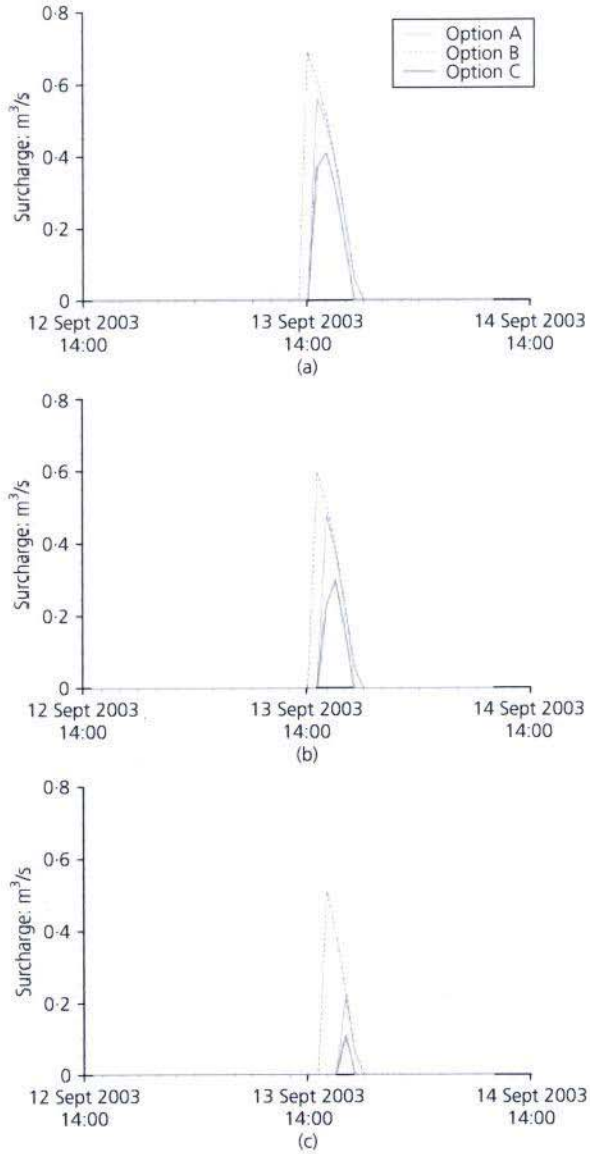

Figure 8. Surcharged flows for each flow route and proposed configuration: (a) $H = 3{\cdot}4$ m; (b) $H = 3{\cdot}7$ m; (c) $H = 4{\cdot}0$ m

Option	Flow route	Conduit no.	Shape	Bottom width: m	Height: m	Length: m
A	MH102→1021→USF	1021	Rect_Closed	1·40	0·90	100
B	MH102→1021→USF→1022→MH101	1022	Rect_Closed	1·20	0·60	80
C	MH102→1021→USF→1023→PS	1023	Rect_Closed	1·20	0·60	100

MH, manhole; USF, underground storage facility; PS, pumping station

Table 6. Assumed flow route and conduit properties

Height H: m	Peak surcharge flow: m^3/s			Surcharge duration: h			Surcharge volume: m^3		
	Option A	Option B	Option C	Option A	Option B	Option C	Option A	Option B	Option C
3·4	0·56	0·69	0·41	5·0	6·0	4·0	6184	8932	4496
3·7	0·48	0·60	0·30	4·0	5·0	3·0	4105	6442	2502
4·0	0·22	0·51	0·11	2·0	4·0	1·0	1024	4259	394

Table 7. Variations of surcharge according to detention basin size and flow

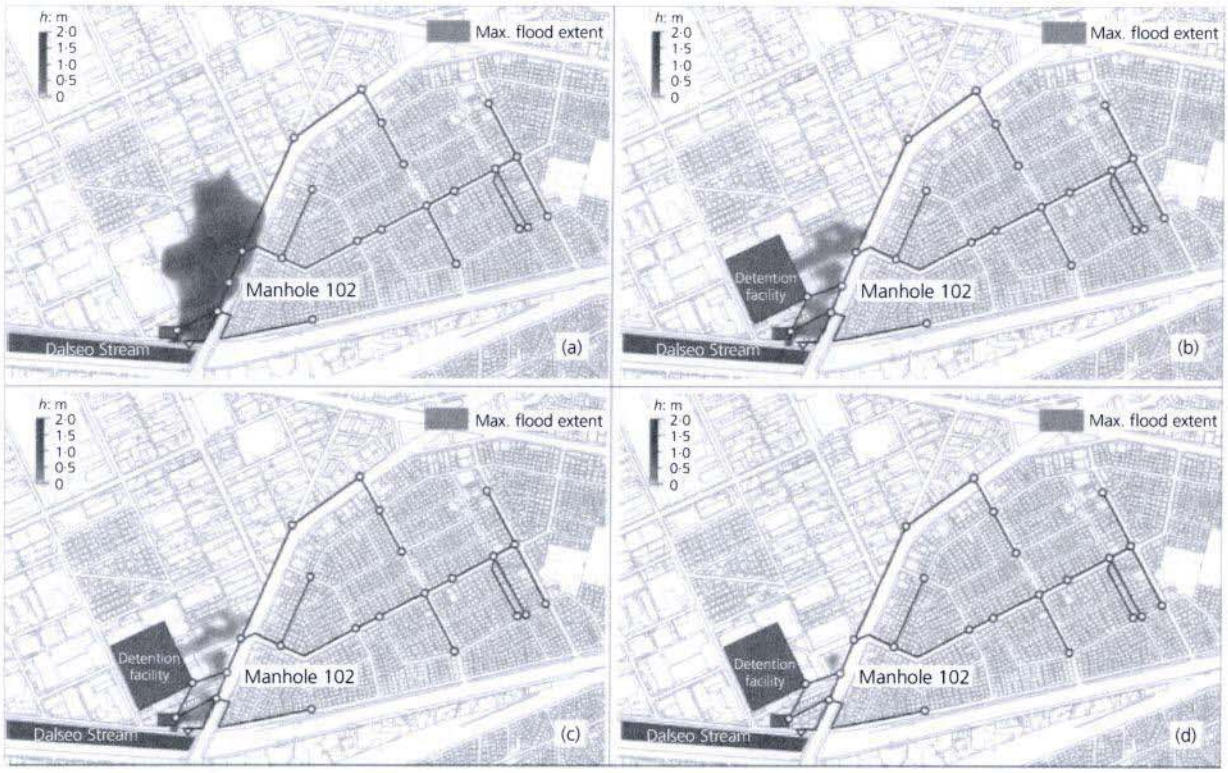

Figure 9. Maximum inundation area: (a) without detention facility; (b) $H = 3{\cdot}4$ m in option C; (c) $H = 3{\cdot}7$ m in option C; (d) $H = 4{\cdot}0$ m in option C

	Flood inundation			
	Maximum area: m^2	Reduced rate: %	Maximum depth: m	Reduced rate: %
Without detention facility	62 127	—	1·22	—
With detention facility ($H = 3{\cdot}4$ m)	27 508	58·69	0·72	41·94
With detention facility ($H = 3{\cdot}7$ m)	22 606	66·06	0·49	60·48
With detention facility ($H = 4{\cdot}0$ m)	1445	97·83	0·12	90·32

Table 8. Variation of maximum inundation area and depth for option C

The study of typhoon Maemi showed that flooding from an extreme event can be significantly reduced by repurposing an existing underground car park as a flood storage facility, but that the effectiveness of the facility would depend on its connections to and configuration within the existing drainage infrastructure. Hence, these deserve careful design and analysis along with the size of the detention basin. The results clearly show that model calibration is important relative to infrastructure design.

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