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Systematic Assessment on Waterlogging Control Facilities in Hefei City of Anhui Province in East China

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Abstract: Both the renovation of rainwater pipes and the addition of sponge city facilities in the low-terrain residences of urban fringes were rarely systematically simulated using the Storm Water Management Model (SWMM). With the waterlogging prevention project in an old residential quarter at a fringe of Hefei city being an example, this study used the SWMM to simulate the effect of the renovation of rainwater pipes and sponge city facilities under different return periods. The results showed the key nodes on the main pipes met the drainage requirements based on water depth analysis after renovation below the 20-year return period, and the reduction rate of the maximum water depth at the key node J5 was the greatest, with 87.7%. The four flow parameters (the average flow rate, the peak flow rate, the total discharge, and the percentage of water flow frequency) for the two outlets (PFK1 and PFK2) all improved after renovation under five return periods (2, 5, 10, 20, and 50 years [a]). The addition of sponge city facilities effectively reduced the amount of rainwater runoff from 28.68% to 14.78% during 2 a to 50 a, and the maximum reduction rate of water depth, being 61.15%, appeared in J5 under 20 a. The curve integral area of the depth over the elapsed time was innovatively used to indirectly express the accumulated rainwater volume through the rainwater well. This study verified that the SWMM model can be well applied to old low-terrain residential quarters in urban fringes and broadened the application scenario of the model.

Keywords: SWMM; rainwater pipe; renovation; return period; sponge city facilities

Citation: Hu, H.; Liu, Y.; Du, J.; Liu, R.; Wu, B.; Zeng, Q. Systematic Assessment on Waterlogging Control Facilities in Hefei City of Anhui Province in East China. *Water* **2024**, *16*, 620. https://doi.org/10.3390/w16040620

Academic Editors: Maria Mimikou, Jinjun Zhou and Hao Wang

Received: 31 December 2023
Revised: 4 February 2024
Accepted: 12 February 2024
Published: 19 February 2024

1. Introduction

In recent years, many Chinese cities have become increasingly waterlogged during the rainy seasons due to climate changes and accelerated urbanization [1,2], with some old low-terrain residential quarters being serious cases. Heavy rains result in small ponds in these areas, which poses a challenge for the residents' life. The other main causes are as follows: to begin with, rainwater and sewage combined sewers were used which, with the development of cities, have been unable to meet the discharge requirements in rainstorm seasons; secondly, some rainwater pipes were damaged, blocked, or corroded seriously, resulting in the failure of rainwater discharge in time [3,4]; thirdly, the bottoms of some rainwater wells were filled with garbage and silt, which will wash to the rainwater inlet and block the outlet when it rains; last, but not least, with the acceleration of urbanization in China, the rising ground hardening rate and proportion of impervious areas has led to the increase in rainwater runoff.

To prevent waterlogging, the rainwater pipes have been renovated. However, the unitary renovation of rainwater pipes is costly, unsatisfactory, and ineffective in the removal of rainwater pollutants. Therefore, Chinese researchers turned to the application of the sponge city or low-development-impact (LID) facilities in 2013. The sponge city facilities, named by Chinese researchers according to a previous study [5], including infiltration roads, planting ditches, permeable pavements, and permeable parking lots, etc., are similar to sponge; that is, rainwater is stored and released for use on sunny days. They believed sponge city facilities could not only reduce the rainwater runoff and the flow rate in rainwater pipes, but also purify the rainwater [6].

However, it is not clear whether the rainwater pipe reconstruction, sponge city, or LID facilities scheme can achieve the expected effect, which, therefore, is to be predicted using the waterlogging model. An urban waterlogging model generally establishes a mathematical model simulating the rainfall runoff on the roads, the underground drainage pipe network, and the river flow [7,8]. The Storm Water Management Model (SWMM), as one of the major prediction models for low-impact development and rainwater pipes, can better simulate the changes in parameters, such as rainwater runoff, the peak flow rate, and water storage, in accordance with the regulation of sponge city facilities [9,10]. One study focused on the SWMM to model and compare the runoff control effects of two LID system schemes, finding that the LID schemes delayed peak flooding and reduced rainwater discharge [11]. And another study researched the rainwater pipes in Windsor, Ontario, using the SWMM and, in turn, developing, calibrating, and validating it [12]. The SWMM was also used to simulate the impact of rainwater facilities on the reduction in rainwater runoff peaks and the increase in the risk of node flooding [13]. Though many studies have dealt with urban waterlogging control by means of the SWMM, few of them focused on old low-terrain residential quarters in urban fringes [14]. It can be said that these areas are more prone to waterlogging, which is supposed to deserve further research amidst blooming urban renewal projects in China in recent years. In addition, there is little systematic research on both rainwater pipe renovation and the addition of sponge city facilities based on an SWMM simulation.

To sum up, this study aims for a systematic assessment of both the rainwater pipe renovation and sponge city facilities construction in old low-terrain residential quarters in urban fringes using the SWMM. It uses the hydraulic model of the SWMM to simulate the actual construction project of rainwater pipes and sponge city facilities in old low-terrain residential quarters on the fringe of Hefei city in Anhui Province as an example, using five return periods and the curve integral area of the depth of the rainwater in the well over the elapsed time for accumulated rainwater to evaluate the effectiveness after the renovation. Therefore, this study can provide a reference for relevant renovation projects in old low-terrain residential quarters in urban fringes.

2. Materials and Methods

2.1. Overview of Research Area

The research area is located in the fringe of Hefei city, near the suburban Feixi under the jurisdiction of the city, in Anhui Province in east China (31°45′19.41″–31°45′26.25″ N, 117°11′57.53″–117°12′4.21″ E), as shown in Figure 1. It is an old residential quarter built in 2004, low-lying and relatively flat, with a slope between 3% and 4%, covering a total land area of about 3.88 hectares and a total construction area of 3.6×10^4 m^2, respectively. Global warming has potential impacts on rainfall patterns, and, in recent years, intense rainfall events have occurred frequently around the world [15], including this study area. These heavy rainfalls led to a sharp increase in the runoff, causing a dramatic increase in the flow rate in rainwater pipes. The design of typical municipal drainage systems is not able to handle such high volumes of rainwater, meaning a potential climatic cause for urban waterlogging in this study area. In addition to climate factors, there are also nonclimatic contributors to this issue, such as the acceleration of urbanization, rainwater and sewage combined sewers, and damaged, blocked, or corroded rainwater pipes.

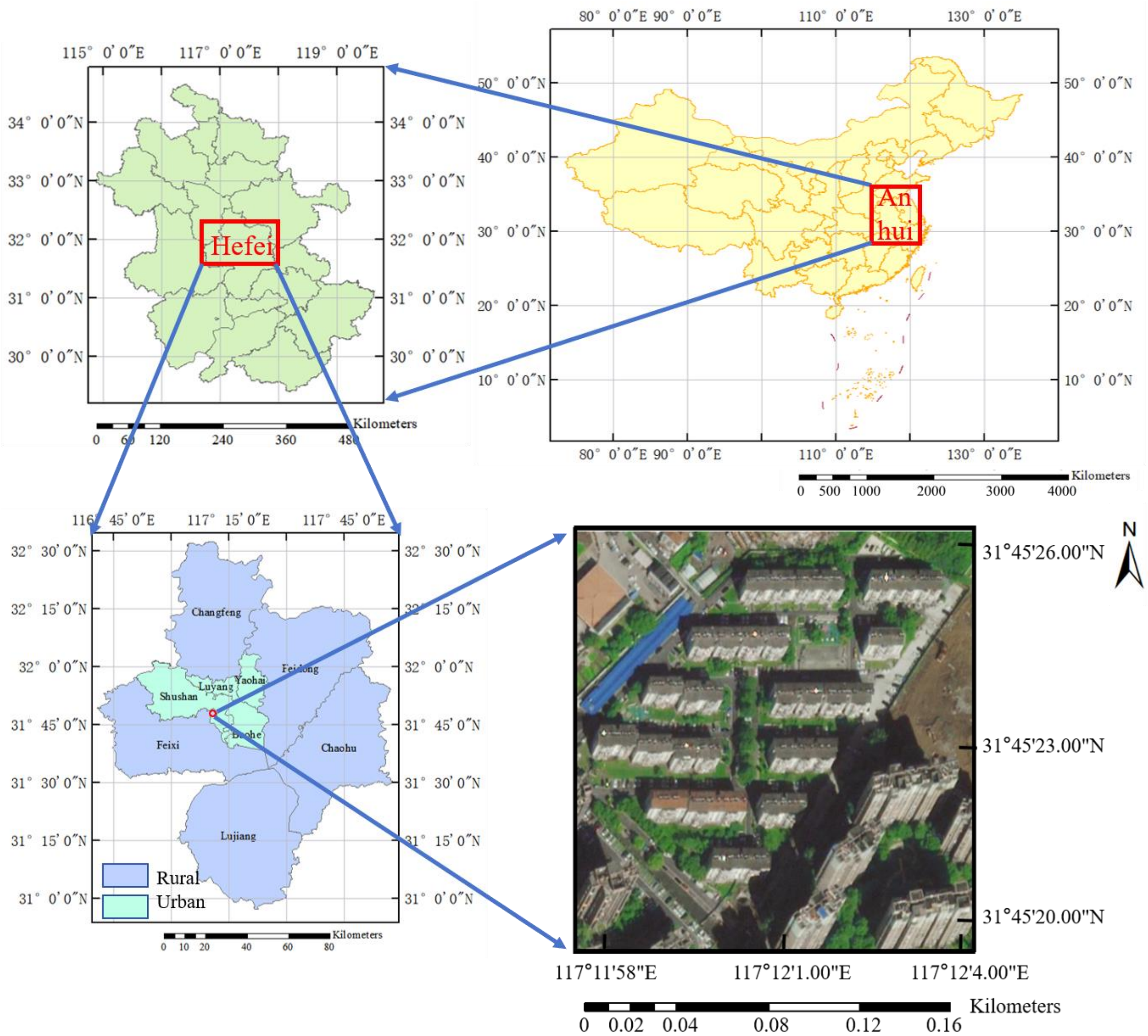

Figure 1. The area of this study, which is located in the fringe of Hefei city, near the suburban Feixi under the jurisdiction of the city, in Anhui Province in east China.

To solve the waterlogging problem in the area, the outdoor drainage pipes of rainwater were renovated, and the original rainwater pipe diameter was expanded (see Table S1); in addition, the sponge city facilities were set (see Table S2 and Figure S1). The SWMM was used to construct a hydraulic model of the rainwater pipe network to assess the drainage capacity of the renovation.

2.2. Construction of Rainfall Model

The area is characteristic of a northern subtropical monsoon climate with high rainfall in the spring and summer. According to a previous study [16], the typical local rainstorm intensity formula was obtained by Equation (1), as follows:

$$\mathrm{i} = \frac{25.8280(1 + 1.3659\lg\mathrm{P})}{(\mathrm{t} + 20.5150)^{0.9126}} \tag{1}$$

In Equation (1), i (mm/min) is the rainfall intensity; P (year, a) is the return period; t (min) is the duration of the rainfall. The rainstorm intensity formula above was chosen to simulate the rainfall amount in the research area, and the Chicago Rain Typer was applied to simulate the rainfall scenario with return periods of 2 a, 5 a, 10 a, 20 a, and 50 a, respectively, with the peak ratio (r) of 0.4 and the elapsed time of 120 min. Process lines of rainfall of different return periods are shown in Figure S2.

2.3. Modeling of Rainwater Pipes by SWMM

The SWMM was selected as the calculation model of the rainwater pipe network. The study area was generalized to 31 subcatchments (ZMJ), 58 rainwater inspection wells (J), 59 rainwater pipes (GQ), and 2 outlets (PFK) by the topography, as shown in Figure S3. The specific geological character of the rainwater pipe foundation is in Table S3. The infiltration model was chosen from Horton according to the previous literature [17]. The kinematic wave model was chosen and the time evolution step was designated as 10 s. The key parameters of the SWMM in this study were based on local engineering technical data, and the measured values are shown in Table 1, with the selected method according to a previous study [18]. The rainfall data constructed by the Chicago Rain Typer were input into the SWMM to simulate the flow rate of the pipes, the nodal rainwater depth, and the outlet discharge flow rate before and after the renovation, respectively.

Table 1. Important parameters of the SWMM model in this study.

Serial Number	Type of Parameter	Parameter Name	Value	Based
1	Roughness factor	Roughness coefficient of impermeable ground	0.024	Local engineering technical data
		Roughness coefficient of permeable ground	0.015	Local engineering technical data
		Roughness coefficient of rainwater pipes	0.013	Local engineering technical data
2	Permeability	Percentage of impermeability	95%	Local engineering technical data
		N value of impermeability	0.015	Local engineering technical data
		N value of permeability	0.024	Local engineering technical data
3	Horton model	Maximum infiltration rate	70 mm/h	Measured value
		Minimum infiltration rate	3.3 mm/h	Measured value
		Decay constant	4 h^{-1}	Local engineering technical data

3. Results and Discussion

3.1. Flow Analysis of Main Pipe Sections

According to the changes in the bottom slope and the diameter of the rainwater pipes in the area, the main trunk pipe sections GQ3, GQ13, GQ23, GQ25, GQ32, and GQ58 on the generalized map (Figure S3) were selected, and their maximum flow rate and water depth were analyzed. Taking the return period of 20 a as an example, the flow rates of the main trunk pipes before and after renovation are shown in Figure 2. It can be seen that, when the return period was 20 a, the maximum flow rate of the main trunk pipes after the renovation greatly increased. Among them, GQ58 increased the most from 0.78 m^3/s to 0.91 m^3/s, and whose discharge capacity increased by 16.70% as a result. The longitudinal diagrams of the main trunk pipes (from node J1 to PFK1) for the 2 a and 20 a return period are shown in Figure 3. It can be seen that, under the 2 a return period, the depth of the rainwater in the wells after the renovation slightly decreased and the rainwater pipe network before and after the renovation could meet the drainage demand. But, at the return period for 20 a, most of the main pipe sections before the renovation took the role of the pressure pipes, and the rainwater poured out of the wells, such as J5, J10, J9, J16, J18, and J21; however, the rainwater after the renovation did not rise out of the wells, although the flows in parts of the main pipe sections became pressure flows. It was a general phenomenon that, beyond a certain return period, the drainage pipes took the role of the risk pipes [19].

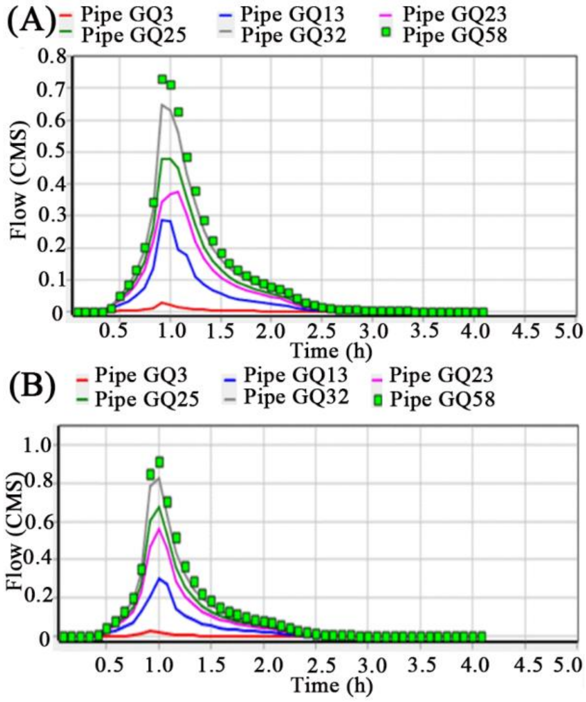

Figure 2. Flow rates of the main trunk pipes for the 20 a return period before renovation over the elapsed time (**A**) and those after renovation (**B**). CMS means cubic meter per second, which equates cubic meter per second (m^3/s).

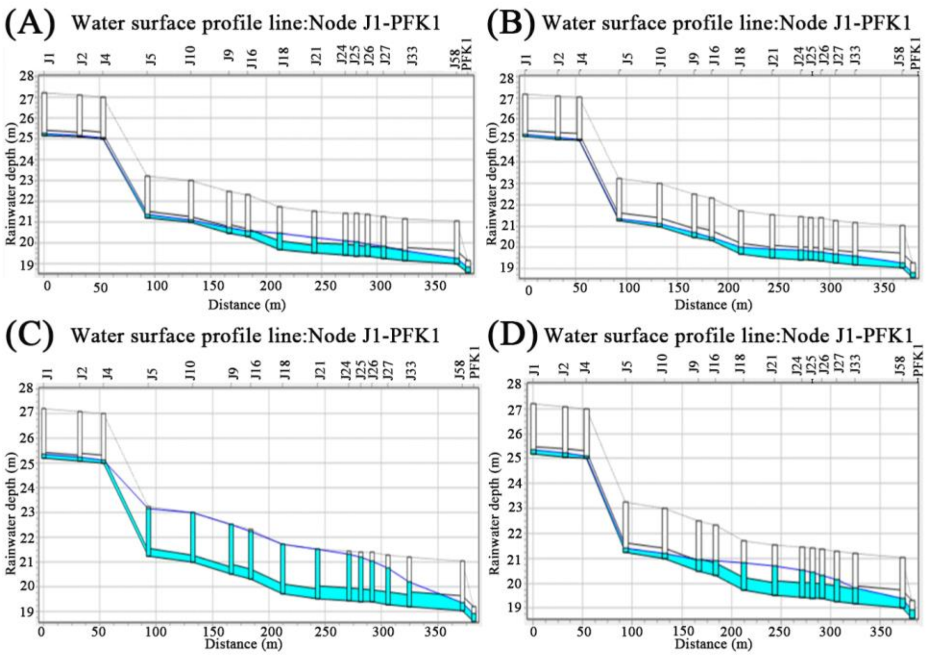

Figure 3. Longitudinal section of main trunk pipes under the 2 a and 20 a return periods: (**A**) Longitudinal section of the main trunk pipes under the 2 a return period before renovation; (**B**) Longitudinal section of main trunk pipes under the 2 a return period after renovation; (**C**) Longitudinal section of the main trunk pipes under the 20 a return period before renovation; (**D**) Longitudinal section of the main trunk pipes under the 20 a return period after renovation. Dark blue line represents the rainwater level of the rainwater wells; light grey line represents the elevation of the top of the rainwater wells along the route; and light blue shaded areas represents the rainwater filling situation in the rainwater wells and rainwater pipes.

Since GQ32 is the main pipe at the end of the pipe net, whose flow convergence area was large, it was considered as the most unfavorable pipe in the generalized area (Figure S3). After its drainage capacity was analyzed, its maximum drainage capacities before and after the renovation were compared, as seen in Figure 4. It can be seen that the latter improved more obviously, by 2.7%, under the return period for 2 a, and by 38.8% under the return period for 50 a. It is also obvious in Figure 4 that, with the return periods increasing, the rainwater pipe drainage capacity increased all the time below the return period of 50 a. The researchers used the SWMM to simulate the collective pipes of wastewater and also found the reduction in the peak flow and an improvement in the drainage capacity [20]. It was obvious that the SWMM was an effective tool to estimate the effectiveness of pipe improvement.

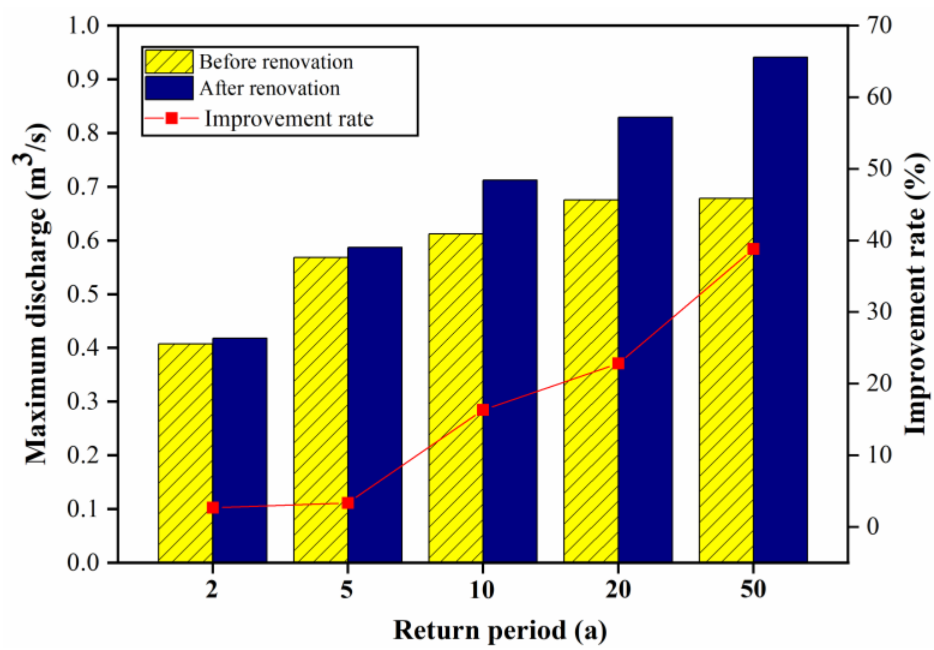

Figure 4. Maximum discharge of rainwater pipe GQ32.

3.2. *Water Depth Analysis for the Rainwater Pipes of Key Nodes*

According to the depth of the rainwater wells and the variation in the diameters of the rainwater pipes, the key pipe nodes on the trunk pipes (numbered J5, J18, J21, J24, J25, and J33) were selected and their maximum depths of rainwater under different return periods were analyzed. The results are shown in Figure 5.

As can be seen from the figure, the maximum water depth of the rainwater wells significantly reduced after the renovation of the rainwater pipes under the return periods of 2 a, 5 a, 10 a, and 20 a. When the return period was 2 a (Figure 5A), among the six key nodes, the gap of the amounts and rates of the maximum water depths at the rainwater well of node J18 before and after renovation was the largest, with the depth dropping by 0.49 m and the reduction rate being 63.5%. Under the return period of 5 a, the reduction rate at node J5 was the greatest, being 87.7% (Figure 5B). When the return period reached 10 a (Figure 5C), before the renovation, the rainwater wells of J5, J18, and J21 were not able to satisfy the drainage demand (the maximum water depth in the rainwater wells was set to 1.8 m and the depth of the wells was 2 m in this study) when the maximum rainwater depth of J18 reached more than 2 m, resulting in impounded surface water; after the renovation, J18, as the most unfavorable node, where the maximum depth was 0.97 m, met the requirements of drainage. Under 20 a return period (Figure 5D), the maximum depth of rainwater wells of J18 and J21 reached up to 2.0 m, and could not drain the water fully. However, after the renovation, the rainwater depths in these wells sharply decreased, with the one in the former dropping by 0.54 m and the one in the latter by 0.58 m, which indicates both of the nodes could meet the drainage capacity requirements, and all the others could do so as well. Finally, before renovation, all key-node rainwater wells could not meet the drainage capacity requirements under 50 a return period. But, after renovation, J24, J25, and J33 could do so, as shown in Figure 5E.

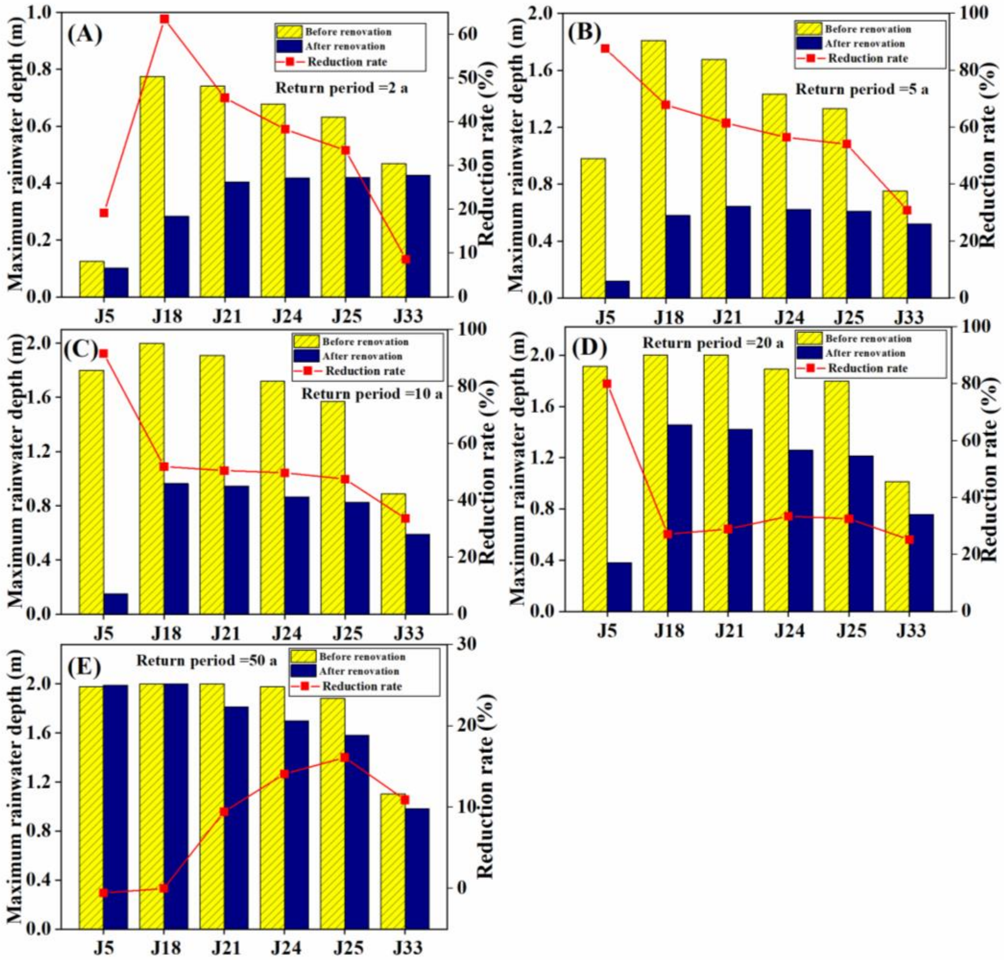

Figure 5. Maximum rainwater depth of rainwater wells at key nodes under different return periods: (**A**) 2 a, (**B**) 5 a, (**C**) 10 a, (**D**) 20 a, and (**E**) 50 a.

Furthermore, the changes in the rainwater depths at these key nodes over the rainfall time for the return periods of 20 a and 50 a before and after the renovation are shown in Figure 6. The rainwater depths at all of the six key nodes (rainwater wells) reached the maximum (in one hour of rainfall), which was much faster than that (over 6–7 h) in the previous study [21]. The main reason for this was the different return periods leading to different rainstorm intensities. The rainwater depths at all six key nodes after the renovation under the 20 a return period (Figure 6A,B) reduced greatly, which indicates that all six key nodes met the drainage requirements. But all the six nodes could not meet the drainage requirements before or after the renovation under the 50 a return period, as shown in Figure 6C,D, and the maximum rainwater depth of all the key nodes had no obvious change, indicating that the renovation could not meet the drainage capacity requirements for the 50 a return period. In fact, the reduction rate in the rainwater depth slowed down as the return periods increased, which could be due to more space being needed for rainwater discharge or because the pipes were already full with the return period increase [22].

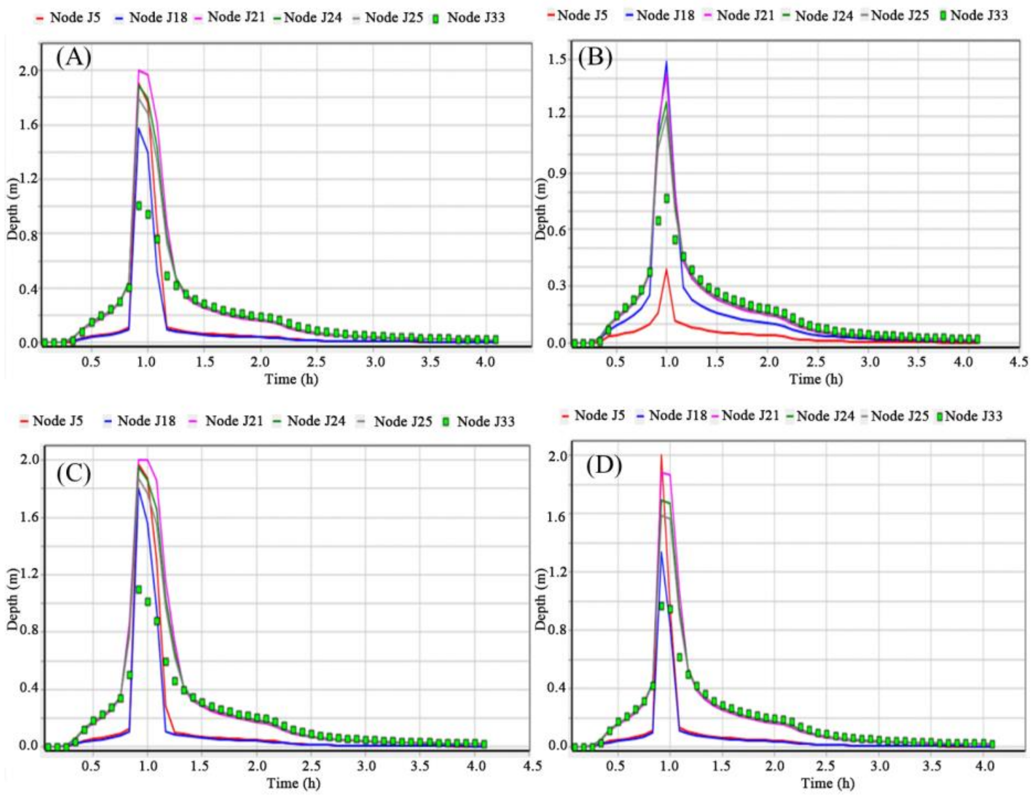

Figure 6. Rainwater depths at key nodes changed over time under the 20 a return period before (**A**) and after (**B**) renovation; rainwater depths at key nodes changed over time under 50 a return period before (**C**) and after (**D**) renovation.

3.3. Analysis of Outlet Flow

The percentage of water flow frequency, the average flow rate, the peak flow rate, and the total discharge for the two outlets (PFK1 and PFK2) in this study are shown in Figure 7. It can be seen that the percentage of water flow frequency slightly decreased after the renovation under different return periods for these two outlets, with that of PFK1 decreasing from 4.69% to 8.02% and that of PFK2 decreasing from 4.72% to 8.12% (Figure 7A). This indicated that the rainwater flow velocity in the pipes became slower after renovation, which played a protective role for the rainwater pipes. The average (Figure 7B) and peak flow rates (Figure 7C) at the two outlets both increased after renovation for all the return periods. This was probably due to the increase in the rainwater drainage capacity, which resulted from the expansion of the diameter of the rainwater pipes. Furthermore, with the return period increasing, the average and peak flow rates of the two outlets increased dramatically. This may be because the total rainwater runoff increased with the increase in the return period. Meanwhile, the total discharge of PFK1 increased obviously, while that of PFK2 increased slightly (Figure 7D). This may be because the flow rate of PFK1 was larger than that of PFK2 because it has a larger catchment area.

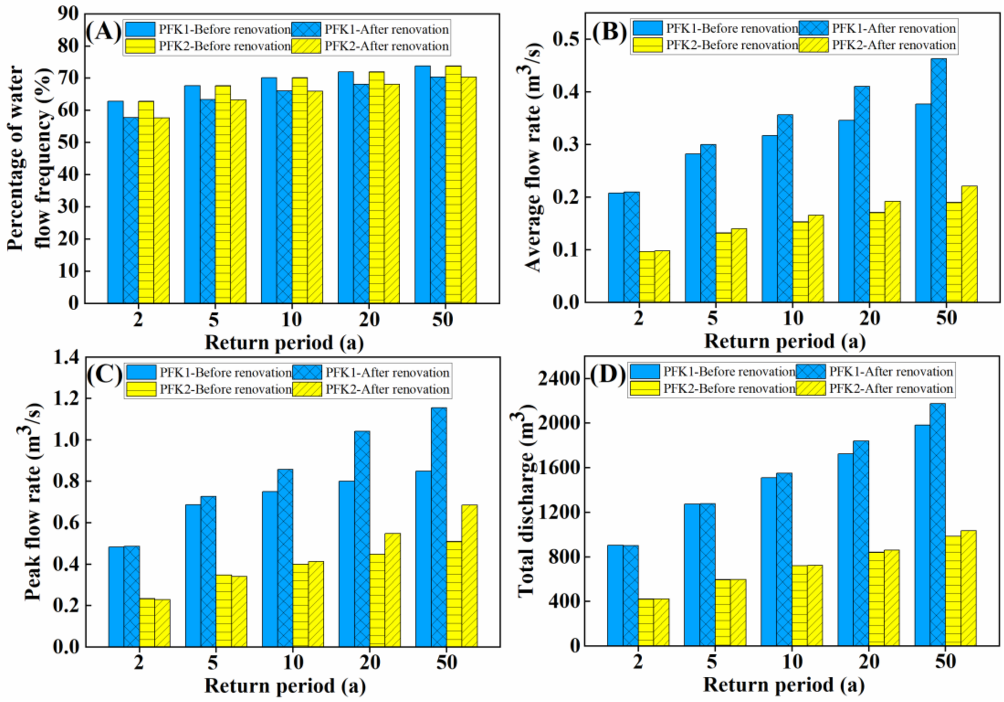

Figure 7. Four flow parameters of two discharge outlets (PFK1 and PFK2) before and after renovation: (**A**) Percentage of water flow frequency; (**B**) Average flow rate; (**C**) Peak flow rate; (**D**) Total discharge.

3.4. Addition of Sponge City Facilities in This Renovation

In addition to largening the diameters of the rainwater pipes, the installation of sponge city facilities, which has been often used in projects of urban waterlogging control in China in recent years, is also considered to an effective way to alleviate urban waterlogging. Therefore, on the basis of the renovation of the rainwater pipe network, sponge city facilities were included in this study. According to the topographic and climatic characteristics of this study area, four types of sponge city facilities (infiltration roads, planting ditches, permeable pavements, and permeable parking lots) were selected. The size and proportion of sponge city facilities are shown in Table S2, and their locations in Figure S1.

In general, the criteria used for assessing waterlogging control by sponge city facilities mainly include the reduction in the total amount of rainwater runoff from the catchment area, the decrease in the maximum water depth in the rainwater wells, and the reductions in the four flow parameters (the average flow rate, the peak flow rate, the total discharge, and the percentage of water flow frequency) of the discharge outlets. Therefore, three sections were explored for the assessment of waterlogging control by sponge city facilities, as follows.

3.4.1. Analysis of Rainwater Runoff Reduction

The addition of sponge city facilities can effectively reduce the amount of surface runoff volume and the surface rainwater accumulation. The total amount of rainwater runoff from the catchment area with and without sponge city facilities after the renovation were simulated by the SWMM. As shown in Table 2, the addition of sponge city facilities reduced the amount of rainwater runoff ranging from 401.5 m^3 under 2 a return period to 504.4 m^3 under 50 a return period, and the rainwater runoff reduction rate decreased from 28.68% in 2 a to 14.78% in 50 a. This was mainly due to the rainwater storage of the sponge city facilities. One study used the LID scheme of segmental detention and retention to reduce the peak outflow to 25% compared to the predesign scheme under 5 a return period [11]. An optimized LID implication was used in Windsor, Ontario, reducing the total volume of the runoff by 13% in a previous study [12]. Another study found that the minimum runoff reduction under five return periods was 42.8% by the combined LID facility [13]. The application of the LID could reduce runoff and flood, but the runoff reduction rates involved in the abovementioned studies vary from each other, between which the rate obtained in this study lies. This was because different proportions and styles of the LID (sponge city facilities) were used in those studies.

Table 2. Total rainwater runoff flow of the catchment area.

Program	Reproduction Period				
	2 a	**5 a**	**10 a**	**20 a**	**50 a**
No sponge city facilities after renovation (m^3)	1399.79	1970.34	2394.97	2838.62	3412.55
Sponging facilities after renovation (m^3)	998.29	1545.72	1950.76	2374.06	2908.15
Runoff reduction (m^3)	401.5	454.62	444.21	464.56	504.4
Runoff reduction rate	28.68%	21.55%	18.55%	16.37%	14.78%

As can be seen from Table 2, the reduction rates of the surface rainwater runoff resulting from the addition of the sponge city facilities were 28.68%, 21.55%, 18.55%, 16.37%, and 14.78% for the return periods of 2 a, 5 a, 10 a, 20 a, and 50 a, respectively. This indicated that the reduction rate decreased as the return periods increased, when the intensity of the storm increased, while the total rainwater storage volume in a sponge facility remained unchanged. Therefore, the addition of sponge city facilities to the rainwater network renovation can effectively hold some of the rainwater and reduce the volume of rainwater runoff, but the effect would become weaker as the return period increased.

3.4.2. Analysis of Key Node Water Depths after Addition of Sponge City Facilities

After the addition of the sponge city facilities in the renovation, the maximum water depth in the rainwater wells decreased compared to that when there were no sponge city facilities. The reductions at key nodes (J5, J18, J21, J24, J25, and J33) are shown in Figure 8. It can be seen the reduction rates in the maximum rainwater depth in the wells changed under different return periods when there were sponge city facilities. The maximum reduction rate, being 61.15%, appeared in the J5 rainwater well under 20 a return period. This might be because the pipe bottom fall between the upstream and the downstream in the J5 rainwater well was bigger than those at other key nodes, which led to the original rainwater depth in the well being lower than those at other key nodes. As regards all the key nodes, the reduction rates in the maximum water depth were obvious below 20 a return period. Previous researchers also reported the rainwater depth of the sponge city facilities group was much lower than that of without the sponge city facilities group at 20 a return period [23]. However, only the rates in J5 and J33 were 43.61% and 33.74%, respectively, and that of the other key nodes was zero or negative under 50 a return period. These phenomena indicated that sponge city facilities could effectively reduce the rainwater depth in the wells below a certain return period. In fact, the decrease was attributed to sponge city facilities' rainwater retention, which depended on the area and style of a certain sponge facility, and which would reach the upper limit with increasing rainfall. Therefore, the maximum rainwater depth in the wells stopped decreasing when the rainfall exceeded a certain amount, which was equivalent to return period pass over a certain value. Hence, in this study, the maximum rainwater depths at the four key nodes (J18, 21, 24, and 25) did not decrease after the return period increased to 50 a.

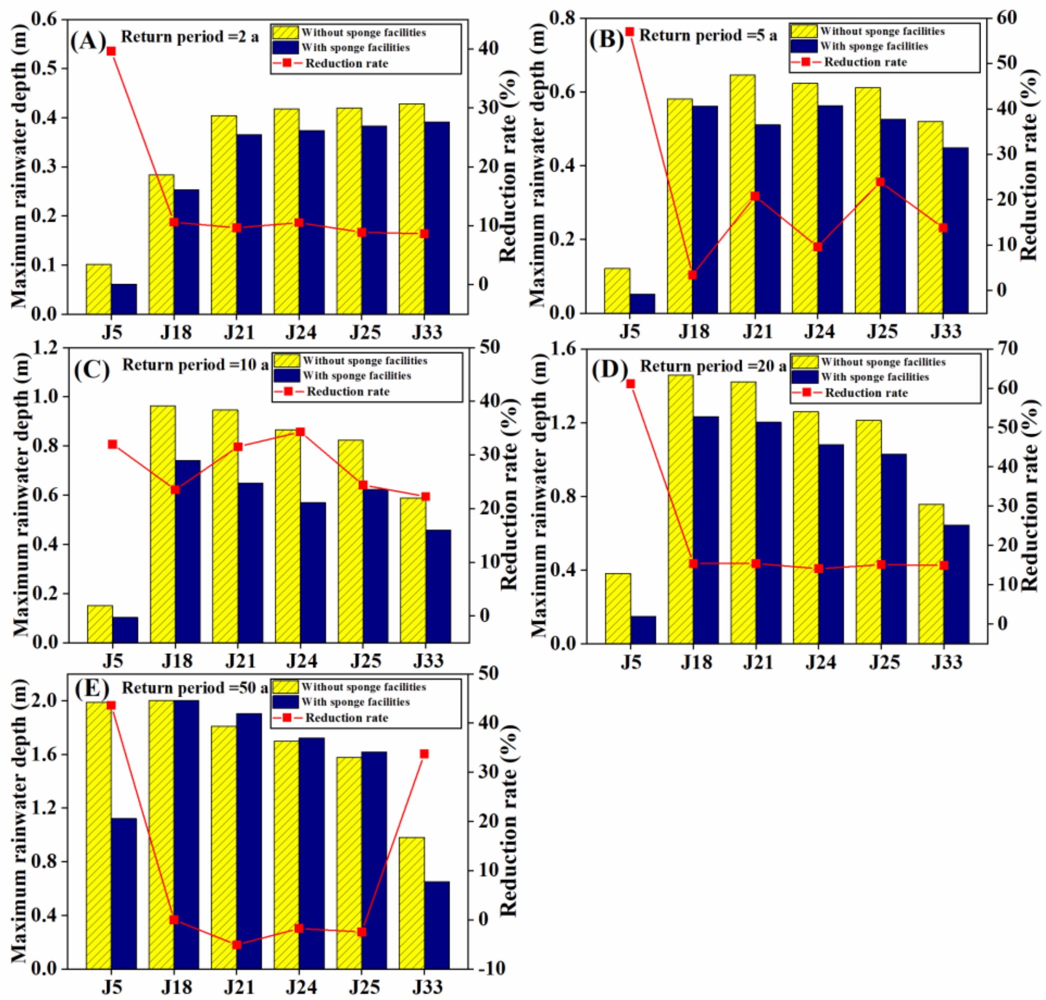

Figure 8. Maximum water depths of rainwater wells at key nodes with or without sponge city facilities under different return periods: (**A**) 2 a, (**B**) 5 a, (**C**) 10 a, (**D**) 20 a, and (**E**) 50 a.

The changes in the rainwater depths at the key nodes over time for the return periods of 20 a and 50 a are shown in Figure 9, which compares the rainwater pipes after renovation with and without sponge city facilities. It can be seen that the peak rainwater depths in all key node wells with the addition of sponge city facilities decreased obviously compared with those without the addition under either 20 a or 50 a. This indicated that the sponge city facilities could detain the rainwater and reduce the volume of rainwater runoff and the pressure on drainage facilities. In this study, the bottom areas of the rainwater wells at these key nodes were the same as each other, which was set to 1 unit (the details see Table S4). Therefore, the curve integral areas of the rainwater depth over the elapsed time in the figure could express the rainwater volume through the rainwater wells. This is why we used the curve integral area to indirectly express the accumulated rainwater volume through the rainwater wells during the elapsed time. The curve integral area of the key nodes is shown in Table S4. The amount of the integral area of the rainwater depths in the wells over the elapsed time with addition of sponge city facilities were obviously less than that of without addition of sponge city facilities for all the key nodes under both the 20 a and 50 a return periods. This further indicated that renovation with sponge city facilities had the effects of rainwater storage and detention and reduced the risks of flooding.

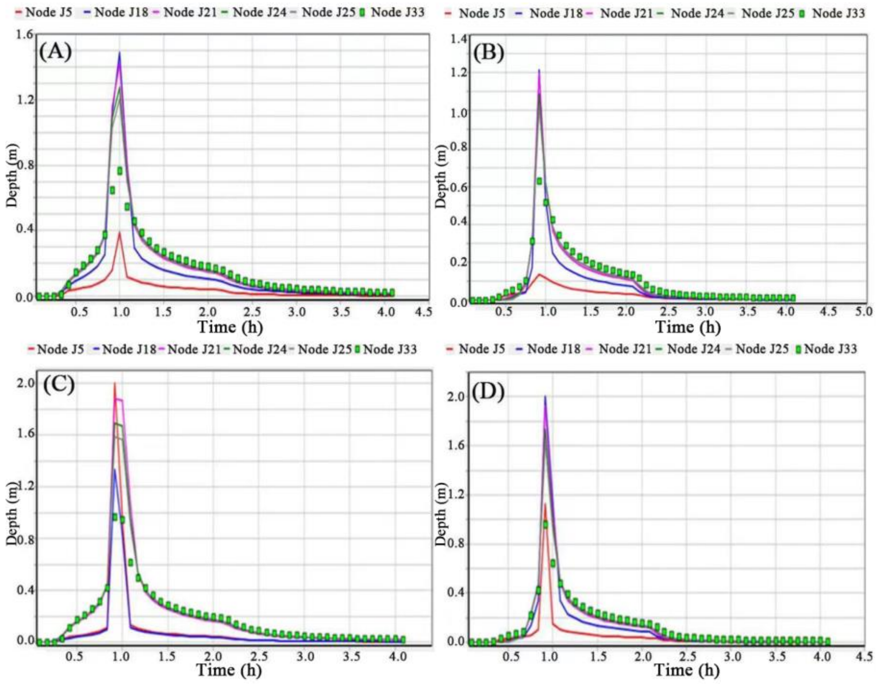

Figure 9. Changes in the rainwater depths of the rainwater wells at key nodes after renovation with or without sponge city facilities over the time: (**A**) 20 a without sponge city facilities; (**B**) 20 a with sponge city facilities; (**C**) 50 a without sponge city facilities; (**D**) 50 a with sponge city facilities.

An unexpected phenomenon is shown in Figure 9. The peak rainwater depth in wells at all key nodes after the addition of sponge city facilities appeared slightly earlier than that without the sponge city facilities. This phenomenon can be explained by the fact that, with the addition of sponge city facilities, the volume of rainwater collected from the catchment areas in the rainwater pipes reduced, resulting in the earlier peak rainwater depth of the rainwater well. And the accumulated rainwater volume through the rainwater wells during the elapsed time reduced with addition of sponge city facilities, as described above, which also verifies the explanation.

3.4.3. The Outlet Flow with and without Sponge City Facilities

Reductions in the four flow parameters (the average flow rate, the peak flow, the total discharge and the percentage of water flow frequency) of the two discharge outlets (PFK1 and PFK2) with the addition of sponge city facilities are shown in Figure 10. It can be seen from Figure 10A,B that changes in the reduction rates of the average flow rate and peak flow under different return periods did not show a clear trend. However, those of PFK2 were higher than those of PFK1 at five different return periods in general. From Figure 10C,D, it can be seen the trend of the reduction rates in the total discharge at different return periods was consistent with those of the water flow frequency, which was that the reduction rates in PFK2 were higher than those of PFK1 at different return periods. In a sense, the water flow frequency indirectly represents the flow rate when the catchment area of the outlet is fixed. Therefore, the reduction rates in the two parameters under different return periods kept the same pace. In fact, the changes in the four flow parameters over different return periods showed that those of PFK2 were higher than those of PFK1. In this study, the catchment area of PFK2 was lower than that of PFK1, resulting in less rainwater in the pipes in the catchment area of PFK2 than of PFK1 (Table S5). When the sponge city facilities collected and stored certain rainwater, it had a greater impact on the four flow parameters of PFK2 than on those of PFK1.

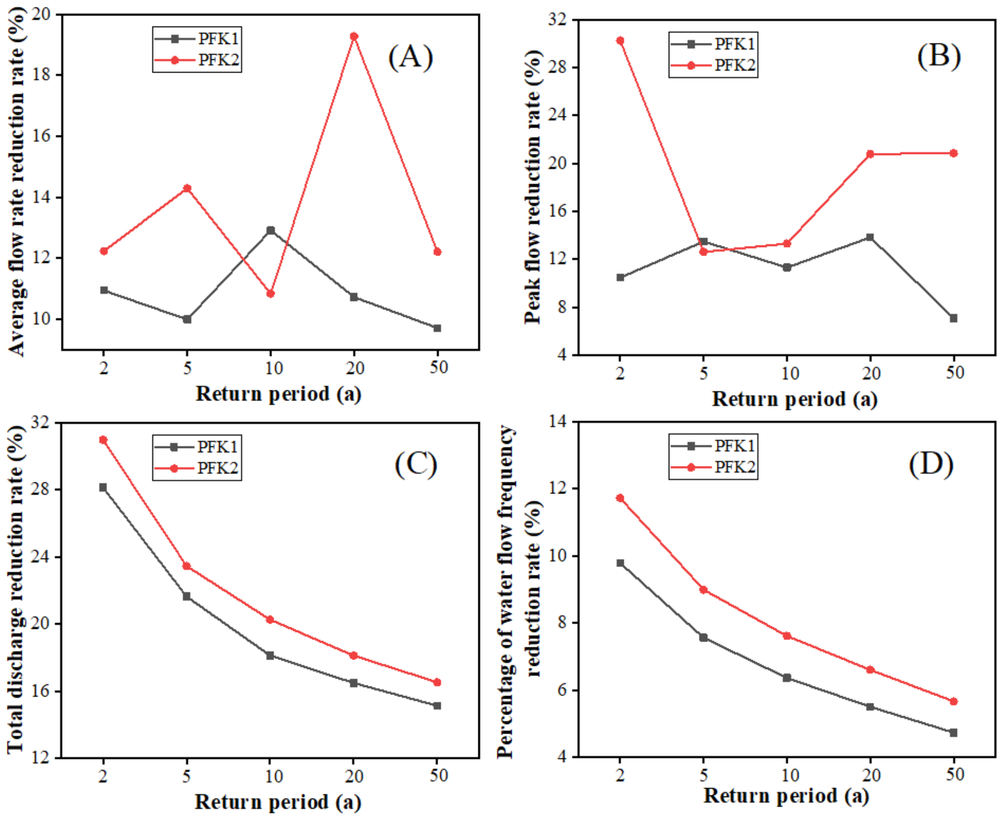

Figure 10. Reduction rates in the four flow parameters of two discharge outlets (PFK1 and PFK2) with addition of sponge city facilities compared to those without: (**A**) Average flow rate reduction; (**B**) Peak flow reduction; (**C**) Total discharge reduction; (**D**) Percentage of water flow frequency reduction.

To sum up, this indicates that the addition of sponge city facilities could reduce the flow rate in the rainwater pipes, relieve the pressure on the rainwater drainage pipes, and, at the same time, have some effect on water storage and retention, effectively avoiding waterlogging.

4. Conclusions and Outlook

In this article, the hydraulic characteristics of rainwater pipes, rainwater wells, and outlets before and after the renovation were systematically analyzed in the urban fringe, including the discharge capacity of rainwater pipes, the maximum depth of rainwater wells, and the flow circumstances of the outlets. The results show: firstly, the renovated rainwater pipes could satisfy greater storm intensity and the discharge capacity significantly increased; secondly, the maximum rainwater depth in the rainwater wells at key nodes decreased significantly after renovation; thirdly, the average flow rates at the outlets increased after renovation and the percentage of water flow frequency decreased, which reduced the pressure of the downstream rainwater pipe discharge; finally, the curve integral areas of the rainwater depth in the rainwater wells over the elapsed time indicated that the accumulated rainwater in the rainwater wells decreased after the renovation of the added sponge city facilities, further indicating that the sponge city facilities played a certain role in water storage and retention, reducing the risks of waterlogging in the fringe of urban areas.

As we know, the SWMM is a good way to quantitatively evaluate the rainfall runoff control involved in a flood control scheme in a residential community and can provide a scientific basis for engineering design [18,24,25]. Though there have been many studies on waterlogging control by the SWMM in urban areas, few have focused on old residential quarters in low-terrain areas in urban fringes. This study verified that the SWMM was well applied to the urban fringes and, therefore, broadened the scenarios of the SWMM application. But it is still difficult for the SWMM parameters in this study to be applied to other locations, especially where geological conditions and climatic conditions are different from of those of this study area, which is the limitation of this study. Additionally, the model parameters were based on location engineering technical data and measured value in this area because the essential facilities were not actually built in the study area at that time; thus, there was not much for the model to predict under different actual rainfall events (see Table S6), which was similar to what it was like in previous study [26]. In spite of this, the main contributions of this study are still invaluable. Therefore, a local model database should be built so that the information disclosure and sharing system will be improved. Nevertheless, future research should aim to the further study of the application of the generalizability of the results to other locations. The SWMM is primarily used for one-dimensional (1D) stormwater network simulations, and is not able to represent the hydrodynamics of 1D networks as well as two-dimensional (2D) surface water accumulation [27]. The accuracy of the SWMM model often depends on the completeness and precision of the underlying data [28]. However, such data are frequently hard to obtain, which results in low accuracy and weak visualization for urban waterlogging. Therefore, integrating the SWMM with other software to develop an accurate, two-dimensional, and visualized urban waterlogging control model is the alternative for the SWMM model in future urban stormwater management.

In this study, four types of sponge city facilities were selected according to the topographic and climatic characteristics of this study area; however, it was rare to optimize the combination of these sponge city facilities. In fact, the optimized combination of different sponge city facilities will be another future research direction [5]. Furthermore, the addition of sponge city facilities can effectively reduce rainwater runoff, and when the sponge city facilities are added to a certain percentage, to lay rainwater pipes or not, or to lay fewer pipes is to be researched in future sponge city construction and rainwater piping projects.

Supplementary Materials: The following supporting information can be downloaded at: https://www.mdpi.com/article/10.3390/w16040620/s1, Table S1: Information about the pipes before and after renovation in this area. The material of the pipes before and after renovation were high-density polyethylene (HD-PE); Table S2: Area size and proportion of sponge facilities in the study area; Table S3: Infiltration rate and geological character in the study area; Table S4: The curve integral areas of the rainwater depth over the elapsed time of the key nodes; Table S5: The variation in the four flow parameters of the two outlets (PFK1 and PFK2) after renovation with and without sponge facilities; Table S6: The measured and predicted flow rates of the two discharge outlets (PFK1 and PFK2); Figure S1: The layout of the sponge city facilities in this study area; Figure S2: Rain intensity under different return periods by the Chicago Rain Typer; Figure S3: Generalized map of rainfall pipes in this study area. ZMJ means subcatchments; J means rainwater inspection wells; GQ means rain water pipe; PFK means outlet; the number represents the sequence number. Pipes GQ3, GQ13, GQ23, GQ25, GQ32, and GQ58 were selected for the maximum flow rate and water depth analysis, which were marked red on the map.

Author Contributions: Conceptualization, H.H. and Y.L.; methodology, H.H. and Y.L.; software, R.L.; validation, H.H. and Y.L.; formal analysis; investigation, Q.Z.; resources, J.D.; data curation, B.W.; writing—original draft preparation, B.W. and J.D.; writing—review and editing, H.H. and Y.L.; visualization, Y.L. and R.L.; supervision, H.H.; project administration, H.H. and Y.L.; funding acquisition, H.H. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by the China Postdoctoral Science Foundation (2021M693071), the Natural Science Research Project of Higher Education Institutions of Anhui (KJ2021A0618), Campus-Level First-Class Course: Sponge City and Stormwater Management (2023xsxx05), An Offline Course for Graduate Students at the Provincial Level: Avanced Fluid Mechanics (2022xxsfkc032) and the Entrepreneurship and Innovation Talent Plan of Taizhou in Jiangsu Province (2021). Anhui Provincial Teaching Quality project: Fluid Mechanics (2021jyxm0270).

Data Availability Statement: Data are contained within the article and Supplementary Materials.

Acknowledgments: We are grateful to the editors and anonymous reviewers for their thoughtful comments.

Conflicts of Interest: The authors declare no conflicts of interest.

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