An optimal scenario for the emergency solution to protect Hanoi Capital from the Red River floodwater using Van Coc Lake

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Abstract
The Red River dike system was built, and an emergency solution was implemented to protect Hanoi Capital city from flood disasters. In the solution, floodwater from the Red River is discharged through the Van Coc Gate, with an overflow point into Van Coc Lake, and is drained downstream through the Day Weir. However, an optimal scenario for the operating procedure of the emergency solution, based on a comprehensive evaluation of the flood risk level from the Red River to the residential areas located outside the protected areas of the dike system, has not been adequately addressed to date. In this study, we employed the latest digital elevation map in a high spatial resolution for the first time, and a two-dimensional depth-integrated hydrodynamic model was utilised to obtain the optimal operation for the emergency solution. The optimal scenario was determined to minimise the flood risk level to the residential areas and optimise the inflow discharge at the Van Coc Gate and the overflow point, and the outflow discharge at the Day Weir for ensuring the safety of the operating system. This study lays a promising foundation for designing risk-reduction strategies in Hanoi Capital.

KEYWORDS
flood disaster, flood risk level, hazard map, two-dimensional depth-averaged model

1 INTRODUCTION

Hanoi is the second-largest city in Vietnam and was built within the Red River Delta, as shown in Figure 1. In common with most regions in northern Vietnam that are located in the Southeast Asian monsoon area, Hanoi is directly affected by the hot and humid climate of the Pacific and Indian Oceans. Consequently, heavy rainfall events and serious floods have frequented the Red River Delta, including Hanoi Capital (Vietnamese Government, 2005). Furthermore, according to the Association of Southeast Asian Nations (ASEAN) Coordinating Centre for Humanitarian Assistance on Disaster Management and Japan International Cooperation Agency in 2015, Vietnam has been identified as one of the five worst-affected countries by climate change, and the Red River basin has been profoundly influenced by climate change (AHA Centre & JICA, 2015). Environmental changes caused by events such...
FIGURE 1  Location of the research area consisting of a part of the Red River area that runs through the Hanoi Capital from Son Tay Station to Hung Yen Station (a). The location of Van Coc Lake (b), the elevation map (c), the protected, and outside protected areas are showed (d).
as urbanisation and/or climate change have worsened the flooding in Hanoi Capital.

The Red River is the largest in the north of Vietnam and runs through the Hanoi Capital. The river faces challenges from floods and drought, especially in the context of climate change and rural development (MONRE, 2012). Flood is characterised by a rapid increase in the river water level, and the flood season in the Red River system occurs from May to September with 3–5 annual floods in the basin (AHA Centre & JICA, 2015). In the last 50 years, floods, which occurred in the years 1945, 1969, 1971, 1984, 1986, 1996, 2002, and 2008, have significantly impacted Hanoi Capital, causing serious damage to homes, possessions, and people (AHA Centre & JICA, 2015). The three recorded flood disaster events occurring in the Red River basin in 1945, 1969, and 1971 had caused huge damage to millions of people, dike failures in numerous places, hundreds of hectares of land inundated, and incurring huge restoration cost (Vietnamese Government, 2005). The flood of 1971 was one of the 10 worst floods of the 20th century. A total of 100,000 people died in this disaster (NOAA, 1993). The high number of casualties indicated the very high vulnerability of the residential areas located in the floodplains. Besides, floods have an extreme effect on a region’s socioeconomic health (Khosravi et al., 2019). Until now, protecting Hanoi Capital from floodwater from the Red River in emergencies has been an important mission of Vietnam, and many solutions have been provided by the government and its stakeholders (Vietnamese Government, 1999, 2007a, 2007b, 2011).

As shown in Figure 1, the Hanoi Capital is protected by the Red River dike system, and in emergencies, floodwater from the Red River is discharged into Van Coc Lake, which is a regulating reservoir located in the Dan Phuong and Phuc Tho Districts (30 km from the centre of Hanoi Capital), through the Van Coc Gate and the overflow point on the bank of the Red River and is drained to the downstream through the Day Weir. Figure 1 shows the Red River dike system that is marked as a red dot line to separate the river from other areas of Hanoi Capital. Therefore, the river area inside the dike system was determined as the outside protected area of the dike system. Further, the protected areas from the floodwater of the Red River are shown in Figure 1. However, the Hanoi Capital has experienced rapid urbanisation during the last few decades, and there are many residential areas located along the river and outside the protected area of the dike system (the river area inside the dike system) in the Hanoi Capital. The residential areas are highly vulnerable during flood disasters. Moreover, moving thousands of households located in the outside-protected areas to safer areas is impossible in the near future in a developing country like Vietnam. Therefore, flood risk management needs to be evaluated in greater detail than ever. In flood risk management, flood risk mapping and modelling are important for preventing flood damage, and determination of flood-prone areas is a fundamental step (Darabi et al., 2019; De Risi, Jalayer, & De Paola, 2015). Moreover, flood risk management requires knowledge of the impacts of flood, including water depth and flow velocity, hazardous processes (Vu & Ranzi, 2017; Zischg, Mosimann, Bernet, & Röthlisberger, 2018), and the application of depth-damage functions is the most common method for the estimation of flood damage (Jongman et al., 2012). An optimal scenario for the operating procedure of the emergency solution based on a comprehensive evaluation of the flood risk level from the Red River to the residential areas is an essential part of the overall development planning of Hanoi to avoid critical developments in the river basin without adequate protection.

The main aim of this paper is to propose an optimal scenario to minimise the flood risk level to residential areas and optimise the inflow discharges at Van Coc Gate and overflow point, and the outflow discharge at the Day Weir to ensure the safety of the operating system in emergencies. Moreover, the comprehensive assessment results of the impact of floods on residential areas located outside the protected area of the Red River dike system has been put at the forefront as important information for the overall development planning of Hanoi. For the first time, the conjunction of a two-dimensional (2D) hydrodynamic model and the latest digital elevation map (DEM) with the high spatial resolution was utilised to assess flood-related risks in setup scenarios precisely and find the optimal solution. The model was validated by using the observed data from the Red River and dealt with the downstream boundary conditions in scenario analyses by utilising the uniform flow assumption. The wetting and drying scheme was introduced for the research area, since the accuracy and stability of the numerical simulation model were affected by the treatments for wet-and-dry fronts, especially in areas with complex geometries (Liu et al., 2013).

2 MATERIALS AND METHODS
2.1 Study area and scenarios

Figure 1 depicts the location and elevation map of the research area comprising a part of the Red River area that runs through Hanoi Capital from the Son Tay Station to Hung Yen Station, and Van Coc Lake area. The Red River area was outside the protected areas of the Red River dike system, approximately 110 km in length from the Son Tay Station to Hung Yen Station. Figure 2 shows...
the longitudinal profile of the elevation of the dike crest, dike toe, and river bottom at the Son Tay, Thuong Cat, Long Bien, and Hung Yen Stations and at the overflow point along the Red River. The elevations of the crest and toe of the right and left dikes decrease from the Son Tay Station to Hung Yen Station. Besides, the river bottom elevation is complex and drops from $-4 \text{ m}$ at the Son Tay Station to $-19 \text{ m}$ at the Hung Yen Station. The highest water level in the Red River when the flood reached its peak at Long Bien Station in the sixth scenario, which is described in Section 3.2.6, is also shown in this figure.

For protecting Hanoi Capital from flood disaster, the Red River dike system was built many centuries ago, and an emergency solution was implemented since 1937. In the solution, floodwater is discharged into Van Coc Lake as a regulating reservoir through the Van Coc Gate and the overflow point located on the bank of the Red River before draining to the Day River through the Day Weir. The Van Coc Gate has 26 controlled gates with a sluice gate form, and Day Weir has six controlled gates with a radial gate form. In emergencies, they are fully open. A conceptual model for the Van Coc Gate, Overflow point, Day Weir, and the Day River in emergencies is shown in Figure 3. The pictures of the upstream and downstream of the Day Weir and Van Coc Gate are shown in Figure 4. To find an optimal scenario to protect Hanoi Capital from floods, 11 scenarios, listed in Table 1, were set up. In Case 0, Van Coc Lake is not utilised as a regulating reservoir. In Case 1, only the Van Coc Gate and the Day Weir are operated. In Case 2, Van Coc Gate, Day Weir, and overflow point with various opening widths and $13.0 \text{ m}$ from the bottom level are operated together (Vietnamese Government, 2016a, 2016b). The dike crest elevation at the overflow point is $15.0 \text{ m}$; therefore, the height of the opening section was $2.0 \text{ m}$.

In Hanoi Capital, there is a flood warning station called Long Bien Station, shown in Figure 1. If the river water level touches $10.5 \text{ m}$ and $11.5 \text{ m}$ at the station, the flood warning levels are put into motion as Warning Levels 2 and 3, respectively, and flood protection measures that depend on the flood warning level are implemented by the Hanoi Capital local government and Vietnamese government.

### 2.2 Hydrodynamic model

Many hydrodynamic models, including 2D and three-dimensional (3D) models, are currently used to simulate water dynamics. Recently, 3D models have been used widely to assess the flow in rivers, lakes, and coastal areas (Abedini, Ghiassi, & Box, 2010; Lu, Zhang, Song, Yue, & Wen, 2015; Wu & Lin, 2015). However, 3D models are time-consuming for a huge research area. In addition, if the research area includes rivers and huge floodplain areas, as in this study, the horizontal flows would be much larger than the vertical ones on the major part of the floodplain when the floodwater flows into the area. In such a case, a 2D model is often used to simulate the flow behaviour effectively (Bates, Wilson, & Stewart, 1999; Bellos & Tsakiris, 2015; Dutta, Alam, Umeda, Hayashi, & Hironaka, 2007; Gharbi, Soualmia, Dartus, & Masbernat, 2016; Hu & Kot, 1997; Tabata, Hiramatsu, Harada, & Hirose, 2013) and the deployment of a
uniform rectangular mesh system in a 2D simulation is convenient for practical studies (Liang, Lin, & Falconer, 2007). Therefore, a 2D model was built herein to simulate the floodwater behaviour in the area. The shallow water equations used in this study are as follows:

![Upstream of the Day Weir](image1.png) ![Downstream of the Day Weir](image2.png) ![The Van Coc Gate](image3.png)

**FIGURE 4** The pictures of the upstream and downstream of the Day Weir and the Van Coc Gate

**TABLE 1** All scenarios for operating procedure of the Van Coc Gate, Day Weir, and various opening widths of the overflow point

| Scenario | Van Coc Gate | Day Weir | Overflow point Width (m) |
|----------|--------------|----------|-------------------------|
| Case 0   | 1st Close    | Close    | Close                   |
| Case 1   | 2nd Open     | Open     | Close                   |
| Case 2   | 3rd Open     | Open     | 450 m                   |
|          | 4th Open     | Open     | 650 m                   |
|          | 5th Open     | Open     | 850 m                   |
|          | 6th Open     | Open     | 1,000 m                 |
|          | 7th Open     | Open     | 1,200 m                 |
|          | 8th Open     | Open     | 1,400 m                 |
|          | 9th Open     | Open     | 1,800 m                 |
|          | 10th Open    | Open     | 2,200 m                 |
|          | 11th Open    | Open     | 2,600 m                 |
Continuity equation:

$$\frac{\partial \eta}{\partial t} + \frac{\partial}{\partial x}\{U(h + \eta)\} + \frac{\partial}{\partial y}\{V(h + \eta)\} = 0$$  \hspace{1cm} (1)

Momentum equation in the x and y directions:

$$\frac{\partial U}{\partial t} + U \frac{\partial U}{\partial x} + V \frac{\partial U}{\partial y} = fV - g \frac{\partial \eta}{\partial x} + v_h \left( \frac{\partial^2 U}{\partial x^2} + \frac{\partial^2 U}{\partial y^2} \right) - \frac{gn^2U\sqrt{U^2 + V^2}}{(h + \eta)^{4/3}}$$ \hspace{1cm} (2)

$$\frac{\partial V}{\partial t} + U \frac{\partial V}{\partial x} + V \frac{\partial V}{\partial y} = -fU - g \frac{\partial \eta}{\partial y} + v_h \left( \frac{\partial^2 V}{\partial x^2} + \frac{\partial^2 V}{\partial y^2} \right) - \frac{gn^2V\sqrt{U^2 + V^2}}{(h + \eta)^{4/3}}$$ \hspace{1cm} (3)

where $U$ and $V$ are the depth-averaged horizontal velocity components (m/s) in the x- and y-directions, respectively; $\eta$ is the river water level (m); $t$ is time (s); $h$ is the bottom elevation (m); $f$ is the Coriolis forces (1/s); $g$ is the gravitational acceleration (m/s$^2$); $n$ is the Manning’s coefficient of roughness (s/m$^{1/3}$); and $v_h$ is the coefficient of the eddy viscosity (m$^2$/s).

The coefficient of the eddy viscosity $v_h$ was calculated using the Smagorinsky model (1963) as follows:

$$v_h = \frac{1}{2} S_m A_G \left( \frac{\partial U}{\partial x} \right)^2 + \frac{1}{2} \left( \frac{\partial V}{\partial x} + \frac{\partial U}{\partial y} \right) \left( \frac{\partial V}{\partial y} \right)^2 \right)^{1/2}$$ \hspace{1cm} (4)

where $S_m$ is the Smagorinsky coefficient, and $A_G$ is the area for each mesh (m$^3$).

The method of Uchiyama (2004) was applied to treat each wetting or drying cell. In this method, the total local depth is used to define the wetting or drying mesh, and a land mask function (LMF) is applied. LMF = 1 indicates that the cell is considered “wet,” whereas LMF = 0 shows that the cell is “land.” The process is recalculated after each time step for all the cells. A threshold depth value $d_{th}$ and a minimum depth $d_{min}$ are defined at the previous step, and the total depth of water $D(m, n) = h(m, n) + \eta(m, n)$ at each cell $(m,n)$ is then compared with $d_{th}$. If the $D(m, n)$ value is greater than or equal to $d_{th}$, LMF(m,n) is set to be 1. Otherwise, the three conditions below are immediately applied:

$$\min[\eta_{m-1,n}, \eta_{m+1,n}, \eta_{m, n-1}, \eta_{m, n+1}] \leq \eta_{m,n}$$

$$\min[D_{m-1,n}, D_{m+1,n}, D_{m, n-1}, D_{m,n+1}] \leq d_{th}$$

$$\max[LMF_{m-1,n}, LMF_{m+1,n}, LMF_{m, n-1}, LMF_{m,n+1}] = 0$$

If at least one of these conditions is satisfied, $LMF(m,n)$ is set to be 0. This means that the cell is dry. If none of the above conditions are satisfied, the total depth $D(m,n)$ of water at each cell $(m,n)$ is compared with $d_{min}$. If the $D(m,n)$ value is smaller than or equal to $d_{min}$, $LMF(m,n)$ is set to 0. Otherwise, $LMF(m,n)$ is set to 1.

### 2.3 Wetting and drying scheme

The wetting and drying process in the hydrodynamic model is very important because it shows the movement of the flow from a wet to dry point in the area and vice versa. It is very common in seas, closed water bodies, and floodplain areas. It affects the accuracy of the simulation results, and the treatment of the wet-and-dry fronts in a complex topography can cause important numerical errors (Brufau, García-Navarro, & Vázquez-Cendón, 2004). In this study, the

### 2.4 Boundary conditions

In the scenario analyses, the observed river water levels from the 11-day data of the 1971 catastrophic flood were
used for the upstream inflow boundary conditions at Son Tay Station. The uniform flow assumption (Pappenberger et al., 2006; Sun, Xue, Wang, Lu, & Liao, 2008) was used to calculate the outflow boundary conditions at the Day Weir, Thuong Cat, and Hung Yen Stations for all scenarios, as shown in Figure 5.

The observed profiles collected in 2013 and 2014 in the Red River area by the Institute of Water Resources Planning, Ministry of Agriculture and Rural Development, Vietnam were used in the model validation. The observed profiles included the river water levels at the Son Tay, Thuong Cat, and Hung Yen Stations. Besides, the uniform flow assumption used in the scenario analyses was introduced at the Thuong Cat and Hung Yen Stations to confirm the assumption. The river water level at Son Tay Station was utilised as the upstream inflow boundary condition. The model validation was conducted without the assistance of Van Coc Lake. The outflow boundary conditions at the Thuong Cat and Hung Yen Stations treated by uniform flow assumption were first validated by comparing the simulated river water levels with the observed levels. Then, the observed river water levels at Long Bien Station were compared with the calculated levels for validation.
RESULTS AND DISCUSSION

3.1 Model validation

The outflow boundary conditions at the Thuong Cat and Hung Yen Stations were treated by the uniform flow assumption that was verified by the observed data. There were two validation periods, from July 29th to August 7th, 2013, and from September 18th to September 29th, 2014. The Nash–Sutcliffe model efficiency (NS) was calculated to evaluate the accuracy, as shown in Figures 6 and 7. The NS coefficient is used as a measure of the agreement between the observed and simulated values and is utilised in many floods scientific researches (Morrissey, McCormack, Naughton, Johnston, & William, 2020; Tanaka, Kiyohara, & Tachikawa, 2020) and strongly recommended by Nash and Sutcliffe (1970). Figure 6 shows good results for 2013 in comparison with the observed river water levels at the outflow boundary stations. The NS of 0.89 and 0.94 for Thuong Cat and Hung Yen stations, respectively, indicate the accuracy of the model. In 2014, the NS also touched 0.78 and 0.95 for the validation results of the outflow boundary stations at the Thuong Cat and Hung Yen Stations, respectively. The NS equation used in this study is as follows:

\[
E = 1 - \frac{\sum_{t=1}^{T} (H_t^o - H_t^m)^2}{\sum_{t=1}^{T} (H_t^o - \overline{H}_o)^2}
\]  

where \(H_o\), \(H_m\), and \(\overline{H}_o\) are the observed, simulated, and mean of the observed water levels (m), respectively.

The model was also validated at Long Bien Station for the two above periods, and the NS coefficients were calculated to assess the accuracy of the model (Figure 8). In 2013 and 2014, the validation results (the NS coefficient was 0.75 and 0.85, respectively) show good agreement between the observed and calculated values. The validation results clearly indicating the accuracy of the proposed 2D hydrodynamic model.

The water balance for the Van Coc Lake and Red river system in the research area is represented explicitly in Figures 9 and 10. The results indicate the accuracy of the model. The water balance equations used in this study are as follows (Güntner, Krol, Araújo, & Bronstert, 2004):

\[
V_t^{VCL} = V_{t-1}^{VCL} + Q_{inVCG} + Q_{inOverF} - Q_{outDW}
\]  

\[
V_t^{RedR} = V_{t-1}^{RedR} + Q_{inST} - Q_{outTC} - Q_{outHY} - Q_{outDW}
\]

FIGURE 6 Calculated and observed river water levels at (a) the Thuong Cat and (b) Hung Yen Stations from July 29th to August 7th, 2013

FIGURE 7 Calculated and observed river water levels at (a) Thuong Cat and (b) Hung Yen Stations from September 18th to September 29th, 2014

FIGURE 8 Calculated and observed river water levels at Long Bien Station (a) from July 29th to August 7th, 2013 and (b) from September 18th to September 29th, 2014
where $V_{\text{VCL}}^t$ and $V_{\text{RedR}}^t$ are the storage volume (m$^3$) in the Van Coc Lake and Red River, respectively, at time $t$; $Q_{\text{inVCG}}$, $Q_{\text{inST}}$, and $Q_{\text{inOverF}}$ are the inflow volume (m$^3$) from the Van Coc Gate, Son Tay Station, and Overflow Point, respectively; $Q_{\text{outDW}}$, $Q_{\text{outTC}}$, and $Q_{\text{outHY}}$ are the outflow volume (m$^3$) from the Day Weir, Thuong Cat Station, and Hung Yen Station, respectively.

### 3.2.1 Water depth in front of the day weir

Figure 11 presents a comparison chart of the time variation of the water depth in front of the Day Weir at the Van Coc Lake side for all simulated scenarios. The water depth in front of the Day Weir depends on the inflow rates at the Van Coc Gate and overflow point. In the second scenario, the water depth had the smallest value with 4.16 m at the peak, which was lower than the maximum weir height of 4.9 m. Although the water depth increased in the third, fourth, fifth, and sixth scenarios, it was still lower than 4.9 m. In contrast, the water depth in the seventh, eighth, ninth, 10th, and 11th scenarios were higher than 4.9 m, with 4.92, 4.95, 4.97, and 4.98 m, respectively. When the width of the overflow point was greater than 1,200 m, the water depth in front of the Day Weir was higher than the maximum weir height of 4.9 m. Moreover, the Day Weir was built in 1937, and overload must be avoided, considering deterioration. Therefore, the maximum water depth in the second to the sixth scenarios is reasonable.

### 3.2.2 Maximum discharge at the day weir

Figure 12 shows the outflow discharge at the Day Weir from the 2nd to 11th scenario. In the second scenario, only the Day Weir and Van Coc Gate were opened, and the discharge touched 1,741 m$^3$/s at the Day Weir. The discharge then increased from 2,157 m$^3$/s in the 3rd scenario to 2,469 m$^3$/s in the 11th scenario, due to the increase in the width of the opening section at the overflow point. However, there were no significant differences in the outflow discharge values of the 9th, 10th, and 11th scenarios with approximately 2,469 m$^3$/s even when the width at the overflow point increased from 1,800 m in the 9th scenario to 2,600 m in the 11th scenario. The results indicate that the maximum outflow discharge at the Day Weir was approximately 2,460 m$^3$/s.

### 3.2.3 Inundated residential areas

Table 2 shows the simulation results of the inundated residential areas when floods reach their peak at the Long Bien Station. The integrated values of the inundated
residential areas, except for Van Coc Lake and their percentages compared to the total residential area, are summarised with five inundation depth levels. Most of the residential areas located outside the protected area of the Red River dike system were inundated by 96% (38.0 km²). The most dangerously inundated residential areas were at Inundation Depth Level 5 from 3.0 m and above for all scenarios. However, there was a decrease in the percentage of...
the inundated residential areas from 46.4% (18.4 km²) in the first scenario to 39.7% (15.7 km²) in the second scenario at Inundation Depth Level 5 before dropping to 37.9% (15.0 km²), 36.2% (13.3 km²), and 35.7% (14.1 km²) in the third, fourth, and fifth scenarios, respectively. Moreover, at Inundation Depth Level 5, the inundated residential areas decreased slightly in the remaining scenarios to 34.7% (13.8 km²) in the 11th scenario. This indicates that the most dangerous flood risk level for the residential areas decreased significantly when the Van Coc Gate, Day Weir, and overflow point were operated. However, owing to the maximum discharge capacity of the Day Weir, the percentage of the inundated residential areas did not drop from the 9th to 11th scenario, with approximately 34.7% (13.8 km²) at Inundation Depth Level 5. Besides, at Inundation Depth Levels 1 and 2, 1.4% (0.6 km²) to 1.9% (0.7 km²) of the residential areas were mainly inundated with a depth of more than 1.0 m in all scenarios. This led to huge damages in residential areas that are highly vulnerable to the impact of floods. Further, there was a slight upward trend in the percentage of the inundated residential areas at Inundation Depth Levels 1 and 2 from 1.4% (0.6 km²) in the first scenario to 1.9% (0.7 km²) in the eighth scenario and from 3.2% (1.3 km²) to 6.7% (2.7 km²), respectively. These results could be explained by the decrease in the percentage at Inundation Depth Level 5, leading to an increase in the remaining Inundation Depth Levels 1, 2, 3, and 4, while the total percentage was almost equal at approximately 96% for all scenarios.

### River water level at Long Bien Station

Figure 13 illustrates the simulated river water level at Long Bien Station for all scenarios. Over the simulation period, the river water level shows a similar trend, with
the biggest value of 12.82 m in the first scenario, which is the non-operating scenario. In the second scenario, with only the Van Coc Gate and Day Weir opened, the river water level decreased sharply to 12.64 m. The river water level declined in the next three scenarios, from 12.57 m in the third scenario to 12.53 m in the fifth scenario, albeit slightly. In the remaining scenarios, although the width of the overflow point rose from 1,000 m in the sixth scenario...
to 2,600 in the 11th scenario, the decrease was insignificant, with approximately 12.51 m, due to the maximum discharge capacity of the Day Weir. The elevations of the right and left dike crests at Long Bien Station shown in Figure 2 are 14.6 m and 13.2 m, respectively, which are higher than the maximum water level. Figure 14 indicates the difference in river water level reduction at Long Bien Station when comparing the first non-operating scenario with the remaining
operating scenarios. In the second scenario, the river water level at Long Bien Station decreased significantly to 0.18 m. When the overflow point was opened with 450 m in the third scenario, the river water level decreased to 0.25 m. The difference in river water level was still high at 0.27 m and 0.29 m in the fourth and fifth scenarios. From the sixth to 11th scenarios, the difference increased slightly from 0.30 m to 0.31 m and did not drop further. Operation of the Van Coc Gate and Day Weir in the second scenario resulted in a sharp drop in the river water level at the Long Bien Station. Especially, when the overflow point was opened together with the Van Coc Gate and Day Weir, the reduction in river water level increased further from 0.25 m to 0.31 m. However, owing to the maximum discharge capacity of the Day Weir, the river water level in the last six scenarios went down slightly, by approximately 0.31 m.

### 3.2.5 Discharge at the Van Coc Gate and overflow point

Figure 15 indicates the discharge at the Van Coc Gate from the 2nd to the 11th scenario; the highest value was 1754 m$^3$/s in the second scenario. The discharge at the gate decreased significantly from 1,528 m$^3$/s in the third scenario to 1,310 m$^3$/s in the 11th scenario due to the width of the overflow point increasing in each scenario. This could be explained by the discharge balance at the Van Coc Gate, overflow point, and Day Weir. When the width of the overflow point increased, the discharge at the overflow point also increased, and consequently, the discharge at the Van Coc Gate decreased. Figure 15 also indicates that in the 9th, 10th, and 11th scenarios, the discharges at the gate were almost equal, at approximately 1,309 m$^3$/s, due to the maximum discharge capacity of the Day Weir.

Figure 16 shows the discharge at the overflow point from the 3rd to 11th scenario; the smallest discharge value was 701 m$^3$/s in the third scenario. The discharge increased from 899 m$^3$/s in the fourth scenario to 1,359 m$^3$/s in the eighth scenario and further increased to the highest discharge of 1752 m$^3$/s in the 11th scenario. In general, the upward trend in discharge is due to the increase in the width of the overflow point.

The discharge at the Van Coc Gate showed a downward trend initially before stabilising at approximately 1,309 m$^3$/s in the 10th and 11th scenarios, while the discharge at the overflow point continued to show an upward trend.

### 3.2.6 The optimal scenario

The optimal scenario was chosen to minimise the flood risk level to residential areas located outside the protected area of the Red River dike system and optimise the inflow discharge at the Van Coc Gate and overflow point, and the outflow discharge at Day Weir to ensure the safety of the operating system.

First, as an important indicator, the maximum water depth in front of the Day Weir must be lower than 4.9 m, which is the maximum weir height, because the Day Weir was built in 1937 and has since deteriorated, so overload must be avoided. The results show that the maximum water depth in front of the Day Weir was lower than 4.9 m in the second to sixth scenarios and is reasonable. Furthermore, the river water level at Long Bien Station did not decrease from the 6th to 11th scenarios owing to the maximum discharge capacity of the Day Weir. However, as shown in Figure 2, the highest water level along the Red River was lower than the crest height at both sides of the dike when the flood reached its peak at Long Bien Station in the 6th scenario and is the same in the 7th and 11th scenarios. Hence, the flood risk levels at Long Bien Station can be considered as the baseline for implementing flood protection measures by the Hanoi Capital and Vietnamese governments. Moreover, the percentages of the inundated residential areas were minimised in the 6th to the 11th scenarios. Besides, the discharge at Van Coc Gate reduced from 1,407 m$^3$/s in the 6th scenario to 1,310 m$^3$/s in the 11th scenario, indicating low overload to the Van Coc Gate. This is necessary because Van Coc Gate has also deteriorated.

Therefore, the sixth scenario was selected as an optimal scenario for the operating procedure of the Van Coc Gate, Day Weir, and overflow point in emergencies to protect the Hanoi Capital. In this case, the bottom elevation of the overflow point is 13.0 m, which is equal to the elevation of the dike toe and surrounding land. The maximum discharges at the Van Coc Gate, overflow point, and Day Weir were 1,407 m$^3$/s, 1224 m$^3$/s, and 2,368 m$^3$/s, respectively, and the maximum river water level at Long Bien Station was 12.52 m, going down by 0.30 m compared to the non-operating scenario.

### 3.2.7 Inundated residential areas when flood reached its peak

Figure 17 shows the inundation depth in the residential areas when the flood reaches its peak at Long Bien Station in the sixth scenario. The research area had a length of over 110 km from Son Tay Station to Hung Yen Station and was classified into five levels to understand the inundated residential areas. When the flood reached its peak, the maximum river water level at Long Bien Station dropped by 0.30 m compared to the non-operating scenario but touched 12.52 m, and 96.0% of the residential areas when the flood reached its peak, the maximum water level at Long Bien Station dropped by 0.30 m compared to the non-operating scenario but touched 12.52 m, and 96.0% of the residential
areas were inundated as shown in Figures 13 and 14, and Table 2.

The inundated residential areas in Zone 1 accounted for 25.1% of the total residential area. The residential areas at Inundation Depth Level 1 from 0.2 m to 0.5 m and at Inundation Depth Level 2 from 0.5 m to 1.0 m were 0.8% and 1.7%, respectively. However, up to 9.5% of the residential areas were underwater at Inundation Depth Level 3 from 1.0 m to 1.5 m.
Depth Level 4 from 2.0 m to 3.0 m. Zone 1 also accounted for large proportions of inundated areas at Inundation Depth Level 5 with 7.6%.

The inundated residential areas in Zone 2 accounted for 15.9% of the total residential area, mainly at Inundation Depth Level 3 from 1.0 m to 2.0 m with 6.3%. The inundated residential areas were small at Inundation Depth Levels 1 and 2, with only 0.5% and 2.3%, respectively, while 3.3% and 3.5% of the residential areas were inundated at Inundation Depth Levels 4 and 5 from 3.0 m and above, respectively.

In Zone 3, the inundated residential areas were 22.4% of the total residential area. The residential areas were mainly underwater at Inundation Depth Levels 3 and 4 from 1.0 to 3.0 m with 15.5%, and the worst affected residential areas accounted for 4.2% at Inundation Depth Level 5.

Zone 4 had the smallest percentage of inundated residential areas, with 12.4% of the total residential area. However, 6.8% of the residential areas were at Inundation Depth Level 5, and consequently, the residential areas were highly vulnerable to the impact of the floods. The inundated residential areas at Inundation Depth Level 4 also accounted for a large proportion with 4.5%, while the total percentage in Inundation Depth Levels 1, 2, and 3 accounted for only 1.2%.

Zone 5 had an inundated residential areas of 20.2% and was most vulnerable to floodwaters with 6.3% and 13.3% at Inundation Depth Levels 4 and 5, respectively. The percentage of the residential areas underwater at

**FIGURE 18** Highest flood flow velocity in the research area, which is divided into five zones when the river water level reached its peak at Long Bien Station in the sixth scenario.
Inundation Depth Levels 1, 2, and 3 accounted for only 0.6%.

The outside-protected areas of the Red River dike system were heavily influenced by floods. More seriously, the simulated results showed that 35.3% of the residential areas were inundated at Inundation Depth Level 5 from 3 m to higher, 31.6% at Inundation Depth Level 4 from 2.0 to 3.0 m, and 20.8% at Inundation Depth Level 3 from 1.0 to 2.0 m.

3.2.8 | Highest velocity

Figure 18 presents the highest flood flow velocities in the research area, which is divided into five zones when the river water level reached its peak at Long Bien Station in the sixth scenario.

In Zone 1, the highest flow velocities were mainly concentrated near the Van Coc Gate, with values ranging from 1.2 m/s to 2.3 m/s. The flood flows with very high velocities were also recorded around the Day Weir from 1.2 m/s to 2.3 m/s. The downstream area of the Day Weir had lower velocities from 0.5 m/s to 0.9 m/s. Besides, the remaining areas in Zone 1 had lower velocities from 0.3 m/s to 0.9 m/s. In Zone 2, the simulation results indicate that the river area has high flow velocities from 0.5 m/s to 2.3 m/s, especially in the curved areas. In Zone 3, the flow velocities fluctuated between 0.9 m/s and 1.2 m/s before the river ran faster and moved into Zone 4 with a velocity from 1.2 m/s to 2.3 m/s. Zone 5 did not have significant differences in flow velocities, and the highest velocities were also concentrated in the river area from 0.5 m/s to 0.9 m/s. Figure 19 shows the velocities near the Hung Yen Station when the river water level reaches its peak at Long Bien Station in the sixth scenario. The highest-velocity areas were primarily concentrated in the riverbed area shown in the inset.

4 | CONCLUSION

This study proposes an optimal operation procedure for the Van Coc Lake to protect Hanoi Capital from the Red River flood disasters, as well as a comprehensive
assessment of the impact of floods on the residential areas located outside the protected area of the Red River dike system.

A 2D hydrodynamic model was constructed, and the unknown outflow boundary conditions were treated by introducing the uniform flow assumption. The research area was divided into grid cells, with a 50-m resolution on the latest digital elevation map that was utilised in numerical calculations involving the 2D hydrodynamic model. A wetting and drying scheme was used to identify the wet and dry areas at each time step. The model was first validated by the observed profiles obtained in the research area. The NS showed the accuracy of the model.

For finding an optimal scenario to operate the Van Coc Lake in emergencies, 11 scenarios were set up by considering the Van Coc Gate, Day Weir, and overflow point with 13.0 m bottom elevation and various widths of the opening section. In the scenario analyses, the observed river water level from the 11-day data of the catastrophic flood in 1971 was used for the upstream inflow boundary conditions. By considering the simulation results of the maximum water depth in front of the Day Weir, the maximum discharges at the Day Weir, Van Coc Gate, overflow point, the inundated residential areas outside the protected area of the Red River dike system, and the river water level at Long Bien Station, the sixth scenario, in which the Van Coc Gate and the Day Weir opened, and the overflow point had 13 m bottom elevation and 1,000 m width for the opening section, was determined as the optimal one. The flood risk level was minimised in the residential areas, and the inflow discharge at the Van Coc Gate, overflow point, and outflow discharge at the Day Weir was optimised to ensure safety for the operating system in the optimal scenario. These are meant for the overall development planning of the Hanoi Capital, especially for critical developments in the Red River basin with adequate protection. However, the residential areas were inundated with 96.0% (38.0 km²) of the total residential areas and vulnerable to flood disasters. This study lays a promising reference for the design of risk-reduction strategies and overall development planning of Hanoi Capital, as well as identifying priorities when conducting related research in the Red River basin system.

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CONFLICT OF INTEREST
The authors declare no conflicts of interest associated with this article.

DATA AVAILABILITY STATEMENT
Some of the datasets during and/or analyzed during the current study available from the corresponding author on reasonable request.

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