Research Article

SPH Simulation on the Coupled Failure of Slope-Building Adjacent to Water Triggered by the Rapid Drawdown of Water Level

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With the development of economy, more and more buildings are constructed on both sides of rivers. The rapid drawdown of water level may induce the change of groundwater seepage in the river bank, thus affecting the stability of buildings on the bank. In this study, the right bank of Qinhuai river with its ancillary building from the Dinghuai Gate to the Qingliang Gate in Nanjing City is analyzed to reveal the failure mechanism and coupled failure mode of slope and building adjacent to water. The soil-water coupled SPH program considering the interaction between soil and structure has been proposed. Then this model is used to study the evolutionary deformation mechanism of slope and building under the rapid drawdown of water level. The results indicate that the potential slip surface of slope and the asymmetrical distribution of plastic zone in the foundation of building become more obvious under the rapid drawdown of water level. Besides, the differential settlement of building induced by the rapid drawdown causes the building tilt. When the sliding surface of the slope passes the building, the differential settlement will become larger. This study is conducive to reveal the coupled failure mechanism of slope and building and also to provide scientific basis for the prevention of such disasters.

1. Introduction

Rapid drawdown of the water level refers to the situation that the river level drops rapidly outside the slope where a building is located [1]. Under this circumstance, water inside the slope cannot drain away immediately and the seepage line in the slope basically remains unchanged. By contrast, the boundary conditions of the seepage would be changed, resulting in deterioration of slope stability. Lots of landslides have occurred under the rapid drawdown of water level. In the United States, about 30% of landslides occurred near the Roosevelt lake, when the water level dropped by 10 ~ 20 meters from 1941 to 1953 [2]. In Spain, a large landslide about $40 \times 10^6 \text{ m}^3$ was reactivated on the left bank of Canelles Reservoir due to drawdown in the summer of 2006 after keeping the high water level for 10 years [3]. In China, the Qianjiangping landslide occurred due to the water level variation of the Three Gorges reservoir on July

14, 2003, which destroyed 4 factories and 129 houses with 24 casualties and 1,200 homeless people [4]. According to the field investigation, the water level of rivers, reservoirs, and canals would drop rapidly after the flood, which would lead the bank slope to landslide, which is adjacent to the water [5]. Therefore, it is urgent to study the failure mechanism of slope under the rapid drawdown of water level.

A large number of methods, including theoretical analyses, model experiments, and numerical simulations, have been introduced to investigate the slope stability under the drawdown of water level from the perspective of pore water pressure inside the water-facing slope and water pressure on the slope caused by the water level of river. Viratjandr [6] used the kinematic limit analysis approach to examine the stability of slope subjected to a rapid or slow drawdown. Based on the theory of limit analysis, Gao [7] discussed the three-dimensional effect of slope under the condition of water level plummeting and found that the plane strain

FIGURE 1: Phreatic level after drawdown: (a) slow drawdown, (b) transient drawdown, and (c) rapid drawdown (based on Berilgen [13]).

analysis was suitable to evaluate the stability of the water-facing slope when the aspect ratio was greater than 10. Meanwhile, some researchers studied the failure mechanism, the failure mode, and the deformation mechanism of water-facing slope by large-scale model tests and centrifuge tests [8–12]. Berilgen [13] classified the drawdown of water level as three cases shown in Figure 1. For the first case, illustrated in Figure 1(a), the soil was totally drained during the process of drawdown, which was called “slow drawdown.” For the generally transient drawdown case shown in Figure 1(b), coupled seepage and consolidation analyses could be carried out concurrently in the stability analysis. For the third case, plotted in Figure 1(c), the soil was considered to be undrained and it was called “rapid drawdown.” “Slow drawdown” and “rapid drawdown” represented the extreme situations in which seepage and consolidation analyses were not done prior to the stability analysis. Berilgen obtained a conclusion similar to the study of Viratjandr [6] with the finite element analysis. The rapid drawdown with the minimum safety coefficient and slow drawdown with the maximum safety coefficient were two extreme situations that affect the slope stability. As the “rapid drawdown” was the most dangerous among three cases, many researchers studied the stability of the slope adjacent to water under rapid drawdown through the simplified model of Figure 1(c) [1, 3, 13–15]. Besides, considering the different infiltration rates, Desai [16] used a numerical approach to locate the internal infiltration line and estimated the global factor of safety with the limit analysis. Desai’s analysis could be considered as a representative of semirapid drawdown and the results showed that the FOS was 40% higher than that of rapid drawdown in some cases. Griffiths et al. [14, 15, 17] analyzed the adverse effects of various factors on the slope including the decreasing amount and speed of water level and the soil permeability coefficient.

As a conclusion, theoretical methods based on the rigid limit equilibrium equation can be used to study the change of slope safety coefficient when the water level drops rapidly, while it cannot take the deformation of the slope into account. Besides, experimental methods can reproduce the instability process of water-facing slope under the sudden drawdown of water level, but the cost is too high and it is inconvenient to perform parameter analysis based on experimental methods. Numerical analyses can take into account the influence of various factors on the stability of water-facing slope, but there

was a large deformation after the slope failure according to the experimental phenomenon. Traditional numerical methods, for instance, FEM and FDM, cannot deal with the large deformation due to the constraints of grids. There is no grids constraint for the method of smoothed particle hydrodynamics (SPH), which can simulate the large deformation and failure process of slopes [18–20]. However, the coupled failure process of slope and building under the action of water level drawdown is rarely studied. In addition, the stability of building on the top of slope is more important than the stability of the slope itself, so it is significant to study the coupled failure process of slope and building.

Therefore, the soil-water coupling and the soil-structure interaction have been introduced into the SPH method in this study. For the soil-water coupling, the fluid phase (pore water) and the solid phase (soil or structure) are simulated on different layers which have the interaction force. With regard to the soil-structure interaction, the coupling force between soil and structure is realized by the reaction force that is the same in size and opposite in direction. Then, the cases of a large-scale model test and a retaining wall are adopted to validate the interaction algorithms. Finally, the right bank of Qinhuai river and its ancillary building from the Dinghuai Gate to the Qingliang Gate in Nanjing City are analyzed to reveal the coupled failure mechanism and failure mode of slope and building adjacent to the river.

2. Water-Soil Coupled SPH Numerical Framework

2.1. Basic Concepts. The basic idea of SPH is to discretize the spatially continuous entity into series of particles that can carry mass, speed, stress, and deformation information. There is no link between these particles. By providing an accurate numerical solution for hydrodynamic equations and tracking the movement of each particle, this method can describe the mechanical behavior of an entire system [21–25]. The meshless characteristics of SPH make it easier to deal with the large deformation problem, sequentially eliminating the mesh distortion problems compared with the traditional Lagrange method [26–30].

The cores of this method are the field function approximation and the particle approximation, respectively [31]. The field function approximation refers to the integral expression

FIGURE 2: Particle approximation in the SPH method.

of the macroscopic variables (such as density, pressure, and acceleration) and the particle approximate means that the weighted summation of motion information is calculated from particles within the influence radius (Figure 2).

The particle approximate function and its derivative can be expressed as follows:

$$\langle f(x) \rangle = \sum_{j=1}^N m_j \frac{f_j(x')}{\rho_j} W(x - x', h) \quad (1)$$

$$\frac{\partial \langle f(x) \rangle}{\partial x_i} = \sum_{j=1}^N \frac{m_j}{\rho_j} f_j(x') \frac{\partial W(x - x', h)}{\partial x_i} \quad (2)$$

where $f(x)$ is the field function; x is the position vector of a computing particle; x' is the position vector of a supporting particle; h is the smoothing length; N is the number of supporting particles inside the influence domain; m is the mass; ρ is the density; and W is the smooth kernel function.

2.2. Mixture Theory for the Two-Phase Coupled Soil. For the complex pore geomaterial, the pore water pressure generated by the flow of pore water has a great influence on the physical and mechanical properties of soil skeleton. The Biot consolidation theory is introduced into the SPH numerical framework due to its wide application. In the proposed SPH method, the fluid phase (pore water) and the solid phase (soil and structure) are simulated on different layers (Figure 3). Accordingly, physical quantities can be distributed according to the following formulas:

$$\begin{aligned} \bar{\rho}^w &= n\rho^w \\ \sigma_{mn}^w &= n p^w \delta_{mn} \end{aligned} \quad (3)$$

(pore water)

$$\begin{aligned} \bar{\rho}^s &= (1 - n) \rho^s \\ \sigma_{mn}^s &= (1 - n) p^w \delta_{mn} + \sigma'_{mn} \end{aligned} \quad (4)$$

(soil or structure)

$$\begin{aligned} \rho &= (1 - n) \rho^s + n\rho^w \\ \sigma &= p^w \delta_{mn} + \sigma'_{mn} \end{aligned} \quad (5)$$

(total)

where ρ^w is the density of water; ρ^s the density of soil or structure; p^w is the pore water pressure; n is the porosity ($n = 0$, when solid particles are structural particles); σ' is the effective stress; and δ_{ij} is the Kronecker function.

The controlling equations of SPH are the continuity equation, the momentum equation, and the energy equation, respectively. This study assumes that the temperature of liquid phase in the geomaterial remains constant, so the energy equation is not considered. The controlling equations of SPH can be obtained by using (1) and (2) for the liquid phase and the solid phase.

The equation of continuity is

$$\frac{d\bar{\rho}_i}{dt} = \sum_{j=1}^N m_j (v_{im} - v_{jm}) \frac{\partial W_{ij}}{\partial x_{im}} \quad (6)$$

The momentum equation of pore water is

$$\begin{aligned} \frac{dv_{im}^w}{dt} &= \sum_{j=1, j \in \text{water}}^N m_j \left(\frac{\sigma_{imn}^w}{(\bar{\rho}_i^w)^2} + \frac{\sigma_{jmn}^w}{(\bar{\rho}_j^w)^2} \right) \frac{\partial W_{ij}}{\partial x_{in}} + F_i \\ &\quad + \frac{R_{im}^{sw}}{\bar{\rho}_i^w} \end{aligned} \quad (7)$$

The momentum equation of soil or structure is

$$\begin{aligned} \frac{dv_{im}^s}{dt} &= \sum_{j=1, j \in \text{soil}}^N m_j \left(\frac{\sigma_{imn}^s}{\bar{\rho}_i^{s2}} + \frac{\sigma_{jmn}^s}{\bar{\rho}_j^{s2}} - \delta_{mn} \Pi_{ij} \right. \\ &\quad \left. + f r_{ijmn} \right) \frac{\partial W_{ij}}{\partial x_{in}} + F_i + \frac{R_{im}^{ws}}{\bar{\rho}_i^s} \end{aligned} \quad (8)$$

where v is the velocity vector; m and n indicate the directions of coordination; i indicates the computing particle; j indicates the supporting particle; σ^w is the pore water stress vector; σ^s is the soil or structure stress vector; F_i is the external force. R^{sw} and R^{ws} are the interaction force between the pore water and the soil or the structure. Π_{ij} is the artificial viscosity that can correct numerical oscillation during the calculation [32].

Water is an incompressible fluid that is difficult to model. Therefore, artificial compressibility is introduced and the pore water is assumed as a kind of compressible fluid that can obtain large pressure changes through small density changes. In addition, the coefficient of viscosity is also considered to acquire the revised formula of pore water pressure (details in [31]).

FIGURE 3: Soil-water coupling in the SPH method.

The interaction force between pore water and soil or structure is calculated as [33]

$$R_{im}^{sw} = \sum_{j=1 \in \text{soil}}^N m_j \frac{R_{ijm}}{\rho_j} W_{ij}, \quad (9)$$

$$R_{ijm} = n^2 \frac{\rho^w g}{k_j} (v_{jm}^s - v_{im}^w)$$

$$R_{im}^{ws} = \sum_{j=1 \in \text{water}}^N m_j \frac{R_{ijm}}{\rho_j} W_{ij}, \quad (10)$$

$$R_{ijm} = n^2 \frac{\rho^w g}{k_i} (v_{jm}^w - v_{im}^s)$$

where k is the permeability coefficient and g is the gravitational acceleration.

2.3. Constitutive Model and Interaction of Soil and Structure

2.3.1. Soil Constitutive Modeling. In this paper, the elastoplastic constitutive model is used to describe the stress-strain relationship of soil. In the framework of plasticity deformation theory, the total strain tensor of an elastoplastic material $\dot{\epsilon}$ is decomposed into two parts: an elastic strain rate tensor $\dot{\epsilon}_e^{\alpha\beta}$ and plastic strain rate tensor $\dot{\epsilon}_p^{\alpha\beta}$.

$$\dot{\epsilon} = \dot{\epsilon}_e^{\alpha\beta} + \dot{\epsilon}_p^{\alpha\beta} \quad (11)$$

The elastic strain rate tensor $\dot{\epsilon}_e^{\alpha\beta}$ is given by a generalized Hooke law and the plastic strain rate tensor is calculated by plastic flow rule.

$$\dot{\epsilon}_e^{\alpha\beta} = \frac{\dot{s}^{\alpha\beta}}{2G} + \frac{1-2\nu}{E} \dot{\sigma}^m \delta^{\alpha\beta} \quad (12)$$

$$\dot{\epsilon}_p^{\alpha\beta} = \dot{\lambda} \frac{\partial g}{\partial \sigma^{\alpha\beta}} \quad (13)$$

where $\dot{s}^{\alpha\beta}$ is the deviatoric shear stress tensor; G is the shear modulus; E is the elastic Young's modulus; ν is Poisson's ratio; $\dot{\sigma}^m$ is the mean stress; and $\dot{\lambda}$ is the rate form of plastic multiplier (when the model is in an elastic state, $\dot{\lambda} = 0$; when the model produces plastic deformation, $\dot{\lambda} > 0$). In this study, the Drucker-Prager model (the DP model for short) with associated flow rule is applied to model the mechanical behavior of soil, which is that the plastic potential function g agrees with the yield function f :

$$g = f(I_1, J_2) = \sqrt{J_2} + \alpha_\phi I_1 - k_c \quad (14)$$

where I_1 and J_2 are, respectively, the first and second invariants of stress tensors:

$$I_1 = \frac{\sigma^{xx} + \sigma^{yy} + \sigma^{zz}}{3} \quad (15)$$

$$J_2 = \frac{1}{2} s^{\alpha\beta} s^{\alpha\beta} \quad (16)$$

where α_ϕ and k_c are Drucker-Prager constants, which are calculated from the Coulomb material constants c (cohesion) and ϕ (internal friction). For the plane-strain problem, the Drucker-Prager's constants are computed by

$$\alpha_\phi = \frac{\tan \phi}{\sqrt{9 + 12 \tan^2 \phi}} \quad (17)$$

$$k_c = \frac{3c}{\sqrt{9 + 12 \tan^2 \phi}} \quad (18)$$

Substituting (12) and (13) into (11) and adopting the Jaumann stress rate for large deformation treatment, the

FIGURE 4: Sketch map of soil and structure interaction.

stress-strain relationship for the current soil model at particle i can be derived to be

$$\frac{d\sigma_i^{\alpha\beta}}{dt} = \sigma_i^{\alpha\gamma} \dot{\omega}_i^{\beta\gamma} + \sigma_i^{\gamma\beta} \dot{\omega}_i^{\alpha\gamma} + 2G_i \dot{e}_i^{\alpha\beta} + K_i \dot{\epsilon}_i^{\gamma\gamma} \delta_i^{\alpha\beta} - \dot{\lambda}_i \left[3\alpha_\phi K_i \delta_i^{\alpha\beta} + \left(\frac{G}{\sqrt{J_2}} \right)_i s_i^{\alpha\beta} \right] \quad (19)$$

$$\dot{\lambda}_i = \frac{3\alpha_\phi K_i \dot{\epsilon}_i^{\gamma\gamma} + (G/\sqrt{J_2})_i s_i^{\alpha\beta} \dot{\epsilon}_i^{\alpha\beta}}{9\alpha_\phi^2 K_i + G_i} \quad (20)$$

where K is elastic bulk modulus; $\dot{\epsilon}^{\alpha\beta}$ and $\dot{\omega}^{\alpha\beta}$ are the strain rate and spin rate tensors defined by

$$\dot{\epsilon}_i^{\alpha\beta} = \frac{1}{2} \left(\frac{\partial v^\alpha}{\partial x^\beta} + \frac{\partial v^\beta}{\partial x^\alpha} \right)_i = \frac{1}{2} \left[\sum_{j=1}^N \frac{m_j}{\rho_j} (v_j^\alpha - v_i^\alpha) \frac{\partial W_{ij}}{\partial x_i^\beta} + \sum_{j=1}^N \frac{m_j}{\rho_j} (v_j^\beta - v_i^\beta) \frac{\partial W_{ij}}{\partial x_i^\alpha} \right] \quad (21)$$

$$\dot{\omega}_i^{\alpha\beta} = \frac{1}{2} \left(\frac{\partial v^\alpha}{\partial x^\beta} - \frac{\partial v^\beta}{\partial x^\alpha} \right)_i = \frac{1}{2} \left[\sum_{j=1}^N \frac{m_j}{\rho_j} (v_j^\alpha - v_i^\alpha) \frac{\partial W_{ij}}{\partial x_i^\beta} - \sum_{j=1}^N \frac{m_j}{\rho_j} (v_j^\beta - v_i^\beta) \frac{\partial W_{ij}}{\partial x_i^\alpha} \right] \quad (22)$$

The above soil constitutive model requires five soil parameters: cohesion (c), friction angle (ϕ), elastic Young's modulus (E), Poisson's ratio (ν), and soil density (ρ). The detailed information can be found in literature [31].

2.3.2. Constitutive Model for Structure. The mechanical behavior of the structure is described by an elastic model, where the stress-strain relationship can be represented as

$$\dot{\sigma}_{mn}^e = D_{mnkl}^e \dot{\epsilon}_{kl}^e \quad (23)$$

where $\dot{\sigma}_{mn}^e$ is elastic stress tensor, D_{mnkl}^e is a fourth-order elastic coefficient tensor, and $\dot{\epsilon}_{kl}^e$ is the elastic strain tensor.

Given Young's modulus E and Poisson's ratio ν , the elastic coefficient matrix is

$$[E] = \begin{bmatrix} \lambda + 2\nu & \lambda & \lambda & 0 & 0 & 0 \\ \lambda & \lambda + 2\nu & \lambda & 0 & 0 & 0 \\ \lambda & \lambda & \lambda + 2\nu & 0 & 0 & 0 \\ 0 & 0 & 0 & \nu & 0 & 0 \\ 0 & 0 & 0 & 0 & \nu & 0 \\ 0 & 0 & 0 & 0 & 0 & \nu \end{bmatrix} \quad (24)$$

2.3.3. Interaction between Soil and Structure. In the proposed SPH model, soil particles and structural particles are both solid particles and their controlling equations are the continuity equation and momentum equation for the solid phase. To distinguish soil and structure and to simulate the interaction between them, different subtypes are set in the solid phase to represent the soil particles and the structural particles, respectively, as shown in Figure 4.

To facilitate the analysis, it can be assumed that the interaction between soil and structure passes through the reaction force that is the same in size and opposite in direction. In the proposed SPH model, the contact stress between soil and structure has an impact on each other's acceleration. Thus, under the soil-structure interaction, the momentum equation of a structure particle can be expressed as

$$\frac{dv_i^\alpha}{dt} = \sum_{\substack{j=1, \\ i \in \text{structure}}}^N m_j \left(\frac{\sigma_i^{\alpha\beta}}{\rho_i^2} + \frac{\sigma_j^{\alpha\beta}}{\rho_j^2} - \delta^{\alpha\beta} \Pi_{ij} \right) \frac{\partial W_{ij}}{\partial x_i^\beta} + \frac{F_i}{\rho_i} \quad (25)$$

Similarly, the momentum equation of a soil particle can be expressed as

$$\frac{dv_i^\alpha}{dt} = \sum_{\substack{j=1, i \in \text{soil} \\ j \in \text{structure}}}^N m_j \left(\frac{\sigma_i^{\alpha\beta}}{\bar{\rho}_i^2} + \frac{\sigma_j^{\alpha\beta}}{\bar{\rho}_j^2} - \delta^{\alpha\beta} \Pi_{ij} \right) \frac{\partial W_{ij}}{\partial x_i^\beta} + \frac{F_i}{\rho_i} \quad (26)$$

where α and β indicate the direction of coordination.

It can be observed that the accelerations of soil and structure have same magnitude and opposite direction under the reaction force. To some extent, the condition of soil-structure deformation coordination is satisfied. Therefore, in theory, the soil-structure interaction treatment method proposed in the present study can be employed to solve the problem of soil-structure interaction in the practical engineering.

In present paper, the linked-list searching method is adopted as the particle searching method. The time integration uses the second-order Runge-Kutta integral method. Besides, the boundary treatment method of Takeda [34] and Morris [35] is adopted to calculate the resisting effect of boundary particles on free particles. This method assumes that the influence of boundary particles on the velocity of free particles is related to the velocity difference and relative distance between them. More details can be found in literature of Zhang [33].

3. Model Validation

3.1. Validation of the Soil-Water Coupling Algorithm. In order to validate the feasibility of the proposed SPH model on the calculation of slope failure process caused by rapid drawdown of water level, the same SPH model as the test in Jia [8] is established, and the simulated results are compared with the test results. According to the experiment, the cohesion of silt is 1 kPa, the internal friction angle is 30° , and the permeability coefficient is 5.3×10^{-6} m/s. A total of 13,023 particles are used in this simulating case, consisting of 936 boundary particles, 5,911 soil particles, and 6,176 water particles. The soil constitutive model adopts the DP model and the incremental time step is 5.0×10^{-5} s.

Figure 5 is the comparison of displacement magnitude between experiment and simulation at different states. As shown in Figure 5(b), the water level decreases from 5.6 m to 3 m. Blocks 1, 2, and 3 are sliders that are observed from the model test, and the frame line presents the side slope shape before collapse. Besides, the color chart of total displacement obtained by SPH simulation after sliding shows that the sliding body (red area) is similar to the sum of sliding blocks 1, 2, and 3 (as shown in Figure 5(b)). Meanwhile, the total displacement of slope crest is 0.48m, which is close to 0.5m measured by the test. This method only considers the situation of sudden drop of water level, assuming that there is no significant drainage outward from the slope, and does not consider the long-term change of seepage field with time. The morphology and displacement of the slope after sliding are

FIGURE 5: Displacement magnitude of the experiment and the simulation (unit: m).

FIGURE 6: Model of retaining wall (unit: m).

similar to those obtained from the model test, so it proves that the proposed SPH model can be used to simulate the failure process of soil slope under the rapid drawdown of water level.

3.2. Validation of the Soil-Structure Interaction Algorithm. The retaining wall is a classical problem for the soil-structure interaction in soil mechanics, so it can verify the rationality of soil-structure interaction proposed in the present paper. As shown in Figure 6, the total length of the plate pile in the calculation example is 5 m and the plate pile is embedded in the soil for 1 m. In this model, sand is adopted as the backfill. Its density and internal friction angle are considered to be 1460 kg/m^3 and 17° , respectively. In the SPH simulation, the stress-strain relationship of soil is expressed by the DP model, while the pipe (structure) is treated as an elastic solid material.

According to the Rankine earth pressure theory, when the soil is in the limit failure state of active earth pressure, the horizontal direction will be the maximal principal stress

TABLE 1: Parameter of soil and structure.

Type	Density (kg/m ³)	Porosity	Elasticity modulus (MPa)	Poisson ratio	Permeability (m/s)	Particle spacing (m)	c (kPa)	Φ (°)
Soil (silt)	1850	0.885	5	0.3	7.68×10^{-6}	0.2	25	17
Structure	1000	1	1000	0.2	1×10^{-10}	0.2	—	—

FIGURE 7: Plastic zone of soil.

plane, and the failure surface angle is $\theta_f = 45^\circ + \varphi/2 = 53.5^\circ$. A plastic zone of failure state can be observed from Figure 7(b). It can be seen that the angle between the plastic zone and horizontal line is 53° which is almost the same as the Rankine earth pressure theory.

Curves of horizontal earth pressure with depth are shown in Figure 8(a). The red curve is simulated by SPH and the black line is calculated by Rankine earth pressure theory. The plate pile wall has both rotary and translational movements, resulting in the nonlinear distribution of earth pressure along the depth. When the soil behind the wall (less than 4.2m) has large deformation due to the excessive active earth pressure, the earth pressure obtained by SPH simulation is less than that of Rankine active earth pressure, which is because the SPH algorithm takes into account the friction between the wall and soil. As shown in Figure 8(b), the variation of horizontal earth pressure with depth simulated by SPH is similar to the result of the other research [36]. At the same time, the distribution rule of upper earth pressure (less than 1m) along the depth simulated by SPH is more accurate than the FEM result [36].

Above all, the angle between the failure surface and horizontal plane simulated by SPH is almost the same as the Rankine active earth pressure theory. The variation of horizontal earth pressure with depth simulated by SPH is also similar to the result of the other methods [36]. So it has been proven that SPH can simulate the interaction force between soil and structure.

4. Simulation of Slope-Building-Coupled Failure under the Rapid Drawdown of Water Level

4.1. Simulating Cases. According to the field investigation, the cross-section diagram of a building on the right bank of Qinhuai river from the Dinghuai Gate to the Qingliang Gate in Nanjing city is selected and discretized into a particle model as shown in Figure 9. The distance (d) between the building and the slope crest is 8m under practical conditions, and the dashed blue line refers to the groundwater level. According to the geological report, the soil parameters of this slope are shown in Table 1, where the density of structure particles is equal to the mass of the whole building divided by the total volume. In order to simulate the large deformation of soil, the constitutive behavior of soil is described by the DP model, while the structure is described by the elastic model. In the calculation, there are 27,358 particles, consisting of 1,476 boundary particles, 11,859 water particles, 10,643 soil particles, and 3,380 structure particles. Besides, the incremental time step is 1.2×10^{-4} s and total steps are 8×10^4 .

Based on previous simplified model for rapid drawdown, this paper analyzes the coupled failure of slope and building adjacent to river triggered by the rapid drawdown. The model of the water-soil coupling problem is shown in Figure 9 and the rapid drawdown problems are evaluated as the following three cases: (i) the water level is 11.6 m, (ii) the water level is 8.3 m, and (iii) the water level is 5 m. For all three cases, the groundwater level inside the slope (the dashed blue line) is considered to be constant. The distance d ($= 0, 2, 4, 6, 8, 10, 12$, and 14 m) between the building and the slope crest is an important parameter to analyze the differential settlement characteristics of building under three water levels.

4.2. Result Discussion. According to the plastic zone shown in Figure 10, it can be found that the failure mode of soil under the rapid drawdown of water level is similar to that of slope foundation obtained by literatures [37, 38]. The changes of water level will cause the plastic zone changes of slope and building. As the water level of a river decreases, the potential sliding surface of the bank slope becomes more obvious and the maximum shear strain increment becomes larger. At the same time, the existence of slope leads the maximum shear strain increment at the bottom of left pile near the slope to be greater than that of right pile, and the gap between them increases with the decrease of water level.

The displacement distribution is shown in Figure 11. It can be seen that when the water level decreases, the sliding displacement of slope will increase, indicating that the more the water level decreases, the worse the stability becomes.

(a) Distribution rules between SPH and Rankine earth pressure theory

(b) Distribution rules between SPH and FEM [36]

FIGURE 8: Horizontal earth pressure distributed along the depth.

FIGURE 9: Sketch map of model (unit: m).

The displacement on both sides of the building foundation shows that the displacement on the side adjacent to the slope is greater than that on the other side. As the water level decreases, the displacement difference between the two shows an increasing trend.

The displacement at the crest of slope under different water levels is also analyzed, and the comparison shows that the lower the water level is, the greater the displacement at the crest of slope is. The horizontal displacement at the slope crest with the water level of 5m increases by 133% (Figure 12(a)) and the vertical displacement increases by 123% (Figure 12(b)) compared with the water level of 11.6m. This is because when the water level drops, the water pressure on the slope decreases, and, as a result, the seepage boundary conditions change as well, which is not conducive to the slope stability. More importantly, the groundwater level inside the slope drops at a significantly lower rate than the water level. The water head difference between inside and outside of the

slope is formed, which generates additional seepage pointing to the outside of slope. Under the action of these mechanisms, the slope is prone to be instable. The greater the water level drops, the greater the possibility of slope instability is. Slope displacements do not reach steady state (Figure 12), because water levels drop caused the change of seepage boundary conditions. In this case, only the slope seepage field achieves stability again; the deformation of slope will be stable. However, it will take a very long time to calculate the seepage stability and the stability of the slope is improving during this process [13]. So the most dangerous condition is considered, which does not calculate the long-term changes of seepage.

When the slope is instable, under the action of sliding force, the process in which the sliding body on the sliding surface slides downward is equivalent to the process of soil inside the sliding surface releasing the earth pressure. Under this mechanism, for the building near the crest of the slope, the earth pressure near the crest of the slope is disturbed to a greater extent, which will cause a larger settlement of the building on this side. However, the earth pressure far from the side of the slope is disturbed to a less extent, which will cause a smaller settlement of the building. Thus, it may lead to the differential settlement of the building, which may cause the building to tilt and produce additional stress or increase the additional stress of the superstructure. Figure 13(a) reveals that as the water level goes down, the sliding deformation of slope goes larger, the disturbance of earth pressure goes more obvious, the differential settlement difference of the building goes greater, and the building safety goes down. In addition, it can be seen from the pile lateral displacement curve in Figure 13(b) that, under the condition of different water levels, the deforming characteristics of the piles are generally the same, but the displacement values are slightly different. When the water level decreases, the lateral displacement of the pile

FIGURE 10: Plastic zone at the end of SPH simulations.

FIGURE 11: Distribution of displacement (unit: m).

FIGURE 12: Time-history curves of displacement at slope crest.

FIGURE 13: Time-history curves of displacement of structure.

will increase. Besides, with the increase of soil depth, the disturbance of soil by the slope sliding decreases and the constraint of soil on the pile increases. It makes the lateral displacement of the pile decrease with the increase of soil depth. The lateral displacement of the pile far away from slope is smaller than that of the pile adjacent to slope and the displacement difference between them increases with the drop of water level.

According to the differential settlement curve under different working conditions in Figure 14, it can be found

that the differential settlement of the building decreases with the increase of distance (d) between the building and the slope crest at the same water level. When d is less than or equal to 4 m, the differential settlement of the building is strongly correlated with d . When d is more than 4 m, there is a significantly weak correlation between the differential settlement and d . According to the plastic zone diagram under different d , when d is less than or equal to 4 m, the slope sliding surface passes through the building, and when d is greater than 4 m, the building is outside the slope sliding

FIGURE 14: Differential settlement curves of the building under different condition.

surface. It can be seen that when the slope sliding surface passes through the building, its differential settlement will become large.

5. Conclusion

Aiming at the problem of the slope-building-coupled failure under the rapid drawdown of water level, this paper took an existing test to verify the feasibility of the proposed SPH model in simulating the soil-water-coupled problem under the rapid drawdown of water level. Then, an example of retaining wall was given to verify the rationality of soil-structure interaction. Besides, the right bank of the Qinhuai river and its ancillary building from the Dinghuai Gate to the Qingliang Gate were analyzed. In the simulation, three different conditions of water level were considered to study the slope-building-coupled failure mechanism and failure mode adjacent to water. Meanwhile, the evolutionary mechanism of slope and building deformation under the rapid drawdown was also revealed. The conclusions could be drawn as follows:

(1) With the rapid drawdown of water level, the potential slip surface of slope and the asymmetrical distribution of plastic zone in the foundation of building become more obvious.

(2) The differential settlement of a building under the rapid drawdown causes the building to tilt and impacts the building utility. In addition, the larger water level drops, the larger differential settlement of the building becomes.

(3) When the sliding surface of slope passes a building, its differential settlement will become large and will be significantly affected by the distance from building to slope crest.

Data Availability

(1) The [generated] data used to support the findings of this study are included within the article. (2) The [analyzed] data

used to support the findings of this study are included within the article. (3) The [nonanalyzed] data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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