Research Article
Nonlinear Simulation and Vulnerability Analysis of Masonry Structures Impacted by Flash Floods

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The vulnerability of buildings impacted by flash floods is often assessed by empirical approaches. However, the damage-developing mechanisms of buildings remain unclear. This study presented a nonlinear numerical analysis for the interaction between flash floods and masonry structures. A new Structured ALE (S-ALE) solver in an explicit finite element platform LS-DYNA was applied. Thirty different scenarios, including two different FE models, three different water depths, and five different flow velocities, were studied. Nonlinear structural responses and failure mechanisms of masonry structures were analyzed and compared. Results showed that due to the consideration of wall damage, the time evolution of the impact force on the entire building was distributed in a multipeak pattern. Under the impact of flash floods, the building was in a complex bending-shear state, and the overturning moment was the principal reason for the building damage. The critical role played by the structural measure was reaffirmed. Moreover, the physical vulnerability was quantified through a macroscopic damage index, the lateral drift ratio of the ground floor. It can be concluded that the physical vulnerability depends on both the local structural strength (local view) and the structural resistance hierarchies (global view).

1. Introduction
Flash floods are regarded as one of the most catastrophic natural hazards and affect many regions, especially mountainous regions [1, 2]. It has caused incalculable consequences throughout history, such as mass casualties, great damage to structures and infrastructures, and heavy economic losses [3]. Because of the changing global climate, unplanned rapid urbanization, poor watershed management, and increased social-economic activities, flash floods are anticipated to erupt more severely and irregularly in the future [4]. Flood risk management is the most effective and positive approach for comprehensive flood control and disaster reduction [5]. As shown in Figure 1, the assessment of flood risks includes evaluating both the hazardous process (hazard assessment) and the consequences (vulnerability assessment).

The multidimensional nature of the vulnerability is responsible for the diversity of definitions available in the literature. From an engineering and natural science perspective, physical or structural vulnerability is a fundamental prerequisite [6, 7]. “Vulnerability” used in this paper refers to the physical vulnerability of masonry structures impacted by flash floods. Besides, masonry structures are widespread in mountain villages and towns in China. It is characterized by clay brick masonry structures combining some structural measures with one or two floors. Because of the lack of professional structural knowledge and financial support, most buildings do not meet specification requirements in the design and construction process, leading to a limited capacity to cope with flood disasters. In this context, it is urgent to investigate the vulnerability of masonry structures. In terms of assessment methods, the statistical and indicator-based methods are more accepted because of the advantages of a straightforward process and clarified vulnerability image over space [8–11]. However, a significant drawback is that the reliability depends on the quality and the quantity of the available empirical data [12]. Considering
the constantly changing built environment and human activities, these empirical methods have some “historical limitations.” An explicit physical mechanism can enhance the understanding of disaster characteristics and the scientificity of the vulnerability assessment, rendering the results more convincing. Physically based methods (scenario analysis), which analyze the structural response in the specific hazard scenarios, have gained increasing popularity [13]. Specifically, the vulnerability of buildings has been analyzed through a variety of physically based methods, including yielding line analysis [14, 15], 2D structural analysis [16], probabilistic and modular analysis [5, 17], and experimental analysis [1]. Based on analytical representations of the failure mechanism of individual components, Nadal et al. [18] investigated the expected flood damage to individual buildings. De Risi et al. [19] proposed an integrated modular probabilistic methodology for calculating the flooding risk of spatially distributed structures. Sturm et al. [20, 21] reproduced a set of fluviatile flood scenarios by a physical model to measure the impact forces and to identify the influencing factors.

The numerical simulation solves complex interaction problems involving nonlinear behaviors in dynamic conditions [22] and provides an effective way to explore the structural response, making it possible to obtain data required to analyze the vulnerability [23]. Based on the computational formula and experimental results about the flood impact, Xiao and Li [24] carried out a static analysis on the rural building numerically. Jalayer et al. [25] calculated flood diffusion by solving the two-dimensional differential equations and simulated the damage to local buildings based on the maximum hydraulic action. Liu et al. [26] revealed the physical response of structures exposed to construction solid landslides.

Until now, most previous researches have failed to explain the structure destruction process. The nonlinear structural response of damaged buildings and the response after the fluid influx have not been described precisely. In addition, the fluid impact model and the structural model were always calculated separately. The structural response was analyzed by applying a load condition obtained from the independent fluid analysis. The load conditions were assumed to be produced by specific water depth and flow velocity utilizing an equivalent pseudo-static formulation, neglecting the dynamic effects produced by the extensive momentum exchange process in the early stage of the impact [27]. Recently, some attempts in the field of geological hazards may provide some references. Cheng et al. [28] studied the fluid-solid interaction dynamic response of masonry structures under debris flows. Feng et al. [29] established a flow-structure coupled numerical model by Smoothed Particle Hydrodynamics to predict the propagation of flow slides and the destruction process of buildings. Luo et al. [12, 30] simulated the interaction between buildings and five mass flows to analyze structural responses and failure patterns of reinforced concrete-framed buildings. However, insufficient attention has been paid to the interaction between flash floods and buildings so far. Although Totschnig and Fuchs [31] suggested that it was not strictly necessary for the vulnerability functions to be classified based on the different sediment-laden torrent process types, flash floods are significantly different from debris flows or landslides. The rheological properties of flash floods are more similar to that of clear water, e.g., negligible yield stress and low viscosity, and the fluid-structure interaction mechanisms are highly dependent on the flow behavior around the structure [32].

While a considerable effort has been devoted to developing the analysis of the physical vulnerability exposed to floods, the real physics behind the phenomena remains unclear, and the vulnerability assessment of masonry structures subject to flash floods is still in its initial stage [13, 18]. To bridge the knowledge gap, this study presented a series of nonlinear numerical analyses to investigate the mechanics of the interaction and the physical vulnerability of buildings. The nonlinear explicit finite element (FE) software LS-DYNA, of which contact algorithms and fluid-structure interaction module are especially suitable for dealing with the complex interaction problem, as well as a new Structured ALE (S-ALE) solve, was applied to establish the model. Thirty scenarios were simulated with two kinds of FE models (with and without structural measures), three kinds of water depths (0.8 m, 1.6 m, and 3.2 m), and five

![Figure 1: Flood risk assessment.](image-url)
kinds of flow velocities (4 m/s, 8 m/s, 12 m/s, 16 m/s, and 20 m/s) to analyze nonlinear structural responses and failure mechanisms of masonry structures. Then, the physical vulnerability was quantified through a macroscopic damage index, the lateral drift ratio of the ground floor. This study contributes to a better understanding of the physical vulnerability of buildings impacted by flash floods and is valuable both for researchers and engineers in the field of flood risk management.

2. Engineering Background and Numerical Models

The study’s engineering background is the catastrophic flash flood disaster that occurred in Wenchuan (Sichuan province, China) on 20 August 2019. The flash flood caused much damage to structures, property, and had casualties. Figure 2 shows photographs of the damaged buildings taken at the disaster sites.

The global disaster scenario is shown in Figure 3(a). For simplicity, the FE model without structural measures is expressed as $s = 0$, while the FE model with structural measures is expressed as $s = 1$. In the specified disaster scenario, the flash flood was initiated by constant water depth and as well as constant flow velocity.

2.1. Masonry Structure FE Model. The masonry structure FE model was established based on the field survey and was simplified to consist of masonry walls, floor slabs, and structural measures (ring beams and structural columns). The wall was modeled by brick masonry, and floor slabs and structural measures were modeled by reinforced concrete. Since the Wenchuan earthquake in 2008, the importance of building structural measures in structural integrity and robustness has been emphasized. However, according to the field survey, there were still some masonry buildings that lack structural measures because of poverty or historical factors. An additional model without structural measures was established to compare the contribution of structural measures to the flood-resistant performance. As an example, only the model with structural measures is introduced here.

The full FE model is illustrated in Figure 3(b). The building was a 2-bay (longitudinal), 3-span (transverse), and 2-story (vertical) masonry structure. It consisted of two floors, each with a height of 3.2 m. The thickness of the exterior wall was 240 mm, while the thickness of the interior wall was slightly smaller. Windows and doors were set on the front and back exterior walls. The window size was $1.2 \times 1.5 \times 0.9$ m height from the bottom. The door size was $0.9 \times 2.2$ m, and it was placed in the middle of the wall. The reinforcing steels of the structural columns and ring beams consisted of 4@12 longitudinal rebars and 6-mm-diameter hoops placed at 240 mm spacing. The transverse and longitudinal reinforcing steels of the slab were modeled with 12-mm-diameter rebars at 240 mm spacing. The section of ring beams and structural columns was rectangular, 240 mm $\times$ 240 mm. The thickness of the cast-in-place floor was 120 mm. Openings (doors and windows) were recognized as the weakest elements, which were broken firstly during the flooding process, so they were assumed to have no resistance to the flash flood. The brick, mortar, and concrete strength values were equal to 15.0 MPa, 10 MPa, and 20 MPa, respectively. HRB 335 was used for the steel. Both dead loads (gravity) and live loads were considered. Pursuant to the load code for the design of building structures [33], uniformly distributed loads of 2 kN/m$^2$ and 0.5 kN/m$^2$ were imposed on the top surface of the slab and the roof, respectively. Masonry and concrete were modeled using eight-node solid elements with reduced integration. The steel was modeled as two-node beam elements using the Hughes Liu beam element formulation. The concrete and the steel were coupled through sharing nodes, where the bond-slip effect was not considered. All degrees of freedom of the bottom nodes were constrained, and the foundation failure was not considered.

The masonry material model was defined by an anisotropic brittle damage model (MAT_BRITTLE_DAMAGE) derived from Govindjee et al. [34]. It can be applied to a wide variety of brittle materials. This model can reflect the progressive degradation of tensile and shear strengths across smeared cracks initiated under tensile loadings. Compressive failure is governed by a simplistic second invariant of the deviatoric stress tensor. The damage is handled by treating the rank four elastic stiffness tensor as an evolving internal variable for the material.

To accurately consider the critical characteristics of concrete under immense strain, high strain rate, and high confining pressure, a computational constitutive model (MAT_JOHNSON_HOLMQUIST_CONCRETE) was used to describe the properties of concrete. It uses the piecewise state equation to describe the relationship between pressure and volume strain [35]. The yield surface equation can be defined as follows:

$$\sigma^* = \frac{\sigma}{f_c} = \left[ A (1 - D) + BP^*N \right] \left[ 1 - C \ln \varepsilon^* \right],$$

where $\sigma^*$ is the normalized stress, $\sigma$ is the actual equivalent stress, $f_c^*$ is the quasi-static uniaxial compressive strength of concrete, $D$ is the damage parameter ($0 \leq D \leq 1$), $P^* = P/f_c^*$ is the normalized pressure (where $P$ is the actual pressure), $\varepsilon^* = \varepsilon/\varepsilon_0$ is the dimensionless strain rate, and $A$, $B$, $N$, and $C$ are material constants (where $A$ is the normalized cohesive strength, $B$ is the normalized pressure hardening coefficient, $N$ is the pressure hardening exponent and $C$ is the strain rate coefficient). The damage is calculated and expressed as a function of the plastic volumetric strain, equivalent strain, and pressure:

$$D = \sum \frac{\Delta \varepsilon_p + \Delta \mu_p}{D_1 (P^* + T^*)^{D_2}},$$

where $\Delta \varepsilon_p$ and $\Delta \mu_p$ are the equivalent plastic strain and plastic volumetric strain, $D_1$ and $D_2$ are material constants, and $T^* = T/f_c^*$ is the normalized maximum tensile hydrostatic pressure (where $T$ is the maximum tensile hydrostatic pressure).
Figure 2: Photographs of damaged buildings due to flash floods in Wenchuan in 2019.

Figure 3: Schematic diagrams: (a) global disaster scenario and (b) finite element model of the masonry building with structural measures.
A kinematic hardening bilinear elastoplastic model (MAT_PLASTIC_KINEMATIC) was employed to model steel rebars. In this model, isotropic, kinematic, or a combination of isotropic and kinematic hardening can be obtained by varying a parameter. In isotropic hardening, the centre of the yield surface is fixed, but the radius is a function of the plastic strain. In kinematic hardening, the radius of the yield surface is fixed, but the centre translates in the direction of the plastic strain. The strain rate effect of a rebar is considered by the Cowper–Symonds model [35], which scales the yield stress with the factor:

\[ 1 + \left( \frac{\dot{\varepsilon}}{C} \right)^{1/p} \]  

where \( \dot{\varepsilon} \) is the strain rate; \( C \) is the hardening parameter of the strain rate.

The constitutive model parameters for materials are listed in Table 1, and detailed descriptions of the material models can be found in Reference [35]. The erosion algorithm lends an excellent means to imitate cracking and failure phenomena with graphical plots. When an element reaches the failure threshold, it will be assumed to have no resistance and is then deleted from the simulation. In the present study, different failure criteria were set for different materials on the basis of experience and relevant literature. The maximum failure principal strain of the masonry element was defined as 0.0015 [36]. A concrete element was allowed to erode when it reached a principal tensile fracture strain of 0.001 [26, 37]. An effective plastic failure strain of 0.1 was adopted for the rebar [30].

### 2.2. The Flash Flood Model

The Arbitrary Lagrangian–Eulerian (ALE) method provides an appropriate option for simulating the mechanical response of highly deformable materials and performing analyses for fluid-structure interaction with a massive momentum transfer in a short period [38]. A multimaterial ALE formulation was chosen for the treatment of the fluid. In this formulation, the computational mesh can be moved in any arbitrarily specified way to be capable of continuously rezoning. Neither the material configuration \( R_x \) nor the spatial configuration \( R_\chi \) is taken as the reference. The referential configuration \( R_\chi \) and the reference coordinates \( \chi \) are introduced to identify the grid points. The relation between velocities, the material velocity \( v \), the mesh velocity \( \dot{v} \), and the particle velocity \( w \) (seen from the referential domain \( R_\chi \)), can be expressed as [39]

\[ c = v - \dot{v} = \frac{\partial x}{\partial \chi} \cdot w, \]  

where \( c \) is the particle velocity relative to the mesh as seen from the spatial domain \( R_x \) and is also defined as convective velocity. The conservation equations of mass, momentum, and total energy can be expressed as

\[ \frac{\partial \rho}{\partial t} + c \cdot \nabla \rho = -\rho \nabla \cdot v, \]

\[ \rho \left( \frac{\partial v}{\partial t} + (c \cdot \nabla) v \right) = \nabla \cdot \sigma + \rho b, \]  

\[ \rho \left( \frac{\partial E}{\partial t} + c \cdot \nabla E \right) = \nabla \cdot (\sigma \cdot v) + v \cdot \rho b, \]

where \( \rho \) is the density, \( \sigma \) represents the Cauchy stress tensor, \( b \) is the specific body force vector, and \( E \) is the specific total energy.

The ALE method in LS-DYNA utilizes an operator splitting scheme to deal with diffusive and advective terms [35]. An ALE formulation consists of a Lagrangian time step, in which the momentum equation is solved to update accelerations, velocities, and displacements, and an “advection” step, in which all history variables are mapped to the updated mesh with fixed topological conditions. The interface construction method is used to reconstruct fluid interfaces between multiphase fluids based on the gradient of the volume for mixed elements. This study applied a new Structured ALE (S-ALE) solver, which is dedicated to solving ALE problems using structured mesh [40]. It takes advantage of the structured mesh, and the element connectivity is patterned. Besides, it has robust control of leakage in fluid-structure interaction and can better capture the peak value of impact pressure.

The fluid domain was discretized through 8-node solid elements with one integration point, and the air was also considered. Generally, flash floods contain a small amount of solid material (such as sand), and its material properties are similar to those of clear water with a density of 1,000–1,200 kg/m³. For simplicity, clear water was applied. The fluid was modelled as a weakly compressible and single-phase viscous flow. The null material model (MAT_NULL), which ignores the yield strength, was selected to describe the physical properties and rheological behavior of flash floods. The model is dominated by viscous stresses with higher mobility and is often used to model a fluid-like material by researchers [12, 27]. The viscous stress can be computed as follows [35]:

\[ \sigma^{\alpha\beta} = 2\mu_d \varepsilon^{\alpha\beta}, \]  

where \( \varepsilon^{\alpha\beta} \) is the deviatoric strain rate, and \( \mu_d \) is the dynamic viscosity. The equation of state (EOS) needs to be defined to express the relation between the pressure and the volumetric change for this model. A classic stiff equation of state (EOS_MURNAGHAN) was applied. This equation of state was designed to model weakly compressible fluid flow. The pressure is defined as [35]

\[ P = k_0 \left( \frac{\rho}{\rho_0} \right)^\gamma - 1, \]

where \( \rho_0 \) is the reference density, \( \gamma \) is a dimensionless parameter typically set to 7, and \( k_0 = c_0^2 \rho_0 \gamma^{-1} \), in which \( c_0 \) represents the sound speed at the reference density. Due to...
the Courant–Friedrichs–Lewy stability condition, the computational time step based on the sound speed is always too small, leading to an extremely high computational effort. For this reason, the computational time step was scaled up by reducing the sound speed. This approach can be regarded as effective if the reduced sound speed is still approximately ten times greater than the expected maximum flow velocity, which causes density variations smaller than 1% [41]. In addition, the ALE mesh needs to be fine enough to capture the highest impact forces well, yet a coarser mesh is favorable in terms of the computational efforts. After testing five mesh sizes, an ALE mesh size of 8 cm was chosen to strike a balance between accuracy and computational efforts. Hourglass control was used with viscous form. Although the adopted method did not capture all the features of natural flow materials, the method has been proved to be feasible to reflect the fundamental physical nature well [12, 30].

### 2.3. Coupling Interfaces and Boundary Conditions

The interaction between the ALE mesh and the Lagrangian mesh was activated by using a penalty-based coupling algorithm. The algorithm places an imaginary normal interface spring at the contact surface. Upon iterations, the impact pressure and boundary conditions are transferred. The coupling force \( F \) is given by

\[
F = k_d d + c d,
\]

where \( k \) is the contact stiffness, \( c \) represents the damping coefficient, and \( d \) is the relative displacement between the slave node and the master particle in the direction normal to the interface. The contact stiffness per unit area is defined as

\[
k_d = \frac{k}{A},
\]

where \( A \) is the average face area of the fluid-structure interface. The value of \( k_d \) can either be specified directly or given in terms of the scale factor for the interface stiffness, the bulk modulus, and the volume of the fluid element. In the present study, \( k_d \) was specified directly, which was helpful to the stability of the calculation and the control of leakage. An appropriate value of the contact stiffness is a critical point to the simulation because too small and excessive contact stiffness will, respectively, lead to excessive nonphysical penetration (leakage) and numerical noise in the results. Hereby, a simplified symmetric model was established and six values of the contact stiffness \( k_d \) (0.015 GPa/m, 0.15 GPa/m, 1.5 GPa/m, 15 GPa/m, 150 GPa/m, and 1500 GPa/m) have been tested to ascertain its appropriate value. From Figure 4, the optimum value, \( k_d = 150 \text{ GPa/m} \), was determined.

The ground was simplified to be a plane surface, and both sides of the fluid were simulated as sliding conditions. In addition, the friction at the interface was also considered using a Coulomb formulation. Luo et al. [12] indicated that the friction coefficient was insensitive to the flood and impact processes. A typical value of 0.1, which was widely used in viscous flow modelling [29, 30], was adopted.

### 3. Results and Discussion

#### 3.1. Descriptions of the Impact Process

The impact process in all scenarios involved in this study can be divided into four classes, namely slight damage, moderate damage, significant damage, and complete damage cases. An overview of the typical simulation scenarios is shown in Table 2.

Figure 5 shows some significant snapshots concerning a typical slight damage case, i.e., the scenario of \( s = 0, h = 0.8 \text{ m}, \text{ and } v = 4.0 \text{ m/s} \). In this scenario, a splash-up and an upward-moving jet were formed. The jet was associated with strong accelerations that produced high-pressure gradients and caused a jump in water depth, making an abrupt increase in the impact force. When the jet, decelerated by gravity, reached the maximum elevation, it posed the highest hydrostatic pressure to the building and started to detach from the wall at 0.70 s (Figures 5(b) and 5(e)). Then, the flood flipped up in front of the incoming flood and moved backward. These phenomena implied that the flood was a supercritical flow, which was characterized by the Froude number larger than unity [42], and can be observed in flume tests [41]. Because of no openings on the sidewalls and interior walls, a large amount of floodwater accumulated inside the building (Figures 5(c) and 5(f)). However, due to low loads caused by the flood, no walls collapsed and there was slight damage or even no damage to the structure. From the perspective of restoration and reconstruction, the building can be immediately used after an event, and the damage can be repaired rapidly at a low cost.

Figure 6 shows numerical results of the impact process concerning a typical moderate damage case, i.e., the scenario of \( s = 1, h = 1.6 \text{ m}, \text{ and } v = 8.0 \text{ m/s} \). In this scenario, a significant splash-up and an upward-moving jet were also seen. The flood broke through masonry walls and directly rushed into the structure (Figures 6(b) and 6(e)). After the

| Material property                        | Masonry wall | Concrete | Rebar |
|------------------------------------------|--------------|----------|-------|
| Density, \( \rho \) (kg/m\(^3\))         | 1,800        | 2,400    | 7,850 |
| Elastic modulus, \( E \) (MPa)           | 3,700        | 23,000   | 207,000 |
| Poisson’s ratio, \( \nu \) (-)           | 0.15         | 0.15     | 0.3   |
| Yield stress (MPa)                       | —            | —        | 335   |
| Compressive strength (MPa)               | 2.31         | 30       | —     |
| Failure criterion (-)                    | Maximum principal strain 0.0015 [36] | Maximum principal strain 0.001 [26, 37] | Effective plastic strain 0.1 [30] |

**Table 1:** Constitutive model parameters of the building model.
destruction of the exterior frontal walls, the flood destroyed the 2nd-row interior walls and the 3rd-row exterior walls subsequently (Figures 6(c) and 6(f)). All first-floor masonry walls orthogonal to the flow direction were almost destroyed. The first-floor masonry walls parallel to the flow were damaged slightly due to the longitudinal backshielding effect of the 1st-row columns. A buffer zone formed behind the columns, where water depth and flow velocity were smaller than that in the undisturbed zone. After losing the partial support of the first-floor load-bearing walls orthogonal to the flow direction, the building relied on other walls and structural measures to undertake the redistributed superstructure loads and avoid the building collapse. Such damaged buildings were observed in the field survey in Wenchuan (Figures 2(c) and 2(e)).

Some simulation results concerning a typical significant damage case, i.e., the scenario of \( s = 0, h = 1.6 \text{ m}, \) and \( v = 8.0 \text{ m/s} \) are illustrated in Figure 7. The physical phenomenon in this scenario was similar to the “moderate damage” case; more load-bearing components were damaged; no building collapse was observed. An upward-moving jet was seen; the brittle failure occurred and led to the forward collapse of the whole building.

| Damage state | Typical simulation scenario | Field evidence | Main failure characteristics |
|--------------|-----------------------------|----------------|----------------------------|
| Slight       | \( s = 0, h = 0.8 \text{ m}, \) and \( v = 4.0 \text{ m/s} \) (Figure 5) | Figure 2 (f)  | A splash-up and an upward-moving jet were formed; a large amount of floodwater accumulated inside the building; there was merely slight damage or even no damage to the structure. |
| Moderate     | \( s = 1, h = 1.6 \text{ m}, \) and \( v = 8.0 \text{ m/s} \) (Figure 6) | Figure 2 (c) and 2 (e) | A significant splash-up and an upward-moving jet were seen; all first-floor masonry walls orthogonal to the flow direction were almost destroyed; the building did not collapse. |
| Significant  | \( s = 0, h = 1.6 \text{ m}, \) and \( v = 8.0 \text{ m/s} \) (Figure 7) | Figure 2 (b)  | The physical phenomenon was similar to the “moderate damage” case; more load-bearing components were damaged; no building collapse was observed. An upward-moving jet was seen; the brittle failure occurred and led to the forward collapse of the whole building. |
| Complete     | \( s = 1, h = 3.2 \text{ m}, \) and \( v = 20.0 \text{ m/s} \) (Figure 9) | Figure 2 (d)  | A high-speed oblique-moving jet flow impacted the roof; bending failure (the roof), three-plastic-hinge bending failure (the 1st-row columns), and two-plastic-hinge bending failure (the 2nd-row and the 3rd-row columns) were observed; the entire building was overturned. |

![Figure 4: Time evolutions of the total impact force for tested values of contact stiffness.](image-url)
Not only the walls were severely damaged but also some columns had failed. The building survived with partial collapse.

Figure 8 depicts the destruction process of the building concerning a typical complete damage case, i.e., the scenario of $s = 0$, $h = 3.2$ m, and $v = 12.0$ m/s. The lower part of the building façade suffered severe damage (Figures 8(b), 8(e), and 8(h)), and an upward-moving jet was seen. With the continuous impact of the flood, the flood flowed into the building and generated an increasing thrust force on the building. Strain gradually developed in the load-bearing components. When the flood reached the 3rd-row exterior walls, the bottom of the 1st-bay longitudinal walls was suddenly broken. Almost at the same time, the bottom of
other load-bearing walls was damaged. Brittle failure occurred and led to the forward collapse of the whole building (Figures 8(c), 8(f) and 8(i)). Such damaged buildings were observed in the field survey (Figure 2(d)). The masonry structure with structural measures collapsed entirely due to the flood impact.

For another typical complete damage case, i.e., the scenario of \( s = 1, h = 3.2 \text{ m}, \) and \( v = 20.0 \text{ m/s}, \) a clear perspective of the building collapse process (one span) is shown in Figure 9. Because of the barrier effect of the ring beam on the flow, a high-speed oblique-moving jet flow was formed. The jet impacted the roof and exerted an out-of-plane force on the roof. When the jet destroyed the 2nd-row interior walls on the second floor, the roof lost effective restraints (Figures 9(a) and 9(d)). After that moment, the connection between walls and structural measures, and the connection between walls and ground were gradually lost due to the combination of the upward force and the thrust force. Many cracks (transverse crack, vertical cracks, and diagonal cracks) appeared, and most of the load-carrying walls were
damaged. At about 0.58s, bending failure (the roof), three-plastic-hinge bending failure (the 1st-row columns), and two-plastic-hinge bending failure (the 2nd-row and the 3rd-row columns) were observed (Figures 9(b) and 9(e)). Eventually, as shown in Figures 9(c) and 9(f), the entire building was overturned.

3.2. Failure Mechanisms of Masonry Structures. The masonry walls orthogonal to the flow direction, which suffered significant out-of-plane loads, were most vulnerable to the flood. Figure 10 shows the maximum principal strain in frontal masonry walls at the failure moment in the scenario of $s = 1$, $h = 1.6$ m, and $v = 8.0$ m/s. Since the masonry elements near openings were in a complex stress state, the flood can easily destroy this part of masonry elements, and the failure mode presented a typical bending failure mode. Apart from some elements directly damaged by the flood impact, the failure of the masonry wall can be attributed to the rotation of the wall along with the hinges, i.e., the fracture lines. These fracture lines were generated in the region of larger bending moments and split the wall into pieces of blocks.

The accurate capture of the dynamic impact of the flash flood against a structure is a crucial phase to explore the failure mechanism of buildings. The impact load derives from the combination of inertia and drag components. It depends on the kinematics of the flood and the building’s geometrical characteristics [20, 43]. The impact is often simply evaluated as the hydrostatic pressure multiplied by an arbitrary coefficient larger than one in the engineering praxis [27, 30]. This approach is empirical and may lead to the underestimation of the correct impact force. A more recommended expression for the hydrodynamic load is calculated by

$$P = 0.5 \cdot C_D p u^2,$$

where $P$ is the impact pressure, $C_D$ is the drag coefficient, and $u$ is the velocity orthogonal to the object. However, this equation may confuse inertial forces with drag forces. The standard published by FEMA (Federal Emergency Management Agency) [44] suggested using a coefficient of 2 when assessing the drag component only while assuming a coefficient of 3 to account for the implosive component. Xiao and Li [24] pointed out that there was a pressure difference on the object’s surface, but using this expression can only give the average pressure.

An example of impact force history recorded in the scenario of $s = 1$, $h = 1.6$ m, and $v = 8.0$ m/s is shown in Figure 11. The evolution of the impact force on the entire building was distributed in a multi-peak pattern, which means that the flood would cause secondary impact and damage to the building after the flood destroyed the frontal masonry walls (Figure 11(a)). In the interaction process, the massive kinetic energy was dissipated and transferred to the deformation energy of the building. Although the 1st-row exterior walls had absorbed part of the kinetic energy, the flood kept marching forward and damaged the 2nd-row and 3rd-row walls. These peaks correspond to the destruction of the 1st-row exterior walls, the 2nd-row interior walls, and the 3rd-row exterior walls, respectively. Moreover, the time history of the impact force on the 1st-row columns presented a sharp (inertia-dominated) peak followed by a less intense but longer-lasting (quasi-static, drag-dominated) force (Figure 11(b)). The difference in the peak value between columns suggested that, for the corner column, the flood can flow through the open area from one side. In contrast, for the middle column, the masonry wall on both sides blocked the flow resulting in a strong interaction mechanism and a larger force. When all the masonry walls near the columns were

![Figure 9: Snapshots in the scenario of $s = 1$, $h = 3.2$ m, and $v = 20.0$ m/s (one span).](image-url)
broken, the lasting drag forces on the columns were similar, about 23 kN. By comparing with the theoretical hydrodynamic load, the simulated peak values were between the calculated values of coefficient 2 and coefficient 3. Thus, the dynamic impact can be captured by the model presented, and the equation proposed by FEMA is relatively reasonable and conservative for the water-like material.

The masonry structure collapse occurs if the total amount of loads exceeds the residual vertical load-bearing capacity of the masonry structure [45]. Based on simulation results, it was argued that the collapse of the masonry structure without structural measures was caused by the brittle in-plane damage of the longitudinal load-bearing walls (bending mechanisms). Figures 12(a), 12(c), and 12(e) demonstrate the vertical force, shear force, and bending moment evolutions at the bottom of longitudinal walls in the scenario of \( s = 0, h = 3.2 \text{ m}, \) and \( v = 12.0 \text{ m/s}, \) respectively. The black dotted line corresponds to the
Figure 12: Continued.
moment when the walls began to break. Positive vertical forces indicated compression and the shear force refers to the force in the flow direction. The bending moment was positive in the direction of the building overturning forward. It can be seen that the structural response gradually shifted from being dominated by gravity to impact forces. The damage of frontal walls resulted in a redistribution of superstructure loads and an increase in vertical forces of the 1st-bay walls. After that, the vertical force in the 1st-bay walls declined amid fluctuations. Whereas the vertical force in the 2nd-bay walls increased with oscillatory behaviors, and the peak vertical force up to 852 kN was obtained at 0.67 s. This difference was due to the continuing flood thrust forces overturning the building. Before the collapse, the shear force performed an apparent upward trend, and the peak values of the 1st-bay and the 2nd-bay walls were 658 kN and 723 kN, respectively. The bending moment of the 1st-bay walls increased in an oscillatory manner until the collapse happened. At about 0.73 s, the bending moment at the bottom of the 1st-bay walls reached the maximum value of 584 kN-m. The bending moment of the 2nd-bay walls increased firstly and then decreased to a negative value due to the overall load-bearing and force-transmitting mechanism of the structure. The elements at the bottom of the walls were in complex compression-shear or tension-shear states, and the bending moment was the primary cause of the wall damage. When the flood reached and destroyed the 3rd-row exterior wall at 0.74 s, the elements at the bottom of the 1st-bay longitudinal walls were damaged firstly, and then all the elements at the bottom failed instantly. The severe brittle in-plane failure occurred and this stage corresponded to the toppling failure of the building.

To indicate the effects of structural measures on the structure, time histories of vertical force, shear force, and bending moment at the bottom of longitudinal walls in the scenario of \( h = 3.2 \text{ m}, v = 12.0 \text{ m/s}, s = 0 \) (a, c, e), and \( s = 1 \) (b, d, f). For the further analysis of the contribution of structural measures, the evolutions of total vertical force, total shear force, and total bending moment at the bottom of reinforced concrete columns (the columns of the masonry building with structural measures) and masonry columns (the columns of the masonry building without structural measures) in the scenario of \( h = 3.2 \text{ m}, v = 12.0 \text{ m/s} \) were obtained separately. From Figure 13, the evolution laws of these physical quantities were similar, but the magnitude of structural columns (RC columns) was much greater than that of masonry columns. Under the flash flood impact, it bore more loads to ensure the safety of the building. The total vertical force, the total shear force, and the total bending moment in the RC columns were about four times, five times, and five times that in the masonry columns, respectively. Structural measures can effectively control the overall structural response. It may ensure alternative load
paths for damage-induced overloads, and improve the robustness and resilience of the building, resulting in a stiffer, stronger, and more stable resisting mechanism.

To explore the detailed failure mechanism of the masonry structure with structural measures, Figure 14 demonstrates the comparisons of vertical forces, shear forces, and bending moments between the different bay columns in the scenarios of $s = 1$, $h = 3.2$ m, and $v = 20.0$ m/s. The continuing flood thrust forces caused the building to undergo a severe overturning moment, resulting in the first two rows of columns in tension and the last row of columns in compression. At 0.23 s, their connection, as well as the frontal walls on the second floor, began to be damaged. The damage of connection cut off the lateral load path leading to a significant change in structural stresses. At 0.42 s, the connection was destroyed severely, and the concrete in the 2nd- and 3rd-row columns was partially damaged, causing another sudden change in structural stresses. Under the complex tension-shear effects, the 1st-row columns reached the ultimate capacity at 0.46 s and began to fail. It was completely destroyed at 0.55 s (three-plastic-hinge mechanisms). At 0.60 s, the remaining columns on the first floor failed simultaneously (two-plastic-hinge mechanisms) and a global toppling failure was forced to occur. Thus, structural measures may change the failure mode of masonry structures.

Figure 13: Comparisons between the scenarios of $s = 0$ and $s = 1$ ($h = 3.2$ m, $v = 12.0$ m/s): (a) total vertical force, (b) total shear force, and (c) total bending moment at the bottom of columns.
In all, the impact of flash floods led the building to a complex bending-shear state, and the overturning moment was the principal reason for the building damage, no matter there were structural measures or not. The destruction of masonry structure without structural measures was almost instantaneous. In contrast, the destruction process of masonry structure with structural measures was similar to that of reinforced-concrete framed structure, i.e., a progressive collapse process. The collapse of the building was not triggered by the failure of masonry walls but by reinforced concrete columns. According to the analysis in the previous texts and the field evidence, the failure mechanism of masonry structure caused by the impact of flash floods can be concluded:

1. Out-of-plane damage to masonry walls (bending mechanisms, especially for transverse walls)
2. In-plane damage to masonry walls (bending and shear mechanisms, especially for longitudinal walls)

Figure 14: The comparison between the different bay columns in the scenarios of $s = 1$, $h = 3.2$ m and $v = 20.0$ m/s: (a) vertical force, (b) shear force, and (c) bending moment.
(3) Formation of plastic hinges in RC columns (bending mechanisms, including two-plastic-hinge mechanisms and three-plastic-hinge mechanisms)

(4) Shear failure of RC columns (brittle shear mechanisms)

(5) Building collapse (insufficient bearing capacity of the remaining structure to bear the superstructure loads)

For the masonry structure without structural measures, the mechanism (1), (2), and (5) are involved, while the failure mechanisms of the masonry structure with structural measures may refer to all these mechanisms. What needs to be illustrated is that the more accepted failure mode of the reinforced concrete column impacted by flow-type disasters is the formation of the plastic hinges. At the same time, if there is a massive energy transfer during the impact process, the shear may be the predominant failure mode due to the high rate of loading [46].

3.3. Physical Vulnerability Analysis. The physical vulnerability of the building is expressed as the expected physical damage state at a given hazard intensity level. To evaluate the physical vulnerability of masonry structures impacted by flash floods, the damage state of the building in the specified scenario needs to be determined [30]. Generally, the damage state represents qualitative descriptions of frequently observed damage features and patterns as categories on an ordinal scale from 0 (no damage) to 1 (total destruction) for engineering applications. They are easily understood by experts and nonexperts, making them easy communication tools about the consequences of hazards. However, as suggested by Mavrouli et al. [47], to analyze the physical vulnerability, it is necessary to establish standard indices that can express the damage state with physical interpretations quantitatively. A combination of qualitative descriptions and quantitative indices will enhance the explanatory power in explaining physical mechanisms and extend its applicability. It should be noticed that there is a wide range of damage modes available to describe how masonry structures respond to flash flood impacts. Including all failure modes will lead to unnecessarily complicated descriptions. Consequently, damage states should capture predominantly damage patterns of the masonry structure to serve as a compromise between comprehensiveness and simplicity [48]. Besides, the working scale of the indices should also be considered, as there is a distinction between damage indices that refer to components, buildings, or even building clusters [47]. When only a few isolated structural elements are involved in the damage, it can often be supposed that the damage of the structure is due to local damage processes. In this case, damage indices usually are calculated as a function of the stresses or strains that develop at the member sections. As for the buildings or building clusters cases, they are usually obtained as a function of the overall macroscopic response, providing a macroscopic view of the response and easy access to engineering applications.

In the literature, maximum-impact pressure is often used as an essential indicator to analyze the physical vulnerability of buildings [32, 49, 50]. However, because of different research methods (field survey, experimental measurement, or numerical simulation), the critical impact pressure given is quite different, and it is difficult to distinguish between the local pressure and the average pressure. The peak interstory drift ratio is recognized as an appropriate description of structural response, which offers a compromise between local and global structural behavior [51]. Here, for the engineering application simplicity, it was decided to introduce a damage index, the lateral drift ratio of the ground floor, i.e., the ratio between the peak lateral displacement of the point \( P_0 \) in the specified scenario \( (\delta_{\text{max}}) \) and the ultimate displacement of the point \( P_0 \) before building collapse \( (\delta_u) \), to analyze overall building performance and physical vulnerability. In detail, that ultimate displacement was assumed to be 0.35 mm and 0.45 mm in the case of masonry structures without and with structural measures, respectively. \( P_0 \) was located at the corner of the first-floor ring beam, as shown in Figure 3(b).

Figure 15 presents the physical vulnerability of the masonry structure obtained by using the defined damage index. For a given structure, the physical vulnerability varied significantly with the flow velocity and the water depth. It can be roughly said that the larger the flow velocity or the water depth, the larger the physical vulnerability was. The vulnerability of the building without structural measures increased more rapidly with the increase of water depth and flow velocity than that of the building with structural measures. There were even some abnormal phenomena, i.e., when the flood intensity increased, the vulnerability was reduced. That means some key physical phenomena, such as the failure of some walls, occurred. An example was the abnormal relation between the scenario of the scenario of \( s = 0 \), \( h = 1.6 \text{ m} \), and \( v = 8 \text{ m/s} \) and the scenario of \( s = 0 \), \( h = 1.6 \text{ m} \), and \( v = 12 \text{ m/s} \). In these two scenarios, both the frontal walls were destroyed, but due to the lower flood intensity in the former scenario, the extent of damage of the frontal walls was smaller than that in the other scenario. This part of the undamaged wall blocked the flow, resulting in a stronger upward-moving jet and a greater force. Therefore, the vulnerability decreased abnormally. Another example was the abnormal relation between the scenario of \( s = 0 \), \( h = 3.2 \text{ m} \), and \( v = 4.0 \text{ m/s} \) and the scenario of \( s = 1 \), \( h = 3.2 \text{ m} \), and \( v = 4.0 \text{ m/s} \). In the former scenario, all the masonry walls orthogonal to the flow direction on the first floor were destroyed. The flood impact pressure was only slightly transmitted to the columns as the pressure only acted on the surface of the columns. However, in the other scenario, these walls were not broken due to the contribution of the structural measures. The pressure acted on the surface of masonry walls and columns, resulting in a larger lateral thrust force and a higher vulnerability value. Strictly speaking, it cannot be simply assumed that the greater the flood intensity, the greater the physical vulnerability. Some key physical phenomena, which would lead to anomalies in the vulnerability analysis, in the interaction process are very important and should be paid special attention to. Thus, being consistent with Luo et al. [12], for the building itself, the physical vulnerability depends on both the local
structural strength (local view) and the structural resistance hierarchies (global view).

According to the above analysis, it is obvious that flash floods pose a great threat to masonry structures. Considering the safety of buildings and inhabitants, there are some countermeasures and suggestions for reference. First, high-strength structural measures should be taken to strengthen the overall flood-resistant performance. Second, referring to the guidelines of FEMA [44], the frontal masonry walls on the first floor are recommended to be set as “breakaway walls” to limit the flood forces on the overall building and individual structural components, under the premise that the building will not collapse without these walls. Third, it is suggested to raise the building elevation or set appropriate measures around the building to avoid the direct impact of the flood. Fourth, the shape of buildings and the layout of openings should be optimized to reduce the flood forces. Fifth, valuables should be placed on the second or higher floor of the building, and in case of floods, inhabitants should escape to the higher floor as soon as possible.

Figure 15: Physical vulnerability: (a) flow velocity evolutions, building without structural measures, (b) flow velocity evolutions, building with structural measures, (c) water depth evolutions, building without structural measures, and (d) water depth evolutions, building with structural measures.
4. Conclusions

Understanding, identifying, and quantifying vulnerability is a prerequisite for assessing flood risk accurately and designing effective risk reduction strategies. A process-specific physical vulnerability analysis, focusing on the real physics behind the phenomena, will result in an appropriate damage mechanism concept. Numerical simulations provide a rational way to bring detailed useful information for understanding the disaster process. In this study, thirty nonlinear numerical models based on S-ALE solver were established to investigate the interaction between flash floods and masonry structures. Nonlinear structural responses, failure mechanisms, and physical vulnerability of masonry structures were analyzed. Hereby, conclusions can be drawn as follows:

(1) Detailed descriptions of the interaction between flash floods and masonry structures in the typical simulation scenarios, including four damage states, were presented and the main failure characteristics were given.

(2) Due to the consideration of wall damage, the time evolution of the impact force on the entire building was distributed in a multi-peak pattern. The impact force on the 1st-row columns presented a sharp (inertia-dominated) peak followed by a less intense but longer-lasting (quasi-static, drag-dominated) force.

(3) Under the impact of flash floods, the building was in a complex bending-shear state, and the overturning moment was the principal reason for the building damage, no matter there were structural measures or not.

(4) The critical role played by the structural measure was reaffirmed. It may ensure alternative load paths for damage-induced overloads, and improve the robustness and resilience of the building, resulting in a stiffer, stronger, and more stable resisting mechanism.

(5) The physical vulnerability was analyzed based on the defined macroscopic damage index (the lateral drift ratio of the ground floor). It can be roughly said that the damage index was positively correlated with flood intensity, and the vulnerability of buildings without structural measures changed more dramatically. Some key physical phenomena in the interaction process, which would lead to anomalies in the vulnerability analysis, should be paid special attention to. Regarding the building itself, the physical vulnerability depends on both the local structural strength (local view) and the structural resistance hierarchies (global view).

This paper gives a more comprehensive understanding of the interaction between flash floods and masonry structures, as well as the physical vulnerability of buildings, promoting the development of flood risk assessment. It can assist researchers, designers, and governments in implementing preventive measures and mitigation strategies to reduce the adverse influence of potential flood events. In the future, more extensive additional studies with longer follow-up are needed to evaluate the influence of incident flow angle, building clusters, and the objects swept away by the flood on structural damage and vulnerability.

Data Availability

The relevant public archived data sets are available from the corresponding author upon request. The model parameters used in this paper are from surveys and related literature.

Conflicts of Interest

The authors declare that they have no conflicts of interest with respect to the research, authorship, and publication of this paper.

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