

Article

Potential Impulse Wave Analysis for Sejiang Deforming Slope on the Near-Dam Reservoir Bank of Bala Hydropower Station of China

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Abstract

Landslide-generated impulse waves in deeply incised gorge regions pose significant risks to hydropower infrastructure, particularly under fluctuating reservoir water levels. This study investigates the potential global instability sliding of the Sejiang deforming slope on the left bank of the Bala Hydropower Station, Sichuan Province of China. A three-dimensional numerical simulation of the landslide–water entry, wave generation, and propagation processes conducted using computational fluid dynamics (CFD) shows a maximum wave height of 5.57 m on the opposite bank. Moreover, the CFD results were compared with those derived from Pan's empirical formula method. In contrast, Pan's empirical formula only provides conservative predictions for water-entry velocity and wave height, with a maximum wave height of 13.60 m on the opposite bank. Notably, the CFD numerical simulations precisely characterize complex topographic effects, such as the approximately 156% wave height amplification at the spoil disposal bend due to reflection and superposition, as well as the localized energy convergence in front of the dam. Furthermore, the impulse wave destructive potential is positively correlated with reservoir water levels. While the assessment confirms no risk of dam overtopping under the current scenarios, it highlights the necessity for differentiated protection strategies targeting three critical zones, i.e., the initial wave impact zone, the multi-directional superposition zone at the spoil disposal area, and the localized energy convergence zone near the dam. This study provides a reliable quantitative basis for refined hazard assessment and disaster mitigation in complex reservoir topographies.



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1. Introduction

As hydropower development in western China continuously expands into high-altitude, tectonically active, and geologically complex regions, the long-term stability of deforming slopes along reservoir banks faces tremendous challenges. These challenges arise from the superposition of multiple factors, including severe cyclic fluctuations of reservoir

water levels, reshaping of high and steep reservoir banks, and strong regional seismic loads [1,2]. The reservoir water level fluctuation is regarded as the critical triggering factor for river bank landslides [3–5]. This impact is particularly pronounced in the deeply incised gorge regions of southwestern China, where deforming slopes are characterized by fragile rocks, and complex geological structures, e.g., anti-dip and thin-ridge formations. Under the coupling effects of groundwater and weak structural planes, these slopes are susceptible to sudden instability and failure, which can subsequently trigger a catastrophic secondary disaster chain of landslide-generated impulse waves [6–8]. The impact pressure exerted by these impulse waves can cause severe damage to the downstream dam and hydraulic structures [9]. Furthermore, these waves generate complex reflection and superposition effects within narrow reservoir areas [10], posing a severe threat to the safety of hydropower stations and the full-lifecycle operation of dams.

Numerous methods have been proposed for assessing landslide-generated impulse waves, in which empirical formula methods and numerical simulation methods have become the most representative approaches in current academic research and engineering practices, owing to their remarkable engineering practicality and broad scenario applicability. The empirical formula method, typified by Pan's Model [11], is based on the rigid-body assumption and the unidirectional reservoir flow theory. The movement velocity and wave height of landslide can be estimated through simplified kinematic and wave equations. On the basis of Pan's Model, Yu et al. [12] derived impulse wave calculation formulae which are applicable to different instability modes of dangerous rock masses, and developed a comprehensive calculation system for impulse waves induced by hazardous rock plunging into water. To enhance the accuracy of landslide velocity calculation, Zhong et al. [13] further incorporated the three-dimensional geometric characteristics of the landslide into Pan's model, while coupling the effects of pore water pressure and hydrodynamic drag force. Validated by the Qianjiangping landslide case, the calculation error of Pan's empirical formula method was merely 6.19%, which is significantly superior to the traditional two-dimensional analytical methods. It fails to fully account for the complex dynamic variations induced by landslide instability and exhibits difficulties in accurately tracking the free water surface under complex topographic conditions [14].

Numerical simulation is of great value in illustrating the evolutionary characteristics of geological disaster within water conservancy and hydropower engineering projects; among them, CFD is widely used, owing to its unique advantages in multiphase flow coupling, free-surface tracking, and complex boundary treatment [15]. Maleki et al. [16] established a three-dimensional model in CFD to systematically investigate the synergistic effects of the slide material, width, thickness, and impact velocity on wave generation. Similarly, Huang et al. [17] utilized CFD to conduct a time-frequency analysis on the near-field propagation of impulse waves generated by subaerial landslides, proposing the concepts of the near-field characteristic wave energy peak and its transmission velocity. Numerical simulations using the CFD software can characterize the spatiotemporal distribution of wave amplitudes at the dam face, variations in propagation velocity, and maximum wave run-up under various landslide failure scenarios. This approach can quantitatively analyze the impacts of complex topography and landslide kinematics on the destructive potential of impulse waves, thereby effectively overcoming the accuracy limitations inherent in traditional empirical methods under complex environmental conditions [18,19]. However, numerical simulation studies primarily focus on parametric sensitivity analyses under idealized topographic conditions or simulations of specific landslides. There remains a lack of a systematic comparison with classical empirical formula methods, nor have the complementary roles of numerical simulations and empirical approaches in engineering practice been clearly elucidated.

In this study, the Sejiang deforming slope on the left bank near the dam of the Bala Hydropower Station is taken as an example, which is located in the upper reaches of the Dadu River, Sichuan province of China. Three-dimensional numerical simulations are conducted to illustrate the entire process of instability and sliding along the slip zone under the reservoir impoundment level. The results obtained from the numerical simulations are systematically compared with that from Pan's empirical formula. Differentiated engineering protection strategies are proposed with integrating the spatiotemporal evolution patterns of impulse waves within the reservoir area for the initial wave impact zone on the opposite bank, the continuous fluctuation zone at the spoil disposal area, and the energy convergence zone in front of the dam.

2. Engineering Background

Bala Hydropower Station, a key water conservancy project in the upper reaches of the Dadu River, is situated within the alpine canyon region of northwestern Sichuan on the eastern margin of the Qinghai–Tibet Plateau (Figure 1). The reservoir area comprises the main stream of the Jiaomuzu River and several dendritic tributaries, such as the Yala and Daercang gullies. The Sejiang deforming slope is located on the left bank of the reservoir, 2.2 km apart from the dam. It extends approximately 270 m along the river, with a relative height difference of 120 m, spanning from an elevation of 2930 m at the slope toe to 3050 m at the crown. The normal pool level of the reservoir is designed at 2910 m. The reservoir area exhibits a deeply incised canyon landform featured by “wide-top and narrow-bottom” cross-section. The river channel meanders gently in a distinct “S” shape, flanked by massive mountain massifs and steep valley slopes (Figure 1c,d). Regional elevations generally range from 3800 to 4500 m. Apart from the slightly gentler wide-valley terraces situated above an elevation of 3050 m, the natural angles of the remaining slopes predominantly range from 40° to 55°. The exposed lithology is dominated by Yanshanian granite and Triassic metamorphic sandstone interbedded with slate. Due to the intense influence of faulting and folding, the attitude of the strata varies drastically, and the shallow subsurface of the rock mass exhibits pronounced weathering and unloading effects. Consequently, extensive colluvial–deluvial deposits, loose accumulations, and small-scale landslides have developed along the river, leading to an inherently poor natural stability.

Affected by the reservoir impoundment in early 2025, the left-bank slope of Sejiang Village experienced significant deformation. The type of rock mass exhibits a composite failure characteristic, comprising early toppling–fracturing and recent retrogressive sliding. Dense arcuate tension cracks and lower compressional reverse scarps were generated within the elevation of 2930–3050 m. The upper part is dominated by river-parallel tension cracks, with the trailing-edge cracks being arcuate and nearly fully interconnected, as shown in Figure 2a. These trailing-edge cracks span an elevation range of approximately 120 m and a longitudinal (upstream–downstream) distance of about 196 m, with an average width of 25 cm and an average vertical subsidence depth of 15 cm, as illustrated in Figure 2b,c. Due to the differential deformation rates across various profiles, the middle portion of the slope predominantly develops river-perpendicular cracks (Figure 2d). In contrast, the lower portion is mainly characterized by river-parallel cracks (Figure 2e), accompanied by a reverse scarp topography resulting from compressional resistance during sliding or the toppling of the underlying anti-dip stratified bedrock.

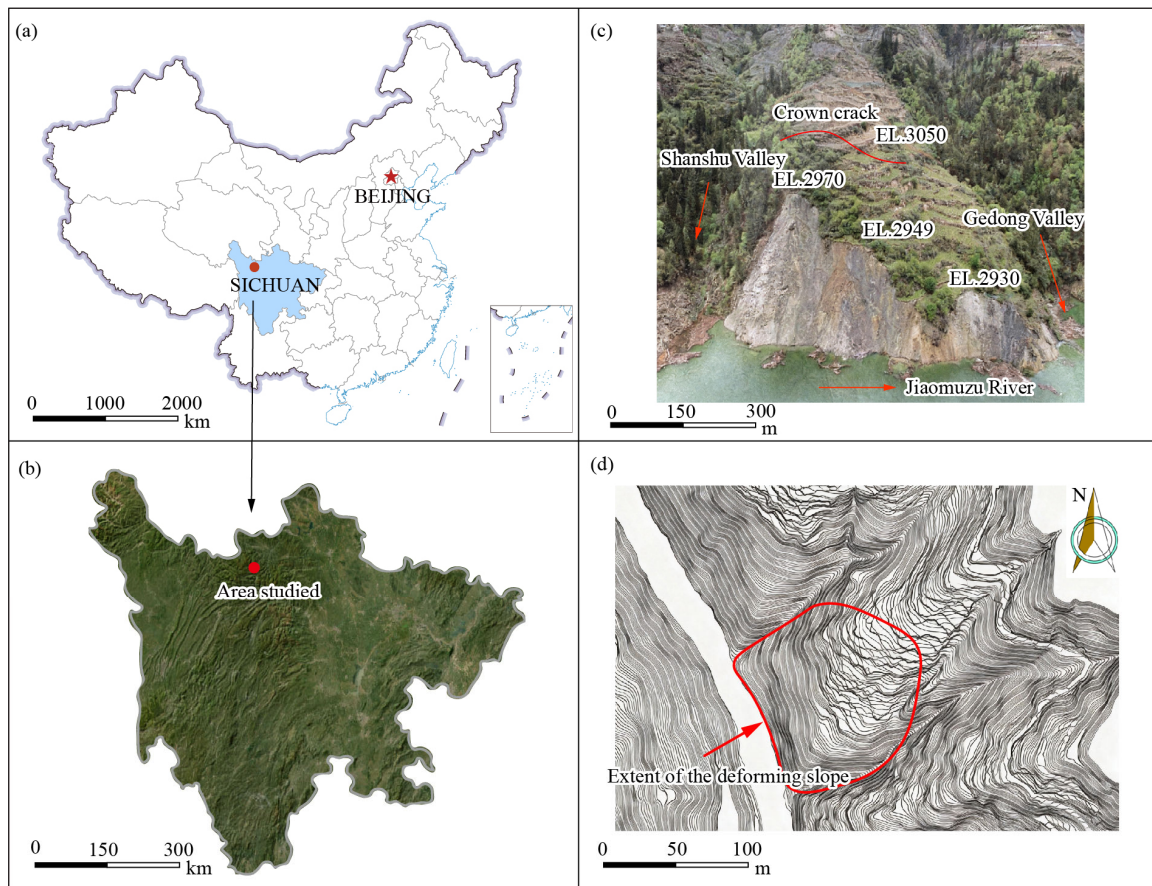


Figure 1. Geographical location and deformation characteristics of the Sejiang deforming slope: (a) location of Sichuan Province in China; (b) specific location of the study area in Sichuan Province; (c) overall geomorphology and deformation characteristics of the Sejiang deforming slope; and (d) spatial extent of the deforming slope and topographic contours.

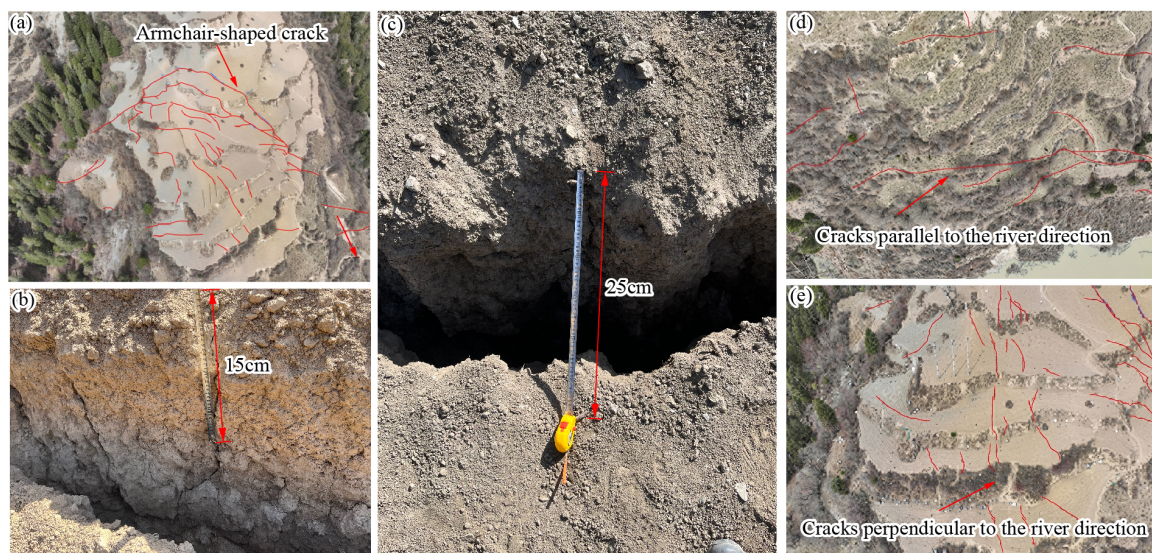


Figure 2. Plan view of crack distribution on the Sejiang deforming slope: (a) armchair-shaped tension cracks at the trailing edge; (b) vertical subsidence characteristics of the crack; (c) measurement of the crack opening width; (d) cracks parallel to the river direction; and (e) cracks perpendicular to the river direction. The red curves indicate the surface crack traces identified from UAV images and field investigations.

Furthermore, investigations of the surrounding rock outcrops reveal that the original attitude of the suspected bedrock on the upper upstream slope possesses a relatively steep dip angle, whereas the bedding planes of outcrops at the lower upstream slope toe are nearly horizontal. This comparison indicates that the slope rock mass had already undergone early-stage toppling and fracturing. Simultaneously, gently dipping intermittent structural planes inclined outward from the slope were identified at the toe of the downstream ridge, suggesting the potential existence of similar adverse structural planes within the interior of this deforming slope.

Figure 3 presents the typical engineering geological cross-section of the Sejiang deforming slope. This profile delineates the evolutionary sequence of the slope from the bedrock to the surficial overburden, clearly illustrating the ‘strongly toppling deformation zone’ and ‘weakly toppling zone’ developed within the underlying metamorphic sandstone intercalated with phyllite due to tectonic stress, as well as the deep bedrock inflection line induced by deep-seated toppling and unloading. The profile identifies the critical interface between the bedrock and the overlying Quaternary loose deposits, namely, the ‘bedrock–overburden interface’. This interface exhibits a significant propensity for shear instability under the influence of reservoir water level fluctuations and gravitational forces, thereby forming the potential sliding channel for the slope. Furthermore, the distribution of rear tension cracks and sliding zone soil depicted in the profile reflects the composite failure mechanism of the landslide, providing precise geometric parameters for determining the spatial distribution of the slip surface and establishing boundary conditions for subsequent three-dimensional numerical simulations.

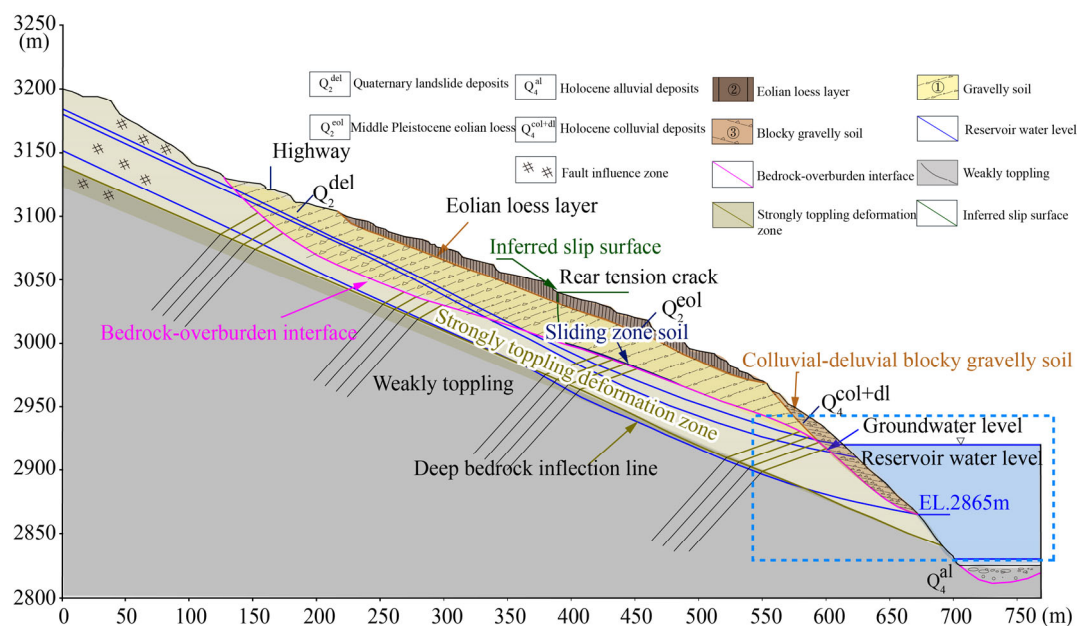


Figure 3. Engineering geological cross-section of the Sejiang deforming slope.

The potential failure surface of the Sejiang deforming slope was determined by integrating field geological mapping, crack distribution, UAV-based terrain interpretation, and the engineering geological cross-section. The rear tension cracks indicate the upper boundary of deformation, whereas the sliding-zone soil and the bedrock-overburden interface provide direct geological evidence for the basal sliding zone. In addition, the strongly toppling deformation zone developed in the underlying metamorphic sandstone interbedded with phyllite constrains the deep deformation boundary of the slope. These features jointly define the spatial position of the potential failure surface used in the numerical model.

3. Numerical Simulation of Impulse Waves

3.1. Establishment of the Three-Dimensional Numerical Model

The three-dimensional landslide–water interaction and impulse-wave propagation processes were simulated using FLOW-3D, a commercial CFD code developed by Flow Science, Inc. The software was adopted because it provides a VOF-based free-surface tracking method, a FAVOR-based geometric representation technique for complex terrain, and a granular material model suitable for simulating the landslide–water interaction. In the present study, the VOF method was used to capture the temporal evolution of the free water surface, the FAVOR technique was used to represent the canyon topography and solid boundaries, and the RNG turbulence model was adopted to describe the turbulent flow generated during rapid landslide entry and the subsequent wave propagation [9,15]. The selection of this CFD-based approach over alternative methods was guided by the specific requirements of the present engineering-scale assessment in a narrow canyon reservoir. Semi-empirical formulae, such as Pan’s method, offer a high computational efficiency but rely on rigid-body assumptions that cannot reproduce the granular deformation, differential acceleration, or the reflection and superposition effects induced by complex topography. Shallow-water equation models, though suitable for far-field wave propagation in wide basins, adopt depth-averaged assumptions that lose the vertical flow structure and are inadequate for accurately capturing near-field three-dimensional free-surface deformations and local wave amplification at river bends. Conversely, DEM-CFD coupled models can resolve particle–fluid interactions at high resolution; yet, their computational cost is prohibitive for the full reservoir domain of approximately $4.3 \text{ km} \times 3.3 \text{ km}$ considered in this study. The present CFD approach strikes a practical balance between physical fidelity and computational efficiency, enabling the explicit simulation of landslide fragmentation, water-entry dynamics, and the subsequent wave propagation while remaining tractable for engineering-scale hazard assessment. However, several limitations should be acknowledged, including the need for the careful calibration of granular flow parameters based on geological data and the absence of direct field validation, which will be addressed through future monitoring and physical model testing. Within these constraints, the adopted CFD method is considered the most appropriate for characterizing the spatiotemporal evolution of impulse waves and their topographic amplification effects in the complex canyon setting of the Bala Hydropower Station.

To simulate the most unfavorable engineering condition of slope instability, the landslide is prescribed to undergo global instability under the action of reservoir water. The boundaries and depths of the sliding face of the unstable rock mass is determined by geotechnical engineering investigations. Following impoundment, the water level in the reservoir area gradually rises from the normal pool level of 2910 m to the check flood level of 2918.5 m. To comprehensively grasp the impulse wave risks under varying water levels, a total of 11 characteristic water levels is established for the simulations.

A three-dimensional solid model was reconstructed based on the measured 1:2000 topographic elevation point cloud data. As illustrated in Figure 4, the computational domain spans a length of 4307.39 m, a width of 3290.14 m, and a height of 1030.82 m, which fully encompasses the area affected by the slope sliding and the primary region of wave propagation toward the dam. Furthermore, the boundaries are extended sufficiently far to eliminate artificial reflection interference. The landslide volume in the model is $1.14 \times 10^6 \text{ m}^3$, with an error of less than 0.5% compared to the field-measured values. Monitoring point A₁ is primarily located at the confluence of the landslide mass and the river. Monitoring point C₁ is established at the river bend, and monitoring point D₁ is set up in the dam area to monitor free surface elevation fluctuations across the dam body. In addition, monitoring points B₁ through B₈ are deployed alternately along both banks of

the river, with B_1 positioned on the bank opposite the landslide mass to collect data on free surface elevation fluctuations.

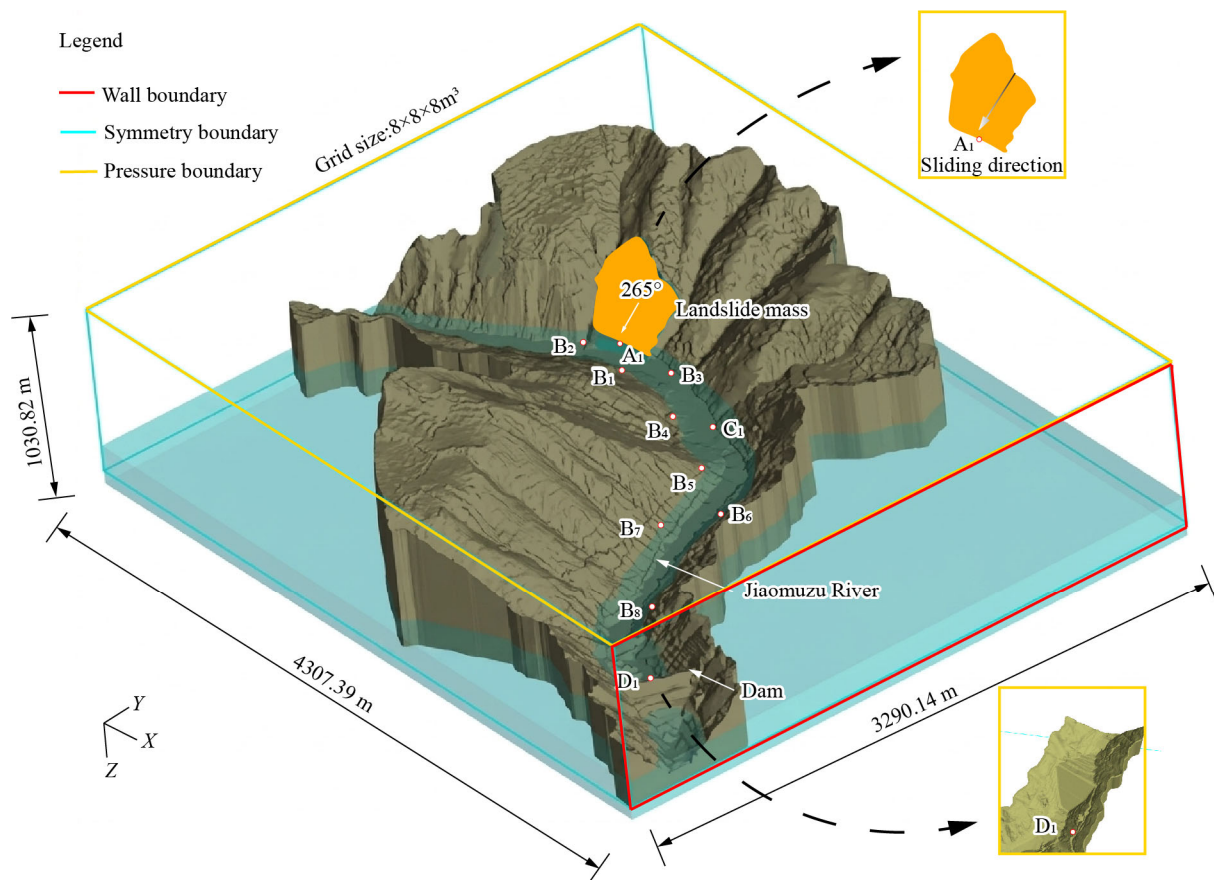


Figure 4. Schematic diagram of the Sejiang deforming slope model.

3.2. Parameter Settings

The landslide body utilizes the built-in granular material model of CFD, with its initial velocity set to 0 m/s to simulate the free sliding process along the slip surface under the action of gravity. The friction coefficient between the landslide mass and the slip surface is determined through conversion based on a 30-degree angle of repose. The coefficients of restitution for particle-to-particle and particle-to-boundary collisions adopt the software default values, rendering the entire movement process of the landslide completely gravity-driven.

The boundary and initial conditions are configured as follows: i.e., the bottom is designated as a wall boundary, the top as a pressure boundary, the downstream river outlet as an outflow boundary, and the remaining directions as symmetry boundaries. The gravitational acceleration is set as 9.81 m/s^2 , the initial flow velocity is 0 m/s, and the total computational duration is set as 600 s. Considering the influences of clay minerals and pore water adhering to the particle surfaces, the particle density is specified as 2600 kg/m^3 . Combined with the lithological characteristics of the Sejiang deforming slope, which consists of metamorphic sandstone interbedded with phyllite, as well as relevant engineering specifications, the friction angle and angle of repose are established at 32° and 30° , respectively. The fluid is specified as pure water at 1°C . The volume fractions for mechanical interference, tight packing, and loose packing that govern the movement of the landslide blocks are defined as 0.7, 0.6, and 0.5, respectively.

In the FLOW-3D granular flow model, the collision energy dissipation of granular material is represented by the grain restitution coefficient. Unlike DEM-based particle simulations, the Eulerian granular flow model adopted in this study does not separately

define particle–particle and particle–boundary restitution coefficients. The grain restitution coefficient controls the kinetic energy retained after granular interactions. A value close to 1.0 indicates weak energy dissipation, whereas a smaller value indicates stronger collision-induced energy loss. In this study, the grain restitution coefficient was set to 0.65 to represent the dissipative behavior of the wet fragmented rock–soil mixture in the Sejiang deforming slope. A summary of all key model parameters is provided in Table 1.

Table 1. Main parameters used in the CFD simulation.

Category	Parameter	Value Adopted in This Study	Basis or Physical Meaning
Computational domain	Domain size	4307.39 m × 3290.14 m × 1030.82 m	Determined from the reconstructed 1:2000 topographic elevation data
Landslide geometry	Landslide volume	$1.14 \times 10^6 \text{ m}^3$	Determined from engineering geological investigation and 3D terrain reconstruction
Initial condition	Initial velocity of landslide mass	0 m/s	The sliding process was assumed to be gravity-driven from an initially unstable state
Initial condition	Initial water velocity	0 m/s	Reservoir water was assumed to be initially still
Gravity	Gravitational acceleration	9.81 m/s^2	Standard gravitational acceleration
Landslide material	Particle density	2600 kg/m^3	Determined by considering the lithology and water-bearing broken rock–soil mixture
Landslide material	Friction angle	32°	Determined from the lithological characteristics of metamorphic sandstone interbedded with phyllite and relevant engineering references
Landslide material	Angle of repose	30°	Used to convert the basal friction coefficient of the sliding mass
Collision parameter	Grain restitution coefficient	$e_g = 0.65$	Controls collision-induced energy dissipation during granular interactions in the FLOW-3D granular flow model
Packing parameter	Mechanical interference volume fraction	0.7	Controls the dense granular motion state of the landslide mass
Packing parameter	Tight packing volume fraction	0.6	Used to characterize compact packing of granular material
Packing parameter	Loose packing volume fraction	0.5	Used to characterize loose packing of granular material
Fluid property	Fluid type	Pure water	Reservoir water was simplified as incompressible pure water
Fluid property	Water temperature	1°C	Set according to the low-temperature reservoir environment
Water level condition	Reservoir water level	2910–2918.5 m	Eleven characteristic water levels between the normal pool level and check flood level were considered
Boundary condition	Bottom boundary	Wall boundary	Represents the impermeable solid terrain boundary
Boundary condition	Top boundary	Pressure boundary	Represents atmospheric pressure condition
Boundary condition	Downstream outlet	Outflow boundary	Allows wave propagation toward the downstream outlet

Table 1. *Cont.*

Category	Parameter	Value Adopted in This Study	Basis or Physical Meaning
Boundary condition	Other lateral boundaries	Symmetry boundary	Reduces artificial lateral constraints in the extended computational domain
Mesh setting	Mesh size	8 m	Selected based on grid-independence analysis
Simulation time	Total computational duration	600 s	Ensures full coverage of wave generation, propagation, reflection, and attenuation

The mechanical and granular-flow parameters of the landslide body were determined from the engineering geological characterization of the slope. The unstable mass is mainly composed of Quaternary loose deposits, colluvial–deluvial blocky gravelly soil, aeolian loess, and intensely fractured metamorphic sandstone interbedded with phyllite. The adopted particle density, friction angle, angle of repose, and packing parameters were determined by considering lithological composition, fragmentation degree, water-bearing state, and engineering geological analogy with similar reservoir-bank landslides.

CFD utilizes the Fractional Area/Volume Obstacle Representation (FAVOR) technique for grid meshing. This method characterizes complex geometric shapes through rectangular grid blocks, thereby efficiently defining the geometric boundaries of the model. To optimize computational efficiency, strategies including multi-mesh block coupling, local refinement via nested meshes, and the addition of mesh planes were employed [20]. A grid independence test was performed using three mesh sizes (5 m, 8 m, and 10 m) under the identical scenario to compare the peak wave elevations and their corresponding occurrence times at the monitoring points. The results demonstrate that the differences in peak wave elevation and peak arrival time between the 8 m and 5 m grids are less than 2.1% and 0.5 s, respectively. Conversely, the discrepancy between the 10 m and 8 m grids reaches 7.6%. Comprehensively weighing the computational accuracy against efficiency, the 8 m grid was ultimately selected for subsequent analyses.

It should be noted that the direct field validation of the CFD model is not currently available because the Sejiang deforming slope has not yet experienced a complete reservoir-entry collapse. Therefore, no measured impulse-wave time history can be used for a direct comparison. In this study, the reliability of the numerical model was evaluated through mesh-independence verification, a comparison with Pan’s semi-empirical analytical method, and an examination of whether the simulated wave propagation pattern is physically consistent with the expected processes of near-source run-up, downstream attenuation, bend-induced reflection, and local wave-energy convergence. Future work should incorporate physical model tests or field monitoring data, including high-frequency water-level gauges, wave pressure sensors, UAV photogrammetry, and slope deformation monitoring, to further calibrate and validate the CFD model.

Several modelling simplifications should be recognized when interpreting the predicted results. First, the Sejiang deforming slope was simulated under a global-instability scenario, which represents an unfavorable engineering condition and may overestimate the peak wave height compared with progressive failure. Second, the failure surface was prescribed based on geological interpretation, with uncertainty remaining in its depth, continuity, and local roughness. Third, the Eulerian granular-flow model does not explicitly resolve particle-scale fragmentation or detailed particle–particle collisions as in DEM-based models. Fourth, the riverbed and bank boundaries were treated as fixed solid boundaries, whereas local erosion and bank deformation may occur in reality. Fifth, the reservoir water was assumed to be initially still, neglecting background currents and operational

fluctuations. Finally, direct field validation is not currently available because the slope has not yet experienced a complete failure event. Therefore, the predicted wave heights should be interpreted as scenario-based estimates for the assumed global-instability condition rather than deterministic values for all possible failure processes.

4. Analysis of Slope Sliding Movement and Impulse Wave Propagation

4.1. Water-Entry Dynamic Characteristics of Slope Sliding

Figure 5 illustrates the spatiotemporal evolution of the velocity fields for both the sliding mass and the water body during the transition from the unstable sliding to water-entry impact of the Sejiang deforming slope under two distinct initial water levels (2910 m and 2918.5 m), revealing that both conditions exhibit similar underlying evolutionary patterns.

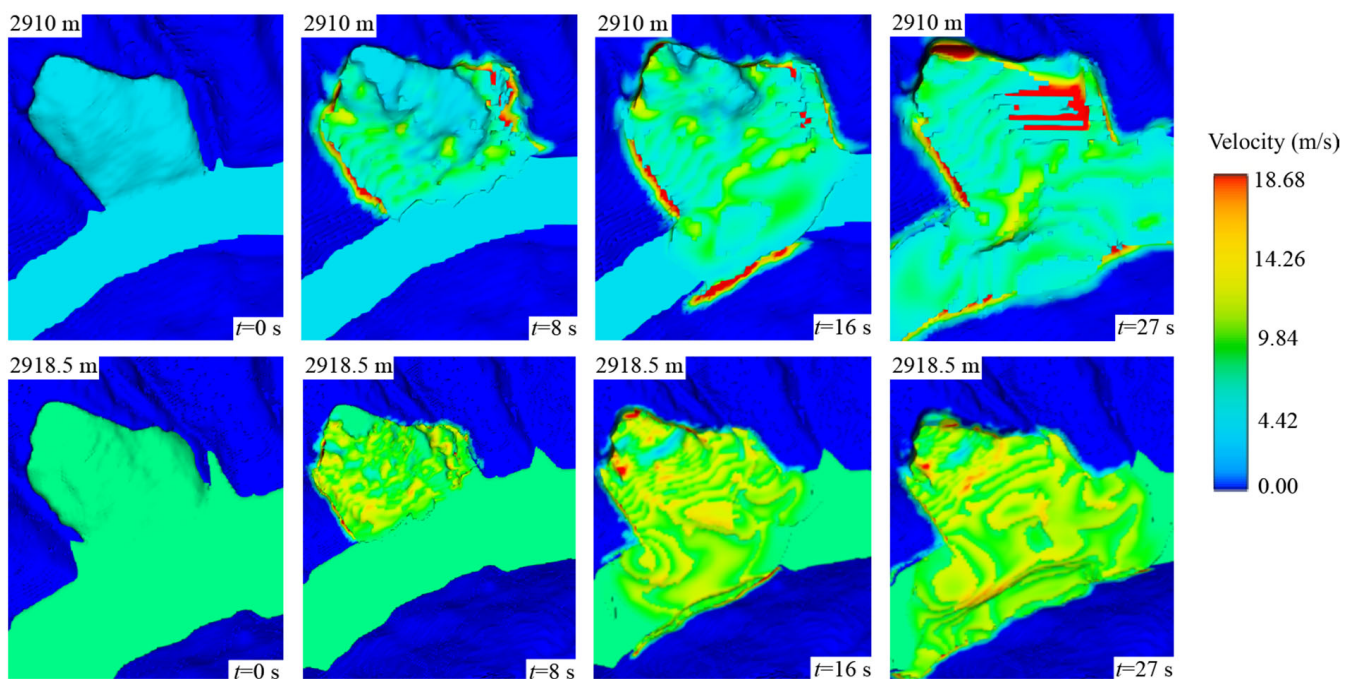


Figure 5. Sliding mass movement and impulse wave generation process at water levels of 2910 m and 2918.5 m.

At time $t = 0$ s, the deformation body is in a critical state of instability with an overall velocity of 0 m/s. By $t = 8$ s, as the shear failure develops, the sliding mass begins to propagate downward. Under the 2910 m water level condition, the material along the lateral edges undergoes preferential shear failure and accelerates due to the shallow burial depth of the slip zone and weak lateral constraints, reaching a localized peak velocity of 18 m/s, while the central mass lags behind owing to the overlying loads and compaction effects. Under the 2918.5 m condition, the higher water level's water-cushion effect causes the leading edge to contact the water earlier, resulting in more extensive compression and disturbance features at the front and a wider lateral shear acceleration zone. At $t = 16$ s, the main body enters the water with a high velocity, intensifying the fluid–structure interaction. Specifically, under the 2910 m condition, the leading edge violently impacts the channel, exerting intense drag and displacement that induces a prominent band-like velocity response downstream and toward the opposite bank, rapidly expanding the impact crater and initial impulse waves. In contrast, under the 2918.5 m condition, the greater water depth leads to more regular, wave-like stratified alternating bands within the slide velocity field, indicating more complex feedback inhibition and energy dissipation mechanisms. Finally, at $t = 27$ s, the water entry of the main slide mass is substantially complete for both

scenarios, generating impulse waves characterized by non-uniform velocities in the vicinity of the impact zone.

Figure 6 presents a schematic model of the landslide–water-entry dynamic process. To ensure consistency with the engineering geological cross-section of the Sejiang deforming slope, the geometric characteristics of the slip surface and the stratigraphic boundaries in this figure are strictly based on the geological interpretation shown in Figure 3. The minor discrepancies in surface morphology reflect the post-failure state simulated by the numerical model, with a focus on illustrating the mass redistribution and water-entry effects following slope instability.

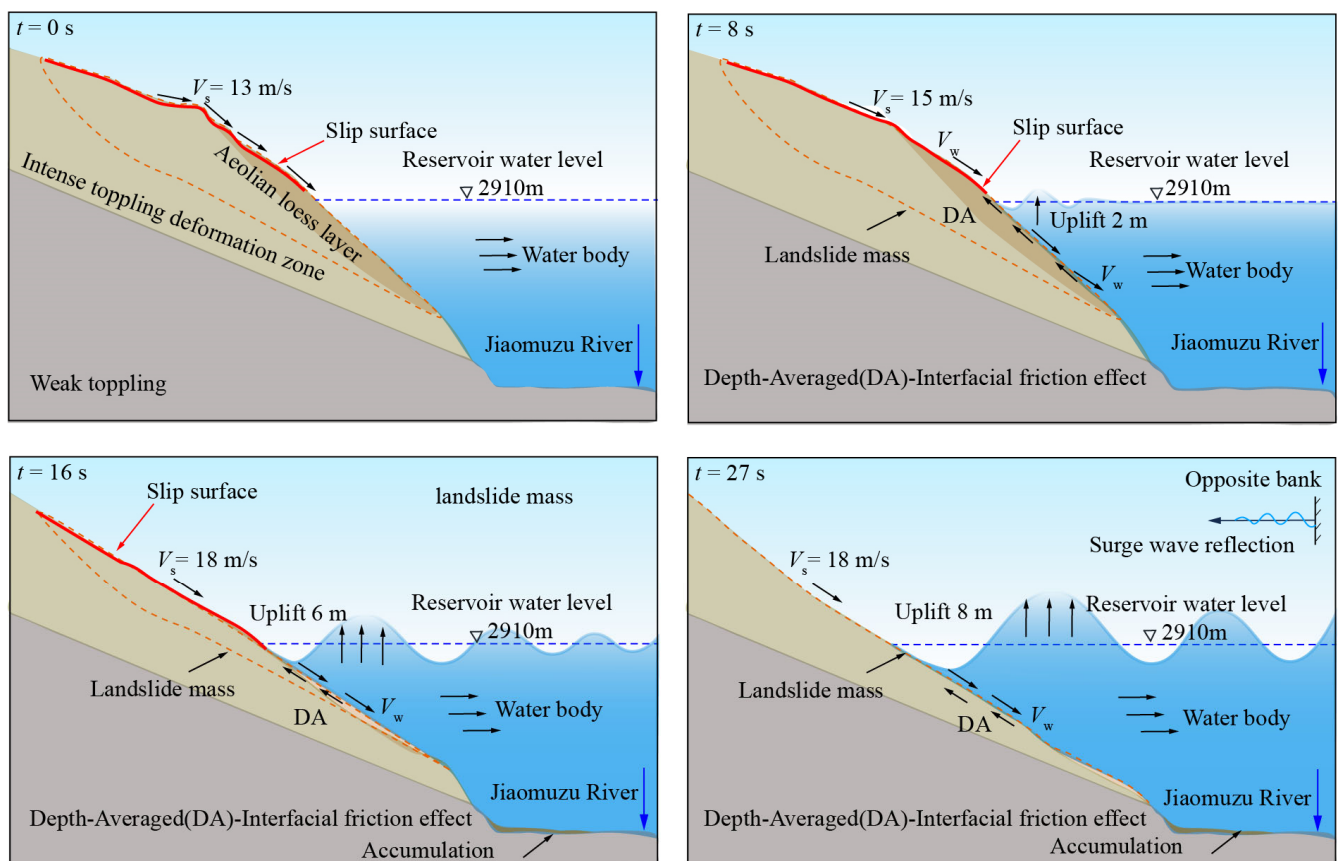


Figure 6. Schematic conceptual model of the landslide–water-entry mechanism and interfacial friction effect (based on the geological structure defined in Figure 3).

As shown in Figure 6, the process illustrates the interfacial effect of a landslide mass entering the water. At $t = 0$ s, the sliding mass, primarily composed of an Aeolian Loess Layer overlying an intense toppling deformation zone, initiates failure. The basal slip surface forms above the weak toppling zone. By $t = 8$ s, as the landslide material penetrates the water body, the sliding velocity accelerates to 15 m/s. The intrusion of the landslide mass triggers fluctuations in the water surface, resulting in an initial free-surface uplift of 2 m. This stage marks the onset of the depth-averaged (DA) interfacial friction effect. At $t = 16$ s, the velocity of the submerged landslide mass reaches 18 m/s. The continuous displacement and momentum transfer significantly amplify the surge wave, leading to a local water uplift of 6 m. Simultaneously, the leading edge of the sliding mass begins to form a subaqueous accumulation zone at the riverbed. By $t = 27$ s, the kinetic energy transferred to the water body generates a maximum surge uplift of approximately 8 m. Upon impacting the opposite bank, the surge wave undergoes reflection.

4.2. Visual Analysis of Swell Impact Areas

Figures 7 and 8 illustrate the three-dimensional visualization results under two initial water level scenarios of 2910 m and 2918.5 m, respectively. Following the initiation of the landslide mass along the slip surface and its high-speed impact into the water body, the massive initial kinetic energy is instantaneously converted into the potential and kinetic energy of the water mass. This process generates violent initial impulse waves at the impact zone, which rapidly propagate outward in a radial pattern. The spatiotemporal evolution reveals that the wave propagation dynamics are governed by profound boundary constraint effects. Shortly after the landslide occurrence, the leading wave directly impacts the opposite bank, triggering a drastic, nonlinear run-up of the free water surface in this region. Given its close proximity to the landslide source zone, this area experiences the earliest wave impact with negligible propagation dissipation, allowing the initial run-up amplitude to reach its peak. Subsequently, the waves undergo intense reflection against the high and steep banks, resulting in the dispersion of wave energy toward both upstream and downstream directions.

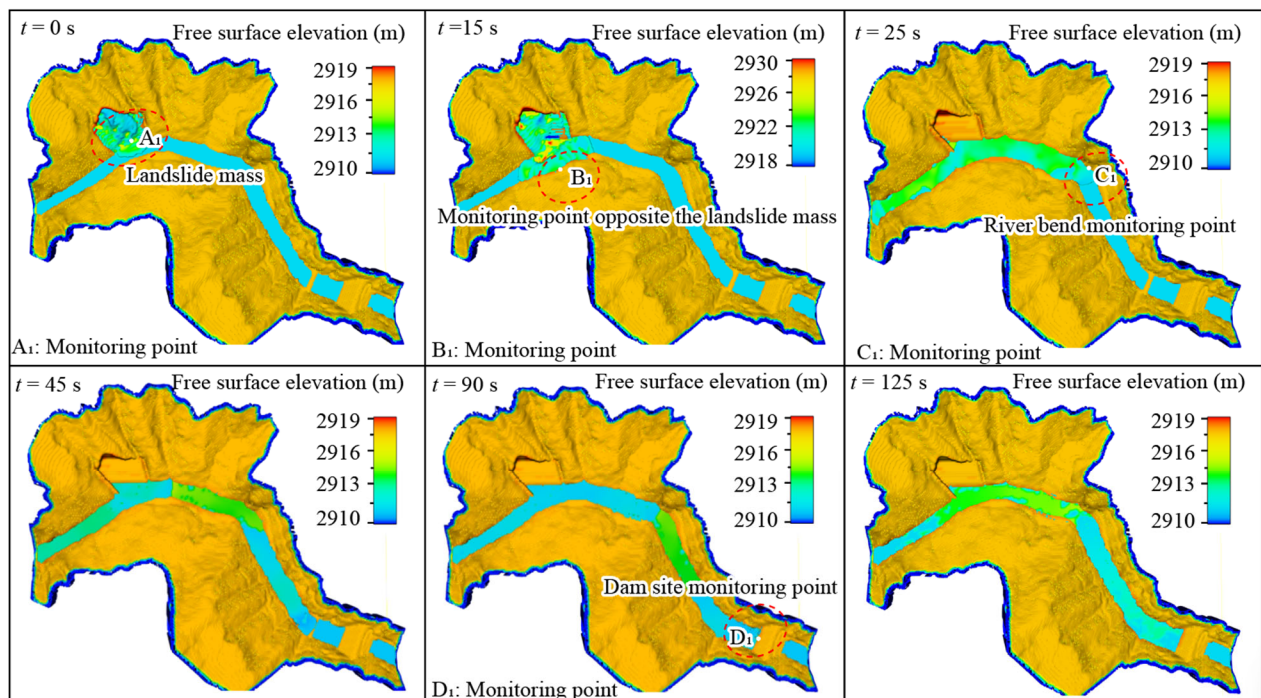


Figure 7. Free surface elevations of waves generated by the Sejiang deforming mass at water level 2910 m.

As the waves propagate towards the spoil disposal area, the abrupt contraction of the river channel topography impedes the rapid dissipation of wave energy, making the region highly susceptible to the superposition effects of multi-directional reflected waves. Concurrently, the accumulations in the spoil disposal area predominantly consist of loose, porous media. Under the continuous cyclic scouring of the waves and the periodic hydrodynamic pressures, these deposits are prone to local or global seepage instability. This poses a potential risk of triggering secondary landslides or debris flows. Therefore, this region must be designated as a critical prevention zone in engineering practice.

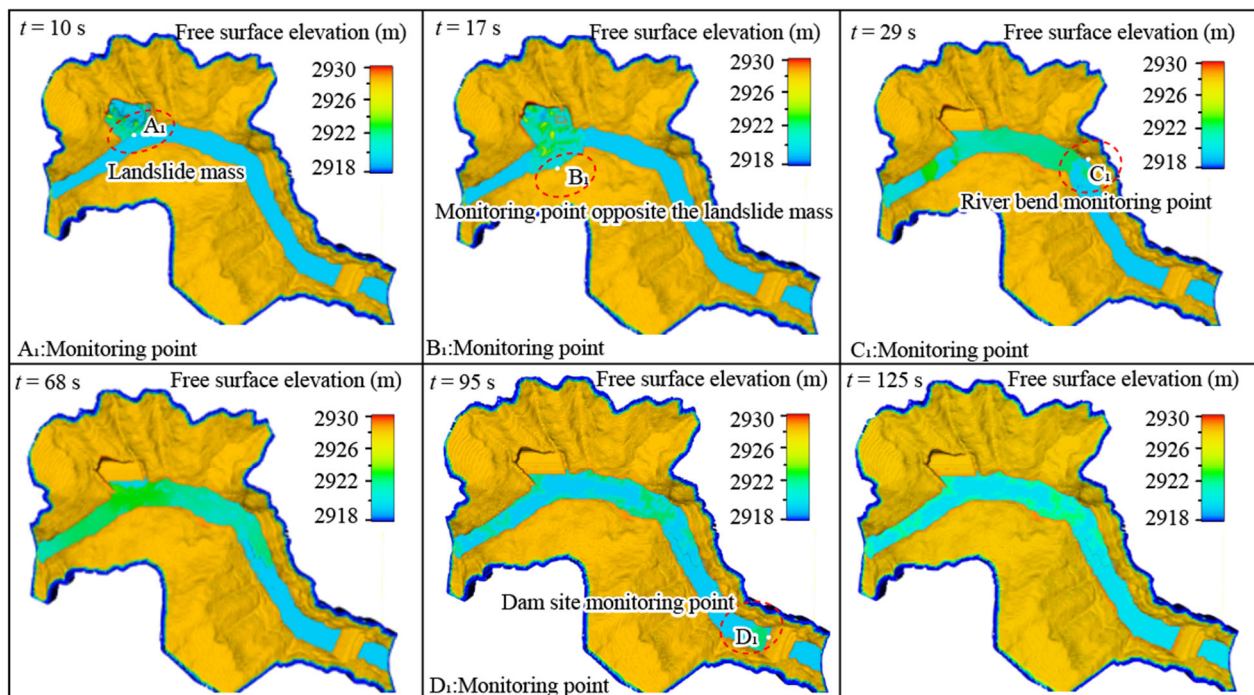


Figure 8. Free surface elevations of waves generated by the Sejiang deforming mass at water level 2918.5 m.

Furthermore, a comparison between Figures 7 and 8 indicates that the initial water level significantly influences the phase velocity and amplitude evolution patterns of the propagating waves. The main wave train continuously advances downstream along the main channel, with the wave front reaching the vicinity of the dam at $t = 66.5$ s. Although the simulation results indicate that the maximum wave crests under both water level scenarios do not cause dam overtopping, the confinement imposed by the local valley boundaries induces an energy convergence effect on the upstream face of the dam, leading to a secondary wave peak in the local water surface elevation. Such high-frequency, localized transient hydrodynamic loads pose a severe threat to the auxiliary facilities of the dam. Therefore, it is imperative to further enhance the wave resistance and structural reinforcement designs for auxiliary structures and equipment in the high-risk zones along the dam shoreline.

4.3. Spatiotemporal Evolution Analysis of Swell Propagation

The results at monitoring points B_1 , C_1 , and D_1 are utilized to comprehensively analyze the impact patterns of landslide-generated impulse waves on the surrounding buildings and key structures under different reservoir water level scenarios. Figure 9a,b clearly present the time-history curves of the free-surface elevation at each monitoring point under the two water level conditions. These curves show the reflection, superposition, and energy attenuation effects during wave propagation, as well as the significant sensitivity of the waves to the reservoir water level.

For monitoring point B_1 on the opposite bank in the near-source region, the initial wave run-up is the most drastic, serving as a direct manifestation of the transient conversion of the landslide's kinetic energy into the water body's potential energy. At the normal pool level (2910 m), the peak wave elevation is 2915.57 m occurring at $t = 13.25$ s. Conversely, when the water level rises to the check flood level (2918.5 m), the peak wave elevation rapidly rises to 2925.36 m, and the peak arrival time advances to $t = 12.55$ s. Clearly, the increased baseline water depth significantly enhances the wave velocity. The initial

fluctuation amplitudes at point B₁ under both scenarios are significantly larger than those at other monitoring points. Thereafter, as the wave energy dissipates outward, the amplitude gradually attenuates and transitions into multi-cycle oscillations, eventually decaying to a stable water level within 100–250 s.

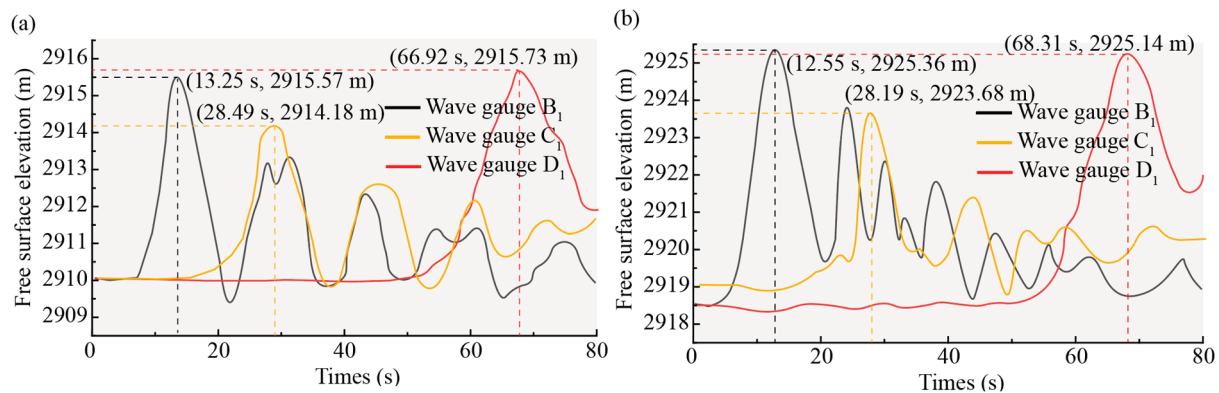


Figure 9. Positions of surge wave crest points at points B, C, and D: (a) at water level 2910 m, and (b) at water level 2918.5 m.

When the impulse wave propagates along the river channel to monitoring point C₁ at the spoil disposal area, the initial wave energy has undergone preliminary attenuation due to geometric spreading and frictional resistance along the path. At the 2910 m water level, the peak elevation of the first round of water level fluctuation is 2914.18 m ($t = 28.49$ s), whereas the peak elevation is 2923.68 m ($t = 28.19$ s) at the 2918.5 m water level. By comparison, the wave peaks under both scenarios exhibit a distinct phase lag relative to point B₁. Strongly constrained by the narrow and meandering topography in this area, the impulse waves undergo multiple boundary reflections and superpositions, forming prolonged secondary wave trains characterized by significant periodicity and relatively gentle overall amplitudes.

Monitoring point D₁ at the dam site mainly reflects the repeated reflection and convergence effects of the complex reservoir topography on the impulse waves. At the 2910 m and 2918.5 m water levels, the peak wave elevations in the dam area are 2915.73 m ($t = 66.92$ s) and 2925.14 m ($t = 68.31$ s), respectively. Point D₁ is the area with the latest peak arrival time among all monitoring points, revealing the secondary energy convergence effect after several bend reflections in the narrow and meandering river channel. Under the high-water-level scenario (2918.5 m), this localized surge in wave height triggered by multiple superpositions is even more pronounced, raising a higher safety warning for the wave-resistant design of the dam and its appurtenant structures.

5. Discussion

The preceding numerical simulations based on CFD have intuitively revealed the spatiotemporal evolution patterns of the impulse waves (spanning their generation, propagation, and attenuation) along with their impacts on critical zones within the reservoir. To further assess the disaster risks and provide a multidimensional reference for engineering protection strategies, empirical formula methods are concurrently employed for a comprehensive evaluation. Consequently, Pan's empirical formula method is applied to calculate the impulse wave heights under the global sliding scenario of the landslide mass. Building upon this, a comparative analysis against the numerical simulation results is conducted to explicitly elucidate the discrepancies and complementary roles of the two approaches.

5.1. Calculation Results and Water Level Sensitivity of Pan's Semi-Empirical Analytical Method

Pan's empirical formula method utilizes the analytical results of unidirectional reservoir flow and adopts fundamental assumptions to resolve complex reservoir impulse wave problems [12]. For the Sejiang deforming slope, the actual water depth (h) in the corresponding reservoir area is 85 m. Based on the engineering geological profile (as shown in Figure 3), the parameters are set as follows: i.e., $H = 32.5$ m, $f = 0.32$, and $\theta = 32^\circ$. Consequently, the equivalent average water-entry velocity of the landslide mass (v) is calculated to be 8.75 m/s. The calculated equivalent water-entry velocity of 8.75 m/s was used as a conservative design input for preliminary hazard screening, because Pan's semi-empirical analytical method simplifies the sliding mass as an equivalent rigid body and, therefore, tends to provide an upper-bound estimate of impulse-wave height.

Pan's method is referred to in this study as a semi-empirical analytical method rather than a purely empirical formula. This is because the method combines simplified analytical derivation for landslide-induced wave generation with empirical coefficients calibrated from engineering practice. The main assumptions are that the landslide mass can be simplified as an equivalent rigid sliding body and that the impulse wave propagation in the reservoir can be approximated using a simplified one-dimensional wave-propagation framework. In this study, the main empirical or semi-empirical parameters include the effective falling height $H = 32.5$ m, sliding friction coefficient $f = 0.32$, sliding surface inclination $\theta = 32^\circ$, local water depth $h = 85$ m, valley width $B = 256$ m, landslide half-width $l = 130$ m, reflection coefficient $k = 0.85$, and sliding duration $t = 27.37$ s. These parameters were determined based on the engineering geological profile, reservoir geometry, and commonly used engineering values.

Other key parameters include a valley width (B) of 256 m, a reflection coefficient (k) of 0.85, a landslide half-width (l) of 130 m, and a sliding duration of 27.37 s. By substituting the sliding velocity of the Sejiang slope sliding mass into the corresponding formulae [11], the horizontal and vertical impulse wave heights are determined. Based on empirical formula methods, the initial wave height (ζ_0) generated by the landslide mass at the impact point is calculated to be 21.38 m. The constants used in Pan's empirical formula method for calculating the impulse waves propagating downstream along the opposite bank are as follows:

$$c = \sqrt{gh} \cdot \sqrt{1 + (\zeta_0/h) + 0.5(\zeta_0^2/h^2)} \quad (1)$$

$$\Delta t = L_B/c \quad (2)$$

$$\lambda = T/\Delta t \quad (3)$$

where ζ_0 is the initial wave height generated by the landslide mass at the impact point (m); c is the wave velocity (m/s); Δt is the time required for the wave to propagate the distance L_B to monitoring point B_1 (s); and λ is the ratio of the landslide duration T to Δt .

The formula for calculating the maximum wave height on the opposite bank induced by the landslide plunging into the reservoir is expressed as follows:

$$\zeta_{\max} = \frac{2\zeta_0(1+k)}{\pi} \sum_{i=1}^{\infty} k^{i-1} \cdot \ln \left\{ \frac{l}{(2i-1)B} + \sqrt{1 + \left[\frac{l}{(2i-1)B} \right]^2} \right\} \quad (4)$$

where i takes all positive integers (corresponding to the wave order number); k is the reflection coefficient (typically $0 < k < 1$); l is the half-width of the landslide mass; and B is the valley width.

The wave velocity c calculated from Equation (1) is 32.71 m/s; and $\lambda = 3.5$ is obtained with Equations (2) and (3). According to the standard procedures of Pan's empirical

formula method, two terms are retained for the summation series in Equation (4), yielding a maximum impulse wave height of 13.60 m propagating to the opposite bank.

Heller and Spinneken [14] modified and derived the calculation formula for the propagating wave height by integrating the mechanical analysis with physical model experiments, which is expressed as follows:

$$H_r/h = (H_{\max}/h) \cdot \exp[-0.4(x/h)^{0.35}] \quad (5)$$

where H_r is the impulse wave height at a specific location along the river channel (m); $H_{\max}$ is the wave height at the landslide impact point (m); x is the distance from a specific location along the propagation path to the impact point (m); and h is the maximum water depth at the impact point (m).

Calculations are performed for the slip surface illustrated in Figure 3 using the aforementioned Pan's empirical formula method. To evaluate the influence of reservoir water level fluctuations on the scale of the impulse waves, extended calculations were conducted under three typical water level scenarios while keeping the slip zone parameters and landslide volume as constant. The results are summarized in Table 2. Obviously, both the initial wave height and the subsequent impulse wave height increase as the water level rises. Specifically, the wave height on the opposite bank under the check flood level is approximately 3.4% higher than that under the normal pool level. This trend indicates that, although the landslide sliding velocity slightly decreases under high-water-level conditions, the enhancement effect of the increased water depth on the wave propagation efficiency plays a dominant role, thereby exacerbating the destructive potential of the impulse waves. Consequently, engineering protection designs should adopt the check flood level as the upper-bound envelope. The wave propagation patterns revealed by the refined simulations, conducted at the baseline normal pool level in this study, are universally applicable across different water level scenarios, and specific wave heights can be proportionally scaled based on Table 2.

Table 2. Calculation results of impulse waves under different water level scenarios.

Calculation Scenario	Water Level (m)	Initial Wave Height (m)	Water Depth (m)	Landslide Half-Width (m)	Water Surface Width (m)	Volume of Deforming Mass ($\times 10^4 \text{ m}^3$)	Initial Sliding Velocity (m/s)	Max. Wave Height on Opposite Bank (m)	Wave Height at Spoil Disposal Area (m)	Wave Height at Dam Site (m)
Dead water level	2880.0	18.64	60.0	130	256	114	9.64	11.83	7.02	5.24
Normal water level	2910.0	21.38	85.0	130	256	114	8.75	13.60	8.25	6.13
Check flood level	2918.5	22.15	93.5	130	256	114	8.12	14.06	8.30	6.42

The results in Table 2 indicate that the maximum wave height increases with the reservoir water level. However, this trend should not be interpreted as a strictly linear relationship. The wave height is jointly controlled by the water depth, landslide water-entry position, sliding velocity, valley width, and wave reflection. As the reservoir level rises, the increased water depth tends to enhance the wave propagation efficiency, whereas the equivalent sliding velocity may slightly decrease because of the modified water-entry condition. Therefore, the relationship between the reservoir water level and maximum wave height is expected to be weakly nonlinear. Since only three representative water-level scenarios were evaluated in this study, a continuous fitted curve was not introduced to avoid overinterpretation. Instead, the check flood level was adopted as the upper-bound water-level condition for engineering hazard assessment.

5.2. Comparison Between Numerical Simulation and Empirical Formula

Based on the water level sensitivity patterns obtained via the empirical formula method, a comparative analysis between the numerical simulation results and the empirical formula predictions is further conducted under identical scenarios to explicitly clarify the discrepancies and complementary roles of the two methods. When the Sejiang deforming slope slides globally along the slip zone, the impulse wave crests in front of the dam calculated by both methods do not exceed the dam crest elevation of 3000.00 m, indicating no risk of dam overtopping, as summarized in Table 3.

Table 3. Comparison of impulse wave heights between empirical formula and numerical simulation.

Calculation Method	Calculation Water Level (m)	Calculation Water Depth (m)	Water-Entry Velocity of Slide Mass (m/s)	Wave Height on Opposite Bank (m)	Corresponding Water Level on Opposite Bank (m)	Wave Height at Spoil Disposal Area (m)	Corresponding Water Level at Spoil Disposal Area (m)	Wave Height at Dam Site (m)	Corresponding Water Level at Dam Site (m)
Empirical Formula	2910.00	85.0	8.75	13.60	2923.60	8.25	2918.25	6.13	2916.13
	2918.50	93.5	8.12	14.06	2932.56	8.30	2926.80	6.42	2924.92
Numerical Simulation	2910.00	85.0	18.00	5.57	2915.57	4.18	2914.18	5.73	2915.73
	2918.50	93.5	17.51	6.86	2925.36	5.18	2923.68	6.64	2925.14

(1) Comparison of landslide water-entry velocity

The “water-entry velocity” column in Table 3 possesses distinct physical connotations for the two methods. The empirical formula method yields an equivalent average water-entry velocity of 8.75 m/s, which serves as a representative value for engineering design derived from the rigid-body block assumption and accounts for the basal frictional resistance effect. Conversely, the 18 m/s obtained from the numerical simulation represents the instantaneous peak velocity during the landslide movement, reflecting the authentic dynamic characteristics of non-uniform initiation and localized kinetic energy concentration of the granular material.

(2) Comparison of impulse wave heights

The maximum impulse wave height on the opposite bank calculated by the empirical formula method (13.60 m) is significantly larger than the numerical simulation result (5.57 m), reflecting a safety margin of approximately 144% brought by the monolithic block assumption and conservative parameter selections. This conservatism aligns with the positioning of risk screening during the preliminary design stage. The numerical simulation results indicate that the peak wave height in the dam area after superposition through channel bends (5.73 m) is slightly higher than the first peak of the incident wave (5.57 m). In the spoil disposal area, which is more strongly constrained by the narrow river channel topography, the superimposed wave height on the opposite bank (4.18 m) is approximately 156% higher than the theoretical attenuated wave height without bends (1.63 m), demonstrating the prominent energy convergence and amplification effects induced by narrow river channel bends. The numerical results show the reflection and superposition of impulse waves at the river bends, which are difficult to represent using the one-dimensional semi-empirical method. Although the empirical method fails to intuitively reflect the impact of channel bends on impulse waves, it accounts for the frictional resistance along the slip surface post-instability as well as the surface friction of the mass deposited in the reservoir. Comparatively, the water-entry velocity and resulting wave heights calculated by the empirical method lean toward being conservative.

In summary, the two methods yield consistent and complementary conclusions regarding the common objective of impulse wave risk assessment. Pan's empirical formula

method provides a conservative safety baseline, while the numerical simulation reveals refined hazard characteristics under real-world topographic conditions. Comparatively, the wave heights calculated by Pan's empirical method are more conservative, making them highly suitable for the envelope design in engineering safety evaluations.

To further quantify the difference between the two methods, the absolute difference, relative difference, and RMSE were calculated using the CFD results as the reference. As shown in Table 4, Pan's semi-empirical analytical method gives considerably larger wave heights at the opposite bank and spoil disposal area, indicating its conservative nature in the near-field impact and intermediate propagation zones. In contrast, the relative differences at the dam site are much smaller, being 6.98% and 3.31% under the normal pool level and check flood level, respectively. Based on the six comparison points listed in Table 4, the RMSE between Pan's method and the CFD simulation is 4.88 m. These results indicate that Pan's method is more suitable for rapid preliminary screening and conservative envelope estimation, whereas the CFD approach is more appropriate for the refined assessment of local wave reflection, superposition, and topographic amplification.

$$RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^n (H_{Pan,i} - H_{CFD,i})^2} \quad (6)$$

Table 4. Quantitative comparison between Pan's semi-empirical analytical method and CFD simulation.

Water Level (m)	Location	Pan's Method (m)	CFD Simulation (m)	Absolute Difference (m)	Relative Difference (%)
2910.0	Opposite bank	13.6	5.57	8.03	144.17
2910.0	Spoil disposal area	8.25	4.18	4.07	97.37
2910.0	Dam site	6.13	5.73	0.40	6.98
2918.5	Opposite bank	14.06	6.86	7.20	104.96
2918.5	Spoil disposal area	8.30	5.18	3.12	60.23
2918.5	Dam site	6.42	6.64	0.22	3.31

5.3. Influence of Failure Mechanism and Model Uncertainty

To evaluate the robustness of the predicted wave heights and quantify the influence of key input parameters, a sensitivity analysis was conducted on four critical parameters: slide impact velocity (V_s), slide volume (V), slide thickness (T), and still water depth (h). For each parameter, the value was varied within a range of $\pm 20\%$ around its baseline value while keeping all other parameters unchanged. The variation ranges were determined based on geological investigation results and literature review.

The average sensitivity coefficient S_{X_i} was adopted to quantify the sensitivity of wave height to each parameter:

$$S_{X_i} = \frac{1}{n} \sum_{j=1}^n \left| \frac{\Delta H_j / H_0}{\Delta X_i / X_0} \right| \quad (7)$$

where H_0 and X_0 are the baseline wave height and baseline parameter value, respectively, and ΔH_j and ΔX_i are the corresponding variations.

The calculated sensitivity coefficients for each parameter are summarized in Table 5. Among all parameters, the slide volume exhibits the highest sensitivity coefficient (approximately 0.72), followed by the slide impact velocity (approximately 0.65), indicating that these two parameters have the most significant control over impulse wave height. Previous studies have also shown that the sensitivity coefficient of wave height to volume changes is approximately 0.858, compared to 0.217 for velocity changes [14,16], which is consistent

with our results. The sensitivity coefficients for slide thickness and still water depth are relatively lower, at approximately 0.35 and 0.28, respectively.

Table 5. Sensitivity analysis results of key parameters.

Parameter	Symbol	Baseline Value	Variation Range	Sensitivity Coefficient S
Slide impact velocity	V_s	8.75 m/s	$\pm 20\%$	0.65
Slide volume	V	$1.14 \times 10^6 \text{ m}^3$	$\pm 20\%$	0.72
Slide thickness	T	32.5 m	$\pm 20\%$	0.35
Still water depth	h	85 m	$\pm 20\%$	0.28

The dominance of the slide volume and impact velocity can be explained as follows: the slide volume directly determines the total momentum transferred to the water body upon water entry, while the impact velocity controls the rate of momentum transfer and the instantaneous impact intensity. The influence of the still water depth on the wave height is relatively complex, exhibiting different effects in the near-field and far-field regions.

The sensitivity analysis results indicate that, in practical engineering applications, the accurate determination of the slide volume and impact velocity should be given priority. For potential landslides with a large volume and high impact velocity, the impulse wave risk should not be overlooked even if other parameters contain certain uncertainties. In this study, the most unfavorable scenario (instantaneous global collapse) was adopted for wave prediction, and the obtained wave heights serve as conservative upper-bound estimates, which can provide a conservative basis for protection design.

6. Conclusions

This study evaluates landslide-generated impulse waves in the Sejiang slope area by integrating Pan's empirical method with 3D numerical simulations. The primary conclusions are as follows:

- (1) **Methodological Complementarity:** Pan's empirical formula provides a conservative estimation of the landslide's average entry velocity, suitable for preliminary risk screening. In contrast, 3D numerical simulation granular models effectively reproduce block-by-block initiation and non-uniform acceleration, capturing localized kinetic energy concentrations missed by empirical approaches.
- (2) **Topographic Amplification Effects:** Numerical simulation quantitatively reveals the topographic control on wave propagation. Due to reflection and superposition, wave heights at river bends are amplified by approximately 156% compared to linear attenuation models. Notably, impulse waves exhibit an energy convergence effect, resulting in a 2.9% increase in the peak wave height at the dam site relative to the initial impact zone, which necessitates rigorous wave-resistant design for appurtenant structures.
- (3) **Hydraulic Destructive Potential:** The destructive potential of impulse waves increases monotonically with reservoir water levels. The impulse wave heights at the check flood level are 3.4% higher than at the normal pool level. Therefore, the check flood level should serve as the upper-bound envelope for safety design. Rapid reservoir drawdown conditions warrant a specialized assessment due to the potential increases in the landslide sliding velocity.
- (4) **Differentiated Protection Strategy:** A comprehensive risk assessment confirms that dam overtopping is unlikely. However, a multi-tiered protection strategy is proposed: (i) anti-scour reinforcement for the direct impact zone on the opposite bank; (ii) drainage and deformation monitoring for the spoil disposal area prone to multi-cycle wave instability; and (iii) structural upgrading for shoreline appurtenant structures in the energy convergence zone.

- (5) Integrated Application Framework: An assessment framework for canyon-reservoir stability is established, utilizing Pan's method for rapid conservative envelope identification during initial design and numerical simulation for a refined topographic impact analysis in the detailed design stage. This integrated approach ensures both safety and economic efficiency in reservoir disaster mitigation.
- (6) The specific wave heights, arrival times, and amplification ratios obtained in this study are site-dependent and should not be directly transferred to other reservoirs. Nevertheless, the proposed assessment framework—combining geological interpretation, semi-empirical estimation, and CFD-based wave propagation simulation—is transferable to other reservoir-bank landslides. This integrated approach is particularly suitable for narrow canyon reservoirs where wave reflection, superposition, and local energy convergence are significant. When applied to other sites, the specific topographic, geological, and hydraulic parameters should be redefined rather than directly adopting the numerical values of the present case.
- (7) Among the available methods for landslide-generated impulse-wave prediction, semi-empirical formulae are suitable for rapid preliminary screening, while three-dimensional CFD provides a practical balance between the computational cost and engineering applicability for refined hazard assessment. CFD is most suitable for detailed engineering evaluation after preliminary screening has identified a potentially hazardous scenario.

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Conflicts of Interest: The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Abbreviations

The following abbreviations are used in this manuscript.

CFD	Computational Fluid Dynamics
RNG	Renormalization Group
FAVOR	Fractional Area/Volume Obstacle Representation
DA	Depth-averaged

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