

Article

Physical Model Experimental Study on Landslide-Generated Impulse Waves: A Case of Near-Dam Reservoir Landslide on the Upper Yellow River of China

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Abstract

Reservoir-bank landslides may become unstable after reservoir impoundment and generate impulse waves, posing serious threats to hydropower hub structures, reservoir navigation, and downstream safety. The H1 landslide, with a volume of approximately 5.35 million m³, is located on the left bank of the near-dam reservoir area of Yangqu Hydropower Station. Reservoir operation and water-level fluctuations may affect the stability of the H1 landslide; if instability occurs, rapid water entry could generate impulse waves that threaten the hydraulic structures. To evaluate the impulse-wave hazards associated with the H1 landslide, a three-dimensional physical model was constructed at a geometric scale of 1:200. Representative experiments were conducted under landslide instability volumes of 100×10^4 , 200×10^4 , and 500×10^4 , reservoir water levels of 2710.0 m and 2715.0 m, and the corresponding prototype-oriented water-entry conditions. The generation, propagation, opposite-bank run-up, and wave responses in front of the hydraulic structures were investigated. The results show that both the initial wave height near the water-entry point and the wave height in front of the hydraulic structures increase with increasing landslide volume. At the water-entry point, the maximum wave height increases from 3.10–3.27 m for the 100×10^4 m³ landslide to 4.50–4.64 m for the 200×10^4 m³ landslide and further to 7.18–7.21 m for the 500×10^4 m³ landslide. In front of the hydraulic structures, the maximum wave height at the spillway reaches 5.61 m for the 200×10^4 m³ landslide under the ecological restricted water level of 2710 m. After superposition with the wind-wave run-up, the corresponding total wave elevation is 2718.8 m, which remains below both the dam crest elevation of 2721.0 m and the wave-wall crest elevation of 2722.2 m. However, under the normal pool level of 2715 m, the total wave elevations at the spillway and power station intake after superposition with wind-wave run-up exceed the wave-wall crest elevation, indicating a potential overtopping risk. The research results provide a scientific basis for reservoir impoundment scheduling, landslide-generated impulse wave risk prevention, and the safety protection of key hydraulic structures at Yangqu Hydropower Station.



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1. Introduction

Landslide-generated impulse waves occur when an unstable reservoir-bank slope rapidly enters the water and displaces a large volume of reservoir water. These waves can have large amplitudes, propagate rapidly, and affect extensive areas. They therefore threaten hydraulic structures, reservoir navigation, and communities along reservoir banks and downstream reaches [1].

The 1963 Vajont Reservoir disaster in Italy illustrates the destructive potential of this hazard. Approximately $2.7 \times 10^8 \text{ m}^3$ of rock entered the reservoir and generated waves exceeding 200 m in height. The waves overtopped the dam and caused nearly 2000 fatalities downstream. This disaster remains an important warning for the planning, design, and operation of reservoir projects.

As hydropower development continues in western China, landslide-generated wave hazards in near-dam reservoir areas require increasing attention. Documented events and potential risk cases have been reported in the Three Gorges, Baihetan, and Xiluodu reservoir areas [2–4]. A clearer understanding of wave generation, propagation, run-up, and interaction with hydraulic structures is therefore needed to support reservoir operation and disaster-risk reduction [5].

Current research methods for reservoir landslide-generated impulse waves mainly include physical model tests, numerical simulations, and empirical formula-based predictions. Physical model testing is one of the most direct and effective approaches for investigating landslide-generated waves, as it can reproduce the complete process of landslide entry, wave generation, propagation, and run-up [6,7]. It also serves as a primary tool for revealing the formation mechanisms of landslide-generated waves and validating predictive models. Robbe-Saule et al. [8] investigated tsunami wave generation induced by subaerial granular column collapse through physical experiments. They proposed that the landslide front could be regarded as a “moving piston” that drives water motion, and demonstrated a clear correlation between wave amplitude and the Froude number of the landslide front. Takabatake et al. [9] conducted three-dimensional physical model tests of tsunami waves induced by partially submerged landslides and examined the effects of initial landslide position, degree of submergence, and landslide motion pattern on wave propagation and nearshore run-up. Their results highlighted the importance of three-dimensional boundary effects in amplifying the spatial distribution of wave heights. Darvenne et al. [10] investigated the coupled landslide–water interaction process using a two-dimensional granular-flow physical model and suggested that non-uniform granular flow can experimentally represent a mechanism for continuous wave generation. Bolin et al. [3] performed three-dimensional physical experiments on impulse-wave mitigation for the Wangjiashan landslide and evaluated the effectiveness of different protective measures. They emphasized that reservoir disaster mitigation should comprehensively consider the landslide source area, propagation path, and threatened targets. Chen et al. [11] established a 1:150 three-dimensional physical model for the high-position Meilishi landslide in the Gushui Reservoir and investigated the influence of landslide velocity on initial wave height, wave height in front of the dam, and wave run-up. They suggested that prototype-topography physical models are necessary for assessing near-dam impulse-wave risks induced by high-position landslides. Tian et al. [12] conducted large-scale geomechanical model tests on the Wangjiashan landslide in the Baihetan Reservoir area and developed a prediction approach for impulse-wave run-up, providing experimental support for hazard assessment in complex reservoir terrain.

Numerical simulation provides an efficient means for multi-scenario analysis and hazard-zone prediction of landslide-generated impulse waves. Wang et al. [13] investigated the near-field characteristics of landslide-generated impulse waves in channel reservoirs

using a soil–water coupled smoothed particle hydrodynamics model and demonstrated that the opposite bank strongly affects wave formation and near-field energy distribution. Zhang et al. [14] proposed a δ -LES-SPH model incorporating density diffusion and particle shifting techniques, which improved the stability of numerical calculations for wave generation and propagation. Xu and Dong [15] developed a three-dimensional SPH–DEM coupling method to simulate the entire process of landslide instability, movement, water entry, and subsequent hydrodynamic response, demonstrating its capability to represent both granular landslide motion and free-surface evolution. Liu et al. [16] proposed a one-dimensional two-layer coupled model and applied it to simulate the generation and long-distance propagation of the giant landslide wave in Lituya Bay. Bu et al. [17] used an SPH–DEM coupled model to study landslide-generated waves and showed that the discrete motion of particles plays an important role in controlling wave height and waveform. Zhao et al. [18] developed a two-layer two-phase material point method for simulating the large deformation of granular landslides and the resulting tsunami waves. Romano et al. [19] developed an OpenFOAM® framework in which granular landslides are represented using a Coulomb viscoplastic rheological model, enabling the coupled simulation of granular motion, wave generation, and free-surface evolution. Mao and Guan [20] examined the relationship between landslide conditions and dam-overtopping waves from a computational fluid dynamics perspective and concluded that landslide volume, initial position, and water depth jointly control the intensity of overtopping.

Theoretical formulas and empirical prediction models are advantageous because they require fewer parameters, allow rapid calculation, and are convenient for preliminary engineering assessment. Sabeti and Heidarzadeh [21] proposed an empirical equation for estimating the maximum initial amplitude of submarine landslide-generated waves. In a subsequent study, they developed a three-dimensional prediction relationship for the maximum initial amplitude of subaerial landslide tsunamis [22]. Sarlin et al. [23] combined granular-spreading dynamics with shallow-water wave theory and proposed a predictive model for the amplitude of waves generated by granular-column collapse.

Nguyen et al. [24] investigated partially and fully submerged granular column collapses using a multiphase flow–particle coupling method and proposed empirical relationships for maximum wave amplitude. Hu et al. [25] analyzed the classification and amplitude characteristics of near-field landslide waves induced by granular debris and suggested that different wave types can be identified according to water-entry patterns and particle motion characteristics. Heller and Ruffini [26] systematically reviewed physical experiments and empirical prediction methods for subaerial landslide tsunamis and pointed out that existing formulas still have limited applicability to complex three-dimensional topography, deformable landslides, and near-dam river-valley conditions. Therefore, in practical hydropower engineering, empirical formulas are more suitable as preliminary screening tools, and their results should be further verified using physical or numerical models.

Although previous studies have substantially improved the understanding of landslide-generated impulse waves, many physical experiments have been conducted in idealized straight flumes or simplified basins. Such models are suitable for identifying general wave-generation mechanisms and parameter relationships, but they cannot fully represent wave reflection, directional change, and local energy concentration in a narrow and curved near-dam reservoir reach. In addition, previous studies have often focused on the initial wave or near-source response, whereas the continuous evolution from landslide entry to opposite-bank run-up and wave loading at individual hydraulic structures has received less attention.

To address these limitations, this study establishes a 1:200 three-dimensional physical model based on the prototype topography of the near-dam reservoir area of Yangqu Hydropower Station. The selected landslide volumes represent characteristic combinations of instability within the geologically identified landslide zones, while the reservoir levels correspond to the ecologically restricted water level and the normal pool level of the project. The measurements cover the near-source wave response, opposite-bank run-up, downstream propagation, and wave heights at the spillway, dam center, and power station intake. Particular attention is given to the spatial redistribution of wave energy in the curved valley, the possibility that subsequent wave crests may exceed the first crest, and the resulting structure-specific overtopping hazard. The principal contribution of this study is therefore an engineering-oriented, source-to-structure experimental assessment that links realistic landslide scenarios, three-dimensional reservoir topography, complete-wave-train behavior, and protection requirements for key hydraulic structures.

2. Engineering Geological Overview

Yangqu Hydropower Station is located on the main stream of the Yellow River at the junction of Xinghai County and Guide County in Hainan Tibetan Autonomous Prefecture, Qinghai Province. It is an important cascade hydropower station planned for the Longyangxia-Laxiwa reach in the upper Yellow River. The dam site controls a drainage area of approximately $1.28 \times 10^5 \text{ km}^2$, with a multi-year average discharge of about $650 \text{ m}^3/\text{s}$. The hydropower complex consists of a concrete-face rockfill dam, a spillway on the left bank, a power station intake on the right bank, and a water-diversion and power-generation system. The dam crest elevation is 2721.0 m, the top elevation of the wave wall is 2722.2 m, the normal pool level is 2715.0 m, and the ecological restricted water level is 2710.0 m. The total reservoir capacity is approximately 1.625 billion m^3 , and the installed capacity is 1200 MW. The river valley in the dam-site area is narrow, with steep bank slopes and complex geological structures. Landslides are well developed along the near-dam reservoir banks, among which the H1 landslide is one of the major geological issues affecting engineering safety.

The H1 landslide is located on the left bank upstream of the Yangqu Hydropower Station dam site, approximately 1.2–2.0 km from the dam site, as shown in Figure 1. The dam axis shown in the figure is schematic. The impulse waves generated by landslide instability are considered to propagate to the dam axis only to provide a basis for flood-control design; theoretically, the issue of dam failure is beyond the scope of this study. The river valley in the area where the H1 landslide is located is characterized by canyon topography. The Yellow River passes along the front edge of the H1 landslide in a direction ranging from NW 334° to NE 7° . The valley is narrow and the slopes are steep, with a valley width of only 30–40 m, bank slope gradients of $40\text{--}60^\circ$, and a relative elevation difference of approximately 400 m.

In plan view, the H1 landslide exhibits a dustpan-shaped morphology, with a narrow front edge and a wide rear edge. The elevation of the shear outlet at the front edge of the landslide is approximately 2620.0–2630.0 m, which is 15–20 m higher than the river water level and is located near the base of the second terrace of the Yellow River. The rear edge of the landslide has an elevation of 2860.0–2960.0 m, with the highest point approximately 360 m above the river water level. The H1 landslide is approximately 290–360 m long in the longitudinal direction, with a maximum length of 490 m, an average thickness of about 25 m, and a volume of approximately $535 \times 10^4 \text{ m}^3$. It is classified as a large-scale rocky bedding-cutting landslide.

The lithology of the H1 landslide is dominated by Lower Permian (P1) phyllite and phyllitic slate, with minor interbeds of sandy slate and metamorphic sandstone. The

rock mass structure is mainly thinly layered, followed by thinly layered-interbedded structures accounting for approximately 23% and fractured-loose structures accounting for approximately 17%. The attitude of the strata is NW $300^{\circ}\sim 330^{\circ}$ /NE $70^{\circ}\sim 80^{\circ}$, and the H1 landslide is primarily a high-dip bedding landslide with an oblique sliding tendency.

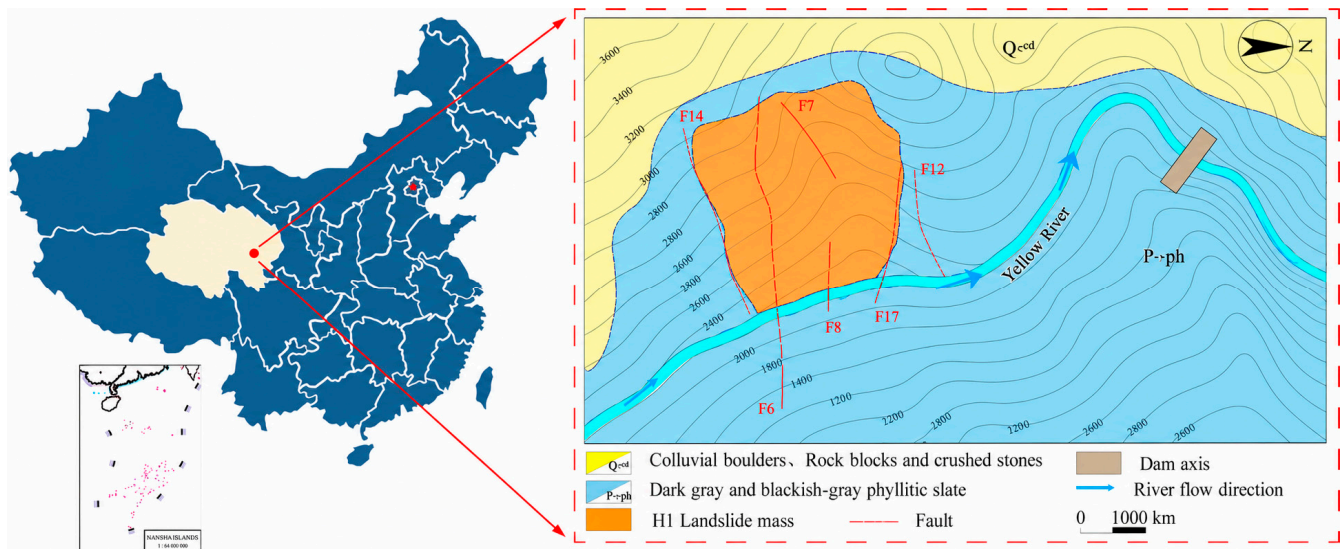


Figure 1. Upstream plane of the dam site of Yangqu Hydropower Station.

Taking the small gully formed by the F6 fault in the middle of the H1 landslide as the boundary, the landslide body is divided into upstream Zone I and downstream Zone II (Figure 1). Zone I is the residual part of an earlier landslide and has undergone later deformation and reworking, with a volume of approximately $135 \times 10^4 \text{ m}^3$. Zone II is in the stage of deformation development, with a large thickness and a volume of approximately $400 \times 10^4 \text{ m}^3$; however, no obvious continuous sliding surface has been observed. Previous stability analyses indicate that the secondary collapse-slide mass and landslide deposits in Zone I, together with the landslide deposits in Zone II, may become unstable under unfavorable reservoir operating conditions and water-level fluctuations, particularly when groundwater pressure cannot dissipate synchronously with changes in the external reservoir level. The maximum volume of a single instability event is estimated to be approximately $300 \times 10^4 \text{ m}^3$ in total, of which the unstable volume is approximately $100 \times 10^4 \text{ m}^3$ in Zone I and approximately $200 \times 10^4 \text{ m}^3$ in Zone II. Accordingly, the 100×10^4 and $200 \times 10^4 \text{ m}^3$ volumes were selected to represent the characteristic partial-instability magnitudes identified for Zones I and II, respectively. The $500 \times 10^4 \text{ m}^3$ volume, which is close to the estimated total H1 landslide volume of approximately $535 \times 10^4 \text{ m}^3$, was adopted as an unfavorable near-total-instability envelope. These scenarios are geological volume bounds for consequence assessment rather than equally probable failure events. More accurate forecasting of the future instability volume requires updated information on sliding-surface continuity, groundwater conditions, shear-strength degradation, deformation development, and reservoir-level history.

Based on the engineering geological conditions and previous stability assessment, the inferred deformation mode of the H1 landslide is characterized by creep sliding at the front edge, tensile cracking at the rear edge, and overall or block-wise push-type sliding. However, its instability cannot be attributed solely to reservoir impoundment. During water-level rise, infiltration and elevated groundwater pressure may reduce the effective normal stress and weaken materials along potential shear zones, whereas buoyancy and external hydrostatic pressure may reduce the effective weight of the submerged portion and provide partial support. During rapid drawdown, external hydrostatic support may

decrease faster than internal pore pressure dissipates, producing an unfavorable transient seepage condition. Progressive deformation, saturation, and strain localization may also reduce shear strength from the peak value toward the residual value, thereby affecting slide acceleration and water-entry velocity. The actual stability response, therefore, depends on permeability, drainage conditions, groundwater evolution, water-level history, and the continuity and residual strength of the potential shear zones. Because these hydro-mechanical processes and site-specific strength-degradation parameters are not simulated in the present physical model, this study does not predict the failure timing or entry velocity from geotechnical parameters; instead, it evaluates the impulse-wave response after representative slide-release scenarios are imposed.

3. Physical Model Experimental Design

3.1. Similarity Criteria

The laboratory physical model experiment was designed according to the Froude gravitational similarity criterion with a geometric scale ratio of ($L_r = 200$). According to the requirements of the DL/T 5246-2010 [27], the landslide body in the physical model satisfies geometric and specific-gravity similarity, while the river channel and bank slopes satisfy resistance similarity. The scale relationships of the physical quantities are listed in Table 1. Because the prototype and the physical model are subjected to the same terrestrial gravitational field, the gravitational-acceleration scale ratio is $g_r = g_p/g_m = 1$; this row is distinct from the roughness-coefficient scale ratio $n_r = L_r^{1/6} = 2.42$.

Table 1. Scale of physical quantities in model test.

Physical Quantity	Scale Relationship	Scale Value
Geometric scale ratio	L_r	200
Flow velocity scale ratio	$L_r^{(0.5)}$	14.14
Time scale ratio	$L_r^{(0.5)}$	14.14
Roughness coefficient scale ratio	$L_r^{(1/6)}$	2.42
Gravitational acceleration scale ratio	1	1

The Froude similarity criterion preserves the dominant balance between gravitational and inertial forces and is therefore appropriate for reproducing the principal processes of landslide entry, water displacement, impulse-wave propagation, and run-up. However, Reynolds-number and Weber-number similarities cannot be simultaneously satisfied with Froude similarity in a 1:200 water model. Consequently, some scale effects should be recognized when interpreting the experimental results. The lower Reynolds number at model scale may produce relatively stronger viscous and boundary-layer dissipation, particularly during shallow-water propagation and near-bank run-up. Surface tension is also relatively more important at the reduced scale and may affect small-amplitude disturbances, local free-surface curvature, splash breakup, and detailed wave-crest morphology. In addition, air entrainment and aerated impact processes cannot be reproduced with strict similarity; therefore, the model-scale bubble distribution, splash fragmentation, and short-duration impact pressures may differ from those under prototype conditions. The present model is consequently more suitable for evaluating the relative effects of the considered engineering scenarios, the spatial pattern of wave propagation, the locations of maximum run-up, and the order of magnitude of wave heights than for reproducing detailed prototype-scale splash, aeration, and local impact-pressure processes.

The geometric scale of 1:200 was selected to incorporate the H1 landslide source area, the near-dam curved reservoir reach, the opposite bank, and the principal hydraulic structures within a single three-dimensional model while retaining sufficient spatial resolution

for wave-height measurements. The similarity design follows DL/T 5246-2010. Geometric similarity was applied to the reservoir topography, sliding path, and relative positions of the measuring points and hydraulic structures. The sliding-body density was configured according to specific-gravity similarity, and the river channel and bank slopes were treated according to resistance similarity. Froude similarity preserves the dominant balance between gravitational and inertial effects and is therefore appropriate for reproducing the main processes of landslide entry, water displacement, free-surface-wave propagation, reflection, and run-up.

The physical model mainly simulates the generation and propagation of impulse waves induced by the instability and subsequent entry of the H1 landslide into the reservoir. The physical model extends 3.2 km upstream from the dam site, with a total model length of approximately 20 m (Figure 2).

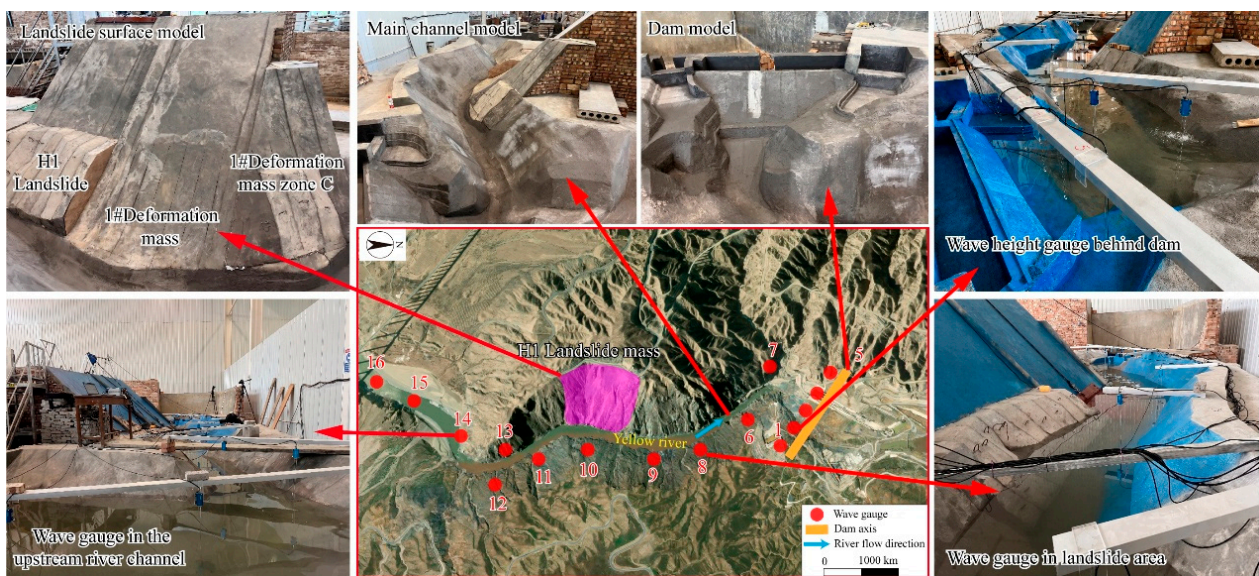


Figure 2. Physical model of the main river.

3.2. Physical Model Generalization and Landslide Body Simulation

The sliding bed of the physical model was constructed using reinforced concrete according to the sliding surface inferred from the engineering geological profile. The surface of the sliding bed was smoothed, and the H1 landslide mass was represented by a multi-segment hinged steel-plate carriage. According to the gravitational similarity requirements, the density of the carriage was approximately 7.81 g/cm^3 . Different instability volumes were reproduced by combining steel blocks of three sizes: small blocks measuring $10 \text{ cm} \times 5 \text{ cm} \times 5 \text{ cm}$, medium blocks measuring $10 \text{ cm} \times 10 \text{ cm} \times 5 \text{ cm}$, and large blocks measuring $20 \text{ cm} \times 10 \text{ cm} \times 5 \text{ cm}$. These block combinations were used to represent the characteristic instability-volume scenarios selected for the engineering hazard assessment.

The multi-segment hinged carriage enabled repeatable control of the bulk volume, density, sliding path, and gravity-driven water-entry process. It should therefore be regarded as a segmented rigid-body analogue rather than a granular-material model. The model reproduces the overall motion and bulk hydrodynamic loading associated with the selected landslide scenarios, but it does not simulate groundwater seepage, pore-pressure evolution, progressive shear-strength degradation along the sliding surface, internal deformation, rock fragmentation, particle segregation, erosion, or water entrainment. These processes may affect landslide acceleration, entry duration, and the efficiency of kinetic-energy transfer to the reservoir. Accordingly, the measured entry velocities and wave responses are conditional on the adopted source scenarios and should not be interpreted as predictions

of the natural failure initiation, sliding-surface evolution, or complete internal deformation process of the prototype H1 landslide.

3.3. Measurement System

In the physical model experiments, 16 CBG03 intelligent wave gauges were arranged along the river channel and in front of the hydraulic structures (Figure 2), and 5 CY201 pressure sensors were used to monitor the hydrodynamic pressure on the dam surface. A high-speed camera was used to record the landslide velocity and wave run-up process. The key measuring points included Points 8~10 near the water-entry location, Points 11~13 on the opposite bank, Points 1~5 in front of the dam, as well as points at the left dam abutment, dam center, right dam abutment, spillway inlet, and power station intake. In the original experimental records, 'Ob9' referred to the same wave-gauge location as Point 9, while figure labels such as '8#Point,' '9#Point,' and '10#Point' denoted Points 8, 9, and 10, respectively. Points 8–10 represent the three wave gauges arranged near the water-entry area. The layout of the measuring points is shown in Figure 2.

Before each test, the initial still-water elevation was recorded and used as the reference datum for processing the wave-gauge signals. Relative wave height represents the difference between the instantaneous water-surface elevation measured at a wave gauge and the initial still-water elevation before release of the sliding body. Accordingly, a value of zero represents the initial undisturbed water surface, a positive value represents a wave crest, and a negative value represents a wave trough. In Figures 3 and 4, the primary vertical axis represents relative wave height converted to the prototype scale, with a unit of metres. The secondary vertical axis labeled 'Reservoir water level' represents the corresponding absolute prototype water-surface elevation relative to the engineering elevation datum, also with a unit of metres. Its initial values are 2710.0 m and 2715.0 m in panels (a) and (b), respectively. The horizontal axis represents prototype time in seconds after conversion according to the Froude similarity relationship. Throughout this study, 'first-wave height' refers to the height of the first positive crest relative to the initial still-water surface, whereas 'maximum wave height' refers to the largest positive crest within the complete recorded wave train. 'Run-up elevation' denotes the absolute elevation reached on the opposite bank, while 'run-up height above the initial reservoir level' denotes the corresponding vertical difference from the initial reservoir surface. These terms are used consistently in the following tables and discussion.

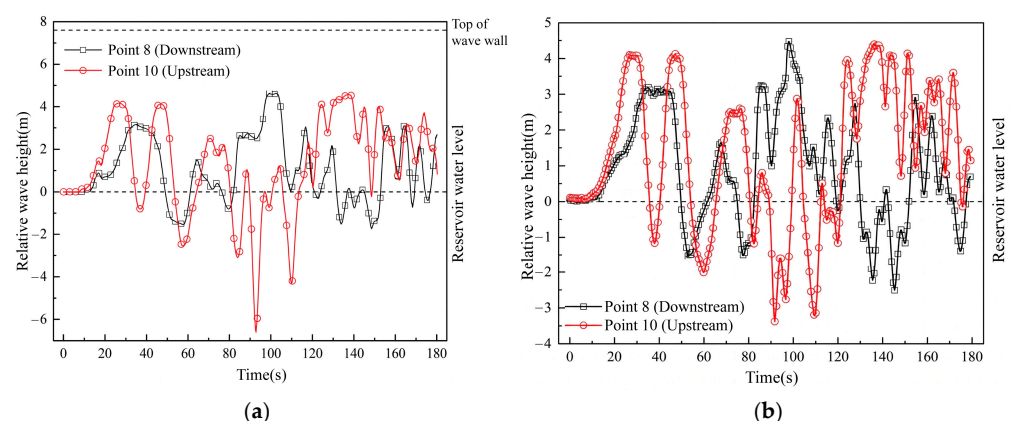


Figure 3. Time histories of landslide-generated impulse waves near the water-entry area for the $200 \times 10^4 \text{ m}^3$ scenario: (a) reservoir level of 2710.0 m; (b) reservoir level of 2715.0 m.

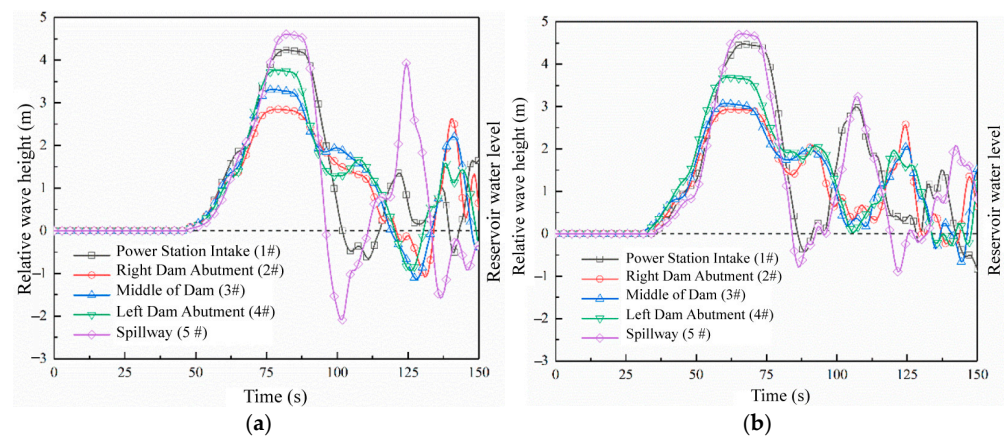


Figure 4. Landslide-generated impulse-wave time histories in front of the hydraulic structures for the $200 \times 10^4 \text{ m}^3$ scenario: (a) reservoir level of 2710.0 m; (b) reservoir level of 2715.0 m.

Data recording began before release of the sliding body and continued throughout water entry, wave generation, propagation, reflection, run-up, and subsequent attenuation. The high-speed camera records were used to determine the water-entry velocity and identify the maximum opposite-bank run-up, while the wave-gauge records were used to extract the first-wave height and the maximum wave height over the complete wave train. The repeated records for each representative condition were checked for abnormal release or acquisition conditions, and representative arithmetic mean values were used in the subsequent analysis.

3.4. Experimental Conditions

A total of 116 experimental conditions were designed in the overall physical-model testing program. Owing to space limitations, only the representative prototype-oriented conditions used for engineering hazard assessment are reported in this study. The selected landslide volumes were 100×10^4 , 200×10^4 , and $500 \times 10^4 \text{ m}^3$. According to the previous geological and stability assessments, the $100 \times 10^4 \text{ m}^3$ scenario represents the characteristic partial instability of Zone I, whereas the $200 \times 10^4 \text{ m}^3$ scenario represents the characteristic partial instability of Zone II. The $500 \times 10^4 \text{ m}^3$ scenario, which is close to the estimated total H1 landslide volume of approximately $535 \times 10^4 \text{ m}^3$, was adopted as a conservative near-total-instability envelope for Zones I and II. The reservoir levels were set at 2710.0 and 2715.0 m, corresponding to the ecological restricted water level and the normal pool level of Yangqu Hydropower Station, respectively.

The three landslide volumes were therefore not selected as an arbitrary scalar range from 1 to 5 million m^3 . Instead, they represent two characteristic partial-instability scenarios derived from the geological zoning of the H1 landslide and one unfavorable near-total-instability envelope. No occurrence probability was assigned to these scenarios, and the physical-model results quantify the hydrodynamic consequences conditional on the adopted source volumes rather than predict the most likely future instability volume. For engineering hazard and emergency planning, the design instability volume should be progressively updated using additional information on sliding-surface continuity, shear-strength degradation, groundwater and pore-pressure conditions, slope deformation, and reservoir-level history.

A broader range of water-entry conditions was considered in the overall testing program. For the representative scenarios reported here, the sliding body was released under gravity-driven conditions, and the water-entry velocities listed in Tables 2–4 were measured results rather than independently prescribed control variables. The reported velocities therefore reflect the combined effects of landslide volume, sliding-carriage configuration, sliding

path, reservoir level, and gravity-driven acceleration. Consequently, the reported cases should be interpreted as coupled prototype scenarios rather than independent single-factor tests of landslide volume, reservoir level, or entry velocity.

Table 2. Near-source impulse-wave characteristics under the representative landslide scenarios.

Experimental Condition	Landslide Velocity (m/s)	First-Wave Height (m)	Maximum Wave Height (m)
100w-2710	6.69	1.73	3.27
100w-2715	6.07	1.47	3.10
200w-2710	8.86	3.19	4.50
200w-2715	8.11	3.18	4.64
500w-2710	10.41	7.21	7.21
500w-2715	10.33	7.18	7.18

Table 3. Opposite-bank run-up characteristics under the representative landslide scenarios.

Experimental Condition	Landslide Velocity (m/s)	First-Wave Run-Up Elevation (m)	First-Wave Run-Up Height Above the Initial Reservoir Level (m)	Maximum Run-Up (m)	Maximum Run-Up Height Above the Initial reservoir Level (m)
100w-2710	6.69	2726.00	16.00	2726.00	16.00
100w-2715	6.07	2731.00	16.00	2731.00	16.00
200w-2710	8.86	2745.00	35.00	2746.00	36.00
200w-2715	8.11	2752.00	37.00	2752.00	37.00
500w-2710	10.41	2769.00	59.00	2769.00	59.00
500w-2715	10.33	2764.00	49.00	2764.00	49.00

Table 4. Impulse-wave heights in front of the hydraulic structures under the representative landslide scenarios.

Experimental Condition	Landslide Velocity (m/s)	Spillway Wave Height (m)	Dam-Center Wave Height (m)	Power Station Intake Wave Height (m)
100w-2710	6.69	2.28	2.06	1.79
100w-2715	6.07	1.85	1.84	1.93
200w-2710	8.86	5.61	3.31	4.64
200w-2715	8.11	4.72	3.07	4.47
500w-2710	10.41	10.17	7.19	9.20
500w-2715	10.33	10.75	6.72	9.00

Each representative condition was repeated no fewer than three times. The repeated runs were conducted primarily as engineering quality-control tests to identify abnormal release or acquisition conditions, examine the consistency of the principal waveform and spatial response pattern, and obtain a representative arithmetic mean for each prototype-oriented scenario. They were not designed as an independent statistical sampling program involving randomised source parameters or boundary conditions. Because only three or slightly more valid repetitions were available for each condition, condition-specific dispersion statistics derived from such a small controlled sample should not be interpreted as statistically robust uncertainty bounds or confidence intervals. Accordingly, Tables 2–4 report representative arithmetic means for comparative scenario assessment rather than probabilistic estimates. Small differences between individual scenarios and mean elevations close to engineering thresholds are therefore interpreted cautiously, and no probability of exceedance is inferred from the repeated tests.

The reliability of the physical-model results was evaluated through several complementary checks. First, the model geometry, sliding path, and hydraulic boundaries were established according to the prototype topography and project operating levels. Second, the selected source volumes were linked to the geologically identified instability zones of the H1 landslide rather than treated as arbitrary laboratory parameters. Third, repeated tests were conducted to examine the consistency of the main waveform and spatial response pattern. Finally, the records obtained near the water-entry area, on the opposite bank, and in front of the hydraulic structures exhibited a coherent source-to-propagation sequence and consistent scenario-dependent variation. These checks support the use of the model for comparative engineering assessment. Nevertheless, because no actual H1 landslide-generated impulse-wave event is available for direct field validation, the results should be regarded as scenario-based physical-model estimates rather than exact deterministic prototype predictions.

4. Analysis of Experimental Results

4.1. Impulse Wave Characteristics at the Water-Entry Point

During the experiments, it was observed that after the landslide body rushed into the water at a certain velocity, intense solid–liquid interaction first occurred near the water-entry point. The water body was rapidly displaced, forming an upward-splashing water curtain and a downward-depressed impact crater. Subsequently, the displaced water flowed back under the action of gravity and propagated forward, forming the first wave crest, namely the first wave. Figure 3 presents the near-source time histories of landslide-generated impulse waves recorded at Point 9 near the water-entry area for the $200 \times 10^4 \text{ m}^3$ scenario under reservoir levels of 2710.0 m and 2715.0 m. In Figure 3, relative wave height is referenced to the initial still-water surface and is expressed in prototype metres, whereas the secondary vertical coordinate represents the corresponding absolute reservoir elevation in metres; the horizontal coordinate represents prototype time in seconds. As shown in Figure 3 and Table 2, the first wave reached the measuring point shortly after water entry and exhibited a steep waveform with a large slope in the rising limb. Under the reservoir water level of 2710.0 m, the first-wave height was 3.19 m, and the maximum wave height was 4.50 m. Under the reservoir water level of 2715.0 m, the first-wave height was 3.18 m, and the maximum wave height was 4.64 m. These results indicate that the first-wave heights under the two water-level conditions were very close, while the maximum wave height under the higher water level was slightly larger because of subsequent wave reflection and superposition.

Table 2 summarizes the characteristic parameters of impulse waves at the water-entry point under different landslide instability volumes. The water-entry velocities listed in Table 2 are the measured velocities corresponding to the selected gravity-driven engineering scenarios. They do not constitute an independently controlled velocity series because landslide volume, reservoir level, and water-entry velocity varied simultaneously among the reported conditions. Therefore, the present data are used to compare the overall wave responses of representative scenarios and should not be interpreted as establishing an independent wave-height–velocity relationship at a fixed landslide volume. As shown in Table 2, both the first-wave height and the maximum wave height increased significantly with increasing landslide instability volume. When the landslide instability volume was $100 \times 10^4 \text{ m}^3$, the first-wave height was 1.47–1.73 m and the maximum wave height was 3.10–3.27 m. When the landslide instability volume increased to $200 \times 10^4 \text{ m}^3$, the first-wave height increased to 3.18–3.19 m, and the maximum wave height increased to 4.50–4.64 m. When the landslide instability volume further increased to $500 \times 10^4 \text{ m}^3$, the first-wave height and the maximum wave height reached 7.18–7.21 m. As the landslide instability

volume increased from $200 \times 10^4 \text{ m}^3$ to $500 \times 10^4 \text{ m}^3$, approximately 2.5 times larger, the maximum wave height increased by approximately 1.55–1.60 times, indicating a clear but sublinear increase in wave height with landslide volume. Under the same landslide instability volume, the influence of reservoir water level on the initial wave height was relatively limited. For the $100 \times 10^4 \text{ m}^3$ landslide, the maximum wave height was 3.27 m at the 2710.0 m water level and 3.10 m at the 2715.00 m water level. For the $200 \times 10^4 \text{ m}^3$ landslide, the maximum wave height was 4.50 m at the 2710.0 m water level and 4.64 m at the 2715.0 m water level. For the $500 \times 10^4 \text{ m}^3$ landslide, the maximum wave heights under the two water levels were 7.21 m and 7.

4.2. Run-Up on the Opposite Bank

When landslide-generated impulse waves propagate to the opposite bank, shallow-water deformation and run-up occur due to the gradually decreasing water depth and the constraint of the bank slope. The run-up on the opposite bank is directly related to the safety of infrastructure such as roads and villages on the bank slope. Table 3 presents the maximum run-up values on the opposite bank under different experimental conditions in the model tests.

Analysis of Table 3 shows that, under the same water-level condition, the run-up height on the opposite bank generally increases with increasing landslide volume. When the landslide volume was $100 \times 10^4 \text{ m}^3$, the maximum run-up height above the reservoir water level was 16 m under both water-level conditions. When the landslide volume increased to $200 \times 10^4 \text{ m}^3$, the maximum run-up height above the reservoir water level increased to 36 m at the 2710.0 m water level and 37 m at the 2715.0 m water level. When the landslide volume further increased to $500 \times 10^4 \text{ m}^3$, the maximum run-up height above the reservoir water level reached 59 m at the 2710.0 m water level and 49 m at the 2715.0 m water level. These results indicate that landslide volume has a significant controlling effect on opposite-bank run-up.

For a given nominal landslide volume, the two reservoir-level conditions did not represent identical source-energy conditions. Changing the initial reservoir level altered the effective subaerial travel distance and vertical drop before water entry, as well as the water-contact position, impact submergence, and measured entry velocity of the sliding body. The measured entry velocities at reservoir levels of 2710.0 and 2715.0 m were 6.69 and 6.07 m/s for the $100 \times 10^4 \text{ m}^3$ scenario, 8.86 and 8.11 m/s for the $200 \times 10^4 \text{ m}^3$ scenario, and 10.41 and 10.33 m/s for the $500 \times 10^4 \text{ m}^3$ scenario, respectively. Therefore, the kinetic energy and momentum transferred to the reservoir differed between the paired water-level cases. The resulting opposite-bank run-up was further affected by local water depth, shoaling, valley confinement, bank reflection, and wave superposition. Accordingly, similar relative run-up heights under some paired conditions do not indicate equivalent energy transfer, and the results should be interpreted as comparisons between coupled prototype scenarios rather than as an independent reservoir-level sensitivity relationship.

Considering the limited number of repeated tests and the absence of a formal statistical uncertainty analysis, the approximately 1 m difference between the two $200 \times 10^4 \text{ m}^3$ scenarios should not be interpreted as an independent reservoir-level effect. For the largest tested landslide volume of $500 \times 10^4 \text{ m}^3$, the maximum run-up elevation on the opposite bank reached 2769.0 m under the 2710.0 m water-level condition and 2764.0 m under the 2715.0 m water-level condition. Although these values represent significant local run-up, they are still much lower than the road elevation on the opposite bank, which is approximately 2930 m. Therefore, the impulse waves generated under the tested conditions would not directly impact the road on the opposite bank.

4.3. Impulse Waves in Front of the Dam

After generation near the water-entry area, the impulse waves propagated downstream and reached the hydraulic structures after a finite travel time. Their amplitudes and temporal evolution are directly relevant to overtopping potential and hydrodynamic loading. Table 4 summarises the characteristic wave heights measured at the spillway, dam centre, and power station intake under the different landslide-volume scenarios. Figure 4 presents the corresponding wave-height time histories for the $200 \times 10^4 \text{ m}^3$ scenario at reservoir levels of 2710.0 m and 2715.0 m. The coordinate definitions are consistent with those used in Figure 3: relative wave height is referenced to the initial still-water surface and expressed in prototype metres; the secondary vertical axis represents the absolute reservoir elevation in metres; and the horizontal axis represents prototype time in seconds.

As shown in Table 4 and Figure 4, the impulse wave heights in front of the hydraulic structures differed markedly under different landslide instability volumes. When the single water-entry volume of the H1 landslide was $100 \times 10^4 \text{ m}^3$, the wave heights at the tabulated measuring locations ranged from 1.79 m to 2.28 m. When the single water-entry volume increased to $200 \times 10^4 \text{ m}^3$, the wave heights increased to 3.07–5.61 m. When the single water-entry volume further increased to $500 \times 10^4 \text{ m}^3$, the wave heights increased sharply to 6.72–10.75 m. These results indicate that the impulse wave height in front of the hydraulic structures increases significantly with increasing landslide instability volume.

For the $200 \times 10^4 \text{ m}^3$ and $500 \times 10^4 \text{ m}^3$ landslide cases, the spillway generally experienced the largest wave height among the three tabulated measuring locations, followed by the power station intake and the dam centre. Under the 2710.0 m water-level condition, the wave heights at the spillway, dam centre, and power station intake were 5.61 m, 3.31 m, and 4.64 m, respectively, for the $200 \times 10^4 \text{ m}^3$ landslide. Under the 2715.0 m water-level condition, the corresponding wave heights were 4.72 m, 3.07 m, and 4.47 m, respectively. For the $500 \times 10^4 \text{ m}^3$ landslide, the wave heights at the spillway, dam centre, and power station intake were 10.17 m, 7.19 m, and 9.20 m, respectively, under the 2710.0 m water-level condition, and 10.75 m, 6.72 m, and 9.00 m, respectively, under the 2715.0 m water-level condition. This spatial distribution indicates that the spillway and power station intake are the key locations requiring special attention in impulse wave hazard assessment.

Further analysis can be conducted in conjunction with Figure 4, which shows the time-history curves of impulse waves propagating to the front of the hydraulic structures under two reservoir water levels when the H1 landslide undergoes instability with a single water-entry volume of $200 \times 10^4 \text{ m}^3$. Under the reservoir water level of 2710.0 m, the maximum wave height at the spillway reached 5.61 m. Under the reservoir water level of 2715.0 m, the maximum wave height at the spillway was 4.72 m. The maximum wave height did not necessarily occur in the first wave crest, indicating that reflected waves from the bank slopes and the curved river-valley boundary may superimpose with subsequent waves and produce larger later-stage wave crests. Therefore, in engineering early warning, the arrival of the first wave should not be used as the sole indicator; instead, attention should be paid to the possible maximum wave height occurring within the entire wave train.

The above phenomena indicate that, during the propagation of impulse waves generated by landslide instability and subsequent entry into the reservoir in a narrow valley, the wave field is affected by multiple superposition effects caused by reflections from the bank slopes on both sides and the curved channel topography, such that subsequent wave crests may exceed the first wave. Therefore, in engineering early warning, the arrival of the first wave should not be used as the sole indicator; instead, attention should be paid to possible peak wave heights occurring within the entire wave train.

The wave heights measured in front of the hydraulic structures were compared with those predicted using commonly applied empirical methods. For the $200 \times 10^4 \text{ m}^3$ scenario

at a reservoir level of 2710.0 m, the measured water-entry velocity was approximately 8.86 m/s, and the spillway wave height obtained from the physical model was 5.61 m. The Pan Jiazheng method predicted a wave height of 1.07–1.63 m, whereas the method proposed by the China Institute of Water Resources and Hydropower Research predicted 0.96–1.45 m.

Empirical methods are advantageous because they require relatively few input parameters and allow rapid preliminary assessment. However, they are generally derived from simplified experimental geometries and represent wave generation and propagation using a limited number of bulk parameters. Consequently, they cannot fully reproduce the three-dimensional effects associated with the narrow and curved reservoir reach considered in this study. In the physical model, repeated reflection from the valley sides, directional changes in wave propagation, and superposition of subsequent wave crests may locally increase the wave height near the hydraulic structures.

The lower empirical estimates should therefore not be interpreted as indicating that these methods are universally invalid. Instead, the discrepancy reflects differences in model assumptions, topographic representation, and the definition and location of the evaluated wave response. Empirical formulas remain useful for rapid preliminary screening, whereas prototype-topography physical models are more suitable for assessing structure-specific wave responses under complex near-dam boundary conditions. Because no actual H1 landslide-generated wave event is available for direct field validation, the present comparison is regarded as a methodological cross-check rather than a definitive validation of either approach. The physical-model results were therefore adopted as the primary basis for the subsequent overtopping screening.

The wind-wave run-up of 3.21 m was adopted directly from the approved engineering report for Yangqu Hydropower Station and was not calculated from the physical-model data obtained in the present study. The value was further cross-checked with the relevant project technical personnel and confirmed to be consistent with the project design basis. The available engineering report provides the final design wind-wave run-up but does not disclose the underlying design wind speed, effective fetch length, return period, mean water depth, or intermediate calculation coefficients. Therefore, these project-specific input parameters are not reconstructed or inferred in the present study.

The general calculation framework was checked against the applicable Chinese standards for embankment-dam design and hydraulic-structure loading. In this framework, the mean wave run-up is converted to the design-frequency run-up as $R_p = K_p R_m$, and the required dam freeboard is generally expressed as $Y = R_p + e + A$, where R_m is the mean run-up, K_p is the frequency coefficient, R_p is the design-frequency run-up, e is the wind setup, and A is the safety allowance [27–29]. These relationships are provided to explain the general design procedure and are not intended to reproduce the undisclosed project-specific calculation of the 3.21 m value.

For the present overtopping screening, the total elevation was calculated as $Z_{\text{screen}} = Z_0 + H_{\text{LGIW}} + R_w$, where Z_0 is the initial reservoir level, H_{LGIW} is the landslide-generated wave height measured in front of the relevant hydraulic structure, and $R_w = 3.21$ m is the wind-wave run-up adopted from the approved project report. The direct superposition represents a conservative, unfavorable envelope and does not reproduce the phase-resolved interaction between wind waves and landslide-generated impulse waves. Therefore, the calculated elevations are interpreted as scenario-based screening values rather than fully coupled deterministic predictions.

Using the screening relationship $Z_{\text{screen}} = Z_0 + H_{\text{LGIW}} + R_w$, the overtopping status was evaluated for all three tested landslide-volume scenarios under both reservoir levels. Here, Z_0 is the initial reservoir level, H_{max} is the maximum measured landslide-generated

wave height among the spillway, dam center, and power station intake, and $R_w = 3.21$ m is the project-design wind-wave run-up adopted from the engineering report.

For the $100 \times 10^4 \text{ m}^3$ scenario at a reservoir level of 2710.0 m, the maximum wave height in front of the hydraulic structures was 2.28 m at the spillway. The corresponding combined screening elevation was $2710.0 + 2.28 + 3.21 = 2715.49$ m, which was 5.51 m below the dam crest elevation of 2721.0 m and 6.71 m below the wave-wall crest elevation of 2722.2 m. At a reservoir level of 2715.0 m, the maximum wave height was 1.93 m at the power station intake, giving a combined elevation of $2715.0 + 1.93 + 3.21 = 2720.14$ m. This value remained 0.86 m below the dam crest and 2.06 m below the wave-wall crest. Therefore, neither of the $100 \times 10^4 \text{ m}^3$ conditions exceeded the two reference crest elevations under the adopted screening method. For the $200 \times 10^4 \text{ m}^3$ scenario, the maximum structural-front wave heights were 5.61 m and 4.72 m at reservoir levels of 2710.0 m and 2715.0 m, respectively. The corresponding combined elevations were 2718.82 m and 2722.93 m. The former remained 2.18 m below the dam crest and 3.38 m below the wave-wall crest based on the representative mean response, whereas the mean-based screening for the 2715.0 m condition yielded nominal elevations 1.93 m above the dam crest and 0.73 m above the wave-wall crest. For the $500 \times 10^4 \text{ m}^3$ scenario, the maximum structural-front wave heights were 10.17 m and 10.75 m at reservoir levels of 2710.0 m and 2715.0 m, respectively. The corresponding combined elevations were 2723.38 m and 2728.96 m. The mean-based combined elevations were nominally 2.38 m and 7.96 m above the dam crest and 1.18 m and 6.76 m above the wave-wall crest, respectively.

Because the underlying parameters used to derive the project-design wind-wave run-up of 3.21 m are unavailable, a deterministic sensitivity analysis was conducted to examine whether the overtopping classifications depend strongly on this adopted value. The wind-wave run-up was varied by $\pm 10\%$ and $\pm 20\%$, corresponding to values of 2.89, 2.57, 3.53, and 3.85 m, respectively. These perturbations are introduced solely as engineering sensitivity scenarios and do not represent statistical confidence bounds. Within the $\pm 20\%$ range, the classification of all six representative scenarios remained unchanged. The $100 \times 10^4 \text{ m}^3$ scenarios and the $200 \times 10^4 \text{ m}^3$ scenario at 2710.0 m remained below both reference crest elevations at the upper sensitivity bound, whereas the $200 \times 10^4 \text{ m}^3$ scenario at 2715.0 m and both $500 \times 10^4 \text{ m}^3$ scenarios remained above both reference elevations at the lower sensitivity bound. The $200 \times 10^4 \text{ m}^3$ scenario at 2715.0 m was the most sensitive case: at the -20% condition, the combined elevation was approximately 2722.29 m, only 0.09 m above the wave-wall crest. The critical wind-wave run-up corresponding to the wave-wall crest for this scenario is approximately 2.48 m, about 22.7% below the adopted value. Thus, the overall classification is stable within the tested sensitivity range, although the margin for this particular scenario remains small.

The mean-based screening indicates that the $100 \times 10^4 \text{ m}^3$ scenarios remain below both reference crest elevations under the two tested reservoir levels. For the $200 \times 10^4 \text{ m}^3$ scenario, the mean combined elevation remains below the reference elevations at 2710.0 m, whereas the 2715.0 m condition indicates a potential exceedance. For the conservative $500 \times 10^4 \text{ m}^3$ scenario, the mean combined elevations are above the reference elevations under both reservoir-level conditions; however, these differences should be interpreted as scenario-screening indicators rather than statistically quantified overtopping margins.

The baseline calculation and the wind-wave-run-up sensitivity analysis consistently indicate that the $100 \times 10^4 \text{ m}^3$ scenarios remain below both reference crest elevations under the two tested reservoir levels. The $200 \times 10^4 \text{ m}^3$ scenario at 2710.0 m also remains below the reference elevations throughout the tested sensitivity range. In contrast, the $200 \times 10^4 \text{ m}^3$ scenario at 2715.0 m and both $500 \times 10^4 \text{ m}^3$ scenarios remain above the reference elevations within the adopted $\pm 20\%$ variation in wind-wave run-up. Nevertheless, the $200 \times 10^4 \text{ m}^3$

scenario at 2715.0 m is close to the wave-wall threshold at the lower sensitivity bound and should therefore be interpreted as a marginal overtopping scenario rather than a deterministic safety-margin exceedance.

At the reservoir water level of 2710.00 m, under the condition of a landslide instability volume of $200 \times 10^4 \text{ m}^3$, the total wave elevation at the spillway was $2710 + 5.61 + 3.21 = 2718.82 \text{ m}$, which was lower than both the dam crest elevation and the wave-wall crest elevation. Therefore, no overtopping risk was identified for this condition. However, when the landslide instability volume increased to $500 \times 10^4 \text{ m}^3$ at the same water level, the total wave elevation at the spillway was $2710 + 10.17 + 3.21 = 2723.38 \text{ m}$. For the $500 \times 10^4 \text{ m}^3$ scenario at a reservoir level of 2710.0 m, the total wave elevation calculated using the mean measured spillway wave height was 2723.38 m. This value corresponds to a nominal exceedance of 1.18 m above the wave-wall crest under the considered scenario. Because the repeated tests were not designed to establish a formal statistical uncertainty distribution, the 1.18 m exceedance should not be interpreted as an exact deterministic safety margin. Rather, it indicates that the unfavourable large-volume scenario may produce a combined wave elevation close to or above the available wave-wall elevation and therefore requires specific engineering attention.

At the reservoir water level of 2715.0 m, under the condition of a landslide instability volume of $200 \times 10^4 \text{ m}^3$, the total wave elevation at the spillway was $2715 + 4.72 + 3.21 = 2722.93 \text{ m}$, and that at the power station intake was $2715 + 4.47 + 3.21 = 2722.68 \text{ m}$. Both values exceeded the wave-wall crest elevation of 2722.2 m, indicating a potential overtopping risk at the spillway and power station intake. The overtopping assessment is based on representative arithmetic mean wave heights obtained from the repeated engineering tests and the project-design wind-wave run-up of 3.21 m. It should therefore be interpreted as a mean-based scenario-screening calculation rather than a probabilistic estimate of overtopping. The repeated tests were not designed to establish statistical confidence intervals or exceedance probabilities, and the calculated differences between the mean combined elevation and the dam or wave-wall crest should not be regarded as uncertainty-adjusted safety margins. This qualification is particularly important for cases in which the nominal mean exceedance is relatively small. Such cases are interpreted as potential or marginal exceedance scenarios requiring further engineering verification rather than as evidence of certain prototype overtopping. The actual prototype uncertainty additionally includes scale effects, landslide-volume and motion uncertainty, reservoir conditions, and the adopted wind-wave run-up.

Although the adopted wind-wave run-up is documented in the approved project report and has been verified with the relevant project technical personnel, its original design wind speed, effective fetch, return period, and intermediate calculation parameters are not available in the report version accessible to the authors. Consequently, the present study does not independently recalculate the 3.21 m value. This limitation should be considered when interpreting the combined elevations, which are used for conservative engineering screening rather than as independently reproducible wind-wave design calculations. Therefore, it is recommended that the reservoir water level be controlled at 2710.00 m during the initial impoundment and operation stages, and that the potential single landslide water-entry volume be strictly limited to within $200 \times 10^4 \text{ m}^3$. If the reservoir water level needs to be raised to 2715.00 m, wave-dissipation facilities or wave-wall-heightening measures should be considered near the spillway and power station intake.

5. Conclusions

In this study, a 1:200-scale physical model was used to investigate the generation, propagation, opposite-bank run-up, and engineering effects of landslide-generated impulse

waves associated with the H1 landslide at Yangqu Hydropower Station. The analysis focused on geologically defined partial-instability scenarios and a conservative near-total-instability envelope under representative reservoir operating levels.

(1) The physical model reproduced the principal processes of landslide entry, impulse-wave generation, propagation, reflection, and opposite-bank run-up in the narrow and curved reservoir reach. Across the selected scenario volumes, the maximum wave height near the water-entry area increased from 3.10–3.27 m for the $100 \times 10^4 \text{ m}^3$ scenario to 4.50–4.64 m for the $200 \times 10^4 \text{ m}^3$ scenario and 7.18–7.21 m for the $500 \times 10^4 \text{ m}^3$ scenario. In front of the hydraulic structures, the maximum recorded wave height increased from 2.28 m to 5.61 m and further to 10.75 m. The 100×10^4 and $200 \times 10^4 \text{ m}^3$ cases represent characteristic partial-instability scenarios associated with Zones I and II, whereas the $500 \times 10^4 \text{ m}^3$ case represents a conservative near-total-instability envelope. These scenarios should therefore be interpreted as consequence-assessment bounds rather than equally probable or exact forecasts of future failure volume.

(2) The impulse-wave response exhibited pronounced spatial and temporal variability during downstream propagation. For the $200 \times 10^4 \text{ m}^3$ scenario, the wave heights at the spillway, dam center, and power station intake were 5.61, 3.31, and 4.64 m, respectively, at a reservoir level of 2710.0 m, and 4.72, 3.07, and 4.47 m, respectively, at 2715.0 m. The maximum response did not always occur at the first crest because bank reflection, curved-channel confinement, and wave superposition could amplify subsequent crests. The spillway and power station intake should therefore be treated as priority locations for wave monitoring and local protection, and early-warning procedures should consider the complete wave train rather than only the first-wave arrival.

(3) The mean-based scenario screening varied substantially with the adopted instability-volume scenarios and reservoir operating levels. For the $100 \times 10^4 \text{ m}^3$ scenarios, the calculated mean combined elevations remained below the dam crest and wave-wall crest. For the $200 \times 10^4 \text{ m}^3$ scenario, the mean-based elevation remained below both reference elevations at 2710.0 m, whereas the screening at 2715.0 m indicated a potential marginal exceedance. For the conservative $500 \times 10^4 \text{ m}^3$ near-total-instability scenario, the mean-based combined elevations were above the reference elevations under both reservoir-level conditions. These results are based on representative arithmetic mean responses and should therefore be interpreted as scenario-based warning indicators rather than statistically quantified overtopping probabilities or uncertainty-adjusted safety margins. A deterministic sensitivity analysis with the adopted wind-wave run-up varied by $\pm 20\%$ did not change the overtopping classification of the six representative scenarios, although the $200 \times 10^4 \text{ m}^3$ scenario at 2715.0 m remained close to the wave-wall threshold at the lower sensitivity bound.

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Data Availability Statement: The data presented in this study are available on request from the corresponding author. The data are not publicly available because they include engineering survey, experimental, and project-related information associated with a major hydropower project and are subject to confidentiality and data-sharing restrictions imposed by the project owner.

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