

Article

Study of Response Characteristics and Strength Parameter Evaluation of Water Intake Tower Under Different Amplitude Modulation Modes

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Abstract

This study selected a simplified water intake tower model, simplifying the physical structure into a cantilever model, and MATLAB software (R2010b) was used to develop a rapid seismic response analysis program for the structure. Thirty near-fault pulse and non-pulse ground motions were selected as the input ground motions for this analysis. Peak ground velocity (*PGV*) was used as the intensity parameter for the ground motions. The acceleration, cross-sectional rotation, and lateral curvature of the simplified water intake tower model were calculated for ground motions modulated with different *PGA* amplitudes. The acceleration, maximum shear force, and cross-sectional rotation of the simplified water intake tower model were also calculated for ground motions modulated with improved effective peak acceleration (*IEPA*) and improved effective peak velocity (*IEPV*). The study showed that the seismic response of the simplified water intake tower model for near-fault ground motions modulated with different intensities of *PGV* amplitude modulation was closer to the unmodulated response order. *PGV* as an intensity parameter did not affect the acceleration response amplification factor of the water intake tower and hoist chamber. The AC coefficient indicated that *PGV* was less suitable for pulse-type earthquake amplitude modulation than *PGA*. Compared with *PGA* amplitude modulation, *IEPA* amplitude modulation is more suitable for pulse-type seismic motion, while *IEPV* amplitude modulation has less impact on pulse-type seismic motion.

Keywords: water intake tower; response characteristics; strength parameter; amplitude modulation modes; seismic performance

Received: 7 November 2025

Revised: 9 January 2026

Accepted: 28 January 2026

Published: 4 February 2026

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1. Introduction

In recent years, near-fault earthquakes have occurred frequently around the world, causing significant property damage and loss of life. Near-fault seismic ground motion causes significant structural responses, beginning with resonance phenomena and localized damage, which may lead to overall structural bending deformation or even collapse. Precisely because of these specific seismic traits, various engineering structures often struggle to appropriately resist seismic damage under near-fault earthquake events, highlighting the gap between existing seismic design theories and practical applications. Con-

sequently, the destructive nature of near-fault earthquakes has garnered widespread attention. The seismic resistance of structures subjected to near-fault earthquakes has also become a focus of recent research, with the damage to industrial structures caused by near-fault earthquakes also drawing widespread attention.

Numerous researchers have paid long-term and continuous attention to the seismic performance of structures under near-fault ground motions and have conducted an abundance series of research works.

In contrast to ordinary earthquake waves, near-fault earthquake motion exhibits characteristics of longer pulse periods, substantial velocity amplitudes, and distinct pulses, which can induce considerable dynamic impacts on structures. Yue Maoguang et al. [1], Wang Changlong [2], and Amiri et al. [3] used different types of steel frame structures and steel structure analysis to prove the dynamic effects of near-fault earthquake motion on different types of steel structures. Liao et al. [4], Li Shuang and Xie Lili et al. [5], and Zhang Bing [6] used reinforced concrete structures and analyzed the seismic responses of structures to two different earthquake motions using computer software. The analysis results indicate that the seismic response to pulse-type earthquake motion is intensified, thereby augmenting the potential for structural damage. This suggests that the seismic performance and earthquake mitigation strategies for structures in proximity to fault lines must be more stringent. Juin-Fu et al. [7], and Alavi and Krawinkler [8] calculated the relationship between the shear response and displacement response of different structures due to pulse earthquake motion by inputting equivalent pulse earthquake motion. Their research results demonstrate that the structural earthquake reaction is directly related to the velocity pulse and period of the pulse earthquake.

Regarding the impact of near-fault ground vibrations on hydraulic structures, academics have conducted several investigations. For instance, Bayraktar et al. [9] evaluated the seismic performance of several types of dams under such motions. Their results show that the seismic performance requirements of various dam bodies under the action of near-fault earthquake motion are much higher than those for far-field earthquake motion. Zhang Sherong et al. [10] and Wang and Alembagheri [11] evaluated the effects of several forms of near-fault ground vibrations on gravity dams. Their results demonstrated that pulse ground motions have a substantial influence on the seismic response of gravity dams.

Regarding the seismic resistance of water intake towers under earthquakes, domestic and foreign scholars have also performed a lot of work. Cheng Hankun et al. (2011) [12] used the finite element program to calculate the seismic response of a water intake tower under three-dimensional earthquakes. Lefrancois et al. (2014) [13] calculated the base shear force based on a simplified water intake tower model using finite element software, and used this research result to evaluate the seismic resistance of the water intake tower construction. Li Ning et al. (2014) [14] used time–distance analysis to examine the seismic response of Zipingpu Reservoir's intake tower to the ground shaking of the Wenchuan mainshock and response spectral analysis to examine the seismic response of Zipingpu to the ground shaking of the Wenchuan aftershock, comparing the results of the two methods. Dang Kangning et al. (2016) [15] employed finite element software to investigate the seismic performance of the hoist chamber at the top of a water intake tower. Alembagheri (2016) [16,17] studied the influence of a dam on the seismic performance of two water intake towers. The findings demonstrated that the dam had a certain influence on the seismic response of the water intake tower. Dang et al. (2017) [18] calculated the influence of different foundations on the seismic response of water intake towers based on the artificial boundary foundation theory using a finite element program. Zhang et al. (2018) [19] studied the influence of the duration and frequency of earthquake motion on the seismic per-

formance of a water intake tower structure. The results showed that the duration of earthquake motion had a significant effect on the seismic performance of the water intake tower structure. Pang et al. [20] and Zou et al. (2019) [21] computed the real-time slip of a dam under the impact of near-fault earthquake motion. The findings demonstrated that near-fault pulse earthquake motion would have a major impact on the slip of a rockfill dam. Altunisik (2022) [22] investigated the influence of near-fault seismic ground motion on the dynamic seismic response of gravity dams, incorporating interactions among the dam body, reservoir, and foundation. The findings indicate that near-fault impulsive earthquakes affect the dynamic seismic responses of gravity dams, necessitating consideration in engineering design. Zhai (2022) [23] researched the actual dynamic damage characteristics under near-fault seismic ground motion, proposing an assessment model for dam body damage under three different types of such motion. The results suggest that impulsive near-fault seismic ground motion causes more pronounced damage. Wu (2023) [24] investigated the dynamic response of liquid storage tanks subjected to near-fault ground motions and pointed out that the existing seismic design provisions for such conditions are incomplete and require further supplementation. Choudhury (2025) [25] explored the interaction between earth-rockfill dams and reservoirs under near-fault ground motions, demonstrating that pulse-like seismic excitations significantly amplify the associated earthquake-induced dynamic responses.

In conclusion, while researchers both domestically and internationally have conducted extensive studies on the damage that near-fault ground shaking causes to engineering structures, seismic response analysis of intake tower structures typically focuses on either the analysis of the seismic response of the intake towers under the action of artificially simulated ground shaking or the seismic response of the intake towers under the action of general near-field earthquakes.

As a result, a water intake tower's structure and hoist chamber were simplified in this study, and a self-developed MATLAB program tool was employed to rapidly examine the seismic reaction of the structure. The study examined the water intake tower's seismic response to earthquake motions, which were influenced by intensity factors such as *PGV* (peak ground velocity), *IEPA*, and *IEPV* (improved effective peak velocity). A rapid seismic response dynamic analysis program was developed to create a simplified model of the water intake tower and hoist chamber in order to perform a rapid seismic response analysis of the water intake tower construction. This rapid seismic response analysis of the simplified model of the water intake tower and hoist chamber was carried out after thirty near-fault earthquake motion data were amplitude-modulated by the three intensity parameters of *PGV*, *IEPA*, and *IEPV*. The selected 30 seismic records originate from two dip-slip earthquakes (the 1994 Northridge earthquake and the 1999 Chi-Chi earthquake). For example, no ground motion with fling-step effects was recorded in the Northridge earthquake; two fling-step impulsive records were obtained in the 1999 Kocaeli, Turkey earthquake (strike-slip earthquake), and about ten were recorded in the Chi-Chi earthquake. Therefore, we selected ten fling-step records from the Chi-Chi earthquake. To consider the representative Northridge records, we selected five forward-directivity records as well as five non-pulse records. Because the real fling-step records are scarce for strike-slip earthquakes in the world, currently this work does not currently consider the near-fault ground motions from strike-slip earthquakes. However, it would be very useful to investigate the seismic responses of intake towers under the effects records of strike-slip earthquakes in the future. The mechanism by which intensity parameter alterations affect the impulse responses of water intake towers and hoist chambers was identified by evaluating the effects of various amplitude modulation techniques on the response characteristics of a water intake tower.

2. Research into Rapid Analysis Method for Seismic Response Characteristics of Water Intake Tower Based on Simplified Mechanical Model

Finite element modeling for analyzing the seismic response of a water intake tower is complex and cumbersome due to the need to reflect the stress and deformation characteristics of the water intake tower under real engineering conditions. Furthermore, when analyzing the seismic effects of large-scale earthquake waves, the calculations are very time-consuming, making it inconvenient for engineering applications. Therefore, the water intake model was simplified from a mechanical perspective, and a rapid dynamic analysis program for seismic response was developed. This program allows a quick, early assessment of the water intake tower's seismic reactivity during near-fault ground movements by accepting a large number of seismic waves as input. The results can be used to predict how the water intake tower's seismic response will change during near-fault ground motions.

The program is differently segmented according to the change in the cross-section of the intake tower, which is wider at the bottom and narrower at the top. Based on its primary load characteristics, the actual structure is simplified into a cantilever structure combining a curved cantilever beam and a shear beam that accounts for shear deformation. A simplified mechanical diagram is shown in Figure 1. The natural vibration period of this simplified model, $T = 0.996$ s, is close to the finite element model of the intake tower based on actual engineering.

The computation of the effective shear-bearing area for each cross-section elucidates the effects of shear pressures on the tower structure. A program is developed utilizing the robust simulation capabilities of MATLAB software [26] and incorporating the Newmark- β method for dynamic time-history analysis to calculate essential parameters, including maximum cross-sectional acceleration, cross-sectional rotation angle, and maximum shear force of the water inlet tower.

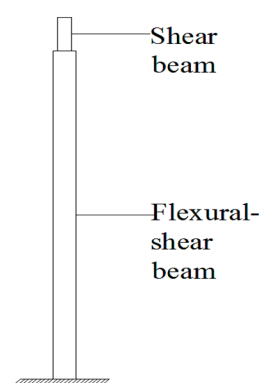


Figure 1. Simplified model of the intake tower.

A water intake tower can be simulated using plane beam elements. Figure 2 is the force diagram of the plane beam element, and Figure 3 is the deformation diagram of the plane beam element.

In Figure 2, M_i , F_{Qi} , F_{Ni} are the bending moment, shear force, and axial force at the unit i end, and the positive directions are as shown in the figure. In Figure 3, u_i , v_i , θ_i are the horizontal displacement, vertical displacement, and angular displacement at the unit i end, and the positive directions are as shown in the figure.

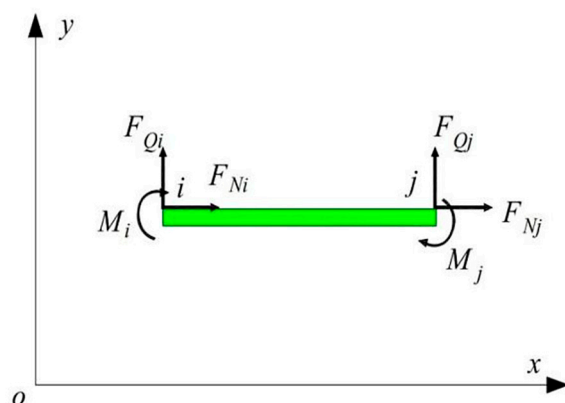


Figure 2. Force of plane beam element.

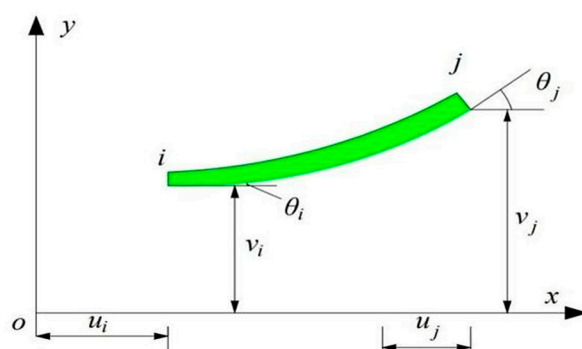


Figure 3. Deformation of plane beam element.

The displacement matrix of the i end can be expressed as follows:

$$\{\delta_i\}_{3 \times 1} = [u_i \ v_i \ \theta_i]^T \quad (1)$$

The i end force matrix can be expressed as follows:

$$\{F_i\}_{3 \times 1} = [F_{Ni} \ F_{Qi} \ M_i]^T \quad (2)$$

The displacement matrix of the j end can be expressed as follows:

$$\{\delta_j\}_{3 \times 1} = [u_j \ v_j \ \theta_j]^T \quad (3)$$

The j end force matrix can be expressed as follows:

$$\{F_j\}_{3 \times 1} = [F_{Nj} \ F_{Qj} \ M_j]^T \quad (4)$$

The displacement vector of the node can be obtained as follows:

$$\{\delta\}_{6 \times 1}^e = \begin{Bmatrix} \delta_i \\ \delta_j \end{Bmatrix}^e \quad (5)$$

The force vector at the node is as follows:

$$\{F\}_{6 \times 1}^e = \begin{Bmatrix} F_i \\ F_j \end{Bmatrix}^e \quad (6)$$

Since the main deformation of the water intake tower is bending, the shear deformation is not directly considered in the rapid seismic response analysis program of the water intake tower. Instead, the influence of shear deformation is reflected by modifying

the stiffness matrix of the plane beam element. The stiffness matrix of the plane beam element is modified as follows:

$$[k] = \begin{bmatrix} \frac{EA}{l} & 0 & 0 & -\frac{EA}{l} & 0 & 0 \\ 0 & \frac{12EI}{l^3(1+\Phi)} & \frac{6EI}{l^2(1+\Phi)} & 0 & -\frac{12EI}{l^3(1+\Phi)} & \frac{6EI}{l^2(1+\Phi)} \\ 0 & \frac{6EI}{l^2(1+\Phi)} & \frac{(4+\Phi)EI}{l(1+\Phi)} & 0 & -\frac{6EI}{l^2(1+\Phi)} & \frac{(2-\Phi)EI}{l(1+\Phi)} \\ -\frac{EA}{l} & 0 & 0 & \frac{EA}{l} & 0 & 0 \\ 0 & -\frac{12EI}{l^3(1+\Phi)} & -\frac{6EI}{l^2(1+\Phi)} & 0 & \frac{12EI}{l^3(1+\Phi)} & -\frac{6EI}{l^2(1+\Phi)} \\ 0 & \frac{6EI}{l^2(1+\Phi)} & \frac{(2-\Phi)EI}{l(1+\Phi)} & 0 & -\frac{6EI}{l^2(1+\Phi)} & \frac{(4+\Phi)EI}{l(1+\Phi)} \end{bmatrix} \quad \text{Symmetry} \quad (7)$$

where $\Phi = \frac{12EI}{GA_s l^2}$, A_s is the effective shear area.

The beam element force vector can be expressed as follows:

$$\{F\}_{6 \times 1}^e = [k]_{6 \times 6}^e \{\delta\}_{6 \times 1}^e \quad (8)$$

Under near-fault seismic ground motions, the response of the intake tower is characterized by a relatively large contribution of shear deformation. The influence of shear deformation is represented by the shear influence coefficient Φ , which is introduced to quantify the relative contribution of shear deformation to the total deformation of the tower. The shear influence coefficient for each section is obtained by calculating the corresponding effective shear area for each section. The intake tower is segmented based on its cross-sectional variations: the tower is divided into 23 sections, including 2 sections (2.5 m high), 3 sections (3.67 m high), 3 sections (3 m high), and 15 sections (4.2 m high). The hoist compartment is divided into two sections, one with a 14 m high section and the other with a 5.5 m high section. The intake tower is divided into 25 sections in total. The rapid seismic response analysis program for the intake tower can be used to calculate the maximum acceleration, maximum section rotation angle, and maximum shear force response at each section.

3. Response Characteristics and Strength Parameter Evaluation Analysis of PGV Intake Tower After Amplitude Modulation

3.1. PGV AM

Peak ground acceleration (PGA) and peak ground velocity (PGV) are extensively used intensity metrics in earthquake engineering. PGA predominantly reflects the instantaneous inertial forces caused by ground motion, whereas PGV more adequately characterizes the energy input of seismic excitation and its influence on structural displacement and damage [27,28]. To investigate the effect of near-fault ground motions modulated by the PGV intensity parameter on the rapid dynamic analysis response characteristics of a simplified water intake tower structure, the PGV of 30 near-fault ground motions were uniformly modulated to 18 cm/s, 28 cm/s, and 36 cm/s. Because the accuracy of ground motions is significantly affected by PGV amplitude modulation, the original ground motions were subjected to a high-pass filter with a filter frequency of $f_c = 0.05$ Hz using

the Butterworth filter method in SeismoSignal software (2016) before *PGV* amplitude modulation. Using the modulated near-fault ground motions as input, the impact of *PGV* amplitude modulation on the rapid seismic response of the water intake tower was analyzed. The applicability of the *PGV* intensity parameter as an amplitude modulation indicator in water intake tower structural analysis was also discussed and evaluated. The seismic waves after *PGV* amplitude modulation are shown in Figure 4.

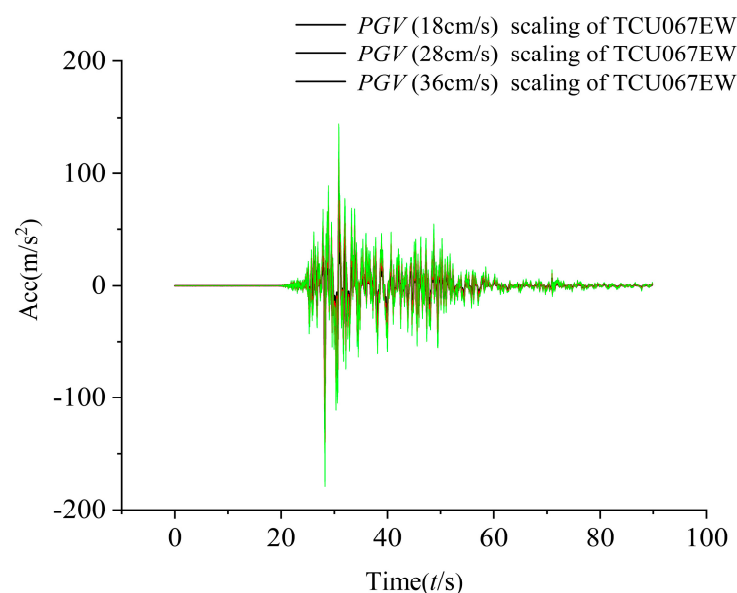


Figure 4. Acceleration time histories of impulsive record TCU67EW with three kinds of *PGV* scaling.

3.2. Acceleration and Shear Response After *PGV* Amplitude Modulation

The maximum acceleration response and maximum shear response of the simplified structure of the water intake tower and hoist chamber after modulation are obtained by modulating near-fault ground motions with various *PGV* amplitudes that are input into the rapid calculation program for the rapid dynamic seismic response of the water intake tower, as illustrated in Figures 5 and 6, respectively. Table 1 displays the acceleration amplification factors and seismic response coefficients for the hoist chamber and intake tower following *PGV* amplitude modulation.

Table 1. Amplification coefficients of acceleration responses for hoist chamber and the amplifying coefficient of near-fault impulsive ground motion of simplified model of intake tower and hoist chamber under near-fault ground motions with three kinds of *PGV* scaling.

Parameter	Fling-Step Pulse			Fling-Step Pulse			Non-Pulse	
	Acceleration (m/s ²)	Ratio	AC	Acceleration (m/s ²)	Ratio	AC	Acceleration (m/s ²)	Ratio
Top acceleration of hoist chamber (<i>PGV</i> = 18 cm/s)	9.61	3.65	0.643	5.39	3.49	0.361	14.95	3.05
Top acceleration of the intake tower (<i>PGV</i> = 18 cm/s)	2.63		0.537	1.54		0.314	4.90	
Top acceleration of hoist chamber (<i>PGA</i> = 28 cm/s)	14.95	3.65	0.642	8.39	3.49	0.361	23.27	3.05
Top acceleration of the intake tower (<i>PGV</i> = 28 cm/s)	4.10		0.537	2.40		0.315	7.63	

Top acceleration of hoist chamber ($PGV = 36 \text{ cm/s}$)	19.23	0.642	10.79	0.361	29.93	
Top acceleration of the intake tower ($PGV = 36 \text{ cm/s}$)	5.27	3.65	0.537	3.49	0.315	9.82

Note: AC is the near-fault pulse earthquake response coefficient.

The acceleration response rises with increased PGV amplitude modulation, as seen in Figure 5. After PGV amplitude modulation, the acceleration response is highest for non-pulse ground motion, with amplified acceleration responses observed in the hoist chamber. Table 1 shows that the maximum acceleration amplification occurs for ground motion with a pulse containing a forward directional effect, while the minimum is for ground motion without a pulse.

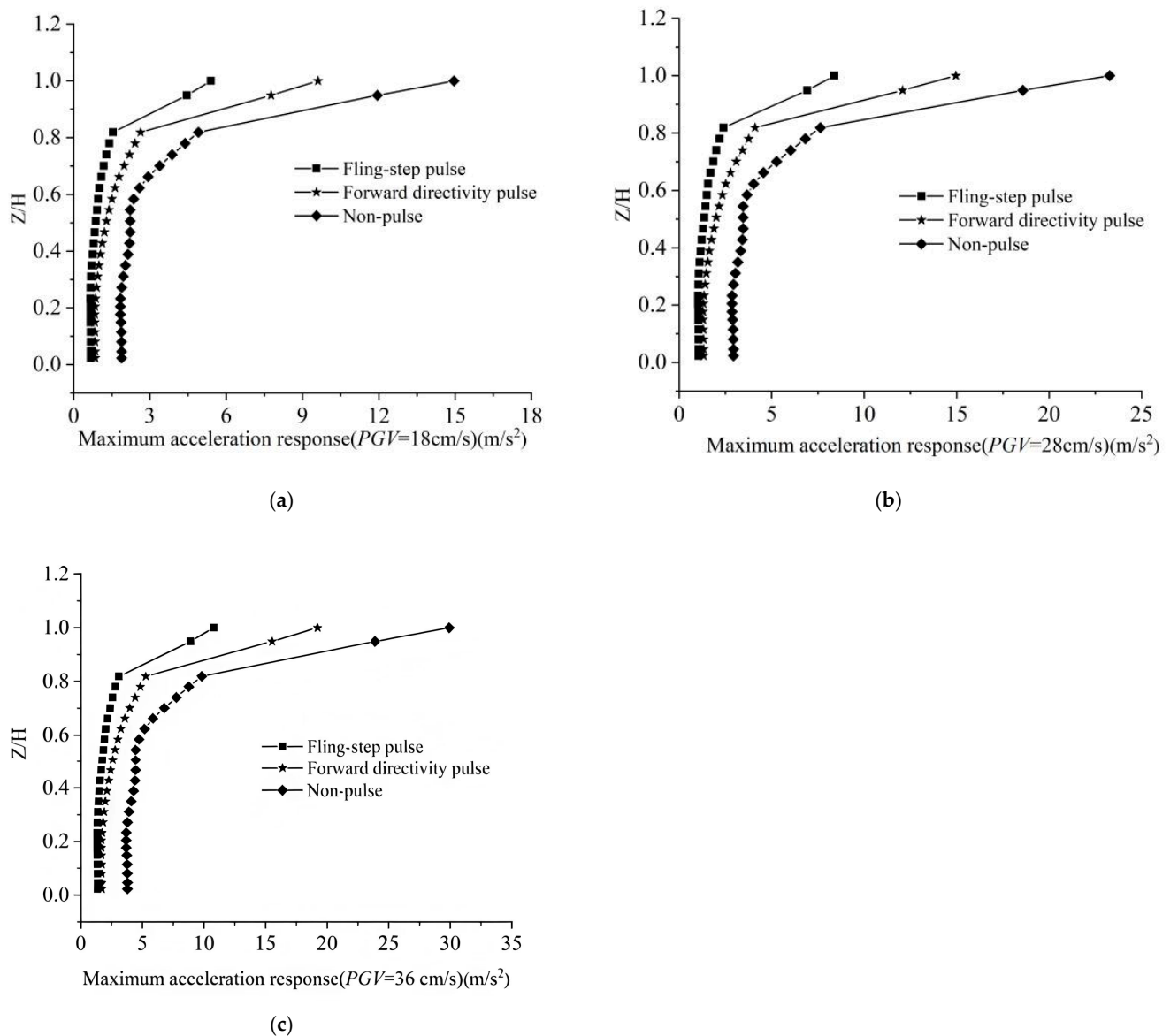


Figure 5. Mean maximum acceleration of simplified model of intake tower and hoist chamber under near-fault ground motions with three kinds of PGV scaling. (a) $PGV = 18 \text{ cm/s}$; (b) $PGV = 28 \text{ cm/s}$; (c) $PGV = 36 \text{ cm/s}$.

Comparing the PGA results with those obtained previously, we can see that the order of the seismic acceleration response changes after PGV amplitude modulation compared

to *PGA* amplitude modulation. Because the amplitude modulation has a smaller effect on the original acceleration peak when the amplitude modulation seismic intensity parameter is *PGV*, the order of the acceleration response is closer to the order of the unmodulated seismic acceleration peaks. *PGV* amplitude modulation also does not affect the acceleration amplification of the gate hoist chamber by pulse-type seismic motion. Compared to *PGA* amplitude modulation, *PGV* amplitude modulation does not affect the amplification factor of the gate hoist chamber. Table 1 shows that the relationship between the acceleration amplification factors does not change for the three amplitude modulation values, and for both pulse-type seismic motions, *PGV* amplitude modulation does not affect the AC coefficient of acceleration.

Figure 6 shows that *PGV* modulation affects the maximum shear response of the simplified water intake tower structure at each segment under the three sets of earthquake motions. The maximum shear response occurs at the base of the water intake tower construction. However, the order of shear response after *PGA* modulation is different. After *PGV* modulation, the shear response is highest for non-pulsed earthquake motions and lowest for near-fault earthquake motions with slip-induced impulses, consistent with the order of acceleration response after *PGV* modulation. After *PGV* modulation, the maximum shear value increases with increasing modulation index value.

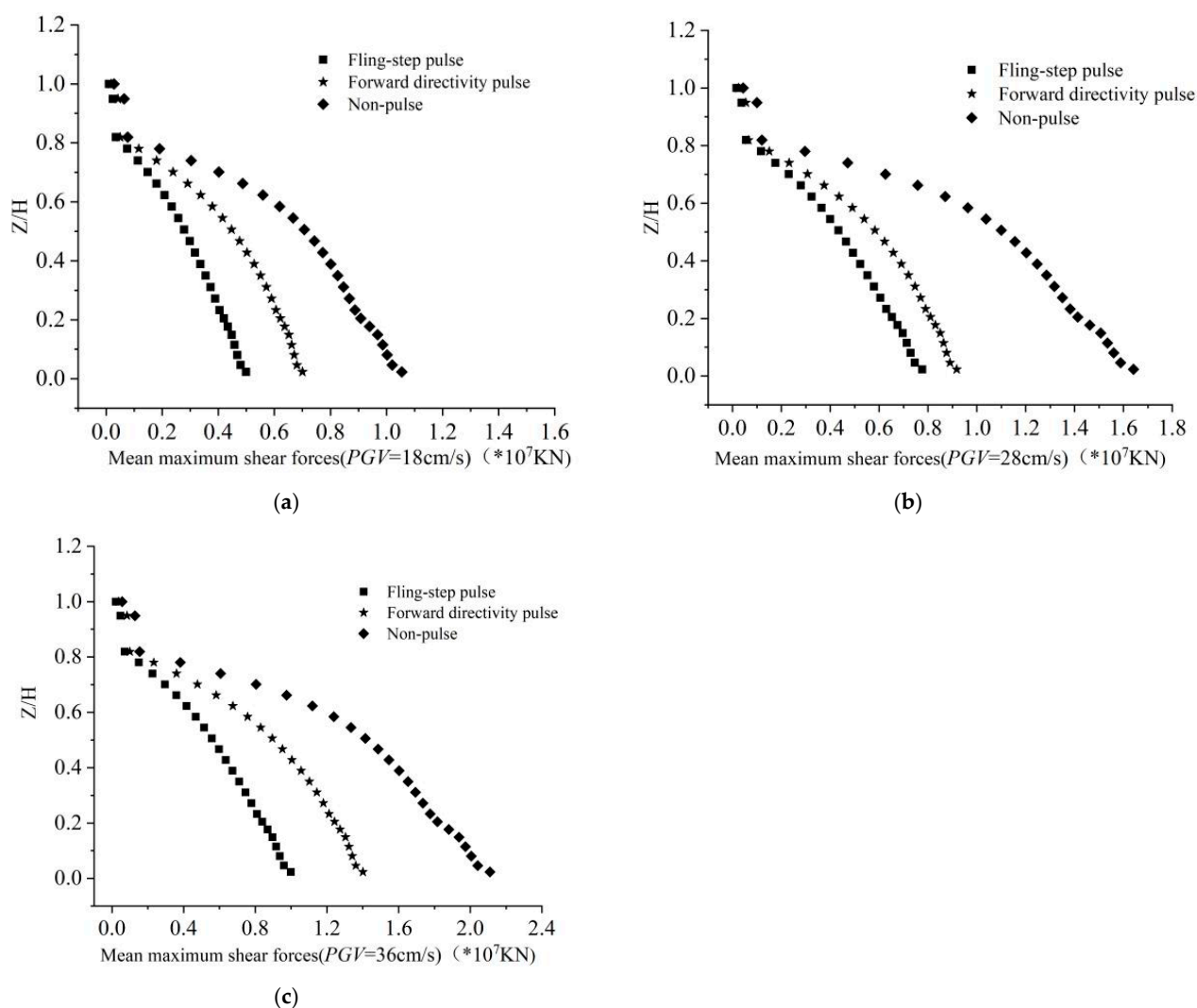


Figure 6. Mean maximum shear forces of simplified model of intake tower and hoist chamber under near-fault ground motions with three kinds of *PGA* scaling. (a) *PGV* = 18 cm/s; (b) *PGV* = 18 cm/s; (c) *PGV* = 18 cm/s.

The shear force and seismic response coefficient at the base of the water intake tower after *PGV* and *PGA* amplitude modulation are contrasted in Table 2. As can be seen from Table 2, for pulse earthquakes, changes in the values of the three different *PGV* amplitude modulation indices do not affect the maximum shear AC coefficient at the bottom of the pulse earthquake with forward directional effects. This phenomenon also appears in the acceleration response after *PGV* amplitude modulation. Pulse earthquakes with slip effects change slightly. *PGA* amplitude modulation has a certain impact on the maximum shear AC coefficient at the bottom of the pulse earthquake, while *PGV* amplitude modulation has almost no effect. This shows that the seismic characteristics of near-fault pulse earthquakes are not greatly impacted by differences in *PGV* amplitude modulation.

Table 2. Comparison of the amplifying coefficient of near-fault impulsive ground motion of the bottom of the shear force of the simplified model of the intake tower under near-fault ground motions with three kinds of *PGV* scaling and *PGA* scaling.

Parameter	Fling-Step Pulse		Fling-Step Pulse		Non-Pulse
	Shear Force ($\times 10^7$ kN)	AC	Shear Force ($\times 10^7$ kN)	AC	Shear Force ($\times 10^7$ kN)
Mean maximum shear forces (<i>PGA</i> = 0.2 g)	1.683	1.436	1.574	1.343	1.172
Mean maximum shear forces (<i>PGA</i> = 0.3 g)	2.516	1.407	2.346	1.312	1.788
Mean maximum shear forces (<i>PGA</i> = 0.4 g)	3.355	1.407	3.127	1.312	2.383
Mean maximum shear forces (<i>PGV</i> = 18 cm/s)	0.700	0.664	0.499	0.473	1.055
Mean maximum shear forces (<i>PGV</i> = 28 cm/s)	0.918	0.559	0.777	0.473	1.641
Mean maximum shear forces (<i>PGV</i> = 36 cm/s)	1.401	0.664	0.999	0.473	2.110

Note: AC is the near-fault pulse earthquake response coefficient.

It can be concluded that, compared with *PGA* amplitude modulation, when the earthquake intensity index is *PGV*, changes in the *PGV* intensity index value have essentially no impact on the maximum shear AC coefficient at the bottom of the slip pulse near-fault ground motion, but have a modest influence on the maximum shear AC coefficient at the bottom of the forward-directional pulse near-fault ground motion, indicating that the influence of the *PGV* intensity index amplitude modulation on the near-fault pulse ground motion is not as obvious as that of the *PGA* intensity index amplitude modulation.

3.3. Evaluation of Cross-Sectional Rotation Angle and Lateral Deformation Curvature Response Characteristics and Strength Parameters After *PGV* Amplitude Modulation

Figures 7–9 show the average maximum cross-sectional rotation angles of the simplified models of the intake tower construction and hoist chamber after *PGV* modulation. As shown in Figures 7–9, the cross-sectional rotation angles of the intake tower segment begin to increase with increasing *PGV*, with higher modulation indexes associated with greater rotation angles. The hoist chamber segment's cross-sectional rotation angles also rise with increasing *PGV* under the three sets of ground movements. Unlike *PGA* modulation, the cross-sectional rotation angles of the intake tower segment after *PGV* modulation are greatest for non-pulse ground motions and smallest for ground motions with slip-induced pulses, consistent with the acceleration response after *PGV* modulation. It is obvious by comparing the seismic responses before and after *PGA* modulation that *PGV* modulation minimizes the near-fault ground motion response with forward-directed pulses.

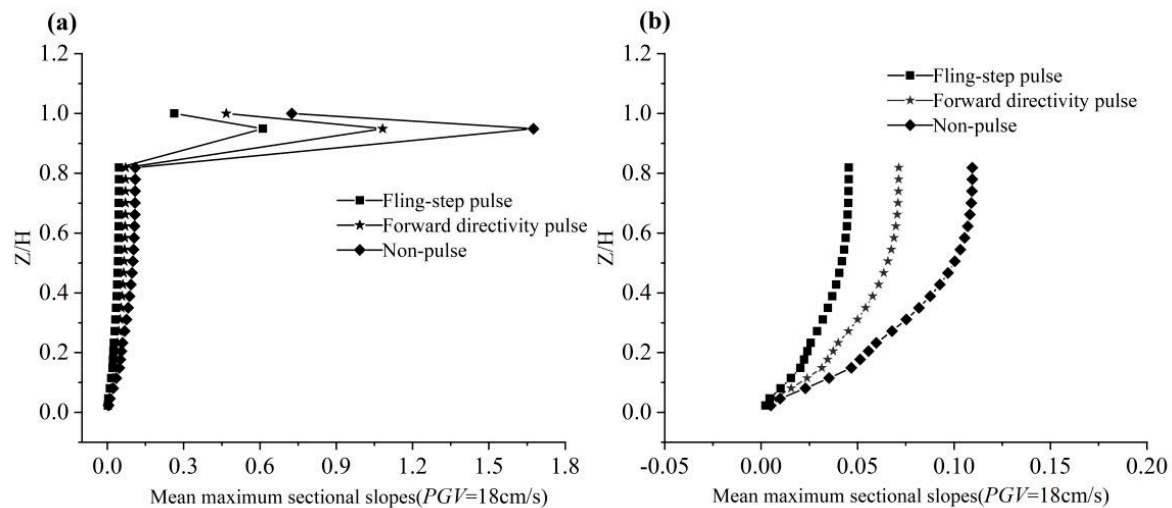


Figure 7. Mean maximum slopes of simplified model of intake tower and hoist chamber under near-fault ground motions with 18 cm/s of PGV scaling. (a) intake tower and hoist chamber; (b) intake tower.

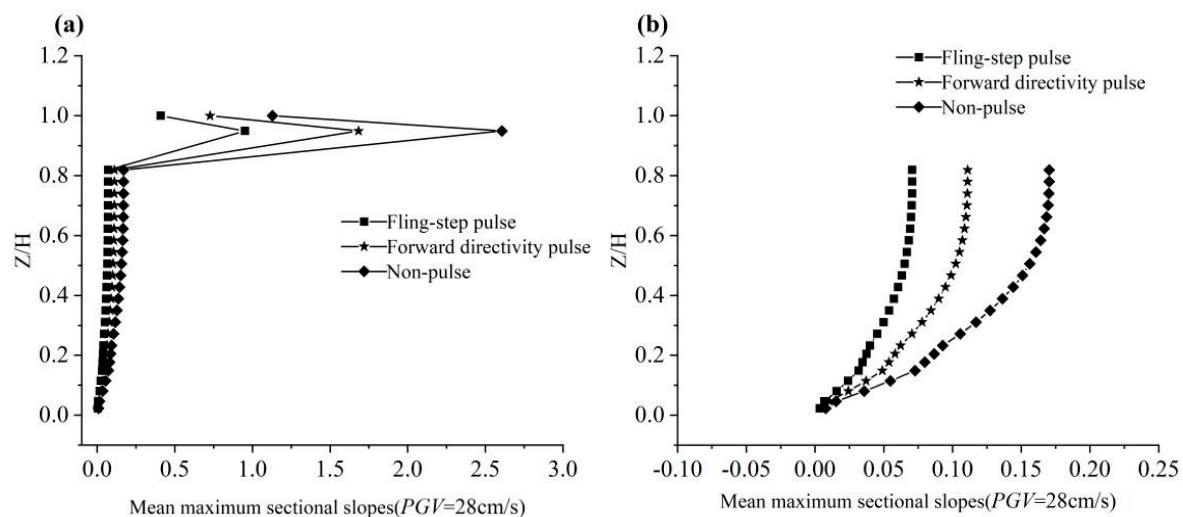


Figure 8. Mean maximum section al slopes of simplified model of intake tower and hoist chamber under near-fault ground motions with 28 cm/s of PGV scaling. (a) intake tower and hoist chamber; (b) Intake tower.

When the angle values of each section along the height of the simplified model of the water intake tower are obtained after different PGV amplitude modulation, the lateral deformation curvature of the simplified structure of the water intake tower along the height can be obtained by differentiating once using formula [29]. Due to the existence of the whip effect, the angle of the gate chamber part changes greatly, so the lateral deformation curvature of the gate chamber is not considered for calculation. As a result, Figures 10–12 demonstrate the variation law of the lateral deformation curvature of the three groups of near-fault earthquake motions along the height of the tower body following various values of PGV amplitude modulation. It can be seen from the figure that as the PGV amplitude modulation value increases, the values of the three groups of lateral deformation curvatures also increase. The largest lateral deformation curvature occurs at the bottom of the water intake tower structure, and the lateral deformation curvature of the tower body decreases as height rises. Different from the unmodulated and PGA modulated ground

motions, after *PGV* modulation, the bottom lateral deformation curvature of the non-pulse ground motion is the biggest, and the near-fault ground motion with slip effect pulse is the smallest. The order is consistent with the order of acceleration response and shear response after *PGV* modulation.

Table 3 shows the average maximum acceleration response, segmental maximum shear force, segmental cross-sectional rotation, and lateral deformation curvature of the water intake tower after *PGV* modulation. Table 3 shows that after varying *PGV* modulation values, the AC coefficients for both the maximum cross-sectional rotation and the maximum lateral deformation curvature show little change. Compared to the AC coefficients under *PGA* modulation, these coefficients are virtually unchanged. This indicates that changes in the *PGV* value, as an earthquake amplitude modulation intensity parameter, have little impact on the seismic response to pulse-type ground motions. Therefore, the *PGV* earthquake intensity parameter is not suitable as an amplitude modulation parameter for near-fault pulse-type earthquakes.

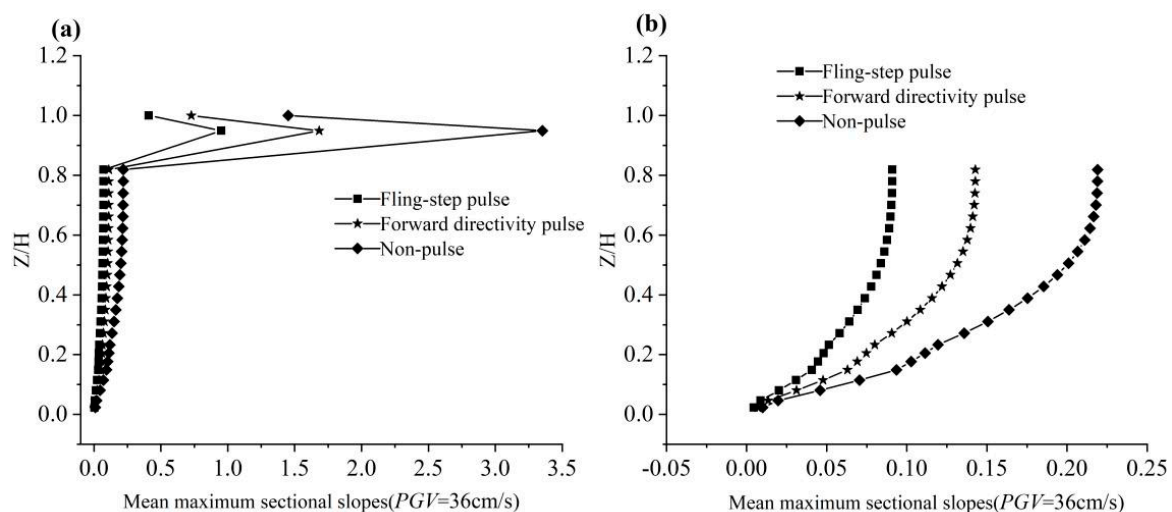
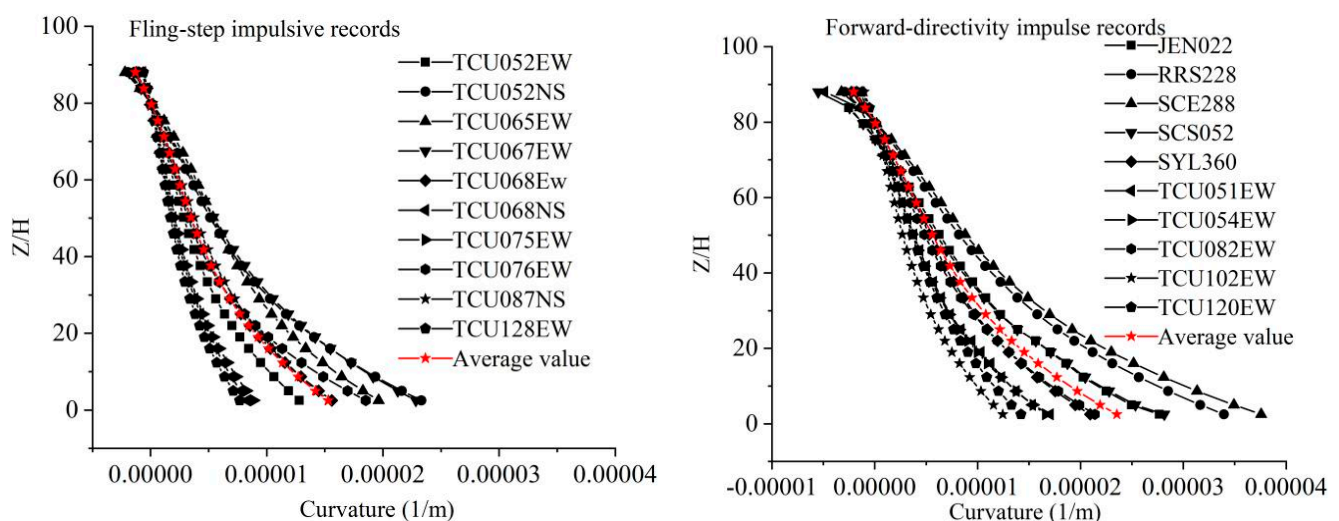


Figure 9. Mean maximum section al slopes of simplified model of intake tower and hoist chamber under near-fault ground motions with 36 cm/s of *PGV* scaling. (a) intake tower and hoist chamber; (b) Intake tower.



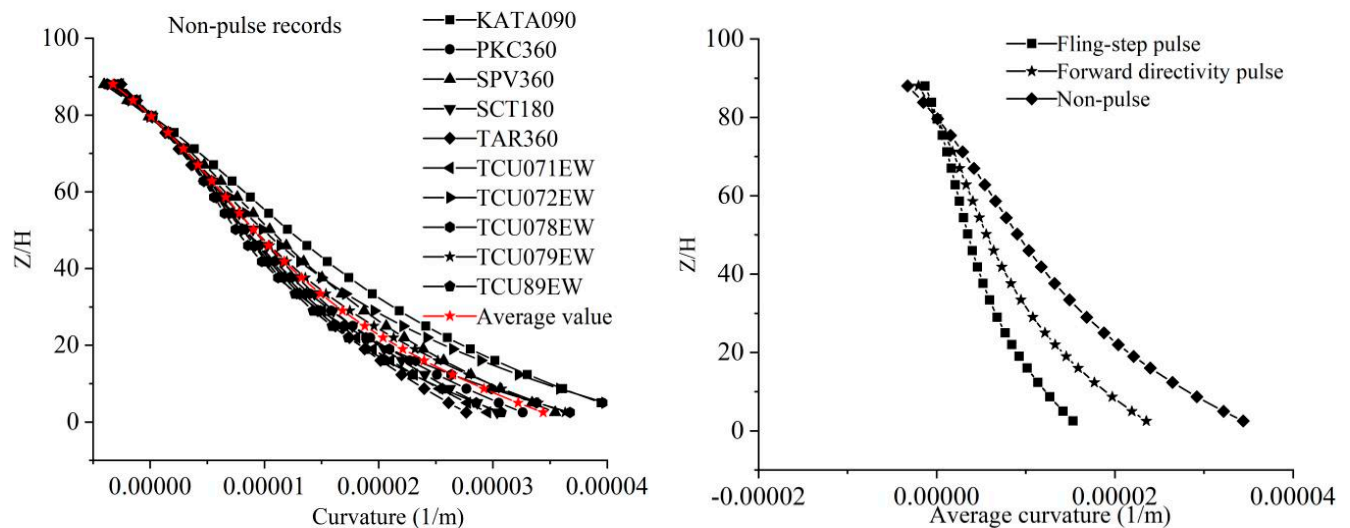


Figure 10. Mean lateral curvatures of intake tower under three groups of near-fault ground motions with 18 cm/s of PGV scaling.

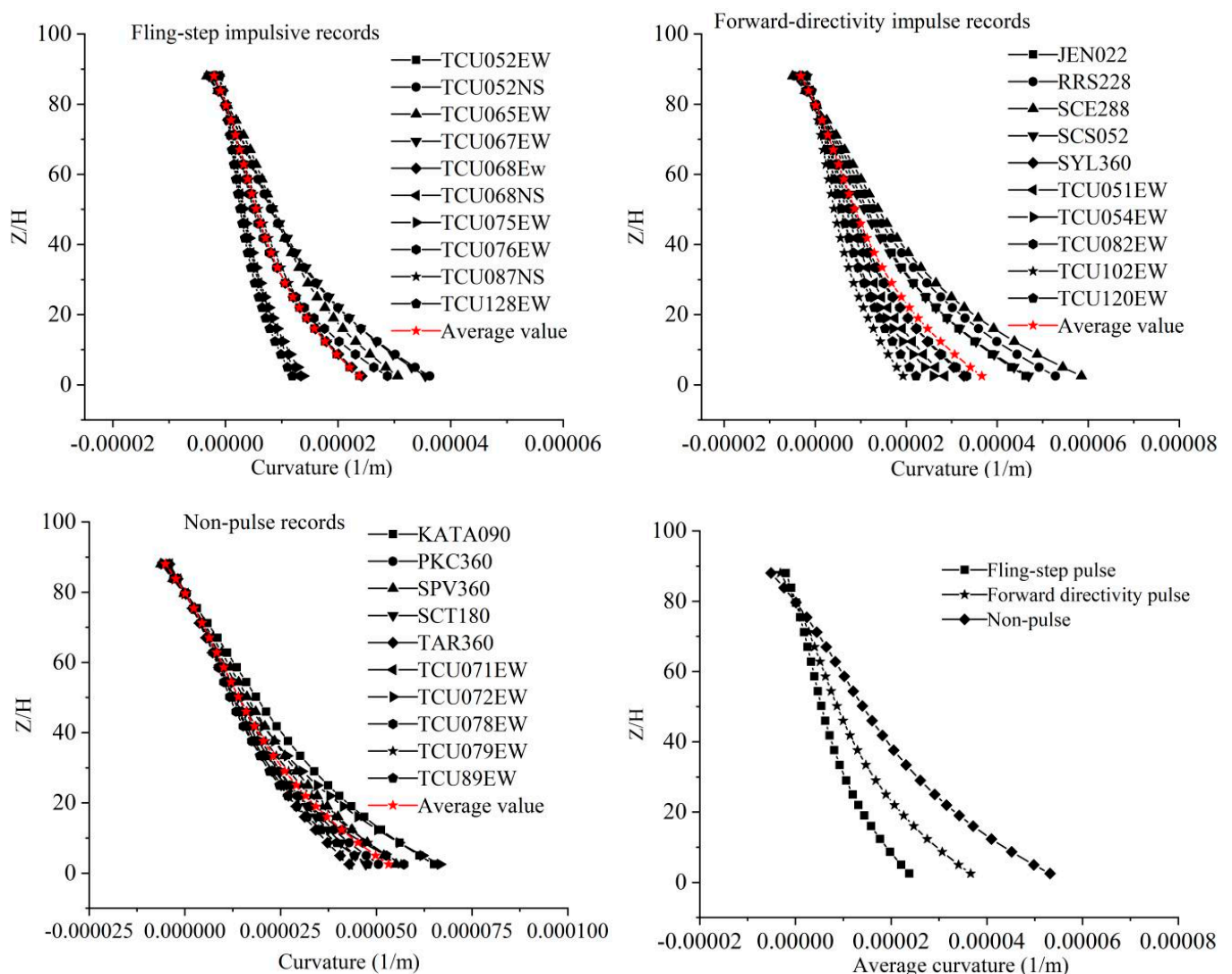


Figure 11. Mean lateral curvatures of intake tower under three groups of near-fault ground motions with 28 cm/s of PGV scaling.

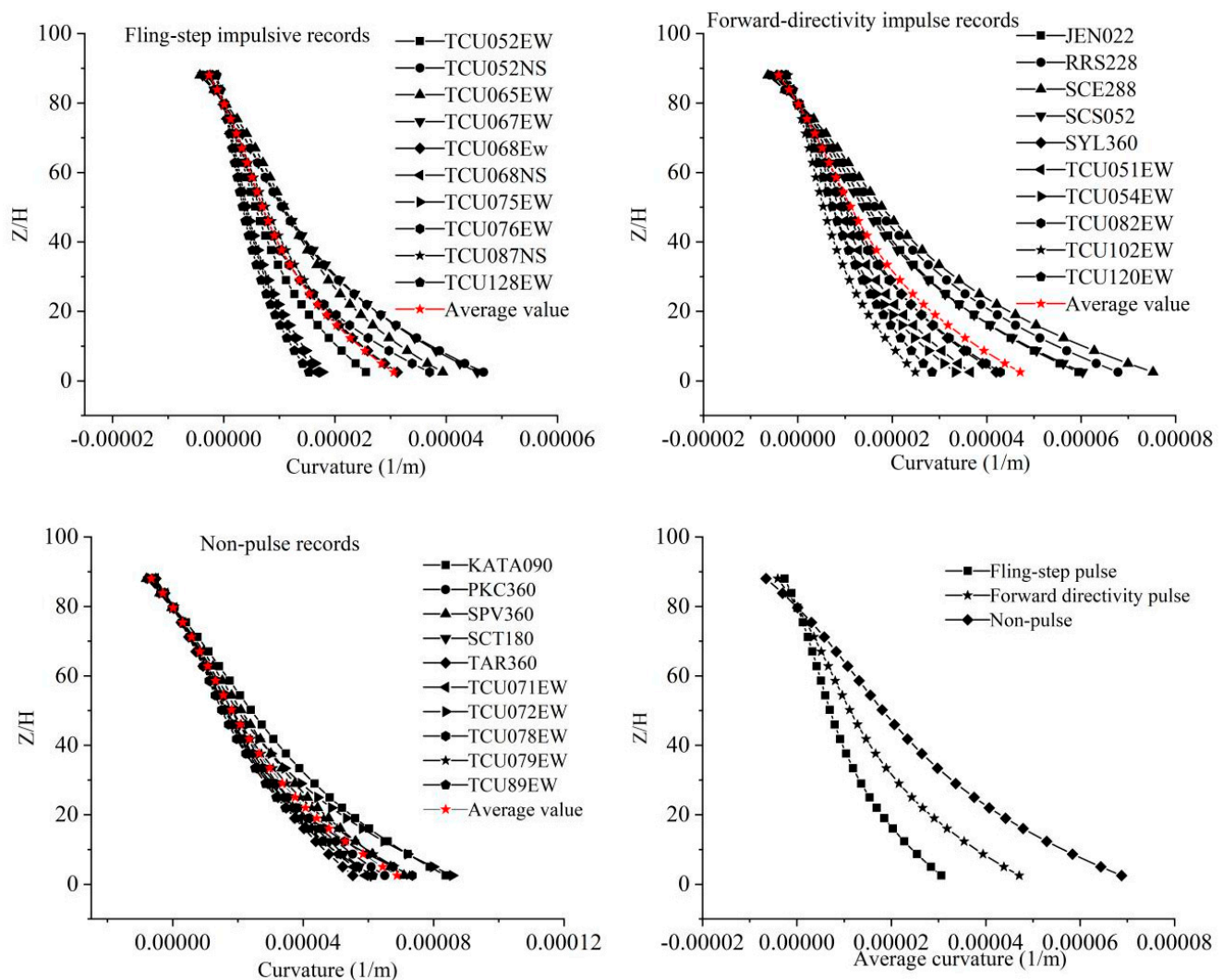


Figure 12. Mean lateral curvatures of intake tower under three groups of near-fault ground motions with 36 cm/s of PGV scaling.

Table 3. Amplifying coefficient of near-fault impulsive ground motion of maximum sectional slopes and lateral curvatures of intake tower under near-fault ground motions with three kinds of PGV scaling.

Parameter	Forward Directivity Pulse		Fling-Step Pulse		Non-Pulse
	AC		AC		
Maximum slope (PGV = 18 cm/s)	0.0714	0.652	0.0455	0.416	0.1095
Maximum Lateral curvature (PGV = 18 cm/s)	2.35×10^{-5}	0.683	1.53×10^{-5}	0.445	3.44×10^{-5}
Maximum slope (PGV = 28 cm/s)	0.111	0.649	0.0707	0.413	0.171
Maximum Lateral curvature (PGV = 28 cm/s)	3.66×10^{-5}	0.688	2.38×10^{-5}	0.477	5.32×10^{-5}
Maximum slope (PGV = 36 cm/s)	0.143	0.653	0.0909	0.415	0.219
Maximum Lateral curvature (PGV = 36 cm/s)	4.71×10^{-5}	0.685	3.06×10^{-5}	0.445	6.88×10^{-5}

Note: AC is the near-fault pulse earthquake response coefficient.

4. Response Analysis of a Simplified Model of a Water Intake Tower Subjected to Near-Fault Ground Motion After IEPA Amplitude Modulation

4.1. Acceleration and Shear Response After IEPA Amplitude Modulation

The effective peak acceleration (*IEPA*) is employed to comprehensively characterize the effective intensity level of seismic acceleration time histories that induce significant dynamic effects on structures, thereby reflecting the equivalent inertial demand under sustained ground motion excitation [30]. The near-fault ground motion data, modulated for intensity indicators according to the *IEPA* method, is sequentially input into the pre-programmed rapid seismic response analysis software. Figures 13 and 14 illustrate the maximum acceleration and maximum shear responses of the simplified structures of the hoist chamber and the water intake tower structure under the *IEPA*-modulated near-fault ground motions. Figure 13 shows that the acceleration of the hoist chamber is amplified by the whip effect after *IEPA* modulation. Unlike the situation where the acceleration responses of the hoist chamber for the near-fault ground motions with a slip-effect pulse and those without a *PGA*-modulated near-fault ground motion are relatively similar, the ground motion with a forward-directed pulse has the highest acceleration response, followed by the ground motion with a slip-effect pulse, and finally the ground motion without a pulse. Figure 14 demonstrates that the ground motion with a forward-directed pulse has the largest shear response of the three near-fault ground movements, as well as the maximum shear force at the base of both pulse-type ground motions, which is much more than that of the ground motion without one.

Table 4 shows the acceleration amplification factors and near-fault pulse earthquake response coefficients of the hoist chamber after *IEPA* modulation. Table 4 shows that the AC coefficient of the near-fault earthquake acceleration response for the forward-directive pulse is 1.542, while the AC coefficient of the near-fault earthquake acceleration response for the slip-effect pulse is 1.155. This indicates that after *IEPA* modulation, the near-fault earthquake acceleration response for the forward-directive pulse has a greater impact on the hoist chamber's seismic response than the near-fault earthquake acceleration response for the slip-effect pulse. Comparing the *PGA* results with Table 4 shows that, similar to the *PGA* modulation results, the near-fault earthquake acceleration amplification coefficient for the hoist chamber is the forward-directive pulse earthquake, indicating that the forward-directive pulse earthquake has the greatest influence on the hoist chamber's structural acceleration response. The forward-directive pulse earthquake acceleration response likewise has the largest value at the top of the tower. As can be seen from Figure 14, the shear response of the three groups of near-fault ground motions is also the largest for the pulse ground motion with forward directional effect, and the maximum shear values at the bottom of the two groups of pulse ground motions are much larger than those without pulse ground motions.

Table 4. Amplification coefficients of acceleration responses for the hoist chamber and the amplifying coefficient of near-fault impulsive ground motion of the simplified model of the intake tower under near-fault ground motions with *IEPA* scaling.

Parameter	Forward Directivity Pulse			Fling-Step Pulse			Non-Pulse	
	Acceleration	Ratio	AC	Acceleration	Ratio	AC	Acceleration	Ratio
Top acceleration of hoist chamber (m/s ²)	25.27			18.93			16.39	
Top acceleration of the intake tower (m/s ²)	6.56	3.85	1.542	5.58	3.39	1.155	5.43	3.02

Note: AC is the near-fault pulse earthquake response coefficient.

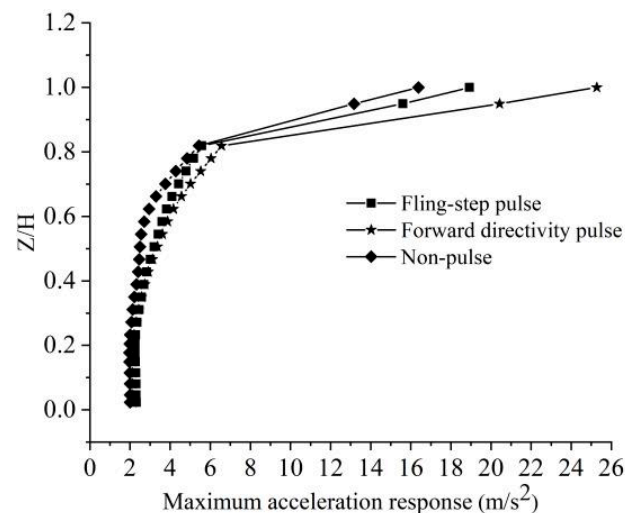


Figure 13. Mean maximum accelerations of simplified model of intake tower and hoist chamber under ground motions with *IEPA* scaling.

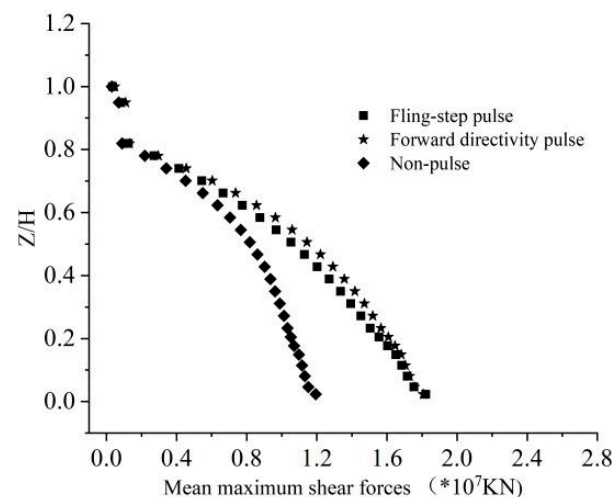


Figure 14. Mean maximum shear forces of simplified model of intake tower and hoist chamber under ground motions with *IEPA* scaling.

4.2. Evaluation of Cross-Sectional Rotation Angle and Lateral Deformation Curvature Response and Strength Parameters After *IEPA* Amplitude Modulation

By inputting near-fault ground motions modulated by *IEPA* into the rapid seismic response analysis program, we can determine the rotation angles of the water intake tower's segmented sections. The water intake tower's lateral deformation curvature can then be ascertained using the lateral deformation curvature method outlined in Chapter 3, as illustrated in Figures 15 and 16. The maximum lateral deformation curvature at the base of both water intake towers is larger than that of non-pulsed ground motions, as shown in Figure 15, suggesting that pulsed ground motions have a stronger effect on the lateral deformation curvature of the structure than non-pulsed ground motions. Figure 17 shows that, when comparing the lateral deformation curvature results for *IEPA* and *PGA* with 0.2 g amplitude modulation, the lateral deformation curvature response for both pulsed ground motions after *IEPA* modulation is greater than that for *PGA* modulation.

Obtained through the rotation angle data of each segmented section of the tower body. Table 5 shows that the AC coefficients of the lateral deformation curvature of the two pulse-type ground motions are close.

Table 5. Maximum section slopes and lateral curvatures of the amplifying coefficient of near-fault ground motion of simplified model of intake tower under near-fault ground motions with *IEPA* scaling.

Parameter	Fling-Step Pulse		Forward Directivity Pulse		Non-Pulse
		AC		AC	
Top max slope of hoist chamber	2.844	1.55	2.147	1.17	1.8339
Top max slope of the intake	0.184	1.49	0.17205	1.39	0.12367
Maximum lateral curvature	6.07175×10^{-5}	1.55	6.11412×10^{-5}	1.55	3.92931×10^{-5}

Note: AC is the near-fault pulse earthquake response coefficient.

Figure 17 compares the average values of lateral deformation curvature after *IEPA* amplitude modulation. As can be seen from these figures, the lateral deformation curvature responses after amplitude modulation for the two non-pulse ground motions are nearly identical, indicating that *IEPA* amplitude modulation has little effect on non-pulse ground motions, while affecting the other two types of pulse ground motions. Therefore, *IEPA* seismic intensity parameters as amplitude modulation indicators are more appropriate for computing amplitude modulation of near-fault ground motions than *PGA* amplitude modulation.

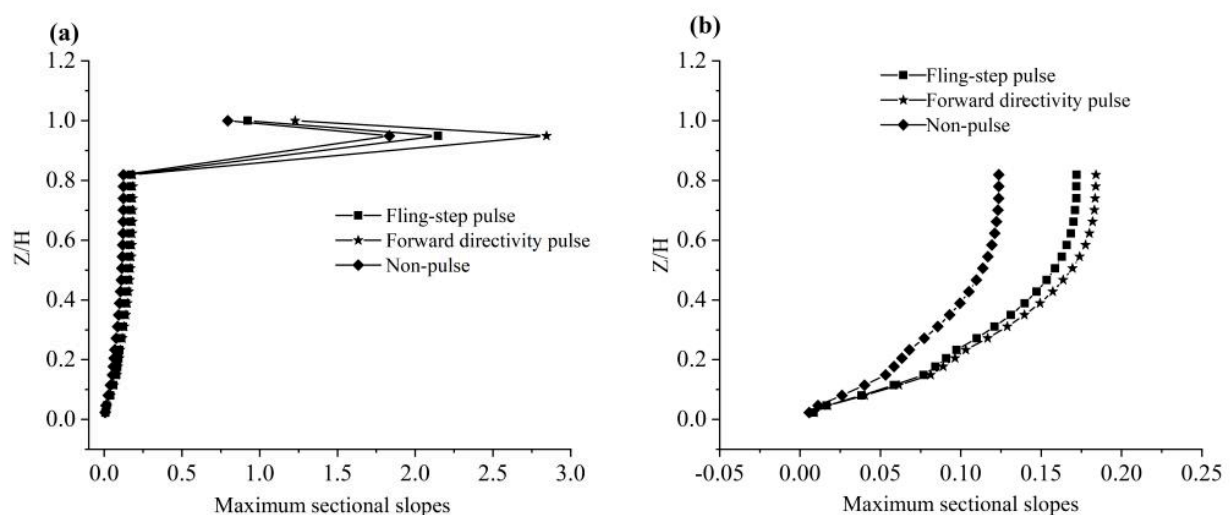


Figure 15. Mean maximum section slopes of simplified model of intake tower and hoist chamber under three groups of near-fault ground motions with *IEPA* scaling. (a) intake tower and hoist chamber; (b) Intake tower.

Table 5 illustrates the maximum cross-sectional rotation, lateral deformation curvature, and seismic response coefficient for the simplified model of the structure after *IEPA* modification. According to Table 5, under the three near-fault ground motions, the order of maximum cross-sectional rotation for the hoist chamber is the highest for pulsed ground motion with a forward directional effect and the lowest for ground motion without a pulse. Unlike the segmented cross-sectional rotation results after *PGA* modulation, the cross-sectional rotation for pulsed ground motion with a slip effect differs significantly from that for ground motion without a pulse, with the former being greater than the latter. Table 5 shows that the AC coefficient of the maximum cross-sectional rotation for the hoist chamber under pulsed ground motion with a slip effect is 1.17, while the AC coefficient for pulsed ground motion with a forward directional effect is 1.55. The AC coefficients for the maximum cross-sectional rotation at the tower apex for these two conditions are 1.39 and 1.49, respectively. This finding indicates that pulsed ground motion with a forward

directional effect exerts a more substantial influence on the cross-sectional rotation than does near-fault ground motion with a slip effect for the hoist chamber.

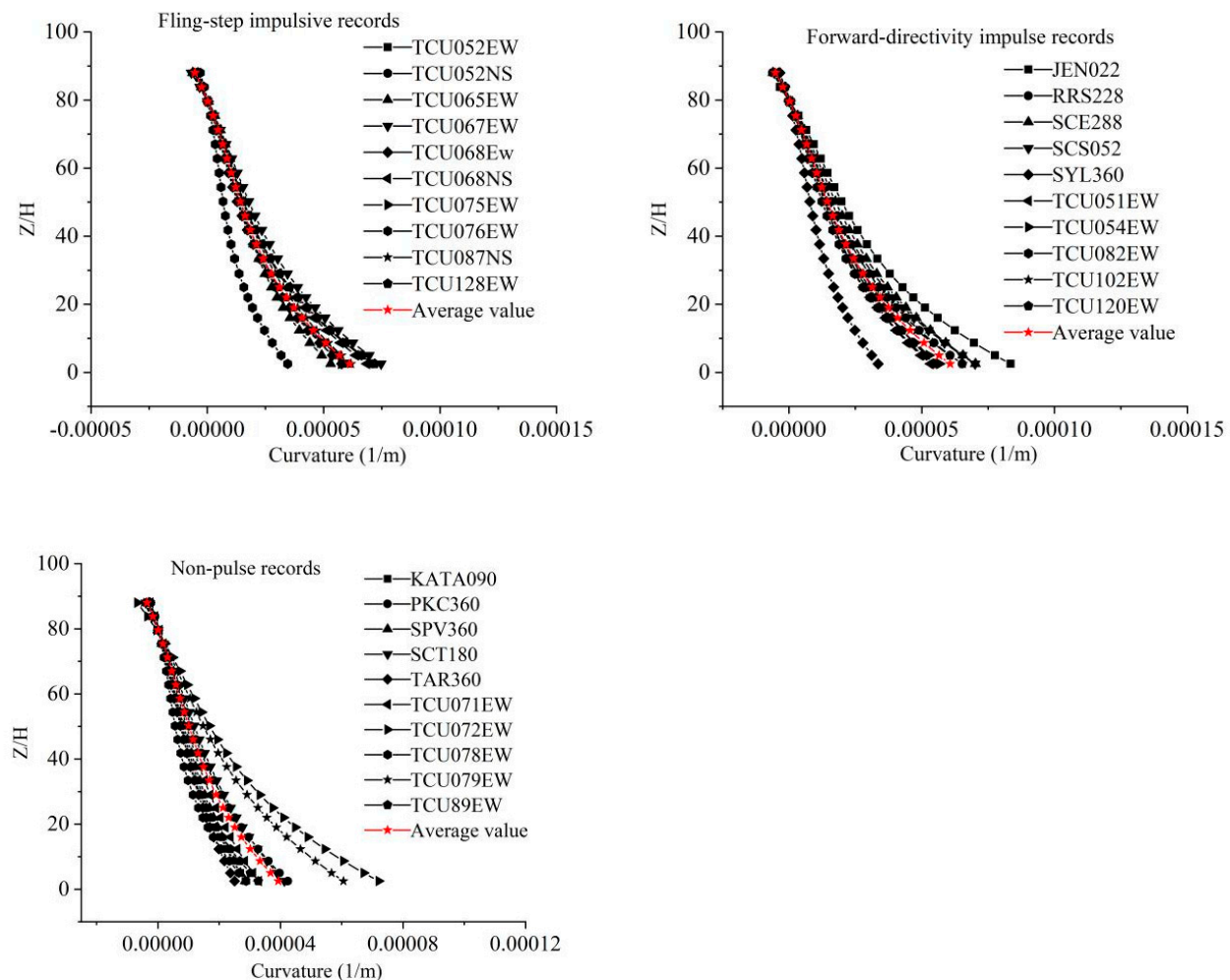


Figure 16. Lateral curvatures of simplified model of intake tower under three groups of near-fault ground motions with *IEPA* scaling.

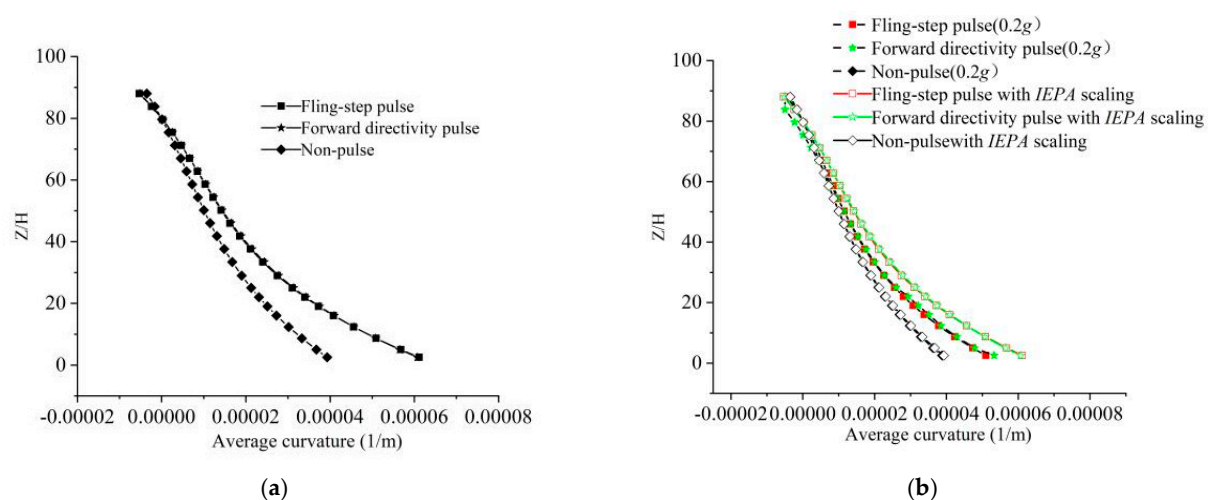


Figure 17. Comparison on mean lateral curvatures of tower intake under near-fault ground motions with *PGA* of 0.2 *g* scaling and *IEPA* scaling. (a) Average curvature with *IEPA* scaling; (b) Average curvature with *IEPA* scaling and *PGA* of 0.2 *g* scaling.

5. Response Analysis of a Simplified Model of a Water Intake Tower Subjected to Near-Fault Ground Motion After *IEPV* Amplitude Modulation

Yang et al. [31] proposed the improved effective peak acceleration (*IEPV*) as an intensity index based on the characteristics of near-fault ground motion to replace the *EPV* that is only applicable to ordinary ground motion. The expression of *IEPV* is Formula (9):

$$IEPV = \frac{S_v(T_{pv} - 0.2, T_{pv} + 0.2)}{2.5} \quad (9)$$

where $S_v(T_{pv} - 0.2, T_{pv} + 0.2)$ represents the average value of the pseudo-velocity spectrum with a damping ratio of 5% T_{pv} near the period. The *IEPV* amplitude modulation coefficient can be obtained.

5.1. Acceleration and Maximum Shear Response

A pre-programmed Matlab program sequentially inputs near-fault ground motions modulated with the *IEPV* seismic intensity parameter. The maximum acceleration response of the simplified structure under near-fault ground motions modulated with the *IEPV* seismic intensity parameter is obtained (see Figure 18); the segmented maximum shear seismic response values are shown in Figure 19. Figure 18 illustrates that, with *IEPV* modulation, the acceleration of the hoist chamber is enhanced due to the whiplash effect. Among the three near-fault ground motions, the highest acceleration in the hoist chamber is still reached with the near-fault ground motion having a forward-directed pulse effect, while the lowest is achieved with the near-fault ground motion containing a slip-pulse effect. Among the three near-fault ground motions, Table 6 demonstrates that the near-fault ground motion containing a forward-directed pulse effect still has the highest acceleration amplification factor in the hoist chamber, indicating that the near-fault ground motion containing a forward-directed pulse effect still has the greatest amplification effect on the hoist chamber structure. Amplitude modulation did not alter the acceleration amplification effect of near-fault ground motions with forward-directed pulse effects on the hoist chamber. However, in comparison with *IEPA* amplitude modulation, the acceleration response of near-fault ground motions with slip pulse effects was smaller, and the AC coefficient of acceleration was also smaller. This indicates that, compared to *IEPA* amplitude modulation, amplitude modulation using *IEPV* as the ground motion parameter has a greater impact on near-fault ground motions with slip pulse effects. Within the tower, except for the slightly larger acceleration response of forward-directed near-fault ground motions at the top of the hoist chamber compared to ground motions without velocity pulses, the acceleration response of near-fault ground motions without velocity pulses was also the largest within the tower, and the order of acceleration responses was closer to that of the unmodulated state. As shown in Figure 19, under the three near-fault ground motions, the shear response was greatest at the base of the structure, with the maximum value occurring for near-fault ground motions with forward-directed pulse effects and the smallest for ground motions with slip pulse effects. Compared with the maximum segmental shear value without amplitude modulation, the shear response with forward directionality becomes larger after *IEPV* amplitude modulation, indicating that *IEPV* amplitude modulation has an impact on the shear response of near-fault ground motion with forward directional pulse effect.

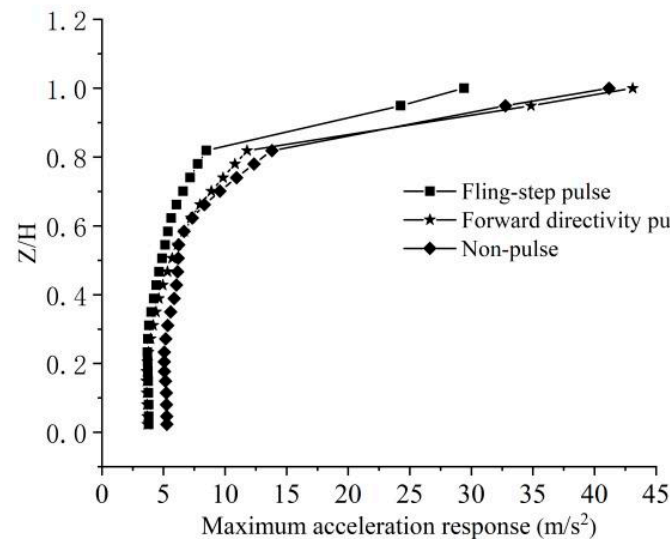


Figure 18. Mean of maximum acceleration of simplified model of the intake tower and hoist chamber under near-fault ground motions with *IEPV* scaling.

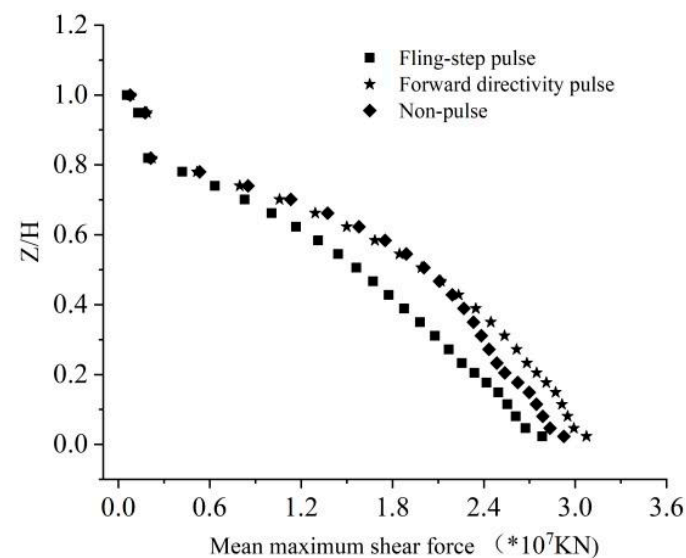


Figure 19. Mean of maximum shear force of simplified model of the intake tower and hoist chamber under near-fault ground motions with *IEPV* scaling.

Table 6. Amplification coefficients of Accelerations and the amplifying coefficient of near-fault ground motion of the simplified model of the intake tower and hoist chamber under near-fault ground motions with *IEPV* scaling.

Parameter	Fling-Step Pulse			Forward Directivity Pulse			Non-Pulse	
	Acceleration	Ratio	AC	Acceleration	Ratio	AC	Acceleration	Ratio
Top acceleration of hoist chamber (m/s ²)	43.097	3.66	1.047	29.403	3.47	0.714	41.179	2.98
Top acceleration of the intake tower (m/s ²)	11.784			8.48			13.807	

Note: AC is the near-fault pulse ground motion response coefficient.

5.2. Segmental Section Rotation

Thirty near-fault ground motions after *IEPV* amplitude modulation were input into a Matlab program, directly yielding the cross-sectional rotation angles of the simplified

water intake tower model at various height intervals. The average cross-sectional rotation angles of the simplified water intake tower structure under three near-fault ground motions vary with height, as shown in Figure 20. Under all three ground motions, the cross-sectional rotation angles increase with increasing height. However, the maximum cross-sectional rotation angle occurs at the tower's top, with the highest angle occurring under forward-directed pulse ground motions and the lowest angle occurring under ground motions containing slip pulses. Compared to the results obtained with *IEPA* amplitude modulation, *IEPV* amplitude modulation more closely matches the cross-sectional rotation angles obtained without amplitude modulation. The cross-sectional rotation angles for near-fault ground motions without velocity pulse effects and forward-directed pulse effects are closer than those obtained with amplitude modulation, with little difference. The cross-sectional rotation angles at the water intake tower elevation, including the hoist chamber, are shown in Figure 20. Due to the whip effect of sudden changes in upper and lower stiffness, the cross-sectional rotation angle at the intake tower hoist room suddenly increases and then decreases. In the hoist room, the maximum vertex cross-sectional rotation angle remains highest for near-fault ground motions with forward-directed pulses. This indicates that *IEPV* amplitude modulation does not alter the characteristic that near-fault ground motions with forward-directed pulses have the greatest impact on the cross-sectional rotation angle at the hoist room. Table 7 shows that under *IEPV* amplitude modulation, the AC coefficient for near-fault ground motions with forward-directed pulses decreases compared to that under *IEPA* amplitude modulation, while the AC coefficient for those with slip pulses is even less than 1. However, the observed patterns closely resemble the response patterns of the three groups of near-fault ground motions with no amplitude modulation, indicating that *IEPV* amplitude modulation has a smaller impact on near-fault ground motions than *IEPA* amplitude modulation.

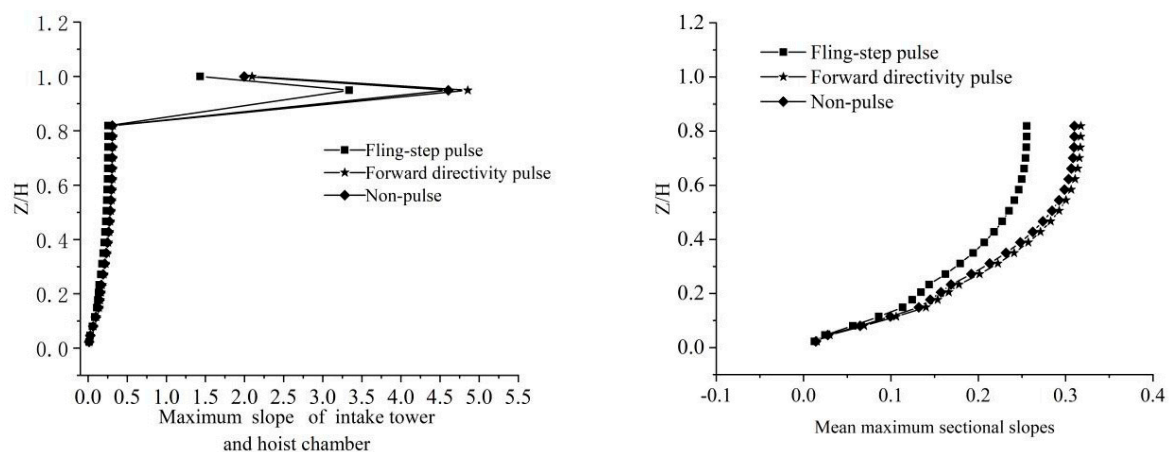


Figure 20. Mean of maximum section angles of the intake tower and hoist chamber under near-fault ground motions with *IEPV* scaling.

Table 7. Amplification coefficients of maximum sectional angles and lateral curvature of simplified model of the intake tower and hoist chamber under near-fault ground motions with *IEPV* scaling.

Parameter	Forward Directivity Pulse		Fling-Step Pulse		Non-Pulse
		AC		AC	
Top max slope of hoist chamber	4.855	1.054	3.337	0.724	4.606
Top max slope of the intake	0.317	1.022	0.256	0.826	0.310
Maximum lateral curvature	1.04552×10^{-4}	1.079	8.51264×10^{-5}	0.879	9.68591×10^{-5}

Note: AC is the near-fault pulse ground motion response coefficient.

6. Conclusions

This study develops a fast seismic response dynamic analysis program for a simplified model of an intake tower and hoist chamber. The seismic response of the simplified structure of the intake tower and hoist chamber, including the chamber, was calculated throughunder the action of an original near-fault ground motion model. Thirty near-fault ground motion records were amplitude-modulated using three intensity parameters: *IEPA*, *PGV*, and *IEPV*. The applicability of different intensity parameters in analyzing the near-fault ground motion response of the intake tower was explored. The specific conclusions are as follows:

- (1) Under the action of near-fault seismic motions after *PGV* modulation with three different intensities, the order of acceleration seismic responses of the simplified structures of the water intake tower and the hoist chamber is closer to the order of acceleration seismic responses without amplitude modulation. Compared with *PGA* modulation, the amplitude modulation using *PGV* as the intensity parameter does not affect the acceleration response amplification factor of the hoist chamber. From the observation of the AC coefficient of the water intake tower acceleration, the maximum shear force at the bottom, and the lateral curvature seismic response after amplitude modulation, it can be seen that the numerical change in *PGV* amplitude modulation has little effect on the pulse-type seismic motion response. *PGV* is not suitable as the amplitude modulation parameter for near-fault pulse-type seismic motions.
- (2) Based on observations of the AC coefficient from the amplified acceleration of the water intake tower, maximum shear force at the base, and lateral curvature seismic response, changes in the *PGV* intensity index value have virtually no effect on the AC coefficient of maximum shear force at the base for near-fault pulse seismic motion. They exert a slight influence on the AC coefficient of maximum shear force at the base for near-fault pulse seismic motion containing forward-directed pulses. Consequently, *PGV* is not suitable as an amplification parameter for near-fault pulse-type seismic motion.
- (3) The near-fault seismic motion was modulated using *IEPA* and *IEPV*, respectively. The calculation results of the water intake tower acceleration, bottom maximum shear force, and lateral curvature seismic response after amplitude modulation were observed. It can be seen that, compared with *PGA* amplitude modulation, the *IEPA* intensity parameter is more suitable for the amplitude modulation calculation of near-fault seismic motion and can better reflect the characteristics of near-fault seismic motion. Compared with *IEPA* amplitude modulation, the seismic response AC coefficient becomes smaller during *IEPV* amplitude modulation, indicating that *IEPV* amplitude modulation has less influence on near-fault seismic motion.

Author Contributions: Conceptualization, X.C. and X.L.; Methodology, X.C. and X.L.; Software, D.C. and B.Z.; Validation, X.C., D.C., and X.L.; Formal analysis, D.C. and B.Z.; Investigation, X.C.; Resources, X.L.; Data curation, D.C.; Writing—original draft, D.C.; Writing—review and editing, B.Z. and X.L.; Visualization, B.Z.; Supervision, X.C.; Project administration, X.C. and X.L.; Funding acquisition, X.C. and X.L. All authors have read and agreed to the published version of the manuscript.

Funding: The support of the National Natural Science Foundation of China (Grant Nos. 023A20673) and the Natural Science Foundation of Shaanxi Province (Grant No. 2025JC-YBMS-042) is greatly appreciated.

Data Availability Statement: The data presented in this study are available on request from the corresponding author. (Data availability is restricted to protect the proprietary interests of Northwest Engineering Corporation Limited, a data provider. Access can be facilitated through the corresponding author upon reasonable request.).

Conflicts of Interest: Author Binpeng Zhou was employed by the company Northwest Engineering Corporation Limited. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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