

Article

Seismic Performance of a Curved Continuous Rigid-Frame Composite Girder Bridge Under Ground Motions

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Abstract

To investigate the seismic performance and damage evolution of a curved continuous rigid-frame composite girder bridge under near-fault velocity pulse-like ground motions, a refined three-dimensional full-bridge finite element model was established, incorporating pile–soil interaction, expansion joint pounding, shear key damage, and nonlinear hysteretic behavior of high damping rubber bearings (HDRBs). Nonlinear time-history analyses were conducted under E1 and E2 seismic levels using near-field pulse records (short, moderate, and long periods), a near-field non-pulse record, and a far-field record. The fiber section capacity-to-demand ratio method was adopted to assess pier damage. Results show that near-field pulse-like motions govern the structural response, with long-pulse records producing the most unfavorable displacements and internal forces. Under E2, HDRBs exhibit significant yielding and hysteretic energy dissipation, effectively protecting the piers but imposing greater deformation demands on expansion joints and unseating preventers. Continuous girder piers display a transverse frame effect and a longitudinal S-shaped moment distribution with a secondary peak at the upper-middle portion due to higher modes. Rigid-frame hollow thin-walled piers exhibit S-shaped internal force distributions associated with abrupt section changes, and the tallest pier reaches a capacity-to-demand ratio of 0.82, indicating moderate yielding. The vertical seismic component amplifies transverse bending–torsion responses of curved girders through spatial coupling. The findings provide a scientific basis for ductility design and damping detailing of similar complex curved bridges.

Keywords: high damping rubber bearing; near-fault ground motion; velocity pulse; nonlinear time-history analysis; damage assessment



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1. Introduction

In the planning of complex transportation routes, curved continuous rigid-frame composite girder bridges have been widely adopted due to constraints imposed by topography and alignment requirements. Especially in complex geological environments such as mountainous regions, the dynamic response characteristics of high-pier long-span continuous girder bridges are highly intricate [1]. Because irregular curved bridges exhibit significant spatial asymmetry in terms of geometric alignment, mass distribution, and boundary conditions, under seismic excitation, the translational and torsional components of a spatially curved continuous girder bridge are coupled, causing the structural response to display pronounced directional sensitivity.

For curved continuous girder bridges, the structural design and parameter optimization of continuous girder bridges based on intelligent finite element modeling combined with evolutionary algorithms [2] or neural network optimization algorithms [3] have also been explored. It is noteworthy that, owing to the distinctive bending–torsion coupling dynamic characteristics of curved bridges, their structural responses exhibit higher sensitivity to the spatial variability of earthquake ground motions and to multi-dimensional non-uniform random excitations. Studies have demonstrated that under spatially varying seismic excitations, the absolute displacement method can effectively reveal the nonlinear elastoplastic response characteristics of curved continuous girder bridges [4], whereas under multi-dimensional non-stationary non-uniform random seismic inputs, the evolution of the system dynamic reliability of isolated curved girder bridges has been verified from a full-probability perspective [5]. In response to the issue of highly concealed post-earthquake damage, intelligent damage identification methods based on signal processing indices such as wavelet packet norm entropy are providing new avenues for the health monitoring of curved girder bridges [6].

When a bridge site is located adjacent to an active fault zone, the adverse effects induced by near-fault ground motions must be given particular consideration in engineering design. Owing to their distinctive velocity pulse and significant fling-step effects, near-field ground motions generally possess higher energy release and more complex damage potential than far-field ground motions [7]. Analyses of strong-motion records in recent years, such as the 2023 Kahramanmaraş MW 7.8 earthquake, have further confirmed the extreme destructiveness of the long-period pulse and large deformation characteristics in near-fault regions [8]. Near-fault ground motions exhibit highly complex spatial variability in the near-field region [9], and significant dependencies exist among the multi-dimensional random parameters of pulse-like ground motions [10]. The coupling of intense vertical seismic excitation with the general lack of tensile capacity in bridge bearings can readily induce vertical separation and pounding effects between the pier and the girder; this process not only alters the horizontal relative displacement between the pier and the girder, exacerbating the damage risk of the bearings, but also significantly amplifies the horizontal dynamic response of the structure, thereby fundamentally changing the failure evolution mechanism of the piers [11,12].

To effectively enhance the energy dissipation capacity of bridge structures under both near-field and far-field earthquakes, seismic isolation and energy dissipation devices such as high damping rubber bearings (HDRBs) are commonly installed at critical pier locations, utilizing their favorable hysteretic energy dissipation characteristics to absorb and dissipate the seismic input energy. Establishing refined mechanical models is crucial for accurately predicting the dynamic response of isolated structures. To this end, researchers have developed a refined multiaxial constitutive model for HDRBs that can account for complex mechanical behaviors [13] and have thoroughly investigated the epistemic uncertainty introduced by different bearing model selections in the performance assessment of isolated structures [14]. These advanced theories and refined modeling methods for HDRB-based seismic isolation and energy dissipation have not only been validated in bridge seismic engineering, but have also been widely applied to the dynamic failure control of complex mountainous building structures [15].

Although seismic isolation and energy dissipation bearings can provide protection, in complex bridge systems that combine continuous girders and continuous rigid-frames, the force-resisting characteristics of each part are fundamentally different owing to the differences in structural systems. The continuous girder units rely on bearing sliding to dissipate energy, whereas the rigid frame units, constructed with pier–girder rigid connections, possess longitudinal and transverse lateral stiffness far greater than that of the

bearing-supported continuous girder system, readily resulting in internal force concentration. In recent years, extensive research has been conducted on the seismic performance and damage mechanisms of long-span high-pier continuous rigid-frame bridges. In particular, quasi-static tests and data-driven modeling of hollow piers with variable cross-sections have elucidated their flexural–shear coupled failure mechanism and equivalent damping characteristics [16–23]. Considering the complex topography and landforms in mountainous regions, existing studies have thoroughly analyzed the amplification and spatial variability effects of various terrain combinations on the seismic response of high-pier long-span continuous rigid-frame bridges [24]. Facing complex ground motion inputs, the fragility analysis and failure collapse mechanisms of high-pier continuous rigid-frame bridges under spatially varying ground motions, multi-dimensional seismic excitations, near-fault pulse effects, and traveling wave effects have received widespread attention [25–27]. Furthermore, for the structural health monitoring and damage identification of continuous rigid-frame bridges during service and after earthquakes, real-time collapse mechanism assessment methods based on advanced sensor networks and machine learning algorithms have also been developed considerably [28].

Based on the aforementioned research background, this study takes a curved continuous rigid-frame composite girder bridge with a main span of 100 m as the research object and establishes a full-bridge three-dimensional spatial finite element model that comprehensively incorporates pile–soil interaction, expansion joint pounding effects, shear key damage, and the nonlinear characteristics of high damping rubber bearings. Near-field velocity pulse records with short-, medium-, and long-pulse features, together with near-field non-pulse and far-field ground motion records, are carefully selected, and nonlinear dynamic time-history analyses are conducted under two seismic fortification levels, E1 and E2. By employing fiber section analysis, the capacity-to-demand ratio (CDR) method at the section level is adopted to rapidly and quantitatively assess the seismic damage evolution states of different types of piers. This study aims to systematically reveal the controlling influences of near-field pulse-like ground motions and their vertical components on the dynamic characteristics, internal force and displacement responses, and damage evolution laws of such complex spatially curved bridges, thereby providing a scientific basis for the seismic ductility design and damping detailing optimization of similar bridges in the future.

2. Materials and Methods

2.1. Bridge Project Overview and Structural System

The curved continuous rigid-frame composite girder bridge system has a total length of 720 m and a standard deck width of 12.6 m. The span arrangement is $(30 + 3 \times 40 + 30) \text{ m} + (4 \times 40) \text{ m} + (50 + 60 + 50) \text{ m} + (60 + 100 + 60) \text{ m}$, and the bridge is divided into four units. Based on the differences in structural systems, the first, second, and third units adopt continuous steel–concrete composite girders, whereas the fourth unit adopts a continuous rigid frame, forming a bridge system that connects composite girders with a rigid frame. Between the units, the release of longitudinal displacements and the orderly transfer of horizontal forces are achieved through transition piers.

Constrained by the horizontal alignment of the route, the bridge centerline is composed of multiple curved elements connected in sequence. From the starting point to the end point, these elements are as follows: a transition curve (parameter $A = 320 \text{ m}$, left-turning), a circular curve (radius $R = 750 \text{ m}$, left-turning), a transition curve ($A = 250 \text{ m}$, left-turning), a tangent section, a transition curve ($A = 100 \text{ m}$, left-turning), a circular curve ($R = 150 \text{ m}$, left-turning), a transition curve ($A = 100 \text{ m}$, left-turning), and a tangent section.

To accommodate the curvature variation and effectively transmit centrifugal forces, all piers and abutments are arranged radially.

The substructure uniformly adopts double-column piers with cap beams, and both the pier shafts and the abutments are supported by pile foundations; the maximum pier height reaches 44.0 m. To enhance the energy dissipation capacity of the structure under both near-field and far-field ground motions, high damping rubber bearings (HDRBs) are installed at critical pier locations throughout the bridge, dissipating seismic energy through their hysteretic characteristics. The overall plan and elevation layouts of the bridge are shown in Figure 1 and Figure 2, respectively, and typical cross-section details of the main girder and substructure are shown in Figure 3.

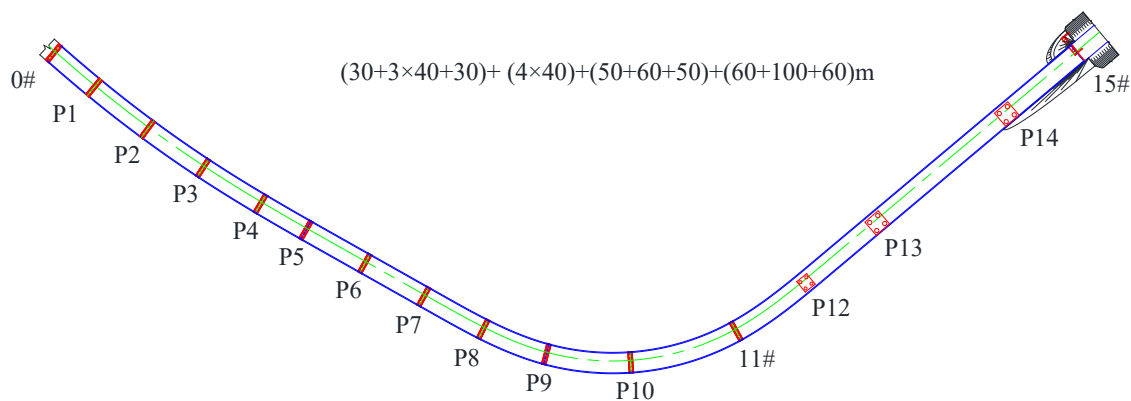


Figure 1. General layout plan of the bridge.

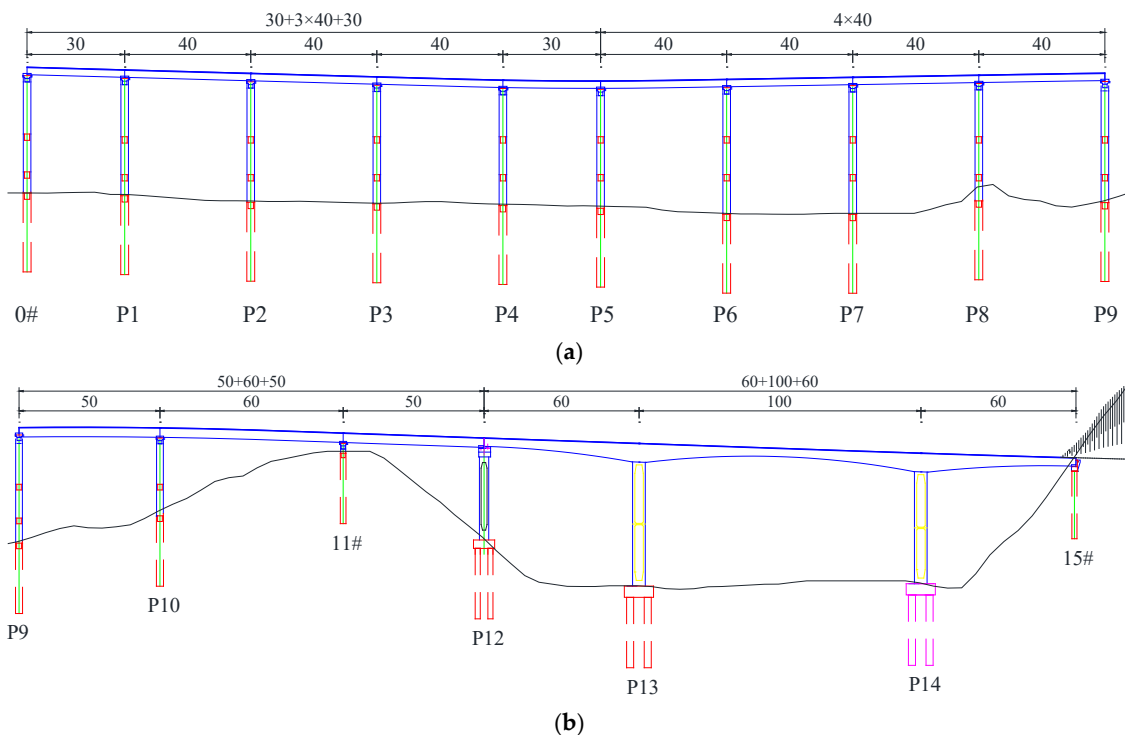


Figure 2. Elevation layout of the bridge: (a) units 1 and 2: $(30 + 3 \times 40 + 30) + (4 \times 40)$ m (unit: m); (b) units 3 and 4: $(50 + 60 + 50) + (60 + 100 + 60)$ m (unit: m).

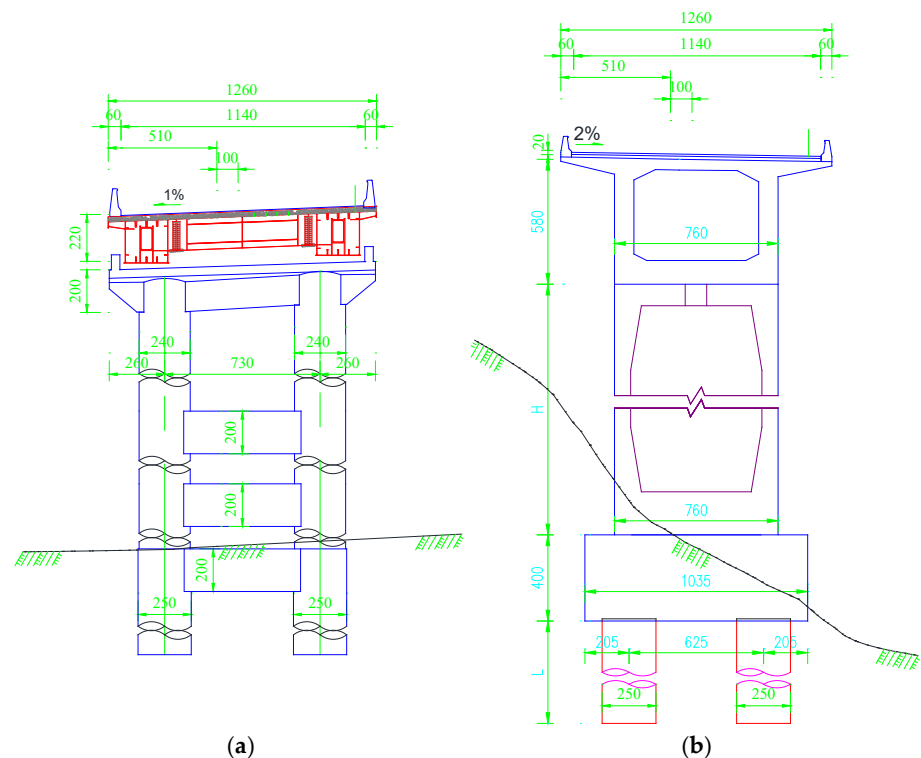


Figure 3. Typical cross-section details of the main girder and substructure: (a) continuous steel-concrete composite girder portion; (b) continuous rigid-frame portion.

The design parameters of all piers, abutments, and bearings of the bridge are summarized in Table 1. The substructure piers are primarily composed of double-column circular section piers with a diameter of 2.4 m, which are widely adopted in the composite girder portions of the first to third units. To accommodate the large span arrangement of 60 m + 100 m + 60 m and the longitudinal stiffness requirements during cantilever construction, hollow thin-walled piers are employed at the three main piers P12, P13, and P14 of the fourth unit (continuous rigid frame), with pier heights of 29.0 m, 44.0 m, and 39.5 m, respectively. Furthermore, pier-girder rigid connections are adopted at P13 and P14, forming rigid joints without bearings.

The bearing system incorporates high damping rubber bearings (HDRBs), PTFE sliding plate bearings (GBZYs), and spherical steel bearings (JQZs), the distribution of which strictly corresponds to the mechanical characteristics of each structural unit. At the expansion ends of the first, second, and third continuous composite girder units and at the transition piers between units, specifically at locations 0#, P5, P9, and P12, GBZY or JQZ bearings are arranged to accommodate the thermal expansion and contraction of the girder and to transmit vehicle braking forces. At the intermediate piers of the first and second composite girder units (P1–P4, P6–P8) and at the side piers of the fourth rigid-frame unit (P10, 11#), high damping rubber bearings, namely HDR(I)-d920 × 325-G1.0 or HDR(I)-d1020 × 374-G1.0 (All the bearings described in this paragraph were supplied by Sichuan Ronghai Yuntong Earthquake-Resistant Technology Co., Ltd. (Chengdu, China)), are uniformly installed to concentrate the dissipation of seismic input energy through their nonlinear hysteretic deformation. The main piers P13 and P14 of the fourth rigid-frame unit, being pier-girder fixed connections, are not equipped with bearings. At the adjacent transition pier P12, both PTFE sliding plate bearings and unidirectional movable spherical bearings are installed to coordinate longitudinal deformations. At the far-end abutment 15# of the fourth unit, bidirectional movable spherical bearings JQZ(II)3.5 SX ± 150 are arranged to accommodate displacement demands in both longitudinal and transverse directions.

Table 1. Parameters of piers and bearings of the bridge.

Pier/Abutment No.	Pier Type	Pier Height (m)	Bearing Type
0#	Double-column circular section pier (D = 2.4 m)	35.5	GBZY 700 × 148
P1	Double-column circular section pier (D = 2.4 m)	35.0	HDR(I)-d920 × 325-G1.0
P2	Double-column circular section pier (D = 2.4 m)	36.0	HDR(I)-d920 × 325-G1.0
P3	Double-column circular section pier (D = 2.4 m)	36.0	HDR(I)-d920 × 325-G1.0
P4	Double-column circular section pier (D = 2.4 m)	35.0	HDR(I)-d920 × 325-G1.0
P5	Double-column circular section pier (D = 2.4 m)	35.5	GBZY 700 × 148
P6	Double-column circular section pier (D = 2.4 m)	38.0	HDR(I)-d920 × 325-G1.0
P7	Double-column circular section pier (D = 2.4 m)	38.5	HDR(I)-d920 × 325-G1.0
P8	Double-column circular section pier (D = 2.4 m)	30.5	HDR(I)-d920 × 325-G1.0
P9	Double-column circular section pier (D = 2.4 m)	35.5	GBZY 700 × 148
P10	Double-column circular section pier (D = 2.4 m)	23.5	HDR(I)-d1020 × 374-G1.0
11#	—	—	HDR(I)-d1020 × 374-G1.0
P12	Hollow thin-walled pier (3.3 m (long.) × 5.6 m (trans.))	29.0	GBZY 700 × 148 JQZ(II)3.5 ZX ± 150
P13	Hollow thin-walled pier (4.5 m (long.) × 7.6 m (trans.))	44.0	—
P14	Hollow thin-walled pier (4.5 m (long.) × 7.6 m (trans.))	39.5	—
15#	—	—	JQZ(II)3.5 SX ± 150

2.2. Finite Element Modeling Method and Element Selection

In view of the fact that the bridge's horizontal alignment comprises multiple transition curves and circular curves, making it a typical spatially curved continuous girder bridge, the translational and torsional components under seismic excitation are coupled, and the structural response exhibits pronounced directional sensitivity. To obtain the most unfavorable force state of the structure, the ground motion input should be applied along the most unfavorable direction, and nonlinear time-history analyses should be conducted on this basis to encompass the envelope values of the structural response in the longitudinal direction, the transverse direction, and their combinations.

The three-dimensional full-bridge finite element model was developed using the SAP2000 v24 structural analysis platform, as shown in Figure 4. The primary structural components were discretized as follows. (1) Main girder (steel–concrete composite deck and rigid-frame box girder): Modeled using three-dimensional linear elastic frame elements with a single-beam spine representation. The curved alignment was approximated by a series of short straight beam segments (typical length 2–4 m) to capture the bending–torsion coupling. The composite cross-section properties were defined using built-in Section Designer tools [29–31]. (2) Piers (double-column circular piers and hollow thin-walled piers): Modeled using three-dimensional elastic frame elements. Each pier shaft was subdivided into at least ten elements along its height to ensure convergence of higher-mode shapes. (3) Pier caps (capping beams): Modeled as rigid linear elastic frame elements. The rigid connection between the pier cap and the pier top was enforced through fully restrained joints (six degrees of freedom fixed).

The self-weight of all structural components was automatically converted to lumped translational masses at the element nodes. Superimposed dead loads (asphalt pavement, barriers, etc., totaling approximately 80 kN/m) were applied as additional lumped masses distributed along the main girder nodes. Rotational inertia of the mass was neglected.

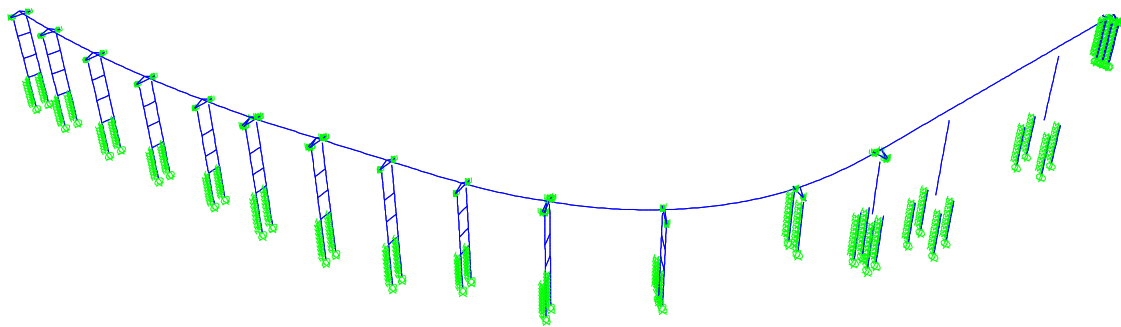


Figure 4. Three-dimensional finite element model of the full bridge.

2.3. Material Constitutive Models and Component Modeling

Pile–soil interaction is simulated using distributed soil springs (Figure 5a). The bearings are assigned corresponding nonlinear constitutive models according to their types: the ordinary PTFE sliding rubber bearings (GBZYs) and spherical steel bearings (JQZs) adopt the bilinear restoring force model shown in Figure 5b, and their mechanical behavior is described through the yield force, pre- and post-yield stiffness, and friction sliding parameters.

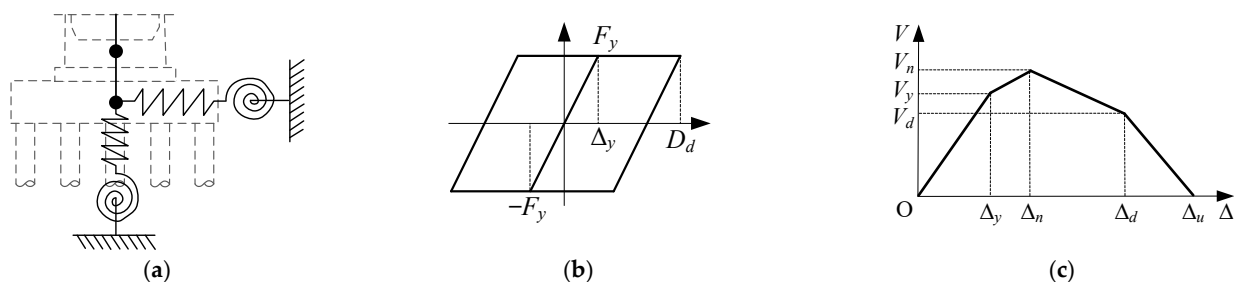


Figure 5. Numerical simulation: (a) pile–soil interaction; (b) bilinear element for PTFE sliding bearings and spherical steel bearings; (c) nonlinear degrading element for reinforced concrete shear keys.

The reinforced concrete shear keys were simulated using the multilinear plastic link element in SAP2000 v24 combined with the Takeda hysteretic rule (Figure 5c). The displacement and force capacity values of the key characteristic points on the backbone curve (yield point, peak point, degradation initiation point, and failure point) were determined based on the experimental studies by Silva et al. [32,33]. A parameter sensitivity analysis (considering $\pm 20\%$ variations in the force capacity and degradation displacement) showed that the variations in both the pier-base moment and the pier-top displacement were less than 1%, indicating that the response of the main structure is insensitive to the shear key parameters and that the model exhibits good robustness.

The pounding effect at the expansion joints is simulated using gap elements combined with contact stiffness; an initial gap is set between adjacent main girders, and the pounding spring stiffness and damping are defined to capture the impact effects that may occur during earthquakes. The pounding stiffness and damping parameters were determined based on the recommendations of Jankowski et al. [34,35]: the pounding stiffness was taken as the axial stiffness of the shorter adjacent girder segment, i.e., $k_{imp} = EA/L \approx 1.2 \times 10^6$ kN/m; the pounding damping was derived from a coefficient of restitution $e = 0.65$, yielding a damping coefficient $c_{imp} \approx 250$ kN·s/m. A parameter sensitivity analysis (simultaneously varying k_{imp} and c_{imp} by $\pm 30\%$) indicated that under E2 NLP ground motions, the variations in both the pier-top displacement and the pier-base moment were less than 2%, suggesting that the pounding parameters have a negligible

influence on the main seismic response of the bridge and that the values adopted in this study exhibit sufficient robustness.

Since the viscoelastic behavior of rubber materials is influenced by multiple factors such as ambient temperature, loading frequency, and shear strain amplitude, scholars worldwide have proposed various mechanical models to describe their nonlinear behavior, among which the commonly used ones include the Maxwell model, the Kelvin model, the fractional derivative model, and the four-parameter model [36]. In the seismic dynamic time-history analysis of the overall structure, in order to balance computational accuracy and efficiency, the equivalent bilinear restoring force model is widely adopted to simulate the force-displacement hysteretic relationship of the bearing. Within the SAP2000 v24 finite element software, the High Damping Rubber Isolator link element can be directly selected to simulate HDRBs. This element is based on a bilinear restoring force model, and its constitutive relationship is illustrated in Figure 6. The key mechanical parameters can be determined according to the following formulas:

$$K_h = G \frac{A_0}{T_r} \quad (1)$$

$$K_1 = G_1 \frac{A_0}{T_r} \quad (2)$$

$$K_2 = G_2 \frac{A_0}{T_r} \quad (3)$$

$$X_y = \frac{Q_y}{K_1} \quad (4)$$

$$\xi = \frac{1}{2\pi} \cdot \frac{W_d}{W} = \frac{W_d}{K_h X^2} \quad (5)$$

where K_h is the horizontal equivalent stiffness of the high damping rubber bearing (kN/mm); G is the shear modulus of the rubber (MPa); A_0 is the effective area (mm²); T_r is the total thickness of the rubber layer (mm); K_1 is the initial horizontal stiffness (kN/mm); G_1 is the shear modulus corresponding to the initial horizontal stiffness (MPa); K_2 is the post-yield horizontal stiffness (kN/mm); D_d is the design horizontal displacement (m); G_2 is the shear modulus corresponding to the post-yield horizontal stiffness (MPa); X_y is the yield displacement (mm); Q_y is the horizontal yield force of the bearing (kN); ξ_{eff} is the equivalent damping ratio (%); W_d is the area enclosed by the hysteretic loop (the energy absorbed in one load cycle); and W is the elastic strain energy corresponding to one load cycle.

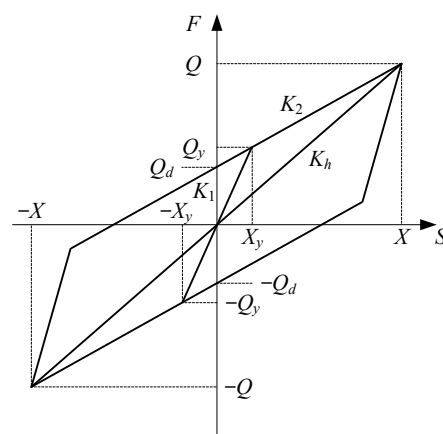


Figure 6. Equivalent bilinear hysteretic model of HDRB.

It is noteworthy that the aforementioned equivalent bilinear model is a rate-independent model and therefore cannot capture the viscoelastic characteristics exhibited by high damping rubber materials under actual seismic loading, such as the influence of loading rate on the restoring force amplitude and the shape of the hysteresis loop. However, given that the primary objective of this study is to evaluate the nonlinear response of the entire bridge and the damage evolution of the piers, adopting the bilinear model allows for the reasonable capture of the main features of post-yield stiffness degradation and hysteretic energy dissipation of the bearings while maintaining computational efficiency. Consequently, this simplification does not compromise the engineering safety of the main conclusions.

2.4. Ground Motion Selection and Input Method

In accordance with GB 17741-2005 [37] and JTG/T 2231-01-2020 [38], combined with the seismotectonic environment and seismicity characteristics of the bridge site, the design ground motion parameters for the project site were determined using a probabilistic seismic hazard analysis method. The bridge site is classified as a Class II site, with a seismic fortification intensity of VIII degrees and a basic peak ground acceleration of 0.3 g. The design response spectrum parameters are listed in Table 2, in which the E1 seismic action corresponds to a 10% probability of exceedance in 50 years ($PGA = 0.37$ g), and the E2 seismic action corresponds to a 2% probability of exceedance in 50 years ($PGA = 0.74$ g).

Table 2. Parameters of design horizontal acceleration response spectra for the bridge ground surface (5% damping).

Exceedance Probability	$\beta_{\max}$	T_1 (s)	T_g (s)	γ	$A_{\max}$ (cm/s ²)	C_d	$S_{\max}$
50 years 10%	2.5	0.10	0.45	1.0	369	1.0	0.941
50 years 5%	2.5	0.10	0.50	1.0	499	1.0	1.273
50 years 2%	2.5	0.10	0.55	1.0	672	1.0	1.714
100 years 2%	2.5	0.10	0.55	1.0	798	1.0	2.036

Considering that the bridge site is adjacent to active faults, in order to adequately reflect the damaging effects of near-fault velocity pulses, four near-field records (including short-pulse NSP, medium-pulse NMP, long-pulse NLP, and near-field without pulse NNP) and one far-field (FF) record were selected from the PEER NGA-West2 strong-motion database based on the pulse identification method proposed by Baker [39]. The original information of these records is listed in Table 3. To make each record suitable for nonlinear time-history analysis at both the E1 and E2 seismic design levels, the horizontal acceleration time histories of each record were linearly scaled such that their peak ground accelerations (PGAs) reached 0.37 g (E1) and 0.74 g (E2), respectively. The vertical acceleration components were scaled using the same factors as the horizontal components, in order to preserve the amplitude ratios among the three components in the original records.

Table 3. Parameters of selected ground motion records.

No.	Station	RSN	Fault Distance (km)	PGA (g)	PGV (m·s ⁻¹)	(PGV/PGA) (s)	Pulse Period (s)
NSP	CHY035	NGA-1202	12.65	0.25	0.44	0.18	1.40
NMP	CHY101	NGA-1244	9.94	0.34	0.65	0.20	5.34
NLP	TCU049	NGA-1489	3.76	0.28	0.54	0.20	12.0
NNP	CHY028	NGA-1197	3.12	0.64	0.61	0.10	—
FF	CHY086	NGA-1234	28.42	0.10	0.17	0.17	—

Since linear scaling does not alter the frequency content or phase of the ground motion, the velocity pulse waveform, pulse period T_p , and PGV/PGA ratio of each record remain exactly the same before and after scaling (the PGV/PGA ratios listed in Table 3 are the original values and remain unchanged after scaling). The comparison of the scaled acceleration response spectra and velocity response spectra is shown in Figure 7.

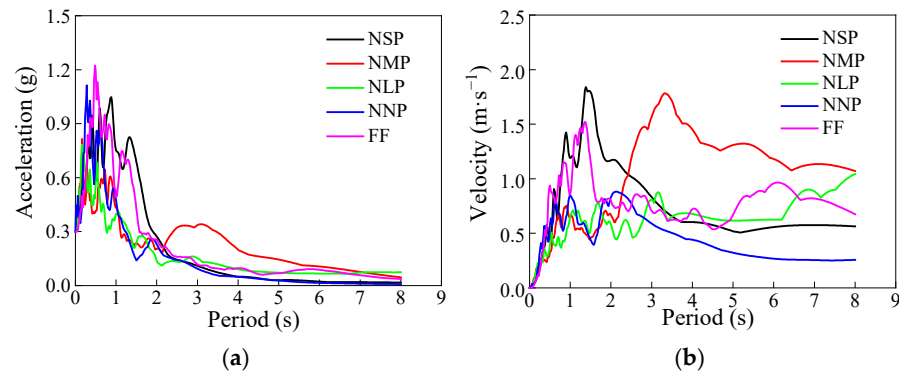


Figure 7. Scaled ground motion response spectra: (a) acceleration response spectra; (b) velocity response spectra.

Because the horizontal alignment of this bridge consists of multiple transition curves and circular curves, the local longitudinal and transverse axes of individual piers deviate from the global coordinate system, potentially inducing directional sensitivity in the seismic response as the input direction varies. To verify whether the orthogonal inputs in the global coordinate system (longitudinal + vertical, transverse + vertical) adopted in this study can envelope the most unfavorable seismic effects, a sensitivity analysis of the horizontal ground motion input angle was conducted using the E2-level near-field long-pulse record NLP. The two horizontal components of NLP were rotated clockwise from the global X-axis (the chord direction from the start to the end of the bridge) in 15° increments up to 180° , resulting in 13 input directions. For each direction, the vertical component was applied simultaneously (scaled by the same factor as the horizontal components), forming three-component ground motion inputs. The envelope values of the longitudinal moment, transverse moment at the pier base, and resultant displacement at the pier top were extracted for a typical continuous-beam pier P7 (pier height 38.5 m) and a rigid-frame pier P13 (pier height 44.0 m). The analysis results indicate the following.

(1) The responses of each pier exhibit a distinct dependence on the input angle, and the most unfavorable response angles are not identical. Specifically, the longitudinal moment of P7 reaches its maximum at an angle of approximately 30° , while the transverse moment of P13 becomes most pronounced at an angle of approximately 120° , reflecting the flexural–torsional coupling effect of the curved bridge and the rotation of the principal axes of local response caused by differences in pier height.

(2) Nevertheless, for all rotation angles considered, the maximum pier-base moments and pier-top displacements of P7 and P13 do not exceed the envelope values obtained from the original orthogonal inputs (X + Z and Y + Z) by more than 5%.

(3) It can thus be concluded that, although each pier of the curved bridge possesses its own most unfavorable response direction, the approach of applying orthogonal inputs in the global coordinate system and taking the envelope values can safely cover all possible worst-case scenarios. Therefore, only the two orthogonal combinations, longitudinal + vertical and transverse + vertical, are adopted in the subsequent analyses of this study, and the results obtained are conservative.

3. Dynamic Characteristics Analysis

3.1. Full-Bridge Structural System

The modal analysis was performed on the curved continuous rigid-frame composite girder bridge, and the first 500 mode shapes were extracted. The cumulative modal participation mass ratios in the three translational directions (longitudinal X, transverse Y, and vertical Z) all reached 100%, satisfying the code requirement that the cumulative participation mass in each direction should not be less than 90%, which demonstrates that the number of extracted modes is sufficient to ensure the accuracy of subsequent mode superposition or response spectrum analyses. Figure 8 shows the distribution of the modal mass participation factors for the first 500 modes of the full bridge, and Table 4 lists the natural periods, frequencies, and corresponding directional mass participation ratios of the first 10 modes.

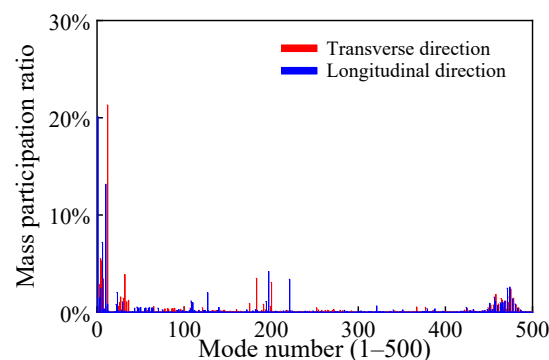


Figure 8. Distribution of modal mass participation ratios for the first 500 modes of the full-bridge system.

As can be seen from Figure 8, the effective modal mass distribution of the structure exhibits distinct multi-peak characteristics. Although a large proportion of the effective mass is concentrated in the first few dozen modes, due to the complex span arrangement, significant variations in pier heights, and the curved horizontal alignment, the higher-order local vibrations still contribute non-negligible participation mass. The cumulative longitudinal participation mass of the first 10 modes is approximately 45%, while the transverse is only about 19%, indicating that both the longitudinal and transverse dynamic responses of this bridge are collectively controlled by dense higher-order modes. Relying solely on a few low-order modes cannot fully reflect the true behavior of the structure under seismic excitation. This characteristic further confirms the necessity of adopting a three-dimensional full-bridge model for nonlinear time-history analysis.

Table 4 also presents the main vibration characteristics of each mode. The first mode ($T_1 = 3.29$ s) is characterized by the first-order longitudinal vibration of the entire bridge, with the main girder drifting longitudinally and causing longitudinal bending of all piers; its longitudinal participation mass reaches 20.1%, making it the key mode controlling the longitudinal seismic response. The second mode has a period of 2.90 s and mainly involves the first-order transverse bending of the composite girders in the first and second units; because only part of the girder is involved, its effective mass is relatively small. The third to fifth modes are densely populated with transverse- or torsion-dominated vibrations: the third mode is the first-order transverse bending of the full bridge accompanied by torsion coupling induced by the curved alignment; the fourth mode exhibits relatively prominent second-order transverse bending with significant pier participation; the fifth mode is the anti-symmetric transverse bending of the continuous girder units. The sixth mode returns to longitudinal vibration, being the second-order longitudinal mode with reverse bending of the main girder and piers, and a longitudinal participation mass of 7.17%. The seventh to ninth modes are primarily dominated by higher-order transverse

bending–torsion coupling of the girder and local bending of the piers. The tenth mode ($T_{10} = 1.55$ s) again exhibits strong longitudinal vibration characteristics, with a longitudinal participation mass of 13.16%, corresponding to the second-order longitudinal vibration of the main girder or large-amplitude bending of the tall piers.

Table 4. Natural periods and modal mass participation ratios of the full-bridge system.

Mode	Period (s)	Frequency (Hz)	Mass Participation Ratio (%)		Torsional Contribution Ratio (%)	Major Vibration Characteristics
			Long.	Trans.		
1	3.29	0.30	20.11%	0.64%	0.10	First-order longitudinal vibration of the whole bridge, longitudinal drifting of the main girder accompanied by longitudinal bending of piers
2	2.90	0.34	0.03%	0.18%	0.22	First-order transverse bending of the composite girders in the first and second units, local vibration
3	2.13	0.47	1.46%	2.83%	44.80	First-order transverse bending of the whole bridge coupled with torsion
4	1.88	0.53	2.46%	5.52%	10.15	Second-order transverse bending of the whole bridge, with significant pier participation
5	1.81	0.55	0.01%	5.25%	5.32	Anti-symmetric transverse bending of the continuous girder units
6	1.76	0.57	7.17%	0.55%	0.35	Second-order longitudinal vibration, reverse bending of the main girder and piers
7	1.73	0.58	0.29%	3.43%	3.68	Higher-order transverse bending–torsion coupling of the main girder
8	1.71	0.58	0.33%	0.35%	1.20	Local bending of piers and torsion of the girder
9	1.62	0.62	0.04%	0.15%	0.12	Higher-order vertical bending or local pier vibration
10	1.55	0.65	13.16%	0.59%	0.40	Second-order longitudinal vibration of the main girder or large bending of tall piers, with significant longitudinal participation

Overall, the first 10 mode shapes already reflect the inherent dynamic characteristics of this type of curved-composite-girder-rigid-frame system under spatial coupling effects: low-order longitudinal drifting and transverse bending appear in sequence, the mass participation is not concentrated, and the modes are closely spaced. These modal parameters will be directly used in the subsequent determination of Rayleigh damping coefficients, the selection of the mode truncation order, and the control of the dynamic integration step size in the seismic response analysis.

3.2. Continuous Rigid-Frame Bridge Subsystem

To further identify the role played by the fourth-unit continuous rigid-frame system in the dynamic response of the full bridge, a modal analysis was separately performed on the rigid-frame bridge of this unit, based on the aforementioned spatial finite element model. The first 500 modes were also extracted; the cumulative modal participating mass ratios in the longitudinal (X), transverse (Y), and vertical (Z) directions all reached 100%, far exceeding the minimum requirement of 90% in each direction, indicating that the number of modes extracted is sufficient to accurately capture the dynamic characteristics of this

substructure. The distribution of the mass participation factors for the first 500 modes is shown in Figure 9, and Table 5 lists the natural periods, frequencies, corresponding mass participation ratios, and vibration shapes of the first 10 modes.

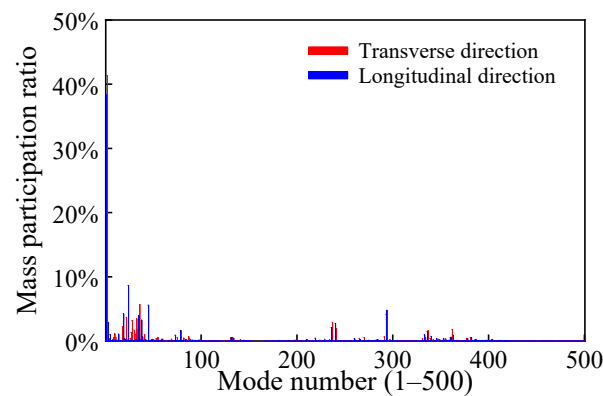


Figure 9. Distribution of modal mass participation ratios for the first 500 modes of the continuous rigid-frame bridge.

Table 5. Natural periods and modal mass participation ratios of the continuous rigid-frame bridge.

Mode	Period (s)	Frequency (Hz)	Mass Participation Ratio (%)		Torsional Contribution Ratio (%)	Major Vibration Characteristics
			Long.	Trans.		
1	1.23	0.81	38.37%	0.02%	0.01	First-order longitudinal vibration of the entire unit, longitudinal drifting of the main girder accompanied by longitudinal bending of thin-walled piers
2	1.18	0.85	0.00%	41.37%	0.05	First-order transverse bending of the entire unit, lateral bending of the main girder and piers in the same direction
3	1.07	0.93	2.89%	0.05%	12.40	Second-order longitudinal vibration, anti-symmetric longitudinal bending of the main girder, reverse displacement of pier tops
4	0.85	1.18	0.00%	0.35%	48.20	Second-order transverse bending–torsion coupled vibration, pronounced local behavior
5	0.82	1.22	0.99%	0.00%	0.10	Higher-order longitudinal bending, multiple inflection points on piers
6	0.58	1.72	0.00%	0.06%	0.86	Higher-order transverse bending of the main girder or distortion of the box girder section
7	0.43	2.33	0.05%	0.00%	2.21	Local longitudinal bending of piers and vertical bending of the girder
8	0.33	3.03	0.01%	0.17%	1.41	Higher-order transverse bending–torsion and local vibration of pier tops
9	0.31	3.23	0.01%	0.55%	0.14	Higher-order transverse bending, multi-wave lateral bending pattern
10	0.30	3.33	0.04%	1.10%	0.65	Higher-order transverse mode, compound transverse bending of the main girder and piers

In contrast to the gently dispersed distribution of the modal mass participation factors for the first 500 modes of the full-bridge system, the effective mass of the continuous

rigid-frame bridge is highly concentrated in the first few modes (Figure 9). As can be seen from the figure, the mass participation factors of the first and second modes reach as high as approximately 38% and 41%, respectively; the cumulative longitudinal participation mass of the first 10 modes exceeds 42%, and the cumulative transverse participation mass exceeds 43%, with the dominant modes being prominent. The reason is that in the rigid-frame bridge, the main piers and the main girder are fixed to form a rigid frame, and the longitudinal and transverse lateral stiffnesses are far greater than those of the bearing-supported continuous girder system. The structural integrity is enhanced, and the low-order modes can mobilize most of the mass participation, while the contribution of higher-order local vibrations is relatively limited.

Table 5 lists the parameters of the first 10 modes. The first mode ($T_1 = 1.23$ s) is the first-order longitudinal vibration of the entire unit, with the main girder translating longitudinally and driving the thin-walled piers to bend longitudinally; its longitudinal mass participation ratio reaches 38.37%, making it the dominant mode controlling the longitudinal seismic response of the rigid-frame system. The second mode ($T_2 = 1.18$ s) is characterized by first-order transverse bending, with the main girder and the piers bending laterally in the same direction, and the transverse participation mass is 41.37%, representing the principal dynamic mode in that direction. The periods of the first two modes are close, but their directions are orthogonal and the degree of coupling is low, indicating that the longitudinal and transverse dynamic characteristics of the rigid-frame part are relatively independent. The third mode ($T_3 = 1.07$ s) exhibits second-order longitudinal vibration, with anti-symmetric longitudinal bending of the main girder accompanied by reverse displacement of the pier tops, and a longitudinal participation of 2.89%. The fourth mode ($T_4 = 0.85$ s) is a second-order transverse bending–torsion coupled vibration, with a transverse participation of only 0.35%, showing a distinct local behavior. The fifth mode ($T_5 = 0.82$ s) again manifests as higher-order longitudinal vibration, with multiple inflection points appearing along the piers, and a longitudinal participation of 0.99%. From the sixth mode onward, the periods shorten rapidly, the participation masses are very small, and the vibration shapes are mostly higher-order distortion of the box girder cross-section, local bending of piers, or coupled bending–torsion combinations. The tenth mode ($T_{10} = 0.30$ s) has a transverse participation of 1.10%, representing higher-order transverse bending, but its absolute contribution remains limited.

A comparison with the dynamic characteristics of the full-bridge system in Section 3.1 reveals that the fundamental frequency of the continuous rigid-frame bridge is significantly higher (0.81 Hz versus 0.30 Hz for the full bridge), the period is substantially shortened, and the frame effect markedly increases the longitudinal and transverse stiffness of the structure. Moreover, the long-period longitudinal drifting mode present in the full-bridge system no longer appears in the isolated rigid-frame bridge, confirming the controlling influence of the bearing arrangement and the differences in structural systems on the overall dynamic characteristics. The above modal parameters will provide a direct basis for the seismic response analysis and damping parameter calibration of the rigid-frame bridge portion.

3.3. Quantitative Assessment of Higher-Mode Contributions

To quantitatively evaluate the contribution of higher modes to the seismic response of the structure, elastic response calculations were performed using the mode superposition response spectrum method (with CQC combination) under the E1-level design response spectrum. The first 10, 50, 150, and 500 modes were successively included in the analysis. The pier-base moments and pier-top displacements of a typical continuous-beam pier P7 (height 38.5 m) and a rigid-frame pier P13 (height 44.0 m) were selected as

comparison indices, and the computed results are listed in Table 6. When only the first 10 modes are considered, the pier-base moment of P13 reaches only 70.2% of the full-modal (first 500 modes) result, and the pier-top displacement reaches only 69.2%; even when the number of modes is increased to 50, the displacement and internal forces still exhibit a deficiency of approximately 10–15%. This indicates that modes beyond the 10th order make a non-negligible contribution to the structural response. In particular, for flexible high-pier structures, the local bending effects excited by higher vibration modes can produce significant increases in internal forces at critical sections. Therefore, it is necessary to adopt the nonlinear time-history direct integration method that accounts for the contributions of all 500 modes in the final analysis.

Table 6. Comparison of key section responses under different modal truncation orders (E1 response spectrum method).

Index	Pier No.	First 10 Modes	First 50 Modes	First 150 Modes	First 500 Modes	First 10/First 500
Pier-base longitudinal moment (kN·m)	P7	4520	5350	5680	5720	79.0%
Pier-base longitudinal moment (kN·m)	P13	18,600	23,700	25,700	26,500	70.2%
Pier-top longitudinal displacement (mm)	P7	12.3	15.1	16.4	16.7	73.7%
Pier-top longitudinal displacement (mm)	P13	28.5	35.6	39.8	41.2	69.2%

It should be noted that the nonlinear time-history analysis employs the Hilber–Hughes–Taylor (HHT) direct integration method, which solves the full system dynamic equilibrium equations at each time step without modal truncation. Consequently, it automatically encompasses the responses of all vibration modes represented by the discretized finite element model (up to the frequency corresponding to the 500th mode).

3.4. Assessment of Torsional Irregularity

To quantify the degree of torsional irregularity of this curved bridge, a torsional irregularity coefficient R_T is defined as the ratio of the cumulative torsional mass participation ratio of the torsion-dominated modes (torsional mass participation ratio $\geq 50\%$) among the first N modes to the total torsional mass participation ratio of the first 500 modes. From the results presented in Tables 4 and 5, the third mode of the entire bridge ($T = 2.13$ s) already exhibits a torsional participation ratio of 44.8%, approaching the dominance threshold. The cumulative torsional participation ratio of the torsion-dominated modes within the first 20 modes is approximately 56%, accounting for 82% of the total torsional mass ($56/68.2$), yielding $R_T = 0.82$. In the rigid-frame portion, the fourth mode ($T = 0.85$ s) attains a torsional participation ratio of 48.2%, which also represents a significant torsional mode. The relatively high R_T value indicates that, under seismic excitation, horizontal ground motions can readily induce substantial torsional vibration components in this bridge. Combined with the inherent flexural–torsional coupling effect of the curved alignment, this is expected to generate non-negligible additional torsional moments.

4. Seismic Response Analysis

4.1. Pier-Top Displacement Response of the Full Bridge

To systematically reveal the displacement response patterns of the full bridge under near-field pulse-like, near-field non-pulse, and far-field ground motions, the five typical seismic records listed in Table 3, namely, near-field short pulse (NSP), near-field moderate pulse (NMP), near-field long pulse (NLP), near-field non-pulse (NNP), and far-field (FF) ground motions, were input under two seismic fortification levels, E1 and E2. The excitations were applied in two orthogonal combinations, i.e., transverse–vertical and longitudinal–vertical, and the maximum horizontal displacements at the top of each pier were extracted. Figure 10 and Figure 11 show the distributions of the pier-top displacements along the bridge under transverse–vertical and longitudinal–vertical excitations for the E1 and E2 levels, respectively.

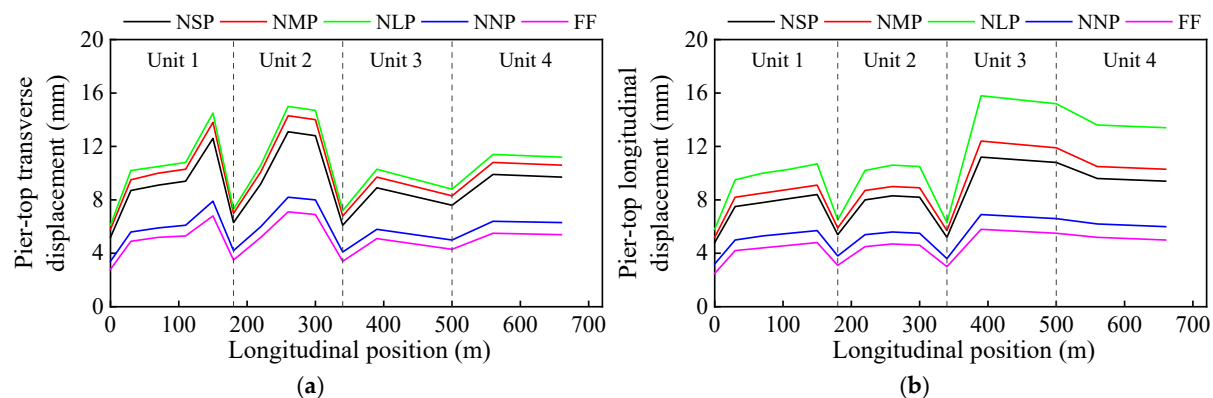


Figure 10. Pier-top displacement response under E1-level earthquakes: (a) transverse–vertical excitation; (b) longitudinal–vertical excitation.

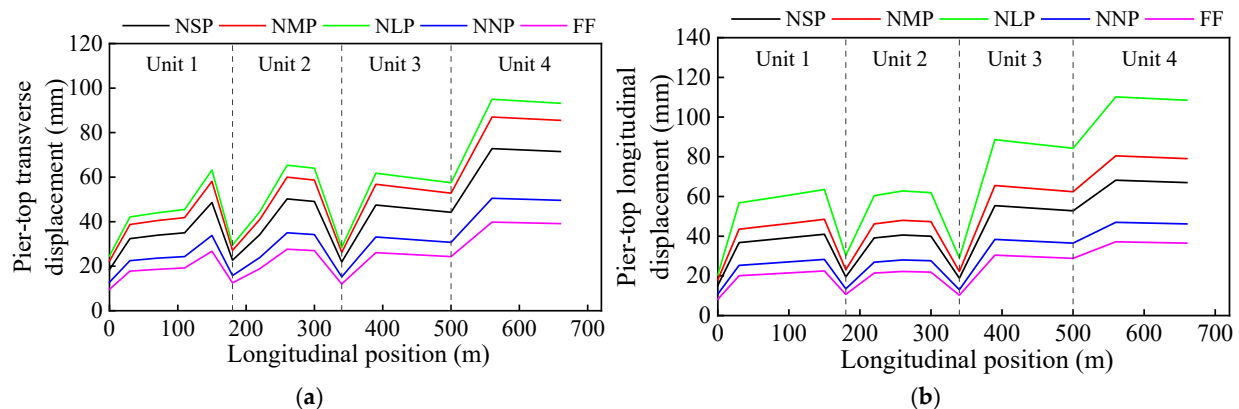


Figure 11. Pier-top displacement response under E2-level earthquakes: (a) transverse–vertical excitation; (b) longitudinal–vertical excitation.

Under the E1 seismic action (Figure 10), the displacement responses of all bridge piers generally remained at low levels. Neither the transverse displacements under transverse–vertical excitation nor the longitudinal displacements under longitudinal–vertical excitation exceeded the allowable design ranges, indicating that the bridge essentially remained elastic at the E1 level. The transverse displacements were relatively prominent at the moderately high piers in the curved segments (e.g., P4, P7, and P8), which is primarily attributable to the bending–torsion coupling effect of the curved bridge and the uneven stiffness distribution caused by differences in pier heights. The longitudinal displacements were comparatively larger at the transition piers connecting the composite girder units and the rigid-frame unit (P10 and P12) and at the side piers of the rigid frame, reflecting the concentration

of longitudinal deformation at the junctions of different structural systems due to abrupt changes in stiffness. Among the five ground motions, the displacement values induced by the near-field pulse records NMP and NLP were noticeably greater than those induced by NNP and FF; in particular, NLP, owing to its longest pulse period and prominent velocity pulse effect, had already caused relatively significant longitudinal displacements.

At the E2 level (Figure 11), the displacements of all piers generally increased substantially, and the degree of nonlinearity intensified. Under transverse-vertical excitation, the peak transverse displacements occurred at the main piers P13 and P14 of the fourth-unit rigid frame and at the adjacent transition piers. This was primarily caused by the strong transverse velocity pulses of the near-field pulse-like ground motions, together with the high transverse stiffness of the rigid-frame system, which led to the direct transfer of inertia forces at the pier-girder rigid joints. Under longitudinal-vertical excitation, the distribution of longitudinal displacements along the bridge exhibited pronounced non-uniformity: in the composite girder portions of the first to third units, where high damping rubber bearings were installed, the relatively low post-yield stiffness of the bearings induced large shear deformations, which increased the relative displacement between the girder and the piers and correspondingly amplified the longitudinal displacements of the piers themselves; in the rigid-frame portion of the fourth unit, owing to the pier-girder rigid connections, the pier-top longitudinal displacements directly reflected the vibration of the main girder. Under the action of long-period pulses such as NLP, the low-order longitudinal drifting mode of the rigid frame was fully excited, and the displacements at the tops of P13 and P14 reached the maximum values for the entire bridge. Comparing the pulse and non-pulse records, at the E2 level, the displacements induced by the three pulse-like ground motions (NSP, NMP, and NLP) were all significantly larger than those induced by NNP and FF, and the longer the pulse period, the more pronounced the displacement amplification effect, which demonstrates the high damage potential of near-fault velocity pulses to long-period structures. Although the far-field ground motion FF induced certain transverse displacements in the higher-mode regions, its amplitude was relatively low, and the longitudinal displacement was even more limited.

It is worth noting that at locations where high damping rubber bearings were installed, P1–P4, P6–P8, P10 and 11#, the bearings entered a distinct nonlinear hysteretic energy dissipation stage under the strong E2 earthquake, effectively limiting the shear forces transmitted to the piers and thereby reducing the displacement ductility demand on the piers themselves. However, the large deformations of the bearings also induced significant relative displacements between the main girder and the piers, imposing higher requirements on the deformation capacity of the expansion joints and unseating prevention devices. By contrast, although the displacements at the rigid-frame main piers P13 and P14 without bearings were large, the force transfer paths were clear and can be controlled through rational ductility design.

Overall, the displacement response of the full bridge exhibits the following regular characteristics: near-field velocity pulse-like ground motions, especially records with long-pulse periods, induce the most unfavorable displacement responses; the vertical ground motion component exacerbates the transverse bending-torsion displacements of the curved girder segments through spatial coupling effects; and the high damping rubber bearings protect the piers through yielding and energy dissipation at the E2 level, but due attention must be given to the bearing displacements and the coordinated design of adjacent components.

4.2. Pier-Base Internal Force Response of the Full Bridge

To evaluate the strength demand and potential damage states of the piers under near-field and far-field ground motions, the maximum shear forces and bending moments at the pier-base sections under transverse–vertical and longitudinal–vertical excitations at the E2 level were extracted. Figure 12 and Figure 13 present the distributions of the pier-base shear forces and bending moments, respectively, along the bridge under the five seismic records.

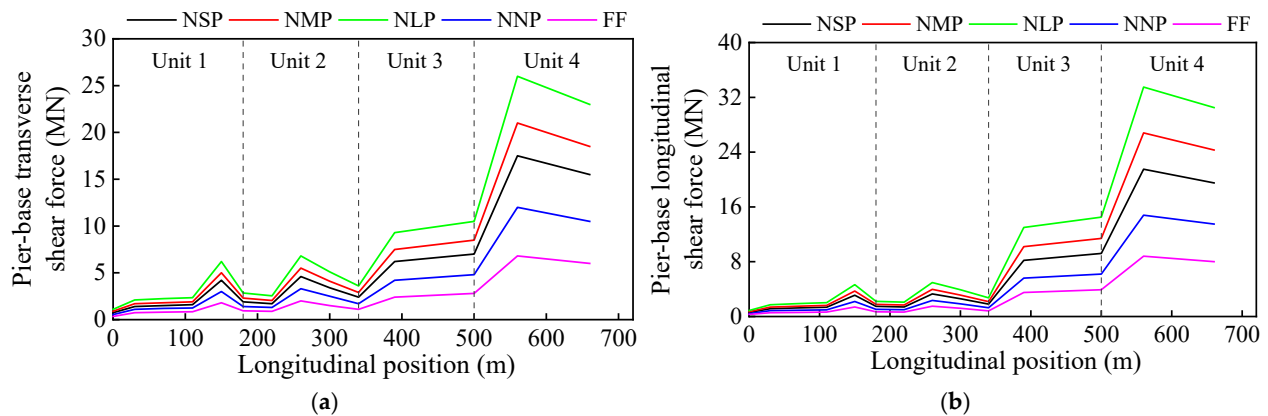


Figure 12. Pier-base shear force response under E2-level earthquakes: (a) transverse–vertical excitation; (b) longitudinal–vertical excitation.

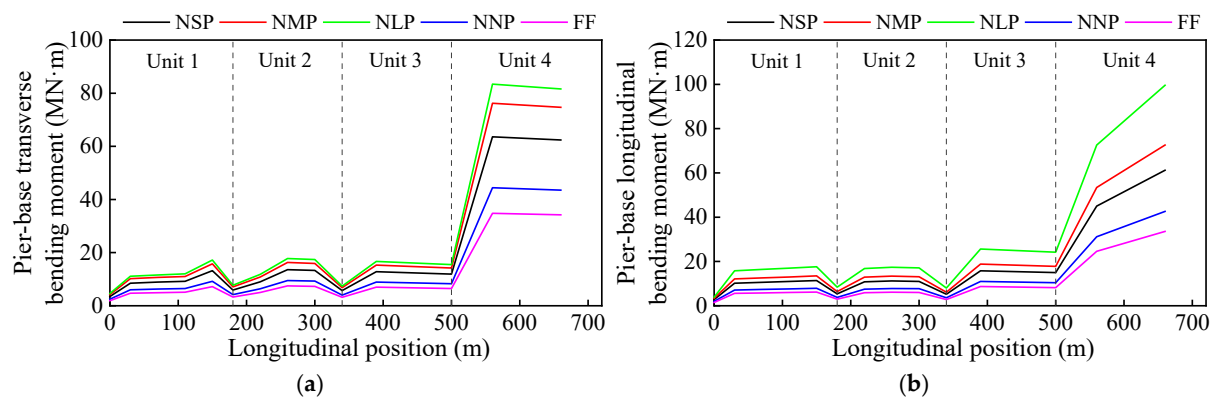


Figure 13. Pier-base bending moment response under E2-level earthquakes: (a) transverse–vertical excitation; (b) longitudinal–vertical excitation.

Under transverse–vertical excitation (Figures 12a and 13a), the distribution patterns of the transverse shear forces and transverse bending moments at the pier bases are consistent with those of the pier-top displacements: the peak internal forces occur at the main piers P13 and P14 of the fourth-unit continuous rigid frame. This is primarily because the pier–girder rigid joints directly transfer the enormous inertia forces of the main girder to the piers, and, combined with the pronounced velocity pulse effect of near-field pulse-like ground motions, the internal forces at the rigid-frame pier bases are far greater than those at the other piers. The internal forces at the transition piers P10 and P12 adjacent to the rigid frame, as well as at the moderately high piers P4, P7, and P8 in the curved segments, are the next highest; the bending–torsion coupling effect of the curved bridge and the differences in pier heights further exacerbate the forces at these locations. For piers P1–P4 and P6–P8 in the composite girder units where high damping rubber bearings are installed, the hysteretic energy dissipation of the bearings limits the shear forces transmitted to the pier tops, resulting in relatively small transverse internal forces at the pier bases. However, the increased deformation after bearing yielding may amplify the P-Δ effect in the piers, which requires due attention. The pier-base internal forces induced by the near-field

moderate-pulse (NMP) and long-pulse (NLP) ground motions are significantly higher than those induced by the near-field non-pulse (NNP) and far-field (FF) records, and the longer the pulse period, the more pronounced the internal force amplification effect. The far-field ground motion FF produces the lowest internal force levels.

Under longitudinal–vertical excitation (Figures 12b and 13b), the longitudinal shear forces and longitudinal bending moments at the pier bases exhibit a markedly non-uniform distribution along the bridge. In the composite girder portions of the first to third units, since the high damping rubber bearings enter the post-yield sliding stage with low stiffness at the E2 level, the bearing shear deformation dissipates part of the seismic energy, and the longitudinal shear forces and bending moments carried by the piers are effectively controlled, though they are slightly higher than those at the E1 level due to the increased relative displacement between the girder and the piers. The peak longitudinal internal forces remain concentrated in the rigid-frame portion: the fixed joints at the main piers P13 and P14 sustain the longitudinal inertia forces and overturning moments of the main girder, and the longitudinal bending moments at the bases of these two piers reach the maximum for the entire bridge under longitudinal–vertical excitation. The NLP long-pulse record, possessing the longest pulse period, significantly amplifies the longitudinal drifting response of the main girder, causing the pier-base internal forces at P13 and P14 to be the highest among all ground motion types. The transition piers P10 and P12, located at the system changeover points, experience considerable longitudinal shear forces due to abrupt stiffness changes and differences in the dynamic characteristics of adjacent units, forming another vulnerable link. Although the near-field non-pulse record NNP has the largest PGA, it contains no regular velocity pulses and its energy is concentrated in the high-frequency range, resulting in internal force amplitudes far lower than those of the pulse-like records.

By synthesizing the displacement and internal force response patterns, it can be concluded that near-field velocity pulse-like ground motions at the E2 level constitute the governing loading scenario for the strength demand of the bridge piers. The long-pulse-period records exert a particularly pronounced internal force amplification effect on the rigid-frame main piers and the system transition piers. The high damping rubber bearings effectively protect the piers in the composite girder units through yielding and hysteretic dissipation; however, due to the pier–girder rigid connections in the rigid-frame portion, internal forces are highly concentrated. It is recommended that ductility enhancement measures be adopted for these pier shafts in design and that a capacity protection design be performed for the transition piers. The vertical ground motion component exerts a non-negligible influence on the flexure–shear coupling and shear capacity of the piers through spatial coupling effects and axial force variations, and should be fully taken into account.

4.3. Displacement and Internal Force Response of Continuous Girder Piers

To investigate the distribution patterns of seismic responses along the pier shaft for the continuous girder bridge portion, three representative piers were selected from the composite girder units for comparative analysis: the tallest pier P7 (38.5 m), a medium-height pier P8 (30.5 m), and the shortest pier P10 (23.5 m). All three piers adopt double-column circular sections with a diameter of 2.4 m, and cap beams are provided at the pier tops, connecting to the main girder via high damping rubber bearings (P7 and P8: HDR(I)-d920×325-G1.0; P10: HDR(I)-d1020×374-G1.0). The envelope values of displacement, shear force, and bending moment distributed along the pier height were extracted for these three piers under the five ground motions (NSP, NMP, NLP, NNP, FF) at the E2 level, applied in the transverse–vertical and longitudinal–vertical directions, respectively. The results are shown in Figures 14 and 15.

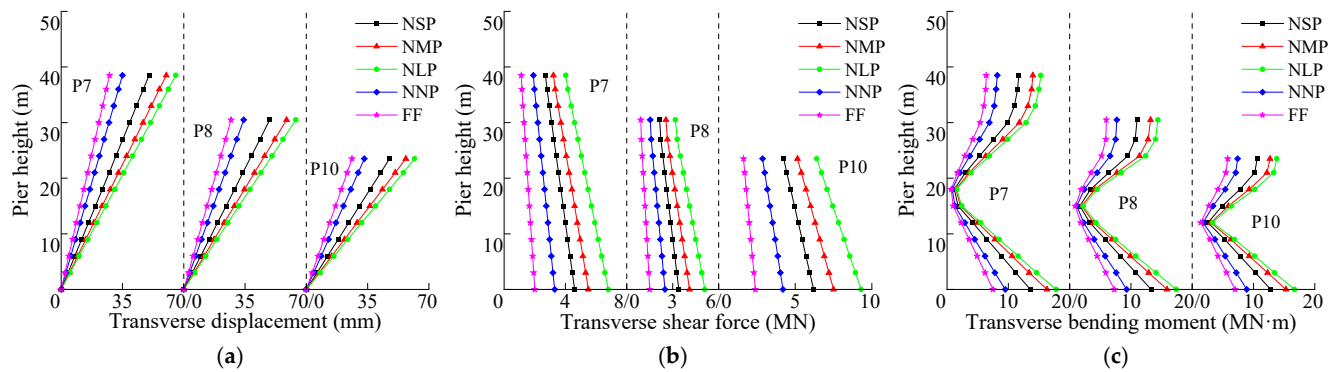


Figure 14. Displacement and internal force responses along pier height of continuous girder piers under E2 transverse-vertical excitation: (a) displacement envelope; (b) shear force envelope; (c) bending moment envelope.

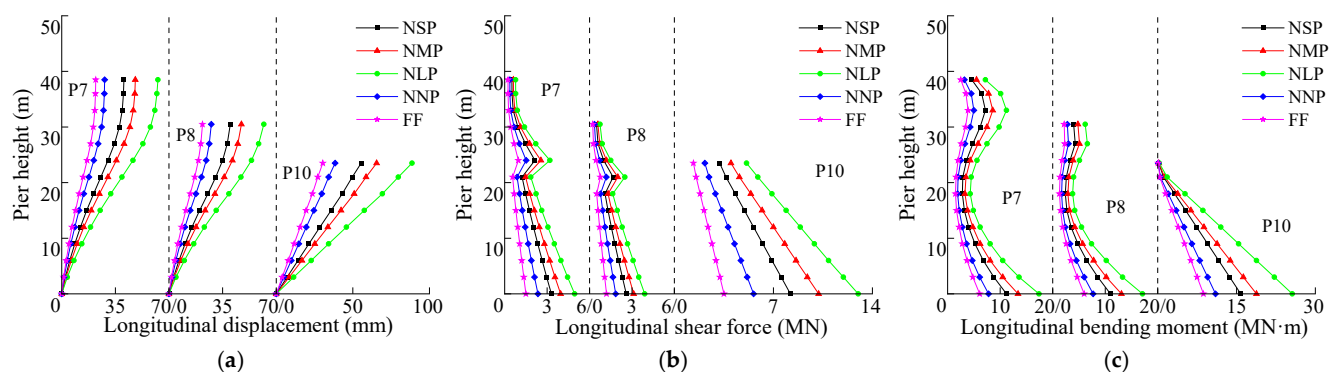


Figure 15. Displacement and internal force responses along pier height of continuous girder piers under E2 longitudinal-vertical excitation: (a) displacement envelope; (b) shear force envelope; (c) bending moment envelope.

As can be seen from Figure 14, under transverse-vertical seismic excitation, large bending moment responses are developed at both the pier top and pier base, with an inflection point located approximately at the mid-height of the pier (about 1/2 pier height). This distribution pattern arises from the closed-frame effect formed by the transverse connection of the cap beams on top of the double-column piers: the cap beams impose significant rotational restraint on the pier tops, resulting in fixed-end moments at both the top and bottom of the pier columns, and the bending moment distribution along the pier height is anti-symmetric, with the inflection point near the mid-height. The displacement envelope exhibits a predominantly shear-type distribution in the transverse direction; the pier-top displacement is the largest for the tall pier P7, followed by the medium-height pier P8, and is the smallest for the short pier P10 due to its greater transverse lateral stiffness. The shear force envelope is relatively uniformly distributed along the pier height. P10 exhibits the largest pier-base shear force among the three piers, reflecting the characteristics of a short pier with high stiffness that attracts larger seismic forces, which is consistent with the shear force distribution pattern of all bridge piers described in Section 4.2. As for the bending moment envelope, P7 and P8, due to their greater pier heights, show relatively significant pier-base bending moments; although P10 has a large base shear, its moment arm is short, so its pier-base bending moment is close to that of P8. Among the five ground motions, the pier responses induced by NLP and NMP are significantly greater than those induced by NNP and FF, and the long-pulse record exerts a more pronounced amplification effect on the displacement and bending moment of the tall pier.

Under longitudinal-vertical seismic excitation (Figure 15), the piers behave in a cantilever manner in the longitudinal direction: the pier base is fixed to the pile cap, while

the pier top is connected to the main girder through HDRBs, whose longitudinal shear deformation provides limited restraint. Significantly different from the transverse direction, the pier column cross-section in the longitudinal direction is relatively smaller, resulting in greater flexibility, and the participation of higher-order modes is pronounced. The displacement envelopes of the tall pier P7 and medium-height pier P8 exhibit curvature bending in the upper portion of the pier, and both the shear force and bending moment envelopes display an approximately S-shaped nonlinear distribution: an inflection point exists at the pier base, the bending moment direction reverses at the mid-height, and a local peak appears at approximately 0.6–0.7 times the pier height, which is closely related to the contributions of the second and higher modes. The higher-order effects may cause the bending moment demand at certain sections in the mid-height of the pier to exceed that at the pier base, imposing additional requirements on the shear design. In contrast, the internal force envelopes of the shortest pier P10 are essentially linear, with the bending moment decreasing monotonically from the base to the top; the influence of higher-order modes is significantly reduced due to the lower pier height, and the displacement is also much smaller than that of the two taller piers. Among the various ground motions, the NLP long-pulse record produces the most drastic amplification of the longitudinal displacement and the S-shaped bending moment envelope of the tall pier P7, followed by NMP, further confirming the extremely adverse effect of near-field velocity pulses on flexible tall piers. The far-field ground motion FF induces the smallest longitudinal response, and the envelope shapes for the three piers are essentially consistent with those under the pulse records, with only the amplitudes reduced.

Based on the above analysis, the combined action of the transverse frame effect and the longitudinal cantilever higher-order effect leads to significant directional differences and height sensitivity in the seismic response of continuous girder bridge piers. For tall piers, the S-shaped bending moment distribution and the phenomenon of the mid-height bending moment exceeding that at the base, caused by higher-order modes under longitudinal earthquakes, should be fully considered in seismic design. Especially for near-field pulse-like ground motions, it is advisable to appropriately densify the stirrup arrangement at the mid-height of the piers and to verify the flexural and shear capacities of the controlling sections.

It should be noted that the S-shaped bending moment distribution described above, along with the phenomenon that the bending moment in the middle of the pier surpasses that at the base, stems from the significant contribution of higher vibration modes. The nonlinear time-history analysis in this study employs the direct integration method without modal truncation, thereby automatically incorporating the contributions of all higher modes represented by the discretized model. Consequently, the secondary peaks observed in the envelope diagrams can be confirmed as a genuine manifestation of higher-mode effects in the time-history response, rather than an indirect inference derived from modal decomposition.

4.4. Displacement and Internal Force Response of Rigid-Frame Piers

To reveal the response patterns of continuous rigid-frame bridge piers under longitudinal seismic excitation, three representative variable-section hollow thin-walled piers were selected from the fourth unit of the rigid-frame bridge for comparative analysis: the tallest pier P13 (44.0 m), a medium-height pier P14 (39.5 m), and the shortest pier P12 (29.0 m). The detailed structural configuration parameters of each pier are listed in Table 7. Their common features are the adoption of a single pier form, wall thickness varying in a stepped manner along the pier height, solid segments provided at the pier top and base, a predominantly thin-walled hollow section in the middle portion, and multiple transition segments arranged at the solid–hollow transition zones as well as in the upper-middle portion of the pier. Given that the longitudinal direction is the primary force-resisting direction for this

type of rigid-frame pier, only the envelopes of pier displacement, shear force, and bending moment under the five ground motions (NSP, NMP, NLP, NNP, FF) at the E2 level with longitudinal-vertical excitation are extracted herein, as shown in Figure 16.

Under longitudinal-vertical seismic excitation, the displacement envelopes of the three piers increase gradually from the pier base to the pier top (Figure 16a). The increment is relatively uniform in the thin-walled hollow segment and tends to level off in the top solid segment due to the abrupt increase in the flexural stiffness of the cross-section. The pier-top displacement increases significantly with the pier height: P13 exhibits the largest displacement, followed by P14, and P12 the smallest, reflecting the controlling effect of pier height on the longitudinal flexibility of the rigid-frame system. Because the near-field long-pulse ground motion NLP has the longest pulse period, the displacements of all piers far exceed those under the other cases; NMP ranks second, and FF produces the smallest displacements.

The shear force and bending moment envelopes exhibit complex nonlinear distributions closely related to the variable cross-section configuration and higher-order modes (Figure 16b,c). The shear force and bending moment envelopes of the tall pier P13 and the medium-height pier P14 both display a typical S-shaped characteristic: a local peak appears within a range of approximately 0.60–0.66 times the pier height measured from the base. This phenomenon primarily arises from two factors. First, the wall thickness in the upper-middle portion of the hollow thin-walled pier is relatively thin, and the cross-sectional flexural stiffness is correspondingly low, which increases the participation of the second and higher modes and induces an internal force redistribution in the upper-middle portion of the pier. Second, this height range coincides exactly with the upper-middle transition segment shown in Table 7, and the sudden change in wall thickness further intensifies the local internal force concentration. In contrast, for the shortest pier P12, because of its limited height, the fundamental mode dominates and the higher-order effects are weak; its shear force and bending moment envelopes decrease essentially linearly from the pier base to the pier top, without exhibiting an obvious S-shaped resurgence. This S-shaped distribution further confirms the dominant influence of higher modes on the internal forces of variable cross-section tall piers. Because the nonlinear time-history direct integration method solves the full-system dynamic equilibrium equations at each integration step, the local bending excited by higher vibration modes beyond the 10th order is fully retained and reflected in the envelope diagrams. Therefore, the secondary peaks located in the $0.6H$ – $0.7H$ region of the pier height, as well as the abrupt changes in internal forces at the transition zone of the variable cross-section, are not numerical noise but rather the direct consequence of the coupling between higher-mode effects and sudden changes in sectional stiffness.

Specifically, the displacement envelope increases most rapidly in the thin-walled hollow segment, and the slope gradually decreases after entering the top transition segment and solid segment. The shear force envelope reaches its maximum value in the bottom solid segment and the lower transition zone, and then decreases toward the pier top. For P13 and P14, the shear force exhibits a secondary rise in the upper-middle portion of the pier (at approximately $0.65H$), forming an S-shaped peak, and the magnitude of the rise increases with the pier height. At the transition positions where the wall thickness changes abruptly, obvious inflection points or steps can be observed in the shear force envelope curve. The pier-base bending moment envelope is the largest; in tall piers it decays rapidly with increasing height and crosses the zero point (inflection point), then increases in the reverse direction and forms a second peak in the upper-middle portion of the pier, after which it gradually decreases toward the pier top. Owing to the pier-girder rigid connection, a certain residual bending moment remains at the pier top. The S-shaped peak bending moment of P13 can reach several times the average bending moment of adjacent sections, imposing a higher demand on the flexural capacity. The bending moment of P12, on the

other hand, decays nearly linearly from the maximum value at the base to the pier top, with the top bending moment approaching zero.

Table 7. Structural configuration of variable-section hollow thin-walled piers of the rigid-frame bridge.

Pier	Height Range (m)	Pier Segment	Cross-Section
P12	27–29	Top solid segment	3.3 m (long.) × 5.6 m (trans.)
	25–27	Top transition segment (from lower edge to upper edge of top hollow chamfer)	0.6 m (wall thickness)–1.1 m (wall thickness)
	5–25	Mid-height hollow segment (from upper edge of bottom hollow chamfer to lower edge of top hollow chamfer)	0.6 m (wall thickness)
	3–5	Bottom transition segment (from lower edge to upper edge of bottom hollow chamfer)	1.1 m (wall thickness)–0.6 m (wall thickness)
	0–3	Bottom solid segment	3.3 m (long.) × 5.6 m (trans.)
P13	43–44	Top solid segment	4.5 m (long.) × 7.6 m (trans.)
	40–43	Top transition segment (from lower edge to upper edge of top hollow chamfer)	0.7 m (wall thickness)–1.2 m (wall thickness)
	23–40	Upper hollow segment (from upper edge of upper-middle hollow chamfer to lower edge of top hollow chamfer)	0.7 m (wall thickness)
	22.5–23	Upper-middle transition segment (from lower edge to upper edge of upper-middle hollow chamfer)	1.2 m (wall thickness)–0.7 m (wall thickness)
	22–22.5	Mid-height hollow segment (from upper edge of lower-middle hollow chamfer to lower edge of upper-middle hollow chamfer)	4.5 m (long.) × 7.6 m (trans.)
	21.5–22	Lower-middle transition segment (from lower edge to upper edge of lower-middle hollow chamfer)	0.7 m (wall thickness)–1.2 m (wall thickness)
	5–21.5	Lower hollow segment (from upper edge of bottom hollow chamfer to lower edge of lower-middle hollow chamfer)	0.7 m (wall thickness)
	2–5	Bottom transition segment (from lower edge to upper edge of bottom hollow chamfer)	1.2 m (wall thickness)–0.7 m (wall thickness)
P14	0–2	Bottom solid segment	4.5 m (long.) × 7.6 m (trans.)
	38.5–39.5	Top solid segment	4.5 m (long.) × 7.6 m (trans.)
	35.5–38.5	Top transition segment (from lower edge to upper edge of top hollow chamfer)	0.7 m (wall thickness)–1.2 m (wall thickness)
	20.5–35.5	Upper hollow segment (from upper edge of upper-middle hollow chamfer to lower edge of top hollow chamfer)	0.7 m (wall thickness)
	20–20.5	Upper-middle transition segment (from lower edge to upper edge of upper-middle hollow chamfer)	1.2 m (wall thickness)–0.7 m (wall thickness)
	19.5–20	Mid-height hollow segment (from upper edge of lower-middle hollow chamfer to lower edge of upper-middle hollow chamfer)	4.5 m (long.) × 7.6 m (trans.)
	19–19.5	Lower-middle transition segment (from lower edge to upper edge of lower-middle hollow chamfer)	0.7 m (wall thickness)–1.2 m (wall thickness)
	5–19	Lower hollow segment (from upper edge of bottom hollow chamfer to lower edge of lower-middle hollow chamfer)	0.7 m (wall thickness)
	2–5	Bottom transition segment (from lower edge to upper edge of bottom hollow chamfer)	1.2 m (wall thickness)–0.7 m (wall thickness)
	0–2	Bottom solid segment	4.5 m (long.) × 7.6 m (trans.)

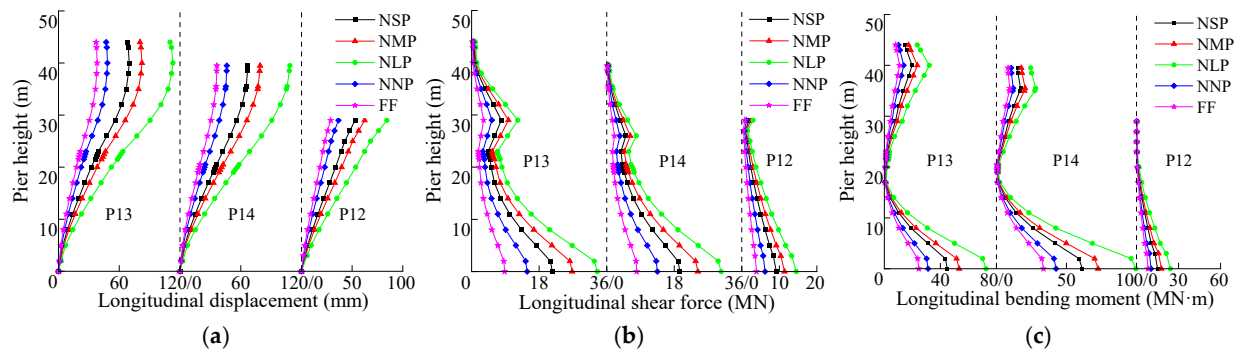


Figure 16. Displacement and internal force responses along pier height of rigid-frame piers under E2 longitudinal-vertical excitation: (a) displacement envelope; (b) shear force envelope; (c) bending moment envelope.

Among the five ground motions, the internal force responses induced by the near-field pulse-like records (NSP, NMP, NLP) are all significantly larger than those induced by the near-field non-pulse (NNP) record and the far-field (FF) record. Under the action of NLP, the S-shaped distribution characteristics of each pier are the most prominent, and the internal force peaks reach the highest among all cases. Furthermore, influenced by the cross-sectional dimensions, wall thickness distribution, and stiffness differences, the pier-base bending moment of P14 is the largest among the three piers, indicating that under specific cross-sectional designs, a medium-height pier may also face relatively high flexural demands.

In summary, the response of variable-section hollow thin-walled piers of the rigid-frame bridge under longitudinal seismic excitation is dually controlled by pier height and cross-sectional variation. The S-shaped internal force distribution in tall piers and the local abrupt changes at the variable cross-section locations constitute seismically vulnerable regions. It is recommended that, in seismic design, the stirrups be appropriately densified in the upper-middle portion of the pier (approximately 0.6–0.7 times the pier height), local strengthening be applied at the wall thickness transition segments, and the flexural and shear capacities of critical sections such as the interface between the bottom solid segment and the transition segment be checked. Meanwhile, the adverse effects of near-fault long-period velocity pulses on tall and flexible bridge piers should be fully considered, and measures such as adding energy dissipation devices or enhancing ductility detailing should be taken where necessary.

4.5. Hysteretic Response of HDRB

To thoroughly investigate the hysteretic energy dissipation characteristics of HDRBs under seismic excitations of different levels and types, the bearing installed at the top of the tallest continuous girder pier, P7 (pier height 38.5 m), was selected as the analysis object. Figure 17 and Figure 18 present the force-displacement hysteretic curves of this bearing under the five typical ground motions applied in the longitudinal-vertical direction at the E1 and E2 fortification levels, respectively.

Under the E1 seismic level (10% probability of exceedance in 50 years), as can be seen from Figure 17, the hysteretic loops of the bearing in all cases exhibit a slender spindle shape with limited enclosed area. The displacement amplitudes under the near-field non-pulse (NNP) and far-field (FF) records are only about 5.6 mm and 4.2 mm, respectively, far smaller than the yield displacement of the bearing; the displacement amplitudes under the near-field pulse-like records NSP, NMP, and NLP increase successively, with NLP reaching a maximum of approximately 10.7 mm. The hysteretic curves are nearly linear, with very small residual displacement after unloading, indicating that the bearing essentially remains

in the elastic shear deformation stage and dissipates only a negligible amount of energy through the inherent damping of the rubber material. In conjunction with Figure 10, the pier-top displacement of P7 at the E1 level remains at a low level, and the bearing provides effective elastic restraint through its initial horizontal stiffness, limiting the excessive development of the relative displacement between the girder and the pier. Meanwhile, Figure 12 shows that the pier-base shear force also remains elastic, indicating that at the E1 level the HDRB primarily performs the function of connection and force transmission, while its seismic isolation and energy dissipation effects have not yet been fully mobilized.

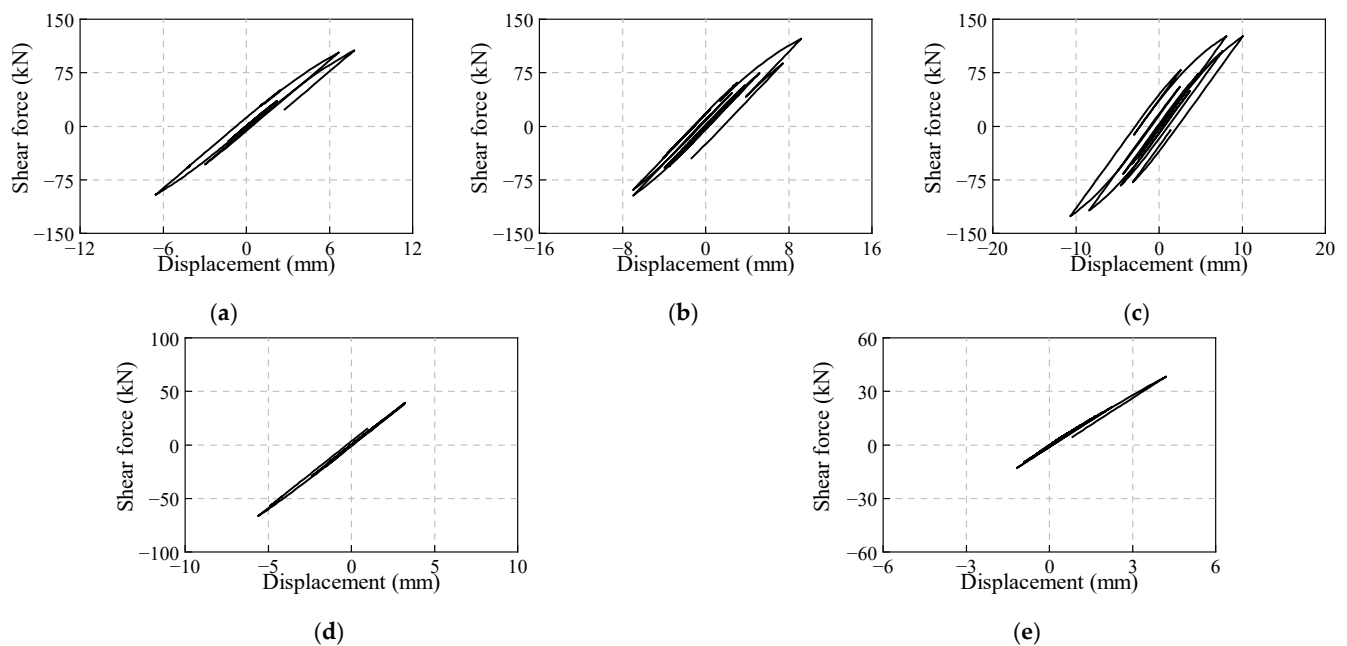


Figure 17. HDRB hysteretic response under E1 longitudinal-vertical excitation: (a) NSP; (b) NMP; (c) NLP; (d) NNP; (e) FF.

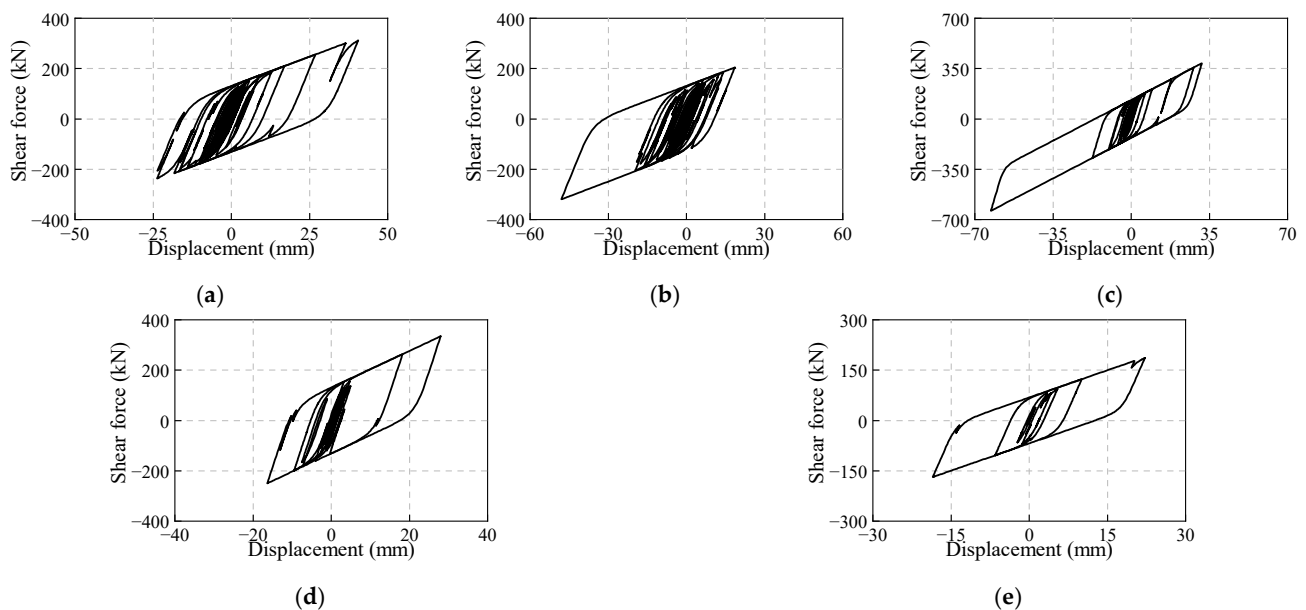


Figure 18. HDRB hysteretic response under E2 longitudinal-vertical excitation: (a) NSP; (b) NMP; (c) NLP; (d) NNP; (e) FF.

After entering the E2 seismic level (2% probability of exceedance in 50 years), the hysteretic behavior of the bearing undergoes an essential change. In Figure 18, the hysteretic

loops under all ground motions are significantly broadened, transitioning from spindle shapes to full parallelogram-like or elliptical shapes, reflecting distinct post-yield stiffening and hysteretic energy dissipation characteristics. The bearing displacements induced by the near-field non-pulse NNP and far-field FF records increase to approximately 28.0 mm and 22.2 mm, respectively, with the hysteretic loops remaining relatively regular; in contrast, the responses induced by the three near-field pulse records are far more intense: the bearing displacement reaches approximately 40.6 mm under NSP, 48.0 mm under NMP, and as high as 62.8 mm under NLP, which is already close to the design allowable displacement of this bearing type. The hysteretic loop under the NLP case is particularly full, with its enclosed area reaching more than ten times that of the FF case, and force cusps as well as brief stiffness degradation signs caused by the velocity pulse effect appear at the loading–unloading reversal points.

The large deformation energy dissipation of the bearing provides a remarkable protective effect for the substructure. A comparison with Figure 13 reveals that, although the bearing displacement increases drastically at the E2 level, the longitudinal bending moment at the base of P7 is still relatively small among all continuous girder piers. This demonstrates that the bearing effectively reduces the seismic shear forces and bending moments transmitted to the pier through its low post-yield stiffness sliding and hysteretic energy dissipation, thereby limiting the degree of plastification of the pier. At the same time, attention must be paid to the unseating risk potentially induced by excessive bearing deformation. The bearing shear deformation of up to 62.8 mm under the NLP case in Figure 18 imposes high demands on the deformation capacity of expansion joints and unseating prevention devices.

Comparing Figures 17 and 18, the hysteretic characteristics of the bearing exhibit a significant sensitivity to the spectral characteristics of the ground motions. Near-field pulse-like ground motions, by virtue of their large amplitudes and prominent velocity pulses, can inject considerable kinetic energy into the structure within a very short time, causing the bearing to yield rapidly and enter large-amplitude cyclic energy dissipation, with full hysteretic loops and high energy dissipation efficiency. The non-pulse near-field (NNP) record, although possessing a relatively large peak ground acceleration, lacks regular pulses and its energy is concentrated in the high-frequency range, resulting in bearing displacements and energy dissipation far inferior to those of the pulse records. The far-field (FF) record, due to its relatively rich long-period components, can still sustain certain reciprocating displacement amplitudes after the bearing yields, but the fullness of its hysteretic loops and the total energy dissipation are still significantly lower than those of the pulse-like ground motions.

The above analysis indicates that the NLP record, with a pulse period $T_p = 12.0$ s that is much longer than the fundamental period of the bridge ($T_1 \approx 3.3$ s) and that of its rigid-frame portion ($T_1 \approx 1.2$ s), nevertheless produced the most unfavorable displacements, internal forces, and damage. This phenomenon can be explained from three aspects: the acceleration response spectrum of the long-period velocity pulse maintains relatively high spectral ordinates in the period range of 0.5–5.0 s, providing sufficient energy input to structures within this period range; the large bearing deformations and substantial girder drift induced by the high PGV lead to a sharp increase in the displacement demand of the isolation system, indirectly amplifying the internal forces in the piers; and the long pulse duration allows for the full accumulation of nonlinear hysteretic energy dissipation and sectional curvature ductility, causing the structure to gradually reach a higher damage state over multiple cycles. This suggests that in the seismic design of bridges in near-field regions, attention should not be limited solely to the proximity between the pulse period

and the fundamental period; the amplification effects of the pulse amplitude and duration on the nonlinear structural response must also be fully considered.

In summary, at the E1 level, the high damping rubber bearing provides elastic restraint and initial stiffness, thereby ensuring the normal service function; at the E2 level, by virtue of its stable post-yield stiffness and high equivalent damping ratio, it concentrates the dissipation of seismic energy through substantial hysteretic deformation, effectively reducing the seismic forces transmitted to the pier and protecting the main structure. Near-field velocity pulse-like ground motions constitute the most unfavorable loading scenario governing the bearing displacement demand and energy dissipation burden. It is recommended that, based on the potential matching effect of the pulse period, a specific check of the bearing displacement capacity be conducted for long-period bridges such as long-span and high-pier structures.

To quantitatively evaluate the energy dissipation contribution of the HDRBs and the plastic energy dissipation level of the bridge piers under near-fault pulse-like ground motions, the most unfavorable E2 NLP longitudinal + vertical record was used as input, and the time histories of various energy components for the entire bridge were extracted from the nonlinear time-history analysis. The total seismic input energy E_I consists of the kinetic energy E_K , the structural damping energy dissipation E_D , the recoverable elastic strain energy E_S , and the irrecoverable hysteretic energy dissipation E_H . The hysteretic energy can be further decomposed into the bearing hysteretic energy $E_{H,bearing}$ and the pier plastic energy dissipation $E_{H,pier}$, among others.

Table 8 presents the cumulative energy distribution over the first 40 s. It can be clearly seen that the hysteretic energy dissipation of the HDRBs accounts for as much as 66.6%, serving as the absolute primary contributor to the dissipation of seismic input energy; the plastic energy dissipation of the piers accounts for only 8.6%; and the Rayleigh damping energy dissipation accounts for approximately 21.0%. This energy distribution pattern strongly confirms the effectiveness of the HDRB isolation system adopted in this study under rare earthquakes, as it concentrates the energy dissipation to protect the main bridge piers.

Table 8. Energy distribution of the entire bridge under E2 NLP longitudinal + vertical input.

Energy Component	Energy Value (kN·m)	Proportion of Total Input Energy (%)
Total seismic input energy E_I	42,500	100.0
Structural Rayleigh damping energy dissipation E_D	8920	21.0
HDRB hysteretic energy $E_{H,bearing}$	28,300	66.6
Pier plastic hysteretic energy $E_{H,pier}$	3650	8.6
Other energy dissipation (shear keys, pounding, etc.)	1630	3.8

4.6. Quantitative Analysis of the Influence of Vertical Seismic Components

To quantify the contribution of the vertical seismic component to the flexural–torsional response of the curved bridge, nonlinear time-history analyses were conducted for two load cases using the most unfavorable near-field long-pulse (NLP) record at the E2 level as input: Case A applied only horizontal ground motions (transverse or longitudinal), while Case B simultaneously applied horizontal and vertical ground motions (i.e., the three-component input method adopted in the original study, with the vertical and horizontal components scaled by the same factor to $PGA = 0.74\text{ g}$). The internal forces at the control sections at the pier base and the pier-top displacements were extracted for a typical continuous-beam pier P7 (pier height 38.5 m) and a rigid-frame pier P13 (pier height 44.0 m), and the percentage changes in internal forces after the inclusion of the vertical component were calculated. The results are listed in Table 9. The inclusion of the vertical seismic component has the

most significant influence on the torsional response of the curved bridge piers. For both the continuous-beam pier P7 and the rigid-frame pier P13, regardless of whether the ground motion is applied in the transverse or longitudinal direction, the increase in the pier-base torque reaches 25–35% or more. The increases in the transverse and longitudinal bending moments are relatively small, generally between 5% and 8%; the increase in pier-top displacement is of the same order of magnitude as that of the bending moments.

Table 9. Influence of vertical seismic components on key sectional internal forces (E2 NLP record).

Pier No.	Input Direction	Internal Force Type	Case A (Horizontal Only)	Case B (Horizontal + Vertical)	Change Ratio (%)
P7	Transverse	Pier-base torque T (kN·m)	1820	2450	+34.6
P7	Transverse	Pier-base transverse moment M_t (kN·m)	8450	9120	+7.9
P7	Longitudinal	Pier-base longitudinal moment M_l (kN·m)	5680	6020	+6.0
P7	Longitudinal	Pier-base torque T (kN·m)	1260	1580	+25.4
P13	Transverse	Pier-base torque T (kN·m)	3650	4800	+31.5
P13	Transverse	Pier-base transverse moment M_t (kN·m)	24,100	25,800	+7.1
P13	Longitudinal	Pier-base longitudinal moment M_l (kN·m)	26,500	27,800	+4.9
P13	Longitudinal	Pier-base torque T (kN·m)	2150	2840	+32.1

The above differences stem from the unique spatial flexural–torsional coupling mechanism of curved bridges: the vertical ground motion generates additional torques through the eccentricity between the shear center and the centroid of the main girder cross-section, while the changes in bearing axial forces induced by vertical inertial forces further alter the load transfer path of horizontal seismic forces. The pier-base bending moments are primarily controlled by horizontal inertial forces, and the vertical component only exerts a secondary influence through the P– Δ effect and axial force–bending moment coupling, resulting in limited increases. However, the significant increase in torque implies that the vertical seismic component cannot be neglected in the seismic design of curved bridges, and its effects on the torsional performance of bearings and the torsional capacity of piers should be specifically checked.

4.7. Analysis of Torsional Amplification for Curved Bridges

To further quantitatively evaluate the degree of torsional response of the curved bridge under earthquakes and its amplification effect relative to a straight bridge, the torsion-to-moment ratio (TMR) at the pier-base section is defined:

$$TMR_T = \frac{T_{\max}}{M_{t,\max}} \quad (6)$$

$$TMR_L = \frac{T_{\max}}{M_{l,\max}} \quad (7)$$

where $T_{\max}$ is the maximum torque at the pier base, and $M_{t,\max}$ and $M_{l,\max}$ are the maximum transverse and longitudinal bending moments, respectively. A larger TMR indicates a stronger accompanying torque under the same bending moment level, i.e., a more pronounced torsional effect.

Table 10 summarizes the TMR values of the typical continuous-beam pier P7 and rigid-frame pier P13 under five ground motions at the E2 level. Under near-field pulse-like records, especially the long-pulse NLP, the TMR generally reaches the highest values: for P7, the transverse TMR is 0.27 and the longitudinal TMR is 0.26; for P13, the transverse TMR is 0.19 and the longitudinal TMR is 0.10. Compared with the far-field FF case, NLP increases the transverse TMR of P7 by approximately 50% and the longitudinal TMR of P13 by approximately 67%. This reveals that the long-period velocity pulse not only proportionally

increases the bending moment and torque of the piers, but also disproportionately amplifies the torque demand by exciting the flexural–torsional spatial coupling of the curved bridge.

Table 10. Comparison of torsion-to-moment ratio (*TMR*) of typical piers (E2 level).

Ground Motion	P7 Transverse <i>TMR</i>	P7 Longitudinal <i>TMR</i>	P13 Transverse <i>TMR</i>	P13 Longitudinal <i>TMR</i>
NSP	0.22	0.21	0.15	0.08
NMP	0.24	0.23	0.17	0.09
NLP	0.27	0.26	0.19	0.10
NNP	0.18	0.17	0.12	0.06
FF	0.18	0.16	0.11	0.06

To separately quantify the contribution of the curved alignment to the torque, an idealized straight bridge comparison model was established (with the curve-induced flexural–torsional coupling removed, while all other parameters such as span lengths, pier heights, and bearing arrangements remained identical), and analyzed under the E2 NLP longitudinal + vertical input. The results show that the pier-base torque of P13 in the straight bridge is only 850 kN·m, whereas in the actual curved bridge this value is 2840 kN·m, yielding a torsional amplification factor of 3.34; the pier-base torque of P7 increases from approximately 480 kN·m to 1820 kN·m, with an amplification factor of about 3.79. This fully confirms that the flexural–torsional coupling effect of the curved bridge is the main source of pier-base torque, and this effect is further amplified under near-field long-pulse ground motions. Therefore, independent checks of the torsional bearing capacity for piers and bearings must be performed during design.

5. Seismic Damage Assessment

To evaluate the potential damage states of the bridge under E2-level seismic action, conventional methods require the introduction of plastic hinge elements into the finite element model to simulate the nonlinear hysteretic behavior and stiffness degradation of the components; however, modeling multiple piers of the full bridge would lead to excessively high computational costs and convergence difficulties. For this reason, a capacity-to-demand ratio (*CDR*) method at the section level is adopted in this study. By comparing the flexural capacity of critical sections with the seismic demand internal forces, the damage degree of the piers is rapidly identified. This method not only avoids the modeling burden of a large number of plastic hinge elements but also provides a quantitative basis for the identification of seismically vulnerable regions and for ductility design.

5.1. Sectional Flexural Capacity Analysis

The sectional flexural capacity of the bridge piers is calculated using the XTRACT fiber model software. Based on the plane section assumption, the software discretizes the cross-section in two directions into unconfined concrete, confined concrete, and longitudinal reinforcement fibers (Figures 19 and 20), assigns the corresponding uniaxial stress–strain constitutive relationships to each, and solves the complete moment–curvature curve by incrementally applying curvature; the nominal flexural strength and the moment limits corresponding to different damage states are then extracted.

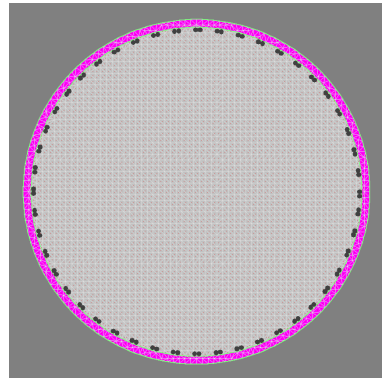


Figure 19. Fiber mesh discretization of the continuous girder pier section (diameter 2.4 m pier).

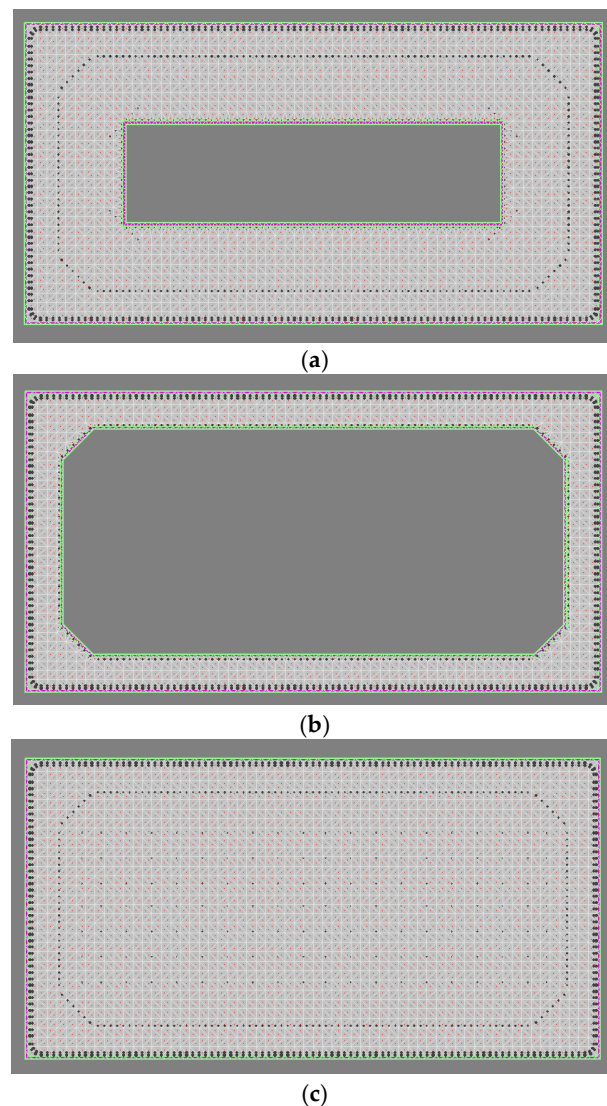


Figure 20. Fiber mesh discretization of the rigid-frame hollow thin-walled pier section (4.5 m (long.) $\times$ 7.6 m (trans.)): (a) section at upper edge of top hollow chamfer and lower edge of bottom hollow chamfer; (b) section at lower edge of top hollow chamfer, mid-pier section, and upper edge of bottom hollow chamfer; (c) solid section at pier base.

The piers of the continuous girder units are circular sections with a diameter of 2.4 m. The longitudinal reinforcing bars are uniformly arranged along the circumference, and the spiral stirrups provide effective confinement to the core concrete. The fiber mesh discretization is shown in Figure 19. The piers of the rigid-frame bridge

are variable-section hollow thin-walled piers, with a typical outer profile dimension of 4.5 m (longitudinal) $\times$ 7.6 m (transverse); the wall thickness and cross-sectional form vary along the pier height. To accurately reflect the capacity differences in the various controlling sections, fiber models are established separately for the top solid segment, the top transition segment, the mid-height hollow segment, the upper-middle transition segment, the bottom transition segment, and the bottom solid segment, as shown in Figure 20. The Mander confined concrete model is adopted to account for the enhancement effect of the stirrups on the peak stress and ultimate compressive strain; the reinforcing steel adopts a bilinear elastoplastic hardening model. Through the fiber section analysis, the moment–curvature relationships and the M–N interaction envelopes for each typical section under different axial force levels are obtained, laying the foundation for the subsequent capacity–demand comparison.

5.2. M–N Capacity-to-Demand Ratio at Critical Sections

Based on the internal force response envelopes of each pier under the five ground motions at the E2 level, as presented in Figures 12–16, and in combination with the sectional flexural capacities computed by XTRACT, the tallest pier of the continuous girder unit, P7 (pier height 38.5 m, 2.4 m diameter circular section), and the tallest pier of the rigid-frame bridge, P13 (pier height 44.0 m, 4.5 m $\times$ 7.6 m hollow thin-walled section), are taken as examples. The moment–axial force (M–N) capacity–demand comparisons at their most critical sections under transverse–vertical and longitudinal–vertical excitations are presented in Figure 21 and Figure 22, respectively. In the figures, the outer envelope represents the flexural capacity limit surface of the section under different axial force levels, and the interior data points represent the most unfavorable internal force combinations (M, N) induced by each ground motion. If an internal force point lies within the envelope, the sectional capacity meets the demand under that load case, and the component is in an elastic or slightly damaged state; if an internal force point exceeds the envelope, the section has entered the plastic stage and may suffer significant damage or failure.

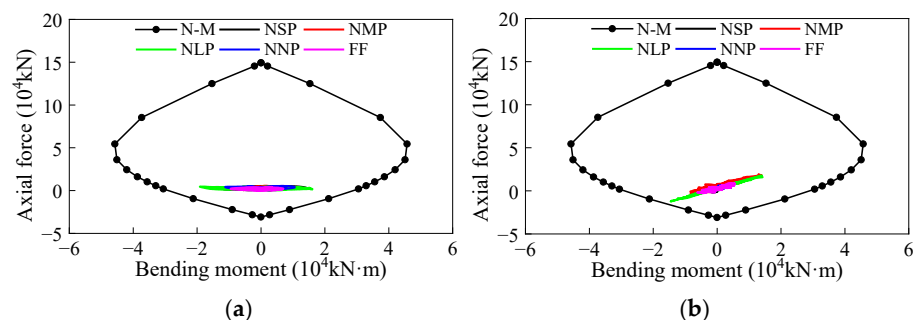


Figure 21. M–N capacity–demand comparison for the most critical section of pier P7 under E2 earthquakes: (a) transverse–vertical excitation; (b) longitudinal–vertical excitation.

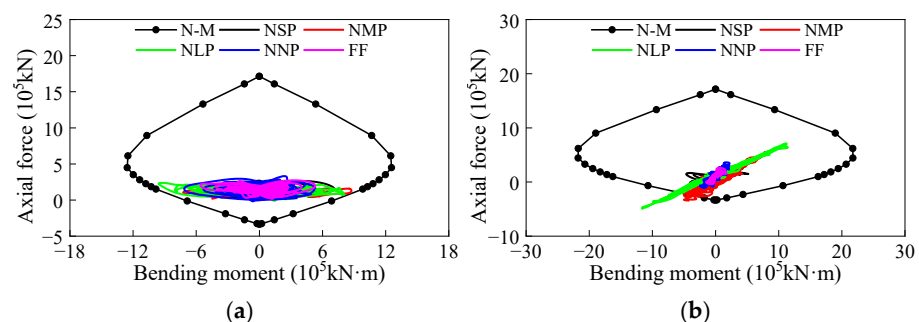


Figure 22. M–N capacity–demand comparison for the most critical section of pier P13 under E2 earthquakes: (a) transverse–vertical excitation; (b) longitudinal–vertical excitation.

Figure 21 shows that, under transverse–vertical excitation, the internal force points of all ground motions for the most critical section of P7 are located within the M–N envelope, indicating a relatively ample capacity reserve. Under longitudinal–vertical excitation, the internal force points of the near-field pulse-like ground motions (especially NLP) are close to the envelope boundary, indicating that this pier faces a high longitudinal flexural demand under long-pulse near-fault ground motions. However, due to the energy dissipation and seismic mitigation effect of the high damping rubber bearings, the section still remains within the safe range, which is consistent with the S-shaped higher-order distribution characteristic of the bending moment along the pier shaft observed in Figure 15.

In Figure 22, the internal force state of the most critical section of P13 is markedly more severe. Under transverse–vertical excitation, some internal force points of the NLP and NMP cases have already touched or even slightly exceeded the envelope, indicating that limited yielding may occur at the pier base or in the transition section regions. Under longitudinal–vertical excitation, the internal force points of the NLP case significantly exceed the envelope, demonstrating that the main pier of the rigid frame has entered a notable plastic state under the long-period pulse resonance effect, and the flexural capacity of the pier base section is insufficient. This result corroborates the pier-base bending moment demand of P13 described in Section 4.4, reflecting the force characteristic whereby the pier–girder rigid connection leads to the transmission of enormous longitudinal inertial forces.

5.3. Capacity-to-Demand Ratio Evolution and Damage State Assessment

To quantify the damage evolution process of each pier under seismic action, based on incremental dynamic analysis (IDA), the peak ground acceleration (PGA) was progressively scaled from 0.1 g to 1.0 g, covering the E1 level (PGA = 0.37 g) and the E2 level (PGA = 0.74 g). The ratio of the moment demand at the most critical section of each pier to the sectional flexural capacity was extracted, and the capacity-to-demand ratio is defined as:

$$CDR = \frac{M_u}{M_d} \quad (8)$$

where M_u is the effective flexural capacity of the section corresponding to the current axial force level, and M_d is the sectional moment demand under seismic action. $CDR > 1.0$ indicates a capacity surplus, while $CDR < 1.0$ indicates insufficient capacity, meaning that the section has yielded or sustained damage.

Figure 23 presents the CDR evolution curves with increasing PGA under transverse–vertical (T–V GM) and longitudinal–vertical (L–V GM) excitations for three representative piers of the continuous girder unit (P7, P8, P10) and three representative piers of the rigid-frame bridge (P13, P14, P12). The CDR of all types of piers decreases monotonically with increasing PGA, but significant differences exist in the rate of decline and the final degree of damage. Among the continuous girder piers, the CDR of the tallest pier, P7, decreases most rapidly; at the E2 level, the longitudinal–vertical CDR drops to 1.02, the safety margin is essentially exhausted, and the pier is on the verge of slight damage. The medium-height pier P8 follows, while the shortest pier P10, due to multiple favorable factors including high stiffness, a short moment arm, and the installation of a large-capacity HDRB, still maintains a CDR greater than 1.55 at the E2 level, with extremely minor damage. The CDR curves of the rigid-frame bridge piers are generally steeper. For the tallest pier P13 under longitudinal–vertical excitation, the CDR already drops to 1.30 at the E1 level and further decreases to 0.82 at the E2 level, indicating that the pier-base section has entered a moderate yielding damage state and constitutes the most seismically vulnerable link of the entire bridge. The longitudinal–vertical CDR of P14 at the E2 level

is approximately 1.05, representing slight yielding, while P12 consistently maintains a relatively large safety margin.

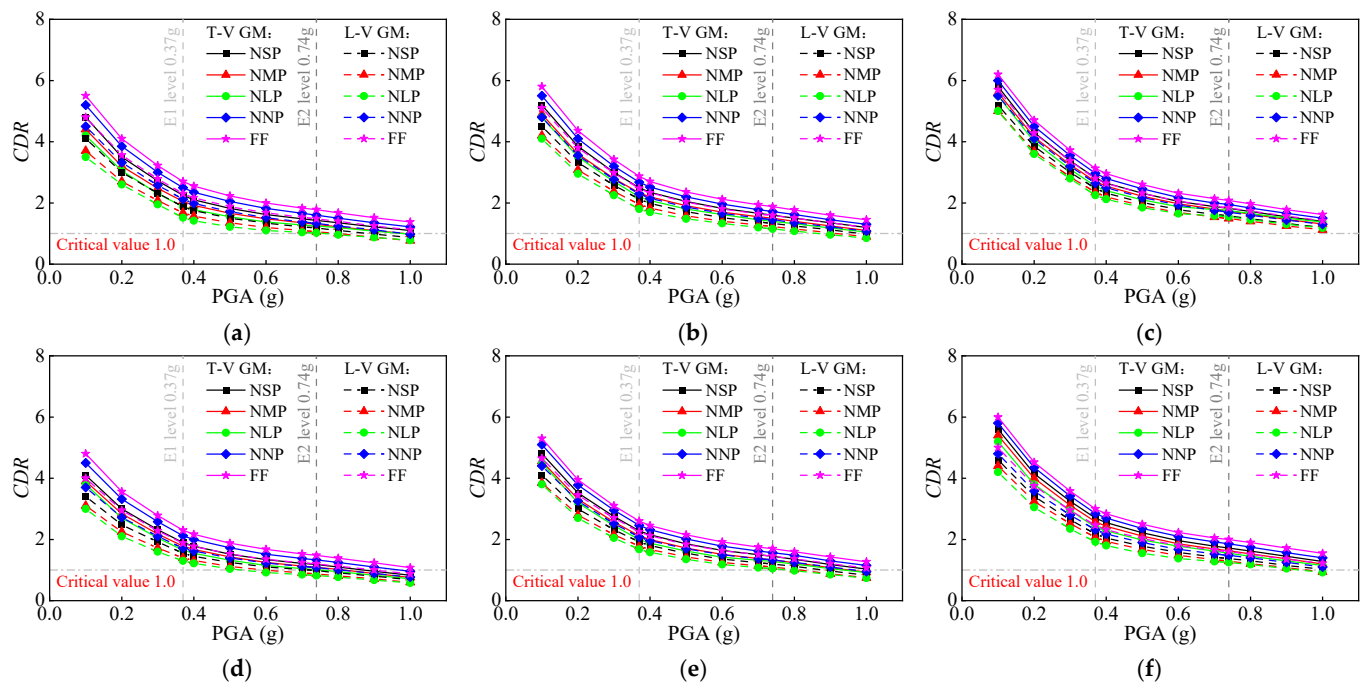


Figure 23. Capacity-to-demand ratio (*CDR*) curves of typical piers: (a) P7; (b) P8; (c) P10; (d) P13; (e) P14; (f) P12.

Comparing the influence of the five ground motions, the NLP long-pulse near-field ground motion causes the most drastic *CDR* reduction, followed by NMP, while NSP causes a relatively smaller reduction. The far-field FF and near-field non-pulse NNP records result in the least damage. Taking the longitudinal–vertical *CDR* of P13 as an example, at the E2 level the *CDR* values under NLP, NMP, NSP, NNP, and FF are 0.82, 0.87, 0.98, 1.08, and 1.20, respectively; the pulse period length has a decisive influence on damage. In addition, the *CDR* values under longitudinal–vertical excitation are generally lower than those under transverse–vertical excitation, indicating that longitudinal bending is the dominant direction controlling pier damage for this type of bridge.

This study selected five ground motion records to cover typical spectral characteristics, including near-field short-, medium-, and long-pulse, near-field no-pulse, and far-field records, with the objective of comparing physical mechanisms rather than performing probabilistic statistics. To assess the sensitivity of the results to the selected records, the range of *CDR* values for typical piers at the E2 level under each record was statistically analyzed based on the completed incremental dynamic analysis (IDA). The *CDR* of pier P13 varies between 0.82 and 1.20 (with a coefficient of variation of 14.1%); however, for all response indices, the long-pulse NLP record consistently yields the most unfavorable results, while the far-field FF record produces the least severe effects, and a clear monotonic relationship exists between the pulse period and the damage level, indicating that the conclusions are not an artifact of individual records. Given the limited sample size, probabilistic fragility derivation is not performed herein [40]; when the results are to be used for quantitative design, it is recommended that the number of records be increased to at least 11, as required by the relevant codes.

5.4. Comprehensive Evaluation of Damage States

Based on the M - N capacity-to-demand comparisons and the CDR evolution curves, the following assessment is made regarding the pier damage states of this curved continuous rigid-frame composite girder bridge under the E2 seismic level:

(1) The damage to the piers of the continuous girder units is generally slight. The longitudinal CDR of the tallest pier, P7, is close to 1.0, indicating that it is on the verge of slight damage. It is recommended that its longitudinal reinforcement ratio be appropriately increased or that displacement-limiting devices be added at the pier-top bearings to control displacements. The sectional capacities of P8 and P10 are ample and satisfy the seismic performance objectives.

(2) The damage to the rigid-frame bridge piers is notably more severe than that of the continuous girder units. P13 and P14 have undergone yielding to varying degrees under longitudinal–vertical excitation, among which the plastic demand at the pier-base section and the upper-middle transition section region of P13 is the most prominent. In the design, ductility detailing enhancements should be implemented for critical regions such as the interface between the bottom solid segment and the lower transition segment of P13, and the upper-middle transition segment, for example, by densifying confining stirrups, adding core columns or structural steel, and verifying the rotational capacity of the plastic hinge zones. Meanwhile, since the rigid-frame piers cannot dissipate energy through bearings, it is recommended that, at the scheme stage, consideration be given to introducing seismic isolation and energy dissipation bearings or supplementary energy dissipation devices at the side piers of the rigid frame to reduce the internal force demand on the fixed piers.

(3) Near-field pulse-like ground motions (especially the long-pulse NLP) constitute the most unfavorable loading scenario controlling pier damage. The CDR reduction caused by such motions can be two to three times that caused by far-field ground motions. For bridges located adjacent to active faults, full attention should be paid to the significant effect of velocity pulses. In the seismic design, ground motion inputs that account for the near-fault effect should be used, and specific damage assessments should be conducted for the critical pier.

6. Results and Discussion

6.1. General Discussion of Key Findings

This study systematically investigated the seismic performance of a curved continuous rigid-frame composite bridge under near-fault pulse-like ground motions. Several salient findings emerge from the analyses:

(1) Dominant effect of near-fault velocity pulses. Among the five ground motion records considered, the long-pulse record (NLP, $T_p = 12.0$ s) consistently produced the most unfavorable pier displacements, internal forces, and damage states, even though its pulse period far exceeds the fundamental periods of the bridge (1–3 s). This counterintuitive result is explained by the broadband spectral energy of the velocity pulse, the large-amplitude bearing deformations induced by the high PGV (0.54 m/s), and the cumulative damage effect under prolonged pulse duration.

(2) Effectiveness of HDRB isolation. Under the E2 level, HDRBs dissipated approximately 66.6% of the total seismic input energy through stable hysteretic behavior, while pier plastic energy dissipation accounted for only 8.6%. This energy distribution pattern confirms the intended seismic isolation function of the bearings, effectively protecting the piers from severe damage. However, bearing shear deformations reached up to 62.8 mm under NLP, imposing high deformation demands on expansion joints and unseating prevention devices.

(3) Vertical ground motion effects. The vertical seismic component amplified pier-base torsional moments by 25–35% and bending moments by 5–8%, primarily through spatial coupling between vertical bending and horizontal torsion of the curved deck. This highlights the necessity of including vertical excitation in the seismic analysis of curved bridges, particularly for near-fault sites with high vertical-to-horizontal spectral ratios.

(4) Higher-mode effects on pier internal force distributions. Modal separation analysis confirmed that the S-shaped bending moment distribution observed in the rigid-frame hollow thin-walled piers arises from the superposition of the fundamental longitudinal mode and the second anti-symmetric longitudinal mode, with abrupt section changes at wall thickness transitions further intensifying the local internal force concentration. Similar higher-mode effects were observed in the continuous girder piers under longitudinal excitation.

6.2. Research Limitations

Several limitations of the present study should be acknowledged:

(1) Limited number of ground motion records. Although the five records were carefully selected to cover distinct spectral and pulse characteristics, this sample size is insufficient for probabilistic seismic fragility analysis. The findings are physically robust for the considered scenarios but may not capture the full variability of near-fault ground motions. Extension to a larger record set (e.g., 11–20 pulse-like records) is recommended for fragility-based design applications.

(2) Rate-independent HDRB model. The equivalent bilinear model does not account for the rate-dependent viscoelastic behavior of high damping rubber. Under near-fault pulse-like motions with rapid loading rate changes, this simplification may introduce deviations in the predicted bearing force and hysteresis loop shape. Comparative studies using rate-dependent models (e.g., fractional derivative or modified Maxwell models) are warranted to quantify these effects.

(3) Lack of experimental validation. The finite element model was validated indirectly through modal property checks, component-level parameter calibration against published test data, and parametric sensitivity analyses. Direct validation against large-scale shake table tests or field monitoring data of similar curved bridges would further enhance model confidence.

(4) Advanced constitutive models for RC thin-walled piers that incorporate axial–flexure–shear interaction [41,42] could be integrated in future refinements of the pier fiber sections.

6.3. Future Research Directions

Based on the findings and limitations of this study, the following research directions are suggested:

(1) Probabilistic seismic fragility analysis of curved continuous rigid-frame composite bridges using a statistically sufficient ensemble of near-fault pulse-like records, accounting for uncertainties in ground motion, material properties, and modeling parameters.

(2) Comparative studies employing rate-dependent HDRB constitutive models to quantify the influence of viscoelastic effects on the seismic response of isolated curved bridges.

(3) Multi-objective optimization of seismic isolation parameters and ductility detailing for curved bridges, considering the competing demands of bearing displacement control, pier damage mitigation, and unseating prevention.

(4) Field monitoring and system identification of instrumented curved bridges to validate and refine the numerical modeling assumptions adopted in this study.

7. Conclusions

This paper presents a comprehensive numerical investigation of the seismic performance and damage evolution of a curved continuous rigid-frame composite bridge under near-fault pulse-like ground motions. The main conclusions are summarized as follows:

(1) The dynamic characteristics of this curved composite bridge are complex, with a long fundamental period (3.29 s) and densely distributed modes. The rigid-frame subsystem exhibits significantly higher stiffness due to pier–girder rigid connections. Modal analysis reveals a torsional irregularity coefficient of 0.82, indicating pronounced torsional sensitivity. (2) Under the E1 earthquake level, the structure remains essentially elastic with limited bearing energy dissipation. Under E2, HDRBs dissipate approximately 66.6% of the total seismic input energy, effectively protecting the piers. However, bearing deformations impose high demands on expansion joints and unseating prevention devices, although all remain within design allowable limits.

(3) Near-fault long-pulse records ($T_p = 12.0$ s) govern the structural response, producing the most unfavorable displacements, internal forces, and damage states due to broadband spectral energy, high PGV-induced bearing deformations, and cumulative damage under prolonged pulse duration. The vertical seismic component amplifies pier torsional moments by 25–35%.

(4) Continuous girder piers exhibit a transverse frame effect and a longitudinal S-shaped bending moment distribution due to higher-mode contributions, confirmed by modal separation analysis. Rigid-frame hollow thin-walled piers display S-shaped internal force distributions associated with abrupt wall thickness changes, with the tallest pier reaching a CDR of 0.82 (moderate damage).

(5) The capacity-to-demand ratio analysis indicates that damage to rigid-frame piers is more severe than that of continuous girder piers, particularly under near-field long-pulse earthquakes. Ductility detailing enhancements are recommended for the upper-middle portions and wall thickness transition zones of rigid-frame piers, where higher-mode effects and abrupt stiffness changes concentrate seismic demands.

The conclusions drawn in this paper should be interpreted in light of the following limitations. The analyses are based on five ground motion records; while these records cover distinct spectral and pulse characteristics essential for the mechanistic investigation, the sample size is insufficient for probabilistic fragility analysis. And the equivalent bilinear model adopted for HDRBs does not capture rate-dependent viscoelastic effects, which may influence the bearing response under near-fault pulses with rapid loading rate variations. These limitations should be considered when extrapolating the findings to other bridge configurations or site conditions.

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References

1. Liu, W.; Luo, Q.; Dai, G.; Tang, X. Experimental and numerical study on dynamic response of high-pier ballastless continuous beam bridge in mountainous area. *Appl. Sci.* **2025**, *15*, 4341. [\[CrossRef\]](#)
2. Liu, J.; Xiong, G.; Xu, T.; Chen, F.; Xiong, Y.; Huang, F. Intelligent FE modelling and optimization of steel-concrete composite continuous beam bridge. *Eng. Struct.* **2025**, *337*, 120568. [\[CrossRef\]](#)
3. Bai, H.; Zhang, Y.; You, B.; Chen, K. Comprehensive design optimization framework for prestressed concrete continuous beam bridge using genetic algorithm and backpropagation neural network. *Buildings* **2025**, *15*, 1344. [\[CrossRef\]](#)
4. Zhao, T.L.; Li, X.M.; Du, Y.F. Nonstationary elastoplastic response analysis of curved beam bridges under spatial variability of earthquake ground motion using absolute displacement method. *Soil Dyn. Earthq. Eng.* **2024**, *181*, 108626. [\[CrossRef\]](#)
5. Li, X.M.; Fan, P.H.; Li, W.R.; Du, Y.F. Study on the dynamic response and system reliability of curved beam bridge under multi-dimensional non-uniform random earthquake excitation. *Soil Dyn. Earthq. Eng.* **2026**, *201*, 109990. [\[CrossRef\]](#)
6. Deng, T.; Huang, J.; Cao, M.; Li, D.; Bayat, M. Seismic damage identification method for curved beam bridges based on wavelet packet norm entropy. *Sensors* **2022**, *22*, 239. [\[PubMed\]](#)
7. Qi, Q.M.; Zhou, H.B.; Yang, H.P.; Chen, X.; Chen, L.; Shao, C.; Cui, H. Seismic responses of an FPB-isolated railway continuous beam bridge under pulse-type motions: Shaking table tests and simulations. *Soil Dyn. Earthq. Eng.* **2026**, *209*, 110506. [\[CrossRef\]](#)
8. Wu, Y.; Liu, A.; Wang, N.; Lu, C. Near-fault ground motion characteristics from strong-motion records of the 2023 Kahramanmaraş MW7.8 earthquake. *J. Asian Earth Sci.* **2026**, *308*, 107109. [\[CrossRef\]](#)
9. Jiang, Y.; Xie, Z.; Zhang, L.; Li, Y.; Lyu, Y. Study on spatial variation characteristics of seismic motion in near-fault zones based on ground motion simulation and probabilistic seismic hazard analysis: A case study of the Nankou-Sunhe Fault. *Soil Dyn. Earthq. Eng.* **2026**, *206*, 110274. [\[CrossRef\]](#)
10. Chen, J.; Sun, B.; Deng, M.; Zhang, S.; Xu, J.; Wang, C.; Tan, H. Seismic performance assessments of ACCWDs considering the multidimensional dependent random parameters of near-fault pulse-like ground motions. *Soil Dyn. Earthq. Eng.* **2026**, *207*, 110336. [\[CrossRef\]](#)
11. Yang, C.; Zhang, M.; Wang, Y.; Lyu, P.; Chao, C.; Chen, Y.; An, W.; Wu, L.; Li, L. Research on vertical separation-collision effects and pier failure mechanism of multi-span continuous beam bridges. *Buildings* **2026**, *16*, 1863. [\[CrossRef\]](#)
12. Xiao, B.; Wei, B.; Huang, Z.; Zhang, R.; Yang, Z. Analysis of seismic damage characteristics of typical continuous beam bridge and CRTS III track structure under near-fault earthquakes. *Structures* **2026**, *88*, 111820. [\[CrossRef\]](#)
13. Gallardo, J.A.; Chen, M.C.; Restrepo, J.I.; de la Llera, J.C. Modeling the multiaxial behavior of high damping rubber bearings (HDRBs). *Earthq. Eng. Struct. Dyn.* **2025**, *54*, 3202–3223. [\[CrossRef\]](#)
14. Gallardo, J.A.; de la Llera, J.C. Epistemic uncertainty in the seismic performance assessment of isolated buildings due to high damping rubber bearing model selection. *Eng. Struct.* **2026**, *348*, 121734. [\[CrossRef\]](#)
15. Wahab, A.G.; Tao, Z.; Li, H.; Rasa, A.Y.; Huma, T.; Liang, Y. Dynamic simulation and seismic analysis of hillside RC buildings isolated by high-damping rubber bearings. *Infrastructures* **2025**, *10*, 239. [\[CrossRef\]](#)
16. Cui, H.M.; Shao, C.J.; Wei, W.; Zhang, Y.; Qi, Q.; Lang, J. Prediction of equivalent damping ratio for round-ended hollow railway bridge piers with variable sections based on GA-BP neural network and quasi-static test. *Mech. Syst. Signal Process.* **2026**, *244*, 113810. [\[CrossRef\]](#)
17. Cui, H.M.; Shao, C.J.; Wei, W.; Zhang, Y.; Qi, Q.; Wang, C.; Long, J. Failure mechanisms and seismic damage control of variable-section round-ended hollow railway piers in the transverse direction. *Eng. Fail. Anal.* **2026**, *193*, 110883. [\[CrossRef\]](#)
18. Cui, H.M.; Shao, C.J.; Wei, W.; Zhang, Y.; Qi, Q.; Wang, C. Data-driven prediction of effective stiffness for thin-walled hollow piers via a GA-BP neural network with quasi-static test validation. *Adv. Eng. Softw.* **2026**, *218*, 104194. [\[CrossRef\]](#)
19. Cui, H.M.; Shao, C.J. Experimental study on seismic performance of circular hollow piers with low-to-medium shear-span ratio in high-seismic intensity regions. *Structures* **2026**, *87*, 111767. [\[CrossRef\]](#)
20. Cui, H.M.; Shao, C.J. Experimental investigation on the flexure-shear coupled deformation performance of circular hollow piers in highway bridges. *Structures* **2026**, *88*, 111799. [\[CrossRef\]](#)
21. Cui, H.M.; Shao, C.J.; Wei, W.; Qi, Q.; Zhang, Y.; Wang, C. A flexure-shear critical failure-based equivalent damping ratio model for variable-section round-ended hollow piers in railway bridges. *Structures* **2026**, *89*, 112178. [\[CrossRef\]](#)
22. Cui, H.M.; Shao, C.J.; Wei, W.; Yi, X.; Qi, Q.; Zhang, Y. Displacement-based seismic performance tests and equivalent linearization modeling of variable-section round-ended hollow railway piers. *Structures* **2026**, *90*, 112376. [\[CrossRef\]](#)
23. Cui, H.M.; Shao, C.J.; Wei, W.; Qi, Q.; Zhang, Y. Seismic performance assessment and damage-based quantification of RC thin-walled hollow piers under flexural-shear coupling. *Structures* **2026**, *90*, 112468. [\[CrossRef\]](#)
24. Chen, X.H.; Liao, B.S.; Islam, M.N. Seismic response analysis of high-pier large-span continuous rigid frame bridges considering various terrain combinations. *J. Earthq. Eng.* **2026**, *30*, 1000–1037.
25. Li, J.F.; Jin, M.M.; Chen, J.B.; Wang, G.J.; Wang, Y.F.; Yan, H.H.; Weng, Z.C.; Huang, Z.G.; Ai, H.; Wu, L. Seismic vulnerability analysis of continuous beam bridge based on finite element method. *Environ. Dev. Sustain.* **2026**, *28*, 3613–3627.

26. Chen, Q.; Zhang, Y.B.; Yu, Q.S.; Li, D.J.; Chen, H.P.; Wang, Z. Dynamic response and failure mechanism of slopes subject to near-fault pulse-like ground motions. *J. Earthq. Eng.* 2026, *early access*. [[CrossRef](#)]
27. Zeng, X.Z.; Deng, K.L.; Wang, Y.C.; Yan, G.H.; Zhao, C.H. Field investigation and numerical analysis of damage to a high-pier long-span continuous rigid frame bridge in the 2008 Wenchuan earthquake. *J. Earthq. Eng.* **2022**, *26*, 5204–5220.
28. Lin, W.; Zhang, Y.; Wang, C. Structural health monitoring and damage identification of continuous rigid frame bridges based on advanced sensor networks. *Sensors* **2025**, *25*, 7141.
29. Huang, W.F.; Zhan, Y.L.; Zhang, C.; Lyu, W.; Li, Y.; Shao, J. Tensile resistance and shear-tension interaction relationship of T-type perfobond rid shear connectors. *J. Build. Eng.* **2024**, *89*, 109183. [[CrossRef](#)]
30. Huang, W.F.; Zhan, Y.L.; Li, J.Y.; Wu, H.; Xu, J.; Shao, J. Effect of flange widths of T-type PBL shear connectors on the mechanical performance in negative bending moment region of steel-concrete composite beams. *Structures* **2024**, *66*, 106846. [[CrossRef](#)]
31. Huang, W.F.; Zhan, Y.L.; Lyu, W.; Zhang, H.; Jiang, H.; Shao, J. Cracking performance in the negative moment zone of steel-concrete composite beams connected by T-shaped PBL shear connectors. *Steel Compos. Struct.* **2026**, *58*, 437–457.
32. Silva, P.F.; Megally, S.; Seible, F. Seismic Performance of Sacrificial Exterior Shear Keys in Bridge Abutments. *Earthq. Spectra* **2009**, *25*, 643–664. [[CrossRef](#)]
33. Bozorgzadeh, A.; Megally, S.; Restrepo, J.I.; Ashford, S.A. Capacity Evaluation of Exterior Sacrificial Shear Keys of Bridge Abutments. *J. Bridge Eng.* **2006**, *11*, 555–565. [[CrossRef](#)]
34. Jankowski, R. Non-linear viscoelastic modelling of earthquake-induced structural pounding. *Earthq. Eng. Struct. Dyn.* **2005**, *34*, 595–611. [[CrossRef](#)]
35. Muthukumar, S.; DesRoches, R. A Hertz contact model with non-linear damping for pounding simulation. *Earthq. Eng. Struct. Dyn.* **2006**, *35*, 811–828. [[CrossRef](#)]
36. Markou, A.; Manolis, G. Mechanical models for shear behavior in high damping rubber bearings. *Soil Dyn. Earthq. Eng.* **2016**, *90*, 221–226. [[CrossRef](#)]
37. GB 17741-2005; Evaluation of Seismic Safety for Engineering Sites. General Administration of Quality Supervision, Inspection and Quarantine of the People's Republic of China, Standardization Administration of the People's Republic of China: Beijing, China, 2005.
38. JTG/T 2231-01-2020; Specifications for Seismic Design of Highway Bridges. Ministry of Transport of the People's Republic of China: Beijing, China, 2020.
39. Baker, J. Quantitative classification of near-fault ground motions using wavelet analysis. *Bull. Seismol. Soc. Am.* **2007**, *97*, 1486–1501. [[CrossRef](#)]
40. Duan, L.X.; Yang, H.P.; Qi, Q.M.; Wu, Q.; Shao, C.; Jiang, L. Seismic fragility of urban rail transport RC solid piers considering multiparameter effects. *Buildings* **2026**, *16*, 2327. [[CrossRef](#)]
41. Qi, Q.M.; Shao, C.J.; Yang, H.P.; Cui, H.; Chen, Z.; Gong, W. Axial-flexure-shear model for seismic analysis of RC thin-walled hollow piers. *Soil Dyn. Earthq. Eng.* **2025**, *195*, 109375. [[CrossRef](#)]
42. Qi, Q.M.; Jiang, L.F.; Shao, C.J.; Yang, H.; Shao, J.; Zhou, Y.; Cui, H. Hysteretic behavior modeling of RC walls considering axial-flexure-shear interaction. *J. Build. Eng.* **2025**, *111*, 113628. [[CrossRef](#)]

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