

Article

Seismic Performance and Reinforcement Ratio Analysis of Thin-Walled Piers of Continuous Girder Bridges in High-Seismic-Intensity Regions

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Abstract

Existing studies on bridge seismic behavior mainly address small- and medium-span bridges with circular column piers, with limited applicability to long-span, thin-walled high piers in mountainous regions. This study investigated the seismic performance and rational reinforcement ratio of thin-walled piers in high-seismic-intensity regions. A beam-element finite element model of a three-span prestressed concrete continuous girder bridge was established. Three pier heights (20 m, 50 m, and 80 m) were considered, and response spectrum analyses were conducted under peak ground accelerations of 0.1 g to 0.3 g. Results indicated that tall piers exhibit pronounced flexibility. The fundamental period increases from 1.02 s to 3.38 s with increasing pier height. For an 80 m pier, the first ten vibration modes are predominantly local bending or torsion, each with mass participation below 1%, requiring many modes for computational accuracy. Under seismic loading, the design of short piers is governed by internal forces, while that of tall piers is controlled by pier-top displacements. The reinforcement ratio of short piers is dictated by seismic demands, and shear requirements are difficult to satisfy solely by increasing transverse reinforcement. For tall piers, the longitudinal reinforcement ratio may follow minimum detailing, while the transverse reinforcement ratio increases significantly with higher seismic intensity. These findings provide a reference for seismic design and reinforcement optimization of long-span, thin-walled pier bridges in high seismic intensity regions.

Keywords: thin-walled pier; seismic performance; reinforcement ratio; response spectrum analysis

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1. Introduction

China is characterized by high seismicity, with frequent and intense earthquakes distributed over a wide area [1,2]. The country accounts for approximately one-third of continental earthquakes of magnitude ≥ 7.0 globally, and most major cities are located in regions of intensity VII or higher [3,4]. As a critical component of transportation lifelines, bridge failure during an earthquake not only incurs direct economic losses but also cripples rescue access. In the 2008 Wenchuan earthquake, dozens of bridge failures isolated

numerous townships, turning them into “islands” and severely hampering disaster relief. In high-seismic-intensity zones, bridge seismic design faces a persistent trade-off between economic feasibility and seismic performance. Conventional bridge strengthening practices, which rely heavily on empirical values, frequently result in excessively high longitudinal reinforcement ratios in piles and columns. This not only increases material costs and carbon emissions (each ton of steel emits ca. 1.85 t of CO₂) but may also shift the failure mode of piers from ductile flexural damage to brittle shear failure [5–7]. Therefore, investigating the seismic damage mechanisms of piers and abutments and optimizing reinforcement ratios in high-seismic-intensity regions are of significant practical importance for reducing life-cycle costs and enabling economical, green engineering.

Current research on bridge design in high-seismic-intensity regions has primarily focused on seismic response analysis. Through methods such as response spectrum analysis, nonlinear time-history analysis, incremental dynamic analysis (IDA), and pseudo-static testing [8–12], scholars have established quantitative mapping relationships among ground motion input and pier internal forces, pier-top displacements, and reinforcement ratios [13,14]. Elucidating how these quantitative relationships vary with pier height is key to achieving performance-based seismic design and reinforcement optimization [15]. At the core of this endeavor lies the translation of earthquake damage observations into practical design techniques [16–18], a process that integrates seismic response studies with damage investigation and countermeasure development. Gao et al. evaluated damage probabilities of piers and bearings in continuous girder bridges under frequent [19], rare, and very rare earthquakes using IDA-based fragility analysis. They found that in high-intensity zones, bearing damage risk often exceeds that of piers, highlighting the need for careful bearing selection and verification. Wang et al. performed longitudinal and transverse analyses under E1 and E2 seismic levels [20], specifying pier and pile dimensions, reinforcement amounts, and seismic isolation bearing parameters for varying pier heights, and proposed key design principles for high-intensity zones. Miao et al. focused on short-to medium-span reinforced concrete bridges [21]. Using pseudo-static tests and finite element analysis, they found that increasing the longitudinal reinforcement ratio from 1% to 2% increased the ultimate curvature by only 15% but improved energy dissipation capacity by over 40%. However, beyond 2.5%, the ductility gain plateaued, confirming the existence of an economical reinforcement ratio. The findings offer a theoretical foundation for performance-based seismic design and retrofitting strategies, and pave the way for future investigations into near-fault pulse effects, lateral spreading of pile foundations in liquefiable soils, and cost-effective reinforcement optimization approaches based on machine learning.

Foreign researchers have also extensively investigated bridge seismic behavior, particularly the response of special bridges to vertical ground motions and the reinforcement and ductility design of conventional piers. Wilson, Kunnath, and colleagues [22] numerically studied the effects of vertical ground motions on a three-span curved girder bridge and a two-span single-deck viaduct, demonstrating that vertical motions significantly affect pier axial forces. Mander et al. conducted theoretical comparative studies on the influence of confined concrete [23], longitudinal reinforcement ratio, and transverse reinforcement on the ductility of conventional reinforced concrete bridge piers, followed by detailed analyses of strength and ductility design for circular-section piers. Lu et al. used pseudo-static tests combined with numerical simulations to investigate the effect of locally increased reinforcement in the plastic hinge region on pier seismic performance [24]. They found that failure in such piers concentrated at the base without shifting to unintended weak zones; the hysteresis loops became fuller, and both load-carrying capacity and energy dissipation increased markedly. Moreover, when the additional reinforcement in the plastic hinge region was less than 65% of the full-section uniform increase, the seismic

performance was comparable to that of fully uniformly reinforced piers. Bouazza et al. employed fiber-based beam–column elements to model circular piers under identical axial load ratios [25], flexural stiffness, and material properties. Using an advanced hysteresis steel model that accounts for inelastic buckling of rebars, they determined effective buckling lengths analytically and provided design guidance for ductile reinforced concrete bridge piers. Di Mucci et al. proposed a computer vision-based assessment framework that employs a convolutional neural network to identify the corrosion levels of circular reinforced concrete piers from surface images [26]. This framework was further coupled with a probabilistic seismic analysis model for rapid prediction of seismic fragility in corroded reinforced concrete simply supported girder bridges. This study thus offers a novel technical pathway for prioritizing maintenance and informing retrofitting decisions for aging bridges in high-seismic-intensity zones.

Domestic and international scholars have developed a variety of research methods and approaches for bridge seismic behavior; however, the primary focus of existing studies remains on conventional small- and medium-span bridges with circular column piers [27–29]. As the construction focus shifts toward improving the road network in mountainous regions [15], the thin-walled piers have gained prominence. Their relatively small longitudinal thickness reduces material consumption and self-weight [30], making them increasingly widespread in long-span overpass bridges. This trend poses serious challenges to both the applicability of existing research and the refinement of code-based design guidance [31–33]. Regrettably, current seismic design codes only specify minimum seismic stirrup ratios in potential plastic hinge regions and a longitudinal reinforcement ratio range of 0.6–4% for ductile piers [34]. They do not differentiate values for different pier heights or seismic intensity levels, nor do they provide any specific provisions for thin-walled piers. Consequently, designers often rely on empirical rules of thumb without clear justifications, making it difficult to achieve accurate design in practice. Accordingly, there is substantial engineering value in investigating how to balance seismic safety and reinforcement economy for long-span thin-walled pier bridges.

Regarding the aforementioned objectives, this study conducts a numerical simulation analysis on a three-span prestressed concrete continuous girder bridge. First, a finite element model is established using Midas Civil (v2024), and the ground motion parameters and design response spectrum are selected to perform dynamic characteristic analysis and seismic response analysis. Subsequently, a sensitivity analysis of the longitudinal reinforcement ratio and stirrup ratio on seismic performance is carried out. Based on the results, recommendations for seismic design reinforcement ratios and minimum reinforcement ratio limits are proposed based on pier height classification. The findings are intended to serve as a reference for the seismic design and code revision of long-span continuous girder bridges with thin-walled piers in high-seismic-intensity regions.

2. Midas-Based Finite Element Model of a Prestressed Concrete Continuous Girder Bridge and Research Framework

2.1. Project Overview

This study addresses the seismic design of a three-span prestressed concrete continuous girder bridge currently under development in coastal Fujian, China. The bridge, which carries a first-class highway over an existing expressway, is situated in a seismic intensity VII zone with high fortification demands.

The bridge has a span arrangement of 35 m + 65 m + 35 m and a total deck width of 26.0 m. The superstructure consists of a three-span prestressed concrete continuous box girder with variable cross-section. The girder was constructed using the cantilever casting method, with a total of 41 casting segments: 32 cantilever segments (Nos. 1 to 8), three

closure segments, and six falsework-cast segments. The main design parameters of the bridge are listed in Figures 1–3.

The box girder has a single-box four-cell cross-section. The top slab is 26.0 m wide, the bottom slab is 21.0 m wide, and each cantilever wing is 2.5 m wide. Longitudinally, within the 4 m length of the 0-block over the pier, the bottom of the girder is flat with a constant depth of 4.295 m. From segment 1 to segment 8, the girder depth follows a 1.8-power parabola, the curve of the bottom flange lower surface of the box girder varies parabolically as $H = (1.6/29.5^{1.8})x^{1.8} + 2.695$ m, and the curve of the bottom flange upper surface varies parabolically as $H = (1.17/28.5^{1.8})x^{1.8} + 2.445$ m, varying from 4.295 m to 2.695 m. The side-span cast-in-place segments, side-span closure segments, and mid-span closure segment all have a constant depth of 2.695 m. The top slab thickness is 0.25 m; the bottom slab thickness varies from 0.25 m to 0.579 m; the web thickness varies from 0.5 m to 0.7 m; and the wing thickness varies from 0.2 m to 0.7 m, with a constant wing width of 2.5 m.

The substructure consists of thin-walled piers, U-shaped abutments at both ends, and friction pile foundations. Each pier is rectangular, measuring 18 m transversely and 4 m longitudinally. The pier cap is also rectangular, with transverse length of 21.6 m, longitudinal length of 7.5 m, and depth of 3.0 m. The foundation is composed of bored piles with a diameter of 2.0 m, and eight piles are arranged under each pier. The main cross-sections properties are listed in Table 1, where the I_{xx} , I_{yy} , and I_{zz} denote the longitudinal, transverse, and vertical directions of the bridge, respectively.

Table 1. The main cross-sections properties of the bridge.

Cross-Section Location	I_{xx} (m ⁴)	I_{yy} (m ⁴)	I_{zz} (m ⁴)
Mid-span of the main girder's middle span	55.311	18.186	985.924
Location at the pier top of the main girder	209.502	79.696	1580.289
The end-support location of the main girder	55.311	18.186	985.924
Pier shaft section	33.025	9.600	1944.000

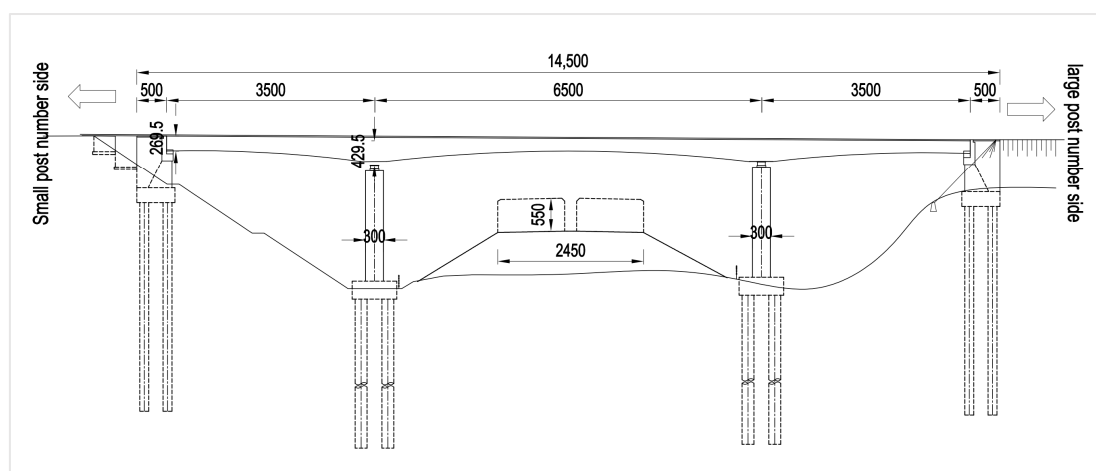


Figure 1. Longitudinal section of the bridge.

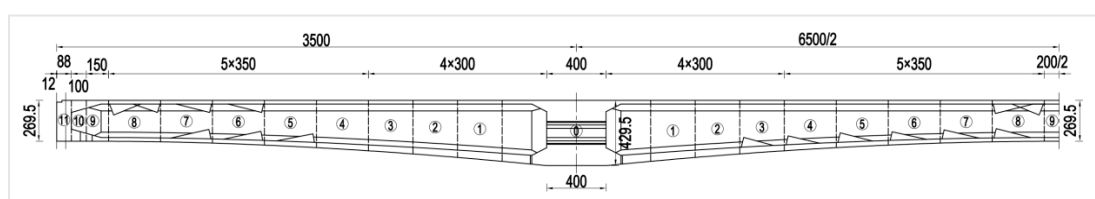


Figure 2. Longitudinal section of main girder (The circled numbers indicate the cast-in-place segment numbers of the main girder).

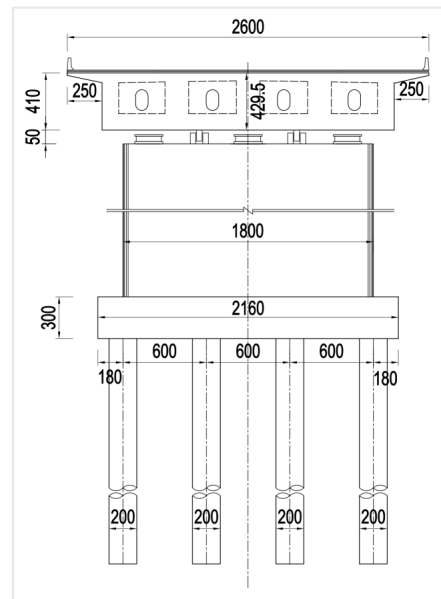


Figure 3. Cross-section of the bridge.

2.2. Model Establishment

As illustrated in Figure 4, a beam-element finite element model was developed using Midas Civil and Midas CDN. To investigate the seismic response characteristics of thin-walled piers with varying heights, three pier heights of 20 m, 50 m, and 80 m were considered in the modeling. The computational model was established in accordance with the Specifications for Seismic Design of Highway Bridges [34].

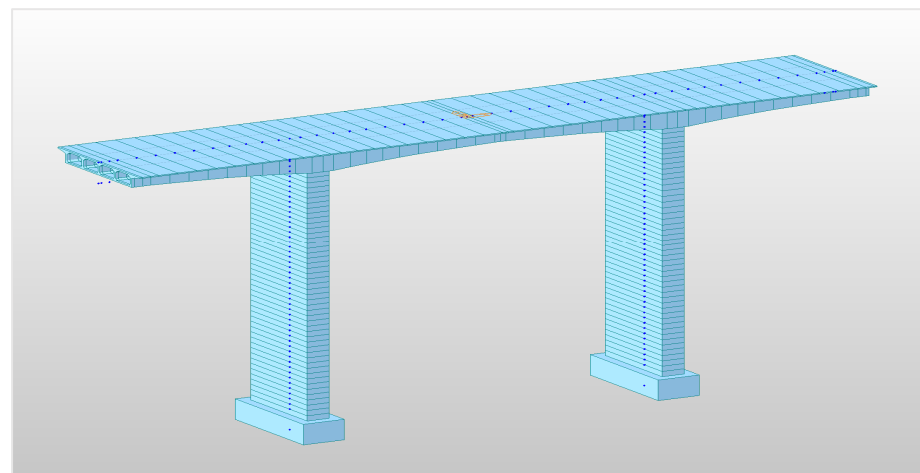


Figure 4. Midas-based finite element dynamic analysis model (Blue dots represent the model nodes).

2.2.1. Actions on the Bridge

The loads considered in the structural analysis included permanent actions (i.e., self-weight of the main girder and cross-beams, secondary dead loads such as deck overlay and barriers, and prestressing forces) and seismic actions. The dead loads were converted into masses, with directions assigned along the X, Y, and Z axes.

2.2.2. Main Bridge Materials and Their Parameters

The cast-in-place box girder and the cast-in-place deck overlay are constructed with C50 concrete, the main bridge piers with C40 concrete, and the pile caps and pile

foundations with C30 concrete. The unit weight of concrete is taken as 26 kN/m^3 . The deck pavement is made of asphalt concrete with a unit weight of 24 kN/m^3 .

The main girder is designed as a prestressed reinforced concrete structure, while the bridge piers are designed as ordinary reinforced concrete structures. The prestressing tendons consist of 15.2 mm diameter steel strands for prestressed concrete, with a total prestressing steel weight of 127 tons. The elastic modulus of the strands is $1.95 \times 10^5 \text{ MPa}$, and the standard tensile strength is 1860 MPa. The jacking stress after anchoring is controlled at 1395 MPa. The curvature friction coefficient of the ducts is taken as 0.0015, the wobble friction coefficient as 0.17, and the anchorage set and tendon draw-in as 0.006 m. Plastic corrugated ducts are adopted.

The stirrups in the bridge piers are fabricated using HPB300 hot-rolled ribbed bars with a nominal diameter of 12 mm, and the longitudinal reinforcing bars in the piers are fabricated using HRB400 hot-rolled ribbed bars with a nominal diameter of 28 mm. In the parametric analysis of reinforcement ratios, the stirrup ratio is varied by adjusting the stirrup spacing and the number of stirrup legs, while the longitudinal reinforcement ratio is varied by adjusting the bar spacing and the number of longitudinal reinforcement rows.

2.2.3. Pier Bearings and Boundary Conditions

Fixed pot bearings are adopted at the left pier of the main bridge, while movable pot bearings are adopted at the right pier and the abutment locations. All degrees of freedom are constrained at the pier bases. The influence of abutment configurations on the seismic response of the main girder is not considered; therefore, the abutments are not established in the modeling process. Instead, the constraint relationship between the main girder and the abutments is simulated by restraining the transverse and vertical translational degrees of freedom at the ends of the main girder. Figure 2 presents the dynamic analysis model of the bridge, in which the boundary conditions of the bridge model are reflected.

2.3. Research Framework

Based on the finite element model described above, the research analysis of this paper is carried out in three progressive stages.

Stage 1: Dynamic characteristic analysis. Through modal analysis, the natural vibration periods, mode shapes, and mass participation factors of the structure with different pier heights (20 m, 50 m, and 80 m) are obtained, and the number of modes required to satisfy the accuracy requirement is determined. These results lay a foundation for the subsequent seismic response analysis.

Stage 2: Seismic response analysis. Under two seismic fortification levels, namely E1 and E2, the response spectra with four peak ground accelerations ranging from 0.1 g to 0.3 g are input. The internal forces (axial force, shear force, and bending moment) at the pier bases and the displacements at the pier tops are extracted. These data are used to evaluate the seismic response variation with pier height and the mechanistic differences between low and high piers.

Stage 3: Sensitivity analysis of reinforcement ratio and optimization recommendations. Taking the 50 m high pier as the research object, the effects of longitudinal reinforcement ratio (ranging from 0.6% to 2.5%) on axial and flexural bearing capacities as well as displacement limits are examined, and the influence of stirrup ratio on shear capacity is investigated. Furthermore, by synthesizing the reinforcement demands of the three pier heights under different seismic intensities, recommendations for seismic design reinforcement ratios and a dimensionless discriminant criterion based on pier height classification are proposed to provide guidance for engineering design.

3. Selection of Ground Motion Parameters

The seismic analysis of the bridge was conducted using the response spectrum method. According to the Seismic Ground Motion Parameters Zonation Map of China [35], the Specifications for Seismic Design of Highway Bridges [34], and the geological survey report at the bridge site, the parameters for the response spectrum analysis were selected as follows: the characteristic period of the response spectrum is 0.40 s; the site classification is Class II; the seismic fortification intensities are taken as VII to VIII; the peak ground accelerations at the bridge site are taken as 0.1 g, 0.15 g, 0.2 g, and 0.3 g, respectively; and the building site soil is classified as Class II.

According to the Specifications for Seismic Design of Highway Bridges [34], the bridge is classified as seismic fortification category B. The design acceleration response spectrum, $S(T)$, is determined by Equation (1).

$$S(T) = \begin{cases} S_{\max} (0.6T/T_0 + 0.4) & T \leq 0 \\ S_{\max} & T_0 \leq T \leq T_g \\ S_{\max} (T_g/T) & T_g \leq T \leq 10 \end{cases} \quad (1)$$

where T_g is the characteristic period; T_0 is the maximum period of the linear ascending branch of the response spectrum; and $S_{\max}$ is the peak value of the design acceleration response spectrum (g), determined by $S_{\max} = 2.5C_iC_sC_dA$, with the specific values of the relevant parameters given in Table 2.

Table 2. Response spectrum parameters.

50-Year Exceedance Probability	T_g/s	T_0/s	C_i (Blast Resistance Importance Coefficient)	C_s (Horizontal Site Coefficient)	C_s (Vertical Site Coefficient)	C_d (Damping Adjustment Coefficient)
10%	0.4	0.1	0.5	1.0	0.7	1
2%	0.4	0.1	1.7	1.0	0.6/0.7/0.8	1

The horizontal basic seismic peak ground acceleration A was varied as 0.1 g, 0.15 g, 0.2 g, and 0.3 g. For horizontal seismic action, the corresponding $S_{\max}$ values are 0.1250 g, 0.1875 g, 0.2500 g, and 0.3750 g under E1 seismic level, and 0.4250 g, 0.6375 g, 0.8500 g, and 1.2750 g under E2 seismic level. The horizontal seismic response spectra corresponding to different peak ground accelerations are shown in Figure 5.

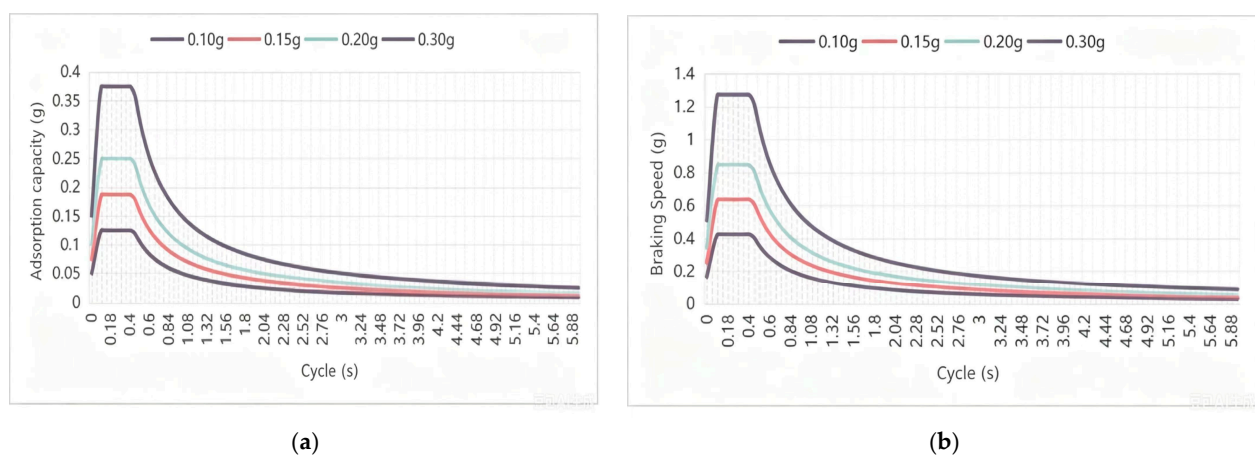


Figure 5. Horizontal design ground motion response spectra. (a) E1 Seismic action; (b) E2 Seismic action.

4. Seismic Performance and Rational Reinforcement Ratio Analysis

4.1. Analysis of Structural Dynamic Characteristics

To rationally determine the cutoff mode number for seismic response analysis, the evolution of vibration modes in continuous girder bridges with varying pier heights was investigated. This evolution depends on the stiffness distribution and mass inertia characteristics of the structure in the elastic stage. For this purpose, a series of dynamic characteristic analysis models of the continuous girder bridge with different pier heights were established. The relationship between the cumulative modal mass participation factor and the number of required modes was examined. According to the Highway Bridge Seismic Design Specifications [34], when the multi-mode response spectrum method is applied, the cumulative modal mass participation factor in the direction of seismic excitation must reach at least 90%. Therefore, the minimum cutoff mode number that satisfies this code requirement was determined for each pier height scenario.

The eigenvalue modes for the dynamic analysis of the bridge are presented in Table 3, and the natural vibration characteristic parameters of the bridge piers under different pier heights are shown in Figure 6. Based on the presented data, as the pier height increases from 20 m to 80 m, the fundamental period of the structure prolongs from 1.02 s to 3.38 s, an increase of more than 230%. This indicates that continuous girder bridges with high piers exhibit pronounced flexibility. In contrast, most stiff and moderately stiff sites (e.g., bedrock, dense sand) have short predominant periods, typically ranging from 0.1 s to 0.6 s; even for deep soft-soil sites, the predominant period generally falls between 1.0 s and 2.0 s. Therefore, the use of high-pier structures can effectively avoid the predominant period of ground motion, thereby reducing seismic response.

Table 3. Eigenvalue modes for dynamic analysis of the bridge.

Pier Height (m)	Longitudinal Participating Mass (%)			Transverse Participating Mass (%)		
	Modes Required	This Mode (%)	Cumulative (%)	Modes Required	This Mode (%)	Cumulative (%)
20	16	2.8	92.8	24	8.2	90.6
50	13	4.9	91.7	30	2.3	90.8
80	17	2.1	91.1	35	0.8	90.6

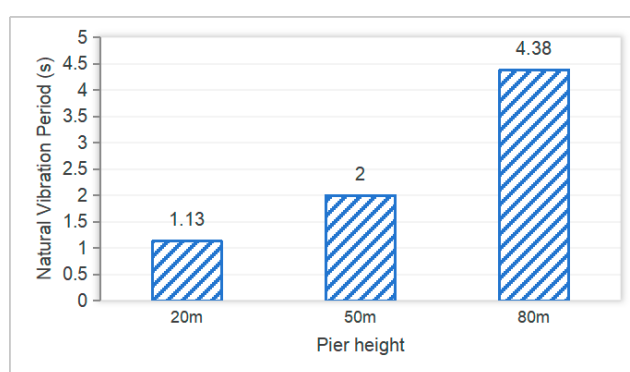


Figure 6. Natural vibration characteristic parameters of bridge pier columns with varying pier heights.

Furthermore, the dynamic response of high-pier continuous girder bridges exhibits significant local vibration characteristics. For a pier height of 20 m, the first three modes accumulate more than 60% of the modal mass participation factor. However, when the pier height reaches 80 m, the first ten modes are primarily local bending or torsional modes of the pier itself, each contributing very inefficiently to the overall mass participation, often less than 1% per mode. To ensure that the cumulative mass participation

coefficient reaches the 90% requirement specified in the code, the number of modes required for extraction increases dramatically with pier height, rising from 24 to 35 as shown in Table 2. This directly reflects the complexity of the dynamic response of high-pier bridges, and the contribution of higher-order modes (particularly multiple bending modes of the pier shaft) to the total response cannot be neglected.

Based on the above convergence analysis, extracting only 35 modes, while theoretically sufficient to marginally meet the 90% mass participation requirement, often results in a noticeable truncation effect. This may lead to errors in the calculated base shear and displacement at the pier top. To ensure high-precision comparisons among models with different pier heights while maintaining computational efficiency, 60 modes were uniformly extracted for all iterative calculations. This approach guarantees that, even in the most demanding case (80 m pier height), the cumulative modal mass participation factor exceeds 95%, effectively controlling modal truncation errors and providing a reliable basis for subsequent response spectrum and time-history analyses.

4.2. Seismic Performance Analysis of Thin-Walled Piers

To evaluate the seismic performance of thin-walled piers with varying heights under E1 and E2 earthquake levels, the internal force responses at the pier base and the displacements at the pier top were analyzed under increasing peak ground accelerations (0.1 g, 0.15 g, 0.2 g, and 0.3 g). The internal force responses at the pier base included axial force, shear force, and bending moments in both the transverse and longitudinal directions.

4.2.1. Axial Force and Shear Force Responses

Comparisons of pier-base axial and shear forces under E1 seismic action (Figure 7) indicate that peak ground acceleration growth leads to a significant increase in shear force, yet axial force remains nearly constant. Taking the 20 m pier as an example, when the acceleration increased from 0.1 g to 0.3 g, the axial force increased from 263,902.8 kN to 264,158.9 kN (an increase of <0.1%), while the shear force increased from 27,337.3 kN to 37,455.8 kN (an increase of 37.0%). For the 80 m pier, the shear force increased from 4202.9 kN to 14,607.7 kN, corresponding to an increase of 247.6%. Furthermore, the longitudinal shear force at the pier base decreased with increasing pier height (the shear force of the 20 m pier was approximately 2.6 times that of the 80 m pier), but the rate of increase in shear force became more pronounced at greater pier heights. Overall, the axial force was insensitive to variations in peak ground acceleration, whereas the shear force increased markedly with acceleration, with the most significant increase observed for the 80 m pier.

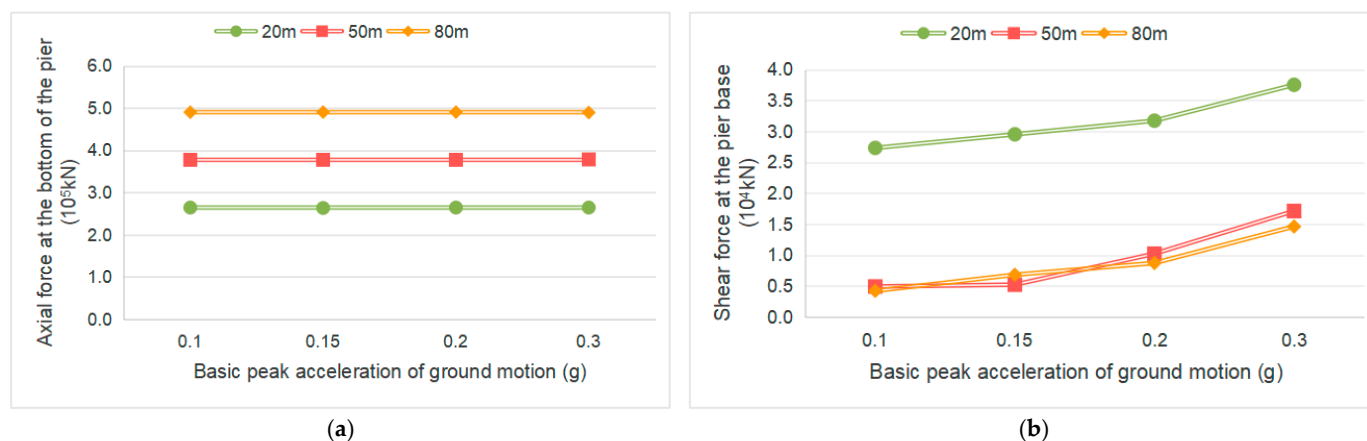


Figure 7. Axial force and shear force at the base of Pier No. 1. (a) Axial force; (b) Shear force.

4.2.2. Longitudinal Bending Moment Response

Comparison of the longitudinal bending moment responses under E1 earthquake actions for different pier heights in Figure 8 reveals the following trends. For the 20 m high pier, the bending moment increases from 6.096×10^5 kN·m to 8.987×10^5 kN·m, representing an increase of approximately 47.4% with an absolute increment of 2.891×10^5 kN·m. For the 50 m high pier, the bending moment increases from 1.975×10^5 kN·m to 4.334×10^5 kN·m, corresponding to an increase of approximately 119.4% with an absolute increment of 2.359×10^5 kN·m. For the 80 m high pier, the bending moment increases from 1.511×10^5 kN·m to 3.889×10^5 kN·m, corresponding to an increase of approximately 157.4% with an absolute increment of 2.378×10^5 kN·m.

The analysis indicates that the bending moment responses of low and high piers follow distinctly different patterns. In terms of the percentage increase, the amplification of the bending moment grows with increasing pier height, and the sensitivity of the bending moment to the peak ground acceleration becomes significantly more pronounced for taller piers. However, in terms of the absolute increment, the increases for the three pier heights are essentially on the same order of magnitude (approximately 2.3×10^5 kN·m– 2.9×10^5 kN·m), suggesting that the increase in seismic forces induced by higher acceleration levels does not exhibit a substantial numerical difference across the pier heights. What truly governs the disparities in the absolute bending moment levels among the piers is the baseline internal force level dictated by their initial stiffness.

Notably, the absolute bending moment of the 20 m high pier (approximately 8.99×10^5 kN·m) is far higher than that of the 50 m high pier (approximately 4.33×10^5 kN·m) and the 80 m high pier (approximately 3.89×10^5 kN·m), reaching 1.6 to 2.2 times the latter values. The primary reason for this phenomenon is that the excessively low pier height results in excessive pier stiffness, causing the structural deformation to be dominated by shear behavior with limited energy dissipation capacity, thereby generating substantial internal forces under seismic excitation. Therefore, in high-seismic-intensity regions, the flexural strength design of low piers, like that of high piers, constitutes a critical issue that warrants particular attention.

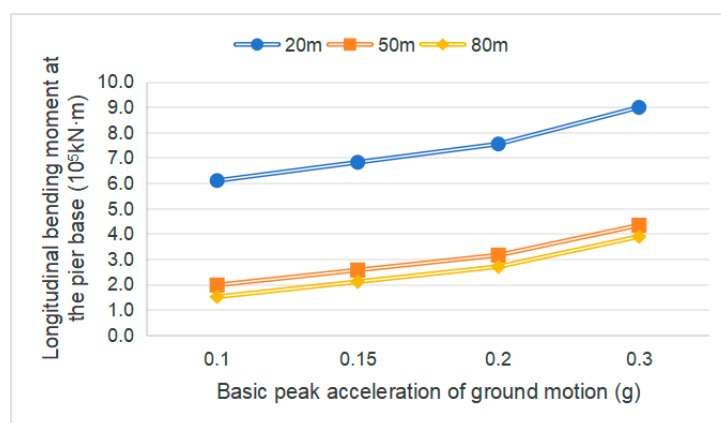


Figure 8. Longitudinal bending moment at the base of the left pier.

4.2.3. Transverse Bending m20 Moment Response

Analysis of the transverse bending moment data under E1 earthquake actions in Figure 9 reveals the following. For the 20 m high pier, the transverse bending moment increases from 0.441×10^4 kN·m to 1.322×10^4 kN·m, with an absolute increment of 0.881×10^4 kN·m, corresponding to an increase of approximately 200%. For the 50 m high pier, the bending moment increases from 0.942×10^4 kN·m to 2.826×10^4 kN·m, with an absolute increment of 1.884×10^4 kN·m, also an increase of approximately 200%. For the 80 m high

pier, the bending moment increases from 0.955×10^4 kN·m to 2.865×10^4 kN·m, with an absolute increment of 1.910×10^4 kN·m, again an increase of approximately 200%. The percentage increases for the three pier heights are highly consistent, indicating that the growth of the transverse bending moment is primarily governed by the amplification effect of the peak ground acceleration, while the pier height exerts no significant influence on the percentage increase.

However, in terms of the absolute increment, the increment for the 20 m high pier (0.881×10^4 kN·m) is considerably lower than those for the 50 m high pier (1.884×10^4 kN·m) and the 80 m high pier (1.910×10^4 kN·m). This suggests that although pier height does not alter the proportional growth of the transverse bending moment, it significantly affects the absolute magnitude of the increment—medium and high piers experience substantially larger seismic internal force increments in the transverse direction than low piers.

In terms of absolute values, the transverse bending moment of the 50 m high pier at 0.3 g reaches 2.826×10^4 kN·m, which is slightly lower than that of the 80 m high pier (2.865×10^4 kN·m) but significantly higher than that of the 20 m high pier (1.322×10^4 kN·m). This indicates that the transverse bending moment is markedly influenced by pier height for low-to-medium height piers, whereas the influence becomes less pronounced for taller piers.

It is worth noting that the transverse bending moment of the 50 m high pier already accounts for 65.2% of the corresponding longitudinal bending moment under the same loading condition (4.334×10^4 kN·m), whereas for the 20 m high pier, this ratio is only 14.7%. This demonstrates that as pier height increases, the seismic threat in the transverse direction rises significantly and constitutes a considerable proportion for medium and high piers, thus deserving due attention in seismic design.

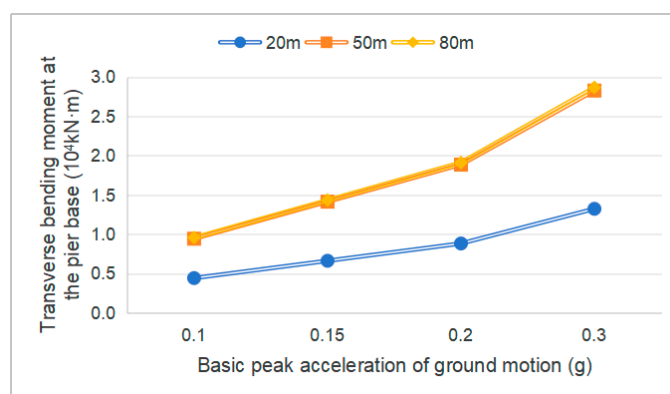


Figure 9. Transverse bending moment at the base of Pier No. 1.

4.2.4. Pier-Top Displacement Response

In a design that balances economy and seismic performance, piers are required to remain elastic under the E1 earthquake level and to enter the plastic state under the E2 earthquake level. This transition shifts the design criterion from strength control to displacement control at the pier top. Once the pier enters the plastic state, the plastic hinge region begins to operate with cracking. To reasonably capture the actual mechanical behaviour after concrete cracking in the plastic hinge zone, the elastic stiffness of the pier should not be overestimated. Therefore, the calculated stiffness must be reduced to the cracked stiffness using an effective stiffness reduction factor. According to the Highway Bridge Seismic Design Specifications [32], the effective stiffness reduction factor is determined through calculation based on the equivalent yield stiffness obtained from moment-curvature analysis and the pre-cracking stiffness.

As shown in Figure 10, for a given pier height, the stiffness reduction factor exhibited no significant variation beyond the second decimal place under different peak ground accelerations. This indicates that, under the E2 earthquake level, the stiffness reduction in the pier column depends primarily on the pier height and direction, and is insensitive to the peak acceleration value. Therefore, the stiffness reduction factor at 0.30 g was adopted as the basis for seismic analysis.

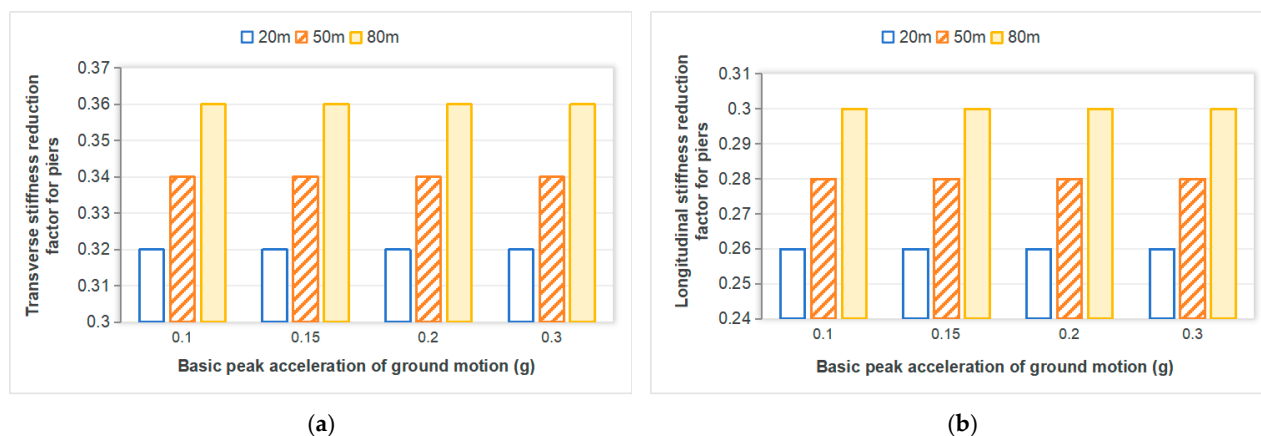


Figure 10. Stiffness reduction factors for pier columns. (a) Transverse stiffness reduction factor for piers; (b) Longitudinal stiffness reduction factor for piers.

Figure 11 lists the pier top displacement and its limit when the peak dynamic acceleration is 0.30 g.

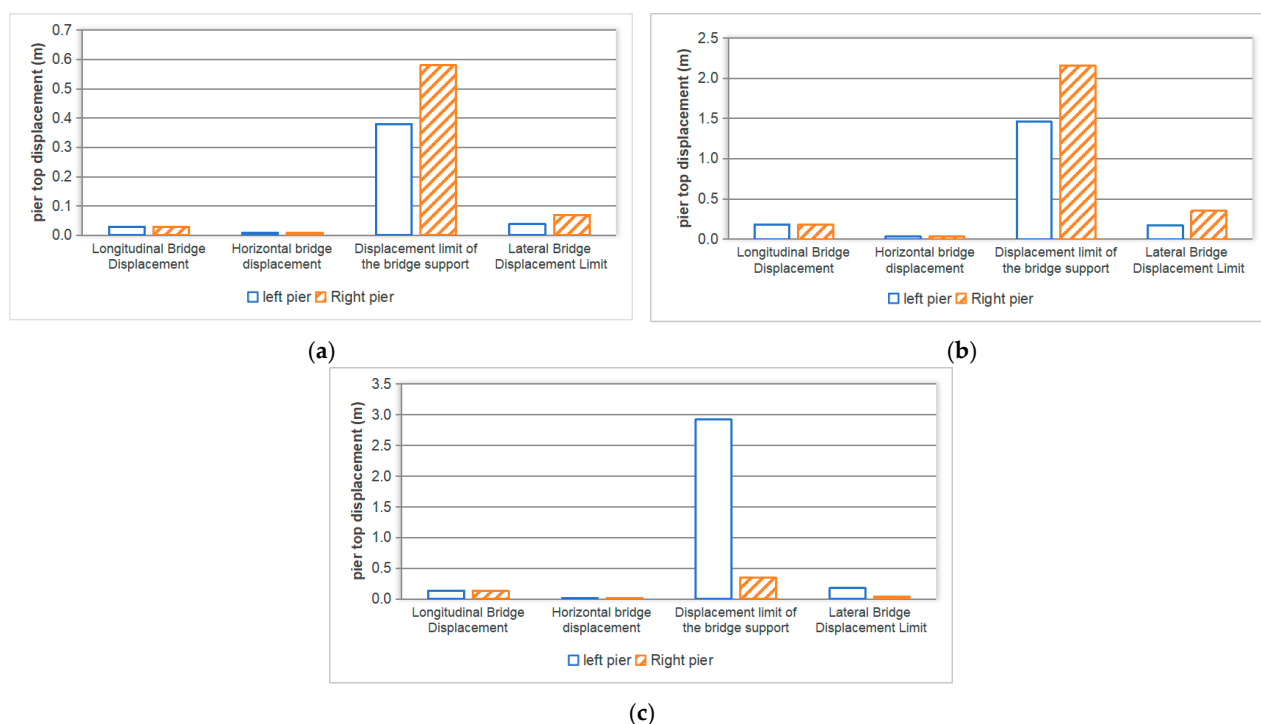


Figure 11. Pier top displacement under E2 earthquake action. (a) The pier top displacement of the 20 m high pier; (b) The pier top displacement of the 50 m high pier; (c) The pier top displacement of the 80 m high pier.

Figure 11 shows that pier height had a significant effect on the seismic displacement response of the thin-walled piers. When the pier height increased from 20 m to 50 m, the longitudinal displacement increased from 0.03 m to 0.18 m, an increase of 500%, while the

transverse displacement increased from 0.01 m to 0.03 m, an increase of 200%. This was attributed to the reduction in overall structural stiffness and increase in flexibility with increasing pier height, which made the structure more prone to large deformations under seismic action. However, when the pier height was further increased to 80 m, the longitudinal displacement decreased to 0.14 m and the transverse displacement decreased to 0.02 m, reversing the increasing trend observed at 50 m. This observation indicates that the displacement response of very tall piers does not increase monotonically with height, and is likely influenced by a combination of factors, including contributions from higher-order modes, variations in mass distribution along the pier, and boundary conditions.

Regarding compliance with displacement limits, the pier top displacements in all cases were well below the corresponding limits, indicating a sufficient safety margin. Taking the longitudinal direction as an example, the displacement of the 20 m pier accounted for 7.9–8.6% of its limit, that of the 50 m pier for 8.3–12.3%, and that of the 80 m pier for only 4.8%. In the transverse direction, the displacement of the 20 m pier accounted for 13.3–25.0% of its limit, that of the 50 m pier for 8.6–17.6%, and that of the 80 m pier (right pier) for 50.0%. The latter case had the most stringent displacement limit among all cases (only 0.04 m). The actual displacement in this case was 0.02 m, reaching 50% of the limit, thus offering a smaller safety margin compared with other cases. It is therefore recommended that particular attention be paid to transverse displacement verification in the design of similar tall piers.

Figure 11 also reveals that at pier heights of 20 m and 50 m, the longitudinal displacement limit of the expansion pier (2.16 m) was considerably larger than that of the fixed pier (1.46 m). At a pier height of 80 m, however, the longitudinal displacement limit of the fixed pier (2.92 m) far exceeded that of the expansion pier (0.35 m), and the transverse limit of the expansion pier (0.04 m) was also significantly lower than that of the fixed pier (0.18 m). These observations indicate that, for tall piers, the stability of the expansion pier and the deformation capacity of the bearings become severely constrained, and the controlling design factor shifts from strength to stability. Based on the findings, the following design recommendations are made: (1) Low piers can be designed according to conventional ductility design principles. (2) For medium piers, attention should be paid to the displacement limit of movable piers, and their deformation capacity should be appropriately enhanced. (3) For high piers, a rigid-frame system is preferable, and particular emphasis should be placed on checking the risk of instability.

Overall, within the pier height range of 20 m to 80 m, the thin-walled piers exhibited satisfactory displacement control performance under the E2 earthquake level. Nevertheless, the 50 m pier showed the largest displacement response, and the 80 m pier exhibited a relatively smaller displacement safety margin. In seismic design, special attention should therefore be paid to the displacement magnitude, redundancy, and stability of tall piers.

4.3. Sensitivity Analysis of Reinforcement Ratios on Seismic Performance

To reveal the influence patterns of longitudinal reinforcement ratio and stirrup ratio on the seismic performance of bridge piers, and to provide a basis for determining reasonable reinforcement ratios, this section takes the 50 m high pier as the research object. Under E2 earthquake actions (with peak ground accelerations of 0.2 g and 0.3 g, respectively), the response patterns of flexural capacity, shear capacity, and displacement limits of the pier are systematically investigated as the reinforcement ratio varies from 0.6% to 2.5%.

4.3.1. Effect of Longitudinal Reinforcement Ratio on Flexural Capacity of the Pier

The eccentric compression bearing capacity of the bridge pier under the minimum axial force and the maximum primary bending moment conditions can reflect the flexural performance of the pier under different loading states. The flexural capacities of the 50 m high pier under E2 earthquake actions with varying longitudinal reinforcement ratios are presented in Figures 12 and 13.

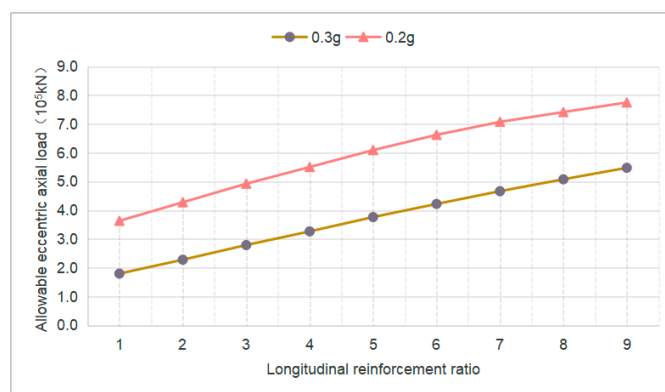


Figure 12. flexural capacity at the minimum axial force.

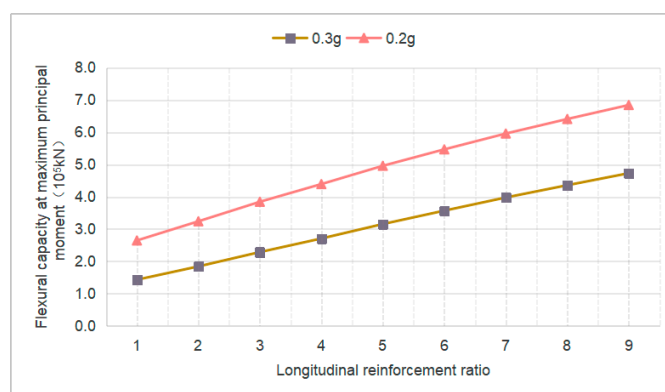


Figure 13. flexural capacity at the maximum primary bending moment.

(1) The flexural capacity at the minimum axial force increases approximately linearly with the increase in reinforcement ratio. When the reinforcement ratio increases from 0.6% to 2.5%, the allowable axial force increases from 3.63×10^5 kN to 7.75×10^5 kN at a peak ground acceleration of 0.2 g, representing an increase of approximately 113%. At a higher peak ground acceleration of 0.3 g, the same reinforcement increase raises the allowable axial force from 1.79×10^5 kN to 5.48×10^5 kN, corresponding to a more pronounced increase of approximately 206%. This indicates that the marginal contribution of longitudinal reinforcement is higher under strong earthquakes, as the reinforcement can effectively delay the crushing process of concrete in the compression zone. Therefore, the effectiveness of increasing reinforcement is more pronounced under severe seismic conditions. However, the absolute bearing capacity at 0.3 g remains lower than that at 0.2 g with a reinforcement ratio of 1.0%.

The reason is that in high-seismic-intensity regions, once the reinforcement ratio exceeds a certain level, the sectional bearing capacity approaches the upper limit governed by the ultimate compressive strain of concrete. To more effectively enhance the seismic performance of the pier, consideration should be given to increasing the sectional dimensions or improving the concrete strength grade.

(2) The flexural capacity at the maximum primary bending moment also increases approximately linearly with the reinforcement ratio, without exhibiting a clear saturation trend. As the reinforcement ratio increases from 0.6% to 2.5%, the flexural capacity increases from 2.65×10^6 kN to 6.84×10^6 kN at a peak ground acceleration of 0.2 g, representing an increase of approximately 231%. At a higher peak ground acceleration of 0.3 g, the same reinforcement increase raises the flexural capacity from 1.43×10^6 kN to 4.73×10^6 kN, also corresponding to an increase of approximately 231%. The flexural capacity maintains a consistently high sensitivity to the reinforcement ratio, exhibiting near-linear growth under both seismic intensity levels without an apparent saturation trend. This suggests that within the reinforcement ratio range of 0.6% to 2.5%, the contribution of increasing longitudinal reinforcement to enhancing the sectional flexural capacity is continuous and stable.

(3) Under the same reinforcement ratio, the flexural capacity at a peak ground acceleration of 0.2 g is approximately 1.4 to 1.8 times that at a peak ground acceleration of 0.3 g. This is because under the 0.3 g earthquake action, the section enters a state of small eccentric compression, where the compression zone height increases and the tensile strain in the longitudinal reinforcement decreases, thereby suppressing the flexural contribution of the longitudinal bars. The flexural capacity exhibits higher sensitivity to the reinforcement ratio under the high acceleration level (0.3 g). Although the absolute values are lower, the percentage increase at 0.3 g (231%) is significantly higher than that at 0.2 g (158%). This indicates that under strong earthquakes, the flexural benefit from increasing the reinforcement ratio is more pronounced—since the longitudinal reinforcement also participates in the compression zone to resist concrete crushing, the enhancement of flexural capacity is more substantial.

4.3.2. Effect of Longitudinal Reinforcement Ratio on the Longitudinal Displacement Limit of the Pier

Unlike the sensitivity of bearing capacity to longitudinal reinforcement configuration, the variation in the displacement limit with the longitudinal reinforcement ratio for the 50 m high pier under E2 earthquake actions is presented in Figure 14. The results indicate that the displacement limit increases only slowly and linearly with increasing reinforcement ratio: as the reinforcement ratio increases from 0.6% to 2.5%, the displacement limit increases from 1.518 m to 1.564 m, representing an increase of merely about 3.0%. This suggests that the displacement limit is far less sensitive to the reinforcement ratio than the bearing capacity is, and that increasing longitudinal reinforcement offers very limited improvement in deformation capacity.

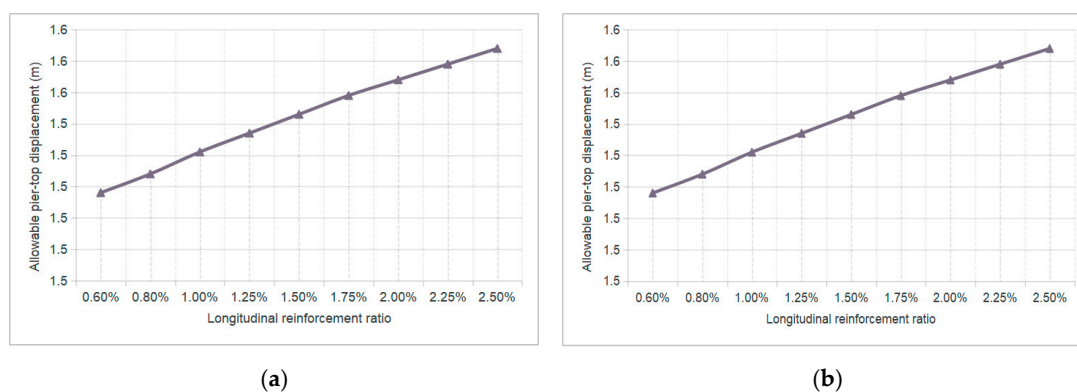


Figure 14. longitudinal displacement limits of the pier under different longitudinal reinforcement ratios. (a) variation in pier longitudinal displacement with longitudinal reinforcement ratio under a

peak ground acceleration of 0.2 g; (b) variation in pier longitudinal displacement with longitudinal reinforcement ratio under a peak ground acceleration of 0.3 g.

4.3.3. Effect of Stirrup Ratio on Allowable Shear Force

The allowable shear forces of the 50 m high pier under E2 earthquake actions for varying stirrup ratios are presented in Figure 15. It can be observed from the figure that the longitudinal allowable shear force ceases to increase once the stirrup ratio exceeds 1.75%. As the stirrup ratio increases from 0.6% to 1.75%, the allowable shear force rises from 0.80×10^5 kN to 2.10×10^5 kN, representing an increase of approximately 163%. However, when the stirrup ratio exceeds 1.75%, the allowable shear force stabilizes at 2.10×10^5 kN and no longer increases with further enhancement of the stirrup ratio. This indicates that the shear capacity of the section is governed by the diagonal compression strength of concrete, and merely increasing the stirrups cannot further improve the shear resistance.

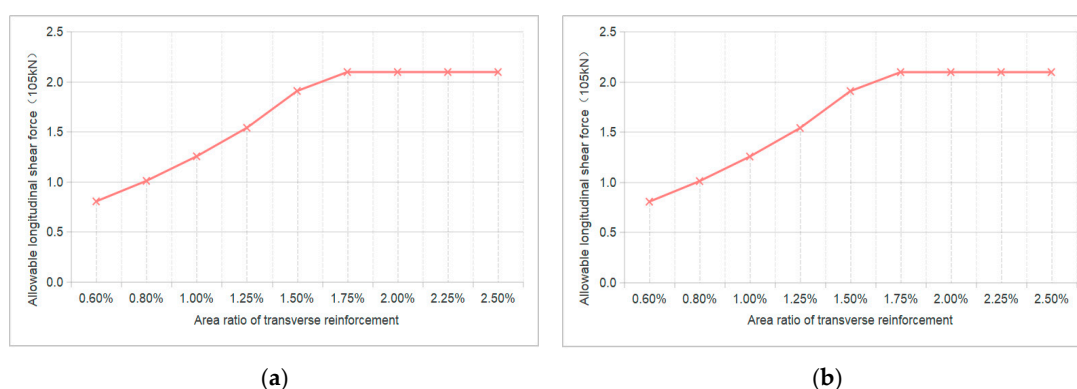


Figure 15. longitudinal allowable shear forces under different stirrup ratios. (a) variation in longitudinal allowable shear force with stirrup ratio under a peak ground acceleration of 0.2 g; (b) variation in longitudinal allowable shear force with stirrup ratio under a peak ground acceleration of 0.3 g.

In summary, the longitudinal reinforcement ratio significantly enhances the bearing capacity of the pier, with an approximately linear growth relationship. And the marginal contribution of longitudinal reinforcement is higher under strong earthquakes (0.3 g). However, the bearing capacity approaches the upper limit governed by the ultimate compressive strain of concrete at high reinforcement ratios, suggesting that increasing the sectional dimensions or improving the concrete strength grade is a more effective approach for further enhancement. In contrast, the displacement limit cannot be improved by adjusting the longitudinal reinforcement ratio.

4.4. Analysis of Reasonable Reinforcement Ratio for Thin-Walled Piers

4.4.1. Longitudinal Reinforcement Ratio

Considering the ability of piers with different heights to remain elastic under the E1 earthquake level, as well as the requirements for plastic development and displacement ductility under the E2 earthquake level, the variation in the minimum required reinforcement ratio with pier height was analyzed. Figure 16 presents the minimum longitudinal reinforcement ratio required to maintain the seismic performance of the piers at different pier heights.

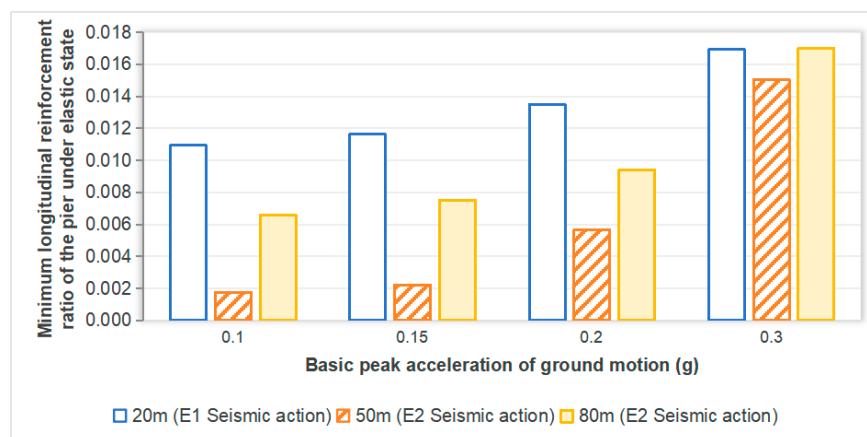


Figure 16. Analysis of longitudinal reinforcement ratio for different pier heights.

It should be noted that, for the 20 m short pier, even increasing the reinforcement ratio could not satisfy the pier top displacement limit under the E2 earthquake level. Therefore, the reinforcement ratio shown for the 20 m pier in Figure 16 corresponds to the minimum ratio required to maintain elastic behavior under the E1 earthquake level (frequent earthquake). For the 50 m and 80 m tall piers, the values represent the minimum longitudinal reinforcement ratios required to meet the displacement ductility capacity under the E2 earthquake level (rare earthquake).

According to the longitudinal reinforcement ratio data for different pier heights presented in Figure 16.

(1) The reinforcement demand for the short pier was significantly higher than that for the tall piers. The minimum reinforcement ratio of the 20 m pier under the E1 earthquake level was still greater than that of the 50 m and 80 m piers under the E2 earthquake level. This was because, under the E1 level, the pier was required to remain elastic and thus experienced larger internal forces, whereas the tall piers under the E2 level could dissipate energy through plastic deformation, resulting in relatively lower internal forces. In seismic design, special attention should be paid to short piers; enlarging the cross-section or adopting seismic isolation measures should be considered when necessary.

(2) As the peak ground acceleration increased, the reinforcement ratio increased, and the rate of increase accelerated. Taking the 50 m pier as an example, the reinforcement ratio increased from approximately 0.8% at 0.1 g to approximately 1.5% at 0.3 g (an increase of about 0.7%), with the increment showing an accelerating trend with higher seismic intensity. This reflected that at higher earthquake levels, the stiffness degradation of the pier became pronounced, requiring the reinforcement to sustain a larger share of the internal forces.

(3) The reinforcement ratio for tall piers was governed by construction (minimum) requirements rather than by seismic demands. Under all seismic accelerations considered, the minimum required reinforcement ratio for the 80 m tall pier was below the 0.6% constructional reinforcement ratio specified in the Highway Bridge Seismic Design Specifications [32]. This indicates that seismic demand does not control the reinforcement design of tall piers, and reinforcement can be provided according to constructional requirements.

4.4.2. Stirrup Reinforcement Ratio

Finite element analysis indicated that for the 20 m short pier, increasing the stirrup reinforcement ratio could not satisfy the shear capacity requirement under seismic action. Therefore, either enlarging the pier cross-section or adopting seismic isolation measures was necessary to achieve the desired seismic performance. Consequently, Figure 17

presents only the minimum stirrup reinforcement ratios for pier heights of 50 m and 80 m under different peak ground accelerations (0.1 g, 0.15 g, 0.20 g, and 0.30 g).

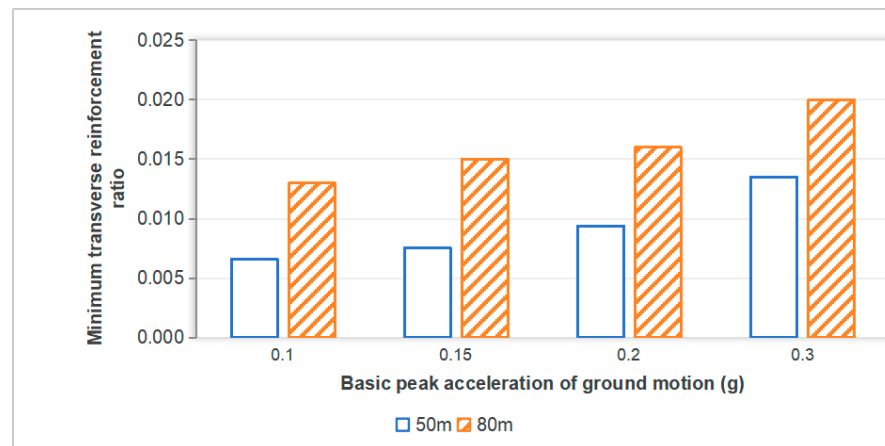


Figure 17. Analysis of Stirrup Reinforcement Ratio for different pier heights.

Overall, as the peak ground acceleration increased, the minimum stirrup reinforcement ratio increased for both pier heights, indicating that higher seismic demands required stronger confinement from the stirrups. When the acceleration increased from 0.1 g to 0.30 g, the minimum stirrup ratio for the 50 m pier increased from 0.007 to 0.014 (an increase of approximately 100%), while that for the 80 m pier increased from 0.013 to 0.020 (an increase of approximately 54%). When the acceleration increased from 0.20 g to 0.30 g, the stirrup ratio for the 50 m pier jumped from 0.009 to 0.014 (an increase of 55.6%), whereas that for the 80 m pier rose from 0.016 to 0.020 (an increase of 25%). The lower pier (50 m) exhibited a higher growth rate of stirrup ratio at high accelerations, indicating that its seismic reinforcement design was more sensitive to changes in ground motion intensity. This was likely because the lower pier had greater stiffness, leading to more concentrated seismic forces.

Under the same acceleration level, a greater pier height required a higher minimum stirrup reinforcement ratio. For example, at 0.20 g, the reinforcement ratio is 0.009 for a pier height of 50 m, compared to 0.016 for a pier height of 80 m, which is approximately 78% higher. This suggests that tall piers had a greater demand for stirrup confinement under seismic action, particularly at higher acceleration levels.

Both pier height and peak ground acceleration significantly influenced the minimum stirrup reinforcement ratio. In seismic design, for relatively low piers, special attention should be paid to the abrupt increase in reinforcement ratio at high acceleration levels. For tall piers, a higher stirrup reinforcement ratio is required to meet shear capacity demands.

4.4.3. Dimensionless Results and Summary of Trends

To extend the above reinforcement ratio analysis results to a broader engineering context beyond the specific bridge type investigated in this study, the ratio of the reinforcement demand to the minimum reinforcement ratio specified in the design code is introduced as a dimensionless index. The longitudinal reinforcement ratio index R_l is defined as $R_l = Q_{req}/Q_{min}$, where Q_{min} is the minimum reinforcement ratio (0.6%) specified in the code. When $R_l > 1.0$, the reinforcement is governed by seismic demand; when $R_l < 1.0$, the reinforcement is governed by detailing requirements, and the reinforcement may be provided in accordance with the minimum detailing provisions.

From the data presented in Table 4, the following observations can be made.

Table 4. Reinforcement ratio indices of piers for different pier heights and seismic intensities.

Height-to-Width Ratio	Longitudinal Reinforcement Ratio Indices at Various Seismic Acceleration Levels R_i				Earthquake Action
	0.1 g	0.15 g	0.2 g	0.3 g	
5.0	1.82	1.94	2.25	2.82	E1
12.5	0.30	0.37	0.94	2.51	E2
20.0	1.10	1.25	1.57	2.83	E2

(1) Even under E1 earthquake actions, for the low pier with a height-to-width ratio of $H/B = 5$, the reinforcement ratio index R_i consistently exceeds 1.0 and increases steadily with increasing seismic intensity (from 1.82 to 2.82). This indicates that the reinforcement of low piers is always governed by seismic demand, and this governing effect becomes more pronounced as the intensity increases. When the peak ground acceleration reaches 0.3 g, the required reinforcement approaches three times the minimum detailing requirement. This implies that simply increasing the reinforcement ratio cannot adequately satisfy the seismic demand under such severe conditions.

(2) For the medium-to-high pier with an aspect ratio of $H/B = 12.5$, under low seismic intensities (0.1 g–0.15 g), R_i is considerably less than 1.0 (0.30–0.37), and the reinforcement is governed by detailing requirements. However, when the seismic intensity increases to 0.2 g, R_i rises sharply to 0.94, approaching the threshold for seismic control. When the peak ground acceleration reaches 0.3 g, R_i increases dramatically to 2.51. This trend indicates that there exists a critical seismic intensity range (approximately 0.15 g–0.2 g) for medium-to-high piers where the control mechanism transitions from detailing requirements to seismic demand. Once this threshold is exceeded, the reinforcement demand grows substantially. In design practice, special attention should be paid to the reinforcement design of medium-to-high piers located in high-seismic-intensity regions.

(3) For the high pier with an aspect ratio of $H/B = 20$, the reinforcement ratio index R_i consistently exceeds 1.0 across all seismic intensity levels (1.10–2.83), and increases steadily with increasing seismic intensity. This indicates that as the pier height continues to increase, the reinforcement of the pier remains governed by seismic control.

The above dimensionless index R_i distills the reinforcement ratio results into a concise discriminant criterion, allowing designers to quickly determine whether the reinforcement of a given pier is governed by seismic demand or by detailing requirements. This can be done by simply calculating the height-to-width ratio in the relevant direction to identify the corresponding R_i value. Subsequently, reinforcement design can be carried out in accordance with the empirical relationships presented above, thereby shortening the trial-and-error process in reinforcement design.

5. Discussion

5.1. Fundamental Differences in Seismic Mechanisms Between Low and High Piers

The results indicate that pier height has a decisive impact on the seismic performance of thin-walled piers. Low piers (20 m) exhibit a distinctly internal-force-dominated response under seismic action, whereas high piers (80 m) display a response pattern characterized by displacement-controlled behavior at the pier top.

In terms of the response characteristics of internal forces and displacements, the low pier exhibits a distinctive feature of “high internal forces and small displacements.” For instance, the bending moment at the pier base of the low pier reaches as high as 7.31×10^5 kN·m, which is approximately 2.9 times that of the 80 m high pier. Similarly, the shear force at the pier base reaches 3.7×10^4 kN, approximately 2.6 times that of the 80 m high pier. Meanwhile, the pier-top displacement is extremely small (with a longitudinal displacement of only 0.03 m under a peak ground acceleration of 0.3 g).

The fundamental reason for this phenomenon is that the low pier possesses excessive stiffness, resulting in a short natural vibration period—the fundamental period of the 20 m pier is only 1.02 s. This period approaches the predominant period of ground motion (0.1 s–0.6 s), leading to a significant amplification of seismic inertial forces. In addition, the structural deformation is dominated by shear behavior, and the plastic deformation capacity is not fully mobilized, resulting in limited energy dissipation capacity. Therefore, the seismic design of low piers is essentially governed by internal force bearing capacity rather than deformation capacity.

Regarding the response characteristics of internal forces and displacement, short piers demonstrate a pronounced pattern of “high internal forces and small displacement.” The bending moment at the base of a short pier attains 731,000 kN·m, which is approximately 2.9 times that of an 80 m high pier; the base shear force reaches 37,000 kN, approximately 2.6 times that of an 80 m high pier. Concurrently, the displacement at the pier top is extremely small (under a peak ground acceleration of 0.3 g, the longitudinal displacement is only 0.03 m). The fundamental reason for this phenomenon lies in the excessive stiffness of low piers. Their natural vibration period is short (the fundamental period of a 20 m pier is only 1.02 s), approaching the predominant period of ground motion (0.1 s–0.6 s), leading to significant amplification of seismic inertial forces. Simultaneously, the structural deformation is primarily shear-type, and the plastic deformation capacity is not fully utilized, resulting in weak energy dissipation capacity. Therefore, the seismic design of low piers is essentially governed by internal force bearing capacity, rather than deformation capacity.

From the reinforcement perspective, even under an E1 earthquake, the minimum longitudinal reinforcement ratio required for a 20 m pier to maintain an elastic working state is higher than the demand for 50 m and 80 m piers under an E2 earthquake. More importantly, under this pier height condition, the shear demand cannot be met by simply increasing the stirrup ratio. This indicates that for low piers, merely increasing the reinforcement ratio is not an effective solution to seismic problems. Instead, measures at the system level should be adopted, such as enlarging the cross-sectional dimensions, using seismic isolation bearings, or adjusting the structural system.

In contrast, the 80 m high pier exhibits typical flexible characteristics. The fundamental period of the structure extends to 3.38 s, effectively avoided the excellent period of seismic motion, thereby significantly reducing the input of seismic inertial forces. Both the bending moment and shear force at the pier base are much smaller than those of the low pier, but the displacement at the pier top is significantly larger.

It is worth noting that under a peak ground acceleration of 0.3 g, the transverse displacement of the 80 m movable pier is relatively small at 0.02 m; however, its displacement limit is only 0.04 m, resulting in a safety margin of merely 50%. This is the most stringent condition among all considered cases. This indicates that the seismic design of high piers shifts from a strength issue to a displacement control problem, and may further evolve into a stability control issue. As the pier height continues to increase, the structure may face challenges such as intensified P- Δ effects, significant contributions from higher modes, and an increased risk of girder unseating.

From the reinforcement perspective, the longitudinal reinforcement ratio for high piers can be governed by detailing requirements (a minimum of 0.6% meets seismic demands), but the stirrup ratio increases significantly with seismic intensity. This indicates that the core of seismic design for high piers lies in ensuring concrete ductility through adequate confinement reinforcement, guaranteeing sufficient deformation capacity in the plastic hinge region, rather than resisting larger internal forces by increasing longitudinal reinforcement.

5.2. Phenomenon and Mechanism of Critical Pier Height

The analysis reveals that the 50 m high pier exhibits “critical” characteristics across multiple response indices, indicating that seismic responses such as bending moment and pier-top displacement approach their peak values at this pier height.

The transverse bending moment results indicate that under a 0.3 g earthquake action, the moment of the 50 m high pier (2.826×10^5 kN·m) is slightly lower than that of the 80 m high pier (2.865×10^5 kN·m) but significantly higher than that of the 20 m high pier (1.322×10^5 kN·m). This suggests that the transverse bending moment increases sharply with pier height up to 50 m, beyond which the increase becomes marginal. A more notable trend is revealed by the proportion of transverse-to-longitudinal bending moments: for the 20 m high pier, this ratio is only 14.7%; for the 50 m pier, it rises to 65.2%; and for the 80 m pier, it reaches 73.7%. This indicates that as pier height increases, the seismic threat in the transverse direction becomes increasingly significant, accounting for a considerable proportion of the total demand for high piers.

The pier-top displacement results indicate that both the longitudinal displacement (0.18 m) and transverse displacement (0.03 m) of the 50 m high pier are greater than those of the 80 m high pier (0.14 m and 0.02 m, respectively), suggesting the existence of a critical pier height at which the displacement response reaches its maximum.

This phenomenon can be explained from the perspective of structural dynamics. As the pier height increases from 20 m to 50 m, the structural stiffness decreases significantly and the natural vibration period prolongs, which reduces the seismic inertial forces. However, the deformation capacity increases concurrently, leading to an upward trend in displacement response. When the pier height continues to increase to 80 m, higher-order vibration modes—particularly the bending and torsional modes of the pier shaft itself—begin to dominate the response. These modes have low mass participation efficiency but may alter the phase relationships of the displacement response, resulting in a non-monotonic increase in pier-top displacement. In addition, the redistribution of mass along the pier shaft and the relative changes in boundary conditions may also contribute to this turning point.

Therefore, during the bridge conceptual design stage, caution should be exercised regarding the possible response peak near the “critical pier height” to avoid selecting a pier height that falls within the most dynamically unfavorable range. For medium-to-high piers, particular attention should be paid to the transverse seismic design, and additional seismic mitigation measures or optimized pier cross-sectional configurations should be considered where necessary.

5.3. Comparison with Existing Studies

5.3.1. Relationship Between Reinforcement Ratio and Ductility

Increasing the stirrup ratio can transition the failure mode of a specimen from brittle shear failure to ductile flexural failure, effectively enhancing the seismic ductility and shear capacity of bridge piers. Miao Jianbao et al. found through quasi-static tests that ductile bridge piers with longitudinal reinforcement ratios in the range of 1.11–2.72% can achieve substantial displacement deformation while minimizing damage indices [21]. Our analysis of the 50 m pier shows that when the peak ground acceleration increases from 0.1 g to 0.3 g, the required longitudinal reinforcement ratio increases from 0.8% to 1.5%, an increase of approximately 0.7%, still below the saturation threshold of 2.72%. This suggests that for medium–high piers around 50 m, within seismic intensity ranges of 0.3 g and below, increasing the longitudinal reinforcement ratio can still effectively enhance seismic performance, leaving room for further optimization.

5.3.2. Detailing-Controlled Characteristics of High Pier Reinforcement

As noted by some researchers, the seismic demand of high piers frequently does not control the reinforcement design; rather, detailing requirements may play a dominant role [36–39]. Our analysis of the 80 m pier aligns highly with this view. For the 80 m pier, under peak ground accelerations of 0.1 g–0.3 g, the minimum required longitudinal reinforcement ratios are all below the minimum detailing requirement of 0.6% specified in the code. This finding further confirms that for very high piers, the key to seismic design is not how much reinforcement to use, but how to arrange confining reinforcement to ensure ductility and how to control pier top displacement and stability.

5.3.3. Transition of Seismic Controlling Factors with Pier Height

Studies by Gao Zeliang et al. and Wang Yingfa et al. respectively from the perspectives of fragility analysis and engineering design, emphasized the importance of bearing damage risk and displacement control in high-intensity regions [19,20]. Our research further reveals a continuous spectrum: as pier height increases, the seismic controlling factor transitions from internal force control (for pier heights of 20 m and below) to displacement control (around 50 m) and further to stability control (for pier heights of 80 m and above). This understanding helps establish a more refined, height-dependent design strategy.

5.4. Implications for Engineering Practice

Based on pier height ranges, the following seismic design recommendations are proposed. For low piers (approximately 20 m in height), priority should be given to enlarging cross-sectional dimensions or using seismic isolation bearings. Solely increasing the reinforcement ratio is not recommended, as it is insufficient to meet the seismic demand under strong earthquakes. If necessary, consider adjusting the structural system (for example, adopting a rigid-frame system) to improve mechanical performance. For medium piers (a pier with a height of approximately 50 m): Special attention should be paid to the transverse seismic threat, as the transverse bending moment at this height may approach or even exceed the longitudinal level. Attention should also be given to the peak displacement response; enhance the deformation capacity at the pier top, and if necessary, increase the deformation allowance of expansion bearings. For high piers (pier height of 80 m and above): It is recommended to prioritize the use of a rigid-frame system to improve overall stability. Seismic design should focus on checking pier top displacement and its safety margin, and assessing P- Δ effects, contributions from higher modes, and potential instability risks.

Regarding reinforcement design: For low piers: Seismic demand governs the reinforcement ratio, but increasing the ratio is not the optimal solution. It is recommended to reduce seismic demand through systemic measures (isolation, section enlargement) rather than passively increasing bearing capacity. For medium–high piers: The longitudinal reinforcement ratio can be selected within the range of 0.8–1.5% based on seismic demand, but the stirrup ratio needs to be significantly increased with higher seismic intensity. Under 0.3 g conditions, the required stirrup ratio is 1.4% for a 50 m pier and 2.0% for an 80 m pier; this should be given sufficient attention in design. For very high piers: The longitudinal reinforcement ratio can be governed by detailing requirements ($\geq 0.6\%$), but stirrup configuration must meet ductility demands. It is recommended to intensify stirrups in the plastic hinge region at the pier base, considering the distribution of bending moment along the pier.

It should be noted that this study is primarily based on response spectrum analysis, aiming to provide design guidance on reinforcement ratios for thin-walled piers of varying heights under different seismic intensities. Although an effective stiffness reduction factor was adopted under the E2 earthquake level to account for stiffness degradation

caused by concrete cracking in the plastic hinge region, this method remains limited in capturing the influence of geometric nonlinearities on structural seismic responses. To address this limitation, nonlinear time-history analyses will be conducted in the next phase of this research to more accurately account for P- Δ effects, higher-mode contributions, and local buckling, which constitutes a key focus of our future work.

6. Conclusions

In this study, a beam-element finite element model of a three-span continuous girder bridge is established using Midas Civil, with three pier heights of 20 m, 50 m, and 80 m considered. Response spectrum analyses are conducted under four peak ground acceleration levels of 0.1 g, 0.15 g, 0.2 g, and 0.3 g. The seismic responses of thin-walled piers with different pier heights, the effects of reinforcement ratios on seismic performance, and the required longitudinal and stirrup reinforcement ratios are systematically investigated. The differences in seismic performance and reinforcement control mechanisms between low and high piers are revealed. Furthermore, by introducing the dimensionless index R_i , the calculated minimum reinforcement ratio requirements are distilled into a concise discriminant criterion, which provides a reference for designers in configuring reinforcements and shortens the design process.

(1) High-pier continuous girder bridges exhibit pronounced flexibility. The fundamental period significantly prolongs with increasing pier height, which is beneficial for avoiding the predominant period of earthquake ground motions. However, higher-order modes contribute considerably to the dynamic response of high piers. Therefore, a sufficient number of modes must be extracted in modal analysis to ensure computational accuracy.

(2) Pier height plays a decisive role in the seismic internal force response of the bridge. Short piers, due to their excessive stiffness, experience substantially higher internal forces than tall piers and thus represent the critical component in seismic design. For medium-high piers, the seismic threat in the transverse direction increases with pier height, approaching the level of the longitudinal direction; thus, equal attention should be paid to both directions in design.

(3) The pier top displacement first increases and then decreases with increasing pier height, indicating the existence of a critical pier height at which the displacement response peaks. Displacements in all cases satisfy the code limits; however, the transverse displacement safety margin for tall piers is relatively small. In seismic design, the displacement magnitude and its redundancy for tall piers should be carefully verified.

(4) The reinforcement control mechanisms differ fundamentally with pier height. For short piers, the reinforcement ratio is governed by seismic demand, and simply increasing the stirrup ratio cannot satisfy the shear capacity requirement. Therefore, measures such as enlarging the cross-section or adopting seismic isolation are necessary. For tall piers, the longitudinal reinforcement ratio can be controlled by constructional requirements; however, the stirrup reinforcement ratio increases significantly with higher seismic intensity and deserves sufficient attention in design.

(5) To facilitate broader application of the research findings in engineering practice, the dimensionless index R_i is introduced to distill the calculated minimum reinforcement ratio requirements into a concise discriminant criterion. Designers can quickly determine whether the reinforcement of a given pier is governed by seismic demand or by detailing requirements by simply calculating the height-to-width ratio in the relevant direction to identify the corresponding R_i value. Subsequently, reinforcement design can be carried out in accordance with the above findings, thereby shortening the trial-and-error process in reinforcement design.

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References

1. Li, X.Z.; Kong, J.M.; Deng, H.Y.; Cui, Y.; Tian, S.; Afa, Y. Analysis on characteristics and deformation failure mode of large-scale landslides induced by “5.12” Wenchuan Earthquake. *J. Sichuan Univ. (Eng. Sci. Ed.)* **2009**, *41*, 72–77.
2. Liu, Y.; Wang, X.; Hu, Y.; Xie, C. How do natural disasters affect agricultural exports? *J. Integr. Agric.* **2026**, *25*, 2148–2166.
3. Wu, Q.; Xu, Y.; Cheng, G.X. Review of soil liquefaction manifestation and its consequences for 11 earthquakes in China over the Last 20 years. *J. Disaster Prev. Mitig. Eng.* **2022**, *42*, 961–975.
4. Zhao, T.Y. Research on Seismic Resilience Assessment and Enhancement Decision Optimization of the Urban Road-Bridge System. Ph.D. Thesis, Southeast University, Nanjing, China, 2023.
5. Jiang, R. Seismic Design and Performance Research of R.C Frame Under Fortification and Major Earthquakes. Ph.D. Thesis, Tongji University, Shanghai, China, 2008.
6. Li, X.Q. Seismic Behavior and Design Method of Concrete Filled Precast Concrete Tubular Column to Steel Beam Mixed Frames. Ph.D. Thesis, Tianjin University, Tianjin, China, 2023.
7. Pham, T.M.; Hyde, H.; Kaung, M.K.; Zhuge, Y.; Tran, D.T.; Vlietstra, D.; Tran, T.M. Comparative impact behaviours of ultra high performance concrete columns reinforced with polypropylene vs. steel fibres. *Def. Technol.* **2024**, *40*, 138–153.
8. Su, S.B.; Hu, M.H.; Jia, X.Z.; Zhang, G.; Xu, S.; Wang, M.; Li, Y.; Han, Q.; Du, X. Seismic performance evaluation of a novel joint system for prefabricated high-strength RC centrifugal pipe column bridges. *China J. Highw. Transp.* **2026**, 1–18.
9. Haseli, B.; Taromi, M.M.; Nouri, G.; Sadeghi, N.; Nowruzianostad, M.; Adili, E. Effect of incident angle of seismic waves on seismic behavior of irregular bridges. *Earthq. Eng. Eng. Vib.* **2026**, *25*, 315–332.
10. Sun, Z.Y.; Guo, Y.T.; Ge, K.; Hou, C.; Hu, Z.-Z. ROM-PINN: A physics-informed neural network with reduced-order modelling for nonlinear structural response prediction. *Structures* **2026**, *86*, 111350.
11. Yan, L.; Cao, L.; Yang, K. Seismic fragility analysis of self-anchored suspension bridge based on incremental dynamic analysis. *J. Chongqing Jiaotong Univ. (Nat. Sci.)* **2019**, *38*, 41–45.
12. Chen, Y.H.; Chu, L.S.; Feng, H.; Cui, W.; Wang, Y.; Zhang, C. The influence of socket depth on the seismic behavior of precast hollow bridge pier. *Structures* **2024**, *69*, 107588.
13. Nie, L.X. Study on Seismic Isolation and Post-Earthquake Running Performance of Railway Tall Pier Simply Supported Bridges Using Rocking Isolation. Ph.D. Thesis, Central South University, Changsha, China, 2025.
14. Liu, X.; Chen, Y.L.; Jiang, L.Z.; Lai, Z.; Pang, L. Deformation patterns and fastener failure criteria of CRTS-III slab-simply supported girder bridges under lateral seismic action. *J. Traffic Transp. Eng.* **2026**, 1–15. <https://doi.org/10.19818/j.cnki.1671-1637.2026.271>.
15. Editorial Department of China Journal of High way and Transport. Review on China’s bridge engineering research·2024. *China J. Highw. Transp.* **2024**, *37*, 1–160.
16. Jiang, S.; Ma, R.; Bi, K.; Du, X.; Song, J. Negative stiffness enhanced tuned mass damper (NS-TMD) for seismic induced response mitigation of isolated bridges. *Eng. Struct.* **2025**, *325*, 119416.
17. Fan, X.; Xu, L.Q.; Sun, Y.H. Seismic response features and anti-seismic countermeasures of horizontally curved girder bridge. *Earthq. Eng. Eng. Dyn.* **2021**, *41*, 177–187.

18. Xiao, X.H. Damage mechanism of simply supported beam bridges pier and girder in wenchuan earthquake. *J. Chongqing Jiaotong Univ. (Nat. Sci.)* **2014**, *33*, 15–20.
19. Gao, Z.L.; Wang, X.M.; Wang, B.Y.; Shi, Y.; Zhang, H.; Wu, Q. Improving seismic performance of in-service continuous beam bridges considering traveling wave effect. *J. Jilin Univ. (Eng. Technol. Ed.)* **2025**, 1–14. <https://doi.org/10.13229/j.cnki.jdxbgxb.20250162>.
20. Wang, Y.F.; Liu, C.F.; He, N. Analysis of ground motion parameters and seismic design of bridge engineering in high seismic intensity area. *Build. Struct.* **2023**, *53*, 758–762.
21. Miao, J.B.; Feng, W.; Zhao, Y. Range of reinforcement ratio of piers of medium and small span bridges in earthquake area. *Sci. Technol. Eng.* **2023**, *23*, 3937–3949.
22. Wilson, T.; Chen, S.; Mahmoud, H. Analytical case study on the seismic performance of a curved and skewed reinforced concrete bridge under vertical ground motion. *Eng. Struct.* **2015**, *100*, 128–136.
23. Mander, J.B.; Priestley, M.J.N.; Park, R. Theoretical stress-strain model for confined concrete. *J. Struct. Eng.* **1988**, *114*–*118*, 1804–1826.
24. Lu, J.H.; Chen, X.C.; Ding, M.B.; Zhang, X.; Liu, Z.; Yuan, H. Experimental and numerical investigation of the seismic performance of railway piers with increasing longitudinal steel in plastic hinge area. *Earthq. Struct.* **2019**, *17*, 545–556.
25. Bouazza, H.; Djelil, M.; Matallah, M. Comparative numerical study of reinforcement buckling in square vs. circular RC bridge piers under seismic loading. *J. Mater. Eng. Struct.* **2026**, *13*, 225–231.
26. Mucci, D.; Mario, V.; Cardellicchio, A.; Ruggieri, S.; Nettis, A.; Renò, V.; Uva, G. Computer vision-based seismic assessment of RC simply supported bridges characterized by corroded circular piers. *Bull. Earthq. Eng.* **2025**, *23*, 6771–6800.
27. Su, J.S.; Lou, C.X.; Han, Q.; Xiang, N.; Li, Z.; Li, Z. Seismic retrofit of highway bridges: State of the art review. *China J. Highw. Transp.* **2026**, *39*, 333–357.
28. Cui, H.; Shao, C. 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.
29. Gao, H.; Li, J.; Chen, X.; Du, X.; Zhong, X. Seismic performance of longitudinal shear keys in preventing girder unseating in highway bridges. *Soil Dyn. Earthq. Eng.* **2026**, *206*, 110245.
30. Xu, Y.; Hao, X.W.; Zhang, L.F. Study on parameter design method of double thin-walled piers for continuous rigid frame bridge. *China J. Highw. Transp.* **2002**, 82–85. <https://doi.org/10.19721/j.cnki.1001-7372.2002.02.020>.
31. Wang, H.L.; Chen, C.; Huang, S.Q.; Ding, Y. Seismic performance analysis of continuous rigid-frame bridges with twin-column thin-walled piers based on seismic vulnerability. *J. China Three Gorges Univ. (Nat. Sci.)* **2026**, *48*, 71–78.
32. Wang, X.; Li, W.; Liu, Z. Experimental investigation and fragility analysis on seismic performance of precast UHPC hollow piers under varying axial load. *Soil Dyn. Earthq. Eng.* **2025**, *196*, 109402.
33. Zong, Z.H.; Xia, J.; Xu, C.R. Seismic study of high piers of large-span bridges: An overview and research development. *J. South-east Univ. (Nat. Sci. Ed.)* **2013**, *43*, 445–452.
34. JTG/T 2231-01—2020; Specifications for Seismic Design of Highway Bridges. China Communications Press, Beijing, China, 2020.
35. GB 18306—2015; Seismic Ground Motion Parameters Zonation Map of China. China Standard Press: Beijing, China, 2015.
36. Editorial Department of China Journal of Highway and Transport. Review on China's bridge engineering research: 2014. *China J. Highw. Transp.* **2014**, *27*, 1–96.
37. Li, S.S. Friction Analysis and Rapid Seismic Response Assessment of High-Speed Railway Track-Bridge Systems. Ph.D. Thesis, Central South University, Changsha, China, 2024.
38. Li, T.H. Seismic Fragility and Seismic Resilience Analysis of Simply Supported Beam Bridges Under Environmental Erosion Conditions. Ph.D. Thesis, Institute of Engineering Mechanics, China Earthquake Administration, Harbin, China, 2023.
39. Xia, X.S.; Wu, S.W.; Shi, J.; Jia, J.; Chen, X.; Ma, H. Seismic response of rocking isolated railway bridge piers with sacrificial components. *Earthq. Eng. Eng. Vib.* **2020**, *19*, 1005–1015.

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