Research Article

Influence of Restrainer Piers on the Seismic Performance of Long Bridges with Equal-Height Piers

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1. Introduction

From a global perspective, China is in a seismically active zone and faces serious earthquake threats [1,2]. For example, in the 2008 Wenchuan earthquake, about 1,400 bridges were in a high-intensity area. Among them, the Baihua Bridge underwent unseating damage due to excessive displacement of the superstructure. Furthermore, many bridges were damaged by the earthquake in Lushan County, Ya’an City, located in Sichuan Province, in 2013. The probability of unseating damage for short- to medium-span bridges in earthquakes is very high [2]. Consequently, transportation systems and modern communication infrastructures may be severely impaired. Hence, an analysis of seismic isolation and pounding during earthquakes is of paramount significance [3–5].

Pounding between bridge decks with insufficient separation distances may result in significant structural damage or even beam unseating, which has been identified as one of the primary causes of bridge damage in many major earthquakes. Li et al. experimentally investigated the influence of spatial variation of ground motions on the pounding behavior of three adjacent bridge segments using three shake tables. The results indicated that the spatially nonuniform ground motions increased the pounding forces [6]. Because a simplified model failed to evaluate the pounding responses, a longitudinal pounding analysis of an isolated continuous beam bridge was carried out under unidirectional ground motions, based on a multiscale simulation scheme [7]. Wei et al. and He and Zhao investigated the seismic performance of bridges based on seismic blocks and pins [8, 9] and found that the pounding between linear bridge structures should be between rigid body pounding and straight bar coaxial pounding. Moreover, the reasonable pounding contact stiffness should be 0.5 times that of the axial stiffness of the shorter main beam [10]. Zhu et al. [11] proposed a 3D contact-friction model to simulate an arbitrary pounding between bridges, combining the advantages of the recovery coefficient method [12–14] and the contact element method [15–17]. The experimental
observations demonstrated that this pounding model can be used to simulate poundings in multiple directions. Li et al. [18] introduced an equivalent Kelvin pounding model, in which the authors presented a parameter determination method for pounding analysis of straight-line bridges. Their research suggested that the range of pounding stiffness for the seismic pounding response analysis of urban bridges was 3E+5 kN/m-6E+5 kN/m. By setting up restrainer piers in a key part of the bridge, the rigidity and strength of the key piers were enhanced, and the overall seismic performance of the bridge was increased accordingly. This was because the excessive displacement of the piers during the earthquake due to insufficient rigidity and strength was avoided [19, 20]. Taking a 32 m simply supported girder bridge as a research object, it was found that the pounding between the shear keys and the bearings restrained the transverse relative displacement between the superstructure and the substructure [21]. The most efficient way to avoid pounding between adjacent structures is to provide sufficient separation distances. However, for bridge structures with conventional expansion joints, complete avoidance of pounding during strong earthquakes is often impossible. Bi et al. proposed a new modular expansion joint (MEJ) and investigated the minimum total gap of a MEJ to avoid pounding at the abutments and between bridge decks [22, 23]. Pounding may be the primary cause of beam unseating. In this study, a 3D contact-friction model was used to simulate bridge pounding.

To dissipate earthquake energy and to decrease damage to the bridge structure, seismic isolation methods have been applied. Seismic isolation is a design strategy to isolate a structure from the ground and thereby protect it from the damaging effects of earthquake motions [24,25]. Moreover, the performance of bridges isolated with shape memory alloy- (SMA-) based FPS bearings at low temperatures was scrutinized. The results showed that the seismic performance could be improved without significantly increasing the base forces at low temperatures [26]. The bridge model supported by four spherically shaped sliding bearings (known as the friction pendulum system or FPS bearings) was tested on a shake table with seismic motions. The experimental results demonstrated a substantial improvement in the ability of the isolated bridge to sustain all levels of seismic excitation under elastic conditions [27]. Seismic performances of bridges installed with a conventional lead rubber bearing (LRB) system and sliding-LRBs were investigated under near-fault excitations [28,29]. By combining SMA wires with sliding-LRBs, thus forming a superelastic-sliding-LRB isolation system, different studies have indicated that the displacement responses can be effectively mitigated [30]. The mechanical parameters of the LRB and viscous damper have been analyzed and optimized and are known to have a positive impact on seismic isolation [31–33]. Xia and Chen [34] proposed a simulation method of restrainer devices and improved the analysis model of high-pier seismic isolation. Several innovative types of piers have been proposed to improve seismic performance, such as rectangular-hollow double-column tall piers with energy dissipation beams, new base rocking isolation bridge piers, and double-deck bridge piers based on a rocking self-centering system [35–37]. Guo et al. [38] presented a new type of multilevel spring restrainer (MLSR) to diminish the seismic responses of a curved bridge and show that the MLSR has high effectiveness in mitigating pier damage to the bridge. Furthermore, other researchers proposed several new isolation systems such as an SMA-based isolation system [39] and a wedge-shaped block [40].

The aim of this research was to study the influence of restrainer piers, EJSs, and the span on the seismic performance of long bridges. A 3D contact-friction model was utilized to simulate bridge pounding using the ABAQUS finite element software. The El-Centro, Northbridge, and Taft seismic waves were selected for the analysis of the dynamic response of long bridges. To reduce the negative effect of pounding and to decrease the relative displacement of pier-beams, different EJSs (30, 50, 70, and 90 mm) and different spans (20, 30, 40, and 50 m) were investigated. The results indicated that the relative displacement of pier-beams could be reduced by increasing the structural stiffness, and the pounding between the pier and beam could likely be avoided. Here, restrainer piers were designed and simulated to increase the structural stiffness and to lessen the relative displacement of the pier-beam under seismic action. Finally, it was concluded that the relative displacement of pier-beams could be significantly decreased when there were three restrainer piers, which could be a falling-off prevention measure.

2. Analytical Model and Algorithm

2.1. Analytical Model. When analyzing the influence of the number of restrainer piers and EJSs on the dynamic response, the span of the bridge was set to 30 m. The bridge consisted of six units, and each unit had four spans. The cross section of the main girder was T-shaped. Figure 1 shows the size parameters of the main girder.

The height of the cap girder was 1.6 m, and its width was 2 m. The length of the transverse cap and main girder was 12.5 m. The bridge adopted a double cylindrical pier, and its height was 10 m. The cross radius of the pier was 0.8 m, and the transverse distance was 7 m. The tie beam size was 1.0 m × 1.3 m. The restrainer pier size was 5.0 m × 4.0 m. The overlapping length of the main girder and restrainer was 1.535 and 1.035 m, respectively. The direction of the falling beam was in the direction of the main girder, deviating from the bearing centerline. The positive direction of the falling beam is shown in Figure 2(a). The bridge was modeled by beam elements except for the cap beam and upper area, which were modeled using solid elements. The local drawing of the restrainer pier is presented in Figure 2(b), and its layout is presented in Table 1.

2.2. 3D Contact-Friction Model. To study the arbitrary contact and pounding between the main girders, Zhu et al. [11] proposed a 3D contact-friction model. The pounding surface contact can be divided into normal and tangential contact methods. The model is based on the point-surface
contact theory, and the boundary interference fit was considered (Figure 3).

The contact surface on the end of the beam on the pounding side is defined as surface \( a-b-c-d \), and the contact point is defined as point \( p \). During the pounding, point \( p \) contacts surface \( a-b-c-d \) and invades point \( k \). The relative displacement between point \( k \) and \( p \) is \( \Delta_k \), and their relative velocity is \( V_{kp} \). They are related using

\[
\Delta_k = X_p - X_k, \\
V_{kp} = V_p - V_k, \tag{1}
\]

where \( X_p \) and \( X_k \) are the displacement of point \( p \) and \( k \), respectively, and \( V_p \) and \( V_k \) are the velocity of point \( p \) and \( k \), respectively.

The contact force between point \( p \) and \( k \) is expressed by the contact stiffness \( K_{cnt} \), and the force is calculated using

\[
F_k = K_{cnt} \cdot \Delta_k, \tag{2}
\]

where the contact force \( F_k \) can be decomposed into a normal component \( F_{kn} \) and a tangential component \( F_{kt} \). The contact states are divided into a sliding and nonsliding state (Figure 3(b)), which are judged using

\[
\text{adhesive contact } F_{kt} < \mu_s F_{kn}, \tag{3}
\]

\[
\text{sliding contact } F_{kt} \geq \mu_s F_{kn},
\]

where \( \mu_s \) is the coefficient of static friction.

The energy loss between the contact point \( p \) and the intrusion point \( k \) is simulated by the normal damping and tangential damping of the contact surface. The damping coefficients are \( C_n \) and \( C_t \), and the damping forces are \( F_{cn} \) and \( F_{ct} \), respectively. Their relationship is expressed through

\[
F_{cn} = -C_n V_{kpn}, \tag{4}
F_{ct} = -C_t V_{kpt}.
\]

The pounding force can be expressed using
adhesive contact $R_k = F_k + F_{cn} + F_{ct}$,  
sliding contact $R_k = F_{kn} + F_{cn} + F_{ft}$,  
\begin{equation}
\text{(5)}
\end{equation}
where $F_{ft}$ is the dynamic friction force, which is defined by $F_{ft} = -\mu_k F_{kn}$.

2.3. Explicit Contact Algorithm. The contact algorithm or
contact pair algorithm can be employed in contact simula-
tion in ABAQUS/Explicit. To define a contact simulation,
we usually need to specify only the contact algorithm and
the contact surface. There are some general contact algorithms,
including the contact pair algorithm, slip formula, penalty
function contact formula, and dynamic contact formula.
Pounding between the abutments and the girder or adjacent
girder was considered through the surface-to-surface con-
tact in ABAQUS. The problem of seismic pounding in which
the pounding point cannot be determined in advance can be
solved by an explicit contact algorithm. The interaction
between two contact surfaces includes two components of
normal contact force and tangential contact force. In this
research, the hard contact behavior was set for normal
contact behavior [7].

The iterative process is displayed in Figure 4.

2.4. Selection of Seismic Waves. The ground motion was
regarded as a random process, and the uncertainty of the
ground motion input in the seismic time-history analysis
inevitably results in uncertainty regarding the structural
response. Analysis of historical ground motion is one of the
most effective methods to understand the real response of
structures in an earthquake. In analyses of seismic perform-
ance, two types of ground motion inputs are generally
adopted (i.e., seismic acceleration response spectrum and
seismic time history). In the seismic design of bridges, there
are three methods to select the seismic time history, namely,
using the acceleration time-history record, adopting the
artificial acceleration time history, and selecting the standard
acceleration time history. The seismic waves are usually
simulated as random processes with prescribed spectral
patterns [41–43]. The selection of seismic waves generally
follows the principle of selecting representative strong
earthquake records. Tianjin, El-Centro, Northridge, and Taft
waves are commonly utilized to analyze the seismic per-
formance of bridges. According to the above analysis, the
representative El-Centro, Northbridge, and Taft waves were
chosen based on the site characteristics of bridges in the
high-intensity areas in central-western China. The peak
ground acceleration (PGA) of selected seismic waves was
adjusted to 0.4, 0.7, and 1.0 g.

3. Influence of the Number of Restrainer
Piers on the Dynamic Response

3.1. Influence of the Number of Restrainer Piers on
the Dynamic Response under the El-Centro Seismic Wave

3.1.1. Dynamic Response under the El-Centro Seismic Wave.
The maximum relative displacement of the pier-beam and its
percentage reduction are plotted in Figure 5. Under the three
different PGAs, the maximum relative displacement of the
pier-beam decreased as the number of restrainer piers in-
creased. Meanwhile, the percentage reduction in relative
displacement increased with an increase in the number of
restrainer piers. When the PGA was 0.4 g and there were
three restrainer piers, the maximum reduction in the relative
displacement occurred, which was 61.67% (Figure 5(b)).
This was mainly because the restrainer pier increased the
structural stiffness. Also, the higher the number of restrainer
piers, the greater the structural stiffness and the smaller the
maximum relative displacement of the pier-beam. These
findings show that setting up a restrainer pier could reduce
the relative displacement between the pier and beam. Hence,
it could be a falling-off prevention measure.

Figure 6 depicts the internal force of the ordinary and the
restrainer piers. Figures 6(a) to 6(c) illustrate the internal
force of the ordinary pier, and the other figure panels il-
illustrate the internal force of the restrainer pier, where $M_O$, $F_{SO}$, and $F_{NO}$ represent the maximum pier bottom moment,
maximum pier bottom shear, and maximum pier bottom
axial force of the ordinary pier, respectively. $M_R$, $F_{SR}$ and $F_{NR}$ represent the maximum pier bottom moment,
maximum pier bottom shear, and maximum pier bottom axial force of the restrainer pier, respectively.

When the PGA was 0.4g and there were three restrainer piers, the maximum pier bottom moment of the restrainer pier was greater than that of the corresponding ordinary pier (Figures 6(a) and 6(d)). When the PGA was 0.7 or 1.0g and only one restrainer pier was set up, the maximum pier bottom moment of the ordinary pier was at its minimum. Moreover, under a PGA of 0.7 or 1.0g, the maximum pier bottom moment of the restrainer pier gradually increased as the number of restrainer piers increased. The maximum pier bottom shear of the restrainer pier was larger than that of the corresponding ordinary pier (Figures 6(b) and 6(e)). When the PGA was 0.4g, the shear of the ordinary pier gradually
decreased with an increase in the number of restrainer piers, while the shear tended to increase when the PGA was 0.7 and 1.0 g. Moreover, the shear of the restrainer pier gradually increased with an increase in the number of restrainer piers. The axial force at the bottom of the restrainer pier was larger than the corresponding ordinary pier, especially when the PGA was 1.0 g (Figures 6(c) and 6(f)). The axial force of the ordinary and restrainer piers gradually increased as the number of restrainer piers increased. Finally, when the number of restrainer piers was three, the axial force of both the ordinary and restrainer piers reached their maximum value.

3.1.2. Displacement Response When the El-Centro Seismic Wave PGA Was 0.4 g. As the number of restrainer piers increased, the maximum relative displacement of the pier-beam gradually decreased (Figure 7), and the maximum relative displacement occurred at the abutment. When there were three restrainer piers, displacement was at its minimum; thus, the effect of reducing the relative displacement was the most effective.

3.1.3. Displacement Response When the El-Centro Seismic Wave PGA Was 0.7 g. On the left and right sides of the restrainer pier, because of the pounding between the beam and retaining wall, the maximum relative displacement of the pier-beam on the left and right sides near the restrainer pier increased. As the number of restrainer piers increased, the relative displacement at the abutment gradually decreased. It could be observed that the relative displacement of the pier-beam gradually decreased as the number of restrainers piers increased (Figures 8(a) and 8(b)). Finally, when three restrainer piers were set up, the relative displacement of the pier-beam was at its minimum.

3.1.4. Displacement Response When the El-Centro Seismic Wave PGA Was 1.0 g. Figure 9 reveals the displacement response when the PGA was 1.0 g. As the number of restrainer piers increased, the relative displacement of the pier-beam gradually decreased. When there were no restrainer piers, the maximum relative displacement occurred at the abutment; however, when restrainer piers were set up, the maximum relative displacement occurred close to the restrainer pier. The relative displacement was obviously reduced after setting up a restrainer pier (Figure 9(b)), and the restrainer pier at the Nos. 12 and 20 piers had almost the same role in limiting the relative displacement of the pier-beam when two or three restrainer piers were set up. Finally, after setting up three restrainer piers, the relative displacement of the pier-beam was at its minimum.
3.2. Influence of the Number of Restrainer Piers on the Dynamic Response under the Taft Seismic Wave

3.2.1. Dynamic Response under the Taft Seismic Wave. The maximum relative displacement of the pier-beam and its percentage reduction are plotted in Figure 10. It can be observed that the maximum relative displacement of the pier-beam increased with an increase in PGA when the number of restrainer piers was the same. Specifically, the maximum displacement occurred when the PGA was 1.0g. Furthermore, the maximum relative displacement of the pier-beam decreased with an increase in the number of restrainer piers (Figure 10(a)). Figure 10(b) demonstrates that the more the number of restrainer piers, the greater the percentage reduction. When there were three restrainer piers and the PGA was 0.4g, the relative displacement of the pier-beam decreased significantly, and the maximum was 62.47%. The analysis above indicates that setting up a restrainer pier could reduce the relative displacement of the pier-beam under the Taft seismic wave. Hence, it could be a falling-off prevention measure.

Figure 11 depicts the internal force of the ordinary and the restrainer piers. Figures 11(a) to 11(c) illustrate the internal force of the ordinary pier, and the other figure panels illustrate the internal force of the restrainer pier. When the number of restrainer piers and PGA were the same, the maximum pier bottom moment of the restrainer pier was greater than that of the corresponding ordinary pier; moreover, the maximum pier bottom moment increased with an increase in PGA. The maximum pier bottom moment of the ordinary pier substantially increased when the number of restrainer pier increased from zero to one under a PGA of 0.7 and 1.0g, and when the number of restrainer pier was more than one, the maximum pier bottom moment varied little. Besides, when the PGA was 0.4g, the maximum pier bottom moment of the ordinary and the restrainer piers did not change much. Under a PGA of 0.7 and 1.0g, the maximum pier bottom moment of the restrainer pier increased with an increase in the number of restrainer piers. It may attribute that the increase in the number of restrainer piers aggravated the pounding between the
main girders and between the main girder and the restrainer pier, which eventually led to the increase of the maximum pier bottom moment of the restrainer pier (Figures 11(a) and 11(d)). When the number of restrainer piers was the same, the maximum pier bottom shear increased with an increase in PGA. When the PGA was 0.7 or 1.0 g, the maximum pier bottom shear of the ordinary and the restrainer piers increased with an increasing number of restrainer piers. Moreover, when the PGA was 0.4 g, the maximum pier bottom shear of the ordinary and restrainer piers changed little, and the maximum pier bottom shear of the ordinary pier got the minimum when the number of restrainer pier was three (Figures 11(b) and 11(e)). The axial force at the bottom of the restrainer pier was larger than that of the corresponding ordinary pier. The maximum axial force at the bottom of the pier increased with the PGA when the number of restrainer piers was the same. When the number of the restrainer pier was more than two, the maximum axial force at the bottom of the ordinary pier did not change much (Figure 11(c)). The maximum axial force at the bottom of the restrainer pier increased significantly with an increase in the number of restrainer piers, especially the PGA was 1.0 g (Figure 11(f)).

3.2.2. Displacement Response When the Taft Seismic Wave PGA Was 0.4 g. Figure 12 depicts the displacement response when the Taft seismic wave PGA was 0.4 g. As the number of restrainer piers increased, the maximum relative displacement of the pier-beam gradually decreased, and the maximum relative displacement occurred at the abutment. When there were three restrainer piers, the relative displacement of the pier-beam changed evenly and reached its minimum at most piers. Thus, the setting up of the restrainer pier could effectively reduce the relative displacement.

3.2.3. Displacement Response When the Taft Seismic Wave PGA Was 0.7 g. Figure 13 shows that the maximum relative displacement of the whole pier-beam was the smallest when there were three restrainer piers. Besides, the setting up of restrainer pier had a significant effect on the displacement of
the abutment. For the abutment of the first unit, when there was no restrainer pier, the relative displacement of the abutment was 0.52 m, and it was 0.29 m when setting up one restrainer pier, which reduced 0.23 m.

3.2.4. Displacement Response When the Taft Seismic Wave PGA Was 1.0 g. Figure 14 depicts the displacement response when the Taft seismic wave PGA was 1.0 g.

When three restrainer piers were set up, the relative displacement of the pier-beam clearly decreased, especially at the abutment. Besides, when there were two restrainer piers (Nos. 8 and 16), the relative displacement of the piers from Nos. 9 to 15 decreased significantly. It was mainly because two restrainer piers limited the displacement of main girders. When the number of the restrainer piers was three, the relative displacement of the pier-beam at most piers was at its minimum.

4. Influence of the EJS on the Dynamic Response

When the moving distance of two beams on the pier is greater than the EJS, the beams will pound at the expansion joint. The smaller the EJS, the greater the chance of pounding. Generally, the displacement of the beam is limited by restricting the pounding between the beam and retaining wall. Here, the EJS was considered to affect the relative displacement of the pier-beam. A single-column restrainer pier was set in the middle of the bridge. In this analysis, the EJS varied in size from 3 to 9 cm in increments of 2 cm. The El-Centro and Taft seismic waves were employed in this analysis. The number of restrainer piers was 0, 1, or 2, and the EJS of the ordinary pier was 7 cm.

4.1. Influence of EJS on the Dynamic Response under the El-Centro Seismic Wave

4.1.1. No Restrainer Piers. Figure 15(a) shows the relative displacement when the PGA was 0.7 g. The relative displacement of the pier-beam was maximum at the No. 0 abutment. With the increase of the EJS, the displacement of the No. 0 abutment increased, and the displacement of the whole bridge gradually decreased along the positive direction. The difference of relative displacement under any cases was small after pier No. 20. In addition, there was little difference between the relative displacement when EJSs of 70 or 90 mm were used.

The maximum internal force of the ordinary pier at the pier bottom when there was no restrainer pier is plotted in Figures 15(b) to 15(d). When the PGA was 0.4 g, the EJS had little impact on the maximum moment. When the PGA was 0.7 g, the maximum moment increased first and then decreased with the increase of the EJS, and when the EJS was greater than 70 mm, the maximum moment changed little (Figure 15(b)). When the PGA was 0.7 g, the EJS had little
effect on the maximum shear. When the PGA was 0.4 g, the maximum shear decreased initially, then increased with the increase of the EJS, and reached its minimum when the EJS was 50 mm (Figure 15(c)). The maximum axial force increased firstly and then decreased with the increase of the EJS. When the EJS was greater than 70 mm, the maximum axial force changed little, and the maximum axial force at the pier bottom reached its maximum value when EJS of 50 mm was used. In conclusion, a bridge with an EJS of 70 or 90 mm could obtain better seismic performance.

4.1.2. One Restrainer Pier. Compared with the case of when no restrainer pier was set up, when there was one restrainer pier, the relative displacement of the pier-beam along the positive direction varied widely. Overall, when the EJS was 70 mm, the relative displacement of the pier-beam was more stable (Figure 16(a)). The relative displacement of the pier-beam increased with an increase in the EJS, and when the PGA was 0.7 g, the relative displacement of the pier-beam was larger than that of the pier-beam with the PGA of 0.4 g (Figure 16(b)). The relative displacement of the pier-beam was reduced because of the restrainer pier. When the PGA was 0.4 and 0.7 g and the EJS was 50 and 70 mm, respectively, the relative displacement of the pier-beam decreased significantly, and the maximum was 40.33 and 46.91%, respectively.

Figure 17 depicts the internal force of the ordinary and the restrainer piers. Figures 17(a) to 17(c) illustrate the internal force of the ordinary pier, and the other figure panels illustrate the internal force of the restrainer pier. The maximum pier bottom moment of the ordinary pier gradually decreased with an increase in the EJS. When the PGA was 0.7 g, the maximum pier bottom moment greatly changed, and it reached its minimum when the EJS was 70 mm. However, the EJS had little impact on the maximum pier bottom moment when the PGA was 0.4 g (Figure 17(a)). Under a PGA of 0.4 or 0.7 g, the maximum pier bottom moment of the restrainer pier decreased with the increase of the EJS, and when the EJS was greater than 70 mm, the maximum pier bottom moment changed little (Figure 17(d)). When PGA and EJS were the same, the
maximum pier bottom shear of the restrainer pier was greater than that of the corresponding ordinary pier. The maximum pier bottom shear of the restrainer pier decreased with an increase in the EJS; moreover, the maximum pier bottom shear of the restrainer pier changed little when the EJS was greater than 70 mm (Figures 17(b) and 17(e)). The axial force at the pier bottom of the restrainer pier was greater than that of the ordinary pier when the PGA and the EJS were the same. With an increase in the EJS, the maximum axial force at the pier bottom of both piers varied little.

The axial force at the pier bottom of the restrainer pier reached its minimum value when the EJS was 70 mm (Figures 17(c) and 17(f)).

4.1.3. Two Restrainer Piers. The relative displacement of the pier-beam under different EJSs changed similarly along the positive direction (Figure 18(a)). When the PGA was 0.7 g, the relative displacement of the pier-beam decreased first and then increased as the EJS increased.
When the EJS was 70 mm, the relative displacement of the pier-beam was at its minimum. However, its relative displacement varied little when the PGA was 0.4 g (Figure 18(b)). Under a PGA of 0.4 and 0.7 g, the relative displacement of the pier-beam was reduced maximally (the maximum was 53.06 and 54.33%, respectively) when the EJS was 70 mm, compared with the case of no restrainer pier (Figure 18(c)).

Figure 19 depicts the internal force of the ordinary and restrainer piers. Figures 19(a) to 19(c) illustrate the internal force of the ordinary pier, and the other figure panels illustrate the internal force of the restrainer pier. The maximum pier bottom moment of both the ordinary and restrainer piers gradually decreased as the EJS increased (Figures 19(a) and 19(d)). When the PGA was 0.7 g, the maximum pier bottom shear of the ordinary pier decreased with the increase of the EJS and then varied little after 50 mm. When the PGA was 0.4 g, the EJS had little influence on the maximum pier bottom shear (Figure 19(b)). Under a PGA of 0.4 and 0.7 g, the maximum pier bottom shear of the restrainer pier changed little and reached its minimum value when the EJS was 70 mm (Figure 19(e)). The EJS had little impact on the axial force at the pier bottom of the ordinary pier in
When the PGA was 0.7 g, the axial force at the pier bottom of the ordinary decreased with the increase of the EJS (Figure 19(c)). When the PGA was 0.4 g, the maximum axial force at the pier bottom of the restrainer pier significantly reduced as the EJS increased and reached its minimum value when the EJS was 70 mm. When the PGA was 0.7 g and the EJS was greater than 50 mm, the maximum axial force at the bottom of the restrainer pier changed little (Figure 19(f)).

4.2. Influence of EJS on the Dynamic Response under the Taft Seismic Wave

4.2.1. No Restrainer Piers.

Figure 20(a) shows the relative displacement when the PGA was 0.7 g. The maximum relative displacement of the No. 0 abutment (Figure 20(a)). With an increase in the EJS, the displacement of the No. 0 abutment progressively increased. When the PGA was 0.4 g and the EJS was less than 50 mm, the relative displacement of the No. 0 abutment was significantly reduced. When the PGA was 0.7 g and the EJS was greater than 50 mm, the relative displacement of the No. 0 abutment changed little (Figure 20(b)).

Figure 18: (a) Relative displacement of the pier-beam, (b) maximum relative displacement of the pier-beam, and (c) percentage reduction in relative displacement.

Figure 19: Internal force at the pier bottom of the ordinary and restrainer piers with two restrainer piers.

4.2.1. No Restrainer Piers. Figure 20(a) shows the relative displacement when the PGA was 0.7 g. The maximum relative displacement of the pier-beam occurred at the No. 0 abutment (Figure 20(a)). With an increase in the EJS, the displacement of the No. 0 abutment progressively increased and the displacement of the whole bridge gradually
decreased along the positive direction. However, there was little difference between the relative displacement when EJSs of 70 or 90 mm were used. Also, in pier Nos. 16 to 24, the EJS had little impact on the displacement. An insufficient EJS had a great impact on pounding between the main girders. Small EJSs cause frequent poundings, which leads to local concrete spalling and bearing damage to the bridge. In conclusion, a bridge with an EJS of 70 or 90 mm could obtain better seismic performance.

The maximum internal force of the ordinary pier when there was no restrainer pier is plotted in Figures 20(b) to 20(d). With an increase in the EJS, the maximum moment and the maximum shear at the pier bottom increased initially and then decreased. When the EJS was greater than 70 mm, the maximum moment changed little. When the EJS was 50 or 70 mm, the maximum moment and the maximum shear force at the pier bottom reached their maximum values. However, when the PGA was 0.4 g and the EJS was 70 mm, both the maximum moment and the maximum axial force reached their minimum values.

4.2.2. One Restrainer Pier. When the EJS was 90 mm, the relative displacement of the pier-beam changed suddenly at the No. 12 restrainer pier when the PGA was 0.7 g (Figure 21(a)). When the PGA was 0.4 or 0.7 g, the maximum relative displacement of the pier-beam increased with an increase in the EJS and reached its maximum value at an EJS of 90 mm (Figure 21(b)). When the EJS was 50 and 70 mm, the PGA was 0.4 and 0.7 g, respectively; the percentage reduction of the maximum relative displacement of the pier-beam reached its maximum value of 49.15 and 32.71%, respectively (Figure 21(c)).

The internal force of the ordinary and the restrainer piers is shown in Figure 22 when there was one restrainer pier. The maximum pier bottom moment of the ordinary pier gradually decreased with an increase in the EJS when the PGA was 0.7 g, and it increased first and then decreased when the PGA was 0.4 g; it was at its maximum when the EJS was 50 mm (Figure 22(a)). Moreover, when the PGA was 0.4 g, the maximum pier bottom moment of the restrainer pier first increased and then decreased with an increase in the EJS; it was at its maximum when the EJS was 50 mm. Meanwhile, the maximum moment also reached its maximum value when the PGA was 0.7 g and the EJS was 50 mm (Figure 22(d)). From the shear curve illustrated in Figures 22(b) and 22(e), it can be seen that when the EJS was 70 mm, the maximum pier bottom shear of the ordinary pier reached its maximum value, while that of the restrainer pier reached its minimum value when the PGA and EJS were 0.7 g and 70 mm, respectively. The axial force curves illustrated in Figures 22(c) and 22(f) indicate that when the PGA
was 0.7 g, the maximum pier bottom axial force of the ordinary and the restrainer piers gradually decreased with an increase in the EJS. When the PGA was 0.4 g, the maximum pier bottom axial force of the ordinary pier increased first and then decreased, while the maximum axial force of the restrainer pier decreased first and then increased; its minimum value was obtained at an EJS of 70 mm.

4.2.3. Two Restrainer Piers. The relative displacement of the pier-beam under different EJSs changed similarly along the positive direction (Figure 23(a)). When the EJS was 70 mm and the PGA was 0.4 g, the relative displacement of the pier-beam was reduced maximally (the maximum was 58.33%). It is believed that from a relative displacement perspective, good performance for a six-unit bridge can be achieved with two restrainer piers and an EJS of 70 mm (Figures 23(b) and 23(c)).

The internal force of the ordinary and the restrainer piers when two restrainer piers were set up is plotted in Figure 24. With an increase in the EJS, the maximum pier bottom moment of the ordinary and the restrainer piers generally decreased (Figures 24(a) and 24(d)). For the restrainer pier, the moment changed little when the EJS was greater than 70 mm. The shear

![Figure 21](image1.png)
![Figure 22](image2.png)
curves illustrated in Figures 24(b) and 24(e) indicate that the maximum pier bottom shear of the ordinary and the restrainer piers gradually decreased with an increase in the EJS. However, when the PGA was 0.4 g and the EJS was 70 mm, the maximum pier bottom shear of the ordinary pier increased, and then gradually decreased. The axial force curves illustrated in Figures 24(c) and 24(f) demonstrate that the maximum pier bottom axial force of the ordinary and the restrainer piers gradually decreased with an increase in the EJS, differing only in the rate of change. When the PGA was 0.4 g, the maximum pier bottom axial force of the restrainer pier increased first, then decreased with an increase in the EJS, and reached its maximum value when the EJS was 50 mm. Moreover, when the PGA was 0.4 or 0.7 g and the EJS was greater than 70 mm, the axial force changed little.

5. Influence of Span on the Structural Dynamic Response

5.1. No Restrainer Piers. The maximum relative displacement of the pier-beam under a PGA of 0.4 or 0.7 g reached its maximum value at the No. 0 abutment (Figure 25). With an
increase in the span, the relative displacement at the No. 0 abutment gradually decreased. The relative displacement of the whole bridge in the positive direction decreased gradually. When the PGA was 0.4 g and the span was 50 m, the relative displacement at each pier was the smallest for pier Nos. 1 through to 17. Also, the relative displacement of each span between pier Nos. 17 and 20 was similar under different spans, and there was a big difference after No. 20 pier. When the PGA was 0.7 g and the span was 50 m, the relative displacement of each pier was the smallest for pier Nos. 1 through to 8. After pier No. 8, the relative displacement at a span of 50 m was less different from that of the other spans (although that of the 50 m span was larger).

The internal force of each pier with no restrainer pier is shown in Figure 26, where the subscripts 0.4 and 0.7 indicate a PGA of 0.4 and 0.7 g, respectively. The maximum moment occurred at pier No. 16 when the span was 20 m and at pier No. 12 for all other spans. When the span was 40 m, the maximum moment at pier Nos. 1 through to 12 was smaller than that of the other spans. After pier No. 12, at a span of 50 m, the maximum moment was smaller than that of the other spans (Figure 26(a)). However, when the PGA was 0.7 g and the span was 50 m, the maximum moment had a smaller value for pier Nos. 1 through to 14 (Figure 26(d)). Figures 26(b) and 26(e) display that the maximum shear occurred at the abutment of both pier Nos. 0 and 24, and the span had little influence on the shear of the No. 0 abutment. However, for the No. 24 abutment, the shear under different spans differed widely. Moreover, when the span was 30 m, the shear of the No. 24 abutment was maximum under a PGA of 0.4 and 0.7 g. However, when the span was 50 m, the shear at the bottom of most piers was at its minimum.

The axial force of the bridge showed an increasing trend along the positive direction (Figures 26(c) and 26(f)). For spans of 20, 30, 40, and 50 m, the maximum axial force occurred from pier Nos. 22 to 24. When the PGA was 0.4 g and the span was 30 m, the axial force at the pier bottom was larger than that of the other spans. When the PGA was 0.7 g and the span was 50 m, the axial force of pier Nos. 0 through to 10 was at its minimum. Under a PGA of 0.4 and 0.7 g, the axial force of each pier showed fewer changes when the span was 50 m.

5.2. One Restrainer Pier. A restrainer pier changes the rigidity of the bridge, resulting in uneven internal force distribution. After setting up one restrainer pier, the relative displacement of the third and fourth unit was large (Figure 27). However, the largest displacement occurred at the No. 0 abutment, and with an increase in the span, the relative displacement along the positive direction decreased first and then increased and was ultimately reduced. When the PGA was 0.7 g, the maximum relative displacement occurred in the middle part of the bridge. For a span of 50 m, the relative displacement varied gently.

The internal force of the ordinary and the restrainer piers with one restrainer pier is plotted in Figure 28. The maximum moment of the ordinary and the restrainer piers gradually decreased with an increase in the span, and its maximum and minimum values were obtained at a span of 20 and 50 m, respectively (Figures 28(a) and 28(d)). The shear curves illustrated in Figures 28(b) and 28(e) reveal that when the PGA was 0.4 g, the maximum shear of the ordinary and the restrainer piers increased first and then decreased with an increase in the span. Its maximum value was obtained at a span of 30 m; a smaller value was obtained at the span of 50 m. When the PGA was 0.7 g and the span was 40 m, the pier bottom shear of the ordinary pier was at its minimum, while the shear of the restrainer pier changed little with an increase in the span. Under a PGA of 0.4 and 0.7 g, when the span was 50 m, the axial force of the ordinary pier bottom was at its minimum (Figures 28(c) and 28(f)). When the PGA was 0.4 g and the span was more than 30 m, the influence of the span on the restrainer pier axial force was insignificant. Therefore, from the perspective of axial force, a 50 m span is suitable for long bridges.

5.3. Two Restrainer Piers. The relative displacement of the pier-beam with two restrainer piers under a PGA of 0.4 or 0.7 g is illustrated in Figure 29. The relative displacement of the pier-beam reached its extremum at the No. 0 abutment, No. 24 abutment, and No. 4 pier. As the span increased, the relative displacement in the positive direction fluctuated substantially. Moreover, when the PGA was 0.7 g, the trend of the relative displacement in the positive direction under each span was essentially the same. The relative displacement in the middle part of the bridge was smaller when the span was 50 m.

The internal force of the ordinary and the restrainer piers is depicted in Figure 30. The moment curves of Figures 30(a) and 30(d) demonstrate that when the PGA was 0.4 g, the maximum moment of the ordinary and the restrainer piers gradually decreased with an increase in the span, reaching its maximum and the minimum values at spans of 20 and 50 m, respectively. However, when the PGA was 0.7 g, the maximum moment of the ordinary and the restrainer piers increased first and then decreased with an increase in the span. Maximum and minimum values were reached at spans of 30 and 50 m, respectively. When the PGA was 0.4 g, the maximum shear of the ordinary pier increased first and then decreased with increases in the span, and its minimum value was reached at a span of 50 m (Figure 30(b)). When the PGA was 0.7 g, its maximum and minimum values were reached at spans of 40 and 50 m, respectively. The shear of the restrainer pier changed little with an increase in the span (Figure 30(e)). When the PGA was 0.4 g, the maximum axial force of the ordinary and the restrainer piers increased first and then decreased with an increase in the span (Figures 30(c) and 30(f)). When the PGA was 0.7 g, the maximum axial force of the ordinary pier gradually decreased with an increase in the span. Finally, under a PGA of 0.4 or 0.7 g, the axial force of the ordinary and the restrainer piers reached its minimum value at a span of 50 m.

5.4. Three Restrainer Piers. The relative displacement of the pier-beam with three restrainer piers under a PGA of 0.4 or 0.7 g is plotted in Figure 31. The relative displacement of the
pier-beam from the second unit to the fourth unit was smallest when the PGA was 0.4 g and the span was 50 m. Moreover, when the PGA was 0.7 g, the relative displacement trend of each span was the same in the positive direction, and the first and sixth units had the largest values.

The internal force of the ordinary and the restrainer piers is plotted in Figure 32. The maximum moment of the ordinary and the restrainer piers decreased with an increase in the span (Figures 32(a) and 32(d)). Under a PGA of 0.4 or 0.7 g, the maximum moment of the ordinary and the restrainer piers reached their minimum value at a span of 50 m. The curves of shear illustrated in Figures 32(b) and 32(e) indicate that when the PGA was 0.4 g, with an increase in the span, the maximum and minimum shear of the ordinary pier was reached at spans of 40 and 30 m, respectively. When the PGA was 0.7 g, the maximum shear increased first and then decreased with an increase in the span, and its minimum value was reached at a span of 50 m. The maximum shear of the restrainer pier was less affected by span and was less varied. Under a PGA of 0.4 or 0.7 g, the maximum axial force of the ordinary pier reached its maximum and
minimum values at spans of 40 and 30 m, respectively, with an increase in the span (Figures 32(c) and 32(f)). However, the maximum axial force of the restrainer pier increased with an increase in span, and its maximum and minimum values were reached at spans of 50 and 20 m, respectively.
Figure 29: Relative displacement of the pier-beam with two restrainer piers.

Figure 30: Internal force at the pier bottom of the ordinary and the restrainer piers with two restrainer piers.
6. Conclusion

After inputting some seismic waves, the influence of a varying number of restrainer piers, EJSs, and spans on the dynamic response and seismic performance of long bridges with equal-height piers was scrutinized. The main conclusions of this study can be summarized as follows:

(1) The influence of the number of restrainer piers on the dynamic response showed that the internal force at the bottom of the pier increased for each additional restrainer pier. Moreover, our results also illustrated that the longitudinal antipushing rigidity of the bridge increased with an increase in the number of restrainer piers, which could reduce the maximum relative displacement of the pier-beam in the longitudinal direction. Considering the occurrence of, and cost incurred by, rare earthquakes, it is recommended to use three restrainer piers in bridge engineering.

(2) The influence of the EJS on the dynamic response showed that the pounding had little impact on the internal force of the pier bottom when one or two restrainer piers were set up. Moreover, in terms of the relative displacement, when there were two

![Figure 31: Relative displacement of the pier-beam with three restrainer piers.](image)

![Figure 32: Internal force at the pier bottom of the ordinary and the restrainer piers with three restrainer piers.](image)
restrainer piers and the EJS was 70 mm, the relative displacement of the pier-beam was at their mini-
mum. Therefore, the seismic performance was best when an EJS of 70 mm was used.

(3) The dynamic response analysis of the main girder with different spans indicated that the relative dis-
placement of the pier-beam gradually decreased with an increase in the span on the whole, and the internal force of the whole bridge had the same trend in general. Using a span of 50 m was proposed for areas with frequent strong earthquakes.

(4) The results demonstrated that efficient performance can be achieved for a 24-span equal-height beam bridge under the following parameters: 3 restrainer piers, an EJS of 70 mm, and a 50 m span. However, in other cases, the determination of the optimal number of restrainer piers, the size of EJS, and the span need further investigation.

Data Availability

The data that support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this manuscript.

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