Research Article

Comparison of Highway Bridge Seismic Design in Europe and China through a Case Study

Qingguo Ben and Xiaoning Zhu

JSTI Group, Nanjing, Jiangsu Province 210017, China

Correspondence should be addressed to Qingguo Ben; benqingguo@outlook.com

Received 14 January 2022; Revised 12 March 2022; Accepted 16 March 2022; Published 27 March 2022

Academic Editor: Shuang Li

Copyright © 2022 Qingguo Ben and Xiaoning Zhu. This is an open access article distributed under the Creative Commons Attribution License, which permits unrestricted use, distribution, and reproduction in any medium, provided the original work is properly cited.

With accumulated experiences of bridge damage learned from past earthquake events, the highway bridge seismic design approach is evolving from time to time. It is important for both China and Europe to learn from each other to improve seismic design approaches. A thorough comparison relating to the seismic hazard level, response spectra, and ductility considerations between Eurocode 8 and the Chinese specification is made in this study. Both of these two specifications are based on performance-based design philosophy. However, the Chinese specification is based more on deformation capacity than on strength. The design approach specified in the Chinese specification is more consistent with experiences obtained from past earthquake events than the method adopted in Eurocode 8. Case study shows that bridge designed in accordance with Eurocode 8 could satisfy the force requirement under earthquake action E1 as specified in the Chinese specification but could not satisfy the displacement requirements under earthquake action E2. It is expected that the method adopted in the Chinese specification could provide conservative seismic design in both aspects of seismic forces and displacements.

1. Introduction

The first bridge design specification which includes seismic design provisions was published in 1925 shortly after the 1923 Kanto earthquake in Japan [1, 2]. In China, the first highway bridge seismic design specification was published in 1977 [3, 4], immediately following the July 28, 1976, Tangshan earthquake, which is one of the most destructive earthquakes ever happened in China. In the past century, specifications for seismic bridge design are evolving from the force-based design method to the displacement-based design method [2, 5]. The landmark earthquake events that significantly promoted the development of bridge seismic design philosophy in countries around the world are the 1971 San Fernando earthquake (USA), the 1995 Kobe earthquake (Japan), and the 2008 Wenchuan earthquake (China) [6, 7]. The development of seismic bridge design specifications was initiated primarily by the fact that the observed representative damage types of bridge in past earthquake events are unseating of girders, longitudinal and transverse offset of decks, concrete spalling, and shear failure of piers. The seismic performance of bridges demonstrated that the force-based seismic design has many shortcomings. The major one is that it is not explicitly related to the seismic performance of bridge, as displacement and deformation determine seismic damage rather than component strength [2]. It has been observed that bridges which possess ductility and can deform inelastically to the required deformations without loss of strength can survive the earthquake successfully [8]. In response to the bridge damage observed in past earthquake events, American Association of State Highway and Transportation Officials (AASHTO) first shifted its seismic design focus from the force-based R-factor design approach [9] to the displacement-based design approach in the Guide Specifications for LRFD Seismic Bridge Design published in 2009. When designed as per current seismic bridge design specifications/codes, proper seismic details such as tightly spaced transverse reinforcements in the bridge columns, well performed shear keys, and long seats should be provided, and satisfactory bridge seismic
performance can usually be achieved. As reported, fewer than 0.15% of highway bridges which possess aforementioned seismic details collapsed or were severely damaged in the February 27, 2010 Maule earthquake, Chile (Mw 8.8) [10, 11]. Nowadays, it is a common practice to involve a large number of probabilistic considerations, relating to the variability of seismic input, material properties, and component dimensions of bridge structures, in the specifications for bridge seismic design. Even more, financial consequences associated with bridge damage, collapse, or loss of usage following seismic attack are considered too [12–15]. However, despite the enhanced emphasis on realistic determination of displacement demand for bridges, design philosophy employed in current specifications could at best be termed as deformation-calculation-based seismic design, as only the detailing of critical sections is related to the deformation demand. Greater effort should be applied to develop a seismic design method which is more compatible with the concept of displacement-based design, so that when bridges are designed accordingly, they can achieve a specified deformation state under the design-level earthquake.

It has been recognized that seismic bridge design specifications of the USA and Japan are most influential in the seismic design field [12, 16, 17]. The newly published Specifications for Seismic Design of Highway Bridges in China in 2020 make a good reference to these two specifications [14]. While the Chinese design specification has relatively few connections with current Eurocode 8 [15], the European Community began its own action program in the field of construction in 1975 and finished the first generation of European codes in the 1980s [15, 18]. The seismic bridge design code currently in effect in Europe is the 2005 version. Due to different origins, evolution progresses, and design philosophies, there are substantial differences between the Chinese seismic bridge design specification and Eurocode 8, in aspects such as the seismic hazard level, response spectra, and behavior modification factors to consider ductility, etc [19]. With the development of economy and close international cooperation between China and European countries, it is essential to differentiate the difference existing in the design specifications. In this paper, a thorough comparison between Eurocode 8 [15] and the Specifications for Seismic Design of Highway Bridges in China (hereafter referred to Chinese specification) [14] relating to the aforementioned aspects is made. It is expected that this paper could be of some value to encourage China and Europe to learn from each other for improving their seismic standards in the future and that this will also promote the development of highway bridge seismic design philosophy.

2. Difference and Similarity between Eurocode 8 and Chinese Specification

2.1. Seismic Hazard Levels. Eurocode 8 specifies a single-level seismic design of new bridges corresponding to the life safety limit state of the general performance-based seismic design framework, requiring that after the seismic events, bridges shall be designed and constructed to retain its structural integrity and have sufficient residual resistance to be used for emergency traffic. The design seismic action for bridges of ordinary importance in Eurocode 8 has a reference return period of 475 years, corresponding to a 10% exceedance probability in 50 years. However, for bridges that are essential for public safety or that are critical for communications in the region, the importance factor \( \gamma_l \) should be applied to the design seismic action for the purpose of achieving better bridge seismic performance. It is implicit in Eurocode 8 that once the life safety limit state of the bridge is achieved, minimal damage of the bridge would occur under earthquake with a high probability of exceedance, but limited only to secondary bridge components. The near-collapse limit state of bridge in an extreme and very rare earthquake is prevented by applying the capacity design concept, reflecting in the clauses relating to ductility and energy dissipation [19]. Single-level seismic design was also used in the repealed Chinese specification published in 1989 (i.e., Specifications of Earthquake Resistant Design for Highway Engineering) [4]. Currently, two-level seismic design for highway bridges, i.e., one for operational performance level and the other for life safety performance level, is employed. Corresponding to earthquakes with return period of about 475 years or 10% exceedance probability in 50 years (i.e., earthquake action E1) and of about 2000 years or 2.5% exceedance probability in 50 years (i.e., earthquake action E2), respectively, the two seismic design levels specified in the Chinese specification are adjusted by using seismic design important factor \( C_d \), which is similar to the importance factor \( \gamma_l \) as specified in Eurocode 8. Under operational performance level design earthquake, highway bridges are required to have only minor damage and to be open to traffic immediately after the earthquake. While the life safety performance level is specified to protect human life during and following a rare earthquake, it corresponds to the near-collapse limit state.

2.2. Response Spectra. The basic shape of the horizontal elastic response spectra in Eurocode 8 and the Chinese specification is illustrated in Figure 1. Obviously, the response spectrum in Eurocode 8 consisted of four branches, while there are only three branches in the response spectrum of the Chinese specification. It is important to notice that these two response spectra are described by the lower limit of the period of the constant spectral acceleration branch \( T_g \) in Eurocode 8 vs. 0.1 in the Chinese specification), the upper limit of the period of the constant spectral acceleration branch \( T_C \) in Eurocode 8 vs. \( T_g \) in the Chinese specification), the soil factor \( S \) in Eurocode 8 vs. \( C_s \) in the Chinese specification), and the damping correction factor \( \eta \) in Eurocode 8 vs. \( C_d \) in the Chinese specification). The importance factor \( \gamma_l \) or the seismic design important factor \( C_d \) is included in the design ground acceleration \( a_g \). Values of the parameters describing both type 1 elastic response spectra in Eurocode 8 and the elastic response spectra in the Chinese specification are given in Table 1. Values of the damping correction factors \( \eta \) and \( C_d \) are determined by (1) and (2), respectively. Comparison of the damping correction factors
is shown in Figure 2. It can be seen from Table 1 and Figure 2 that these values vary significantly in each specification.

\[
\eta = \sqrt{\frac{10}{5 + \xi}} \geq 0.55, \quad (1)
\]

\[
C_d = 1 + \frac{0.05 - \xi/100}{0.08 + 1.6\xi/100} \geq 0.55, \quad (2)
\]

where \(\xi\) is the damping ratio of the bridge, expressed as a percentage.

For the vertical elastic response spectrum in Eurocode 8, the amplification factor in the constant spectral pseudo-acceleration plateau is 3 instead of 2.5 as in the horizontal direction, the period parameters \(T_B, T_C, \) and \(T_D\) are fixed for all soil types (see Table 2), and there is no amplification factor due to soil type for the vertical spectrum. However, the soil factor and the upper limit of the period of the constant spectral acceleration branch for the vertical elastic response spectrum in the Chinese specification are all different (see Table 3). The shape of the vertical elastic response spectrum is the same as the horizontal elastic response spectrum in both Eurocode 8 and the Chinese specification.

2.3. Ductility Seismic Design. It is today commonplace that the two fundamental options for the seismic design of bridges are seismic isolation design and ductility seismic design. For the seismic isolation design, horizontal displacement demand imposed by earthquake excitation is accommodated by placing bridge deck on a system of sliding or horizontally flexible bearings at the top of the abutments and piers [20], while for the ductility seismic design, bridge deck is fixed or rigidly connected to at least one pier and the fixed pier is required to accommodate the horizontal displacement demand by developing inelastic rotations in the assigned plastic hinge regions [21]. Only ductility seismic design is discussed in this paper. Ductility seismic design in Eurocode 8 is force-based as the inelastic response spectrum used is obtained from the elastic response spectrum by applying a so-called behavior factor \(q\). The behavior factor \(q\) is the ratio of \(F_{el}\) (i.e., peak force that would have developed if the bridge is elastic) to \(F_y\) (i.e., yield force of the bridge). Equal displacement rule, i.e., the peak displacement response of the inelastic and elastic bridges under earthquake excitation are about the same, is adopted to determine the value of \(q\). The behavior factor \(q\) is expected to reflect the global inelastic deformations of bridge under the design seismic action, and a safety factor between 1.5 and 2 is expected to be achieved by properly dimensioning and detailing the plastic hinges in the piers. It should be noted that the behavior factor \(q\) in Eurocode 8 enters in the inelastic design response spectrum and must be determined beforehand; therefore, iterative dynamic analysis is inevitable.

Force-based seismic design usually can produce safe and satisfactory designs when combined with capacity design principle and careful detailing of plastic hinges. However, it should be emphasized that force-based seismic design implicitly implies that the elastic characteristics of the bridge are the best indicators of inelastic performance of the bridge. Additionally, the component stiffness in force-based seismic design is traditionally assumed to be independent of the component strength, and hence according to equal displacement principle, increasing the strength of a bridge would improve its safety. But accompanying the crushing of

---

**Table 1: Values of the parameters describing the basic shape of the horizontal elastic response spectra.**

| Eurocode 8 | Chinese specification |
|-----------|-----------------------|
| Ground type | \(S\) | \(T_B\) (s) | \(T_C\) (s) | \(T_D\) (s) | Ground type | \(C_s\) (s) | \(T_g\) (s) |
| A | 1.00 | 0.15 | 0.40 | 2.00 | \(I_0\) | 0.72–0.90 | 0.20–0.30 |
| B | 1.20 | 0.15 | 0.50 | 2.00 | \(I_1\) | 0.80–1.00 | 0.25–0.35 |
| C | 1.15 | 0.20 | 0.60 | 2.00 | II | 1.00 | 0.35–0.45 |
| D | 1.35 | 0.20 | 0.80 | 2.00 | III | 1.30–1.00 | 0.45–0.65 |
| E | 1.40 | 0.15 | 0.50 | 2.00 | IV | 1.25–0.90 | 0.65–0.90 |
concrete and yielding of longitudinal reinforcements in the plastic hinge regions of piers, the initial bridge elastic stiffness will be irrelevant even to the subsequent elastic response of bridge following inelastic deformation of piers. Besides, the assumption of stiffness independent of strength is proved to be invalid by detailed analysis and experimental evidence [22]. Another problem with force-based seismic design is the selection of appropriate member stiffness. The assumed effective component stiffness used in force-based seismic design will significantly affect the design seismic forces. It is specified in Eurocode 8 that the cracked bending and shear stiffness may be taken as one half of the uncracked elastic stiffness of the gross section or can be estimated from (3) or (4). If inaccurate stiffness is assumed, the calculated displacement demand will also be inaccurate and probably be nonconservative.

\[ I_{\text{eff}} = 0.08I_{\text{un}} + I_{\text{cr}}, \]  

\[ E_{c}I_{\text{eff}} = \frac{\gamma M_{d}}{\phi_{y}}, \]  

where \( \gamma = 1.2 \); \( M_{d} \) is the design ultimate moment; and \( \phi_{y} \) is the curvature of pier section at first yield of the reinforcing steel.

In the Chinese specification, because the aforementioned two-level seismic design approach is adopted, calculation of bridge responses under earthquake actions \( E_{1} \) and \( E_{2} \) should be performed. Different amplified important factors \( C_{i} \) are applied to earthquake actions \( E_{1} \) and \( E_{2} \). Under earthquake action \( E_{1} \), bridges are required to remain essentially elastic and immediate service should be available following the earthquake. Therefore, component forces are more important under earthquake action \( E_{1} \). Also, elastic analysis procedure is employed and gross section area of the piers is used to obtain a conservative assessment of the seismic design force. Under earthquake action \( E_{2} \), bridge displacements are more critical. Inelastic action of bridge pier is allowed and is intended to be restricted only to the plastic hinge regions. Consequently, nonlinear analysis is a necessity and time history analysis or equivalent elastic analysis is employed. In order to obtain the realistic maximum displacement demand under earthquake action \( E_{2} \), effective section properties should be used when modeling ductile piers. Effective section properties as specified in the Chinese specification should be obtained from the \( M-\phi \) curve analysis (see (5)) of the section.

![Comparison of the damping correction factors in Eurocode 8 and the Chinese specification.](image)

**Figure 2:** Comparison of the damping correction factors in Eurocode 8 and the Chinese specification.

**Table 2:** Parameters of the vertical elastic response spectra in Eurocode 8.

| Spectrum | \( a_{ug}/a_{g} \) | \( T_{B} \) (s) | \( T_{C} \) (s) | \( T_{D} \) (s) |
|----------|-----------------|-------------|-------------|-------------|
| Type 1   | 0.90            | 0.05        | 0.15        | 1.00        |

**Table 3:** Parameters of the vertical elastic response spectra in the Chinese specification.

| Ground type | \( C_{g} \) (s) | \( T_{g} \) (s) |
|-------------|----------------|-------------|
| I\(_{0}\)    | 0.60           | 0.15-0.25   |
| I\(_{1}\)    | 0.60-0.70      | 0.20-0.30   |
| II          | 0.60-0.80      | 0.25-0.40   |
| III         | 0.70-0.80      | 0.30-0.50   |
| IV          | 0.80-0.90      | 0.55-0.75   |
where $M_y$ is the moment capacity of pier section at first yield of the reinforcing steel; $\phi_y$ is the curvature of pier section at first yield of the reinforcing steel; $E_x$ is the modulus of elasticity of concrete; and $I_{eff}$ is the effective moment of inertia of the pier section.

3. Case Study

Extensive worked examples about design of highway bridges as to Eurocode 8 have been given in reference [23]. In this section, however, a prototype bridge chosen from realistic project is first designed according to Eurocode 8 and then checked with the Chinese specification.

3.1. Prototype Bridge. The prototype bridge is a 4-span overpass, with spans 65 + 95 + 95 + 65 m and total length of 320 m, as shown in Figure 3. The deck is a post-tensioned cast in situ concrete box girder. Pier heights are 14 m for P1, 38 m for P2, and 30 m for P3. Illustration of the pier section is shown in Figure 4. For pier P1, $H$ is 2.5 m; for pier P2, $H$ is 5.02 m at the bottom and is 3.5 m at the top; for pier P3, $H$ is 2.5 m. The deck is rigidly supported on P2 and P3 and supported on P1 and the abutments through bearings allowing free sliding and rotation in and about both horizontal axes. The piers and abutments are founded on pile groups. The piers are made of concrete C40/50 with $f_{ck}=40$ MPa and $E_x=35$ GPa and reinforcing steel S500 with $f_{y}=500$ MPa. The cover to the reinforcement center is $c=40$ mm. The piles are made of concrete C30/37 with $f_{ck}=30$ MPa and $E_x=33$ GPa and of reinforcing steel S500 with $f_{y}=500$ MPa. The cover to the reinforcement center is $c=75$ mm. The main elements resisting seismic forces are piers P2 and P3. A limited ductile seismic behavior is suggested for piers P2 and P3. The value of the behavior factors $q$ in the horizontal direction is 1.5, while that in the vertical direction is 1.0.

3.2. Response Spectra. The design seismic action is calculated by using response spectrum of type 1. The ground type is $B$, so the characteristic periods are $T_B=0.15 s$, $T_C=0.5 s$, and $T_D=2 s$, while the soil factor is $S=1.2$. The bridge is located at seismic zone with a reference peak ground acceleration $a_{gR}=0.26 g$. The importance factor is $\gamma_I=1.3$, and the lower bound factor is $\beta=0.20$. Therefore, the seismic action in the horizontal direction is $a_g=\gamma_I \cdot a_{gR}=1.3 \times 0.26 g=0.338 g$. In the vertical direction, $a_{gV}/a_g=0.9$. The horizontal and vertical design response spectra calculated according to Eurocode 8 are presented in Figures 5 and 6, respectively.

3.3. Seismic Design Results. The effective moment of inertia of the cracked pier, calculated as per (3), is 0.4 times the moment of inertia of the gross section of the uncracked pier. The fundamental periods of the bridge estimated according to the effective moment of inertia of the pier are 4.7 s, 2.3 s, and 1.3 s in the transverse, longitudinal, and vertical directions, respectively. The combination rule of the three components of the seismic action in the response spectrum analysis is the linear combination rule of the type given in (6). Summary of the suggested seismic reinforcements designed according to Eurocode 8 is given in Tables 4 and 5. Combination of seismic design forces and the capacity of critical pier sections are shown in Figure 7. It can be seen from Figure 7 that the most critical section is of the transverse direction at the top of pier 2, at which the capacity of the section is marginally larger than the demand of seismic design force.

$$E_{1eff} = \frac{M_y}{\phi_y},$$

$$E = \left\{ \begin{array}{ll}
|E_x| + 0.3|E_y| + 0.3|E_z|, \\
0.3|E_x| + |E_y| + 0.3|E_z|, \\
0.3|E_x| + 0.3|E_y| + |E_z|.
\end{array} \right.$$
Figure 3: Illustration of the prototype bridge.

Figure 4: Illustration of the pier section (unit: m).

Figure 5: Comparison of horizontal design response spectra.

Figure 6: Comparison of vertical design response spectra.
Table 4: Suggested longitudinal reinforcements of piers.

| Location | Reinforcement | P1 | P2 | P3 |
|----------|---------------|----|----|----|
| Bottom   | Number        | 382| 529| 510|
|          | Size (mm)     | 25 | 32 | 32 |
|          | Ratio (%)     | 1.3| 2.3| 2.8|
| Top      | Number        | SC | SC | SC |
|          | Size (mm)     | 32 | 32 | 32 |
|          | Ratio (%)     | 2.2| 2.8| 2.8|

Note. SC means reinforcement ratios should be determined based on static analysis.

Table 5: Suggested transverse reinforcements of piers.

| Location | Position | Reinforcement | P1 | P2 | P3 |
|----------|----------|---------------|----|----|----|
| Bottom   | Longitudinal | Shear | Not required | $A_{wm}/s \geq 121.5 \text{ cm}^2/\text{m}$ | $A_{wm}/s \geq 166.6 \text{ cm}^2/\text{m}$ |
|          | Transverse | Shear      | Not required | $A_{wm}/s \geq 53.8 \text{ cm}^2/\text{m}$ | $A_{wm}/s \geq 61.2 \text{ cm}^2/\text{m}$ |
| Top      | Longitudinal | Shear | -- | $A_{wm}/s \geq 171.1 \text{ cm}^2/\text{m}$ | $A_{wm}/s \geq 153.3 \text{ cm}^2/\text{m}$ |
|          | Transverse | Shear | -- | Not required | Not required |
|          | Confinement | -- | Not required | Not required | Not required |

Against buckling

$s \leq 12.8 \text{ cm}$

$s \leq 16.4 \text{ cm}$

Note. $A_{wm}$ is the total cross-sectional area of hoops or ties in the one transverse direction of confinement; $s$ is spacing of tie legs on centers.

Figure 7: Continued.
Figure 7: Moment-axial force interaction diagram for critical pier sections.

Figure 8: Continued.
3.6. Earthquake Action E2. As per the Chinese specification, seismic responses of the bridge under earthquake action E2 can be obtained either from nonlinear time history analysis or from response spectrum analysis. Nonlinear time history analysis is thought to be more accurate than the response spectrum analysis. However, by using the nonlinear time history analysis, response spectrum compatible artificial ground motions [25, 26] have to be generated first because the recorded ground motions are usually different in overall ground motion level and spectral shape from the design spectrum. Besides, nonlinear frame hinge models [27, 28] have to be inserted in the potential hinge regions, which will unnecessarily complicate the comparison process. Therefore, response spectrum analysis is chosen. Deformation of the piers obtained from response spectrum analysis shall be multiplied by the magnification factor specified in the following equation:

\[
R_d = \begin{cases} 
1 - \frac{1}{\mu_{\Delta}} \frac{T^*}{T} & \text{for } \frac{T^*}{T} \geq 1.0, \\
1.0 & \text{for } \frac{T^*}{T} \leq 1.0,
\end{cases}
\]

where \(T^* = 1.25T_p\), \(\mu_{\Delta}\) is the maximum pier displacement ductility demand (approximately equal to 6.0), \(T\) is the fundamental period in the calculation direction, and \(T_p\) is the characteristic period shown in Figure 1(b).

Displacement capacity of the pier \(\Delta_u\) is given by (9) based on the moment-area method for determining the pier’s rotation and deflection (see Figure 8). (9) is dependent on the following three assumptions: (a) the plastic rotation \(\theta_u\) of the pier is concentrated at the center of the analytical plastic hinge; (b) the distribution of elastic curvature along the pier is linear; and (c) the plastic curvature of the analytical plastic hinge is constant.

\[
\Delta_u = \frac{1}{3}H^2 \times \phi_y + \left( H - \frac{L_p}{2} \right) \times \theta_u, \\
L_p = \min(L_{p1}, L_{p2}), \\
L_{p1} = 0.08H + 0.022f_y d_s \geq 0.044f_y d_s, \\
L_{p2} = \frac{2}{3}b, \\
\theta_u = \frac{L_p(\phi_u - \phi_y)}{K_{ds}},
\]

where \(H\) is the height of the pier from point of maximum moment to the point of moment contraflexure, \(\phi_y\) is the idealized yield curvature (see Figure 8), \(\phi_u\) is the ultimate curvature (see Figure 8), \(L_p\) is the analytical plastic hinge length, \(b\) is the width of the pier, \(f_y\) is the yield strength of longitudinal reinforcement, \(d_s\) is the reinforcement diameter, and \(K_{ds}\) is the safety factor taken as 2.0.

Checking of the displacement capacity of the pier under earthquake action E2 is shown in Table 6. It is clearly shown in Table 6 that longitudinal displacement capacity of pier 1
Table 6: Checking of displacement capacity of piers.

| Direction     | Pier | $R_d$ | $\Delta_d$ (cm) | $\Delta_u$ (cm) | Check |
|---------------|------|-------|------------------|------------------|-------|
| Longitudinal  | 1    | 1.0   | 1.7              | 60.8             | Y     |
|               | 2    | 1.0   | 19.3             | 17.5             | N     |
|               | 3    | 1.0   | 20.1             | 19.3             | N     |
|               | 1    | 1.0   | 0.1              | 4.8              | Y     |
| Transverse    | 2    | 1.0   | 9.9              | 32.7             | Y     |
|               | 3    | 1.0   | 7.7              | 16.8             | Y     |

and pier 2 does not satisfy the requirements of the Chinese specification, which means pier 1 and pier 2 could not maintain their load resistance under seismic-induced deformations.

4. Conclusions

Eurocode 8 currently in effect was published seventeen years ago, a time before the state-of-the-art highway bridge seismic design approach was developed. Therefore, single-level seismic design of new bridges was adopted which is different from current common practice of adopting two-level seismic design. For this reason, response spectra defined in Eurocode 8 depend not only on seismic zone and soil conditions, such as that defined in the Chinese specification, but also on the structural system, i.e., relating to the behavior factor $q$ of the bridge. Ductility seismic design in Eurocode 8 is force-based, and structural behavior factor $q$ is used to reflect the global inelastic deformations of bridge under design seismic action. This approach implicitly implies that the elastic characteristics of the bridge are the best indicators of inelastic performance of the bridge. However, in the Chinese specification, by employing the two-level seismic design approach, sufficient bridge strength is stressed under earthquake action $E_1$, while sufficient displacement capacity of the bridge is stressed under earthquake action $E_2$. This design approach is more consistent with experiences obtained from past earthquake events, and it appears more straightforward and reasonable for the seismic design of bridges by employing the method specified in the Chinese specification. Case study shows that bridge designed in accordance with Eurocode 8 could satisfy the force requirement under earthquake action $E_1$ as specified in the Chinese specification but may not satisfy the displacement requirements under earthquake action $E_2$ as specified in the Chinese specification. It is expected that the method adopted in the Chinese specification would provide conservative seismic design in both aspects of seismic forces and displacements.

Data Availability

The numerical data used to 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 paper.

References

[1] W. P. Yen and S. Unjoh, *Comparison of U.S. and Japanese Highway Bridge Seismic Retrofitting Measures*, Tsukuba, Japan, 1999.
[2] M. J. N. Priestley, G. M. Calvi, and M. J. Kowalsky, *Displacement-Based Seismic Design of Structures*, IUSS Press, Pavia, Italy, 2007.
[3] Y Wancheng and F Licu, “Ductility and isolation in asseismic designs for bridges-development tendency of Chinese asismatic code for bridges from the view of Eurocode 8,” *Journal of Tongji University*, vol. 22, no. 4, pp. 481–485, 1994.
[4] MCPRC, *Specifications of Earthquake Resistant Design for Highway Engineering*, China Communications Press Co., Ltd., Beijing, China, 1989.
[5] M. J. N. Priestley, G. M. Calvi, and M. J. Kowalsky, “Direct displacement-based seismic design of structures,” in *Proceedings of the 5th New Zealand Society for Earthquake Engineering Conference*, Palmerston North, New Zealand, March 2007.
[6] W. H. P. Yen, G. Chen, M. Yashinsky, Y. Hashash, C. Holub, and K. Wang, *China Earthquake Reconnaissance Report: Performance of Transportation Structures during the May 12, 2008, M7 Wenchuan Earthquake*, U.S. Department of Transportation, Federal Highway Administration, Research, Development, and Technology, Turner-Fairbank Highway Research Center, Washington DC, USA, 2011.
[7] S. Shekhar, J. Ghosh, and S. Ghosh, “Impact of design code evolution on failure mechanism and seismic fragility of highway bridge piers,” *Journal of Bridge Engineering*, vol. 25, no. 2, Article ID 04019140, 2020.
[8] S. D. C. Hampshir, S. BucurZanaica, S. D. S. Lima, C. Bucur, and S. D. Lima, “Comparative study of codes for seismic design of structures,” *Mathematical Modelling in Civil Engineering*, vol. 9, no. 1, pp. 1–12, 2013.
[9] A. Aashto, *LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington DC, USA, 8th edition, 2017.
[10] W. H. P. Yen, G. Chen, I. Buckle, T. Allen, D. Alzamora, and J. Ger, *Postearthquake Reconnaissance Report on Transportation Infrastructure Impact of the February 27, 2010, Offshore Maule Earthquake in Chile*, U.S. Department of Transportation, Federal Highway Administration, Research, Development, and Technology, Turner-Fairbank Highway Research Center, Washington DC, USA, 2011.
[11] C. Cui and Y. Xu, “Mechanism study of vehicle-bridge dynamic interaction under earthquake ground motion,” *Earthquake Engineering & Structural Dynamics*, vol. 50, no. 7, pp. 1931–1947, 2021.
[12] AASHTO Guide, *Specifications for LRFD Seismic Bridge Design*, American Association of State Highway and Transportation Officials, p. 309, Washington DC, USA, 2nd edition, 2015.
[13] MOHURD, *Code for Seismic Design of Urban Bridges*, China Architecture and Building Press, Beijing, China, 2011.
[14] MCPRC, *Specifications for seismic design of highway bridges*, China Communications Press Co., Ltd., Beijing, China, 2020.
[15] European Committee for Standardization, *Eurocode8: Design of Structures for Earthquake Resistance-Part2: Bridges*, European Committee for Standardization, Brussels, Belgium, 2005.
[16] Japan Road Association, *Design Specifications for Highway Bridges: Part V Seismic Design*, Japan Road Association, Tokyo, Japan, 2012.

[17] A. D. E. Sebai, *Comparisons of international seismic code provisions for bridges*, McGill University, Montreal, Canada, 2009.

[18] A. Ansal, “Perspectives on European Earthquake Engineering and Seismology: Volume 1,” *Geotechnical, Geological and Earthquake Engineering*, vol. 34, 2014.

[19] B. Kolias, M. N. Fardis, and A. Pecker, *Designers’ Guide to Eurocode 8: Design of Bridges for Earthquake Resistance*, ICE Publishing, London, UK, 2012.

[20] X. Li and Y. Shi, “Seismic design of bridges against near-fault ground motions using combined seismic isolation and restraining systems of LRBs and CDRs,” *Shock and Vibration*, vol. 2019, Article ID 4067915, 11 pages, 2019.

[21] Q. Ben, “Research on correlation of ground motion parameters and seismic performance of bridge,” *Northern Communications*, no. 10, pp. 1–3, 2016.

[22] H Li, Q Ben, Z Yu, Y Zhang, and X Lu, “Analysis and experiment of cumulated damage of steel frame structures under earthquake action,” *Journal of Building Structures*, vol. 25, no. 3, pp. 69–74, 2004.

[23] Y. Bouassida, E. Bouchon, P. Crespo, P. Croce, L. Davaine, and S. Denton, *Bridge Design to Eurocodes-Worked Examples*, Publications Office of the European Union, Luxembourg, Europe, 2012.

[24] W. Smeby and A. D. Kiureghian, “Modal combination rules for multicomponent earthquake excitation,” *Earthquake Engineering & Structural Dynamics*, vol. 13, no. 1, pp. 1–12, 1985.

[25] D. A. Gasparini and E. H. Vanmarcke, *SIMQKE: A Program for Artificial Motion Generation*, MIT, Cambridge, England, 1976.

[26] F. Ferreira, C. Moutinho, A. Cunha, and E. Caetano, “An artificial accelerogram generator code written in matlab,” *Engineering Reports*, vol. 2, no. 3, pp. 1–17, 2020.

[27] S. El-Tawil and G. G. Deierlein, “Nonlinear analysis of mixed steel-concrete frames. I: element formulation,” *Journal of Structural Engineering*, vol. 127, no. 6, pp. 647–655, 2001.

[28] S. El-Tawil and G. G. Deierlein, “Nonlinear analysis of mixed steel-concrete frames. II: implementation and verification,” *Journal of Structural Engineering*, vol. 127, no. 6, pp. 656–665, 2001.