DUCTILITY OPTIMIZATION BASED COLLAPSE RESISTANCE DESIGN OF MULTI-SPAN BRIDGES CONSIDERING SPATIALLY VARIABLE GROUND MOTIONS

Ning Li¹, Zhong-Xian Li², Xiaoqiong Li³

ABSTRACT

The spatially variable ground motions (SVGM) usually result in non-uniform distribution patterns of seismic demands, leading to a considerable conflict with the uniform design capacity on bridges. A seismic performance optimization method for multi-span bridges is proposed as a solution to the conflict, through the modification of the ductility capacity patterns in seismic design procedures. A series of three-dimensional SVGM is generated and applied to the finite element model of a multi-span highway bridge in the southwest of China. The original bridge model is analyzed and modified by iteratively correcting the longitudinal reinforcement ratio of each pier until the optimal ductility pattern and configuration for the bridge is obtained. The results show that all the piers in the case of the optimal bridge are supposed to resist collapse synchronously. In this way, the redundant or deficient seismic ductility capacity of individual piers is avoided so that the optimal ductility capacity distribution is determined. Furthermore, the improvement of the overall seismic capacity for the entire bridge system achieves the objectives of minimal ductility demands. This proposed optimization methodology is proven to be efficient and practical in obtaining the optimal ductility capacity distribution patterns and the structural configurations for collapse resistance of multi-span highway bridges.

Keywords: Multi-span bridge; spatially variable ground motions; optimization design; ductility capacity; collapse resistance

1. INTRODUCTION

Performance-based seismic designs guarantee that the deformation and strength capacity of a structural system surpass the seismic demands in cases where the system is subjected to a predefined level of earthquake ground motions, with adequate margins of safety (Fajfar, 2000). Deformations are generally considered to be more critical than forces for defining performance. Displacement-based approaches are more rational than force-based approaches in terms of bridge designs because seismic damages to bridges are usually induced by the excessive structural displacements (Ghobarah, 2001). These approaches integrated with nonlinear analyses are critical in identifying the patterns and levels of damage for assessing the inelastic behavior and failure modes of structures. The preliminary design for most structures is based on the equivalent static forces specified by the relevant design code. However, they usually suffer from nonlinear seismic responses when subjected to severe earthquakes. Particularly, for long-span structures such as bridges, the spatially variable ground motions usually result in unexpected distribution patterns of seismic demands. Therefore, the current distribution of seismic forces may not necessarily lead to the best seismic performance for a particular structure (Mohammadi, 2004). Chopra (Chopra, 2001) suggested that the height-wise distribution patterns of strength for some shear building models, which conform to the uniform building code, can result in non-uniform distributions and unsatisfactory seismic responses, including ductility demands, drift, and damage. The conclusions drawn by Moghaddam et al. (Moghaddam, 1999)

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verified that the strength pattern recommended by the design code achieves a uniform distribution of ductility, and suggested that some arbitrarily chosen patterns can also generate uniform and smaller ductility. Zou et al. (Zou et al. 2005) developed an optimal drift performance design technique for reinforced concrete buildings, incorporating the pushover analysis and numerical procedure. Steel reinforcement ratios are also considered as design variables during the procedure, similar to the optimization of the index, as presented in Section 3.2 of this paper. Mohammadi et al. (Mohammadi et al. 2004) proposed a performance-based seismic design methodology to determine the optimum strength and stiffness pattern for the ductility distribution of multistory structures. However, most relevant studies have been based on multistory structures rather than on long-span bridges. Similar ideas could be extended to the design of seismic performance of bridges, after appropriate consideration of the spatial variation of earthquakes.

Long-span bridges are believed to experience different earthquake excitations at each support point (Zerva et al. 2002) owing to the spatially variable ground motions (SVGM), namely, the wave passage, loss of coherency, and site amplification effects (Der Kiureghian, 1996). Thus, the distribution of ductility demands subjected to SVGM is non-uniform for each pier. However, piers in the same bridge are usually designed to have a similar configuration, and thus the ductility capacities are quite close to each other. As a result, the ductility demands for each pier and their capacities are not equivalent. In other words, when the vulnerable piers suffer extensive damage during an earthquake, others may experience no damage. If all the components could reach the same damage state during the same earthquake synchronously, then there are no vulnerable components. In this way, the overall seismic performance of the bridge can be improved, and the material strength is fully utilized. Therefore, one of the objectives of this study is to address the question on how the ductility capacity pattern should be modified in a seismic design procedure to guarantee the minimal ductility demand of multi-span bridges.

This study proposes an equivalent seismic performance optimization method to ensure that all the components could resist collapse synchronously under differential earthquake ground motions. An iterative approach is developed to estimate the optimal ductility capacity pattern based on the multiple dynamic characteristics of SVGM. The optimization procedure can be performed by correcting the structural parameters such as the section’s dimensions, reinforcement ratio, and material strength. The optimal configuration for a damage state that is minor may not satisfy the ductility demand subjected to stronger earthquakes. Because the structure component is assumed to be near collapse in the case of a complete damage state, assuming that the lower bound ductility capacity of the complete damage state as the seismic performance index should provide adequate redundancy for the optimization procedure. In this way, all the components could remain below the level of complete damage, namely, resist collapse synchronously during earthquakes.

For the seismic design of bridge structures, which can be employed to achieve the optimal ductility capacity pattern for the minimum ductility demands, as well as the optimal configuration for collapse resistance are challenge issues. This methodology provides a solution to the current conflict between the uniform design capacity of the piers and their non-uniform seismic demands. The ductility optimization method is proposed and verified using an engineering example. A series of nonlinear time-history analyses (incremental dynamic analysis, IDA) are conducted in the process. This is then followed by validations of fragility analyses for each pier of the bridge. This approach is proved useful in the planning stages where decisions can be made to control the ductility demands and in the final design stages where details for structural and nonstructural elements are established.

2. PROPOSED DUCTILITY OPTIMIZATION ALGORITHM

2.1 Ductility Capacity Definitions

Various states of seismic damage of structures may varying from light damage to collapse, owing to the excessive deformation or displacement of members (Lin, 2003). In this study, the five limit states of damage (Hazus-MH, 2015) are defined in terms of the displacement ductility demand \( \mu_d \) (Hwang et al. 2000), as shown in Table 1 (\( \Delta \) is the relative displacement at the top of the pier, and \( \Delta_{cy1} \) is the specific value of \( \Delta \) when the longitudinal reinforcement first yields at the bottom of the pier).
Table 1. Bridge damage states (taken from HAZUS-MH 2.1)

<table>
<thead>
<tr>
<th>Damage states</th>
<th>Description</th>
<th>Ductility index</th>
</tr>
</thead>
<tbody>
<tr>
<td>No damage (N)</td>
<td>First yield</td>
<td>$\mu_d \leq \mu_{c1}$</td>
</tr>
<tr>
<td>Slight damage (S)</td>
<td>Minor cracking and spalling</td>
<td>$\mu_{c1} &lt; \mu_d \leq \mu_{c2}$</td>
</tr>
<tr>
<td>Moderate damage (M)</td>
<td>Moderate cracking and spalling</td>
<td>$\mu_{c2} &lt; \mu_d \leq \mu_a$</td>
</tr>
<tr>
<td>Extensive damage (E)</td>
<td>Any pier degrading without collapse</td>
<td>$\mu_a &gt; \mu_u$</td>
</tr>
<tr>
<td>Complete damage (C)</td>
<td>Collapse of any pier</td>
<td></td>
</tr>
</tbody>
</table>

All the ductility indices in Table 1 are based on $\Delta_{c1}$. The symbol $\mu_{c1}$ denotes the ductility demand at the first yield, which implies that $\mu_{c1} = 1$. Correspondingly, $\mu_{c2}$ is the equivalent yield ductility demand (Fajfar, 1992), $\mu_{c2}$ is the ductility demand when the compressive strain of the cover concrete reaches 0.004, and $\mu_u$ is the ultimate ductility demand when $\Delta$ denotes the ultimate displacements at pier top. In most cases $\mu_u = \mu_{c2} + 3$. This ductility index can be determined through IDA on the pier of the bridge. The ductility capacity of the component for each damage state is represented by the corresponding ductility index. It is obvious that $\mu_u$ is the upper bound of the ductility demand to guarantee that the pier will not collapse. Thus, in this study, $\mu_u$ is considered as the seismic performance index to provide adequate redundancy for the optimization procedure. In this way, the state of all the piers should remain below the level of complete damage, namely, they should resist collapse synchronously during earthquakes.

2.2 Optimization process

Traditional optimization design methods for structures usually aim to obtain the lowest cost or weight by adjusting the component dimensions, under the specific requirements of static stress or displacement. The failures of structures subjected to earthquake ground motions start from vulnerable components, then develop, and spread to the entire system. If all the components are designed with a similar seismic performance, there are no obvious vulnerable parts. Thus, the overall seismic performance of the structure is improved, and the material strength is fully utilized. This study proposes an equivalent seismic performance optimization method, to ensure that all the components could resist collapse synchronously under differential earthquake ground motions.

The seismic capacity of a bridge system with $n$ piers in series is defined as the minimum value of the seismic capacity for all piers,

$$M = \text{min} \left[ \frac{\mu_i}{\zeta_i} \right] \quad i = 1, 2, ..., n$$

where $\mu_i$ is the seismic capacity of the $i^{th}$ pier, considered as the ultimate ductility capacity in this study, and $\zeta_i$ is the importance factor of the $i^{th}$ pier, defined as,

$$\zeta_i = \frac{\frac{n \mu_i^d}{\sum_{j=1}^{n} \mu_j^d}}{\sum_{j=1}^{n} \mu_j^d}$$

where $\mu_i^d$ is the ductility demand of the $i^{th}$ pier, calculated as the mean value at multiple peak ground accelerations (PGA). The optimization function for an $n$-pier bridge could be expressed as

$$\text{min} \quad F_i(S)$$

subject to

$$\mu_i^d(S) < \mu_u \quad i = 1, 2, ..., n$$

$$S_i \in \mathbb{R}^q \quad l = 1, 2, ..., q$$
where the objective function \( F(S) \) is defined by the seismic performance index. Equivalently, \( \mu_i^d(S) < \mu_\Delta \) is the constraint equation formulated on the basis of the ductility demand, and \( S_l (l=1, 2, \ldots, q) \) is the \( q \)-dimensional optimization vector. According to the optimization scheme, parameters such as the properties of the material, reinforcement ratio, and section dimensions are varied because these factors have been proven to influence the ductility capacity of piers (Paultre et al. 2008). Compared with concrete materials, the steel reinforcement is more cost-efficient, and significantly affects the occurrence of first yielding. Thus, in this study, the reinforcement ratio is considered as the optimization index.

![Flow chart of the equivalent seismic performance optimization process](image)

The objective of the equivalent seismic performance optimization method is to reduce the difference of the seismic performance between each component. As aforementioned, the ultimate ductility
capacity \( \mu_t \) is considered as the seismic performance index. As indicated in Eq. (1), the seismic capacity of the system is determined by both the ductility capacity and the importance factor of the \( i^{th} \) pier. Thus, the seismic performance index for the \( i^{th} \) pier is \( \mu_t / \xi_i \). The objective of the optimization process could be performed by modifying the section dimensions, reinforcement ratio, or material strength of the piers. Consequently, the objective function is

\[
|\mu_t^i / \xi_i - \mu^i| \leq \varepsilon \quad i = 1, 2, ..., n
\]  

(4)

where \( \varepsilon \) is an arbitrary, real number, and \( \mu^i \) is the target ductility capacity of the entire bridge. According to Eq. (1), in order to obtain the best seismic performance of the system, \( \mu^i \) should be consistent for each pier. Thus, the optimal ductility capacity of each pier is in accordance with the weights of the most importance factors.

First, the constraint equation can be determined in accordance to the seismic design code based on the local site condition of the bridge. The spatially varying ground motions were simulated, and they were determined to be compatible with the prescribed response spectra. The ductility capacity and demand of each pier is calculated by IDA under differential excitation. Therefore, the importance factor can be derived from the ductility demand distribution, and the evaluation of the target ductility capacity pattern can be performed based on the original data. Assuming that the ductility demand at multiple PGAs is lognormal distributed as \( \mu_d = \ln(\hat{\mu}_d, \beta_d) \), it is denoted by the mean value \( \hat{\mu}_d \) during the optimization process in this study.

The step length of the optimizing index needs to be adjusted, and the new bridge model is modified with the upgraded optimized index value. Correspondingly, this implies that in this study, an evolutionary reinforcement ratio must be assigned to each pier. Subsequently, IDA is conducted on the new bridge model to obtain the ductility demand distribution for each pier. The constraint equation is then checked to ensure that the new configuration of the bridge meets the compliance standard in accordance to the design code. If not, the target ductility capacity should be re-evaluated. The ductility capacity and importance factors of the new bridge are compared with the target ductility capacity patterns. If the differences are unacceptable, the step length is re-adjusted, and the model updated until convergence on the target is achieved. The optimal configuration of the bridge can be obtained after several iterations.

The flow chart for the equivalent seismic performance optimization process are indicated in Fig 1. The ductility capacity is redistributed in accordance to the ductility demand (importance factor) for each pier in the case of the optimized bridge model. Thus, the quotient of the ductility capacity and the importance factor are equivalent for each pier.

3. NUMERICAL MODEL

3.1 Bridge Model

Jinghe’s bridge is a typical continuous rigid-frame bridge located in the mountainous region in the southwest of China. The elevation and cross-sections of this prestressed reinforced concrete highway bridge are shown in Figs. 2 and 3. The continuous girder is 984 m long with different site conditions at each support. Thus, the spatially variable ground motions can lead to differential seismic responses for each pier. Nevertheless, the configurations of the four middle piers that are fixed on the main girder are almost the same. The seismic capacity of each pier is not compatible with their demands. The two abutment piers (1 and 6) are single columns, and the middle piers (2–5) are double-limb. The structure uses C50 concrete, HPB300 steel for transverse reinforcement, and HRB335 steel for longitudinal reinforcement, with a tri-axial tension control stress of 1395 MPa applied to the box girder. Other details on the structural properties for this bridge can be found in (Du et al. 2011). The three-dimensional finite element model of the bridge is established with inelastic frame elements using OpenSees. The C50 concrete adopted in the entire bridge is simulated through a uniaxial, nonlinear, constant confinement model (Mander et al. 1988), and the reinforcements are arranged based on the relevant design code (China Communications Press, 2008) and in accordance to a bilinear steel model.
The pier bases are assumed to be fixed, while the two abutments (0 and 7) were modeled as roller supports, and restrained in the transverse direction (Y), vertical direction (Z), and accounted for X and Z rotations (Rx and Rz). There are two bearings at the joints of piers 1 and 6 with the main girder, with a longitudinal stiffness of 75 MN/m and a vertical stiffness of 19231 MN/m. The DoF at other joints are all coupled with the corresponding DoF of the girder, because they are fixed as a continuous rigid frame bridge. The dynamic interaction between the pile foundation and the surrounding soil is usually the main SSI effect that is considered in the analysis.

Because all the piles are rested on bedrock, the soil-pile interaction can be simplified in the ground motion transfer function.

![Figure 2. Elevation view for Jinghe’s bridge](image)

**Figure 2. Elevation view for Jinghe’s bridge**

**Figure 3. Cross-sections of Jinghe’s bridge**

### 3.2 Ground Motions

Fig. 2 shows the local conditions of the bridge based on geological exploration. The corresponding characteristic parameters are shown in Table 2. Points 0 and 7 are abutments and points 1–6 indicate spatial locations on the lower parts of the six piers.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Density ( \rho ) (g cm(^{-3}))</th>
<th>Shear wave velocity ( v ) (m/s)</th>
<th>Damping ratio ( \zeta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedrock</td>
<td>2.8</td>
<td>1500</td>
<td>0.05</td>
</tr>
<tr>
<td>Strongly weathered shale</td>
<td>2.7</td>
<td>700</td>
<td>0.05</td>
</tr>
<tr>
<td>Pebble soil</td>
<td>2.1</td>
<td>500</td>
<td>0.1</td>
</tr>
<tr>
<td>Soft clay</td>
<td>1.6</td>
<td>150</td>
<td>0.1</td>
</tr>
</tbody>
</table>

The ground motions on bedrock can be assumed to be zero mean stationary stochastic processes with identical power spectral density, which is modelled in accordance to (Bi, 2013)

\[
S_g(\omega) = |H_p(\omega)|^2 S_0(\omega) = \frac{\omega^4}{(\omega_0^2 - \omega^2)^2 + (2\xi\omega_0\omega)^2 (1 - \omega^2/\omega_0^2)^2} + 4\xi^2\omega^2/\omega_0^2 
\]

(5)
where $S_0$ is the Kanai-Tajimi power spectral density function (Tajimi, 1960), and $\omega_k$ and $\xi_k$ are the central frequency and damping ratio, respectively. In addition, $H_j(\omega)$ is a high-pass filter applied to maintain only the low-frequency components and correct the singularity with $\omega_l$ and $\xi_l$ denoting the central frequency and damping ratio. There is substantial physical basis for the choice of $\omega_l$ and $\xi_l$ (Hao et al. 1989), and different values are used based on the nature of the problem. The symbol $\Gamma$ denotes a constant scale factor relevant to the intensity of ground motions. It can be calculated using standard random vibration approaches (Der Kiureghian, 1980).

The auto-power spectral density functions of ground motion in Eq. (5) on the ground surface, and the cross-power spectral density function can be expressed as

$$S_j(\omega)=[H_j(i\omega)]^2S_x(\omega)$$

$$S_{jk}(\omega)=H_j(i\omega)H_k^*(i\omega)S_x(\omega)\gamma_{jk}(d_{jk},i\omega) \quad j,k=1,2,...,n$$

where $H_j(i\omega)$ and $H_k(i\omega)$ are the site transfer functions of two arbitrary points $j$ and $k$ on the ground surface, while * denotes the complex conjugate. Additionally, $\gamma_{jk}(i\omega)$ indicates the coherency between points with the distance between points $j'$ and $k'$ (denoted as $d_{jk}$), and utilization of the Sobczyk model (Sobczyk 1991),

$$\gamma_{jk}(i\omega)=|\gamma_{jk}(i\omega)|e^{-i\alpha_{jk}\cos\nu_{eq}}=e^{-i\beta\cos\nu_{eq}}e^{-i\alpha\cos\nu_{eq}}=e^{-i\beta\cos\nu_{eq}}$$

where $\beta$ is the coherence coefficient, $\alpha$ is the angle of the incident wave on the bedrock, and $v_{eq}$ is the apparent velocity. The power spectral density function of the stationary stochastic time histories at $n$ points can be described using an $n\times n$ density matrix (Bi et al. 2013)

$$S(i\omega)=\begin{bmatrix}
S_{11}(i\omega) & S_{12}(i\omega) & \cdots & S_{1n}(i\omega) \\
S_{21}(i\omega) & S_{22}(i\omega) & \cdots & S_{2n}(i\omega) \\
\vdots & \vdots & \ddots & \vdots \\
S_{n1}(i\omega) & S_{n2}(i\omega) & \cdots & S_{nn}(i\omega)
\end{bmatrix}$$

where the diagonal elements $S_{jj}(i\omega)$ ($j=1,2,...,n$) are the auto-power spectral densities. The off-diagonal elements $S_{jk}(i\omega)$ ($j,k=1,2,...,n, j\neq k$) are cross-power spectral densities. The ground motion acceleration time series generated using spectral representation method (Hao et al. 1989), and a nonstationary process can be obtained using an envelope function.

For example, the parameters for the ground motions in Eq. (5) are assumed to be $\omega_x=10\pi \text{ rad/s}$, $\xi_x=0.6$, $\omega_f=0.5\pi \text{ rad/s}$, $\xi_f=0.6$, and $\Gamma=0.0034 \text{ m}^2/\text{s}^2$ (Bi et al. 2012), which correspond to a ground motion time history with a time duration of 20 s (PGA = 0.2 g, PGD = 0.082 m). The parameters for the coherency loss function in Eq. (7) are $\beta=0.0005$, $\alpha=60^\circ$, and $v_{eq}=1768 \text{ m/s}$.

The fortification intensities for the bridge locations are VII according to JTG/T B02-2008, and the simulated ground motions can be compatible with the prescribed response spectra. The upper cut-off frequency $\omega_u$ in the simulation process is assigned to be equal to 50 $\text{rad/s}$, the time duration $T$ of the ground motion to 20 s, and the time step $dt$ to 0.01 s.

Based on the aforementioned method, the spatially variable ground motions compatible with local site conditions can be obtained using MATLAB. Differential acceleration time histories and displacement time histories at each ground surface support point are shown in Fig. 4. The eight time histories indicated in Fig. 4 (a) and (b) correspond to support points 0 to 7 in Fig. 2. The smallest and largest PGAs of the simulated earthquake time histories on bedrock are 1.53 m/s$^2$ and 2.27 m/s$^2$, which are close to the theoretical characteristics of the selected ground motion (PGA = 0.2g). The largest PGA of the differential acceleration time histories in Fig. 4(a) is 3.89 m/s$^2$, indicating the significant site amplification effect of the soil layers. It is noteworthy to state that the peak ground displacement (PGD) is larger at points 1 and 5. As aforementioned, the ductility demand
of each pier at each iterative step is calculated as the mean value of elicited results following exposure to multiple differential ground motion scenarios, considering eight different central frequencies \( \omega \) (0.3\( \pi \) to \( \pi \) at 0.1\( \pi \) increments) of a Kanai–Tajimi high-pass filter, five different incident angles \( \alpha \) (15°, 30°, 45°, 60°, and 75°) of the incoming wave to the site, and 10 evolutionary PGAs (0.05g to 0.7g). The ductility capacity is represented by \( \mu_u \), as discussed in Section 3.1. All the ground motion excitations are applied in three dimensions.

![Acceleration and Displacement Time Histories](image)

**Figure 4.** Simulated differential ground motion time histories (eight supports)

### 4. ANALYSIS RESULTS

**Table 3. Importance factor of piers at each optimization step**

<table>
<thead>
<tr>
<th>Step number</th>
<th>Pier 1</th>
<th>Pier 2</th>
<th>Pier 3</th>
<th>Pier 4</th>
<th>Pier 5</th>
<th>Pier 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original</td>
<td>1.485</td>
<td>0.797</td>
<td>0.519</td>
<td>0.511</td>
<td>1.557</td>
<td>1.130</td>
</tr>
<tr>
<td>1</td>
<td>1.454</td>
<td>0.821</td>
<td>0.538</td>
<td>0.566</td>
<td>1.510</td>
<td>1.111</td>
</tr>
<tr>
<td>2</td>
<td>1.431</td>
<td>0.822</td>
<td>0.588</td>
<td>0.629</td>
<td>1.441</td>
<td>1.090</td>
</tr>
<tr>
<td>3</td>
<td>1.400</td>
<td>0.856</td>
<td>0.613</td>
<td>0.644</td>
<td>1.402</td>
<td>1.085</td>
</tr>
<tr>
<td>4</td>
<td>1.341</td>
<td>0.917</td>
<td>0.673</td>
<td>0.669</td>
<td>1.319</td>
<td>1.080</td>
</tr>
<tr>
<td>5</td>
<td>1.294</td>
<td>0.953</td>
<td>0.701</td>
<td>0.697</td>
<td>1.286</td>
<td>1.069</td>
</tr>
</tbody>
</table>

The optimization process is performed by correcting the longitudinal reinforcement ratio of the cross-section for each pier. Fragility analyses on continuous rigid frame bridges show that the transverse...
ductility capacity is enhanced owing to the double-limb configuration of the piers in this direction. Therefore, these piers are more vulnerable to the spatially variable ground motions in the longitudinal direction. In this study, the longitudinal ductility capacity is considered as the seismic performance index. The importance factor of each pier is calculated as indicated by Eq. (2). The optimization process is performed. According to the IDA result of the original bridge model, the seismic capacity of the bridge system $\mu^t$ is evaluated to be approximately equal to 9, as defined by Eq. (1). Thus, the target ductility capacity for each pier is the product of $\mu^t$, and their importance factors are shown in Fig. 1. For the efficiency and accuracy of the iteration, the first three step lengths of the optimized index are assigned to 0.1%, and the lengths for the subsequent steps are adjusted until the optimization objective is met. The importance factors of piers for the original bridge model, and at each subsequent optimization step, are shown in Table 3. The corresponding values of the ductility capacities for each pier are listed in Table 4.

### Table 4. Ductility capacity of piers at each optimization step

<table>
<thead>
<tr>
<th>Step number</th>
<th>Pier 1</th>
<th>Pier 2</th>
<th>Pier 3</th>
<th>Pier 4</th>
<th>Pier 5</th>
<th>Pier 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.367</td>
<td>9.172</td>
<td>8.987</td>
<td>8.783</td>
<td>10.072</td>
<td>7.982</td>
</tr>
<tr>
<td>3</td>
<td>10.098</td>
<td>8.983</td>
<td>7.763</td>
<td>7.824</td>
<td>10.973</td>
<td>8.527</td>
</tr>
</tbody>
</table>

The importance factors for each pier presents minor differences during the optimization process, because the correction of the reinforcement ratio does not have a major influence on the overall rigidity of the entire bridge. Thus, if an excessive number of iterations is required to attain the optimizing objective, the importance factor could be calculated every 3 or 4 steps to simplify the process. Normally, the optimization could be fulfilled within 7 or 8 iterations. Fig. 5 shows the envelope curves of the seismic performance index at each step. It is important to note that the seismic performance index is the product of the ductility capacity and the importance factor of each pier. It is clear that Eq. (4) is applicable in the optimal bridge model.

As can be seen from Tables 3 and 4, the original piers are classified into two types, according to the cross-section. For each type, the longitudinal reinforcement ratio of the original bridge is determined based on JTG/T B02-01-2008 to be 1.2%, so the ductility capacity is consistent. In later iterations, the step length for the reinforcement ratio of each pier is adjusted according to the difference between the real-time ductility capacity and the target. It is obvious that the spatially varying ground motions result in significantly different seismic responses at each pier. For instance, the importance factors of piers 1 and 5 are larger than others. Therefore, the longitudinal reinforcement ratios for these two piers of the final, optimal, bridge model are higher. Table 5 shows the longitudinal reinforcement ratio of the bridge at three iteration steps.
The bridge can be regarded as a system with a serial connectivity of all its constituent piers. Therefore, the failures of the piers are considered as independent events under earthquake ground motions. The probability of damage for the entire bridge subjected to the ground motion excitation case \( k \) (\( P_k \)) is determined by the possibility that the \( i \)th pier exceeds its ductility index for this damage state (\( P_{i,k} \)), and could be evaluated in accordance to (Lupoi et al. 2005),

\[
P_k = 1 - \prod_{i=1}^{n} \left(1 - P_{i,k}\right)
\]

(9)

Table 5. Modification of the longitudinal reinforcement ratios of piers (%)

<table>
<thead>
<tr>
<th>Step number</th>
<th>Pier 1</th>
<th>Pier 2</th>
<th>Pier 3</th>
<th>Pier 4</th>
<th>Pier 5</th>
<th>Pier 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>Third</td>
<td>1.944</td>
<td>1.073</td>
<td>0.990</td>
<td>0.959</td>
<td>1.678</td>
<td>1.689</td>
</tr>
<tr>
<td>Optimal</td>
<td>2.758</td>
<td>0.943</td>
<td>0.811</td>
<td>0.798</td>
<td>2.260</td>
<td>2.056</td>
</tr>
</tbody>
</table>

Fig. 6 presents the fragility curves for the original bridge model and the optimal bridge model. It is obvious that the overall seismic capacity of the optimal bridge is significantly improved.

(a) Original bridge response curves          (b) Optimal bridge response curves

Figure 6. Fragility curves for the original and optimal bridges (S-slight damage; M-moderate damage; E-extensive damage; C-complete damage)

5. CONCLUSIONS

A seismic performance optimization method was developed that aimed to modify the bridge configuration (reinforcement ratio) during the seismic design procedure to achieve the optimal ductility capacity pattern for minimal ductility demands. In this way, without vulnerable components, all the piers in the same bridge are expected to resist collapse synchronously.

The optimization method was performed on a seven-span, continuous highway bridge, in the southwest of China. A series of spatially variable ground motions were generated with particular consideration of the local site conditions, and were applied to the finite element model of the bridge. The original bridge model was analyzed and modified by iterative corrections of the longitudinal reinforcement ratio of the cross-section for each pier, until the optimal configuration and ductility pattern for the bridge was obtained. Results show that the fragility curves of complete damage for each pier of the optimal bridge model are obviously closer to those of the original model. Thus, it is reasonable to conclude that under an earthquake with a specific PGA on the bedrock, each pier of the optimal bridge model would exhibit a state that is far below the level of complete damage, and all piers would assume similar probabilities of damage. Equivalently, all the piers of the optimal model could resist collapse synchronously. In this way, the redundant or deficient seismic ductility capacity of individual piers is avoided, and there are no obvious vulnerable components. Thus, the optimal
ductility capacity pattern is obtained. Additionally, the fragility curves for the original bridge model and the optimal bridge model explicitly indicate the improvement of the overall seismic capacity of the entire bridge system. Therefore, the aim for minimal ductility demand is achieved. The proposed optimization approach provides a good basis for additional, performance-based optimizations for bridges. Given consideration of additional structural components and accurate nonlinear analyses, multiple levels of seismic performance criteria and design objectives can be fulfilled simultaneously.

6. ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial supports for this research by the National Natural Science Foundation of China under grant numbers 51427901, 51378341, 51678407, and 91315301, and the National Basic Research Program (Program 973) of China under grant number 2011CB013603.

7. REFERENCES


