Seismic Fortification Intensity Evaluation by a Cost-Benefits Analysis– Case Study of Three Bridges

Tao Xiaxin 1,2, a, Tao Zhengru 2b, Li Dong 2, c
1 School of Civil Engineering, Harbin Institute of Technology, Harbin, 150090, China
2 Institute of Engineering Mechanics, CEA, Harbin, 150080, China
ataoxiaxin@aliyun.com, taozr@foxmail.com, 823571742@qq.com

Abstract. A case study on evaluation of seismic fortification intensity by cost-benefit analysis is presented in this paper. Three bridges designed with intensities VI, VII and VIII are modeled and their seismic responses are calculated by push-over approach, and the damage indices are estimated by a stiffness degradation model. The vulnerability matrices are built from a set of probability density functions and the damage results. In seismic risk cost assessments of the bridges, both direct and indirect losses are taken into account by ratios to the original construction costs. The fortification intensity is evaluated from the minimum sum of the original cost and seismic risk cost in bridge life-cycle. The result shows that the most effective intensities at three sites with different hazards are close to those with exceeding probability 10 % in 50 years, except that intensity VIII is little bit better at a site in intensity VII zone.

1. Introduction
Seismic fortification of building is the main engineering countermeasure against earthquake disaster at present. In general, fortification intensity for seismic design of an engineering structure is chosen from national or regional seismic zoning map. Probabilistic seismic hazard assessment (PSHA) has been developed and widely adopted in compiling of the hazard map and in evaluating of major project sites for about half a century. Cornell emphasized, when he built the foundation and the frame of PSHA, to express seismic risk in terms of return periods from the requirement for seismic design of engineering project, so that engineers can make a trade-off costly between higher resistances and higher risks of economic loss [1]. By this way, engineers could consider the performance of the project under moderate as well as strong motions, and how quickly the risk decreases as the intensity increases, so intensity versus average return period is far more useful than single numbers as the "expected lifetime maximum" or "50-year" intensity. Fifty years later, the fortification intensity now on the zoning map is still just intercepted from hazard curve at that location by PSHA with exceeding probability 10% in 50 years worldwide, while the other valuable information from the curve is almost ignored unfortunately.

A way to improve determination procedure of fortification intensity from hazard curve at a given engineering site by means of cost-benefit analysis is dealt with in this paper as a case study, in which three bridges designed with intensities VI, VII and VIII are evaluated on three sites with different hazards.

2. Cost-benefit analysis framework
In life-cycle cost analysis of engineering structure against natural hazard risk, four costs generally are considered, such as agency cost, user cost, environment cost and risk cost [2, 3, 4]. To simplify the
analysis, just original construction cost in the first and earthquake risk cost in the fourth are taken into account in the cost-benefit analysis for seismic fortification intensity evaluation in this paper. The criterion of the evaluation is to minimize the sum of the two costs, as in the following Eq. 1

\[ C_L = \min(C_o + C_R), \]

where \( C_L, C_o \) and \( C_R \) are the total cost, original construction cost and earthquake risk cost, respectively. The latter two, of course, depend on fortification intensity. The earthquake risk cost can be calculated by Eq.2 (referred to [5])

\[ C_R(I_f) = \sum_{I=1}^{12} \sum_{j=1}^{5} C_o(I_f) L_j P(D_j|I,I_f) P(I), \]

where \( I_f \) is fortification intensity; \( I \) is the intensity in case of happening in the engineering life-cycle; \( C_o(I_f) \) is the original construction cost with \( I_f \); \( L_j \) is loss function to show the loss by the structure with damage grade \( j \); \( D \) is for damage grades with \( j = 1, ..., 5 \), for intact, slightly, moderately, severely damage and collapsed, respectively; \( P(D_j|I,I_f) \) is the probability of the \( j \)th damage grade of the structure with fortification intensity \( I_f \) if \( I \) happens, i. e. the vulnerability; \( P(I) \) is the occurrence probability of intensity \( I \) from the seismic hazard assessment at the site. The loss in Eq.2 is commonly classified into two, direct loss and indirect loss.

**Direct Loss.** It is mainly in physical form, the cost to repair, replacement of component or even reconstruction of the damage structure itself. Obviously it depends on damage grade. For a given bridge, the worse damage is, the more direct loss must be; the higher fortification intensity is adopted, the original construction cost will be, while the lighter damage should be from the same intensity. For a same damage grade, direct loss should be more if higher fortification intensity is adopted. Therefore, the direct loss \( C_d \) is usually defined by a loss ratio \( \gamma \) as in the following equation

\[ C_d(I_f) = \sum_{I=6}^{9} \sum_{j=1}^{5} \gamma_j C_o(I_f) L_j P(D_j|I,I_f) P(I), \]

where the summation for intensity \( I \) is cut down from 6 to 9, since no structural loss in general in intensity less than 6, and the \( P(I) \) of intensity larger than 9 is very small at most sites; \( \gamma_j \) is the ratio of the direct loss of damage grade \( j \), with definition as

\[ \gamma_j = \frac{\text{the cost of repair in damage grade } j \text{ for } I_f}{\text{the cost of relacement for } I_f}. \]

**Indirect Loss.** It is very hard to be assessed in most cases, since function loss of railway network is difficult to be counted from the bridge damage, and the loss of whole society is much more difficult from the railway system function loss. The modern economic society is very complicated so that a shortage of raw and processed material from railway broken down may cause bad losses of enterprises at the next points of production chain, and then even larger loss could be caused from the dependence between industries. Therefore, a ratio \( \eta \) is adopted actually to define indirect loss by the direct ones as in Eq. 5.

\[ C_i(I_f) = \sum_{I=6}^{9} \sum_{j=1}^{5} \eta_j \gamma_j C_o(I_{f0}) L_j P(D_j|I,I_f) P(I), \]
where $\eta_j$ is the ratio of indirect loss to direct one in damage grade $j$, $C_o(I_0)$ is the original construction cost without seismic fortification.

3. Vulnerability of the three bridges

Modeling of the bridge. For the case study, three bridges with standard design with fortification intensities VI, VII and VIII respectively are selected from a practical railway project [6]. The bridges with almost the same geometrical shapes and sizes, are modeled by Midas Civil software tool as independent pier models, shown in Fig. 1. In the modeling, the dead loads of the superstructures, such as gird and so on, are concentrated at the center of pallet as additional mass, and the pile-soil interaction is described by M method. From a natural vibration analysis, it is shown that the first modes are predominate in vibrations of all three bridges, the basic periods are about 0.56 seconds, and the participating mass percentages of all first mode are all more than 72%.

![Fig. 1 Bridge independent pier model for the case study](image)

Damage indices of the bridges from static nonlinear analysis. A stiffness degradation model for seismic damage is adopted, with an improvement of the existing formula for damage index to place the yield point in between grades of intact and slight damage, as shown in Eq. 6.

$$
\begin{align*}
DM &= 0.1 \frac{u_m}{u_y}, \quad \text{for linear range} \\
DM &= 1.0 - 0.9 \frac{k_r}{k_0}, \quad \text{for nonlinear range}
\end{align*}
$$

(6)

where $DM$ is the damage index, $u_m$ and $u_y$ are maximum response displacement and the yielded displacements, $k_r$ and $k_0$ are the degraded and elastic stiffness, respectively. It is obvious that value of $DM$ must be in between 0.0 to 1.0. The damage indices of the three bridges caused by four intensities are calculated from their nonlinear displacement responses.

The capability curves of the three bridges are calculated by a push-over analysis with the Vidic’s strength reduction model [7], as shown in Fig. 2.
The inelastic demand spectra corresponding to the design response spectra for six basic peak accelerations (corresponding to intensities 6, 7, 8 and 9 respectively) are built from the code for seismic design of railway engineering [8]. Then the nonlinear responses of each bridge caused by the six accelerations are calculated by capacity spectrum method [7]. The $DM$ values of the bridges by the accelerations are calculated by Eq. 6, and listed in table 1, in which bridge I, II and III are for the bridges designed with intensities VI, VII (0.15g) and VIII respectively.

![Fig. 2 Capacity curves of the three bridges](image)

$$\text{Table 1 Index values of the three bridges by six accelerations}$$

| Bridge | Basic peak acceleration |
|--------|-------------------------|
|        | 0.05g | 0.10g | 0.15g | 0.20g | 0.30g | 0.40g |
| I      | 0.063 | 0.330 | 0.505 | 0.596 | 0.691 | 0.750 |
| II     | 0.058 | 0.251 | 0.448 | 0.549 | 0.655 | 0.723 |
| III    | 0.049 | 0.097 | 0.369 | 0.481 | 0.597 | 0.665 |

The vulnerability matrices. Probabilistic evaluation model for earthquake damage grade is established with guiding of the membership function in fuzzy set theory, by the means of processing fuzziness in partition of earthquake damage grades and referring three existing partition suggestions listed in table 2.

$$\text{Table 2 Damage grades with } DM$$

| Suggestions | Intact | Slight | Moderate | Severe | Collapsed |
|-------------|--------|--------|----------|--------|-----------|
| From [9]    | 0–0.2  | 0.2–0.4| 0.4–0.65 | 0.65–0.9| 0.9–1     |
| From [10]   | 0–0.1  | 0.1–0.3| 0.3–0.6  | 0.6–0.85| 0.85–1    |
| From [11]   | 0–0.2  | 0.2–0.4| 0.4–0.6  | 0.6–0.9 | 0.9–1     |

Normal probability density function is adopted for each grade with values of the mean and standard deviation in table 3, while the density functions at the two end of $DM$ field are truncated at 0.0 and 1.0 respectively. In the table, $k$ is for the suggestions in table 2, the value at left of “/” is the mean value, the one at right is the standard deviation value.
Table 3 The values of mean and standard deviation of Normal Probability Function for each damage grade

| k | Intact | Slight | Moderate | Severe | Collapsed |
|---|--------|--------|----------|--------|-----------|
| 1 | 0.0/0.2 | 0.3/0.1 | 0.525/0.125 | 0.775/0.125 | 1.0/0.1 |
| 2 | 0.0/0.1 | 0.2/0.1 | 0.45/0.15 | 0.725/0.125 | 1.0/0.15 |
| 3 | 0.0/0.2 | 0.3/0.1 | 0.5/0.1 | 0.75/0.15 | 1.0/0.1 |

Fig. 3 The five probability density function curves

Five probability density functions are built initially by summing up the probabilities from the three functions for each damage grade at each $DM$ value, and then normalized for each damage grade. The result curves are shown in Fig. 3.

The seismic vulnerability matrices of three bridges are calculated from the result $DM$ values in table 1 with the above probability density functions, as listed in table 4, table 5 and table 6.

Table 4 Seismic vulnerability matrix of bridge I (％)

| Intensity | Intact | Slight | Moderate | Severe | Collapsed |
|-----------|--------|--------|----------|--------|-----------|
| VI        | 89.6   | 10.0   | 0.4      | 0.0    | 0.0       |
| VII       | 12.6   | 60.0   | 26.8     | 0.6    | 0.0       |
| VIII      | 0.8    | 0.9    | 57.4     | 39.8   | 1.2       |
| IX        | 0.0    | 0.1    | 8.8      | 76.2   | 14.9      |

Table 5 Seismic vulnerability matrix of bridge II (％)

| Intensity | Intact | Slight | Moderate | Severe | Collapsed |
|-----------|--------|--------|----------|--------|-----------|
| VI        | 90.6   | 9.1    | 0.3      | 0.0    | 0.0       |
| VII       | 24.8   | 65.7   | 9.4      | 0.1    | 0.0       |
| VIII      | 1.4    | 2.9    | 70.6     | 24.5   | 0.5       |
| IX        | 0.0    | 0.1    | 13.9     | 76.4   | 9.5       |

Table 6 Seismic vulnerability matrix of bridge III (％)

| Intensity | Intact | Slight | Moderate | Severe | Collapsed |
|-----------|--------|--------|----------|--------|-----------|
| VI        |        |        |          |        |           |
| VII       |        |        |          |        |           |
| VIII      |        |        |          |        |           |
| IX        |        |        |          |        |           |
4. Seismic risk costs of the bridges at three sites

Occurrence probability of the four intensities at three sites. Three engineering sites with different seismic hazards are selected for the case study. The intensities at the sites with exceeding probability 10% in 50 years are VI, VII and VIII respectively. The probabilities of intensity exceeding VI, VII, VIII and IX at the sites in 50 years are listed in table 7.

| Table 7 Seismic hazards at three engineering sites (exceeding probability in 50 years) |
|---------|-------|-------|-------|-------|
| Site    | VI    | VII   | VIII  | IX    |
| 1       | 0.1608| 0.0149| 0.0000| 0.0000|
| 2       | 0.1733| 0.0816| 0.0198| 0.0000|
| 3       | 0.6302| 0.2855| 0.0677| 0.0100|

The occurrence probabilities of the four intensities in 100 years at each of the sites are calculated from the values in table 7 by means of a procedure presented by Tao (2007), and listed in table 8.

| Table 8 Occurrence probabilities at the three sites in 100 years |
|---------|-------|-------|-------|-------|
| Site    | VI    | VII   | VIII  | IX    |
| 1       | 0.2742| 0.0296| 0.0000| 0.0000|
| 2       | 0.1896| 0.1220| 0.0392| 0.0000|
| 3       | 0.7298| 0.4122| 0.1131| 0.0198|

Loss ratios. The values of direct and indirect loss ratios for the case study are adopted as listed in table 9. The ratios of indirect loss are adopted from the available empirical data and reports in references for two kinds of damage situation, traffic interruption and passable with low speed limit, respectively.

| Table 9 Loss ratios adopted for the case study |
|------|-------|-------|-------|-------|-------|-------|
| Damage grade | Intact | Slight | Moderate | Severe | Collapsed |
| Direct loss $\gamma_i$ | 0.06 | 0.17 | 0.30 | 0.70 | 1.0 |
| Pass slowly | 0 | 0 | 1 | 5 | 10 |
| Indirect loss $\eta_j$ | 0 | 0 | 10 | 20 | 50 |

Seismic risk costs of the bridges. The seismic risk costs of the three bridges at the three sites are calculated by Eq. 3 and Eq. 5 respectively with the values of loss ratios in table 9, the vulnerability matrices in table 4, table 5 and table 6, and occurrence probability of four intensities in table 8. The results are listed in table 10, in terms of the percentage of the corresponding original construction costs without seismic fortification, while the three values at left of “/”, in between of the two “/” and at right of “/” in the table, are the cost of direct loss, indirect loss from passing slowly and traffic interruption respectively.
Table 10 Seismic risk cost of the three bridges at the three sites

| Bridge | Site 1 | Site 2 | Site 3 |
|--------|--------|--------|--------|
| I      | 2.74/0.36/3.29 | 5.97/8.44/57.36 | 21.40/33.94/222.22 |
| II     | 2.76/0.13/1.27 | 5.50/5.17/35.92 | 19.98/22.94/150.98 |
| III    | 2.78/0.03/0.33 | 4.62/2.54/19.12 | 17.12/13.14/90.26 |

5. Result of cost-benefit analysis

**Original construction costs with different fortification intensities.** In general, the higher fortification intensity is adopted, the more construction cost must be, for example more and higher strength steels must applied in this case. The original construction costs of railway bridge for different fortification intensities, $C_0(I_f)$, can be described as in Eq. 7.

$$C_o(I_f) = (1.0 + a(I_f))C^*,$$

where $a(I_f)$ is the increasing coefficient of original cost for fortification intensity $I_f$, and $C^*$ is original construction cost of bridge without any seismic fortification. The values of the increasing coefficient are listed in table 11. It is obviously that the the values are the same as the additional cost in the original construction of the bridges for seismic fortification in terms of percentage of $C^*$.

Table 11 The values of the increasing coefficients

| Increasing coefficient | Fortification intensity |
|------------------------|------------------------|
| $a(I_f)$               | VI         | VII | VIII | IX  |
| 0.075                  | 0.15       | 0.30 | 0.50 |

**The benefit of fortification intensity.** It is taken as the criterion to evaluate the benefits of bridges seismic fortification with intensity VI, VII and VIII that to compare the sums of additional costs in original constructions for the fortifications and the corresponding seismic risk costs in 100 years. The two costs of each bridge at the three sites are listed in table 12.

Table 12 Total costs of the three bridges at the three sites (in terms of $C^*$ %)

| Bridge | Original construction cost | Site 1 | Site 2 | Site 3 |
|--------|----------------------------|--------|--------|--------|
| I      | 7.5 | 6.39 | 71.77 | 277.56 |
| II     | 15  | 4.16 | 46.59 | 193.89 |
| III    | 30  | 3.14 | 26.28 | 120.51 |

One can find from the table that the most effective fortification intensities are usually the same as those determined with the exceeding probabilities 10% in 50 years, only at the site 2 in intensity VII zone, total cost of bridge III fortified with VIII is slightly lower than that of bridge II. It means that further study on cost-benefit analysis for bridges and other structures are noteworthy.

6. Conclusion

In order to deal with the benefit of seismic fortification, a cost-benefit analysis is carried out for a case study of three bridges designed with fortification intensity VI, VII and VIII, by means of the criterion to
minimum sum of original construction cost and seismic risk cost. The result shows that the most effective fortification intensities at two sites are the same as those determined with exceeding probability 10% in 50 years, but the total cost of bridge fortified with intensity VIII at site in intensity VII zone is slightly lower than that of fortified with VII. It means that the further study on procedure to determine fortification intensity is noteworthy.

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