Lifecycle environmental impact and cost analyses of steel bridge piers with seismic risk

Y. Itoh & M. Wada
Department of Civil Engineering, Nagoya University, Nagoya, Japan

C. Liu
School of Architecture and Building, Deakin University, Geelong, Australia

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ABSTRACT: Lifecycle approaches have found their wide applications in analyzing, evaluating and assessing technologies and management methods in the infrastructure systems. While environmental problems such as global warming have become a serious issue in the world, researchers and practicing engineers in civil engineering have to pay attention to environmental impacts as well as function, safety, cost and aesthetics in the whole lifecycle of civil infrastructures. In addition to the normal lifecycle activities accompanied with operation and aging, the effects of natural hazards such as earthquakes with a low occurrence probability but a high hazard loss require a full consideration in determining both lifecycle cost and lifecycle environmental impact. In this research, an approach is proposed to predetermine the lifecycle environmental impact and costs of bridges from their construction and maintenance as well as the losses and recovery after natural hazards. Based on this research, it becomes possible to quantitatively outline the roles of bridge construction, maintenance and earthquake in both environmental impact and cost in the whole lifetime of a bridge, especially their constituent parts from seismic losses and recoveries.

1 INTRODUCTION

Lifecycle assessment (LCA) was originally developed for analyzing the lifecycle of a new type of industrial product from the purchase stage of raw materials, production stage, shipment stage to customer, use stage by final customer, secondary use stage by customer, to the final product disposition stage. It is also used to compare a new industrial product with the available products or with alternative strategy in which at least one stage is of different contents. The early studies to look at lifecycle aspects of products and materials date from the late 1960s and early 1970s (EEA 1998). In 1969, for example, the Coca Cola Company funded a study to compare the resource consumption and environmental releases associated with beverage containers. Meanwhile, in Europe, a similar inventory approach was being developed, which was later known as the Ecobalance. Despite several decades of development, LCA is still a young tool and it does not have a fixed methodology and there is no single way to conduct it. It has to be modified with the specific characters of the product. In addition, LCA is not the only way to assess the lifecycle performance and other methods must sometimes be used to supplement the assessment, such as input-output analysis.

LCA has played an important role in the bridge management as it enables to consider all bridge lifecycle stages and their involved activities at the same time. Most of the previous researchers focused their research efforts on lifecycle cost analysis. Chang and Shinozuka (1996) focused on the development of conceptual models of bridge lifecycle cost. Frangopol et al. (1997) carried out a study on the lifecycle cost based on the deterioration of existing bridge structures. Liu and Itoh (1997) used optimization of maintenance strategies for lifecycle management of network level bridges. Efforts are also made to reduce the lifecycle cost by the use of high performance steel (Wright 1998).

Besides the lifecycle cost, the environmental impact is also important in infrastructure management. Since environmental impact assessment of large projects is made mandatory in many countries, various research efforts are ongoing about the evaluations of environmental impact from infrastructure lifecycle. In a previous study, global environmental impact has been considered as one factor for selecting the bridge type (Itoh et al. 1996). Taking the energy consumption and CO_2 emissions from bridge construction activities as the indicators of global environmental impact, a system was developed to compare the candidate bridge types. Horvath and Hendrickson (1998) considered the comparison of a steel bridge and a reinforced concrete bridge with respect to the environmental impact from the bridge lifecycle.

Various types of environmental effects are caused due to the bridge construction and maintenance activities like the generation of toxic materials, hazardous wastes, local air pollutant emissions and global environmental effects. For example, high performance coating systems were developed to reduce various environmental hazards from bridge paints (Calzone 1998).

Global warming is one major threat to the earth, which is caused by emissions of greenhouse gases. Due to the use of construction equipment and the consumption of fossil fuels during the related activities, emissions of greenhouse gases are caused from the construction activities of a bridge. Therefore, this study focuses on the global environmental effects. Greenhouse gases like CO_2, CH_4, N_2O and so on are emitted during the different activities of bridge lifecycle. These emissions are the consequence of various activities that are dependent upon the consumption of natural resources and industrial activities consuming fossil fuels and energies. Since CO_2 comprises the majority of greenhouse gas emissions, it is considered as the indicator of environmental impact in this study.

Thus, the aim of this paper is to extend a previously developed method of calculating the environmental impact and costs from construction and maintenance stages so that the CO_2 emissions and costs of bridges from the losses and recoveries due to earthquakes can be predetermined. Based on this research, it becomes possible to quantitatively outline the roles of bridge construction, maintenance and earthquake in both environmental impact and cost in the whole lifetime of a bridge, especially their constituent parts from seismic losses and recoveries.

Although difficulties still prevail in predicting the lifecycle CO_2 emissions and costs of bridges with required accuracy, these values would be useful for a comparative analysis because consistent methods are followed in the numerical analyses. Because of the insufficiency of data, simplifications and assumptions were adopted in this research for the bridge lifecycle analyses. The numerical results from this section could become more accurate with the accumulation of the necessary data.

2 LIFECYLE ASSESSMENT OF BRIDGES

There are three types of service lives of a bridge or a bridge component, in terms of structural service life, functional service life and economic service life (Nishikawa 1994). The structural service life is determined according to the deterioration of materials used in each bridge component, and the reduction of the entirety of a bridge. The functional service life of a bridge is the time from the construction to the replacement due to the lack of its function such as the increasing requirements on the loading capacity, the traffic volume, its length, width and height, and the seismic capacity. The economic service life of a bridge is determined according to the economic benefit to keep the bridge open for the service. In this research, the structural service life is taken consideration as the service life of a bridge without a specific definition to avoid the subjective effects in the evaluation of results.

In this research, the bridge lifecycle represents the construction stage, the maintenance stage and the seismic losses and recoveries after earthquakes, which cover the major on-site activities and resource consumption of a bridge. Lifecycle evaluation at the construction stage needs the primary data of a bridge including its cross section data, span arrangement, structure type and others. In the previous research, a bridge type selection system was developed to determine these primary data, and the environmental impact and cost from the construction stage of a bridge with the selected type (Itoh et al. 1996). These outputs are parts of the lifecycle environmental impact and cost of a bridge.

The maintenance requirements and specific techniques of a bridge or its components are determined according to periodic inspection and further testing in detail if necessary (Itoh et al. 2001). The seismic loss and recovery analysis after earthquake needs the inputs of the probabilities of earthquake occurrence, the seismic damage condition prediction and the selection of recovery activities, which will be detailed in the next section

Based on the above assumption on lifecycle, the lifecycle environmental impact and cost could be summed as follows:

$$E_t = E_c + E_m + E_r \tag{1}$$

$$C_t = C_c + C_m + C_r \tag{2}$$

where, E_t and C_t are the environmental impact and cost within the whole lifecycle of a bridge, respectively; E_c and C_c are the environmental impact and cost from the construction stage, respectively; E_m and C_m are the environmental impact and cost from the maintenance stage, respectively; and E_r and C_r are the environmental impact and cost from the seismic recovery after earthquake, respectively.

A previous study detailed the lifecycle assessment in construction and maintenance stages (Itoh et al. 2001). The present paper extends the previously developed lifecycle assessment method to explore the effect of the seismic recovery after earthquake. The following two sections will report the three steps in achieving this objective and the numerical results in an analytical example, respectively.

3 LIFECYCLE ASSESSMENT PROCEDURE OF SEISMIC RISK

In performing lifecycle seismic loss and recovery analysis of steel bridge piers, it is imperative quantitatively to identify (1) the probability of earthquake occurrence, (2) the seismic vulnerability of structures associated with various states of damage, and (3) the estimations of cost and environmental impact of each damage state. The remaining of this section will report the details of these three components, which are based on an existing probabilistic estimation method, newly developed seismic fragility curves, and the real damage data of the 1995 Hyogo-ken Nanbu earthquake, respectively.

3.1 *Modeling probability of earthquake occurrence*

Japan is often portrayed as a country subject to frequent earthquakes. As it is located in the circum-Pacific mobile zone, seismic activities occur constantly. The Meteorological Agency in Japan monitors earthquake activity utilizing a network of seismic intensity indicators and seismographs positioned throughout the country. Based on the previously monitored seismic data, the earthquake engineering committee of the Japan Society of Civil Engineers computed the annual probability of earthquake occurrence in 246 locations in the light of an existing model of predicting earthquake occurrence (JSCE 2000). Furthermore, this committee extracted three fractal hazard curves as shown in Figure 1, in terms of 0.16, 0.50 and 0.84 fractal hazard curves.

These curves should be read in the following manner: for example, the 0.84 fractal hazard curve represents the annual probability that 84% of locations do not exceed for certain a peak ground acceleration. In the following calculation, the 0.5 fractal hazard curve will be adopted to represent the annual probability of earthquake occurrence given the peak ground acceleration if an additional explanation is not given.

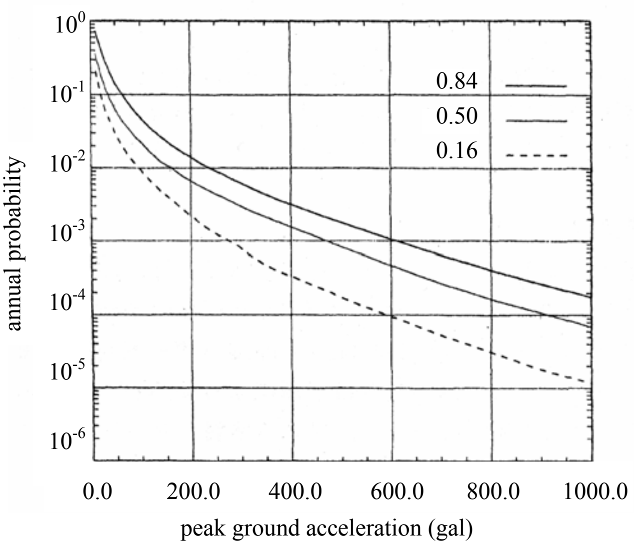

Figure 1. Fractal seismic hazard curves.

3.2 *Development of fragility curves*

The development of vulnerability information in the form of fragility curves is a widely practiced approach when the information is to be developed by accounting for a multitude of uncertain sources involved such as in estimation of seismic hazard (Shinozuka et al. 2000). Fragility curves are therefore functions that represent the probability that a given structure's response to various seismic excitations exceeds performance limit states. Steel piers are much less adopted worldwide in highway bridge systems than in Japan, and this makes their seismic analysis not as well researched as for concrete structures. Under such a circumstance, few fragility curves have been developed for steel bridge piers. The fragility curves constructed in the present paper are on the basis of synergistic use of bridge damage data obtained from the 1995 Hyogo-ken Nanbu earthquake and nonlinear dynamic analysis.

3.2.1 *Seismic response of steel bridge piers*

Since the 1980s, several databases have been developed at Nagoya University to preserve and share experimental data on steel bridge piers acquired at this university and others (Itoh et al. 2002). These databases made it possible and convenient to simulate the seismic performance of steel bridge piers. A total of 15 non-concrete-filled steel box piers with stiffened cross sections are retrieved from existing databases and their specific numerical parameters are shown in Table 1. The symbols h, R_f, $\bar{\lambda}$, and T(s) represent the height of a bridge pier in meters (abbreviated to m), its flange width-thickness parameter, slenderness parameter and the natural period in seconds (s), respectively. Single-degree-of-freedom seismic design is utilized in consideration of the availability of data, and the boundary condition at the foundation is considered as a fixed end.

Table 1. Parameters of experimental and analytical bridge piers.

No.	h (m)	R_f	$\bar{\lambda}$	T (s)
1	3.95	0.30	0.25	0.38
2	5.53	0.30	0.35	0.54
3	7.11	0.30	0.45	0.69
4	8.69	0.30	0.55	0.85
5	10.3	0.30	0.65	1.02
6	3.59	0.45	0.25	0.47
7	8.39	0.45	0.35	0.66
8	10.8	0.45	0.45	0.86
9	13.2	0.45	0.55	1.06
10	15.6	0.45	0.65	1.26
11	8.04	0.60	0.25	0.52
12	11.3	0.60	0.35	0.74
13	14.5	0.60	0.45	0.96
14	17.7	0.60	0.55	1.19
15	20.9	0.60	0.65	1.41

The hysteretic model adopted in the analysis was reported in a previous study (Suzuki et al. 1996). This method gives the relatively accurate prediction and requests a time history record of ground motion to represent the seismic inputs. Three input accelerograms are chosen in order to include ground motions of different characteristics. All of them are the modified ground motion records obtained in the 1995 Hyogo-ken Nanbu earthquake and are adopted in Japanese codes (JRA 1996). Ten peak ground acceleration values are applied for all 15 steel piers shown in Table 1, which start from 100 gal to 1000 gal with an increase of 100 gal. At any one of peak ground acceleration levels, two different ground motions from each of three input accelerograms (totally 6 ground motions) are chosen for a particular steel pier to determine the ratio between the maximum response displacement (δ_{max}) and the yield top displacement (δ_y), δ_{max}/δ_y. Thus, 900 seismic response results are obtained in total and all δ_{max}/δ_y ratios are plotted in Figure 2.

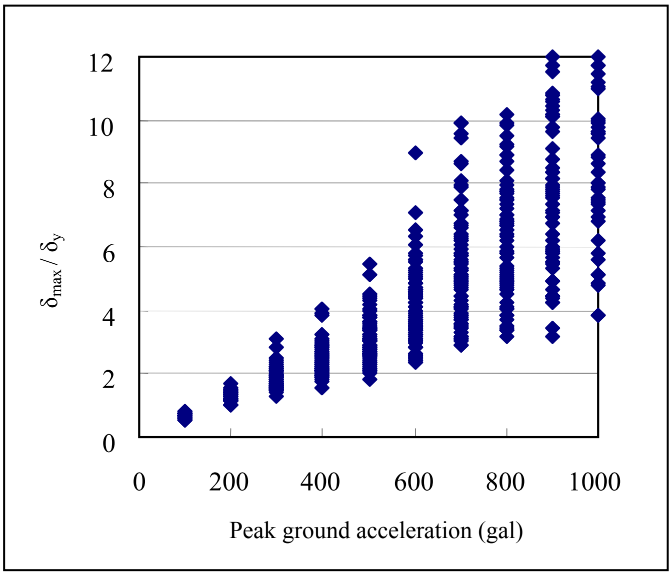

Figure 2. Maximum response displacements.

In bridge systems, the control of residual displacement is likewise significant as that of the maximum displacement. For the practical utilization, the residual displacement of a structure induced by a ground motion, however, is very difficult to accurately calculate by a numerical analysis unless an elaborately developed hysteretic model is available. As the maximum displacement is not sensitive to the hysteretic model, the residual displacement can be obtained by using the relation of the residual displacement to the maximum displacement if existing. In a previous study, the existence of such a relation is verified and corresponding empirical equations have been proposed through the pseudo-dynamic tests on cantilever-type steel thin-walled columns, which are approximately equivalent to a single-degree-of-freedom system (Usami et al. 1998). The equations are given by:

Average curve:

$$\frac{\delta_R}{\delta_y} = 3.37\tan(0.0879(\delta_{\max}/\delta_y - 1)) \geq 0.0 \qquad (3)$$

Lower bound curve:

$$\frac{\delta_R}{\delta_y} = \tan(0.208\delta_{\max y}/\delta_y - 1.46) + 2.7 \geq 0.0 \qquad (4)$$

These two equations allow the residual displacement demand of a single-degree-of-freedom system, δ_R, to be conveniently computed, provided that the maximum displacement demand, δ_{max}, is known. Following these two equations, further calculation is carried out to determine the ratio between the residual displacement (δ_R) and the height of the steel bridge pier (h) for all 900 responses, δ_{max} / h, and the new distribution over peak ground accelerations is plotted in Figure 3. This ratio is a critical indicator to determine the seismic damage state and the potential losses and recovery methods after an earthquake. The values and distributions of all ratios in this figure also enable to construct the fragility curves of steel bridge piers as described below.

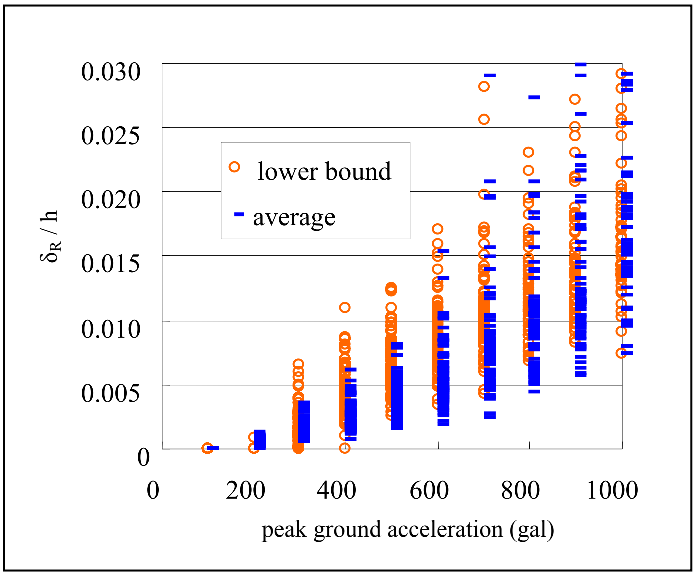

Figure 3. Distribution of residual displacements.

3.2.2 *Fragility curves of steel bridge piers*

It has been widely realized in modern seismic assessment philosophy that a rational seismic assessment method should be able to explicitly reflect the structural and functional conditions, and the potential recovery methodologies as well as their corresponding social, economic and environmental impacts. In this research, the seismic damage condition of a bridge steel pier is assessed to be one of five states, which are quantitatively dominated by δ_{max} / h, the ratio between the residual displacement (δ_R) and the height of the bridge pier (h). By referring to a research report released by Japan Society of Civil Engineers (JSCE 1996), Table 2 describes the δ_{max} / h ratios and general descriptions of the five seismic damage states.

Table 2. Seismic damage states.

Damage state	$\dfrac{\delta_{max}}{h}$	Seismic damage condition description
A_s	>1/100	The bridge is seriously damaged and reconstruction is required.
A	>1/150 and ≤1/100	The structural damage is serious and the function is completely lost. It needs more than 2 months to recover.
B	>1/300 and ≤1/150	The structural damage is obvious, but the minimum function for emergent usage is achievable. The potential recovery duration is from 2 weeks to 2 months.
C	>1/1000 and ≤1/300	The function is not damaged obviously and the damage can be repaired within a couple of days.
D	≤1/1000	The structural damage isn't obvious and specific repair isn't recommended.

In the light of δ_{max} / h ratios generated from the 900 seismic analyses of 15 steel bridge piers at 10 peak ground acceleration values, which are plotted in Figure 3, research is further carried out to study the distribution of these ratios at certain peak ground acceleration points. Based on a trial and error approach to examine with popular distributions, it is found that a logarithmic normal distribution precisely presents all the 90 ratios at most peak ground acceleration points. Taking the 500 gal as an example, Figure 4 shows the number of calculations located in a section of δ_{max} / h ratios as well as the estimated logarithmic normal distribution curve. The total number in the calculation pool is 90.

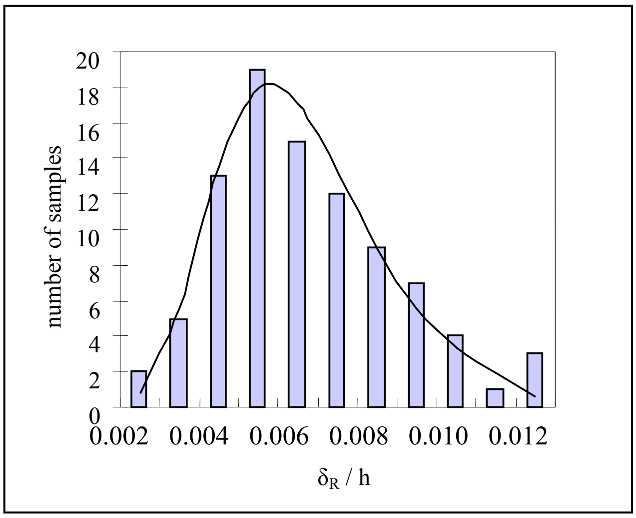

Figure 4. Histogram of residual displacement.

According to the distribution of δ_{max} / h ratios of steel bridge piers at each peak ground acceleration point, five probabilities can be estimated, which represent the fact that these ratios are larger than 1/100, between 1/150 and 1/100, between 1/300 and 1/150, between 1/1000 and 1/300, and less than 1/1000. These five probabilities also indicate the potentials that a steel bridge pier will be damaged at the category states A_s, A, B, C and D after an earthquake at the given peak ground acceleration value. As the residual displacement may be determined using both the average and lower bound curves in Eqs. (4) and (5), both an average probability and a lower bound probability exist for each seismic damage category. It should be noticed that these five probabilities, no matter the average or lower bound, amount to 1. In addition, the accumulated probabilities being greater than 1/100, 1/150, 1/300 and 1/1000 can be calculated for both the average and lower bound cases and these four groups of probabilities are linked with the increase of maximum acceleration. These probabilities connected by lines, which are shown in Figure 5, are also called fragility curves.

Obviously, the fragility curves are a measure of performance in probabilistic terms to present the possibility that a given structure's response to various seismic excitations exceeds performance limit states. It should be highlighted that these fragility curves are constructed on the basis of the damage data associated with the 1995 Hyogo-ken Nanbu earthquake and a dynamic analysis on the seismic response of steel bridge piers under a hysteretic model.

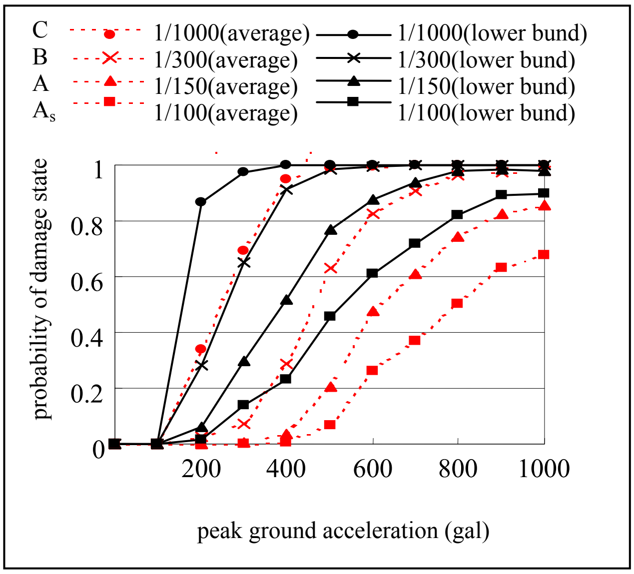

Figure 5. Fragility curves of steel bridge piers.

If no specific explanation is offered, the average fragility curves are adopted in the following analyses. Figure 6 indicates the probabilities that each seismic damage category may happen given the peak ground acceleration. In addition, it should be noticed that the difference between a point in the curve of seismic category C and 1 is the probability that the δ_{max} / h ratio is less than 1/1000, namely seismic damage category D. As this category does not request specific seismic repair and the bridge fully functions after the earthquake, no cost or environmental impact is added to the lifecycle assessment. Therefore, this seismic category is not considered in the following numerical analyses of this research.

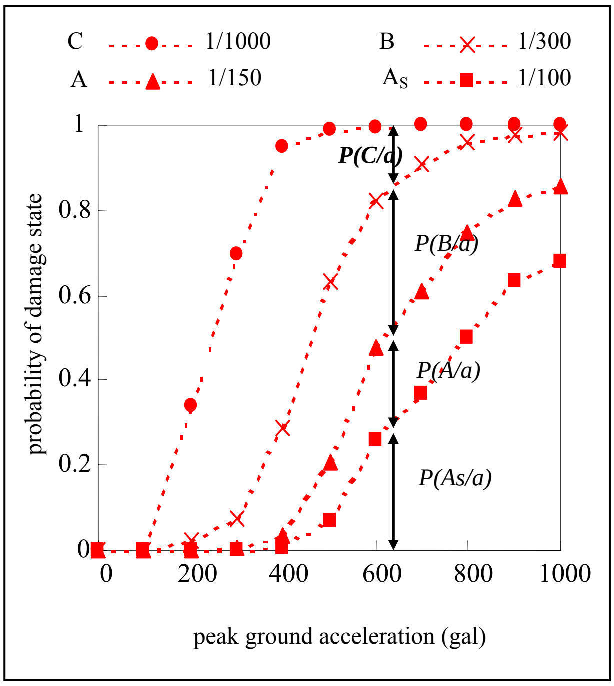

Figure 6. Damage probability of fragility curves.

3.3 *Lifecycle environmental impact and cost formulations of seismic risk*

As mentioned above, the seismic hazard risk is estimated on the basis of the seismic hazard curves and fragility curves, in terms of the probability of earthquake occurrence at certain a peak ground acceleration level and the corresponding probabilistic response of a bridge steel pier respectively. If the seismic hazard curve h(a) is represented as a function of the maximum acceleration *a*, the probability of earthquake occurrence H(a) at the peak ground acceleration *a* can be formulated as:

$$H(a) = -\frac{dh(a)}{da} \tag{5}$$

Following the seismic damage probabilities given the peak ground acceleration as shown in Figure 6, the probability of seismic damage occurrence can be represented as:

$$P_D = H(a)(P(As/a) + P(A/a) + P(B/a) + P(C/a)) \tag{6}$$

or

$$P_D = -\tfrac{dh(a)}{da}(P(As/a) + P(A/a) + P(B/a) + P(C/a)) \tag{7}$$

If the seismic recovery cost can be estimated for each seismic damage state, the potential annual seismic damage cost will be a function of the peak ground acceleration:

$$\sum P_D C_D = \int_0^{1000} -\tfrac{dh(a)}{da}\{(P(As/a)\cdot C_{As} + P(A/a)\cdot C_A + P(B/a)\cdot C_B + P(C/a)\cdot C_C\}da \tag{8}$$

where, C_{As}, C_A, C_B and C_C are the estimated seismic recovery costs for the seismic damage at states A_s, A, B and C, respectively. Similarly, if E_{As}, E_A, E_B and E_C represent the estimated seismic recovery environmental impact for damage states A_s, A, B and C respectively, the seismic recovery environmental impact can be formulated as:

$$\sum P_D E_D = \int_0^{1000} -\tfrac{dh(a)}{da}\{(P(As/a)\cdot E_{As} + P(A/a)\cdot E_A + P(B/a)\cdot E_B + P(C/a)\cdot E_C\}da \tag{9}$$

It should be noticed that the probabilities given in this section are annual values. In case of LCA, the service life must be considered. In addition, the proposed formulation is based on the peak ground acceleration and it is rational to conclude that other intensity measures of the seismic ground motion such as peak ground velocity, spectral acceleration and spectral intensity can also be used.

4 A NUMERICAL EXAMPLE

4.1 *Basic assumptions*

In order to develop an equitable comparison, the costs and environmental impacts of construction and maintenance stages as well as the seismic hazard risk are calculated in relative ratios. First, both cost and environmental impact in the construction stage are proposed as 100 units. A unit is an elemental constituent to measure either cost or environmental impact. Furthermore, only painting of steel bridge piers is considered as the potential maintenance activity and the service life of the coating system is assumed to be 20 years. The seismic recovery method is assumed to completely depend on the seismic damage state classified in Table 2.

To determine the environmental impact and cost of a seismic recovery activity as well as a maintenance activity, the consideration of use of materials and machinery of each operation is essential. However, such data have not been summarized well so as to be able to utilize them for further calculation. Therefore, the environmental impact and cost from the maintenance stage and seismic recovery are assumed to be constant without considering the possible change due to the different conditions. Considering this assumption, the individual values are still difficult to estimate from every section in detail. In this research, only the environmental impact from the operation phase materials, painting, welding, machinery and transportation is considered, and the operation cost is obtained from interviews. The authors of this paper interviewed several practicing engineers who were in charge of the seismic recovery after the Hyogo-ken Nanbu earthquake and collected the recovery cost of a range of replacement, rehabilitation and repair operations. It is surprising to find that the seismic recovery costs largely differ from project to project even though similar methods are applied. After a rough sample screening to remove the isolations of recovery costs, an average value is generated to be the seismic recovery cost for each recovery method. Both environmental impacts and costs of all seismic damage categories are summarized in Table 3.

Table 3. Environmental impact and cost of seismic recovery.

Damage level	Seismic recovery	Co_2 emission	Recovery cost
A_s	Replacement (3months)	101	120
A	Rehabilitation (2 months)	27	43
B	Rehabilitation (2 weeks)	18	21
C	Repair (some days)	0.9	1.7

4.2 *Lifecycle assessment implementation*

It is theoretically ideal to use infinity as the analytical lifecycle, but it cannot be accepted from the viewpoint of bridge engineering. The service life of 100 years is therefore used in this paper. It is widely noted that the discount rate has a large effect onto the results of LCA. However, it is very difficult to predict the exact values of discount rate at each year within the lifecycle because of the long cycle and the difficulty of long-term prediction. The commodity prices usually increase year by year due to inflation, which plays an opposite role in LCA to the discount rate. Therefore, a relatively low discount rate of 2% as well as 0% (no discount rate) is adopted in this research. In addition, no discount rate is applied to the environmental impact, which is a common approach in LCA at the time being.

The effect of the discount rate is shown in Figure 7 in the ratios to construction cost and environmental impact. Both average and lower bound curves of residual displacement formulated in Eqs. (3) and (4) respectively are used to calculate the cost and environmental impact of seismic hazard risk, which are numbered in the figure. The discount rate lessens the values of both maintenance and seismic hazard risk costs. The seismic recovery cost may culminate in 32.2 units of the construction cost if the lower bound curve is applied for the residual displacement with no consideration of discount rate. As far as the environmental impact, the seismic recovery may cause 6.3 and 25.7 units in case of the average and lower bound curves, respectively. Apparently, the lower bound curve brings about larger costs and environmental impact because of the severe seismic fragility damage.

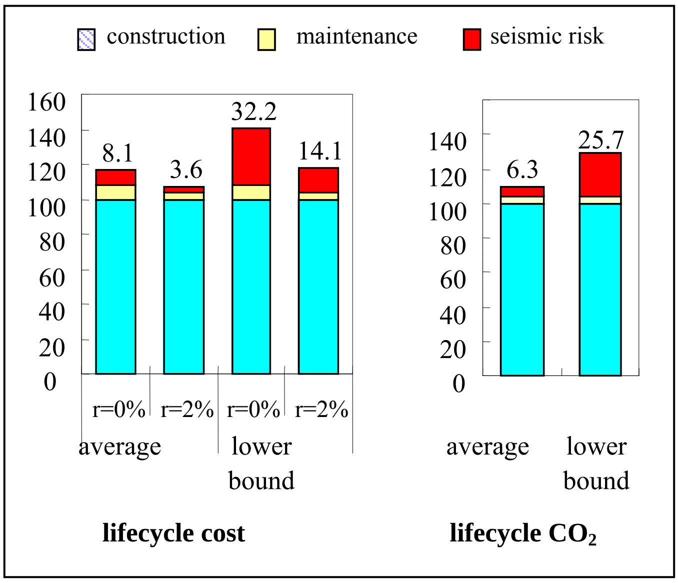

Figure 7. LCC and $LCCO_2$.

Further research has been carried out to study the effects of the seismic hazard curves and the fragility damage risks. The scenario is that the discount rate is 2% and the average curve of residual displacement is adopted. Figure 8 shows the lifecycle costs in conformity to the three fractal seismic hazard lines in Figure 1. Although the seismic recovery costs in three cases are not large due to the effect of the discount rate, the changes are very obvious.

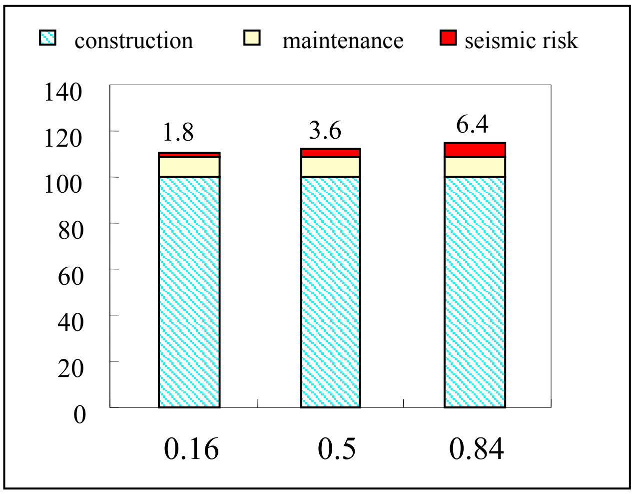

Figure 8. Differences of lifecycle costs in fractal hazard curves.

The distribution of lifecycle cost over the four seismic damage states is shown in Figure 9. It is found that nearly one-half of the seismic recovery cost over the lifecycle of 100 years is caused by the seismic damage state B. Although the recovery costs of damage categories A_s and A are assumed to be six and two times that of damage category B, their lifecycle cost percents are only 16.2 and 15.8, respectively. This is because of their lower damage probabilities, especially while the maximum acceleration is low. In fact, as shown in Figure 1, the probability of earthquake occurrence drastically decreases with the increase of the maximum acceleration. In addition, similar results are found for no discount rate and the lower bound curve of residual displacement although the specific amounts are different.

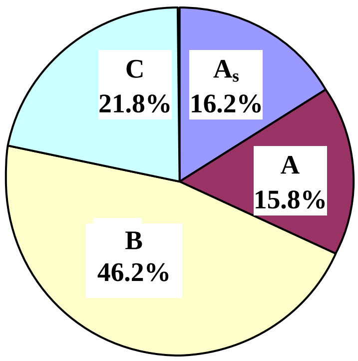

Figure 9. Shares of seismic recovery cost in damage states.

5 CONCLUSIONS

This paper has extended an approach previously developed for lifecycle environmental impact and cost analyses in construction and maintenance stages to include the seismic recovery after earthquake. The environmental impact and cost estimation procedure for seismic hazard risk was proposed in this research based on an existing probabilistic estimation method of earthquake occurrence, newly developed seismic fragility curves, and the real damage data of previous earthquakes. The investigation led to the following observations:

(1) Fragility curves are originally developed for steel bridge piers with consideration of residual displacements. Thus, a probability-based evaluation approach for seismic damage risk is made possible for steel bridges according to fragility curves in combination with seismic hazard curves.

(2) As far as the environmental impacts and costs of seismic recovery operations vary with the discount rates, fragility curves, and hazard curves, it was found from the numerical example that they are generally small compared to those from the construction stage. In addition, the cost distribution over seismic damage states presents that about one-half of lifecycle cost for seismic recovery operations is spent on damage category B because of the probabilistic distributions of both seismic hazard and fragility curves.

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