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LIFE-CYCLE DESIGN OF BRIDGES IN SEISMIC REGIONS BASED ON LESSONS FROM THE 2011 GREAT EAST JAPAN EARTHQUAKE

Mitsuyoshi Akiyama[1], and Dan M. Frangopol[2]

[1] Department of Civil and Environmental Engineering, Waseda University
3-4-1 Shinjuku-Ku Okubo, Tokyo 169-8555, Japan
e-mail: akiyama617@wasada.jp

[2] Department of Civil and Environmental Engineering, ATLSS Engineering Research Center,
Lehigh University, 117 ATLSS Drive, Bethlehem, PA 18015-4729, USA
e-mail: dan.frangopol@lehigh.edu

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Abstract. *Many bridges in Tohoku region of Japan were damaged and collapsed due to strong ground motions and/or liquefaction during the 2011 Great East Japan earthquake. Bridge superstructures were washed away due to the subsequent tsunami. In addition, bridges in the coastal area deteriorated further due to chloride induced corrosion. It is important to note that the seismic events can cause multiple disasters. The bridges located near the coastline in Japan need to be designed taking into consideration the risk associated with seismic and tsunami hazards, and continuous deterioration. This paper provides a key aspect of life-cycle design of bridges under multiple hazards, with an emphasis on earthquake, tsunami and continuous deterioration based on lessons learned from the 2011 Great Japan earthquake. A simple durability design criterion of reinforced concrete (RC) bridge in a marine environment is proposed to determine the concrete quality and concrete cover to prevent the chloride-induced reinforcement corrosion causing the deterioration of structural performance during the whole lifetime. In addition, reliability-based capacity design of bridges with a hierarchy of resistance of the various structural components necessary to avoid catastrophic damage and to ensure prompt restoration after an extreme event is discussed.*

1 INTRODUCTION

The 2011 Great East Japan earthquake (i.e. Off Pacific Coast of Tohoku Region, Japan earthquake) with the magnitude of 9.0 occurred at 2:46 p.m. (local time) on March 11, 2011. The fault zone extended 450 km and 200 km in the north-south and west-east directions, respectively [1]. Extensive damage occurred in a wide region in the east Japan. In Tohoku Region, where the effects of the seismic shocks and the tsunami waves due to the 2011 Great East Japan earthquake were felt with very high intensity, many structures and infrastructures were severely damaged due to the strong ground motion or washed away by the giant tsunami.

Bridges may be susceptible to damage during an earthquake event, particularly if they were designed without adequate seismic detailing. Structures built using earlier design specifications (i.e., without proper seismic detailing) often lack adequate flexural strength and ductility capacity, and/or shear strength. When subjected to strong ground motions, these structures have the potential to exhibit brittle failure. Several destructive earthquakes in Japan (e.g., 1978 Miyagiken-Oki earthquake, 1995 Hyogoken-Nanbu earthquake, 2003 Sanriku-Minami earthquake, and 2004 Niigataken-Chuetsu earthquake) inflicted various levels of damage on the structures and infrastructures. The investigation of these negative consequences gave rise to serious discussions about seismic design philosophy and to extensive research activity on the retrofit of as-built bridges. The seismic design methodology for new bridges has been also enhanced. Comparing the damage state of bridges before and after the seismic retrofits, or the performance of bridges designed according to old and latest seismic specifications during the 2011 Great East Japan earthquake, the effectiveness of seismic retrofit and upgrade of seismic specification against the strong ground motions could be investigated.

The giant tsunami due to the 2011 Great East Japan earthquake inflicted substantial damage to many coastal communities in Japan, including their critical port facilities, residential and commercial buildings, and infrastructures. Although several past earthquakes generated the tsunamis, extensive damage did not occurred to the transportation facilities in Japan. The tsunami effects have not been taken into consideration in the seismic design and retrofit specifications. The code provisions against tsunami need to be established based on the failure mechanisms gathered from the tsunami induced damage effects on bridges.

A team of structural and bridge engineers from the Japan Society of Civil Engineers (JSCE) visited the Tohoku Region shortly after the 2011 Great East Japan earthquake to investigate the performance of structures during the disaster [2]. Based on their field investigation, Akiyama and Frangopol reported the following lessons from the 2011 Great East Japan earthquake [3].

- Since the 1995 Hyogoken-Nanbu earthquake, seismic retrofit has been conducted for RC bridge piers which have insufficient shear reinforcement and/or have cut-offs of longitudinal rebars without adequate anchorage length. However, there were still a significant number of bridge piers which required retrofitting before the 2011 Great East Japan earthquake. As a result, some as-built bridges exhibited similar failure modes as observed in the past earthquakes.
- Comparing the damage state of bridges before and after the seismic retrofits, or the performance of bridges designed according to old and latest seismic specifications during the 2011 Great East Japan earthquake, the effectiveness of seismic retrofit and of the improved seismic specifications against the strong ground motions was proved.
- Several bridges were not washed away even though they were engulfed by the tsunami. These bridges have several common structural features; concrete superstructures with a wider width of the road and shallow girder height, or RC moment-resisting frames.

Based on these structural features, the code provisions against tsunami effects need to be established.

- Some of RC structures and steel bearings were severely deteriorated due to chloride-induced corrosion. The effects of corrosion on the deterioration of the capacity of bridges under seismic and tsunami hazards have to be considered.

Based on the above lessons, the reliability-based life-cycle design of bridges under multiple hazards is discussed in the present paper. This consists of reliability-based durability design of bridges to prevent the material deterioration from the chloride attack, and reliability-based capacity design of bridges with a hierarchy of resistance of the various structural components necessary to avoid catastrophic damage and to ensure prompt restoration after an extreme event.

2 2011 GREAT EAST JAPAN EARTHQUAKE

In Japan, the first seismic design code which included the seismic analysis taking into consideration the inelastic bridge behavior was issued in 1990. The Hyogoken-Nanbu earthquake of January 17, 1995, caused destructive damage to bridges. For this reason, the design code for road bridges was revised in 1996 [4]. Kawashima reported the details of the evolution of seismic design code for road bridge in Japan [1]. Since the 1995 Hyogoken-Nanbu earthquake, seismic retrofits have been conducted for the as-built bridges. However, because there have been still a number of bridges which required retrofitting and several large earthquakes with the magnitude larger than 6.5 have occurred since the 1995 Hyogoken-Nanbu earthquake, some as-built bridges have been damaged. Figure 1 shows RC columns of Shinkansen viaducts failed in shear during the 2003 Sanriku-Minami earthquake and damage of RC bridge piers due to the insufficient anchorage length at the cut-off point of longitudinal rebars during the 2003 Tokachi-Oki earthquake. These RC columns were not retrofitted before the occurrence of these earthquakes.

Figure 1. Damages to as-built RC columns observed in the recent earthquakes in Japan [5]

The retrofitted columns of the viaducts performed well, with almost no damage during the 2011 Great East Japan earthquake as shown in Figure 2. The effectiveness of seismic retrofitting using steel jacketing to prevent significant damage to the Shinkansen viaducts was proved. Field investigation conducted after the 2011 Great East Japan earthquake also demonstrated a significant improvement of the energy dissipation capability of the as-built structures to which the seismic devices were attached as shown in Figure 2. This resulted in substantial longitudinal displacement reductions and under certain conditions in reduction of inertia forces. It is necessary to establish a seismic retrofit strategy to improve the seismic performance of existing bridges in order to minimize the difference between their seismic performance and the performance required according to the latest seismic specifications. Seismic hazard, importance of bridge, failure mode of components, and seismic specifications used need to be considered when determining priorities for seismic retrofit [6].

Figure 2. Effectiveness of seismic retrofitting using steel jacketing and dampers to as-built RC columns

Figure 3. Corrosion of bearings and reinforcing bars attacked by chloride

Some of RC structures and steel bearings were severely deteriorated due to chloride-induced corrosion, as shown in Figure 3. The reinforcing bars in the RC bridge pier showed pitting corrosion. Pitting corrosion is a highly localized form of steel corrosion, and this can cause significant reductions in cross section areas of reinforcing bars, resulting in the reductions of structural capacity [7]. Mechanical model to consider the effect of corrosion on the seismic behavior of steel bearings needs to be improved. Akiyama et al. [8] presented the life-cycle reliability of RC structures under seismic hazards and hazards associated with airborne chloride. They pointed out in an illustrative example that even if the failure probability of RC bridge pier is lower at the beginning (immediately after construction), it would increase with time due to the chloride-induced corrosion. Material corrosion strongly affects the seismic performance of concrete structures. Since it is very difficult to predict the spatial distribution of material corrosion by inspection and/or non-destructive test, the estimated seismic performance of deteriorated bridges must involve large uncertainty. This would increase the life-cycle cost of bridges. For new structures, it is necessary to determine the concrete quality and concrete cover to prevent the chloride-induced reinforcement corrosion causing the deterioration of structural performance during the whole lifetime of RC structures.

As indicated previously, many superstructures were washed away by the tsunami waves from the 2011 Great East Japan earthquake. The failure modes of bridges due the tsunami wave including the washout of the superstructures due to the hydrodynamic uplift force and the horizontal hydrodynamic pressure and collapse of the bridge piers due to the horizontal load are reported in [3, 9, 10]. Tsunami forces caused the collapse of a number of bridges that formed a vital link between towns in the Tohoku Region.

Abutment backfill of Niju-ichigahama bridge with simple span and steel pile foundation was completely destroyed under the tsunami wave. However, since the original bridge had minor damage due to the tsunami wave, temporary bridges could be constructed within one month after the 2011 Great East Japan earthquake as shown in Figure 4. Temporary bridges were used for the evacuation of affected people and the transportation of emergency goods and materials. The code provisions against tsunami effects need to be established by taking into consideration the recovery time to restore the functionality of the bridge.

Figure 4. Desired tsunami induced-damage to bridge in terms of the recovery time to restore the functionality

3 LIFE-CYCLE DESIGN OF BRIDGES IN SEISMIC REGIONS

The 2011 Great East Japan Earthquake demonstrated that seismic events can cause multiple disasters, including damage to structures due to strong ground motions and/or liquefaction and the washout of structures due to subsequent tsunamis, fires, and landslides. In addition, bridges in the coastal area deteriorated further due to chloride induced corrosion. A probabilistic framework for quantifying the life-cycle reliability of bridges considering the occurrence of a mainshock and subsequent cascading hazard events (i.e., tsunami and aftershocks), and the deterioration of structural performance is needed to ensure satisfactory performance of bridges during their lifetime.

Reliability-based life-cycle design of the new RC bridges under seismic and tsunami hazards, and hazard associated with airborne chloride is discussed herein. The life-cycle reliability of existing bridges under multiple hazards is outside the scope of the current investigation. The following aspects are considered:

- Corrosion is initiated by chloride contamination if the structures have poor quality concrete and/or inadequate concrete cover. Corrosion initiation could lead to cracking due to corrosion products and concrete cover spalling. Cracking and/or spalling accelerate the corrosion rate and finally lead to serviceability failure and a deterioration of long-term structural performance. To reduce the life-cycle cost of bridges, it is very important to prevent the steel corrosion or cover concrete spalling from the chloride attack even if the bridge is located in an aggressive environment. In order to do this, a simple reliability-based durability design criterion of RC structures in a marine environment with durability design factors that satisfy the target reliability level is formulated.

- To ensure functionality of the transportation network after an event, an optimal hierarchy of resistance of the various bridge components has to be identified depending on the hazard environment. To establish the reliability-based capacity design of bridges under seismic and tsunami hazards, the effects of the ratio of lateral strength of bridge pier to that of pile foundation, and the ratio of lateral strength of bearing to that of bridge pier on the seismic and tsunami reliabilities are investigated.

3.1 Reliability-based durability design of concrete bridges in a marine environment

Due to the increase of steel corrosion after the cracking of concrete cover, the structural performance of RC structures may be rapidly diminishing. This indicates the need for an assessment of existing safety, repair or replacement of damaged structural elements, or the need for more frequent inspections. All these cases will require the allocation of additional financial resources [11]. It is necessary to design RC structures with high concrete quality, adequate concrete cover, and additional preventative measures (e.g. epoxy coated bars). These requirements tend to increase the initial cost of concrete structures; however, expected reductions in maintenance and repair costs can justify their use on a life-cycle cost basis.

Li [12, 13] proposed a performance-based assessment criterion provided by

$$P[R(t) \geq R_a] \geq p_a \tag{1}$$

where P = probability of an event, $R(t)$ = structural resistance varying with time t, i.e., deterioration, R_a = minimum acceptable resistance, and p_a = minimum acceptable probability of occurrence of the event $R(t) \geq R_a$.

Using Monte Carlo Simulation (MCS), it is possible to ensure that probability P in Equation (1) is close to the target value p_a, or that the reliability index is close to the target reliability index. However, practical design would require complex reliability computations. A durability design criterion is presented such that the reliability is close to the target value without performing a complex reliability analysis by the designers. The designers determine the concrete cover and concrete quality (water to cement ratio = W/C) by prescribing the time T_s larger than the lifetime T_d of the structure. In the present paper, since RC structures which will not need any future maintenance are designed, T_s has to be determined to be the time to corrosion induced concrete cracking. The equation of T_s has a durability design factor taking into account the uncertainties in the computation of T_s. The formulations are as follows [14, 15]:

$$T_d \leq \phi T_s \tag{2}$$

where

$$T_s = T_1 + T_2 \tag{3}$$

$$C_{0,d}\left\{1-\mathrm{erf}\left(\frac{0.1 \cdot c_d}{2\sqrt{D_{c,d}T_1}}\right)\right\} = C_{\mathrm{lim},d} \tag{4}$$

$$T_2 = \frac{Q_{cr,d}}{V_{d,2}} \tag{5}$$

$$C_{0,d} = 4.2 r^{0.25} u^{0.1} d^{-0.25} \tag{6}$$

$$\log D_{c,d} = -6.77(\mathrm{W/C})^2 + 10.10(\mathrm{W/C}) - 3.14 \tag{7}$$

where ϕ = durability design factor, c_d = concrete cover prescribed at design stage, $C_{lim,d}$ = critical threshold of chloride concentration prescribed at design stage, r = ratio of sea wind (defined as the percentage of time during one day when the wind is blowing from sea toward land), u = average wind speed, d = distance from the coastline, $Q_{cr,d}$ = amount of steel weight loss associated with the occurrence of corrosion induced concrete cracking prescribed at design stage, and $V_{d,2}$ = averaged corrosion rate prescribed at design stage associated during T_2.

Water to cement ratio and concrete cover can be determined by Equation (2) independent of bridge location (i.e. differences in marine environment). The procedure to determine the durability design factor is based on code calibration such that Equation (8) is minimized [14]:

$$U = \sum_i \left(\beta_{\text{target}} - \beta_i(\phi)\right)^2 \tag{8}$$

where β_{target} = target reliability index, and $\beta_i(\phi)$ = reliability index of each structure at location i transformed from the probability P provided by Equation (1).

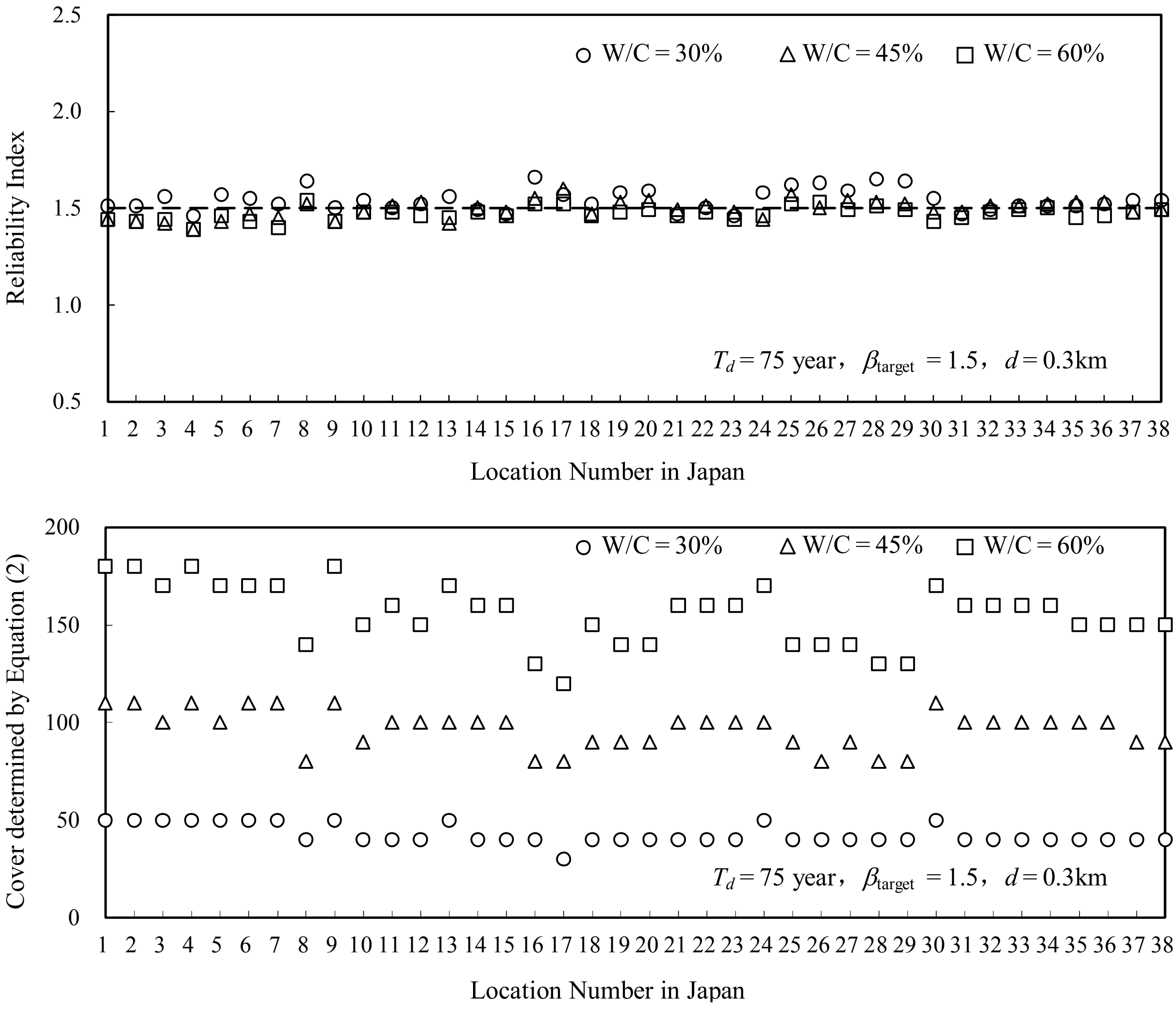

Figure 5. Reliability index and cover of RC structures designed using Equation (2) [15]

The details of steps and random variables used in Equation (1) are shown in [14, 15]. When T_d =75 years, β_{target} =1.5, d = 0.3 km and 38 locations in Japan are selected (i.e., i = 1, 2, 3, ..., 38 in Equation (8)), the durability design factors ϕ minimizing U is 0.91. These reliability indices of RC structures designed by Equation (2) with ϕ = 0.91 are indicated in Figure 5. The reliability indices are very close to the target values independent of water to cement ratio and bridge locations. Figure 5 also shows that the designed concrete cover of each structure in location i (i = 1, 2, 3, ..., 38) depends on the marine environment.

When a RC structure in a marine environment is designed by Equation (2), it is not necessary to consider the effect of material corrosion on the deterioration of structural performance. The hierarchy of resistance of the various structural components will be discussed in the following sections assuming that material properties of concrete and rebars would not be varied during the lifetime of the structure.

3.2 Reliability-based capacity design of bridges under seismic hazard

By allowing one part of a structural system to be damaged and keeping the other parts behaving elastically, a structure can dissipate a significant amount of the input energy from a severe earthquake [16]. The accessibility of the plastic hinges for future inspection and repair must also be accounted for when selecting the location of the hinges. Normally, hinges are placed at the bottom of the piers for maximizing post-event operability and minimizing the cost of repairing bridges after a severe earthquake. Figure 6 shows the plastic hinge introduced at the bottom of RC bridge pier. This bridge pier was damaged during the 2003 Tokachi-Oki earthquake.

To carry out a capacity design based on reliability concepts and methods, it is necessary to satisfy the following three conditions under uncertainties: first, the hierarchy between the flexural and shear capacity ensures the ductile failure mode of the components; second, the capacity hierarchy between components induces the plastic hinge to the appropriate component; and finally, the probability of failure is at most equal to some specified value. To satisfy these design conditions, some design criteria using three partial factors determined by a seismic reliability analysis are proposed, as follows [17]:

$$\gamma_{\mathrm{I}} \frac{M/a}{V} \leq 1 \tag{9}$$

$$\gamma_{\mathrm{II}} \frac{P_a}{P_F} \leq 1 \tag{10}$$

$$\gamma_{\mathrm{III}} \frac{\delta_{p,d}}{\delta_u} \leq 1 \tag{11}$$

where M and V are the design flexural capacity and the shear capacity, respectively, of a given component (i.e., the pier or pile), a is the shear span of the component, P_a is the lateral capacity of the pier (i.e., the ratio of the design flexural capacity M to the shear span a of the pier), P_F is the design yield capacity of the pile foundation, $\delta_{p,d}$ and δ_u are the design displacement ductility demand and the design displacement ductility capacity of the bridge pier, respectively, and γ_{I}, γ_{II}, and γ_{III} are partial factors.

Figure 6. Desired damage of RC bridge column due to the strong ground motion to sustain the actions from emergency traffic and to perform inspection and repair easily [5]

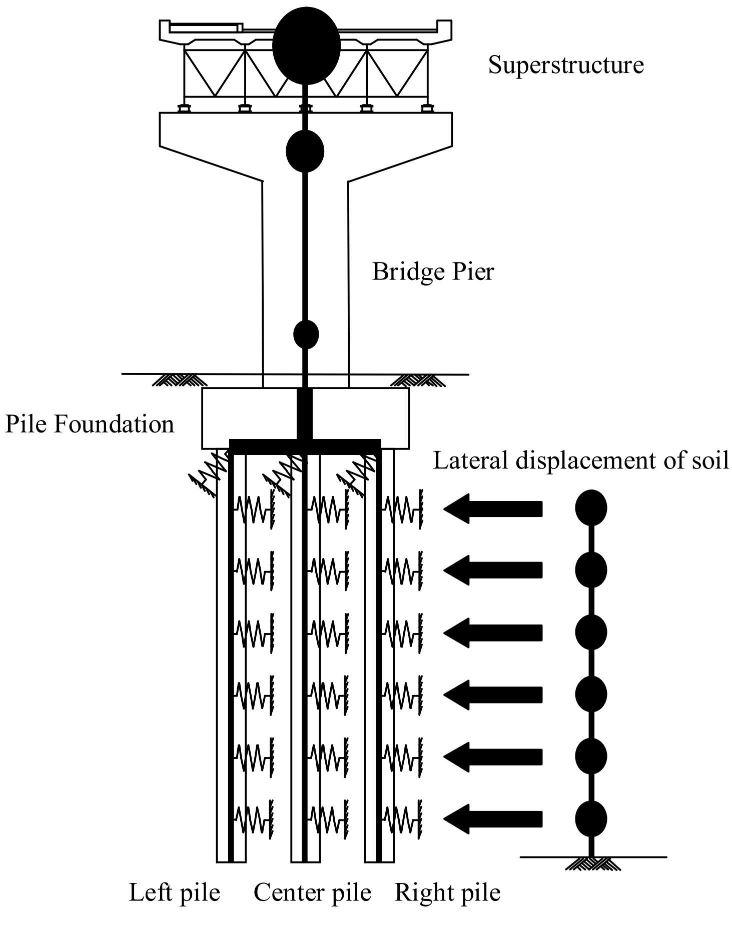

Figure 7. Analytical model for time-history analysis

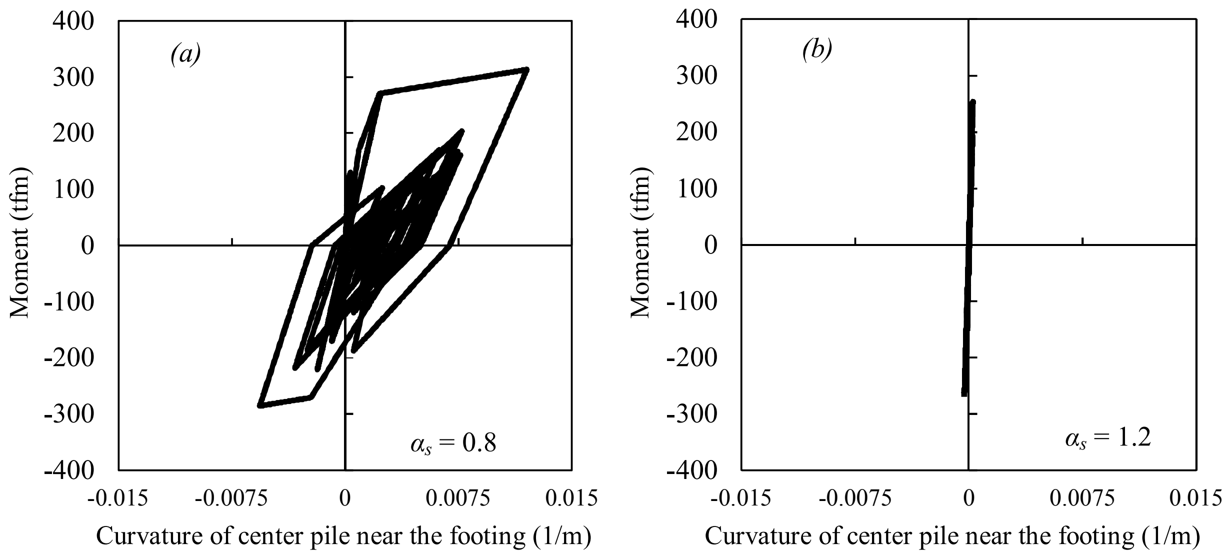

Figure 8. Relationship between moment and curvature of piles ((*a*) when lateral strength of bridge pier is larger than that of pile foundation; and (*b*) when lateral strength of bridge pier is less than that of pile foundation)

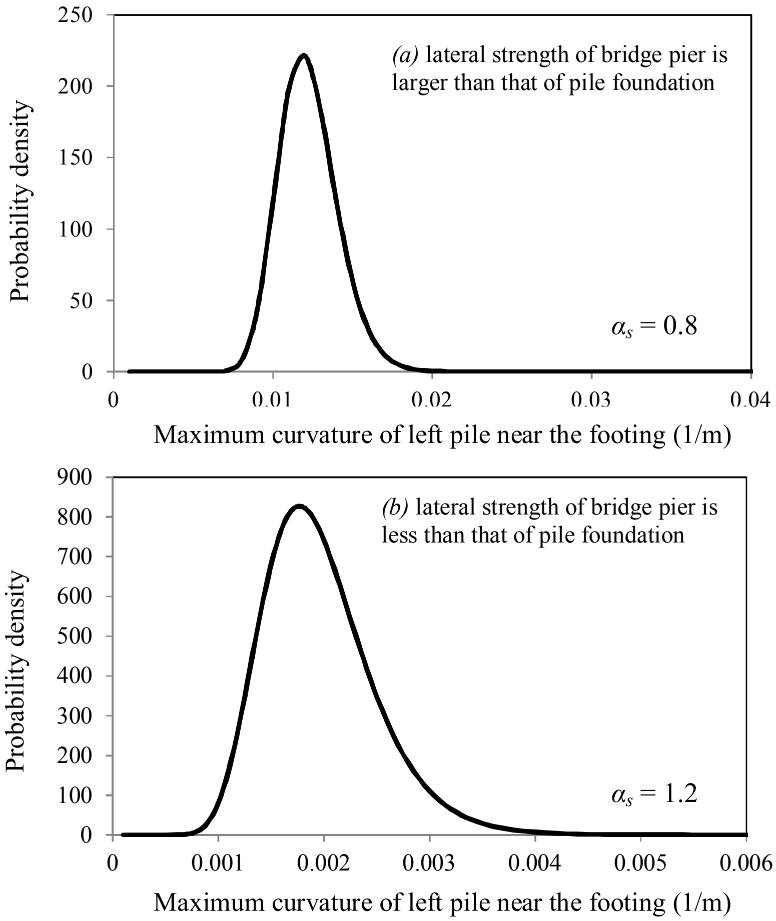

Figure 9. PDF of maximum curvature obtained by MCS

The uncertainties associated with the prediction of soil parameters are very large. To prevent the non-linear behavior of the pile foundation and to ensure that plastic hinge is placed at the bottom of the pier, it is necessary to confirm that the ratio α_s of lateral strength of foundation to that of pier is larger than 1.0. If α_s of bridge shown in Figure 7 is less than 1.0, the pile would exhibit non-linear behavior. In that case, the predicted restoration time and estimated repair costs could become larger because of the difficulties in repair work for the pile foundations.

Figure 8 shows that the relationship between the moment and curvature of RC piles depends on α_s. α_s are 0.8 and 1.2 in Figures 8 (a) and Figure 8 (b), respectively. These seismic responses are obtained by the non-linear dynamic analysis using the ground motion measured during the 1995 Hyogoken-Nanbu earthquake. The probabilistic density functions (PDFs) of maximum curvature of RC pile could be obtained by MCS. The uncertainties associated with the prediction of soil parameters and material strength in MCS are shown in [17]. Figures 9 (a) and (b) show the PDFs of maximum curvatures of pile foundations with α_s = 0.8 and α_s = 1.2, respectively. As indicated, the capacity hierarchy of α_s = 1.2 between bridge pier and pile foundation is not sufficient to keep the pile foundation behaving elastically. Akiyama et al. [17] pointed out that it is necessary to design the pile foundation with $\alpha_s \approx 2.0$ if the probability associated with the yielding of the pile foundation has to be less than 10^{-2}. The pile foundation with higher α_s is desirable, because the damage of foundation due to the strong ground motion is generally difficult to detect and repair. However, it becomes very difficult to design the pile foundation with higher α_s if the bridge is constructed on strata such as very soft soil that can experience soil liquefaction in a severe earthquake. Increased attention should be paid to improving the flexural strength and ductility capacity of precast concrete piles that are driven into liquefiable soil [18]. In addition, initial cost of the bridge increases with α_s. Further research is needed to identify the optimal α_s based on the seismic risk analysis.

3.3 Reliability-based capacity design of bridges under tsunami hazard

It is not feasible to design a bridge that will remain intact under all hazards that might impact its performance. Hazards to a bridge must be selected on the basis of their significance and must be incorporated into the risk assessment [19]. Hazards associated with the hydrodynamic features of a tsunami inundation flow, such as the inundation depth and current velocity, depend on the bridge location, especially on the distance from the coastline. Based on the tsunami fragility and hazards and bridge importance, bridges that require additional attention to achieve tsunami resistance performance objectives must be identified.

Much of the technology already exists for enhancing the structural ductility and integrity of bridges against damage and collapse when additional requirements beyond those provided in the current codes and standards are required. However, no structural system can be engineered and constructed to be absolutely risk free because of existence of uncertainties and economic constraints. Although comparing the structural reliabilities among bridges belonging to a network under tsunami hazard makes the determination of the priority for upgrade and/or repair actions possible, constructing bridges with very high performance requirements to prevent any damage or failure from a giant tsunami would be unfeasible.

If a bridge does not have a device to connect the superstructure with the bridge pier, the superstructure could be washed away due to the tsunami. Meanwhile, if a bridge has a device to connect the superstructure with the bridge pier, the bridge pier could have more severe damage caused by the tsunami. As observed in the 2011 Great East Japan Earthquake, the temporary bridges shown in Figure 4 could be constructed within a few months. In addition, in tsunami risk assessment, it is not necessary to consider human deaths due to the collapse of the bridge because there would be no people or vehicles on the bridge when the tsunami arrives. Therefore, for a giant tsunami with a lower probability of occurrence, washout of the superstructure could be acceptable.

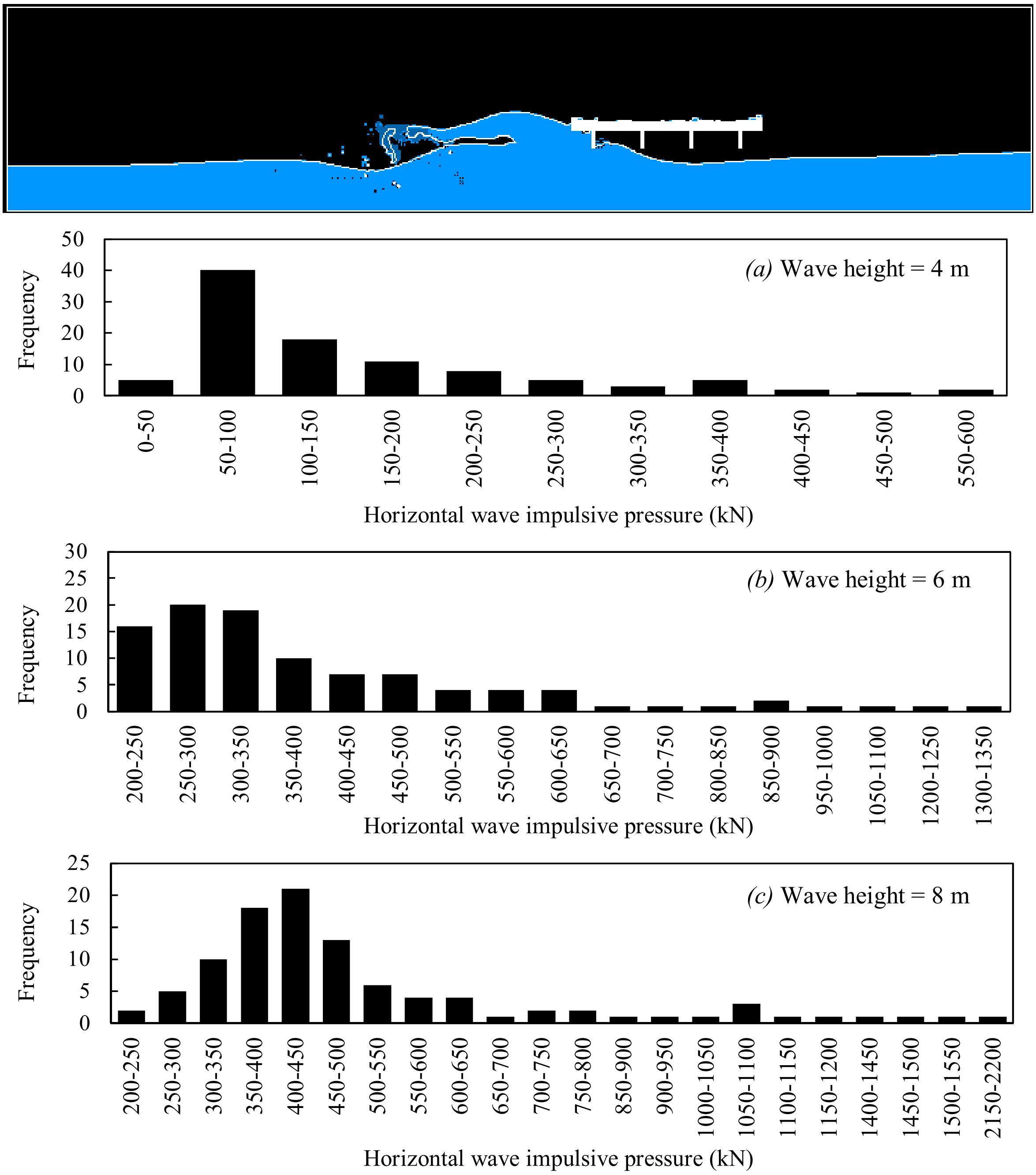

Figure 10. Horizontal wave impulsive pressure histogram obtained by CADMUS-SURF

The effect of the ratio α_t of lateral strength of bearing to that of RC pier on the failure probability of bridges associated with tsunami hazards is investigated using tsunami fragility curves based on simulation. The procedure to obtain the tsunami fragility curve is shown in [9, 10]. In the present study, the hydro-dynamic feature γ is tsunami height. A number of tsunamis with the same tsunami height but different tsunami periods and velocities are generated to take into consideration the tsunami pressure fluctuations using the CADMUS-SURF software developed by a group of Japanese coastal engineers [20]. In CADMUS-SURF, the Navier-Stokes equations in combination with the VOF (Volume Of Fluid) method and the LEF (Large Eddy Simulation) technique are solved. They are applied to a bridge model to estimate the hydro-dynamic horizontal and uplift pressures. These pressures can be calculated at any arbitrary points in the bridge model. Figure 10 shows an example of horizontal wave impulsive pressure histogram obtained by MCS with the number of samples of 100 under the condition that tsunami wave heights are 4m, 6m and 8m.

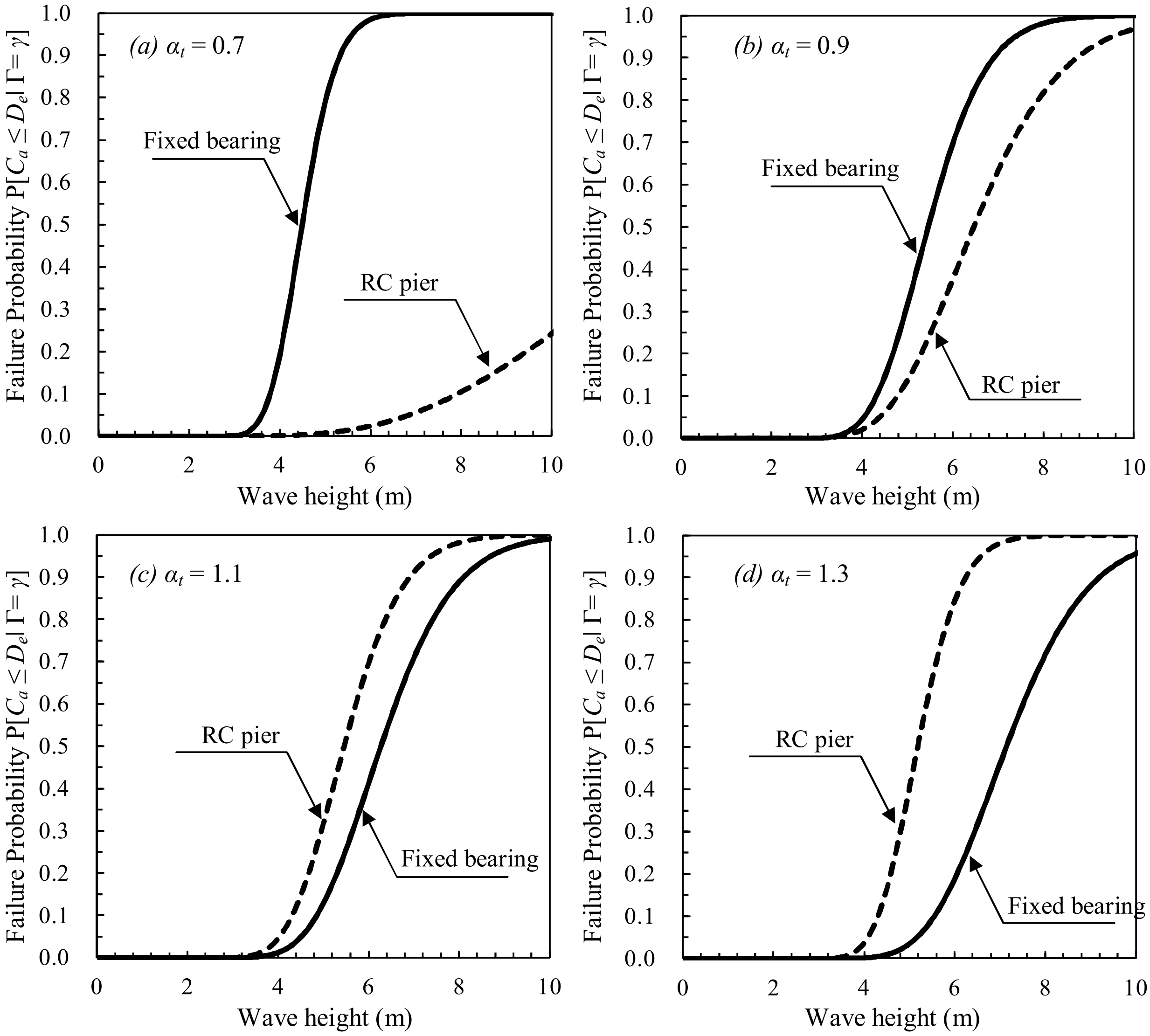

Figure 11. Effect of the ratio α_t of lateral strength of fixed bearing to that of RC bridge pier on the fragility curves

The capacity C_a is compared with the demand D_e in the push-over analysis for the tsunami fragility of bridges. As a case study, the concrete girder bridge is analyzed. This bridge with the length of 90m is continuous over three spans. Structural details of this bridge are shown in [21]. Figure 11 shows the comparison of fragility curves of fixed bearing and RC bridge pier with α_t = 0.7, 0.9, 1.1 and 1.3. When considering the post-event operability after a giant tsunami, the fragility curve associated with the bridge pier is located to the right of that associated with the fixed bearing. The optimal resistance hierarchy must be identified taking into consideration the level of the regional seismic and tsunami hazards, functionality loss (i.e., traffic capacity loss) after an event, economic impacts, time to restore the bridge, and life-cycle costs [22, 23]. The relationship between bridge damage and the resulting loss of functionality of the bridge must be considered when assessing the impact of an event on the performance of the transportation network.

4 CONCLUSIONS

- The key aspects of life-cycle design of bridges under multiple hazards, with an emphasis on earthquake, tsunami and continuous deterioration based on lessons learned from the 2011 Great Japan earthquake are discussed.
- In order to reduce the life-cycle cost of bridges, it is important to prevent the steel corrosion and/or cover concrete spalling even if the bridge is located in an aggressive environment. A simple reliability-based durability design criterion of RC structures in a marine environment with durability design factors that satisfy the target reliability level is presented. Although designing RC structures with high concrete quality, adequate concrete cover, and additional preventative measures (e.g. epoxy coated bars) increases the initial cost of concrete structures, expected reductions in maintenance and repair costs could justify their use on a life-cycle cost basis.
- The hierarchy of resistance of the various bridge components necessary to avoid catastrophic damage and to ensure prompt restoration from a severe earthquake and giant tsunami is investigated. A reliability-based design methodology for bridges under seismic and tsunami hazards is needed to improve structural performance for avoiding catastrophic damage and ensuring prompt restoration.
- Further research on life-cycle performance of single bridges and bridge networks under seismic and other hazards, including reliability, risk, resilience and sustainability has to be performed along the lines reported in [24-32].

REFERENCES

[1] K. Kawashima, Damage of bridges due to the 2011 Great East Japan earthquake. *Proceedings of the International Symposium on Engineering Lessons Learned from the 2011 Great East Japan earthquake*, Tokyo, Japan, 2012.

[2] K. Kawashima, K. Kosa, Y. Takahashi, M. Akiyama, T. Nishioka, G. Watanabe, H. Koga, H. Matsuzaki, Damages of bridges during the 2011 Great East Japan earthquake. *Proceedings of 43rd Joint Meeting, US-Japan 34 Panel on Wind and Seismic Effects, UJNR*, Tsukuba Science City, Japan, 2011.

[3] M. Akiyama, D. M. Frangopol, Lessons from the 2011 Great East Japan earthquake: Emphasis on life-cycle structural performance. *Proceedings of the Third International Symposium on Life-Cycle Civil Engineering*, IALCCE2012, Vienna, Austria, October 3-6, 2012; in *Life-Cycle and Sustainability of Civil Infrastructure Systems*, A. Strauss, D.M. Frangopol and K. Bergmeister eds., CRC Press/Balkema, Taylor & Francis Group plc, London, 13-20, and full paper on DVD, Taylor & Francis Group plc, London, 18-35, 2012.

[4] K. Kawashima, Seismic design and retrofit of bridges. *Proceedings of the 12th World Conference on Earthquake Engineering*, Auckland, New Zealand, 2000.

[5] Japan Society of Civil Engineers, *Damage analysis of concrete structures due the earthquakes in 2003, Concrete Library*, **114**, 2004.

[6] M. Akiyama, D. M. Frangopol, K. Mizuno, *Performance analysis of Tohoku-Shinkansen viaducts affected by the 2011 Great East Japan earthquake*. Structure and Infrastructure Engineering, **10**, 1228-1247, 2014.

[7] S. Kim, D. M. Frangopol, B. Zhu, Probabilistic optimum inspection/repair planning of deteriorating structures. *Journal of Performance of Constructed Facilities*, **25**, 534–544, 2011.

[8] M. Akiyama, D. M. Frangopol, H. Matsuzaki, Life-cycle reliability of RC bridge piers under seismic and airborne chloride hazards, *Earthquake Engineering and Structural Dynamics*, **40**, 1671–1687, 2011.

[9] M. Akiyama, D. M. Frangopol, M. Arai, S. Koshimura, Reliability of bridges under tsunami hazards: emphasis on the 2011 Tohoku-Oki earthquake. *Earthquake Spectra*, **29**, S295-S314, 2013.

[10] M. Akiyama, D. M. Frangopol, Life-cycle design of bridges under multiple hazards: Earthquake, tsunami and continuous deterioration. *Proceedings of the Eleventh International Conference on Structural Safety and Reliability*, ICOSSAR2013, New York, NY, USA, June 16-20, 2013; in *Safety, Reliability, Risk, and Life-Cycle Performance of Structures and Infrastructures*, G. Deodatis, B.R. Ellingwood, and D. M. Frangopol, eds., CRC Press, Taylor & Francis Group, London, 2013, 3-4, and full paper on USB stick, Taylor and Francis Group, London, 2014, 3-16.

[11] D. V. Val, M. G. Stewart, Life-cycle cost analysis of reinforced concrete structures in marine environments. *Structural Safety*, **25**, 343-362, 2003.

[12] C. Q. Li, Life cycle modeling of corrosion affected concrete structures-initiation. *Journal of Materials in Civil Engineering*, **15**, 594-601, 2003.

[13] C. Q. Li, Life cycle modeling of corrosion-affected concrete structures: propagation. *Journal of Structural Engineering*, **129**, 753-761, 2003.

[14] M. Akiyama, D. M. Frangopol, M. Suzuki, Integration of the effects of airborne chlorides into reliability-based durability design of reinforced concrete structures in a marine environment. *Structure and Infrastructure Engineering*, **8**, 125-134, 2012.

[15] M. Akiyama, D. M. Frangopol, H. Matsuzaki, Reliability-based durability design and service life assessment of concrete structures in an aggressive environment. Chapter 1 in *Maintenance and Safety of Aging Infrastructure* (Edited by D. M. Frangopol and Y. Tsompanakis), CRC Press/Balkema, Taylor & Francis Group, London, 1-26, 2014.

[16] M. J. N. Priestley, F. Seible, G. M. Calve, *Seismic Design and Retrofit of Bridges*. John Wiley & Sons, Inc., 1996.

[17] M. Akiyama, H. Matsuzaki, H. T. Dan, M. Suzuki, Reliability-based capacity design for reinforced concrete bridge structures. *Structure and Infrastructure Engineering*, **8**, 1096-1107, 2012.

[18] M. Akiyama, S. Abe, N. Aoki, M. Suzuki, Flexural test of precast high-strength reinforced concrete pile prestressed with unbonded bars arranged at the center of the cross-section. *Engineering Structures*, **34**, 259-270, 2012.

[19] B. R. Ellingwood, Mitigation risk from abnormal loads and progressive collapse. *Journal of Performance of Constructed Facilities*, *ASCE*, **20**, 315-323, 2006.

[20] M. Isobe, X. Yu and K. Umemura, S. Takahashi, Study on development of numerical wave flume. *Proceedings of Coastal Engineering, JSCE*, **46**, 36-40. (in Japanese)

[21] M. Akiyama, D. M. Frangopol, Reliability of bridges under seismic and tsunami hazards. *Proceedings of the Second International Conference on Vulnerability and Risk*

Analysis and Management (ICVRAM2014) *& Sixth International Symposium on Uncertainty Modeling and Analysis* (ISUMA2014), Liverpool, UK, July 13-16, 2014; Abstract in Book of Abstracts, M. Beer, S-K. Au, and J. W. Hall, eds.,Printed by Research Publishing Services, Singapore, 500-501; and full paper in *Vulnerability, Uncertainty and Risk: Quantification, Mitigation, and Management*, M. Beer, S-K. Au, and J. W. Hall, eds., CDRM9, ASCE, 2014, 1696-1705.

[22] D. M. Frangopol, P. Bocchini, Bridge network performance, maintenance, and optimization under uncertainty: Accomplishments and challenges. *Structure and Infrastructure Engineering*, **8**, 341-356, 2012.

[23] P. Bocchini, D. M. Frangopol, Restoration of bridge networks after an earthquake: Multi-criteria intervention optimization. *Earthquake Spectra*, **28**, 426-455, 2012.

[24] D. M. Frangopol, Life-cycle performance, management, and optimization of structural systems under uncertainty: Accomplishments and challenges. *Structure and Infrastructure Engineering*, **7**, 389-413, 2011.

[25] B. Zhu, D. M. Frangopol, Reliability, redundancy and risk as performance indicators of structural systems during their life-cycle. *Engineering Structures*, **41**, 34-49, 2012.

[26] B. Zhu, D. M. Frangopol, Risk-based approach for optimum maintenance of structures under traffic and earthquake loads. *Journal of Structural Engineering*, **139**, 422-434, 2013.

[27] A. Decò, D. M. Frangopol, Life-cycle risk assessment of spatially distributed aging bridges under seismic and traffic hazards. *Earthquake Spectra*, **29**, 127-153, 2013.

[28] Y. Dong, D. M. Frangopol, D. Saydam, Time-variant sustainability assessment of seismically vulnerable bridges subjected to multiple hazards. *Earthquake Engineering and Structural Dynamics*, **42**, 1451-1467, 2013.

[29] A. Decò, P. Bocchini, D. M. Frangopol, A probabilistic approach for the prediction of seismic resilience of bridges. *Earthquake Engineering and Structural Dynamics*, **42**, 1469-1487, 2013.

[30] Y. Dong, D. M. Frangopol, D. Saydam, Sustainability of highway bridge networks under seismic hazard. *Journal of Earthquake Engineering*, **18**, 41-66, 2014.

[31] Y. Dong, D. M. Frangopol, D. Saydam, Pre-earthquake probabilistic retrofit optimization of bridge networks based on sustainability. *Journal of Bridge Engineering*, **19**, 1-10, 2014.

[32] D. M. Frangopol, M. Soliman, Life-cycle of structural systems: recent achievements and future directions. *Structure and Infrastructure Engineering*, published on line January 15, 2015, doi:10.1080/15732479.2014.999794