A simplified approach for seismic reliability assessment of simply supported RC bridges under nonlinear deformation and uncertainties

Long Zhang
CCCC Highway Consultants CO., Ltd., Beijing, China. Email: zhanglong200809@163.com

Long Pang
Department of Civil Engineering, Tsinghua University, Beijing, China. Email: pangl1092@163.com

Cao Wang
School of Civil Engineering, The University of Sydney, Sydney, NSW, Australia. Email: cao.wang@sydney.edu.au

Hao Zhang
School of Civil Engineering, The University of Sydney, Sydney, NSW, Australia. Email: hao.zhang@sydney.edu.au

ABSTRACT: The nonlinear static pushover analysis provides a useful tool to describe the seismic behaviour of bridges subjected to moderate to high seismicity. This paper develops a simplified pushover analysis procedure for the seismic assessment of simply supported reinforced concrete (RC) bridges. With the proposed method, the pushover curve of the bridge can be obtained explicitly without complex finite element modelling. A random factor is introduced to reflect the uncertainty associated with the pushover curve. The role of the correlation in the pier performance in the bridge's seismic reliability is also considered. Illustrative examples are presented to demonstrate the applicability of the method. Sensitivity analysis is conducted to investigate the impact of variation and correlation in pier behaviour on the bridge's seismic performance.

1. INTRODUCTION

Bridges play a critical role in the traffic network, providing physical support to a region's transportation capacity. While the design provisions in current codes and standards are enforced to guarantee adequate levels of serviceability for the bridges, the severe damage or loss of function posed by hazardous events is continuously a great concern to the asset owners and civil engineers, which may lead to substantial economic losses and even ripple effect in the surrounding community. Earthquakes are among the hazardous events responsible for the damage and failure of bridges. For instance, during the 2008 Wenchuan Earthquake in Sichuan province, China, about 24 highways, 6140 bridges and 156 tunnels were severely destroyed, resulting in 67 billion RMB of losses to the traffic and infrastructure system (Du et al., 2008). Moreover, many bridges that were constructed according to historical codes and standards with insufficient safety levels are still in use today due to the socio-economic constraints. As a result, it is essentially important to assess and maintain the safety levels of these in-service bridges subjected to earthquake hazards so that their service reliability may be guaranteed beyond the baseline in the context of probability.

Significant efforts have been made in the scientific community during the past decades regarding the seismic performance assessment of civil infrastructures (Der Kiureghian, 1996; Ghobarah et al., 1998; Li and Ellingwood, 2008). The nonlinear static pushover analysis method, originally developed by Freeman (1975; 1978), has been widely accepted and used to estimate the seismic response for structures because it provides a practical description for the structural elastoplastic behaviour in relation to moderate to high seismicity (Zordan et al., 2011; Camara and Astiz, 2012). However, the generation of pushover curve needs tremendous amount of computational costs and requires skills, which may halter the application of pushover analysis in practice. The University of Ljubljana developed a simplified technique for seismic analyses named N2 method (Fajfar, 2007), which was further implemented in the European standard Eurocode 8 (2005). However, the bridge performance has been modeled as deterministic, with which the variation associated with the structural material and mechanical properties remains unaddressed. Subsequently, the correlation between the behavior of different components, arising from the common design provisions and construction conditions (Lee and Kiremidjian, 2007; Goda and Hong, 2008), yet has neither been taken into account in existing works.

In this paper, a simplified method is developed for the seismic performance assessment of RC bridges. Considering the variation associated with the bridge pier performance, a random factor is introduced to reflect the uncertainty in relation to its pushover curve. The correlation between the performance of the bridge piers is also considered. This paper chooses an in-service bridge to demonstrate the application of the proposed method and to investigate the role of bridge pier variation and correlation in the estimate of bridge seismic performance.

2. NONLINEAR PUSHOVER ANALYSIS OF SIMPLY SUPPORTED RC BRIDGES

The nonlinear static procedure (NSP) offers an insight to the structural nonlinear (inelastic) seismic behaviour for the engineers who are familiar with the linear seismic response of structures, especially in the era with an emphasis on the inelastic-deformation-based design for structures subjected to moderate to high seismicity. The nonlinear pushover analysis with an outcome of "pushover curve" is key in the NSP, which is representative of the relationship between the base shear and the roof displacement. The ultimate objective of NSP is to compare the peak elastoplastic deformation with the critical value so as to judge the displacement-based structural seismic behaviour (Chopra and Goel, 2000; Aydinoğlu, 2003).

The bridge behavior is expected to be ductile in sites associated with moderate to high seismicity due to the consideration of both economic and safety reasons, implying that the bridge components should dissipate a considerable amount of the input earthquake energy themselves. The presence of flexural plastic hinges provides physical support to this bridge performance goal, which can be found in the bridge piers accessible for routine inspection and repair. In this paper, only the longitudinal seismic response is taken into account, with which the formation of plastic hinge (PH) only occurs at the bottom of the bridge piers. The PH can be modeled as a rotary spring at the middle of the effective length L_p (Eurocode 8, 2005).

For a well-design simply supported concrete bridge, the superstructure such as the bent cap and the deck contributes to the majority of bridge mass. As a result, one may simplify the MDOF (multiple degree of freedom) system to a SDOF (single degree of freedom) system when performing seismic analysis for a simply supported concrete bridge pier (Wang et al., 2014).

The nonlinear pushover curve represents the relationship between the roof displacement and the base shear force. Consider the bridge pier as shown in Fig. 1, the objective in NSP is to find the relationship between the roof displacement Δ_{tot} and the shear force F. With the mechanical equilibrium condition, we have

$$M = FL + P\Delta_{\text{tot}} = FL + P(\Delta_{\text{u}} + \Delta_{\text{b}}) \qquad (1)$$

where M is the moment at the pier bottom, Δ_{u} is the displacement posed by the elastoplastic behavior of the pier, and Δ_{b} is the displacement due to
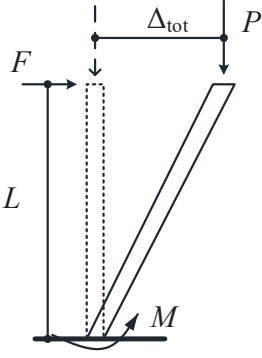
Figure 1: Force diagram for a bridge pier.

the shearing stiffness of the bearing pad. Note that in Eq. (1), Δ_u differs before and after the formation of the plastic hinge. As a result, we will discuss the F-Δ_{tot} relationship respectively for both stages (i.e., elastic and plastic ranges).

Stage 1: Elastic range

Within the elastic range, $\Delta_u = \Delta_{ela} = \frac{1}{3}\varphi L^2$, and $M = EI_{eff}\varphi$, where φ is the section curvature, EI_{eff} is the effective stiffness, and L is the pier length. Since $\Delta_b = \frac{F}{K}$ according to the well-known Hooker's law, where K is the shearing stiffness of the bearing pad, Eq. (1) is rewritten as

$$M = F\left(L + \frac{P}{K}\right) + P\Delta_{ela} \qquad (2)$$

Thus,

$$F = \frac{M - P\Delta_{ela}}{L + \frac{P}{K}} \qquad (3)$$

Note that

$$\Delta_{tot} = \Delta_{ela} + \Delta_b = \Delta_{ela} + \frac{EI_{eff}\varphi - P\Delta_{ela}}{KL + P} \qquad (4)$$

with which the relationship between Δ_{tot} and Δ_{ela} is obtained as

$$\Delta_{tot} = \Delta_{ela}\left[1 + \frac{3EI_{eff} - PL^2}{L^2(KL + P)}\right] \qquad (5)$$

Substituting Eq. (5) into (3), we have

$$F = \frac{3EI_{eff} - PL^2}{\left(L^3 + \frac{PL^2}{K}\right)\left(1 + \frac{3EI_{eff} - PL^2}{L^2(KL + P)}\right)}\Delta_{tot} \qquad (6)$$

Note that Eq. (6) works when the bridge pier section is within the elastic range. Since $\Delta_{ela} \leq \frac{1}{3}\varphi_y L^2$, where φ_y is the yield curvature,

$$0 < \Delta_{tot} \leq \frac{1}{3}\varphi_y L^2\left(1 + \frac{3EI_{eff} - PL^2}{L^2(KL + P)}\right) \qquad (7)$$

Stage 2: Plastic range

After the formation of the plastic hinge at the bottom of the bridge pier, $M = M_y$, where M_y is the yielding moment. Thus, Eq. (1) becomes

$$M_y = FL + P\Delta_{tot} \qquad (8)$$

with which the relationship between F and Δ_{tot} is obtained as

$$F = \frac{M_y - \Delta_{tot}}{L} \qquad (9)$$

Note that for stage 2, $\Delta_y + \Delta_p \leq \frac{1}{3}\varphi_y L^2 + L_p\varphi_{p,u}(L - 0.5L_p)$, where Δ_y is the yielding displacement, Δ_p is the plastic displacement, L_p is the effective length of plastic hinge, and $\varphi_{p,u}$ is the ultimate plastic curvature at the plastic hinge area. Thus,

$$\frac{1}{3}\varphi_y L^2\left(1 + \frac{3EI_{eff} - PL^2}{L^2(KL + P)}\right) < \Delta_{tot}$$
$$\leq \frac{KL\left[\frac{1}{3}\varphi_y L^2 + L_p\varphi_{p,u}(L - 0.5L_p)\right] + M_y}{KL + P} \qquad (10)$$

Finally, by noting that

$$\Delta_{tot} = \Delta_u + \frac{F}{K} = \Delta_u + \frac{M_y - P\Delta_{tot}}{KL} \qquad (11)$$

It follows,

$$\Delta_u = \Delta_{tot}\left(1 + \frac{P}{KL}\right) - \frac{M_y}{KL} \qquad (12)$$

Eq. (9) implies that after the pier bottom yields, F decreases with the increase of Δ_{tot}. This observation interestingly reflects the basic idea of the performance-based design: the ductility of structures may reduce the structural stiffness and enhance the viscous damping ratio and as a result mitigate the earthquake effect.

The pushover curve is given by the relationship between F and Δ_{tot}. Correspondingly, the capacity diagram for a single bridge pier is obtained by
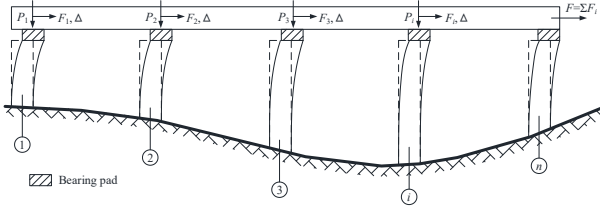
Figure 2: Considering the bridge pier inventory as a whole to derive the force-displacement relationship.

the functional relationship between Δ_{tot} and gF/P, where g is the gravitational acceleration. Clearly, this explicit relationship as developed herein is beneficial for determining the demand point of the structure due to its simplicity. The NSP works well with the proposed method since the simply supported bridge pier can be reasonably assumed as a SDOF as mentioned above.

Now we consider the bridge pier inventory as a whole, where the roof displacement of the bridge deck is identical for each pier, as shown in Fig. 2. In such a case, the pushover curve for the whole bridge is obtained according to Eq. (13), where n is the number of bridge piers, and F_i is the pushover curve function associated with the ith pier.

$$F_{\text{tot}}(\Delta) = \sum_{i=1}^{n} F_i(\Delta) \qquad (13)$$

Further, the simplified capacity diagram is obtained by the relationship between Δ and $g\dfrac{F_{\text{tot}}(\Delta)}{\sum_{i=1}^{n} P_i}$, where P_i is the vertical load associated with the ith pier.

3. VARIATION AND CORRELATION OF THE BRIDGE PIER PERFORMANCE

Note that the aforementioned pushover curve has been modeled as deterministic, with which the variation associated with the bridge pier performance remains unaddressed. Practically, the uncertainties arise in the non-exact structural performance modelling, the variability in material properties, geometry, environmental conditions and deterioration process (Stewart and Val, 1999). In order to reflect the uncertainty associated with the structural property, we introduce a random factor Λ which satisfies

$$\widetilde{F}(\Delta) = \Lambda \cdot F(\Delta) \qquad (14)$$

where $\widetilde{F}(\Delta)$ is the random pushover curve function of the bridge pier. Λ is assumed to follow a lognormal distribution with a mean value of 1 and a standard deviation of σ_Λ. Note that the random factor Λ indeed represents both the uncertainties associated with the bridge pier and the probabilistic behavior of the bearing pad (the shearing stiffness); in this paper, we do not distinguish these two types of uncertainties and refer them to as the pier performance uncertainty for the purpose of simplicity.

Further, we consider Eq. (13), which is revised as,

$$\widetilde{F}_{\text{tot}}(\Delta) = \sum_{i=1}^{n} \widetilde{F}_i(\Delta) = \sum_{i=1}^{n} \Lambda_i F_i(\Delta) \qquad (15)$$

where Λ_i is the modification factor associated with the ith bridge pier. Note that each Λ_i may be correlated due to the correlation in the bridge pier performance. We use the linear (Pearson) coefficient of correlation to model the correlation between two Λ_i's, and employ the Gaussian copula function to help construct the joint CDF for $\{\Lambda_i\}$ provided the marginal distributions and the correlation matrix of $\{\Lambda_i\}$.

4. ILLUSTRATIVE EXAMPLE

4.1. Bridge configuration

Shuangying Bridge, located over the Liangshui River in Beijing, China, has a service life of 26 years since the completion year of 1990. It is a continuous five-span reinforced concrete bridge with an overall length of 82m and spans of 17m, 16m×3 and 17m as shown in Fig. 3(a). This bridge has four bent caps and each of them contains 6 columns. The bridge piers at axes 1 and 4 have a height of 5m and the piers at axes 2 and 3 have a height of 9m (axes 3 and 4 not shown in Fig. 3). There are 19 T beams at each span with a height of 0.8m, and 38 neoprene bearing pads at each bent cap with a shearing stiffness of 7.395×10^4kN/m. All the bridge piers are concrete filled tubes with a diameter of 0.8m, whose cross section is illustrated in Fig. 3(b). *Shuangying* bridge was designed and constructed following the 1989 Chinese code for seismic design of highway bridges (JTJ004-89, 1989) and may have an unsatisfied safety level as required in the currently enforced code (CJJ 166,
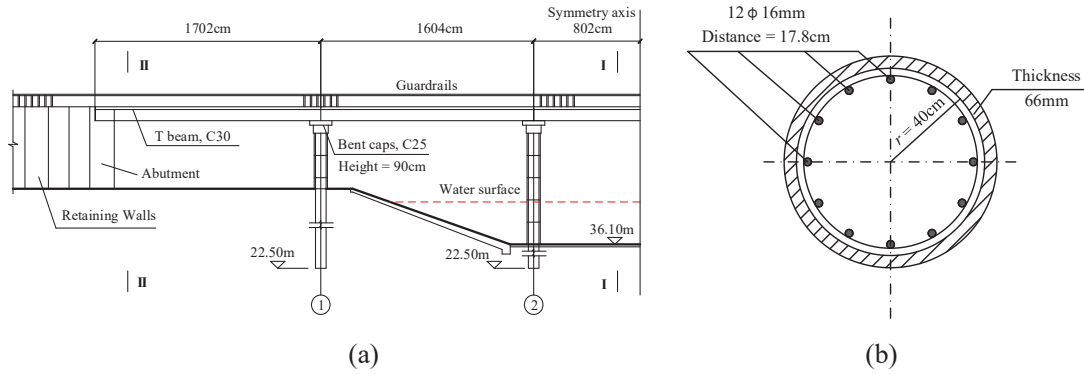
Figure 3: The Shuangying Bridge. (a) side view; (b) pier section.

2011). As a result, the seismic behavior of the bridge is verified in this section according to the latter code provisions, where the displacement-based verification is only required in relation to the E2 earthquake with a return period of 2000 years.

4.2. Seismic performance assessment before consolidation

The key parameters for the moment-curvature curve of the bridge pier are calculated first and the pushover curve for the whole bridge is obtained. Next, we transform the inelastic response spectrum from the A-T_n form to the A-D form and plot the demand and capacity diagrams in the same coordinate system (see Fig. 4). It is seen that the two diagrams have no intersection, implying that the bridge will collapse subjected to E2 earthquake and needs consolidation measures to improve its seismic safety.

4.3. Seismic performance assessment after consolidation

It was recognized from Fig. 4 that the bending moment capacity of the bridge piers is not satisfied in relation to the E2 earthquake seismicity following the currently enforced code (CJJ 166, 2011). A preliminary consolidation scheme is to enhance the bridge piers with larger diameter, as illustrated in Fig. 5. Here, the seismic performance assessment is performed employing the method proposed in this paper for the consolidated bridge piers aimed at evaluating this consolidation scheme.

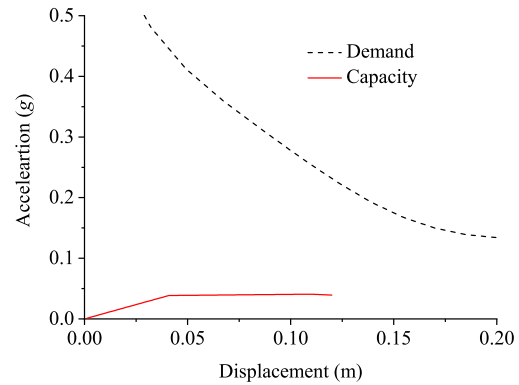The key parameters for the moment-curvature

Figure 4: Finding the demand point for Shuangying Bridge before consolidation.

curve of the consolidated bridge piers are found first, with which the capacity diagram is obtained, as shown in Fig. 6. It is found that the demand displacement is determined as 0.12m. It is emphasized that this figure is not the roof displacement of the bridge pier but the displacement of the bridge deck. The maximum rotation capacity for the bridge pier at axis 1 or 4 is 1/45 and 1/60 for that associated with axis 2 or 3. With this, the critical value for the bridge deck displacement is found as min {0.14m, 0.17m} = 0.14m referring to Eq. (9). Clearly, the consolidation scheme as illustrated in Fig. 5 satisfies the displacement requirement subjected to E2 earthquake.

Note that the bridge pier performance has been modelled as deterministic in Fig. 6. To reflect the
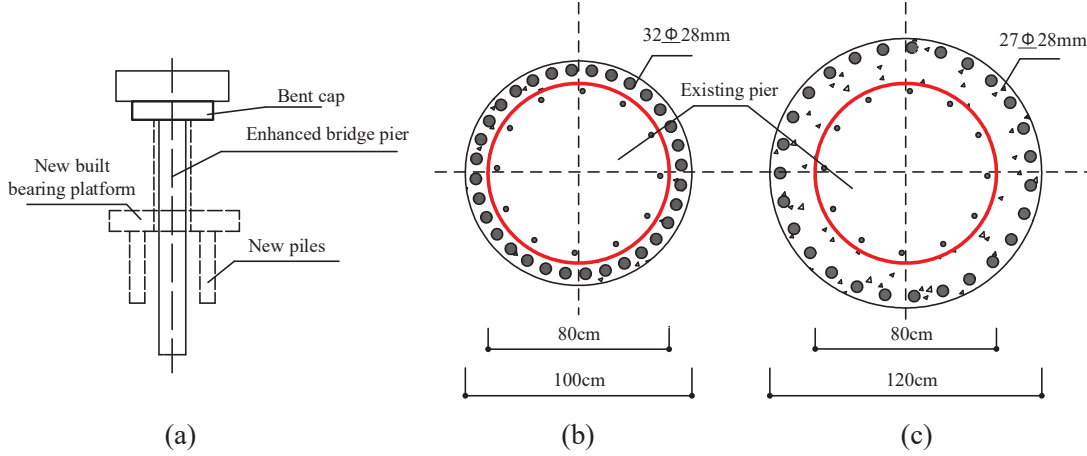
Figure 5: The consolidation of Shuangying Bridge piers. (a) construction scheme; (b) enhanced pier for axes 1 and 4; (c) enhanced pier for axes 2 and 3.
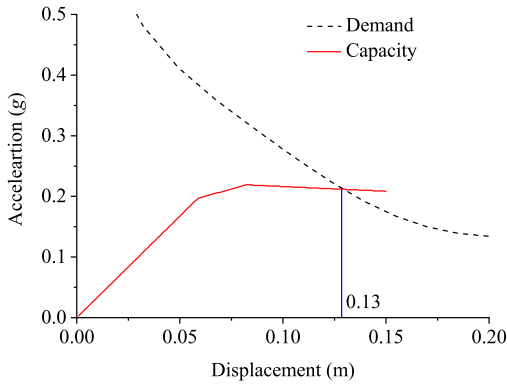

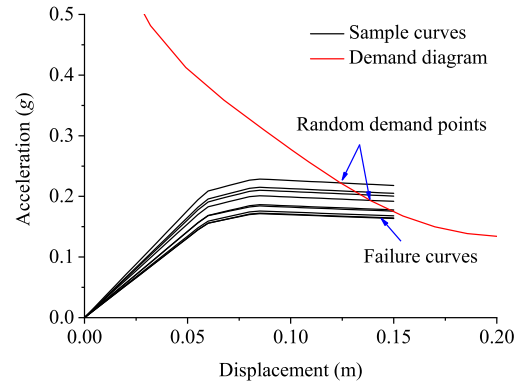*Figure 6: Finding the demand point for Shuangying Bridge after consolidation.*

Figure 7: Sample curves of the capacity diagram with $\sigma_\Lambda = 0.2$ and $\rho = 0.5$.

variation and correlation associated with the consolidated bridge piers, we consider the correlated random factors in relation to each bridge pier. As there is no enough evidence on the variation and correlation associated with the pier performance, we assume that the standard deviation of each Λ is identically σ_Λ and the coefficient of correlation between different piers equals ρ. The values of the two parameters can be obtained with practical investigation on the realistic pier properties and can be substituted to the present analysis once they become available. Fig. 7 plots the sample curves for the pushover curve associated with the whole

bridge inventory for the case of $\sigma_\Lambda = 0.2$ and $\rho = 0.5$. Obviously, the variation and correlation affects the demand point and further has an impact on the demand displacement. For the most sampled curves they intersect with the demand diagram; however, if the sample curve is associated with low seismic capacity, there may be no such intersection, implying that the bridge pier ductility is not satisfied subjected to the E2 earthquake.

Fig. 8 plots the probability distribution of the demand displacement associated with different σ_Λ for the case of $\rho = 0.3$ using 100,000 replications. Here, if there were no intersection between the ca-
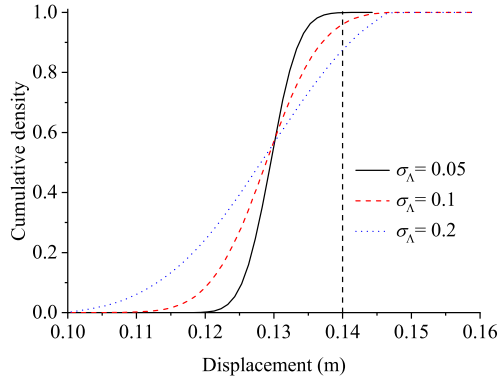
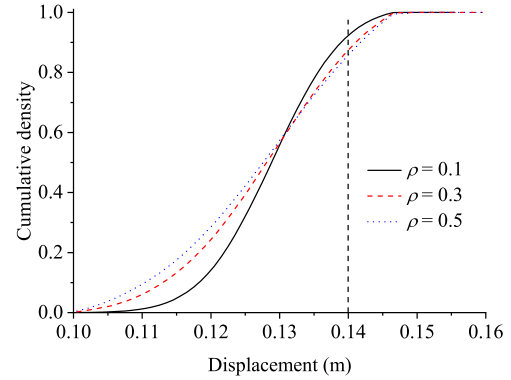*Figure 8: Effect of pier variation on the probability distribution of demand displacement with $\rho = 0.3$.*

Figure 9: Effect of pier correlation on the probability distribution of demand displacement with $\sigma_\Lambda = 0.2$.

pacity and demand diagrams, the demand displacement is set to be infinite. The increase of standard deviation associated with each Λ does not affect the mean value of the demand displacement, which equals 0.13m for all the three cases. However, the variability of the demand point increases as a result of the increase of the random factor. Keeping in mind that the critical displacement for the consolidated bridge is 0.14m, the probabilities that the demand displacement exceeds this critical value are 0, 0.35% and 9.4% corresponding to the cases of $\sigma_\Lambda = 0.05$, 0.1 and 0.2 respectively, implying that increase of the pier performance variation leads to greater seismic risk significantly.

Next, in order to investigate the role of pier performance correlation in the seismic performance assessment, Fig. 9 plots the probability distribution of the demand displacement associated with different ρ for the case of $\sigma_\Lambda = 0.2$. The increase of correlation between each Λ has no impact on the mean value of the demand displacement as before since all the three curves intersect at the same point. However, the increase of the correlation leads to greater variability of the demand point and longer upper tail behaviour. The probabilities that the demand displacement exceeds the critical value 0.14m are 1.8%, 9.4% and 15.4% respectively corresponding to $\rho = 0.1$, 0.3 and 0.5, indicating that the increase of correlation in bridge pier performance results in greater failure probability for

the bridge subjected to E2 earthquake.

It is noticed that the aforementioned exceeding probabilities obtained from Figs. 8 and 9 are indeed conditional on the assumption that the design spectrum represents the realization of the random earthquake demand. The uncertainty associated with the earthquake demand can be further considered by employing the total probability theorem (Bradley, 2013). Nevertheless, the analytical results herein qualitatively suggest that developing proper construction program with the objective of reducing the variation and correlation associated with the pier performance is of significant importance in practical engineering. These probabilities may be further utilized to help develop as a quantitative indicator representing the construction quality as soon as the practical knowledge on the bridge pier variation and correlation is accessible.

5. CONCLUSIONS

A probability-based method has been proposed in this paper for the seismic behaviour assessment of simply supported RC bridges. The proposed method enables the bridge pushover curve to be obtained explicitly without complex finite element modelling. Moreover, both the uncertainty associated with the bridge pier performance arising from the variability in material and mechanical properties and the correlation in the performance of different piers due to common design provisions and construction conditions are taken into account in

the proposed method. An illustrative bridge is chosen to demonstrate the applicability of the proposed method and to investigate the role of variation and correlation in bridge pier performance in the seismic behavior assessment. It is found that the variation and correlation in bridge pier performance contribute to the failure probability of the bridge. The bridge seismic behavior is more sensitive to the former one, implying the relative importance of controlling the construction quality by enhancing the construction management. Moreover, the analytical results reveal that the reduction of pier performance correlation is beneficial for the bridge seismic safety, suggesting the importance of optimizing the construction program aimed at reducing the correlation between the performance of different piers.

REFERENCES

Aydinoğlu, M. N. (2003). "An incremental response spectrum analysis procedure based on inelastic spectral displacements for multi-mode seismic performance evaluation." *Bulletin of Earthquake Engineering*, 1, 3–36.

Bradley, B. A. (2013). "Practice-oriented estimation of the seismic demand hazard using ground motions at few intensity levels." *Earthquake Engineering & Structural Dynamics*, 42(14), 2167–2185.

Camara, A. and Astiz, M. (2012). "Pushover analysis for the seismic response prediction of cable-stayed bridges under multi-directional excitation." *Engineering Structures*, 41, 444–455.

Chopra, A. K. and Goel, R. K. (2000). "Evaluation of nsp to estimate seismic deformation: Sdf systems." *Journal of Structural Engineering*, 126(4), 482–490.

CJJ 166 (2011). *Code for seismic design of urban bridges*. Ministry of housing and urban-rural development of the people's republic of China. China agriculture and building press. Beijing, China.

Der Kiureghian, A. (1996). "Structural reliability methods for seismic safety assessment: a review." *Engineering Structures*, 18(6), 412–424.

Du, X., Han, Q., Li, Z., and et al (2008). "The seismic damage of bridges in the 2008 wenchuan Earthquake and lessons from its damage (in chinese)." *Journal of Beijing University of Technology*, 34(12), 1270–1279.

Eurocode 8 (2005). *Design of structures for earthquake resistance – Part 2: Bridges*. European Committee for Standardization.

Fajfar, P. (2007). "Seismic assessment of structures by a practice-oriented method." *Extreme Man-Made and Natural Hazards in Dynamics of Structures*, Springer, 257–284.

Freeman, S. (1975). "Evaluations of existing buildings for seismic risk-a case study of puget sound naval shipyard." *Proc. 1st US Nat. Conf. on Earthquake Engrg., Bremerton, Washington*, 113–122.

Freeman, S. A. (1978). "Prediction of response of concrete buildings to severe earthquake motion." *Special Publication*, 55, 589–606.

Ghobarah, A., Aly, N., and El-Attar, M. (1998). "Seismic reliability assessment of existing reinforced concrete buildings." *Journal of Earthquake Engineering*, 2(04), 569–592.

Goda, K. and Hong, H. (2008). "Estimation of seismic loss for spatially distributed buildings." *Earthquake Spectra*, 24(4), 889–910.

JTJ004-89 (1989). *Code for seismic design of highway bridges*. Ministry of Transport of the People's Republic of China.

Lee, R. and Kiremidjian, A. S. (2007). "Uncertainty and correlation for loss assessment of spatially distributed systems." *Earthquake Spectra*, 23(4), 753–770.

Li, Q. and Ellingwood, B. R. (2008). "Damage inspection and vulnerability analysis of existing buildings with steel moment-resisting frames." *Engineering Structures*, 30(2), 338–351.

Stewart, M. G. and Val, D. V. (1999). "Role of load history in reliability-based decision analysis of aging bridges." *Journal of Structural Engineering*, 125(7), 776–783.

Wang, C., Bao, Q., and Li, Q. (2014). "Simplified pushover analysis method for simply supported concrete bridges in Chinese." *Technology for earthquake disaster prevention*, 9(3), 439–446.

Zordan, T., Briseghella, B., and Lan, C. (2011). "Parametric and pushover analyses on integral abutment bridge." *Engineering Structures*, 33(2), 502–515.