Proc. of the 6th Intl. Symposium on Reliability Engineering and Risk Management (6ISRERM)
31 May – 1 June 2018, Singapore
Editor(s) Xudong Qian, Sze Dai Pang, Ghim Ping Raymond Ong, Kok-Kwang Phoon

CORRODED RC BRIDGE SEISMIC RELIABILITY ASSESSMENT VIA PUSHOVER ANALYSIS

John Thedy[1,*], Kuo-Wei Liao[2], Fu-Sheng Chien[1], Ming-Yao Chang[1]

1 Department of Civil and Construction Engineering, National Taiwan University of Science and Technology
2 Department of Bioenvironmental Systems Engineering, National Taiwan University
**Corresponding Author E-mail: thedy.john@yahoo.com*

An innovative methodology for reliability assessment of a corrosion-induced deterioration bridge is presented in this study. The seismic performance of a bridge in the marine environment is significantly affected by corrosion. To address this issue, this study develops a time dependent reinforced concrete (RC) bridge assessment framework, in which the deterioration effect due to diffusion of chloride through concrete surface is considered. Once the chloride concentration on the rebar reaches the threshold value, the phenomenon of corrosion is occurred resulting to reduction in steel bar, spalling of concrete cover and loss of rebar bonding strength. Aforementioned corrosion effects will be considered in the proposed time dependent material property degradation model. Uncertainties in both material and corrosion parameter will be included in determining the time dependent plastic hinge property of a bridge. To simplify and reduce the computational time, pushover analyses in SAP2000 is automated by utilizing Open Application Programming Interface (OAPI), which fastens the evaluation the PGA capacity of the investigated bridge. In the proposed evaluation process, the seismic hazard curve on bridge site is also considered and the performance of a deterioration bridge is described by PGA.

Keywords: Bridges; corrosion; deterioration; reliability assessment; reinforced concrete.

1 Introduction

A thousand of bridges are currently operated in Taiwan, in which most of them are highway bridges. Highway bridges plays an important role in the economy foundation of Taiwan. As time progresses, materials of bridge will degrade mostly caused by corrosion. Reliability level of a bridge needs to be accurately evaluated by considering time dependent degradation and the other important uncertainties associated with the bridge performance. To achieve this target, several methods have been proposed. Because uncertainties associated with bridge performance need to be considered. This study will perform a seismic reliability assessment incorporating corrosion effect on material degradation and uncertainty associated in material and corrosion parameter. Mark G

Stewart [5] stated that corrosion is not only affect the loss of steel bar diameter but also the expansion of rusty bar inside the column will cause the cracking of concrete cover and bonds slip of longitudinal bar. Therefore, the effect of corrosion will be considered in this study. Peak Ground Acceleration (PGA) capacity will be used as the indicator of structure capacity.

2 Modeling of Wu Wang Bridge

Wu Wang Bridge is a RC highway bridge and only one segmental of the bridge is being assessed in this study in which the segment is taken from one roller to another roller support (*P08* to *P11*). The chosen bridge segment has four columns as shown in Figure 1 and the dimension information of the bridge is described in Figure 1. This bridge consists of three spans with 30 m long. There are three pile foundations and one caisson foundation. *SAP2000* will be used to perform the static nonlinear analysis. Superstructure of the bridge is modeled as elastic element and the stiffness of the in–plane bridge deck is assumed to be infinite.

Nonlinearity only occurs at the bottom of the pier since the moment capacity of caisson and number of pile used in the foundation are more than enough to keep foundation in elastic while pier already enter near failure condition. The effect of soil is considered in the proposed model using a linear link with the stiffness calculated using Taiwan Seismic Highway Design Code [7].

3 Corrosion Model

Chloride pitting corrosion is considered here. The effect of carbonation corrosion is insignificant in this case and therefore, is not included in the following analysis. Choe's Model [1] is used to represent the rebar diameter reduction with the Diffusion formula described in Equation 1.

$$C(x,t) = C_s\left[1 - erf\left(\frac{x}{2\sqrt{D_c \times t}}\right)\right] \qquad \textbf{Equation 1}$$

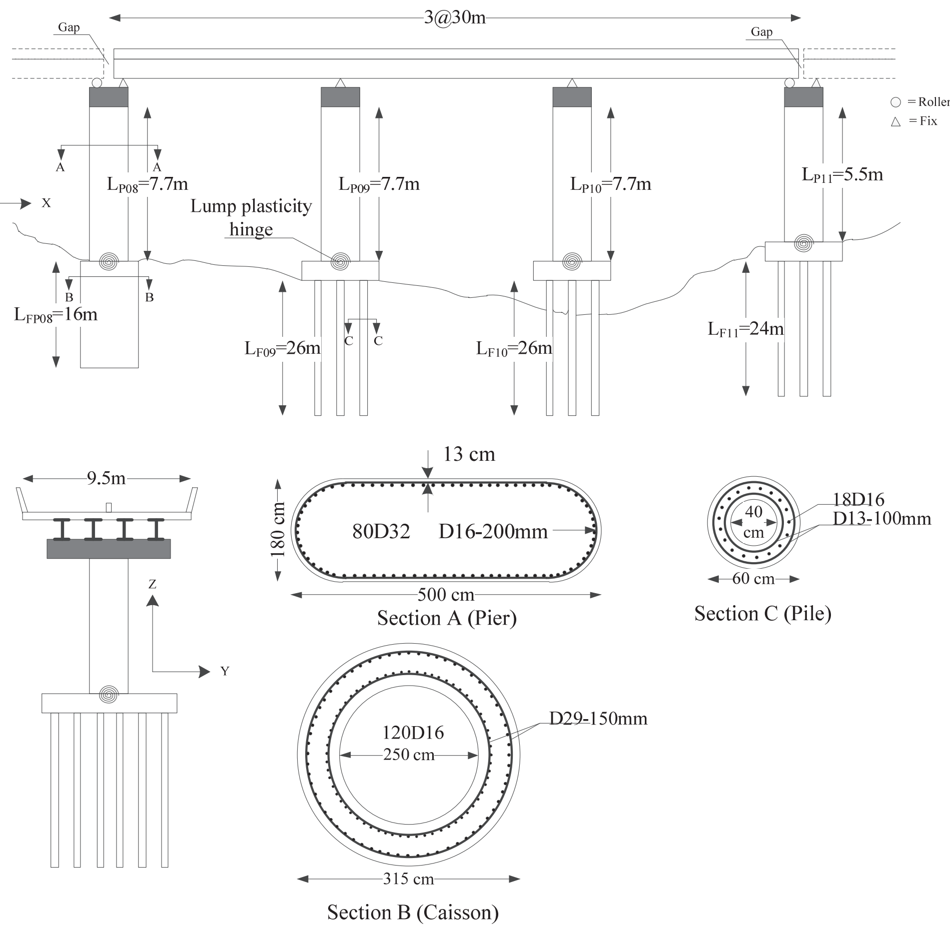

Figure 1 Bridge Model and Dimension Information.

Uncertainty of the corrosion parameter is displayed in Table 1. The coefficient of variation in this study is stochastically determined. In addition to bar diameter reduction, steel property reduction also considered in this analysis by adopting the study from Lee et al [3] in which the steel property reduction is based on the bar reduction percentage. Table 2 shows the steel property reduction formula in which stress, elongation and modulus is assumed to be decreased as corrosion increase.

Table 1 Uncertainty of Corrosion Parameter follows Gaussian distribution.

Parameter	Mean	*cov*	Unit
w/c	0.55	0.1	-
d_{s0}	3.8	0.1	km
w_s	2.175	0.1	m/s
w_r	0.3	0.1	-

Table 2 Steel property reduction factor.

Type of corrosion	Steel Properties	Reduction factor
Pitting corrosion	Yield stress	1-1.98 (%)
	Ultimate stress	1-1.57 (%)
	Elastic modulus	1-1.15 (%)
	Elongation	1-2.59 (%)

4 Material Stress Strain Curve Modeling

For core concrete and steel stress stain model, models proposed by Mander et al [4] and Priestley et al [9] are adopted. Ou et al [10] proposed a practical procedure to account the effect of bond slip by enlarge the strain value with modifier factor (Φ). This modifier factor is calculated from ratio between slip elongations between after and before corrosion and the bond strength before and after corrosion also need to be modified based on Bhargava et al [6] as shown in Figure 2.

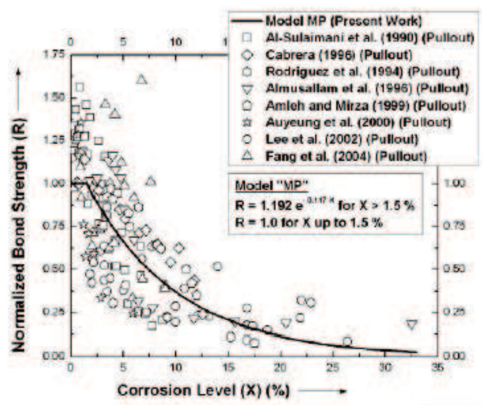

Figure 2 Bond Strength and Corrosion level correlation.

For the cover concrete, models of Molina et al [6] and Hsu et al [7] are adopted. The corrosion effect is introduced as the softening coefficient for stress value. Stress strain curve model for steel, core concrete, and cover concrete will be updated for each year increment by considering the effect of corrosion. In this study, 50 years increment analysis will be performed starting from no corrosion condition (0 year) until 400 year.

5 Pushover Analysis

A *MATLAB* code is developed to calculate the moment curvature of pier section using information obtained from stress strain curve model. Moment curvature results obtained from *MATLAB* will transform into moment rotation adopting the framework proposed by Sung et al [11]. In Sung et al work, moment rotation input for pushover curve is the combination of shear and moment. Using 500 samples of steel, core concrete and cover concrete stress strain curve as the input, moment rotation of each sample could be generated. Figure 4 shows the moment rotation at 0 year and 400 year with 16%, 50%, and 84% interval. Obtaining the moment rotation of each pier for 500 samples at each year increment, pushover analysis is performed. A formula shown in Table 3 is used to get the PGA capacity and the ultimate point in the pushover curve after aforementioned transformation will represent the collapse PGA (A_c) and yielding point in the pushover curve after transformation will be termed as yielding PGA (A_y).

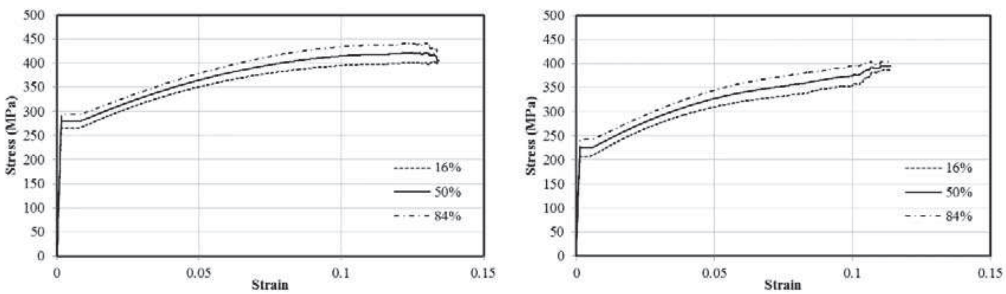

Figure 3 Steel stress strain model at 0 year (left) and 400 year (right)

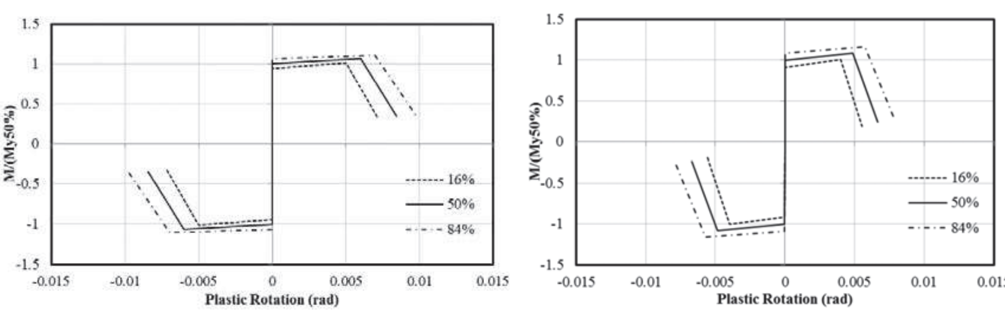

Figure 4 Moment Rotation at P08 at 0 year (left, $M_y50\%$ = 5030kNm) and 400 year (right, $M_y50\%$ = 4060kNm)

In Table 3, β represent the damping factor, T is the structure period that will change in every point of pushover curve as the structure stiffness reduce, S_{aD} is the design code spectral acceleration based on the structure period at certain stage and S_a is the capacity acceleration spectrum obtained from ATC 40 procedure.

Table 3 Acceleration Spectrum to PGA formulation.

Short Period	Medium Period	Long Period

$PGA = \dfrac{S_a \times \beta}{2.5}$	$PGA = \dfrac{S_a \times \beta}{\left(\dfrac{2.5S_{aD}(T)}{S_{DS}}\right)}$	$PGA = \dfrac{S_a \times \beta}{\left(\dfrac{2.5S_{aD}(T)}{S_{DS}}\right)}$

6 Fragility Curve

In this study, A_y and A_c occurrence rate is assumed as normal distribution with mean and standard deviation. Using aforementioned A_y and A_c distribution information and by assuming the earthquake occurrence also follow normal distribution with standard deviation of 0.1, the fragility curve of Ay and Ac could be constructed. From the Ac fragility curve, Reliability index of the bridge from each increment year could be observed as shown in Figure 5. Wu Wang Bridge is designed using PGA 0.308 g based on Taiwan earthquake design code, and from the reliability index degradation graph as shown in Figure 5, performance level of the bridge could be determined using reliability index measurement. If the reliability index is under tolerable value, the bridge should be put into maintenance status.

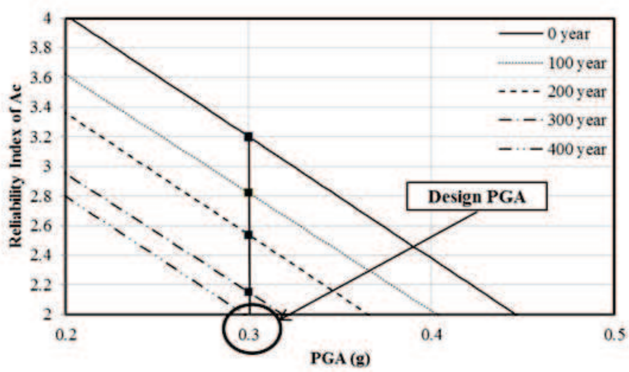

Figure 5 Degradation of Reliability Index for each increment year.

7 Conclusion

From this study, several conclusions are drawn as follows:

1 Reliability assessment via nonlinear static analysis (i.e., pushover) could be done in much less computation time compared to that of a nonlinear time history analysis.

2 Concrete cover has an important role to delay the corrosion process, especially, in the region of plastic hinges.

3 Since longitudinal bars location are deeper compared to that of transverse bars, transverse bars suffer more severe corrosion resulting in a faster decay in its shear strength.

8 References

[1] D.-E. Choe, P. Gardoni, D. Rosowsky, T. Haukaas, Probabilistic capacity models and seismic fragility estimates for RC columns subject to corrosion, Reliability Engineering & System Safety 93(3) (2008) 383-393.

[2] F.J. Molina, C. Alonso, C. Andrade, Cover cracking as a function of rebar corrosion: Part 2—Numerical model, Materials and Structures 26(9) (1993) 532-548.

[3] H.-S. Lee, Y.-S. Cho, Evaluation of the mechanical properties of steel reinforcement embedded in concrete specimen as a function of the degree of reinforcement corrosion, International Journal of Fracture 157(1) (2009) 81-88.

[4] J.B. Mander, M.J.N. Priestley, R. Park, Theoretical Stress-Strain Model for Confined Concrete, Journal of Structural Engineering 114(8) (1988) 1804-1826.

[5] K.A.T. Vu, M.G. Stewart, Structural reliability of concrete bridges including improved chloride-induced corrosion models, Structural Safety 22(4) (2000) 313-333.

[6] K. Bhargava, A.K. Ghosh, Y. Mori, S. Ramanujam, Suggested Empirical Models for Corrosion-Induced Bond Degradation in Reinforced Concrete, Journal of Structural Engineering 134(2) (2008) 221-230.

[7] Taiwan Transportation Department, Taiwan's Normative-Supplementary Study of Seismic Design Codes for Highway Bridges, 1997. (In Chinese)

[8] T.T.C. Hsu, Unified Theory of Reinforced Concrete, Taylor & Francis1992.

[9] T. Paulay, M.J.N. Priestley, Seismic Design of Reinforced Concrete and Masonry Buildings, 2009.

[10] Y.-C. Ou, P.-H. Wang, M.-S. Tsai, K.-C. Chang, C. Lee George, Large-Scale Experimental Study of Precast Segmental Unbonded Posttensioned Concrete Bridge Columns for Seismic Regions, Journal of Structural Engineering 136(3) (2010) 255-264.

[11] Y.-C. Sung, K.Y. Liu, C.-K. Su, I.C. Tsai, K.-C. Chang, A study on pushover analyses of reinforced concrete columns, 2005.