Reliability Analysis of River Bridge against Scours and Earthquakes

Kuo-Wei Liao[1], Yasunori Muto[2], Jessica Gitomarsono[3]

Abstract

This study proposes a bridge safety evaluation process against seismic and flood hazards. Because uncertainties in the scours, seismic hazard, and structural performance for a given seismic excitation are inevitable and important, reliability analysis is adopted. A scour prediction equation for a bridge with a complicated foundation system is developed and a probabilistic scour curve is constructed to measure the risk of scours using the Monte Carlo simulation. The seismic hazard is measured using the probabilistic seismic-hazard analysis. A series of nonlinear time-history analyses are performed to determine the structural performance under different peak-ground-acceleration values. SAP2000 is used to build the finite-element model wherein the soil is modeled using a bilinear link. A plastic hinge is predefined to simulate the nonlinear behaviors of the pier and caisson of the bridge. The displacement ductility is used to measure the structural performance and to construct the fragility curve for various limit states. The Nanyun Bridge located in central Taiwan is selected as an example to demonstrate the proposed safety-evaluation procedure. The results show that the probable scour depth of the Nanyun bridge is from 3 to 5 m. The failure probability considering the floods and earthquakes is insignificant. A deterministic design value, considering both the hazards, is provided for a given reliability target (e.g., β = 3) to help engineers in their present design processes.

Keywords: seismic excitation, reliability analysis, multi-hazards, scour depth, displacement

---

[1] *Corresponding Author, Associate Professor, Department of Bioenvironmental Systems Engineering, National Taiwan University, No. 1, Sec. 4, Roosevelt Rd., Taipei 10617, Taiwan*, Email: kliao@ntu.edu.tw
[2] *Professor, Department of Civil & Environmental Engineering, Tokushima University, Tokushima, Japan*
[3] *Former graduate student*

ductility

Introduction

In earthquake engineering, many efforts are targeted on correlating earthquake intensities and damages of buildings or bridges. Bazos et al. (1999) developed a fragility curve for empirical relationship between ground motion and bridge damage for Northridge earthquake, in which Caltrans was used to define the damage states. Based on the on-site investigation, Hsu and Fu (2004) found several types of bridge damage in Chi-Chi earthquake such as unseating span failure, abutment failure, joint failure, substructure damage, footing settlement, and so on. Elnashai et al. (2012) analyzed the earthquake effect on the buildings and bridges for Chile earthquake. They first developed site specific ground motions and then several typical failures observed in the engineered buildings and bridges were investigated. Based on the field investigation, it was found that excessive displacements of the superstructure lead to unseating and collapse of several bridges. The on-site bridge damage reports often implied that an earthquake-induced damage is not easily classified. However, displacement related damage is often found on the field and is a suitable choice to measure the bridge performance under earthquake excitations.

In addition to the earthquake hazard, flood hazard is another important risk should be considered. For example, Padgett et al. (2008) reported that 44 bridges were damaged from Hurricane Katrina. Bridge damages are primarily due to debris impact. According to Andric´ and Lu (2016), the potential hazards of bridge are classified as geological, windstorms and hydraulic hazards, in which geological hazard includes earthquake, tsunami, liquefaction, soil, and landslides; hydraulic hazards includes flood, debris, scour and drift. Based on literature survey, the primary reason for bridge damage in the United States is related to flood-induced damage. According to a report of Construction Research Institute in Taiwan, bridges in Taiwan also have the same trend. Taiwan is a seismically-active and flood-prone region.

Thus, the goal of this study to investigate the bridge performance under earthquake attacks in the presence of flood-induced scour. To be specific, this study is aimed to evaluate the joint-failure probability of a river bridge subjected to multi-hazard conditions.

There are thousands of bridges in Taiwan. Many of these bridges were built several decades ago and need to be examined to ensure operational safety. Among the different disasters, floods and earthquakes frequently occur in Taiwan and their influences are significant. Typhoon-induced floods often result in a serious scour problem. This study considers the two hazards simultaneously to ensure the safety of the bridge. Many uncertainties are involved in the considered hazards, and therefore, a probabilistic approach is adopted. The reliability of the bridge is calculated considering uncertainties in the scours, seismic hazard, and structural performance under a given seismic excitation.

Many formulae have been proposed to determine the scour depth. Melville and Coleman (2000) and Hydraulic Engineering Circular No. 18 (HEC-18, US Department of Transportation 2012) provide methodologies to consider the non-uniform pier effect. To employ the uniform pier formula, Melville and Coleman (2000) converted the non-uniform pier width to an equivalent uniform pier width to predict the scour depth. However, in HEC-18, the considered foundation was divided into three parts and the scour depth of each part was calculated separately. In the earlier time, the non-uniform foundation effect is rarely considered. Thus, scour depth is often calculated using the approach of uniform pier formula in Taiwan. To avoid extra burden in practice, the approach used by Melville and Coleman is employed to develop a scour-prediction formula using collected scour data and an optimization algorithm. Please note that this selection does not include accuracy judgement between Melville and Coleman’s approach and HEC-18. Further, a probabilistic scour curve is constructed to measure the risk of scours using the Monte Carlo simulation (MCS).

The seismic hazard is evaluated using probabilistic seismic hazard analysis (PSHA). To obtain the structural performance under different peak ground acceleration (PGA), the nonlinear time-history analysis is performed wherein seven recorded ground motions published in the Pacific Earthquake Engineering Research Ground Motion Database (PEER) are used. The ground motions are fitted and scaled to the response spectrum at the bridge location using the Taiwan code corresponding to the return periods of 475 and 2500 years. The mechanical properties of the cover and core concretes are considered. The detailed modeling procedure of the concrete mechanism is provided in the "Simulation of nonlinear behaviors of pier and caisson" section. The simulations of the plastic hinges of the pier and caisson are major factors in this mechanism.

The displacement ductility is used as the parameter in constructing the fragility curve. A finite element model of the Nanyun bridge is built to apply the proposed methodology. In the end, a design scour depth, which is a deterministic value, is provided to help engineers in their practice. That is, if the safety of a bridge with design scour depth is ensured by the current practice, such bridge will meet the target reliability for both the hazards. Several values for target reliability have been suggested (Honjo et al. 2002), ranging from 1.75 to 7.5 for different structural member (e.g, beam in shear or wall in compression) and different failure mechanism (e.g., ductile or brittle). Using $\beta = 3$ as the target reliability, which is roughly equal to the threshold value (1.00×10^{-3}) suggested by the International Organization for Standardization (ISO) (Davis-McDaniel et al., 2013), is often acceptable and therefore, is adopted in this study.

Proposed methodology

Figure 1 shows the flowchart of the proposed methodology. The joint-failure probability of a bridge is the product of three probabilities (Alipour et al. 2013): the probability of seismic hazard, scour depth, and bridge failure for a given limit state. The seismic hazard developed by the National Center for Research on Earthquake Engineering

(NCREE) is adopted in this study (Yeh and Jean 2007). From the experiments, 176 scour depths are obtained to develop a scour-prediction formula using the methodology proposed by Melville and Coleman (2000). Subsequently, a probabilistic scour curve is established. The fragility analysis is a common tool to determine the structural-failure probability under different limit states. To build the fragility curve, several nonlinear time-history analyses are conducted. The fragility curve is a conditional probability wherein the "condition" refers to a given scour depth. Thus, a predefined scour depth is given for the bridge model in the time-history analysis. Because the modeling of a bridge plays an important role in evaluating the structural performance, the nonlinear behaviors of the pier, caisson, and soil are carefully simulated. The details of the proposed methodology are provided in the following sections.

Building the probabilistic scour curve

Melville and Coleman (2000) proposed a formula to predict the scour depth of a complicated foundation. The calculation method is expressed in Eq. (1).

$$d_s = K_{yb} K_s K_\theta K_I K_t K_d \tag{1}$$

where K_{yb} is the water depth – bridge shape impact factor, as expressed in Eq. (2). K_s is the pier-shape correction factor, K_θ is the correction coefficient of the angle of attack of flow, K_I is the flow intensity correction coefficient, K_t is the time-factor correction coefficient, and K_d is the river-bed-material characteristic correction coefficient.

$$\begin{cases} K_{yb} = 2.4 b_e & \dfrac{b_e}{y} < 0.7 \\ K_{yb} = 2\sqrt{y b_e} & 0.7 < \dfrac{b_e}{y} < 5 \\ K_{yb} = 4.5 y & \dfrac{b_e}{y} > 5 \end{cases} \tag{2}$$

where b_e represents the equivalent pier width perpendicular to the flow, y is the flow depth. In the approach proposed by Melville and Coleman, the equivalent pier width (b_e) plays a key role. Additionally, when the water depth, river-bed location, and pier type are considered, b_e

may be slightly different, which can be classified mainly into four cases and are expressed as Eq. (3).

$$
\begin{cases}
b_e = b_c & Y > b_{pc} \text{ (case 1)} \\
b_e = b\left(\dfrac{y+Y}{y+b_{pc}}\right) + b_{pc}\left(\dfrac{b_{pc}-Y}{y+b_{pc}}\right) & b_{pc} \geq Y \geq 0 \text{ (case 2)} \\
b_e = b\left(\dfrac{y+Y}{y+b_{pc}}\right) + b_{pc}\left(\dfrac{b_{pc}-Y}{y+b_{pc}}\right) & 0 \geq Y \geq -y \text{ (case 3)} \\
b_e = b_{pc} & -Y > y \text{ (case 4)}
\end{cases}
\tag{3}
$$

Eq. (3) shows that b_e is interpolated using b_c and b_{pc}, as shown in Eq. (4).

$$
b_e = \begin{cases}
Ab_c + Bb_{pc}, \text{ where} \\
A = \dfrac{x_1 y + x_2 Y}{x_3 y + x_4 b_{pc}} \\
B = \dfrac{x_5 b_{pc} - x_6 Y}{x_7 y + x_8 b_{pc}} \\
A + B = 1
\end{cases}
\tag{4}
$$

where A and B are the weights for b_c and b_{pc}, respectively, and the sum of the two weights is one. According to Melville and Coleman (2000), A and B are functions of the flow depth (y), level of the top surface of the pile cap below the surrounding bed level (Y), and pile-cap width perpendicular to the flow (b_{pc}). In this study, an optimization technique is employed to obtain the functions of A and B, as described in Eq. (4), where x_i refers to the coefficient to be determined. The mathematical formulation of the optimization problem is described as follows.

$$
\mathrm{Min}_x \quad D_s - d_s = D_s - f(x) \tag{5}
$$

where D_s is the scour depth obtained from the experiment, d_s is the calculated scour depth using Eq. (1) which is a function of x described in Eq. (4). The experimental data of the 176 entries are obtained, and the sequential quadratic programming tool from the MATLAB toolbox is used to solve the optimization problem described in Eq. (5). The objective of the optimization is to obtain eight coefficients in Eq. (4) that can help minimize the estimation errors. However, when $Y > 2.4b_c$, it is typically not scoured to the location of the pile-cap and so that the influence of pile-cap and pile groups can be ignored. Therefore, under such conditions, optimization is not performed, indicating that $b_e = b_c$. The optimization results for the other three cases are described in Eqs. (6), (7), and (8).

$$b_e = g(y, Y, b_c, b_{pc}) = \left(\frac{0.80y + 0.31Y}{0.83y + 1.00b_{pc}} \right) b_c + \left(\frac{0.02b_{pc} - 0.07Y}{0.75y + 0.56b_{pc}} \right) b_{pc} \text{ (case 2)} \quad (6)$$

$$b_e = h(y, Y, b_c, b_{pc}) = \left(\frac{0.22y + 0.38Y}{0.16y + 0.31b_{pc}} \right) b_c + \left(\frac{0.05b_{pc} - 0.16Y}{-0.03y + 0.57b_{pc}} \right) b_{pc} \text{ (case 3)} \quad (7)$$

$$b_e = k(y, Y, b_c, b_{pc}) = \left(\frac{0.42y + 0.11Y}{0.24y + 0.90b_{pc}} \right) b_c + \left(\frac{0.11b_{pc} - 0.08Y}{0.20y + 1.00b_{pc}} \right) b_{pc} \text{ (case 4)} \quad (8)$$

Table 1 presents the predicted result of the proposed approach. In general, the result shows that the accuracy of the formula proposed by Melville and Coleman (2000) is significantly improved. The proposed formula is conceptually consistent with the observed scour behaviors and helps predict the scour-depth accurately.

Based on the built scour prediction formula, it is known that scour depth is a function of water depth and water velocity. That is, scour depth is a function of random variables and its probabilistic characteristics (such as mean value, standard deviation and probability density function) are described using MCS followed by Goodness of fit test. The design/target values specified in the code (2009) are used as the mean values of water depth and water velocity. Based on earlier studies (Liao et al. 2012), the water depth and water velocity were found to often follow the log-normal distribution and are adopted in this study. In addition, the coefficients of variation for the water depth and water velocity are assumed as 0.135 and 0.35, respectively (Liao et al. 2012).

Simulation of nonlinear behaviors of pier and caisson

Two types of mechanical properties of the concrete are considered: the cover and core concretes. The behavior of the cover concrete is considered unconfined using a model proposed by Coronelli and Gambarova (2004). The stress–strain correlation is calculated using Eqs. (9) and (10) for ascending and descending branches, respectively. The parameter ζ represents the softening effect resulting from the corrosion. Because the corrosion is not considered, the value of ζ becomes one.

$$\sigma_a = \zeta f_c \left[2\left(\frac{\varepsilon}{\zeta\varepsilon_0}\right) - \left(\frac{\varepsilon}{\zeta\varepsilon_0}\right)^2 \right] \tag{9}$$

$$\sigma_d = \zeta f_c \left[1 - \left(\frac{\frac{\varepsilon}{\zeta\varepsilon_0} - 1}{\frac{2}{\zeta} - 1} \right)^2 \right] \tag{10}$$

The strength of the core concrete is greater than that of the cover section because of the presence of transverse reinforcement. The model proposed by Mander et al. (1988) is adopted in this study to evaluate the confinement effect. Because the pier has a solid circular section whereas the caisson has a hollow section, two types of core concretes are considered. The circular section is evaluated using the model proposed by Mander et al. (1988). However, the hollow section should be modified to consider the different force distributions. The general equation of the model proposed by Mander is expressed in Eq. (11).

$$f_c = \frac{f'_{cc}\, x r}{r - 1 + x^r} \tag{11}$$

where f_c is the longitudinal compressive concrete stress and f'_{cc} is the compressive strength for the confined concrete, which can be determined as follows.

$$f'_{cc} = f'_{co}\left(-1.254 + 2.254\sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2\frac{f'_l}{f'_{co}}\right) \tag{12}$$

where f'_{co} is the unconfined concrete compressive strength and f'_l is the effective confining stress on the concrete. x in Eq. (11) is calculated as follows.

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \tag{13}$$

where ε_c is the longitudinal compressive concrete strain. ε_{cc} is calculated as follows.

$$\varepsilon_{cc} = \varepsilon_{co}[1 + 5(\frac{f'_{cc}}{f'_{co}} - 1)] \tag{14}$$

where ε_{co} is the corresponding unconfined concrete strain of f'_{co} and is 0.002, as suggested by Mander et al. (1988). r in Eq. (11) is calculated as follows.

$$r = \frac{E_c}{E_c - E_{\sec}} \tag{15}$$

where $E_c = 5{,}000\sqrt{f'_{co}}\,\mathrm{MPa}$ is the tangent modulus of elasticity of the concrete and

$$E_{\sec} = \frac{f'_{cc}}{\varepsilon_{cc}} \tag{16}$$

For foundation with a hollow section (i.e., the investigated bridge), the effective confined stress (f'_l) is different from that of a solid pier and is determined using Eq. (17).

$$f_l' = effective\ \ confinement\ pressure = k_e f_l = k_e \left[\frac{2 f_{yh} A_{sp}}{s\left(d_{s_2} - d_{s_2}\right)} \right] \tag{17}$$

The stress–strain curve of the steel used in this study is described below.

For $0 \le \varepsilon_s \le \varepsilon_y$

$$f_s = E_s \varepsilon_s \tag{18}$$

For $\varepsilon_y \le \varepsilon_s \le \varepsilon_{sh}$

$$f_s = f_y \tag{19}$$

For $\varepsilon_{sh} \le \varepsilon_s \le \varepsilon_{su}$,

$$f_s = f_y\left[\frac{f_u}{f_y}-\left(\frac{f_u-f_y}{f_y}\right)\left(\frac{\varepsilon_u-\varepsilon_s}{\varepsilon_u-\varepsilon_{sh}}\right)^2\right] \tag{20}$$

where f_s is the stress of the steel, E_s is the elastic modulus of the steel, ε_s is the strain in the steel, f_y is the yield stress of the steel, f_u is the ultimate stress of the steel, ε_{sh} is the strain hardening of the steel, and ε_u is the ultimate steel strain.

According to Sung et al. (2005), the shear mode should be converted to the corresponding bending mode to determine the failure mode of the pier or caisson. Accordingly, three types of failure modes are classified: shear failure mode, flexural-to-shear failure mode, and flexural failure mode. The nonlinear behaviors of the pier and caisson are largely described via the P-M3 plastic hinge using the proposed SAP2000 model. The shear plastic hinge is not used.

Simulation of nonlinear behaviors of soil

Many methods are available to model the soil behavior. The regulations suggested by the Taiwan code are adopted in this study (Chang et al. 2009). The soil behavior is simulated using the bilinear link element provided in SAP2000. The link is divided into three types, which include horizontal resistance on the peripheral side of the caisson, and vertical and friction resistances on the bottom plane of the caisson. The soil behavior is simulated using a bilinear model wherein the passive-earth force is employed as the upper bound. The friction effect between the caisson and the soil along the peripheral side area is ignored. Similarly, the link property in the vertical direction of the bottom surface is simulated using a bilinear model wherein the bearing force is employed to determine the upper limit, as shown in Eq. (21). The stiffness in the linear part is simulated using Eq. (22). The upper limit and stiffness in the linear part for the frictional force are described in Eqs. (23) and (24), respectively. The friction link is placed at the bottom of the caisson using the same partition method.

$$q_u = \alpha c N_c + \gamma_2 D_f N_f + 0.5\beta\gamma_1 B N_r \quad (21)$$

Here, q_u is the bearing force, α and β are the base factors based on the foundation shape, c is the soil cohesion, and γ_1 is the effective unit of the bottom surface of the lower base of the soil. γ_2 is the average effective unit weight of the soil above the bottom surface, D_f is the foundation depth, B is the base width of the foundation, and N_c, N_f and N_r are the factors for the supporting forces.

$$k_V = k_{V0}\left(B_V / 30\right)^{-3/4} \quad (22)$$

where k_{vo} is the coefficient of the vertical ground reaction force, and B_V is the base equivalent load width.

$$R_f = N \tan\delta + AC_a \quad (23)$$

$$k_s = 0.3k_v \quad (24)$$

where R_f is the frictional resistance of the bottom surface (tf), N is the effective vertical load acting on the basis (tf), δ is the angle of friction (°), A is the effective contact area between the bottom surfaces of the base (m^2), and C_a is the effective adhesion (t/m^2)

Ground motions and seismic hazard

A series of nonlinear time-history analyses are performed to develop the fragility curve. Based on the American Association of State Highway and Transportation Officials (AASTHO) guide specification for Load and Resistance Factor Design (LRFD) seismic bridge design (AASTHO 2007), a nonlinear time-history analysis should be performed for critical and essential bridges as approved, for which the definitions, limitations, and requirements are given in Provision 4.2.2 of the AASTHO guide specification for LRFD seismic bridge design (AASTHO 2007). The design action is considered to be the maximum response calculated for three ground motions in each principal direction. If a minimum of seven time histories are used for each component of motion, the design actions are considered as the mean responses calculated for each principal direction. According to the AASTHO guide specification for LRFD seismic bridge design (AASTHO 2007), seven ground motions obtained from the PEER ground motion are used in the nonlinear time history in this study. As indicated in AASTHO 2007, "response-spectrum-compatible time histories are used developed from the representative recorded motion." Specifically, a response-spectrum-compatible time history refers to the response spectrum of the selected earthquakes falling in between 0.2 T and 1.5 T (T is the fundamental period); however, it may not be less than 90% of the corresponding design spectral acceleration for a damping ratio of 5%. In addition, the average value of the response spectrum within the designated period range may not be less than the average value of the corresponding design spectral accelerations. The ground motions used in this study are converted into response-spectrum-compatible data for return periods of 30, 475, and 2500 years.

This study aims to investigate the safety of the bridge against two hazards simultaneously through a probabilistic approach. The probability density distributions of the scour and earthquake magnitudes are incorporated into the evaluation process. The aforementioned probabilistic scour curve is used to address this fact with respect to the flood hazard. The seismic risk is measured using PSHA. The purpose of PSHA is to evaluate the hazard of seismic ground motion at a site by considering all possible earthquakes in the area, estimating the associated shaking at the site, and calculating the probabilities of these occurrences (McGuire, 2004). There are many assessments for seismic hazard analysis and two recent works related to Taiwan are described below. Campbell et al. (2002) developed a seismic hazard model for Taiwan to estimate earthquake losses and risk management. Their seismic hazard model is composed of two major components: a seismotectonic model and a ground-shaking model. Seismic hazard curves at a grid of sites across the island of Taiwan were calculated resulting in to a seismic hazard map. Wang et al. (2015) developed a seismic hazard assessment using MCS with earthquake statistics and local ground motion models. They found that the current seismic design in Taipei might not be as conservative as expected. Although the seismic hazard is important, developing a new seismic hazard model is beyond the scope of the current study. Instead, the model built from NCREE is commonly accepted in Taiwan and therefore, is adopted here. For details, please refer to Yeh and Jean (2007). Based on their model, a seismic hazard curve at a location close to the investigated bridge is built, as shown in Figure 2.

Construction of fragility curve

The displacement ductility (μ_Δ) is used to measure the structural performance under seismic excitations. The displacement ductility is defined as the ratio of the displacement of the bridge girder to the yield displacement of a pier, as indicated in Eq. (25) (Caltrans 2006).

$$\mu_\Delta = \mu_D / \mu_y \tag{25}$$

The yield displacement for a pier is the product of the yield rotation of the plastic section and the length of the pier, as shown in Eq. (26).

$$\mu_y = \theta_y l \tag{26}$$

where θ_y is the yield rotation corresponding to the condition wherein the reinforced bar starts to yield in the plastic hinge.

Eq. (27) is used to establish the relationship between the PGA and the displacement ductility.

$$\mu_\Delta = a(PGA)^b \tag{27}$$

Here, a and b are constants derived from the regression analysis. The fragility curve is a conditional probability computation, representing a failure probability for a given intensity measurement. For example, when the PGA is given, assuming that the capacity and demand of the bridge are log-normally distributed, the corresponding failure probability can be calculated using Eq. (28) as follows.

$$P_f\left(\mu_\Delta > \mu \middle| PGA = x\right) = 1 - \Phi\left(\frac{\ln\left(\frac{\mu}{aPGA^b}\right)}{\sigma_{\mu_\Delta|PGA}}\right) \tag{28}$$

Here, μ is the mean value of the capacity (Alipour et al. 2013), $a \times PGA^b$ is the mean value of the demand in terms of the displacement ductility, σ is the standard deviation with respect to the limit state, and Φ is the cumulative probability density function of the standard normal. Based on the study by Alipour (2013), the capacity of the displacement ductility for varied limit states are $1 < \mu < 2$, $2 < \mu < 4$, $4 < \mu < 7$, $\mu > 7$ for slight, moderate, major, and complete collapse damages, respectively. The standard deviation for a given PGA ($\sigma_{\mu_\Delta|PGA}$) is calculated using Eq. (29).

$$\sigma_{\mu_\Delta|PGA} = \sqrt{\sigma^2_{D|PGA} + \sigma^2_c} \tag{29}$$

where $\sigma_{D|PGA}$ is the standard deviation of the demand for a given PGA, and σ_c is the standard deviation of the capacity (i.e., 0.5) (NCREE 2009). $\sigma_{D|PGA}$ is obtained by performing another regression analysis as indicated in Eq. (30).

$$\sigma_{D|PGA} = c\left(PGA\right)^f \tag{30}$$

Case study

General information of the investigated bridge

The Nanyun Bridge, located in the central Taiwan, is selected for the case study. Specifically, pier 14 (P14), pier 15 (P15), and the superstructure between them are considered. Both piers are solid concrete section. However, the caissons below are hollow cylinders with an outside diameter of 5.5 m and an inside diameter of 4.5 m. The concrete strengths are 28 MPa and 21 MPa for the bridge pier and caisson, respectively. The SD280 steel bar is used for diameters less than or equal to 16 mm whereas SD420W is used for diameters greater than 16 mm.

Analyses results

The MCS is used to simulate the variation in the scour depth for the Nanyun Bridge, wherein the water depth and water velocity are reproduced via LN (1.5933, 0.1798) and LN (0.6692, 0.4173), respectively (Liao et al. 2012). The histogram of scour depth is obtained through a simulation with a sample size of 10^6. Based on the histogram, the scour risk curve can be established as shown in Figure 3.

To determine the failure probability of the scoured Nanyun Bridge for a given PGA, three different scour depths and five different sets of ground motions are used. A total of 105 time-history analyses are performed, as given in Table 2. In addition to return periods of 30, 475, and 2500 years, this study performs another two sets of ground motions corresponding to PGAs of 1.007 and 1.510. To draw a fragility curve for a given limit state, a continuous failure-probability function in terms of PGA is required. The 105 time-history analyses only provide failure probabilities at five different PGA values. Therefore, as explained, the regression analysis is employed to build the fragility curve. Table 3 lists the mean values and standard deviations of the ductility displacement for a bridge with a scour depth of 4 m under five different PGA values. Each set of PGA has seven different ground motions. The average of the seven responses yields the mean value. Similarly, Tables 4 and 5 list mean values and standard deviations of the ductility displacement for a bridge with scour depths of 8 and 10 m under five different PGA values, respectively. Table 6 provides detailed regression results for mean and standard deviation of displacement ductility for scour depths of 4, 8, and 10 m.

Figure 4 shows the fragility curves for a bridge with scour depths of 10 m. Figure 5 shows the fragility curves with different scour depths at moderate damage state. The results show that the failure probability increases with the increase in the scour depth and decreases as the limit state changes from slight to collapse. More importantly, the failure probability was found to increase significantly as the scour depth changes from 8 to 10 m for each limit state.

The probability of bridge failure by exceeding a given limit state of k, DS_k, under the scour event of SC_i, and the earthquake demand of EQ_j can be calculated as shown in Eq. (31) (Alipour et al. 2013).

$$\left(P_f\right)_{ijk} = P\left(SC_i \cap DS_j \cap EQ_k\right) \tag{31}$$

The probability of the simultaneous occurrence of two extreme events (i.e., scour and earthquake) is generally small. Three models for considering the combination effects of extreme loads using reliability approaches are often adopted in practical applications. They are: (1) Turkstra’s rule, (2) the Ferry Borges–Castanheta model, and (3) Wen’s load coincidence method (Ghosn et al. 2003). Turkstra’s model considers one load reaching its maximum value combined with another load with its mean value, which looks rational, but the results are generally unconservative (Sun et al. 2014). The Ferry-Borges model, on the other hand, is more accurate than Turkstra’s rule because it takes the rate of occurrence of the loads and their time duration into consideration (Ghosn et al. 2003). The Turkstra’s rule and the Ferry Borges–Castanheta model assume independence between two different load types. Conversely, the Wen’s method considers the rate of occurrence of each load event and the rate of simultaneous occurrences of a combination of two or more correlated loads (Wen, 1990). Many researchers have made great efforts on investigating the load combination effect. It is very unusual to find scour occurs that follow earthquakes in Taiwan. This study investigated the safety performance of a scoured bridge under seismic excitations. The time difference between the occurrence of a flood and an earthquake would justify assuming independence between earthquakes and scour events. Thus, this study considers the occurrence probabilities of scour and earthquake events to be statistically independent in calculating their combination effects using simulation approach. Eq. (31) can be calculated as shown in Eq. (32).

$$\left(P_f\right)_{ijk} = P\left(SC_i\right)P\left(DS_j\right)P\left(EQ_k\right) \tag{32}$$

Here, $P(SC_i)$ is the probability of experiencing the i^{th} scour scenario, which is obtained from the scour risk curve, shown in Figure 3. $P(DS_j)$ is the probability of failure under a specific-damage state, which is estimated from the seismic-fragility curve obtained for different scour depths as shown in Figures 4–5. $P(EQ_k)$ is the occurrence probability of the k^{th} earthquake scenario defined in the probabilistic seismic-hazard curve in terms of PGA as shown in Figure 2. The joint-failure probabilities are developed within a PGA range of 0.1 to 0.7 because of the data span of the NCREE seismic-hazard curve. Interpolation and extrapolation are used to estimate the failure probability for scour depths of 4, 8, and 10 m. Figures 6 shows the joint probability of failure for moderate damage state.

A deterministic design value (i.e., scour depth), considering both the hazards, for a given reliability target is derived to help engineers in their present design processes as described below. The 3D plot of the joint probability of failure (Figure 6) is reduced to a 2D plot using a fixed PGA value. To be compatible with the present practice, the design PGA of the Nanyun Bridge is used (i.e., 0.32 g). Figure 7 illustrates an example for the moderate-damage state. If the target of reliability index (β) is three, the required scour depth can be derived, which is approximately equal to 5 m as indicated in Figure 7. That is, engineers can follow their regular process in designing bridges and if the safety of a bridge with a scour depth of 5 m is confirmed, the reliability of such bridges against floods and earthquakes is ensured at a value of 0.99865 for a moderate-damage state.

A sudden increase in the probability is observed for scour depths greater than 8 m, as shown in Figures 6–7. The joint-failure probability increases if the scour depth is greater than 8 m. As shown in Eq. (32), the joint-failure probability largely depends on two probabilities: the probability of a given scour depth ($P(SC_i)$) and failure probability of a bridge with the specified scour depth ($P(DS_j)$). The occurrence rate of a given scour depth ($P(SC_i)$) is a monotonically decreasing function as shown in Figure 3. However, the failure probability of the Nanyun Bridge increases significantly for scour depths greater than 8 m, as shown in Figure 5. The caisson depth for the Nanyun Bridge is approximately 14 m, thereby increasing the failure probability considerably. The failure probability dominates the joint probability for all the damage states in this case study.

Conclusions

Bridges are important infrastructures and their safety should be ensured. Based on the literature, both floods and earthquakes are found to be the main threats concerning the safety of bridges in Taiwan. The uncertainties involved in such hazards are inevitable; hence, a probabilistic approach is employed in this study. This study integrates the non-uniform scour-depth prediction, nonlinear time-history analyses, nonlinear soil property, and moment-curvature analyses to establish fragility curves to evaluate the safety of a bridge against floods and earthquakes. To demonstrate the proposed evaluation process, the Nanyun Bridge, which is located in the Nantou County, is selected for the case study. Piers 14 and 15 of the Nanyun Bridge are modeled for a sour depth of 4 m, which is currently observed. The plastic hinges are predefined at each pier located 1 m below the ground level because of the presence of nonlinear soil link. Based on the results, the conclusions of this study are as follows.

1. The Nanyun Bridge is likely to experience a flood scour with a depth in the range of 3–5 m, based on calculations from the proposed formula, which is consistent with the on-site observation.

2. The failure probability for each limit state is insignificant. The failure probability is significant only for the slight and moderate-damage states. For example, the failure probabilities are 0.42 and 0.84 for moderate and slight limit states, respectively (for a PGA of 0.5 g and a scour depth of 4 m).

3. The failure probability against seismic attacks is not proportional to the scour depth. The results show that the failure probability does not significantly increase when the scour depth increases from 4 m to 8 m. However, the failure probability considerably changes when the scour depth increases from 8 m to 10 m. This significant change in the failure probability affects the shape of the joint-failure probability in the range of 8–10 m.

4. A deterministic design value, considering both the scour and seismic hazards, is proposed for a given reliability target. For example, if the reliability target index (β) of three is specified, the corresponding design scour depth is approximately 5 m for the moderate-limit state.

Acknowledgments

This study was supported by the TU-NTUST Joint Research Program. The support is gratefully acknowledged.

References

Alipour, A., Shafei, B., and Shinozuka, M. (2013). "Reliability-Based Calibration of Load and Resistance Factors for Design of RC Bridges under Multiple Extreme Events: Scour and Earthquake." Journal of Bridge Engineering ASCE 18(5) 362–371.

Andric´, J. M., and Lu, D. G. (2016). "Risk Assessment of Bridges under Multiple Hazards in Operation Period." Safety Science 83 80–92.

Basoz, N. I., Kiremidjian, A. S., King, S. A., and Law, K. H. (1999). "Statistical Analysis of Bridge Damage Data from the 1994 Northridge, CA, earthquake." Earthquake Spectra, 15(1) 25–54.

Campbell, K. W., Thenhaus, P. C., Barnhard, T. P., and Hampson, D. B. (2002). "Seismic Hazard Model for Loss Estimation and Risk Management in Taiwan." Soil Dynamics and Earthquake Engineering, 22(9) 743–754.

Chang, K. C., Tsai, Y. C., Chang, D. W., Sung, Y. C., Liao, W. Y., Chai, J. F., Hung, H. H., Liu, K. Y., Wu, H. M., Qi, S. R., and Chen, Y. H. (2009). "Seismic Assessment and Retrofit Manual for Highway Bridges." NCREE 0-09-028, Taipei.

Davis-McDaniel C., Chowdhury M., Pang W. C., and Dey K. (2013) "Fault-Tree Model for Risk Assessment of Bridge Failure: Case Study for Segmental Box Girder Bridges." Journal of Infrastructure Systems, 19(3) 326–334.

Elnashai, A. S., Gencturk, B., Kwon, O. S., Hashash, Y. M., Kim, S. J., Jeong, S. H., and Dukes, J. (2012). "The Maule (Chile) Earthquake of February 27, 2010: Development of Hazard, Site Specific Ground Motions and Back-Analysis of Structures." Soil Dynamics and Earthquake Engineering, 42 229-245.

Ghosn, M., Moses, F., and Wang, J. (2003). "Design of highway bridges for extreme events (No. 489)." Transportation Research Board.

HEC-18 (2012). “Evaluating Scour at Bridges, Fifth Edition.” F. H. Administration. US, U.S Department of Transportation.

Honjo, Y., Kusakabe, O., Matsui, K., Kouda, M., and Pokhard, G. (2002) “Foundation Design Codes and Soil Investigation in View of International Harmonization and Performance Based Design.” Proceedings of the IWS Kamakura Conference, Japan, April, 10–12.

Hsu, Y. T., and Fu, C. C. (2004). “Seismic Effect on Highway Bridges in Chi Chi Earthquake.” Journal of performance of constructed facilities, 18(1) 47–53.

Liao, K. W., Lu, H. J., and Wang, C.Y. (2015). “A Probabilistic Evaluation of Pier-Scour Potential in the Gaoping River Basin of Taiwan.” J. Civ. Eng. Manag. 21 637–653.

Mander, J. B., Priestley, M. J., and Park, R. (1988). “Theoretical stress-strain model for confined concrete.” Journal of structural engineering, 114(8) 1804-1826.

McGuire, R. K. (2004). “Seismic Hazard and Risk Analysis.” Earthquake Engineering Research Institute.

Melville, B. W. and Coleman, S. E. (2000). “Bridge Scour.” United States of America, Water Resources Publications, LLC.

Ministry of Transportation and Communications (2009). “Code for Design of Highway Bridges.” Taipei, Taiwan.

Ministry of Transportation and Communications (2015). “Seismic Design Specifications for Highway Bridges.” Taipei, Taiwan.

Padgett, J., DesRoches, R., Nielson, B., Yashinsky, M., Kwon, O. S., Burdette, N., and Tavera, E. (2008). “Bridge Damage and Repair Costs from Hurricane Katrina.” Journal of Bridge Engineering, 13(1) 6–14.

Sun, D., Wang, X., and Sun, B. (2014). “A Methodology for Multihazards Load Combinations of Earthquake and Heavy Trucks for Bridges.” The Scientific World

Journal, 2014 1–9.

Sung Y. C., Liu, K. Y., Su, C. K., Tsai, I. C., and Chang K. C. (2005). “A study on pushover analyses of reinforced concrete columns.” J Struct Eng Mech, 21(1) 35–52.

Wang, J. P., Wu, Y. M., and Huang, D. (2015). “Major Earthquakes around Taipei and a Seismic Hazard Assessment with Monte Carlo Simulation.” Natural Hazards Review, 16(4) 1–7.

Wen, Y. K. (1990). “Structural Load Modeling and Combination for Performance and Safety Evaluation.” Elsevier Science Ltd.

Yeh, C. H., and Jean, W. Y. (2007) “Study on the Integration of Seismic Hazard Analysis and Disaster Simulation Technology (II).” NCREE-07-040, Taipei.

Table 1. Accuracies comparison among different approaches (2000)

Soil covering depth	mean absolute percentage error (MAPE)	
	Proposed approach	Melville and Coleman (2000)
(1) $Y > 2.4b_c$	5.1	12.22
(2) $2.4b_c > Y \geq 0$	30.4	106.28
(3) $0 > Y > -y$	34.2	93.50
(4) $Y \leq -y$	24.8	236.69
Average	28.9	102.75

Table 2. Summary of total time-history analyses conducted in this study

Name	Earthquake	PGA (Return period)	Scour depth	Total No.
Contents	San Fernando			
	Imperial Valley	0.091 (30 years)		
	Loma Prieta	0.363 (475 years)	4 m	
	Northridge	0.453 (2500 years)	8 m	105
	Kobe	1.007	10 m	
	Chi-Chi (TCU52)	1.510		
	Chi-Chi (TCU68)			

Table 3. PGAs corresponding to μ_Δ and $\sigma_{D|PGA}$ for scour depth of 4 m

PGA	Mean of μ_Δ	$\sigma_{D|PGA}$
0.091	0.318	0.012
0.363	1.243	0.095
0.453	1.600	0.133
1.007	4.756	0.545
1.510	5.118	0.802

Table 4. PGAs corresponding to μ_Δ and $\sigma_{D|PGA}$ for scour depth of 8 m

PGA	Mean of μ_Δ	$\sigma_{D|PGA}$
0.091	0.386	0.0217
0.363	1.402	0.0826
0.453	1.635	0.118
1.007	5.634	1.080
1.510	6.312	1.069

Table 5. PGAs corresponding μ_Δ and $\sigma_{D|PGA}$ for scour depth of 10 m

PGA	Mean of μ_Δ	$\sigma_{D|PGA}$
0.091	0.493	0.035
0.363	1.934	0.146
0.453	2.353	0.240
1.007	5.616	0.889
1.510	8.485	0.934

Table 6. Regression results for mean and standard deviation of displacement ductility for scour depths of 4, 8, and 10 m

Scour Depth (m)	Constant			
	a	b	c	f
4	3.80	1.04	0.46	1.53
8	4.40	1.05	0.60	1.51
10	5.48	1.01	0.65	1.25