# Earthquake Damage Potential of Bridges Subjected to Flood and Seismic Hazards

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**SUMMARY:**
Many bridges located in seismic risk regions also suffer serious foundation exposure due to riverbed scour. Loss of surrounding soil can significantly reduce the lateral strength of the pile foundation, resulting in potential for undesired damages in the piles during an earthquake. This paper presents an analytical approach suitable for assessing the seismic damage potential of bridges with foundation exposure. The approach follows the well-accepted response spectrum analysis method to determine the maximum seismic response of a bridge. The influence of soil-structure interaction on the seismic performance of bridges is incorporated in the analysis process. The damage potential of the bridge is assessed by comparing the imposed seismic demand with the lateral strengths of the column and pile foundation. The versatility of the proposed approach is illustrated using a numerical example, which highlights that undesired damages can be expected in pile foundations even if the scour-depth is relatively small.

*Keywords: soil-structure interaction; seismic performance assessment; pile exposure; bridges; multiple hazards*

## 1. INTRODUCTION

According to a study published by the World Bank (Dilley *et al.* 2005), approximately 3% of the Earth's land area and more than 11% of the world's population are highly exposed to at least two natural hazards, including earthquakes, floods, cyclones, etc. Civil structures located in these regions must withstand the impacts from various natural hazards. Although different natural hazards rarely strike simultaneously, damages caused by one natural disaster can affect the performance of a structure in the hazard events occurred afterwards. A notable example is bridges located in the regions subjected to flood and earthquake hazards. During a flood, the rapid water flow causes severe erosion of the river channel. As the water flow hits the bridge pier, a strong downward current impinges the riverbed and digs a scour hole adjacent to the pier. A vortex system is then developed around the pier when the oncoming flow runs into the scour hole and passes through the pier. The interaction between the downward current and the vortices exacerbates the erosion of the soil around the pier, resulting in bridge foundation exposure after the flood, as shown in Fig. 1.1. For a bridge with an exposed pile foundation subjected to a horizontal seismic motion, the lateral force from the inertia of the superstructure induces a large flexure demand in the above-ground portion of the pile, particularly at the pile/pile-cap connection. However, the lateral strength of the foundation may be reduced considerably due to exposure of piles. Loss of the surrounding soil also brings down the lateral stiffness of the bridge foundation. The reduction of lateral stiffness gives the bridge a longer natural period of vibration, and possibly a different seismic force. Consequently, the seismic performance of a bridge with foundation exposure is totally different from what the bridge was originally designed.

In current practices, foundation systems of most bridges are strategically designed to remain elastic even under severe seismic demands. The approach is intended to avoid the difficulty of the post-earthquake inspection and the high cost associated with repairing the foundation if damage or failure occurs. By following a capacity design principle, an elastic response of the foundation can be ensured by increasing the strength of the foundation above that of the bridge columns, as well as the seismic demand imposed on the foundation, so that plastic hinges develop in the column instead of the

(a) (b)

**Figure 1.1.** Bridge foundation exposure due to riverbed scour (courtesy of (a) Dr. C. Lin and (b) Dr. Y.C. Lin)

foundation. For bridges located in the regions under both flood and seismic hazards, however, pile exposure due to riverbed scour cuts down the lateral strength of the foundation. When the scour depth exceeds a critical level, the lateral strength of the foundation may not be sufficient to protect piles from yielding, leading to potential for undesired damages in the foundation when subjected to earthquake excitations. Depending on the intensity of the ground motion, bridges supported on damaged piles may suffer unacceptable level of tilting and residual deformation, significantly affecting the serviceability of the structure. Since many bridges located in seismic regions also suffer serious scour problems, an assessment of the seismic damage potential of a bridge with foundation exposure can help the engineers to evaluate the need for foundation retrofit, particularly if a certain level of performance is to be guaranteed for the structure.

An analytical approach suitable for assessing the seismic damage potential of bridges with foundation exposure is presented in this paper. The approach idealizes the soil-foundation-structure system of a bridge bent to a two-degree-of-freedom system, as shown in Fig. 1.2. The seismic performance of the bridge bent is assessed by comparing the imposed seismic demand with the lateral strengths of the column and the foundation. The procedure follows the well-accepted response spectrum analysis method to obtain the seismic demand on the bridge (Chopra 2011). The strength of foundations is assessed using an analytical model, which is capable of assessing the performance of a soil-pile system at its pertinent yield limit states (Song *et al.* 2008). The influence of soil-structure interaction on the seismic response of a bridge is incorporated in the analysis process. The versatility of the proposed approach is illustrated using an illustrative example. Results highlight that the lateral strength of a foundation reduces significantly with increasing the depth of pile exposure. Undesired foundation damages can be expected under a relatively small scour-depth.

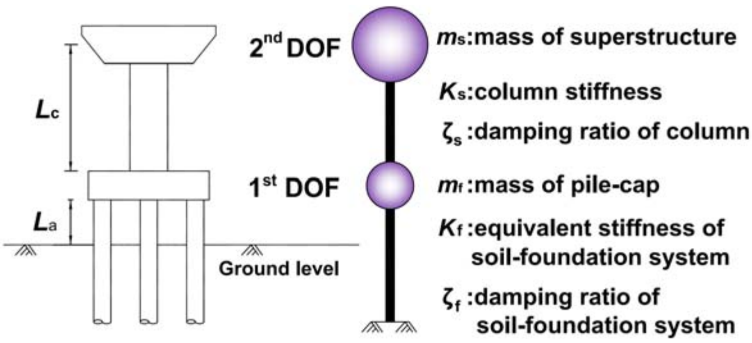


**Figure 1.2.** Idealized two-degree-of-freedom model of a bridge bent

## 2. DAMAGE POTENTIAL ASSESSMENT OF BRIDGES WITH FOUNDATION EXPOSURE

### 2.1. Bridge Model

Current seismic design of bridges primarily considers the performance of the structure under transverse earthquake loads. When subjected to earthquake excitations, the overall response of a bridge structure is closely related to its mass and stiffness. Guidance on properly modelling a bridge structure for the assessment of its seismic response is available in Priestley *et al*. (1996). The analytical approach proposed in this paper is intended for straight multi-span bridges located in midstream or downstream of the river. The structure is assumed to have a fairly uniform distribution of mass, stiffness and strength between bents. It is further assumed that all piles in the bridge foundation undergo approximately the same level of exposure. The seismic response of the bridge may be characterized by the response of a single bridge bent subjected to transverse earthquake excitation. With a consideration of the interaction between the foundation and structure, the bridge bent is modelled as a two-degree- of-freedom system, as illustrated earlier in Fig. 1.2.

The seismic mass of the superstructure $m_s$ is obtained from the two adjacent half spans of the superstructure. The mass of the columns is assumed to be relatively small, comparing to that of the superstructure, and has little influence on the overall seismic response (Priestley *et al*. 1996). Similarly, the mass of the foundation $m_f$ is assumed to be contributed by the pile-cap alone and the mass of piles is negligible. A coefficient $\beta_m$ is defined as the ratio between the mass of the pile-cap and the mass of the superstructure, i.e. $\beta_m \equiv m_f / m_s$. By assigning the translational displacement of the pile-cap as the first degree-of-freedom (DOF) and the translational movement of the superstructure as the second DOF, the mass matrix $\mathbf{m}$ of the bridge bent model can be expressed as:

$$\mathbf{m} = m_s \begin{bmatrix} \beta_m & 0 \\ 0 & 1 \end{bmatrix} \tag{2.1}$$

Under a lateral load, the stiffness of bridge columns $K_s$ may be calculated by (Priestley *et al*. 1996):

$$K_s = N_c \alpha_c \frac{(EI_e)_c}{L_c^{\,3}} \tag{2.2}$$

where $N_c$ is the number of columns in the bridge bent, $(EI_e)_c$ is the effective flexural rigidity of the column and $L_c$ is the column height. The coefficient $\alpha_c$ represents the boundary condition of the column, where $\alpha_c = 3$ for the single-column bridge bent with a cantilever column and $\alpha_c = 12$ for the multi-column bridge bent with a very stiff cap-beam. A common approach to determine the lateral stiffness of a pile foundation assumes that the soil-pile system can be modelled as a flexural member supported laterally by a series of closely spaced springs, which provide a soil reaction that is proportional to the lateral deflection of the pile. In current practices, the soil is broadly divided into cohesive and cohesionless soils. The stiffness of cohesive soils is assumed to be independent of the depth, resulting in a constant horizontal subgrade reaction $k_h$ (in units of force/length$^2$). The soil resistance of cohesionless soils is commonly modelled with a constant rate of increase of modulus of horizontal reaction, denoted by $n_h$ (in units of force/length$^3$). Guidance for selecting the appropriate values of soil subgrade coefficients is available in literature. The constant modulus of subgrade reaction $k_h$ for cohesive soils may be taken as $k_h = 67 s_u$, as suggested by Davisson (1970), where $s_u$ is the undrained shear strength of the soil. For cohesionless soils, an estimation of $n_h$ and its correlation with the effective friction angle $\overline{\phi}$ and relative density $D_r$ of the soil can be made following the suggestion of ATC-32 (1996). For bridge foundations with a group of piles, the correlation between the lateral stiffness of the soil-foundation system and the above-ground height $L_a$ of the piles is given by (Song *et al*. 2008):

$$K_f = \begin{cases} N_p \times \dfrac{1}{\frac{1}{12}\xi_a^{\,3} + \frac{1}{2\sqrt{2}}\xi_a^{\,2} + \frac{1}{2}\xi_a + \frac{1}{\sqrt{2}}} \dfrac{(EI_e)_p}{R_c^{\,3}} & \text{for cohesive soils} \\[2ex] N_p \times \dfrac{1}{\frac{1}{12}\xi_a^{\,3} + \frac{7}{16}\xi_a^{\,2} + \frac{6}{7}\xi_a + \frac{15}{16}} \dfrac{(EI_e)_p}{R_n^{\,3}} & \text{for cohesionless soils} \end{cases} \tag{2.3}$$

where $N_p$ is the number of piles in the foundation, $(EI_e)_p$ is the effective flexural rigidity of the pile, $R_c$ is the characteristic length of a pile in cohesive soils, which is defined as $R_c \equiv \sqrt[4]{(EI_e)_p / k_h}$, and $R_n$ is the characteristic length of a pile in cohesionless soils, which is defined as $R_n \equiv \sqrt[5]{(EI_e)_p / n_h}$. The above-ground height coefficient $\xi_a$ is defined as the pile above-ground height $L_a$ normalized by the characteristic length of the soil-pile system, i.e. $\xi_a \equiv L_a / R_c$ for cohesive soils and $\xi_a \equiv L_a / R_n$ for cohesionless soils. In this paper, the level of the riverbed is assumed to be at the pile/pile-cap interface before the commencement of scour, and hence the scour-depth is taken to be equal to the above-ground height $L_a$ of the pile. It can be seen from Eqn. 2.3 that the lateral stiffness of the pile foundation decreases with increasing the scour-depth. By defining the stiffness ratio $\beta_k \equiv K_f / K_s$, the stiffness matrix $\mathbf{k}$ of the bridge bent can be expressed as:

$$\mathbf{k} = K_s \begin{bmatrix} \beta_k + 1 & -1 \\ -1 & 1 \end{bmatrix} \tag{2.4}$$

The mass matrix $\mathbf{m}$ and stiffness matrix $\mathbf{k}$ are used to assess the dynamic response characteristics, including the natural periods of vibration and mode shapes, of the bridge bent.

## 2.2. Natural Periods and Mode Shapes of Vibration

Seismic design of bridges is generally based on the peak response of the structure under a design level earthquake. A handy approach to assess the peak seismic response quantities relies on the earthquake response spectrum, once the natural vibration period of the structure has been determined. Since the damping in most bridges is significantly less than the critical damping, the influence of structural damping on the natural period and vibration mode shape is negligible. For a bridge bent modelled as a two-degree-of-freedom system, the undamped free vibration of the bridge bent can be described by:

$$\mathbf{m} \cdot \ddot{\mathbf{x}}(t) + \mathbf{k} \cdot \mathbf{x}(t) = 0 \tag{2.5}$$

where $\mathbf{x}(t)$ is the displacement vector of the structure. The characteristic equation of Eqn. 2.5 is expressed as

$$(\mathbf{k} - \omega^2 \mathbf{m})\mathbf{x} = 0 \tag{2.6}$$

where the two roots of $\omega^2$ represents the natural circular frequencies of the two vibration modes. Substituting the mass matrix $\mathbf{m}$ in Eqn. 2.1 and the stiffness matrix $\mathbf{k}$ in Eqn. 2.4 into the characteristic equation 2.6, the natural circular frequencies of the first vibration mode, $\omega_1$, and the second vibration mode, $\omega_2$, are given by:

$$\omega_1 = \sqrt{\frac{\lambda_a - \lambda_c}{2\beta_m} \frac{K_s}{m_s}} \qquad \omega_2 = \sqrt{\frac{\lambda_a + \lambda_c}{2\beta_m} \frac{K_s}{m_s}} \tag{2.7}$$

where $\beta_m$ is the mass ratio, $m_s$ is the mass of the superstructure and $K_s$ is the lateral stiffness of the bridge columns. The coefficients $\lambda_a$ and $\lambda_c$ are defined as $\lambda_a \equiv \beta_k + \beta_m + 1$ and $\lambda_c \equiv \sqrt{\beta_k^2 - 2\beta_k(\beta_m - 1) + (\beta_m + 1)^2}$. The natural period of each vibration mode can be then calculated by:

$$T_1 = 2\pi\sqrt{\frac{2\beta_m}{\lambda_a - \lambda_c} \frac{m_s}{K_s}} \qquad T_2 = 2\pi\sqrt{\frac{2\beta_m}{\lambda_a + \lambda_c} \frac{m_s}{K_s}} \tag{2.8}$$

The mode shape of the each vibration mode is determined by substituting the individual circular frequency from Eqn. 2.7 into the characteristic equation 2.6. Assuming a unit displacement in the second DOF, the mode shapes corresponding to the first and second vibration modes, denoted as $\boldsymbol{\phi}_1$ and $\boldsymbol{\phi}_2$, respectively, are expressed as:

$$\boldsymbol{\phi}_1 = \{\phi_{11} \quad \phi_{21}\}^T = \left\{\frac{2}{\lambda_b + \lambda_c} \quad 1\right\}^T \qquad \boldsymbol{\phi}_2 = \{\phi_{12} \quad \phi_{22}\}^T = \left\{\frac{2}{\lambda_b - \lambda_c} \quad 1\right\}^T \tag{2.9}$$

where the coefficient $\lambda_b$ is defined as $\lambda_b \equiv \beta_k - \beta_m + 1$.

## 2.3. Equivalent Damping Ratio

Although the damping of a structure system has very little influence on the dynamic response characteristics, it could significantly affect the magnitude of the structure's response during an earthquake. Seismic design of bridges using the response spectra requires identifying the equivalent damping ratios for the vibration modes considered in the design. For a bridge bent shown in Fig. 1.2, the compliance of the surrounding soil gives the soil-foundation system a damping ratio, which is higher than that of the bridge column and superstructure. The idealized two-degree-of-freedom system indeed consists of two subsystems with different levels of damping. In this case, the equivalent damping ratio corresponding to the first vibration mode is expected to be different from that of the second vibration mode. Guidance on assessing the modal damping ratio of a soil-structure system with a consideration of the additional damping from the surrounding soil is available in literatures (Priestley *et al*. 2007; Chopra 2011). In this paper, the equivalent modal damping ratio is calculated using an equation originally proposed for based-isolated bridges (Hwang *et al*. 1994) and later adopted in soil-structure interaction analyses (MOTC 2008). For a multiple-degree-of-freedom system comprising total $n$ subsystems with different levels of damping, the equivalent damping ratio of the $i^{th}$ vibration mode $(\zeta_e)_i$ can be estimated by:

$$(\zeta_e)_i = \frac{\sum_{j=1}^{n} K_j \, \phi_{ji}^2 \, \zeta_j}{\boldsymbol{\phi}_i^T \mathbf{k} \, \boldsymbol{\phi}_i} \tag{2.10}$$

where $\mathbf{k}$ is the stiffness matrix of the multiple-degree-of-freedom system, $\boldsymbol{\phi}_i$ is the mode shape of the $i^{th}$ vibration mode, $K_j$ is the stiffness of the $j^{th}$ subsystem, $\phi_{ji}$ is the deformation of the $j^{th}$ subsystem in the $i^{th}$ vibration mode shape, and $\zeta_j$ is the damping ratio of the $j^{th}$ subsystem. Substituting the stiffness matrix in Eqn. 2.4 and vibration mode shapes in Eqn. 2.9 into Eqn. 2.10, the equivalent damping ratio $(\zeta_e)_1$ corresponding to the first vibration mode of the bridge bent is given by:

$$(\zeta_e)_1 = \frac{K_f \phi_{11}^2 \zeta_f + K_s (\phi_{21} - \phi_{11})^2 \zeta_s}{\boldsymbol{\phi}_1^T \mathbf{k} \, \boldsymbol{\phi}_1} = \frac{4\beta_k \zeta_f + (\lambda_b + \lambda_c - 2)^2 \zeta_s}{4\beta_k + (\lambda_b + \lambda_c - 2)^2} \tag{2.11}$$

and the equivalent damping ratio $(\zeta_e)_2$ of the second vibration mode can be calculated by:

$$(\zeta_e)_2 = \frac{K_f \phi_{12}^2 \zeta_f + K_s (\phi_{22} - \phi_{12})^2 \zeta_s}{\boldsymbol{\phi}_2^T \mathbf{k} \, \boldsymbol{\phi}_2} = \frac{4\beta_k \zeta_f + (\lambda_b - \lambda_c - 2)^2 \zeta_s}{4\beta_k + (\lambda_b - \lambda_c - 2)^2} \tag{2.12}$$

where $\zeta_f$ is the damping ratio of the soil-foundation system, $\zeta_s$ is the damping ratio of the above-ground portion of the bridge, and $\beta_k$ is the stiffness ratio coefficient.

## 2.4. Seismic Demand Assessment: Response Spectrum Analysis Approach

For a multiple-degree-of-freedom system subjected to an earthquake excitation, the maximum seismic demand can be estimated by a combination of the peak response quantities for individual vibration modes (Priestley *et al*. 1996; Chopra 2011). The level of a specific vibration mode participating to the overall dynamic response is evaluated by the modal participation factor, which depends on the mass of each DOF and the mode shape of the vibration mode considered. For the two-degree-of-freedom bridge bent shown in Fig. 1.2, the modal participation factors of the first vibration mode $\Gamma_1$ may be obtained by:

$$\Gamma_1 = \frac{m_f \phi_{11} + m_s \phi_{21}}{m_f {\phi_{11}}^2 + m_s {\phi_{21}}^2} = \frac{(\lambda_b + \lambda_c)(2\beta_m + \lambda_b + \lambda_c)}{4\beta_m + (\lambda_b + \lambda_c)^2} \tag{2.13}$$

and the modal participation factors of the second vibration mode $\Gamma_2$ is given by:

$$\Gamma_2 = \frac{m_f \phi_{12} + m_s \phi_{22}}{m_f {\phi_{12}}^2 + m_s {\phi_{22}}^2} = \frac{(\lambda_b - \lambda_c)(2\beta_m + \lambda_b - \lambda_c)}{4\beta_m + (\lambda_b - \lambda_c)^2} \tag{2.14}$$

where $\beta_m$ is the mass ratio of the bridge bent. Upon the determination of the modal participation factors $\Gamma$, the peak seismic response for the $i^{th}$ vibration mode, denoted as $R_i$, can be assessed by:

$$\mathbf{R}_i = \phi_i \, \Gamma_i \, S_i(T_i) \tag{2.15}$$

where $\phi_i$ is the mode shape of the $i^{th}$ vibration mode, $\Gamma_i$ is the modal participation factor of the $i^{th}$ vibration mode and $S_i(T_i)$ is the spectral response predicted for a damping ratio of $\zeta_i$ at the natural period $T_i$, where $\zeta_i$ and $T_i$ are the damping ratio and natural period of the $i^{th}$ vibration mode, respectively. Techniques for combining the modal responses are discussed in literature (Priestley 1996; Chopra 2011). A direct summation of the peak response quantities for all vibration modes overestimates the seismic demand on the structure. In this paper, the square-root-of-sum-of-squares (SRSS) rule, which is suitable for peak response combination of strictures with well separated natural periods, is used to assess the maximum seismic demand **R** imposed on the bridge bent, i.e.

$$\mathbf{R} = \sqrt{\sum_i \mathbf{R}_i} \tag{2.16}$$

By combining of Eqns. 2.9, and 2.13—2.16, the maximum lateral force applied on the bridge columns during an earthquake can be estimated by:

$$(V_s)_{dem} = \sqrt{\left(\Gamma_1 \, m_s \, S_{a1}(T_1)\right)^2 + \left(\Gamma_2 \, m_s \, S_{a2}(T_2)\right)^2} \tag{2.17}$$

where $S_{a1}(T_1)$ is the spectral acceleration for a system with a equivalent damping ratio equal to $(\zeta_e)_1$ and a natural period of $T_1$, and $S_{a2}(T_2)$ is the spectral acceleration for a system with a damping ratio equal to $(\zeta_e)_2$ and a natural period $T_2$. When subjected to an earthquake excitation, the lateral forces acting on the pile foundation include the base shear of the bridge column and the seismic force associated with the inertia of the pile-cap. The combination of Eqns. 2.9 and 2.13—2.16 also gives the maximum force applied on the pile group, i.e.:

$$(V_f)_{dem} = V_s + \sqrt{\left(\frac{2\beta_m}{\lambda_b + \lambda_c} \Gamma_1 m_s S_{a1}(T_1)\right)^2 + \left(\frac{2\beta_m}{\lambda_b - \lambda_c} \Gamma_2 m_s S_{a2}(T_2)\right)^2} \tag{2.18}$$

The set of Eqns. 2.8, 2.11, 2.12, 2.17 and 2.18 allows the seismic demand imposed on the bridge bent to be assessed using the mass ratio $\beta_m$ and stiffness ratio $\beta_k$. It is worth to note that since the effective stiffness of the foundation $K_f$ decreases with increasing the scour-depth of the pile, as evident in Eqn. 2.3, the seismic demands imposed to the bridge column and foundation can be significantly influenced by the riverbed scour.

### 2.5. Lateral Strengths of Bridge Column and Foundation

In this paper, the seismic performance of a bridge bent with foundation exposure is assessed by comparing the imposed seismic demand given by Eqns. 2.17 and 2.18 with the lateral strengths of the columns and foundation at their yield limit states. For bridge columns subjected to lateral loads, the force required to cause the formation of the plastic hinges $(V_s)_{cap}$ can be determined by:

$$(V_s)_{cap} = N_c \frac{(M_u)_c}{L_c'} \tag{2.19}$$

where $N_c$ is the number of columns, $(M_u)_c$ is the flexural strength of the column and $L_c'$ is the distance from the plastic hinge to the point of contraflexure in the column. For a pile foundation subjected to a horizontal seismic motion, the lateral force acting on the piles may be significant, in which case, can result in sequential yielding along the pile. The first yield limit state is identified by the bending moment reaching the flexural strength of the pile $(M_u)_p$ at the pile/pile-cap connection. The lateral force required to cause the formation of the plastic hinge at the pile-head is given by (Song *et al.* 2008):

$$(V_f)_{cap} = \begin{cases} N_p \dfrac{\xi_a + \sqrt{2}}{\frac{1}{2}\xi_a^2 + \sqrt{2}\,\xi_a + 1} \dfrac{(M_u)_p}{R_c} & \text{for cohesive soils} \\ N_p \dfrac{\xi_a + \frac{7}{4}}{\frac{1}{2}\xi_a^2 + \frac{7}{4}\xi_a + \frac{13}{8}} \dfrac{(M_u)_p}{R_n} & \text{for cohesionless soils} \end{cases} \tag{2.20}$$

where $N_p$ is the number of piles in the foundation, $R_c$ is the characteristic length of a pile in cohesive soils, $R_n$ is the characteristic length of a pile in cohesionless soils and $\xi_a$ is above-ground height coefficient, which is defined as the scour-depth $L_a$ normalized by the characteristic length of the soil-pile system in this paper. Eqn. 2.20 shows that the lateral strength of the piles decreases with increasing the scour-depth, indicating a higher seismic damage potential for bridge foundations suffering serious scour problems.

## 3. ILLUSTRATIVE EXAMPLE: SINGLE COLUMN BRIDGE BENT IN MEDIUM SAND

The analytical approach is illustrated using a reinforced concrete single-column bridge bent located in a Class E cohesionless soil site with an effective friction angle of $\bar{\phi} = 33^{\circ}$. The mass of adjacent half spans of the superstructure is $m_s = 585000$ kg. The superstructure is supported by a 7.5 m tall ($N_c = 1$, $\alpha_c = 3$ and $L_c = L_c' = 7.5$ m) circular column with a diameter of $D_c = 2$ m. The bridge foundation consists of eight 0.9 m diameter circular piles ($N_p = 8$ and $D_p = 0.9$ m). The pile-cap is assumed to be a 7.5 m × 7.5 m × 1.5 m concrete block, with a mass of $m_f = 202500$ kg. The cross section of the bridge bent and the reinforcement details of the column and piles are shown in Fig. 3.1. The compressive strength of the concrete is taken as $f'_c = 35$ MPa. The longitudinal and transverse reinforcements are provided by A706 steel with a yield strength of $f_{ye} = 420$ MPa. The axial load applied to the column is 5740 kN, and each pile is subjected to an axial compression of 1035 kN. The moment-curvature responses of the column and piles are idealized to the elasto-plastic response. The effective flexural rigidity of the bridge column is $(EI_e)_c = 1.07 \times 10^7$ kN – m$^2$. The ultimate bending moment of the column, based on the elasto-plastic idealization, is $(M_u)_c = 33300\ \text{kN}-\text{m}$. The effective flexural rigidity and flexural strength of the pile are $(EI_e)_p = 3.38\times10^5\ \text{kN}-\text{m}^2$ and $(M_u)_p = 2370\ \text{kN}-\text{m}$, respectively. For cohesionless soils with an effective friction angle of $\bar{\phi} = 33^\text{o}$, the rate of increase of modulus of horizontal subgrade reaction is $n_h = 5500\ \text{kN/m}^3$, per the recommendation of ATC-32 (1996). The characteristic length of the soil-pile system is $R_n = \sqrt[5]{(EI_e)_p / n_h} = 2.28\ \text{m}$. The damping ratio of the reinforced concrete bridge structure is taken as $\zeta_s = 5\%$, and the effective damping ratio of the soil-foundation system is assumed to be $\zeta_f = 15\%$.

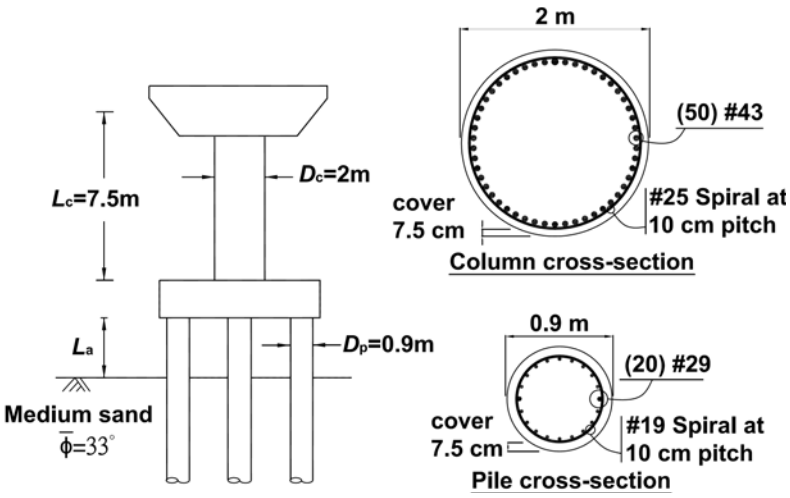


**Figure 3.1.** Cross section of a bridge bent and the reinforcement details of the column and piles

The soil-foundation-structure system of the bridge bent is idealized to the two-degree-of-freedom system shown in Fig. 1.2. The lateral stiffness of the bridge column, as calculated from Eqn. 2.2, is $K_s = 7.61\times10^4\ \text{kN/m}$. The lateral strength of the column is $(V_s)_{cap} = 4440\ \text{kN}$ .per Eqn. 2.19. Before the commencement of river scour, the riverbed is assumed to be at the pile/pile-cap interface, i.e. $L_a = 0$. For $N_p = 8$, $R_n = 2.28\ \text{m}$, $\xi_a = L_a/R_n = 0$ and $(EI_e)_p = 3.38\times10^5\ \text{kN}-\text{m}^2$, the lateral stiffness of the soil-foundation system is $K_f = 2.43\times10^5\ \text{kN/m}$, as given by Eqn. 2.3. The substitution of $N_p = 8$, $R_n = 2.28\ \text{m}$, $\xi_a = 0$ and $(M_u)_p = 2370\ \text{kN}-\text{m}$ into Eqn. 2.20 gives the lateral strength of the bridge foundation at its yield limit state, i.e. $(V_f)_{cap} = 8972\ \text{kN}$. Note that the bridge is originally designed following the capacity design principle where the force required to cause inelastic deformation in the foundation is twice over the lateral strength of the bridge column.

Due to scour of the riverbed, the bridge is suffering the foundation exposure. The pile above-ground height is observed to be $L_a = 3.5\ \text{m}$, approximately 3.9 times the pile diameter. For a bridge foundation with $N_p = 8$, $R_n = 2.28\ \text{m}$, $(EI_e)_p = 3.38\times10^5\ \text{kN}-\text{m}^2$, $(M_u)_p = 2370\ \text{kN}-\text{m}$ and the pile above-ground height coefficient of $\xi_a = L_a / R_n = 1.54$, the lateral stiffness of the soil-foundation system given by Eqn. 2.3 is $K_f = 6.35\times10^4\ \text{kN/m}$, and the lateral forced required to cause formation of plastic hinges at the pile-head is $(V_f)_{cap} = 4970\ \text{kN}$, per Eqn. 2.20. Although the strength of the foundation is still above that of the bridge column, i.e. $(V_f)_{cap} > (V_s)_{cap}$, a scour-depth of $L_a = 3.5\ \text{m}$ has cut down the lateral strength of the bridge foundation at its yield limit state for 45%. The natural vibration periods and mode shapes of the idealized two-degree-of-freedom system is then assessed. The ratio between the mass of the pile-cap and the mass of the superstructure is $\beta_m = m_f / m_s = 0.35$ and the stiffness ratio of the bridge bent is now $\beta_k = K_f / K_s = 0.83$. Upon the determination of the mass and stiffness ratios, the three coefficients $\lambda_a$, $\lambda_b$ and $\lambda_c$ needed for assessing the dynamic response of the bridge bent are calculated as: $\lambda_a = \beta_k + \beta_m + 1 = 2.18$, $\lambda_b = \beta_k - \beta_m + 1 = 1.49$ and $\lambda_c = \sqrt{\beta_k^2 - 2\beta_k(\beta_m - 1) + (\beta_m + 1)^2} = 1.90$. The natural period of the first and second vibration modes are $T_1 = 0.86\ \text{s}$ and $T_2 = 0.23\ \text{s}$, respectively, per Eqn. 2.8. From Eqn. 2.9, the mode shape of the first vibration mode is $\phi_1 = \{0.59 \quad 1\}^T$, and the mode shape of the second vibration mode is $\phi_2 = \{-4.89 \quad 1\}^T$, assuming a unit displacement at the top of the column. For $\beta_k = 0.83$, $\lambda_b = 1.49$, $\lambda_c = 1.90$, $\zeta_s = 5\%$ and $\zeta_f = 15\%$, the equivalent damping ratio of the first vibration mode is estimated to be $(\zeta_e)_1 = 11.3\%$, as given by Eqn. 2.11. The same set of values can be substituted into Eqn. 2.12 to find the equivalent damping ratio of the second vibration mode, i.e. $(\zeta_e)_2 = 8.7\%$.

An elastic design spectrum is needed to impose the seismic demand on the structure. While many different elastic design spectra are available, a design spectrum constructed following the methodology provided in FEMA-P750 (NEHRP 2010) is used to illustrate the analytical approach. The peak ground acceleration of the design level earthquake at the bridge site is assumed to be $0.35\ \text{g}$. In order to consider the influence of damping on the seismic response of structures, a set of damping coefficients is also given in FEMA-450 (NEHRP 2003) to modify the spectral response for different levels of damping ratios. The acceleration response spectra for damping ratios of 5% and 15% are plotted in Fig. 3.2. For the first vibration mode of the bridge bent, where the equivalent damping ratio is $(\zeta_e)_1 = 11.3\%$ and the natural period is $T_1 = 0.86\ \text{s}$, the spectral acceleration is estimated to be $S_{a1}(T_1) = 0.70\ \text{g} = 6.91\ \text{m/s}^2$. The spectral acceleration for the second vibration mode, where the equivalent damping ratio is $(\zeta_e)_2 = 8.7\%$ and the natural period is $T_2 = 0.23\ \text{s}$, is taken as $S_{a2}(T_2) = 0.71\text{g} = 6.96\ \text{m/s}^2$. The modal participation factor of the first vibration mode can be determined by substituting $\beta_m = 0.35$, $\lambda_b = 1.49$ and $\lambda_c = 1.90$ into Eqn. 2.13, i.e. $\Gamma_1 = 1.08$. By substituting of the same set of values into Eqn. 2.14, the modal participation factor of the second vibration mode is given as $\Gamma_2 = -0.075$. For $m_s = 585000\ \text{kg}$, $\Gamma_1 = 1.08$, $S_{a1}(T_1) = 6.91\ \text{m/s}^2$, $\Gamma_2 = -0.075$ and $S_{a2}(T_2) = 6.96\ \text{m/s}^2$, the maximum lateral force applied on the bridge column is $(V_s)_{dem} = 4360\ \text{kN}$, per Eqn. 2.17. The substitution of the same set of values plus $\beta_m = 0.35$, $\lambda_b = 1.49$ and $\lambda_c = 1.90$ into Eqn. 2.18 gives the seismic demand imposed to the bridge foundation $(V_f)_{dem} = 5380\ \text{kN}$. The seismic performance of the bridge bent is now assessed by comparing the imposed seismic demand with the lateral strengths of the bridge column and foundation, which are calculated earlier as $(V_s)_{cap} = 4440\ \text{kN}$ and $(V_f)_{cap} = 4970\ \text{kN}$, respectively. The maximum lateral force applied on the bridge column is less than the strength of the column, i.e. $(V_s)_{dem} < (V_s)_{cap}$, indicating an elastic response during the earthquake. However, the lateral strength of the foundation at its yield limit state, which is reduced considerably due to the riverbed scour, is not sufficient to endure the imposed seismic demand, i.e. $(V_f)_{cap} < (V_f)_{dem}$. Inelastic deformation, and possibly undesired damages, may occur in the piles during the design level earthquake.

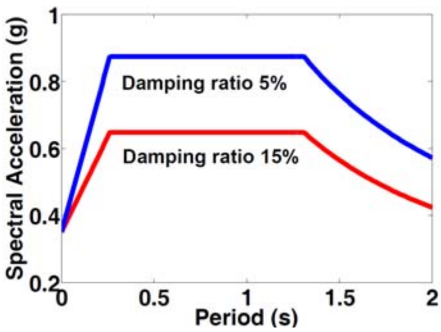


**Figure 3.2.** The design spectra used in the example

## 4. CONCLUSIONS

In current bridge engineering practice, the foundation system is normally designed with some level of over-strength so that piles in the foundation can remain elastic while the energy dissipation due to structural yielding takes place in the bridge column. However, pile exposure due to riverbed scour may significantly cut down the lateral strength of a bridge foundation, leading to potential for undesired damages in the piles during a design level earthquake. In this paper, an analytical approach suitable for assessing the seismic damage potential of bridges with foundation exposure is presented. In the proposed approach, the bridge bent is idealized to a two-degree-of-freedom system to account for the interaction between the foundation and structure during an earthquake excitation. The damage potential of the bridge is evaluated by comparing the imposed seismic demand with the strengths of the column and the foundation with pile exposure. The maximum seismic response of the bridge is assessed following the well-accepted response spectrum analysis method, requiring relatively few parameters: (1) mass ratio of the bridge bent, (2) stiffness ratio of the bridge bent, (3) mass of the superstructure and (4) damping ratios of the structure and foundation. The strength of the foundations at its yield limit state is assessed using an analytical model capable of assessing the performance of soil-pile systems at pertinent yield limit states. The proposed approach incorporates soil effects into the analysis process so that the influence of soil stiffness and damping on the vibration period, system damping and lateral strength of the bridge can be also accounted for. The versatility of the proposed approach is illustrated using a numerical example. Results show that the lateral strength of the foundation reduces significantly with increasing the depth of pile exposure. Although the foundation was originally designed with a satisfactory seismic performance, undesired pile damage can be expected even if the scour-depth is relatively small.

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