PERFORMANCE ASSESSMENT OF MULTI-COLUMN BENT EXTENDED PILE-SHAFTS UNDER LATERAL EARTHQUAKE LOADS

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ABSTRACT

Under seismic loads, bridges supported by multi-column bent extended pile-shafts may be subjected to large curvature demands at the pile/cap-beam connection, with potential of severe damage of the structure. An analytical model incorporating soil properties in the process of seismic performance assessment is proposed. The model is capable of estimating the severity of local damage in the pile-shaft for a wide range of pile and soil properties, and therefore is useful in performance-based engineering. Performance assessment is illustrated using details of the Struve Slough Bridge, which collapsed after the 1989 Loma Prieta Earthquake. Results indicated that the bridge failure was initiated by longitudinal reinforcement buckling due to insufficient transverse confinement in the pile-shaft.

Keywords: Soil-Pile Interaction, Extended Pile-Shafts, Ductility, Seismic Performance, Bridges

INTRODUCTION

A common type of foundation design for bridge structures involves the use of multi-column bent extended pile-shafts, as shown in Fig. 1, where the supporting pile-shaft is extended above ground as a column having approximately the same diameter. The top of the pile, terminated above the ground-level, is restrained from rotation by fully anchoring the reinforcement into the cap-beam to develop the strength of the reinforcement. The popularity of this foundation type arises from its cost-effectiveness when compared to an equivalent group of smaller diameter piles since the construction of a heavily reinforced pile-cap can be eliminated. In seismic design, it is important to recognize that the overall response of a bridge supported on extended pile-shafts is characterized by an increased flexibility due to the compliance of the soil compared to fixed-base columns of similar diameter. The design of these piles, as driven by their flexural strength, depends on the properties of the surrounding soil. For proper design of the extended pile-shaft, the influence of the surrounding soil on the overall performance of the structure is essential and must be taken into account accordingly.

For a multi-column bent extended pile-shaft subjected to a horizontal seismic motion, the lateral force associated with the inertia of the superstructure may induce a large bending moment at the pile/cap-beam connection. The magnitude of the bending moment under a design level earthquake can be sufficiently large to cause plastic hinging in the pile-shaft at different locations, as shown in Figs. 1(a) and (b), with potential for severe damage to the structure. Since yielding of the pile-shaft can be expected at the design level earthquake, post-yield performance of the soil-pile system, particularly the global displacement and local inelastic deformation, becomes critically important. As practices move towards performance-based engineering, an approach capable of incorporating soil properties in the process of seismic performance assessment will help to advance the state-of-the-art in bridge engineering.

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The procedure for seismic performance assessment of extended pile-shafts in multi-column bents is presented in this paper using the Struve Slough Bridge, which collapsed during the 1989 Loma Prieta Earthquake due to serious damage having occurred at the pile-head. The pseudo spectral acceleration and the lateral displacement ductility imposed on the bridge are estimated using the 2004 Caltrans Design Spectra (Caltrans 2004) for the appropriate soil category. The level of damage is assessed by comparing the local curvature ductility demand with the curvature ductility capacity of the pile section. Results highlighting the influence of the soil and structural parameters, and design implications will be presented.

ANALYTICAL MODEL FOR DUCTILITY ASSESSMENT OF EXTENDED PILE-SHAFTS

Seismic performance of extended pile-shafts depends on the imposed level of inelastic deformation in the critical regions of the pile-shaft. Inelastic deformation, commonly characterized in terms of curvature demand, depends on the displacement ductility imposed on the structure as well as the properties of the soil and pile-shaft. Since a wide range of soil conditions exists in practice, seismic performance of extended pile-shafts varies significantly from one design to another. Detailed assessment of the local curvature ductility demand and the damage potential of extended pile-shafts under various levels of displacement ductility factors are important for seismic design of bridge structures.

The curvature ductility demand imposed in the yielding regions of a multi-column bent extended pile-shaft depends on the lateral stiffness and strength of the pile-shaft and surrounding soil, as well as the location and length of the plastic hinges. Although many techniques may be used to estimate the curvature ductility demand, an acceptable approach, which characterizes the response of a laterally loaded extended pile-shaft for a number of significant limit states, lends itself to rather convenient closed-form solutions (Song et al. 2005). For an extended pile-shaft subjected to a large lateral load, sequential yielding occurs along the length of the pile-shaft until a plastic mechanism is fully developed. The idealized load-displacement relation and significant limit states of a laterally loaded extended pile-shaft are plotted in Fig. 2. The first yield limit state is characterized by the formation of the plastic hinge at the pile/cap-beam connection and is defined by the pile-head displacement of \( \Delta_{y1} \). Displacement beyond \( \Delta_{y1} \) induces a plastic rotation at the first plastic hinge and causes bending moment redistribution in the pile-shaft. The second limit state, which occurs when the second plastic hinge develops at a depth of \( L_{m} \), is signified by the lateral displacement of \( \Delta_{y2} \). Lateral displacement beyond \( \Delta_{y2} \) is facilitated by inelastic
deformations in both plastic hinges. The lateral force-displacement response of an extended pile-shaft can be further idealized by a bilinear elasto-plastic response, as shown in Fig. 2, using an equivalent elasto-plastic yield displacement $\Delta_y$. The identification of these limit states allows a set of kinematic equations relating the curvature ductility demand to the imposed displacement ductility factor to be developed.

For the case where the lateral displacement $\Delta_y$ imposed on the pile-shaft is greater than $\Delta_{y1}$ but less than $\Delta_{y2}$, i.e. $\Delta_{y1}<\Delta_y<\Delta_{y2}$, only one plastic hinge is developed at the pile/cap-beam connection. The curvature ductility factor at the first hinge $\mu_{\phi1}$ may be given by (Song et al. 2005):

$$\mu_{\phi1} = 1 + \frac{\beta(L_m + L_a)}{\eta \lambda_{p1}} (\mu_\Delta - \alpha)$$  \hspace{1cm} (1)

where $\mu_\Delta$ is the displacement ductility factor, defined as the ratio of the imposed displacement $\Delta_y$ to the elasto-plastic yield displacement $\Delta_y$, $L_m$ is the depth to the second plastic hinge $L_n$ normalized by the pile diameter $D$, $L_a$ is the above-ground height $L_a$ normalized by the pile diameter $D$, $\lambda_{p1}$ is the length of the first plastic hinge $L_p1$ normalized by the pile diameter $D$, and $\alpha$, $\beta$ and $\eta$ are coefficients used for the assessment of the curvature ductility demand. The coefficient $\alpha = \Delta_{y1}/\Delta_y$ is the ratio between the pile-head displacement at the first limit state $\Delta_{y1}$ and the equivalent elasto-plastic yield displacement $\Delta_y$. The coefficient $\beta = \Delta_y/\langle \phi_i (L_n + L_m)^2 \rangle$ relates the equivalent elasto-plastic displacement $\Delta_y$ to the elasto-plastic yield curvature $\phi_i$ of the pile-shaft. The coefficient $\eta$ is related to the ratio between the characteristic length of the soil-pile system and the distance between the two plastic hinges, and is given by (Song 2005):

$$\eta = \left\{ \begin{array}{ll}
\frac{1}{\delta_{y1}^2} \langle \frac{\xi_a^2 + \frac{1}{2} \xi_a^2 + 2 \xi_a + 2^n}{\xi_a^2 + \frac{1}{2} \xi_a^2 + 1} \rangle R_c & \text{for cohesive soils} \\
\frac{1}{\delta_{y1}^2} \langle \frac{\xi_a^2 + \frac{7}{4} \xi_a^2 + \frac{13}{4} \xi_a + \frac{17}{8}}{\xi_a^2 + \frac{7}{4} \xi_a^2 + \frac{13}{8}} \rangle R_n & \text{for cohesionless soils}
\end{array} \right.$$ \hspace{1cm} (2)

where $R_c$ and $R_n$ are characteristic length of a pile-shaft embedded in cohesive and cohesionless soils, respectively, and will be defined later in the paper. The coefficient $\xi_n$ is the above-ground height coefficient, which is defined as the above-ground height $L_a$ normalized by the characteristic length of the soil-pile system, i.e. $\xi_n = L_a/R_c$ for cohesive soils and $\xi_n = L_a/R_n$ for cohesionless soils.

For the case where the imposed displacement $\Delta_y$ is sufficiently large to cause inelastic deformation in both plastic hinges, i.e. $\Delta_y \geq \Delta_{y2}$, the curvature ductility factor in the first plastic hinge $\mu_{\phi1}$ and the second plastic hinge $\mu_{\phi2}$ may be obtained by (Song et al. 2005):

$$\mu_{\phi1} = \frac{K_1}{K_2} \cdot \frac{\beta(L_m + L_a)}{\lambda_{p1}} (1 - \alpha) + \frac{\beta(L_m + L_a)}{\lambda_{p1}} (\mu_\Delta - \alpha)$$ \hspace{1cm} (3)

$$\mu_{\phi2} = 1 - \frac{K_1}{K_2} \cdot \frac{\beta(L_m + L_a)}{\lambda_{p2}} (1 - \alpha) + \frac{\beta(L_m + L_a)}{\lambda_{p2}} (\mu_\Delta - \alpha)$$ \hspace{1cm} (4)

where $K_1$ and $K_2$ are the initial and reduced lateral stiffness of the soil-pile system, as shown in Fig. 2, $\lambda_{p2}$ is the length of the second plastic hinge $L_p2$ normalized by the pile diameter $D$, and $\mu_{\phi2}$ is the curvature ductility demand at the first plastic hinge as the pile-head displacement equals to $\Delta_{y2}$. The curvature ductility $\mu_{\phi2}$ may be determined by (Song et al. 2005):

$$\mu_{\phi2} = 1 + \frac{K_1}{K_2} \cdot \frac{\beta(L_m + L_a)}{\eta \lambda_{p2}} (1 - \alpha)$$ \hspace{1cm} (5)

This set of equations, namely Eqs. 1, 3, 4 and 5, allows the full range of the curvature ductility factor to be assessed for a given displacement ductility demand.
Lateral Stiffness of the Soil-Pile System

Kinematic equations, i.e. Eqs. 1, 3, 4 and 5, indicate that the curvature ductility demand is closely related to the lateral stiffness of the soil-pile system. A common approach for determining the lateral stiffness of the pile foundation assumes that the soil-pile system can be analyzed as a flexural member supported by an elastic Winkler foundation. For cohesive soils, the stiffness of the soil is assumed to be independent of the depth, resulting in a constant horizontal subgrade reaction $k_h$ (in units of force/length$^2$). For cohesionless soils, the lateral stiffness may be modeled assuming a constant rate of stiffness increase with depth, denoted by $n_h$ (in units of force/length$^3$). For a multi-column bent extended pile-shaft under a lateral load, the initial stiffness $K_1$ of the soil-pile system is given by (Song 2005):

$$K_1 = \frac{1 + \xi_a \zeta_b + \sqrt{2} \xi_b}{1 + \frac{\xi_a}{2} \zeta_a + \frac{7}{3} \xi_b + \frac{\xi_b}{2} + \frac{17}{8} \zeta_b} \frac{E I_s}{R_c^3} \text{ for cohesive soils}$$

$$K_1 = \frac{1 + \xi_a \zeta_b + \sqrt{2} \xi_b}{1 + \frac{\xi_a}{2} \zeta_a + \frac{7}{3} \xi_b + \frac{\xi_b}{2} + \frac{17}{8} \zeta_b} \frac{E I}{n_h R_n^3} \text{ for cohesionless soils}$$

where $EI_s$ is the effective flexural rigidity of the pile-shaft and $\xi_a$ is the above-ground height coefficient. The characteristic length of the soil-pile system is defined as $R_c = \frac{\sqrt{EI_s}}{k_h}$ for cohesive soils and $R_n = \frac{\sqrt{EI_s}}{n_h}$ for cohesionless soils, respectively. The coefficient $\zeta_b$ relates the rotational stiffness of the cap-beam, $K_\theta$, to the lateral stiffness of the soil-pile system and is defined as $\zeta_b = \frac{K_\theta}{\frac{E I_s}{R_c}}$ for cohesive soils and $\zeta_b = \frac{K_\theta}{\frac{E I_s}{R_n}}$ for cohesionless soils. The lateral force at the formation of the first plastic hinge is given by (Song 2005):

$$V_1 = \frac{1 + \xi_a \zeta_b + \sqrt{2} \xi_b}{1 + \frac{\xi_a}{2} \zeta_a + \frac{7}{3} \xi_b + \frac{\xi_b}{2} + \frac{17}{8} \zeta_b} \frac{M_u}{R_c} \text{ for cohesive soils}$$

$$V_1 = \frac{1 + \xi_a \zeta_b + \sqrt{2} \xi_b}{1 + \frac{\xi_a}{2} \zeta_a + \frac{7}{3} \xi_b + \frac{\xi_b}{2} + \frac{17}{8} \zeta_b} \frac{M_u}{n_h R_n} \text{ for cohesionless soils}$$

where $M_u$ is the flexural strength of the pile-shaft. Using the lateral stiffness $K_1$ of Eq. 6 and the lateral yield force $V_1$ of Eq. 7, the pile-head displacement $\Delta_1$ at the first yield limit state can be determined by $\Delta_1 = V_1/K_1$. After the formation of the first plastic hinge, the restraint at the pile-head effectively changes to a free-head condition, where the reduced lateral stiffness $K_2$ may be calculated by (Song 2005):

$$K_2 = \frac{1 + \xi_a \zeta_b + \sqrt{2} \xi_b}{1 + \frac{\xi_a}{2} \zeta_a + \frac{7}{3} \xi_b + \frac{\xi_b}{2} + \frac{17}{8} \zeta_b} \frac{E I_s}{R_c^3} \text{ for cohesive soils}$$

$$K_2 = \frac{1 + \xi_a \zeta_b + \sqrt{2} \xi_b}{1 + \frac{\xi_a}{2} \zeta_a + \frac{7}{3} \xi_b + \frac{\xi_b}{2} + \frac{17}{8} \zeta_b} \frac{E I}{R_n^3} \text{ for cohesionless soils}$$

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 $k_h = 67s_u$, 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 $\phi$ and relative density $D_r$ of the soil can be made following the suggestion of ATC-32 (ATC 1996).

Depth to the Second Plastic Hinge

For a fixed-head extended pile-shaft where potential for double hinging occurs, the location of the second hinge depends on the flexural strength of the pile and ultimate pressure of the soil. The depth to the second plastic hinge $L_m$, which is essentially a strength parameter of the soil-pile system, influences the curvature ductility demand of the pile. For pile-shafts in cohesive soils, the depth to the second plastic hinge normalized by pile diameter, i.e. $L_m = L_m/D$, may be given by the solution of the equation (Song 2005):
where $L_a = L_u / D$ is the normalized above-ground height and $M_u^* = M_u / (s_u D^2)$ is the normalized flexural strength of the pile, where $s_u$ is the undrained shear strength of the cohesive soil. The critical depth coefficient of the soil-pile system $\psi_r$ is defined as $\psi_r = 0.5 \gamma / (\gamma' D + 0.5 s_u)$, where $\gamma'$ is the effective unit weight of the soil. Note that Eq. 9 is derived using the ultimate soil pressure distribution proposed by Matlock (1970). Upon the determination of the normalized depth to the second plastic hinge $L_m^*$, the normalized lateral strength of the pile-shaft, defined as $V_u^* = V_u / (s_u D^2)$, may be determined by (Song 2005):

$$V_u^* = \begin{cases} \frac{3}{\psi_r} L_m^{* 3} + \frac{3}{4} L_m^{* 2} & \text{for } L_m^* \leq \psi_r \\ 9 L_m^{* 2} - 3 \psi_r & \text{for } L_m^* > \psi_r \end{cases}$$  

(10)

For pile-shafts in cohesionless soils, where a linearly increasing ultimate soil pressure distribution as suggested by Broms (1964) is assumed, relevant equations governing the normalized depth to the second plastic hinge $L_m^*$ and the normalized lateral strength $V_u^*$ are (Song 2005):

$$M_u^* = \frac{L_m^*}{\psi_r} + \left( \frac{3}{4} + \frac{3 L_a^*}{2 \psi_r} \right) L_m^{* 2} + \frac{3}{2} L_a^* L_m^*$$  

(9)

$$V_u^* = \frac{3}{\psi_r} L_m^{* 3} + \frac{3}{4} L_m^{* 2}$$  

(11)

$$V_u^* = \frac{3}{2} L_m^{* 2}$$  

(12)

where the normalized flexural strength $M_u^*$ is defined as $M_u^* = M_u / (K_p \psi' D^3)$ and the normalized lateral strength $V_u^*$ is defined as $V_u^* = V_u / (K_p \psi' D^3)$, where $K_p = (1 + \sin \phi) / (1 + \sin \phi)$ is the passive soil pressure coefficient and $\phi$ is the effective friction angle of cohesionless soils. Upon the determination of ultimate lateral strength $V_u$, the equivalent elasto-plastic yield displacement $\Delta$ may be obtained from the idealized elasto-plastic relation in Fig. 2, i.e. $\Delta = V_u / K_1$, while the lateral displacement at the second yield limit state $\Delta_2$ may be determined from the idealized tri-linear response, i.e.: $\Delta_2 = \Delta_1 + (V_u - V_1) / K_2$.

### Equivalent Plastic Hinge Length of Pile-Shafts

As evident in the kinematic equations (Eqs. 1, 3, 4 and 5), the ductility demand in the yielding region of an extended pile-shaft depends upon the equivalent plastic hinge length. For the case of pile-shafts with a fixed pile/cap-beam connection, the length of the first plastic hinge, denoted as $L_{p1}$, is assumed to be similar to the plastic hinge length of a fixed-based column since the first plastic hinge of the pile-shaft forms against a supporting member. Therefore, the equivalent plastic hinge length for the first hinge may be obtained from the equation given in Caltrans Seismic Design Criteria (Caltrans 2004):

$$L_{p1} = 0.04 (L_m + L_u) + 0.022 f_y e d_{bl} \geq 0.044 f_y e d_{bl}; \quad L_{p1} \leq D$$  

(13)

where $f_y$ is the expected yield strength of the reinforcing steel (in the unit of MPa) and $d_{bl}$ is the diameter of the longitudinal reinforcement of the pile-shaft (in the unit of mm). For the second plastic hinge, the equivalent plastic hinge length $L_{p2}$ may be taken from the plastic hinge length for single extended pile-shafts, which varies with the above-ground height (Chai 2002), i.e.:

$$L_{p2} = 0.1 L_u + D \leq 1.6 D$$  

(14)

The analytical model is now complete for lateral push-over analysis. The model enables the estimation of the lateral stiffness and strength of a multi-column bent extended pile-shaft, and also quantifies the magnitude of the local curvature ductility demand for a given displacement ductility factor.
Seismic performance assessment of multi-column bent extended pile-shafts is illustrated using the failure of the Struve Slough Bridge during the 1989 Loma Prieta Earthquake (Mw = 6.9). The Struve Slough Bridge was a pair of reinforced concrete bridges located on the State Highway-1 near Watsonville, CA, approximately 20 km from the epicenter of the earthquake (EERC 1990). The bridge was approximately 250 m long and supported by 22 multi-column bents with a span length of 11 m. Each bent consisted of four 0.38 m diameter ($D = 0.38$ m) circular concrete extended pile-shafts, which were 28 m long and spaced 2.54 m apart. For a span length of 11 m and pile-shaft spacing of 2.54 m, the weight of the superstructure tributary to each pile-shaft was approximately equal to 236 kN (6.7% of $f_c' A_g$). The above-ground height of the pile-shafts was $L_a = 3.66$ m (EERC 1990). The longitudinal reinforcement ratio of the pile section was $\rho_l = 1.5\%$, and the transverse steel ratio was $\rho_s = 0.28\%$. Fig. 3, which is reproduced from Caltrans As-Built Plan (1964), shows the reinforcement details of the pile-shaft and cap-beam. It should be noted that, although step-tapered piles were used below ground level in the bridge, the pile section shown in Fig. 3 is assumed to be continued throughout the entire pile-shaft length in this example. According to the information obtained by soil sampling, the soil in the upper 20 m of the site was soft grey clay with occasional pockets of fine sand (Caltrans 1964). As the soil condition varied somewhat along the length of the bridge, the soil property near the middle span of the bridge is selected for performance assessment. The soft clay had an undrained shear strength of $s_u = 6.5$ kPa, an effective unit weight of $\gamma' = 12$ kN/m$^3$ (Caltrans 1964) and was classified as soil profile SE per current NEHRP seismic provision (NEHRP 2001).

Due to the presence of the soft clay deposit at the site and the close proximity of the bridge to the earthquake epicenter, the Struve Slough Bridge was severely damaged during the Loma Prieta Earthquake. The bridge superstructure displaced excessively in the transverse direction and collapsed with several columns punched through the roadway. Pile-shafts suffered severe concrete crushing, buckling of longitudinal reinforcement and fracture of transverse reinforcement. Many pile-shafts were reported to have sheared off near its interface with the cap-beams, while several pile-shafts sustained damage at some distance below the ground level (EERC 1990). A view of the collapsed structure and the failure near the pile/cap-beam connection are shown in Figs. 4(a) and (b). Although some have speculated that vertical ground acceleration may have played an important role in the damage sustained by the Struve Slough Bridge, a study of the seismic response of the bridge by Saadeghvaziri (1990) using dynamic finite element analysis showed that the model participation factor for the vertical vibration is very small. It was concluded that the vertical excitation did not produce sufficient seismic demand to cause the structural failure. The collapse of the bridge was more likely a result of lateral deformation imposed on the structure, which caused failure of the pile-shafts resulting in the transverse cap-beams lost their seating (Saadeghvaziri 1990). In this paper, the performance of the extended pile-shafts under lateral seismic demand is assessed. Possible failure mechanisms examined include (1) bucking of the longitudinal reinforcement, (2) crushing of the concrete and (3) shear failure of the pile element.

**Properties of Pile-Shaft Section**

The cross-section for the pile-shaft of the Struve Slough Bridge is shown in Fig. 3. Following the specification in the Caltrans Seismic Design Criteria (Caltrans 2004), the nominal shear strength of the pile-shaft is $V_n = 107$ kN. The axial force $P$ applied at the pile-shaft is assumed to arise entirely from the tributary weight of the superstructure, i.e. $P = 236$ kN. Additional axial force due to vertical excitation or
frame-action of the structure is not considered in this study. The moment-curvature response of the pile section may be idealized by an elasto-plastic response. In this case, the effective flexural rigidity of the pile-shaft is $EI_e = 8584 \text{ kN-m}^2$, and the ultimate bending moment of the pile section, based on the elasto-plastic idealization, is $M_u = 117.5 \text{ kN-m}$. The equivalent elasto-plastic yield curvature is $\phi_y = 0.0137 \text{ rad/m}$, and the curvature associated with the crushing of the confined concrete core is $\phi_{cc} = 0.119 \text{ rad/m}$, giving a curvature ductility capacity of $(\mu \phi)_{cap} = 8.69$. As given in Fig. 3, the transverse steel ratio of the pile section was only 0.28%, and the confining steel was provided at a spacing of 152 mm. Since the spacing was less than the maximum spacing limit of $6d_{th}$, where $d_{th}$ is the longitudinal reinforcement diameter, as specified in the Caltrans Bridge Design Specifications (Caltrans 2003), the longitudinal reinforcement may not be adequately restrained against buckling. Buckling of the longitudinal reinforcing bars was a distinct possibility after the unconfined concrete cover has spalled off. According to the moment-curvature analysis, the curvature associated with the loss of the concrete cover is $\phi_{cc} = 0.0854 \text{ rad/m}$, and the curvature ductility capacity is $(\mu \phi)_{cap} = 6.23$ at longitudinal reinforcing bar buckling and $(\mu \phi)_{cap} = 8.69$ at concrete core crushing are selected as the ultimate limit states for performance assessment.

Stiffness and Strength of Extended Pile-Shafts

The effective unit weight of the soft clay near the middle span of the bridge is $\gamma = 12 \text{ kN/m}^3$, and the undrained shear strength is $\gamma_s = 6.5 \text{ kPa}$. The modulus of horizontal subgrade reaction of the soil is $k_h = 67s_u = 435.5 \text{ kN/m}^2$. The critical depth coefficient of the soil-pile system is $\psi_s = 6s_u/(\gamma D + 0.5s_u) = 4.88$. For $k_h = 435.5 \text{ kN/m}^2$ and $EI_e = 8584 \text{ kN-m}^2$, the characteristic length of the pile-shaft is $a = \sqrt{EI_e / k_h} = 2.11 \text{ m}$, which also gives an above-ground height coefficient of $\xi_{su} = L_u / R_c = 1.74$ for an above ground height of $L_u = 3.66 \text{ m}$. The effective flexural rigidity of the cap-beam section, estimated from a moment-curvature analysis, is $EI_b = 675000 \text{ kN-m}^2$. With the cap-beam flexural rigidity of $EI_b = 675000 \text{ kN-m}^2$ and a column spacing of $L_b = 2.54 \text{ m}$, the rotational stiffness of the cap-beam is equal to $K_{db} = 12EI_b / L_b = 3190000 \text{ kN-m/ rad}$, corresponding to a coefficient of $\xi_{su} = K_{ub}/(EI_b / R_c) = 782.8$. For $\xi_{su} = 782.8$, $EI_b = 8584 \text{ kN-m}^2$ and $R_c = 2.11 \text{ m}$, the initial lateral stiffness of the soil-pile system, as calculated from Eq. 6, is $K_1 = 297.7 \text{ kN/m}$, whereas the reduced lateral stiffness, due to the first plastic hinge formation, is $K_2 = 84.2 \text{ kN/m}$, as given by Eq. 8. The ratio of the two lateral stiffness coefficients is $K/K_2 = 3.5$. From Eq. 7, the lateral force at the formation of the plastic hinge at the pile/cap-beam connection is $V_y = 35.5 \text{kN}$. The lateral displacement at the first yield limit state is $\Delta_y = V_y / K_1 = 0.119 \text{ m}$. Using a normalized flexural strength of $M_{fs}^* = M_{fs}/(s_u D^2) = 329.4$, a normalized above ground height of $L_u^* = L_u / D = 9.63$ and a critical depth coefficient of $\psi_s = 4.88$, the normalized depth to the second plastic hinge is obtained by solving Eq. 9, yielding a $L_m^* = 6.97$, or an actual depth of $L_m = 2.65 \text{ m}$. The corresponding normalized lateral strength may be estimated by Eq. 10, which gives $V_y^* = 48.2$, or an actual lateral strength of $V_y = 45.2 \text{ kN}$. Upon the determination of the lateral strength $V_y$, the elasto-plastic yield displacement is $\Delta_y = V_y / K_1 = 0.152 \text{ m}$. The lateral displacement at the formation of the second plastic hinge is $\Delta_{y2} = \Delta_{y1} + (V_y - V_y^*) / K_2 = 0.235 \text{ m}$, which is associated with a displacement ductility factor of $\mu_{\Delta} = 1.55$.

Seismic Demand on the Bridge

The bridge was not instrumented and no ground motion was recorded at the site during the earthquake. To assess the seismic performance of the bridge, the Caltrans Elastic Response Spectrum (Caltrans 2004) for
soil profile $S_E$ with magnitude of 7.25 ± 0.25, as reproduced in Fig. 5(a), is used to impose the seismic demand on the structure. The horizontal peak ground acceleration (PGA) is estimated from an attenuation relation using the distance between the bridge and the fault rupture surface. For the Struve Slough Bridge, which is approximately 17 km from the fault rupture surface (Housner 1990), the PGA from the attenuation relation proposed by Boore et al. (1997) is estimated to be 0.38 g. The attenuation relation is plotted against recorded values of PGA at soft soil sites in Fig. 5(b). A recording (CSMIP Station 47459) in the Watsonville Telephone Building, which was built in a fill over deep alluvium and is located 1 km from the Struve Slough Bridge, showed a horizontal PGA of 0.39 g (CSMIP 1990). The recorded PGA is very close to the PGA estimated by the Boore et al. (1997) attenuation relation. Another recording (CSMIP Station 57180) with similar epicentral distance as the Struve Slough Bridge but located on a rock site showed a PGA of 0.45 g (CSMIP 1990). From these comparisons, a PGA of 0.38 g (PGA ≈ 0.4 g for rock) is felt to be a reasonable estimate of the intensity of the ground shaking at the Struve Slough Bridge, and this value of PGA will be used for performance assessment of the bridge.

The seismic mass tributary to each pile-shaft is $m = P/g = 24060$ kg. For such mass and an initial stiffness of $K_1 = 297.7$ kN/m, the elastic vibration period of the bridge in the transverse direction is $T = 2\pi \sqrt{m/K_1} = 1.8$ s. The spectral acceleration, estimated by scaling the response spectrum in Fig. 5(a) linearly, is $Sp_a = 0.52$ g, and the elastic seismic lateral force acting on the pile-shaft is $V_e = m \cdot Sp_a = 123$ kN. The ratio between the elastic seismic demand $V_e$ and lateral strength of the pile-shaft $V_u$ is $V_e/V_u = 2.72$. Following the $R-\mu \Delta - T$ relation given in the Caltrans Bridge Memo to Designers (Caltrans 1995), with a value of $R = V_e/V_u = 2.72$, $T = 1.8$ s and the period delineating the equal-energy to the equal-displacement region equal to 0.75 s, the displacement ductility demand imposed on the pile-shaft is $(\mu \Delta)_{dem} = R = 2.72$.

![Figure 5.](image)

**Damage Assessment of the Struve Slough Bridge**

Three ultimate limit states are deemed possible for the Struve Slough Bridge: (1) longitudinal reinforcement bucking at the plastic hinge, (2) concrete core crushing and (3) shear failure of the pile element. The idealized lateral load displacement relation and significant limit states of the pile-shaft are plotted in Fig. 6 (a). The nominal shear strength of the pile section, as obtained per Caltrans criteria (Caltrans 2004), is also plotted in Fig. 6 (a). It can be seen from the figure that, although little transverse reinforcement was provided for the pile-shaft, the nominal shear strength of $V_u = 107$ kN is significantly greater than the lateral earthquake forces causing the formation of the plastic hinges at the first and second yield limit states, i.e. $V_1$ and $V_{y2}$, respectively. Upon the formation the second plastic hinge, a fully plastic response with a constant lateral force level is assumed to be developed in the pile shaft, as shown in Fig. 6(a). The nominal shear strength of pile section $V_e$ however would not be reached by the seismic demand after the plastic mechanism is developed. Therefore, it is reasonable to conclude that the observed pile-shaft damage in the Struve Slough Bridge was due to local inelastic deformation at the plastic hinge region instead of shear failure of the pile.
Since the seismic demand gave a displacement ductility factor of \((\mu_\Delta)_{dem} = 2.72\), considerable structural yielding of the pile-shafts was likely developed. The kinematic relation between the curvature ductility factor and the displacement ductility demand in the pile-shaft depends on the ratio between \(\Delta_y\) and \(\Delta_y\), which is represented by the coefficient of \(\alpha = \Delta_y / \Delta_y = 0.78\). Using the elasto-plastic yield displacement of \(\Delta_y = 0.152\ m\), the yield curvature of \(\phi_y = 0.152\ rad/m\), a depth to the second plastic hinge of \(L_m = 2.65\ m\) and an above-ground height of \(L_a = 3.66\ m\), the coefficient \(\beta = \Delta_y / [\phi_y (L_a + L_m)] = 0.28\). Applying \(\xi_a = 1.74\), \(R_c = 2.11\ m\), \(L_a = 3.66\ m\) and \(L_m = 2.65\ m\) into Eq. 2, the coefficient \(\eta = 0.73\). For \(L_a = 3.66\ m\) and \(L_m = 2.65\ m\), the equivalent plastic hinge length of the first plastic hinge is taken as \(L_{p1} = 0.38\ m\) from Eq. 13, which corresponds to a normalized length of \(\lambda_{p1} = 1\). The plastic hinge length for the second hinge is found to be \(L_{p2} = 0.61\ m\) (Eq. 14), corresponding to a normalized length of \(\lambda_{p2} = 1.6\). The curvature ductility demand \(\mu_\phi\) in the first plastic hinge at the lateral displacement \(\Delta y\) is \(5.84\), as calculated from Eq. 5. The substitution of \(\alpha = 0.78, \beta = 0.28, \eta = 0.73, \lambda_{p1} = 1, L_a = 9.63\) and \(L_m = 6.97\) into Eq.1 gives the kinematic relation for small lateral displacements where only one plastic hinge forms. The same set of values plus \(K_1/K_2 = 3.5, \lambda_{p2} = 1.6\) and \(\mu_\phi = 5.84\) may be substituted into Eqs. 3 and 4 for the case of large lateral displacements where both plastic hinges form.

The relation between the imposed displacement ductility factor and local curvature ductility demand for the first and second plastic hinges are plotted in Fig. 6(b). For a curvature ductility capacity of \((\mu_\phi)_{cap} = 6.23\), as estimated for the longitudinal reinforcement buckling, the result in Fig. 6(b) indicates that the pile-shaft can tolerate a displacement ductility factor of \(\mu_\Delta = 1.64\), or a lateral displacement of \(\Delta = 0.25\ m\). The figure also shows that the curvature ductility capacity \((\mu_\phi)_{cap} = 8.69\) for crushing of the concrete core corresponds to a displacement ductility factor of \(\mu_\Delta = 2.17\), or a lateral displacement of \(\Delta = 0.33\ m\). Note that the curvature ductility demand in the second plastic hinge is only \((\mu_\phi)_{dem} = 1.25\) when the buckling of longitudinal reinforcements occurs in the first hinge, and \((\mu_\phi)_{dem} = 2.79\) when the crushing of the concrete in the first hinge occurred. Accordingly, only minor damage was expected to occur in the pile below the ground level. For the seismic demand of \((\mu_\Delta)_{dem} = 2.72\) imposed on the bridge, the curvature ductility demand in the first plastic hinge is \((\mu_\phi)_{dem} = 11.2\), which is significantly greater than the curvature ductility capacities of the pile-shaft. Consequently, flexural failures at the extended pile-shaft of the bridge were expected. Thus, according to the procedure and assumptions made in this paper, the collapse of the Struve Slough Bridge structure was due to lack of transverse reinforcement in the pile-shaft. The low confining steel ratio along with the large transverse steel spacing did not provide adequate restraint against longitudinal reinforcement buckling and confinement against early concrete crushing as a result of significant seismic demand on the bridge. It was probable that the cap-beam lost its vertical support after the flexural failure of the pile-shaft, precipitating the dislocation of the superstructure from the substructure and hence the collapse of the bridge.
CONCLUSIONS

Satisfactory seismic performance of pile-supported bridge structures depends on the level of inelastic deformation experienced by the pile-shaft during the earthquake. Inelastic deformation, commonly characterized by a curvature demand, is correlated to the lateral displacement ductility imposed on the bridge, as well as the properties of the pile-shaft and its surrounding soils. In this paper, an analytical model incorporating soil properties is proposed for seismic performance assessment for extended pile-shafts embedded in cohesive and cohesionless soils. The model is capable of relating the local curvature ductility demand to the displacement ductility factor. The kinematic equations proposed in this paper indicate that the curvature ductility demand depends on the strength and stiffness of the soil-pile system, as well as the location and length of the plastic hinges. The versatility of the proposed model is illustrated using the case history of the Struve Slough Bridge collapse during the 1989 Loma Prieta Earthquake. Three ultimate limit states, namely (1) longitudinal reinforcement bucking, (2) concrete core crushing, and (3) shear failure of the pile, form the basis of the damage assessment in this paper. Results indicated that the observed damage was consistent with the flexural failure expected of the inadequate transverse reinforcement in the pile-shaft. The poor detailing of the transverse reinforcement was the likely cause of buckling of the longitudinal reinforcement, which was followed by crushing of the concrete and eventually led to the dislocation of the superstructure from the substructure and final collapse of the bridge.

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