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A simplified procedure for estimating nonlinear seismic demand of tall piers

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ABSTRACT

More than 40% of the bridges in mountainous areas of Southwest China are constructed with piers having a height of over 40 m. The seismic response of such tall pier structures is usually estimated by nonlinear response history analysis and/or adaptive pushover procedures; while providing satisfactory results, these methods can be quite time consuming and computational demanding. Therefore, this paper proposes a simplified procedure for estimating the nonlinear seismic response of tall piers. The influence of plastic deformation at the base on dynamic properties (natural periods and mode shapes) of a linearized system for the tall piers is first investigated, using a numerical model of the prototype bridge, based on an equivalent linearization technique. Using these results, a simplified procedure is proposed to predict the distribution of seismic shear force and bending moment along the pier height, as well as the curvature ductility ratio at the pier base. The efficiency of the proposed procedure is verified with numerical examples of tall piers subjected to recorded ground motions, through comparing the seismic demands determined by this method with rigorous nonlinear response history analysis. The results show that the proposed method can efficiently estimate the distribution of both shear force and bending moment along the height of the tall pier; the curvature ductility ratio at pier base can be predicted with errors around 10%.

1. Introduction

In recent years, numerous highway bridges have been built or are under construction in the mountainous areas of southwest China. Because of the rugged terrain in the region, more than 40% of these bridges have piers with a height of over 40 m, with some exceeding 100 m. Southwest China is known to be a region of high seismicity, experiencing many strong earthquakes (e.g., the magnitude 8.0 Wenchuan earthquake in 2008 and the magnitude 7.0 Lushan earthquake in 2013). As the bridges with tall piers are key links of the local highway network, their seismic performance and safety is of great concern.

For bridges with tall piers, the mass of the piers is often larger than the corresponding superstructure, which can lead to significant seismic inertial forces [1]. These piers are also flexible, which leads to inherently long fundamental periods that are out of the dominant frequency content of many seismic events; however, higher modes may still have significant contributions to the seismic response [2].

Some research [3–5] was conducted to investigate the higher-mode effects of tall piers with numerical models. The results showed that seismic performance of tall piers was significantly affected by the higher-order modes. Moreover, current displacement-based and capacity design methods could lead to large errors in the case of tall bridge piers, although they are suitable for bridges with short-to-medium height piers. In these studies, the nonlinear response history analysis (NRHA) procedure was used to investigate the seismic performance of tall piers. While recognized as the most rigorous analysis method when nonlinear performance is involved, the NRHA procedure can be prohibitively time consuming and computationally demanding.

In engineering practice, some methods other than NRHA are used to estimate the seismic performance of structures. For example, the multi-mode response spectrum analysis (RSA) procedure is a commonly used method in design practice. However, when nonlinear performance is involved, the seismic responses of structures can be significantly underestimated by this multi-mode RSA method, even though behavior factors are introduced [6,7]. On the other hand, capacity spectrum methods have been used to estimate seismic demands of structural engineering practice [8–10]. In these procedures, the structural capacity curves are computed by nonlinear static analysis of the structure subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a predetermined target displacement is reached; while the seismic demand curves can be represented by earthquake response spectra. By plotting the capacity and demand curves together, structures can be designed accordingly [11]. However,

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these methods are based on the assumption that the structural seismic response is dominated by the fundamental vibration mode; and thus, the effects of higher modes on seismic demands cannot be considered.

Chopra and Goel [12] proposed a modal pushover analysis (MPA) procedure for building structures, which was subsequently extended to bridges [13]. The authors demonstrated that the coupling among vibration modes arising from yielding of the system is negligible. Therefore, the seismic response caused by different vibration modes is calculated separately and combined using the SRSS method in the MPA procedure to estimate the seismic demand of inelastic structures. However, for each vibration mode, the lateral load pattern is assumed to be unchanged throughout the analysis process, which ignores the influence of nonlinearity on the structural dynamics such as vibration periods and modal vectors. Furthermore, the force-displacement relationship is simplified as bi-linear or tri-linear in the MPA procedure, which might lead to errors due to the idealization process.

To better predict the nonlinear seismic responses of structures, numerous adaptive pushover analysis procedures [14–17] have been proposed and applied to both buildings and bridges, using adaptive capacity spectrum and adaptive force/displacement distributions. As the effects of higher modes and stiffness degradation can be considered in these adaptive pushover methods, they are able to provide satisfactory estimates of seismic demands. However, these procedures are generally conceptually complicated and computationally demanding for routine application in structural engineering practice. As demonstrated by previous investigations, the computational effort required for some adaptive pushover procedures can even be comparable to nonlinear time history analysis [12].

The primary objective of this study is to propose a simplified procedure for estimating the nonlinear seismic response of tall bridge piers, which are widely used in southwest China. The influence of plastic deformation at pier base on dynamic properties (natural periods and mode shapes) of a linearized system for the tall piers is first investigated, using a numerical model of the prototype bridge based on an equivalent linearization technique. Using these results, a simplified procedure was proposed to predict the distribution of seismic shear force and bending moment along the pier height, as well as the curvature ductility ratio at the pier base. The efficiency of the proposed procedure was verified with numerical examples of tall piers, comparing the results determined by the proposed method with the NRHA procedure.

2. Bridge prototype and analysis model

2.1. Bridge description

The prototype bridge shown in Fig. 1(a) is typical of bridges with tall piers in southwest China and will be considered herein. The prestressed and precast box girders are composed of C50 concrete and integrated into a four-span continuous girder by the cast-in-place joints. The cross section of the box girder is uniform, and the width and height are 7.45 m and 3.30 m, respectively; one span (30 m) of the girder weighs approximately 700 t. The joints are socket connected to the pier top and thus provide hinge-like connections between the girder and the piers. Unidirectional slider bearings that allow for thermal expansion in the longitudinal direction are used at the transitional piers. Each bridge pier is composed of variable hollow sections with external dimensions ranging from 3.35 m × 5.0 m (pier base) to 2.1 m × 5.0 m (pier top) and wall thickness of 0.6 m, as presented in Fig. 1(b). Each pier is supported by a pile cap sitting on 4 cast-in-place piles, which are 1.8 m in diameter and 30 m in length. More information about the geometrical configurations and material properties of this bridge prototype is shown in Tables 1 and 2.

2.2. Analysis model

To explore the seismic performance of the piers, the prototype bridge was simplified and represented as a single mass-column system as shown in Fig. 2. The plastic deformation was assumed to concentrate near the pier base, while other parts of the pier were expected to respond elastically when subjected to earthquake excitations.

In this single column model, the tributary mass (M = 723 tons) from the two-adjacent half spans of the superstructure (see Fig. 1) was concentrated at the pier top, which is located at a height of H (50.8 m) above the pier base. Linear-elastic beam elements were used to simulate the pier columns, and the flexure stiffness ($EI(h)$) of the variable cross sections was determined by the material properties and cross section dimensions. The distributed pier masses were lumped at the nodes of column elements; the mass at the i-th node was denoted as (55–94 tons) and located at height h_i above the pier base. An equivalent linear rotational spring with stiffness of k_{r} was adopted at the pier base, accounting for the plastic deformation [18], i.e. the rotation of the plastic hinge region is equivalently concentrated in this linear rotational spring. Soil-foundation-interaction effects were neglected.

For the concrete cross-section in the potential plastic region near the pier base, the effective (EI_{eff}) and equivalent (EI_{eq}) flexure stiffness can be determined through moment-curvature (M-ϕ) analysis, as shown in Fig. 3; the actual M-ϕ relation can be approximated as a bilinear curve by various methods [12,19], and thus the effective yield state of the cross section can be determined. The relationships between the parameters shown in Fig. 3 are given by:

$$EI_{\mathrm{eff}} = M_{\mathrm{y}}/\phi_{\mathrm{y}} EI_{\mathrm{eq}} = M_{\mathrm{y}}/\phi\mu_{\phi} = \phi/\phi_{\mathrm{y}} = EI_{\mathrm{eff}}/EI_{\mathrm{eq}} \tag{1}$$

Additionally, to compute the rotation of the equivalent plastic hinge length (L_{p}) at pier base, the elastic curvature is assumed to be trapezoidal and plastic curvature assumed to be uniform distributed in this region. Furthermore, both the elastic and plastic rotation of the plastic hinge is assumed to concentrated at the linear rotational spring. The moment-rotation relationship of the plastic hinge can be obtained as shown in Fig. 4, where the parameters are computed by:

$$\theta_{\mathrm{b}} = \begin{cases} \left(1-\frac{L_{\mathrm{p}}}{2H}\right)\phi_{\mathrm{b}} L_{\mathrm{p}} & \phi_{\mathrm{b}} < \phi_y \\ \phi_{\mathrm{b}} L_{\mathrm{p}} - \frac{L_{\mathrm{p}}}{2H}\phi_{\mathrm{y}} L_{\mathrm{p}} & \phi_{\mathrm{b}} > \phi_y \end{cases}$$
$$M_{\mathrm{b}} = \begin{cases} EI_{\mathrm{eff}} \times \phi_{\mathrm{b}} = k_{\mathrm{r,eff}} \times \theta_{\mathrm{b}} & \phi_{\mathrm{b}} < \phi_y \\ EI_{\mathrm{eq}} \times \phi_{\mathrm{b}} = k_{\mathrm{r,eq}} \times \theta_{\mathrm{b}} & \phi_{\mathrm{b}} > \phi_y \end{cases} \tag{2}$$

In Eq. (2), ϕ_{b} is the curvature at pier base; θ_{b} is the rotation of the plastic hinge; M_{b} is the bending moment at pier base. Note that no investigations about the plastic hinge length of tall piers have been reported to date, the value of L_{p} is assumed as $0.1H$, based on the results of high-rise buildings as given by Ref. [20]. And thus, θ_{b} can be approximated as $\phi_{\mathrm{b}} L_{\mathrm{p}}$ for both elastic and plastic response.

Consequently, the equivalent stiffness of the linear rotational spring corresponding to $\phi_{\mathrm{b}} = \phi_{\mathrm{y}}$ and $\phi_{\mathrm{b}} = \mu_{\phi}\phi_{\mathrm{y}}$, referred to as effective stiffness ($k_{\mathrm{r,eff}}$) and equivalent stiffness ($k_{\mathrm{r,eq}}$), respectively, can be expressed as:

$$k_{\mathrm{r,eff}} \approx EI_{\mathrm{eff}}/L_{\mathrm{p}}$$
$$k_{\mathrm{r,eq}} \approx EI_{\mathrm{eq}}/L_{\mathrm{p}} = EI_{\mathrm{eff}}/(\mu_{\phi} L_{\mathrm{p}}) \tag{3}$$

Eq. (3) shows that the development of plastic deformation at the pier base, measured by the curvature ductility ratio μ_{ϕ}, can be directly modelled by a linear rotational spring with an equivalent stiffness of $k_{\mathrm{r,eq}}$; the tall pier model system with the equivalent linear rotational spring at the base will be referred to as the 'linearized system' in this paper. Note that as μ_{ϕ} increases to infinity, the equivalent stiffness $k_{\mathrm{r,eq}}$ tends to 0; herein, this extreme state is referred to as 'pinned'.

This analysis model will be used in the next sections to assess the dynamic properties of the linearized system for the tall bridge piers.