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Effects of strong ground motions of near source earthquakes on response of thin-walled L-shaped steel bridge piers

Guanmo Xie†

Department of Engineering Mechanics, Wuhan University of Technology, Wuhan 430070, China

Takeo Taniguchi‡ and Nawawi Chouw†

Department of Environmental and Civil Engineering, Okayama University, Okayama 700-8530, Japan

Abstract. Near source earthquakes can be characterized not only by strong horizontal but also by strong vertical ground motions with broad range of dominant frequencies. The inelastic horizontal response of thin-walled L-shaped steel bridge piers, which are popularly used as highway bridge supports, subjected to simultaneous horizontal and vertical ground excitations of near source earthquakes is investigated. A comprehensive damage index and an evolutionary-degrading hysteretic model are applied. Numerical analysis reveals that the strong vertical excitation of a near source earthquake exerts considerable influences on the damage development and horizontal response of thin-walled L-shaped steel bridge piers.

Key words: near source earthquakes; strong vertical ground motion; thin-walled steel bridge piers; inelastic response; damage index; evolutionary-degrading hysteretic model.

1. Introduction

In many of current seismic codes vertical earthquake load is neglected or treated just by scaling the amplitude of the horizontal load without a proper consideration of the frequency content. The reason for this is the belief that vertical ground motion is smaller than the horizontal one, and structures are already over-designed by a large factor of safety to resist gravity loads, hence will be able to resist additional forces from the vertical ground excitation. Records of recent earthquakes like Northridge (1994) and Kobe (1995) have indicated that peak vertical ground acceleration PVGA can be much larger than peak horizontal ground acceleration PHGA, especially near the source (Chouw *et al.* 1999). During these earthquakes not only old but also new buildings suffered severe damages. Field investigations suggested that some of these damages could be attributed to strong vertical ground excitations (Papazoglou *et al.* 1996).

Thin-walled steel bridge piers are usually in T-shaped as well as inverted L-shaped forms, fixed at the base and free at the top. Cross sections of these steel bridge piers are mostly box section with longitudinal stiffeners or circular pipe section. Compared with conventional portal frame type bridge

† Associate Professor
† Professor

supports, such cantilever type piers occupy less ground area. Distinctly different from steel members commonly used in building structures, highway steel bridge piers are characterized by large width-thickness ratio of component plates, which makes them susceptible to local and overall interaction buckling under seismic excitations. Based upon two damage indices (Krawinkler *et al.* 1983, Park *et al.* 1985) and the results of a variety of tests on thin-walled steel bridge piers, a comprehensive damage index and an evolutionary-degrading hysteretic model to evaluate the damage and response of steel structures under cyclic loading were presented by Kumar and Usami (1996).

By means of the damage index and evolutionary-degrading hysteretic model presented by Kumar and Usami, this paper addresses the inelastic horizontal seismic response of thin-walled steel bridge piers to simultaneous horizontal and vertical ground excitations of near source earthquakes.

2. Characteristics of near source earthquakes

Since water and even poorly compacted soil are able to transmit compressive waves, as opposed to shear waves, nonlinear soil behavior like liquefaction does not dampen vertical soil shaking but influence horizontal soil motion. The vertical accelerations which are associated with propagating compressive waves can be amplified towards the soil surface where horizontal accelerations decrease. Since surface layers generally have higher natural frequencies vertically than horizontally the soil prefers to transmit compressive waves with high frequencies and shear waves with low frequencies. The vertical ground accelerations therefore have higher frequency contents than the horizontal ground motions. The large accelerations and higher frequency content of the vertical ground motion of Northridge earthquake can be seen in Fig. 1. While the horizontal ground motion has a dominant frequency of 2.4 Hz, the vertical one has a dominant frequency range around 10 Hz.

3. SDOF modeling of thin-walled L-shaped steel bridge piers

Fig. 2 shows the SDOF modeling of an eccentrically loaded steel bridge pier. The mass M is lumped as $P/g+m/3$, where P is the weight of the upper structure and m is the mass of the pier

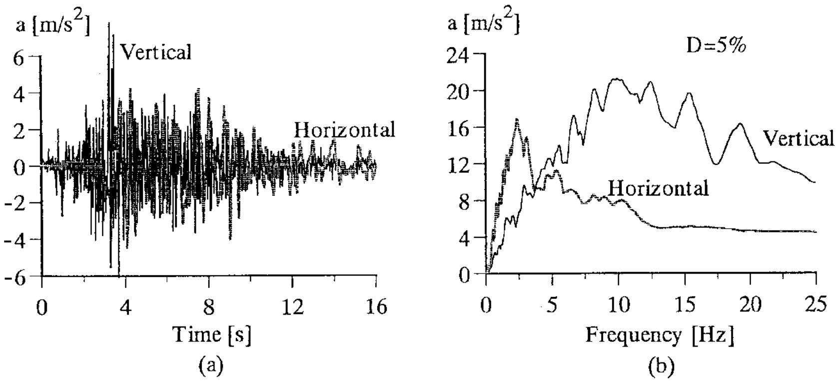

Fig. 1 Vertical and horizontal components of the Northridge earthquake at Station NRG and their response spectra

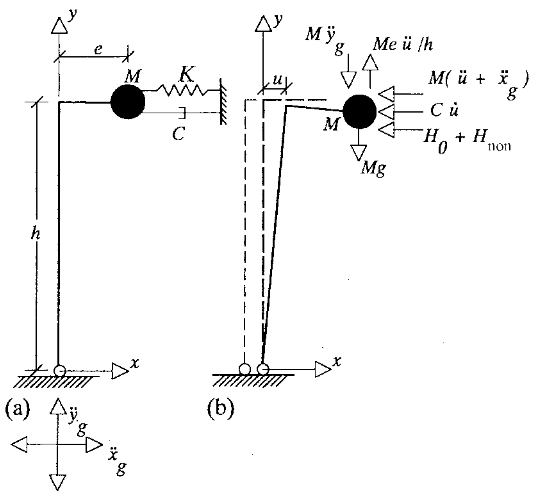

Fig. 2 SDOF modeling of a L-shaped steel bridge pier

column. The nonlinear spring is responsible for the horizontal restoring force. Fig. 2(b) shows the dynamic equilibrium of the mass under simultaneous horizontal and vertical excitations of a near source earthquake.

Equilibrium of bending moments at the hinge gives:

$$[M(\ddot{u}+\ddot{x}_g)+C\dot{u}+H_0+H_{non}]\left(h-e\frac{u}{h}\right)+\left(Me\frac{\ddot{u}}{h}-M\ddot{y}_g-Mg\right)(e+u)=0 \tag{1}$$

where H_{non} is the restoring force of the nonlinear spring.

Dividing both sides of Eq. (1) by h and omitting higher order terms, one can get:

$$M\left(1+\frac{e^2}{h^2}\right)\ddot{u}+C\dot{u}+\left(H_{\text{non}}-\frac{Mg}{h}u-\frac{M\ddot{y}_g}{h}u\right)=-M\ddot{x}_g+\frac{e}{h}M\ddot{y}_g \tag{2}$$

in which $H_{\text{non}}-Mgu/h-M\ddot{y}_g u/h$ represents the restoring force taking P-Δ effect into account. Noting $H_{\text{non}}-Mgu/h-M\ddot{y}_g u/h$ as $H(u)$, Eq. (2) can be written as

$$M\left(1+\frac{e^2}{h^2}\right)\ddot{u}+C\dot{u}+H(u)=-M\ddot{x}_g+\frac{e}{h}M\ddot{y}_g \tag{3}$$

If only the horizontal ground excitation is considered, the equation of motion becomes:

$$M\left(1+\frac{e^2}{h^2}\right)\ddot{u}+C\dot{u}+H(u)=-M\ddot{x}_g \tag{4}$$

Since the vertical displacement of the mass is not regarded as a degree of freedom, the vertical inertia force $M\ddot{y}_g$ caused by the vertical excitation can also point upward. In this case, the equation of motion would be:

$$M\left(1+\frac{e^2}{h^2}\right)\ddot{u}+C\dot{u}+H(u)=-M\ddot{x}_g-\frac{e}{h}M\ddot{y}_g \qquad (5)$$

In numerical analysis we should consider the worst result which is obtained from Eq. (5) or Eq. (3).

4. Numerical results

Figs. 3, 4 and 5 show the horizontal response of three L-shaped steel bridge piers with different eccentricity ratios to the Northridge earthquake. Table 1 gives the necessary parameters of these three steel bridge piers and the hysteretic model.

Since the horizontal excitation has relatively lower frequency contents and the horizontal

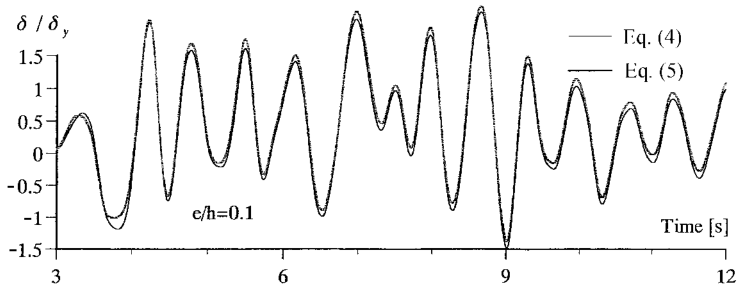

Fig. 3 The horizontal response of pier No. 1 due to the Northridge earthquake

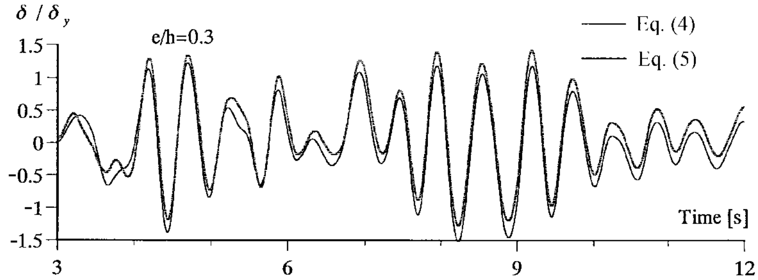

Fig. 4 The horizontal response of pier No. 2 due to the Northridge earthquake

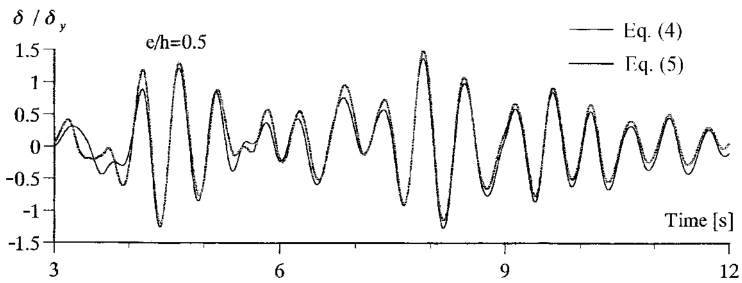

Fig. 5 The horizontal response of pier No. 3 due to the Northridge earthquake

Table 1 Parameters for the bridge piers and the hysteretic model

Bridge Pier	$\frac{e}{h}$	M (10^6 kg)	H_y (10^6 N)	δ_y (10^{-2} m)	α	β	c	$\frac{H_{\max}}{H_y}$	$\frac{\delta_u}{\delta_y}$	$\frac{\delta_m}{\delta_y}$
1	0.1	1.61	6.44	4.36	0.3	0.11	1.8	1.568	34.0	14.05
2	0.3	1.09	6.76	4.58	0.3	0.11	1.8	1.54	36.795	14.05
3	0.5	0.818	6.92	4.68	0.3	0.11	1.8	1.526	38.39	14.05

frequencies of the piers are small (the horizontal frequency of the pier No. 1 is 1.52 Hz, for the pier No. 2 it is 1.85 Hz, for the pier No. 3 it is 2.14 Hz), and in Eq. (3) or Eq. (5) the equivalent inertia force caused by the vertical excitation is the eccentricity ratio e/h times the real vertical inertia force $M\ddot{y}_g$, the horizontal response of the pier is controlled by the horizontal excitation. However, due to the strong vertical component of the ground motion, see Fig. 1(a), the vertical excitation has considerable effects on the piers horizontal response, even for the case of relatively small eccentricity ratio $e/h = 0.1$. As can be seen in Fig. 3, the greatest increase of the horizontal displacement due to the vertical excitation is 7.3 mm ($0.17\delta_y$) which occurs at approximately 5.5 s. In Fig. 4 the largest increase is 11.45 mm ($0.25\delta_y$) occurring at nearly 9.3 s. In Fig. 5 the largest increase is 15.6 mm ($0.33\delta_y$) at approximately 4.3 s. The influences of the vertical ground motion become greater when the eccentricity ratio increases.

Since the mass from the upper structure decreases with the increase in the eccentricity ratio when it is assumed that the weight of the upper structure takes the value of the maximum allowable load from elastic seismic design, the horizontal response of the pier decreases with increasing eccentricity ratio. The mass M of the pier No. 2 is close to that of the pier No. 3, there are small differences between amplitudes of the horizontal response of these two piers, as can be seen in Fig. 4 and Fig. 5.

Because the horizontal excitation defines the horizontal response, it can be observed that the horizontal response peaks due to simultaneous excitations approximately coincide with those due to the horizontal excitation only, except at the initial elastic range.

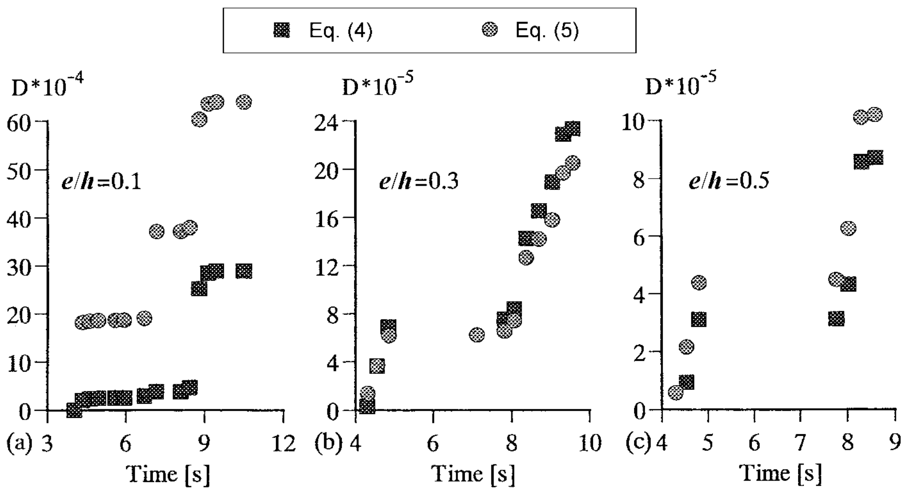

Fig. 6 The damage index of three L-shaped steel bridge piers

Fig. 6 shows values of the damage index *D* of these three L-shaped steel bridge piers during the Northridge earthquake. The horizontal coordinate is time at the end of every half-cycle in which yield occurs, the vertical coordinate is the value of the corresponding damage index *D*. Since the mass *M* decreases when the eccentricity ratio becomes larger, the values of the damage index *D* decrease with the increase in the eccentricity ratio. The values of the damage index of the pier No. 1 are much larger than those of the pier No. 2 or No. 3. The number of half-cycles in which plastic yield occurs also decreases from the pier No. 1 to No. 3 because the pier No. 1 has larger natural horizontal frequency than the pier No. 3. The differences between the values of the damage index *D* due to horizontal excitation and those due to simultaneous excitations also become smaller when the eccentricity ratio increases. This can be explained by the fact that the values of the damage index *D* and the number of half-cycles with yielding are large for the pier No. 1, therefore the differences accumulate gradually. The great differences in Fig. 6(a) reveal that the strong vertical ground motion of a near source earthquake causes the damage of the L-shaped steel bridge pier severely. For the pier with small eccentricity ratio the yield appears early and last longer, as can be observed in the horizontal axis from Figs. 6(a) to (c).

5. Conclusions

In this study the influences of the strong vertical ground motions of near source earthquakes on the damage development and horizontal response of thin-walled L-shaped steel bridge piers are evaluated. Although the horizontal response of the pier is governed by the horizontal excitation, the vertical excitation affects the damage index and horizontal response strongly, even for the pier with relatively small eccentricity ratio. The influences of the vertical ground motion become greater when the eccentricity ratio increases. The horizontal response peaks due to simultaneous excitation approximately coincide with those due to the horizontal excitation only, except at the initial elastic range. The horizontal response of the pier, the values of the damage index, the number of half-cycles with yield occurring and the differences between the values of the damage index due to horizontal excitation and that due to simultaneous excitations decrease with an increase in the eccentricity ratio due to the decrease of the considered mass. For the pier with small eccentricity ratio the yield occurs early and last longer.

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