Effect of Pounding and Restrainers on Seismic Response of Multiple-Frame Bridges

Reginald DesRoches, M.ASCE,[1] and Susendar Muthukumar[2]

Abstract: Pounding between adjacent frames in a multiple-frame bridge produces undesirable forces resulting in large displacements, local damage, and possible failure of column bents. In this study, analytical models are used to examine the factors affecting the global response of a multiple-frame bridge due to pounding of adjacent frames. Parameter studies of one-sided and two-sided pounding are conducted to determine the effects of frame stiffness ratio, ground motion characteristics, frame yielding, and restrainers on the pounding response of bridge frames. It is determined that the most important parameters are the frame period ratio and the characteristic period of the ground motion. The amplification in the frame response due to one-sided pounding is most severe for cases with highly out-of-phase frames, in particular for short period structures. Two-sided pounding amplifies the stiff frame response, and reduces the flexible frame response. The addition of restrainers has a minor effect on the one-sided pounding response of highly out-of-phase frames. Current recommendations by Caltrans for limitations in frame period ratios to reduce the effects of pounding are evaluated through an example case.

DOI: 10.1061/(ASCE)0733-9445(2002)128:7(860)

CE Database keywords: Earthquakes; Pounding; Bridges; Seismic response.

Introduction

Moderate to strong earthquakes can lead to out-of-phase motion of frames in a multiple-frame bridge due to differences in frame dynamic characteristics, variability in ground motion, and traveling wave effects resulting in impact at the expansion hinges. This impact can result in structural damage including spalling of concrete at the hinges and create undesirable forces in the adjacent frames. These forces are typically much larger than those for which the frames are designed, resulting in increased displacements and possible failure of column bents, abutments, and bearings.

Seismic pounding of bridge frames has led to several instances of bridge damage. Impact between bridge decks and abutments were the source of extensive damage to highway bridges with seat type abutments during the 1971 San Fernando earthquake (Jennings 1971). More recently, the 1989 Loma Prieta earthquake also showed several instances of pounding in multiple-frame bridges (NZNSEE 1990). During the earthquake, pounding of the lower roadway and columns supporting the upper deck of the Southern viaduct section at the China Basin in California occurred due to the height differences between the neighboring bridge members.

Significant pounding damage was observed at the expansion hinges and abutments of several bridges at the Interstate 5 and State road 14 interchange which were located at close proximity to the epicenter during the 1994 Northridge earthquake (EERI 1995a). Reconnaissance reports from the 1995 Hyogo-Ken Nanbu earthquake identify pounding as a major cause of fracture of bearing supports (EERI 1995b). This subsequently led to the collapse of several bridge decks. The Hanshin expressway deck had considerable longitudinal movement (up to 0.3 m), thus resulting in significant pounding damage at the intermediate hinges. Surveys conducted after the 1999 Chi-Chi Taiwan earthquake revealed that 30 bridges suffered some damage including pounding at the expansion joints (EERI 2001).

Parametric studies on pounding have shown that the frame stiffness ratios, earthquake loading, hinge gap, frame yield strength, and restrainer stiffness are important factors in determining the effects of pounding in multiple-frame bridges (DesRoches and Fenves 1997). Maragakis et al. (1991) studied the effects of energy losses during impact between bridge decks and abutments. Equivalent damper elements were used at the abutment locations to simulate energy dissipation during impact. The parameters considered included the coefficient of restitution, abutment and deck stiffness, gap, and deck to abutment mass ratio. Impact effects were found to be significant for bridge systems with flexible abutments.

The forces acting on the piers and deck deformations increase as a result of pounding (Jankowski et al. 1998; Ma and Pantelides 1998). However, studies by Malhotra (1998) show that a two-span reinforced concrete bridge with a stiffness ratio of 1.14 has a reduction in response due to pounding. Other researchers have suggested that pounding generally reduces the response of bridge frames because of the energy dissipated during pounding and because pounding disrupts the buildup of resonance (Priestly et al. 1995).

Several studies have evaluated mitigation strategies for pounding in bridges. Kim et al. (2000) found that restrainers reduce the

---

[1] Assistant Professor, Georgia Institute of Technology, School of Civil and Environmental Engineering, Atlanta, GA 30332-0355. E-mail: reginald.desroches@ce.gatech.edu

[2] Graduate Research Assistant, Georgia Institute of Technology, School of Civil and Environmental Engineering, Atlanta, GA 30332-0355.

Note. Associate Editor: Andrei M. Reinhorn. Discussion open until December 1, 2002. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on February 27, 2001; approved on October 16, 2001. This paper is part of the ***Journal of Structural Engineering***, Vol. 128, No. 7, July 1, 2002. ©ASCE, ISSN 0733-9445/2002/7-860–869/$8.00+$.50 per page.

relative displacements between pounding frames and prevent the collapse of spans. Several researchers have shown that shock absorbers, connectors with high damping or stiffness, and energy dissipation devices are effective in mitigating the pounding effects in bridges (Kawashime and Yabe 1996; Jankowski et al. 1999; Kawashima and Shoji 2000).

As mentioned, the effect of pounding on bridge response has led to conflicting results. To better understand the parameters affecting pounding, and to investigate the effects of restrainers and yielding frames on the demands in bridges, a comprehensive study is performed to evaluate the combined effects of restrainers and dynamic impact on the nonlinear response of bridges. In addition, current design recommendations for limiting pounding are evaluated. The 1999 Caltrans Seismic Design Criteria (SDC, 1999) recommends that in all new construction, frames in a multiple-frame bridge should have period ratios $T_i/T_j > 0.70$, where i is the shorter period frame and j is the longer period frame. New bridges that do not satisfy this criterion should be modified by changing effective column lengths, modifying end fixities, or relocating columns. While these recommendations are based on preliminary studies and anecdotal evidence, they have yet to be analytically validated.

A simplified two-degrees of freedom (DOF) bridge model is used to determine the effects of frame dynamic characteristics, ground motion, frame yielding, and restrainers on the frame response during one-sided pounding. Two-sided pounding is then studied through a four DOF model. Finally, the recommendations by Caltrans to reduce the effects of pounding are evaluated.

This study is limited to examining the effects of pounding between bridge decks due to longitudinal ground motion input. The effects of bridge skew, multisupport excitation, and soil structure interaction are not considered.

Simplified Model to Investigate One-Sided Impact

The opening and closing of intermediate hinges, yielding of bridge frames, and engaging of cable restrainers constitute nonlinearities inherent with the interaction of adjacent bridge frames during strong ground motion. Two adjacent frames of a typical multiple-frame bridge, shown in Fig. 1(a), are selected to investigate the effects of one-sided pounding. Impact between bridge decks is assumed to occur from only one side of a deck. A simplified two-DOF nonlinear model is used to evaluate the bridge response, as shown in Fig. 1(b). Each bridge frame is idealized as a single DOF yielding element with mass m_i and a viscous damping coefficient c_i. The inelastic restoring force for each frame (R_i) is obtained from the force deformation relationship for the frame. The Q-Hyst stiffness degrading hysteresis model is used for the frame force deformation relationship (Saiidi and Sozen 1979). The restrainers are represented as bilinear spring elements. The equations of motion for the two-DOF system are

$$\mathbf{M}\ddot{\mathbf{u}}(\mathbf{t})+\mathbf{C}\dot{\mathbf{u}}(\mathbf{t})+\mathbf{R}[\mathbf{u}(\mathbf{t})]+\mathbf{F}[\mathbf{d}(\mathbf{t})]=-\mathbf{M}\mathbf{1}\ddot{\mathbf{u}}_{\mathbf{g}}(\mathbf{t}) \tag{1}$$

where $\mathbf{M}$ and $\mathbf{C}$ = the mass and damping matrices; $\mathbf{R}$ = the vector of restoring forces for the frames; and $\mathbf{F}$ = the restoring force due to restrainers. The vector $\mathbf{1}$ = the earthquake influence coefficient vector; $\ddot{\mathbf{u}}_{\mathbf{g}}(\mathbf{t})$ = the input motion; $\ddot{\mathbf{u}}$, $\dot{\mathbf{u}}$, and $\mathbf{u}$ = the frame acceleration, velocity, and displacement vectors, respectively; and $\mathbf{d}$ = the relative displacement between adjacent frames.

The solution of Eq. (1) is obtained numerically using Newmark's average acceleration method (Newmark 1959). When the relative displacement of the adjacent frames becomes less than the hinge gap (g_p), impact occurs. Pounding is accounted for using a stereomechanical approach, which is a macroscopic attempt to model impact. Impact is assumed to be instantaneous and momentum conservation is used along with the coefficient of restitution to model impact. Other approaches to model pounding include the linear spring element, nonlinear spring (Hertz element), and linear spring in conjunction with damper (Kelvin element) (Maison and Kasai 1992; Jankowski et al. 1998; Ma and Pantelides 1998). In the stereomechanical approach, the velocities of the colliding masses are modified at the instant of impact as

$$v_1' = v_1 - (1+e)\frac{m_2(v_1 - v_2)}{m_1 + m_2} \tag{2}$$

$$v_2' = v_2 + (1+e)\frac{m_1(v_1 - v_2)}{m_1 + m_2} \tag{3}$$

where v_1', v_2' = the velocities of frames 1 and 2 after impact; v_1, v_2 = the frame velocities before impact; and e = the coefficient of restitution. Previous studies show that for realistic values of e, the relative hinge displacement is not sensitive to values of e (Athanassiadou et al. 1994; DesRoches and Fenves 1997). A value of $e = 0.8$ is used in this study. A variable time stepping procedure is used to determine the time of impact and to solve the equations of motion.

Sample Pounding Analysis

Consider the response of two frames in a typical multiple-frame bridge, as shown in Fig. 1(b), subjected to the 1940 El Centro earthquake, scaled to 0.70 g to coincide with typical design response spectra. To simplify the analysis and to better understand the factors affecting pounding, only single-sided pounding will be considered (i.e., effects of abutments or adjacent frames are ignored). Two cases are evaluated: a case with a frame period ratio of $T_1/T_2 = 0.32$, and a case with a frame period ratio of $T_1/T_2 = 0.71$. Case 1 represents the response of highly out-of-phase frames, and case 2 represents the response of slightly out-of-phase frames. In both cases, the modes are assigned 5% critical damping, and the frames have a gap of 12.5 mm (1/2 in.). The frames are designed to have an individual displacement ductility demand of $\mu = 4.0$, for the scaled 1940 El Centro record.

Fig. 2 shows the time history of frame displacements for the no-pounding and pounding studies for case 1 ($T_1/T_2 = 0.32$). The comparison shows that pounding significantly increases the maximum displacement of the stiff frame from 15 mm for the no-pounding case, to over 40 mm for the case when the frames are pounding. Conversely, for the flexible frame, pounding reduces the displacement from 130 mm in the no-pounding case to 90 mm when the frames pound. The flexible frame, which has a larger displacement, pounds the stiff frame increasing its response. Similarly, the stiff frame acts as a barrier to the flexible frame, thereby limiting the flexible frame response.

Fig. 3 shows the same analysis, except the stiffness of frame 1 has been modified such that the frames now have a period ratio of $T_1/T_2 = 0.71$. A comparison of the pounding and no-pounding response shows that the effect of pounding is considerably reduced in this case. For the stiff frame, pounding increases the response from 72 to 100 mm. For the flexible frame, the pounding and no-pounding maximum absolute displacements are nearly identical (approximately 125 mm).

This case study illustrates the significant effect that frame period ratio has on the response of pounding bridge frames. How-

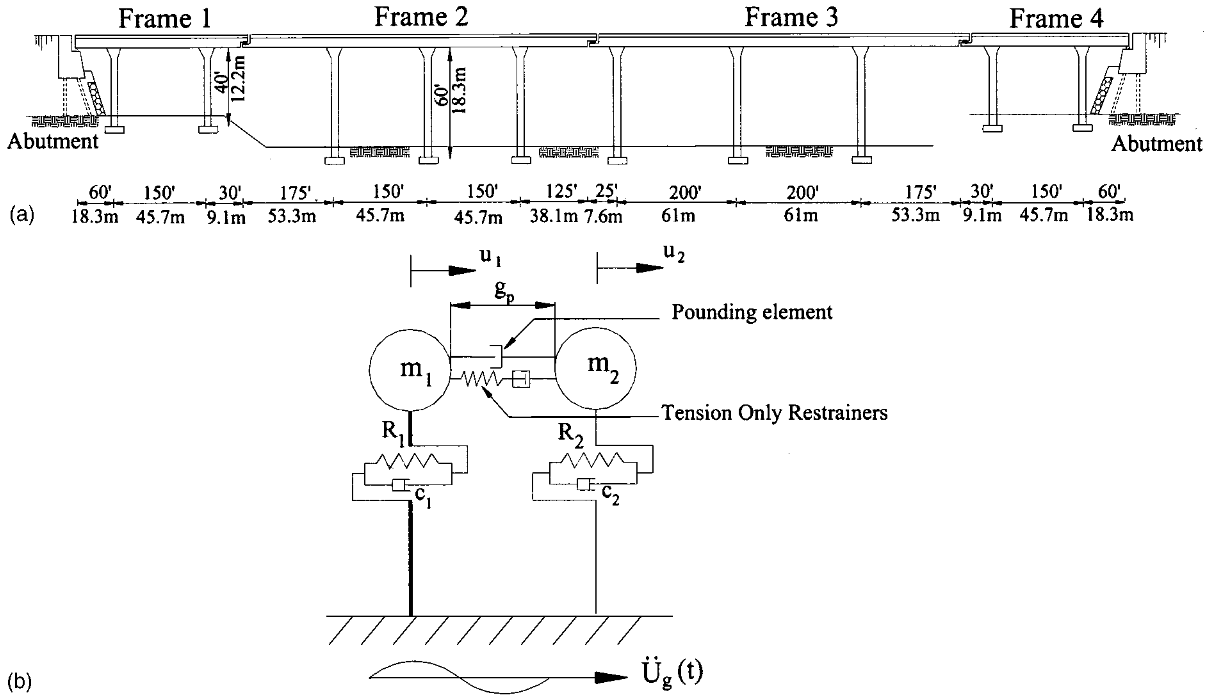

Fig. 1. (a) Typical multiple frame bridge; (b) model idealization

ever, it is not clear what effect other parameters, such as ground motion characteristics, have on the response of pounding frames. In addition, bridges that have been retrofitted with restrainers may have a different pounding response than those without restrainers. In the next section, a parametric study is conducted to investigate these parameters.

Parameter Studies

To mitigate pounding effects in multiple-frame bridges, it is important to determine the factors affecting the pounding response of bridge frames. The analysis shown in the previous section illustrated that the frame period ratio (T_1/T_2) is a major factor affecting the seismic response of pounding bridge frames. Past research has shown that additional primary factors affecting pounding response include the system configuration, earthquake loading, hinge gap (g_p), column yield strength (F_y), restrainer stiffness (K_r), and slack (s) (DesRoches and Fenves 1997). Secondary factors that affect the pounding response include the coefficient of restitution (e), relative masses of the frames (m_1/m_2), impact spring stiffness (K_{imp}), and spring damping. It has been shown that the relative mass does not play a significant role as long as the differences in the structure periods result from the differences in stiffnesses (Athaniassiadou et al. 1994; DesRoches and Fenves 1997; Trochalakis 1997).

Preliminary investigations are conducted with the two-DOF simplified model shown in Fig. 1(b) subjected to longitudinal ground motion to determine parameters affecting the pounding response. The mass ratio of the frames is taken as unity and the coefficient of restitution (e) is taken as 0.8 throughout the study. The principal parameters that are evaluated are the frame period ratio (T_1/T_2) or stiffness ratio (K_1/K_2), ground motion period ratio (T_2/T_g), hinge gap (g_p), and normalized restrainer stiffness (κ). The characteristic period of the ground motion (T_g) is defined as the period at which the input energy of a 5% damped linear elastic system is maximum (Miranda and Bertero 1994). The normalized restrainer stiffness (κ) is given as $\kappa = K_r/K_{\mathrm{mod}}$, where $1/K_{\mathrm{mod}} = 1/K_{\mathrm{eff1}} + 1/K_{\mathrm{eff2}}$ is the sum of the flexibilities of the two adjacent bridge frames, based on effective stiffness properties. The hinge gap g_p is given as

$$g_p = \alpha * D_{np} \tag{4}$$

where α=the gap ratio parameter and D_{np}=the relative displacement of the hinge when pounding of the frames does not occur. $\alpha = 1.0$ corresponds to the critical gap; i.e., the gap which is just sufficient to preclude pounding.

The frames are subjected to the suite of strong ground motion records given in Table 1. All records are scaled to 0.7 g peak ground acceleration, to coincide with typical design response spectra. The input records cover a wide range of characteristic periods (T_g), peak ground accelerations (PGA), and are of magnitude six or greater. Both elastic and inelastic frame responses are studied. Previous earthquakes have shown the relative hinge displacement in bridges subjected to strong ground motion ranges from 100 to 300 mm (DesRoches and Fenves 1997). Since typical hinge gaps are approximately 6.25–12.5 mm, this results in a range of gap ratios from 0.02 to 0.13. Preliminary studies show small differences in bridge response for gap ratios in this range. Therefore, to simplify the analysis, the gap parameter, α, is set at 0.02 in further investigations. The effect of pounding is expressed in terms of the displacement amplification ratio (γ), which is the ratio of the maximum pounding displacement to the maximum frame displacement if pounding does not occur.

Elastic Response

Fig. 4 shows a plot of the mean displacement amplification as a function of T_1/T_g and T_2/T_g for four values of T_1/T_2. Thin dashed lines indicate the variability in response, in terms of mean ± 1 standard deviation. Pounding reduces the frame response when vibrating at a period near the characteristic period of the

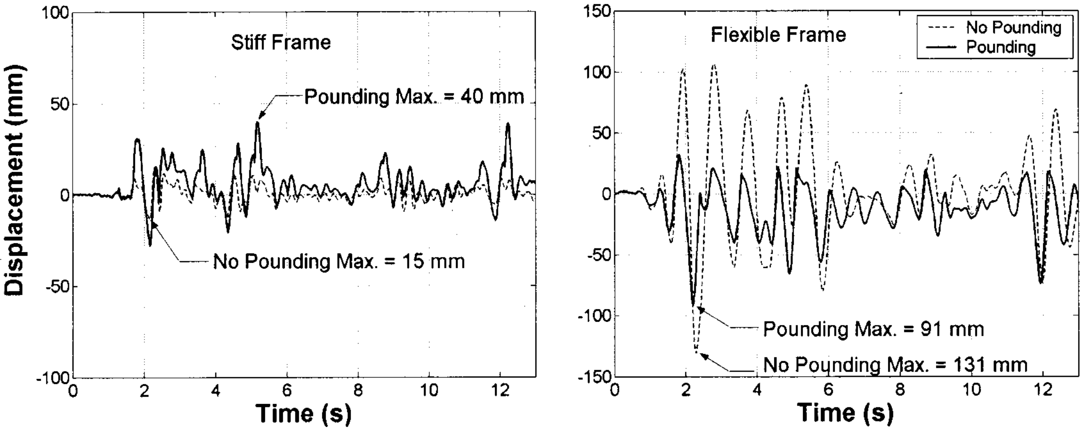

Fig. 2. Time history of frame displacements for the pounding and no-pounding studies. Inelastic frames ($T_1/T_2 = 0.32$); $T_{2\text{eff}}/T_g = 1.0$; El Centro scaled to 0.7 g; stereomechanical approach.

ground motion record (T_g). For example, when $T_2/T_g = 1$, the response of the flexible frame is reduced, while the response of the stiff frame is amplified. The significant reduction is observed since pounding prevents the build up of resonance energy in the frame subjected to input at its resonant frequency. Similarly, at $T_1/T_g = 1$, the response of the stiff frame (Frame 1) is reduced while that of the flexible frame (Frame 2) is amplified.

The displacement amplification plot has three distinct zones. In Zone I, where $T_2/T_g < 1$, the stiff frame amplification is greater than one and the flexible frame amplification is less than one. The mean response of the stiff frame is increased by as much as 180% and the mean response of the flexible frame is reduced by 30% in Zone I, for $T_1/T_2 = 0.32$. In Zone III, when $T_1/T_g > 1$, the flexible frame response increases and the stiff frame response is reduced. In Zone II, $T_1/T_g < 1 < T_2/T_g$ the frame displacement amplification is slightly greater than one for both the frames. The coefficient of variation (COV) defined as the ratio of the standard deviation to the mean can be as high as 57% for the stiff frame and 52% for the flexible frame when $T_1/T_2 = 0.32$. In general, the displacement amplification decreases as the frame period ratio approaches unity, for all values of T_2/T_g, as shown in Fig. 4. For the case with $T_1/T_2 = 0.71$ ($K_1/K_2 = 2.0$), only slight displacement amplifications of the stiff frame are observed for the entire range of T_2/T_g values. The maximum increase in the mean response of the stiff frame is 43% which is much less than that observed for $T_1/T_2 = 0.32$.

Inelastic Behavior of Frames and Effect of Restrainers

The response of any structure subjected to strong ground shaking often extends into the inelastic range and can be significantly different from the corresponding linear response. The inelastic behavior of the frame is characterized by a force–deformation relationship, which is an idealization of the actual behavior of the frame during cyclic load. The yield force of the frame (F_y) is established by dividing the elastic force demand (F_e) by a yield reduction factor R_y, in order to obtain a specified target ductility (μ), using a constant ductility spectrum.

Cable restrainers are often used at intermediate hinges as a retrofit measure to limit relative hinge displacement and prevent unseating during an earthquake. However, the presence of restrainers alters the behavior of adjacent frames by transferring forces as the frame opening exceeds the slack in the cable. While it has been shown that pounding can increase the linear and nonlinear response of the frames, it is not clear how the restrainers affect pounding in nonlinear frames. Hence, the effect of restrainers on pounding is also evaluated for yielding frames. The restrainers are modeled as tension-only springs with a slack. The slack is assumed to equal the hinge gap g_p.

To adequately represent the frame period ratio for all the yielding frames, the frame period is written using the effective stiffness, K_{eff}, where $K_{\text{eff}} = K/\mu$. Therefore, the effective frame period ratio, $T_{2\text{eff}}$ can be written as

$$T_{2\text{eff}} = 2\pi\sqrt{\frac{m}{K_{\text{eff}}}} = 2\pi\sqrt{\frac{m}{K/\mu}} = T_2\sqrt{\mu} \tag{5}$$

where T_2 = the period of the flexible frame in the elastic range. In order to enable comparison with the linear behavior of the frames, both the frames are designed for the same target ductility of $\mu = 4$. Thus, the frame period ratio remains as T_1/T_2 and is varied as done in the linear study. The ground motion effective period ratio $T_{2\text{eff}}/T_g$ is varied from 0.25 to 5.0 s in increments of 0.05 s. The reduction factors necessary to maintain a constant frame ductility depend on the frame period. An iterative procedure is used to determine the reduction factors required to give $\mu = 4$ for the individual frame response. The normalized restrainer stiffness (κ) is still given as K_r/K_{mod}. For yielding frames, K_{mod} can be expressed as

$$K_{\text{mod}} = \frac{K_1 K_2}{\mu(K_1 + K_2)} \tag{6}$$

where K_1 and K_2 = the elastic stiffness of the frames and μ = the design ductility demand. Values of $\kappa = 0$, 0.5 and 1.0 are considered for this study, where $\kappa = 0$ corresponds to the case with no hinge restrainers. The frames are subjected to five ground motion records as indicated in Table 1. Fig. 5 presents the pounding and no-pounding responses for $\kappa = 0$. Displacement amplification and ductility demand are presented.

As observed in the linear case, pounding is more critical for highly out-of-phase frames. Pounding reduces the frame response when the effective frame period (T_{eff}) is close to the characteristic period of the earthquake (T_g). The displacement amplification curves due to pounding can once again be classified into three zones depending on the effective ground motion period ratio.

Zone I covers the region where $T_{2\text{eff}}/T_g \leq 1.0$. The stiff frame ductility demand is increased by as much as 300% and the flexible frame ductility demand is reduced by approximately 40% in

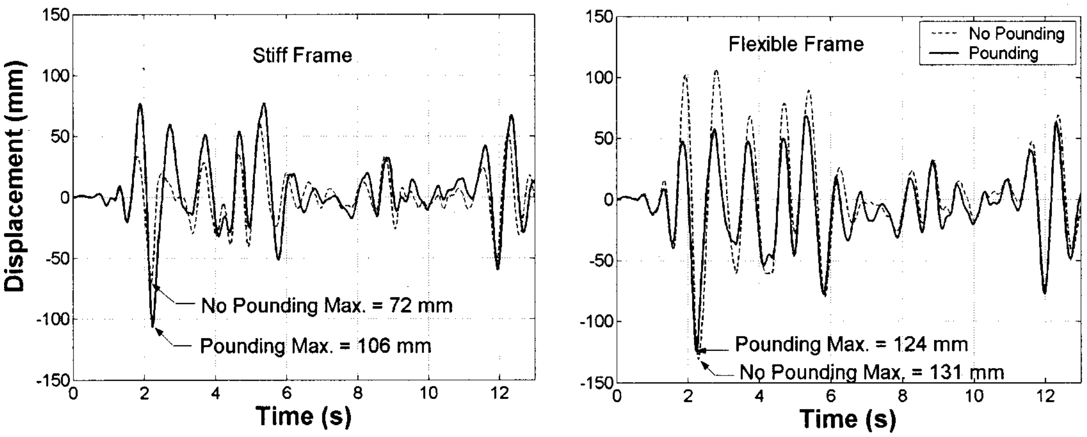

Fig. 3. Time history of frame displacements for the pounding and no-pounding studies. Inelastic frames ($T_1/T_2=0.71$); $T_{2\mathrm{eff}}/T_g=1.0$; El Centro scaled to 0.7 g; stereomechanical approach.

Zone I, for $T_1/T_2=0.32$ ($K_1/K_2=10$). The maximum COVs for the stiff and flexible frames are 57 and 58% respectively. The elastic frames had a corresponding Zone I displacement increase of 180% for the stiff frame and a reduction of 30% for the flexible frame.

Zone II exhibits an increase in both frame ductility demands, as observed for the displacement amplification in the linear study. Zone III shows similar behavior to the elastic cases. However, the increase in the flexible frame demand is slightly less in the inelastic behavior of the frames than exhibited in the corresponding linear study. This effect is attributed to yielding and hysteretic damping. The yielding of the frames results in a smaller relative velocity before impact than if they were elastic, and thus the pounding response is reduced. The hysteretic behavior of the frames in the nonlinear range results in significant energy dissipation that could also affect the pounding response. The effects of pounding are less pronounced when $K_1/K_2=2.0$ ($T_1/T_2=0.71$), similar to the elastic case.

The effect of restrainers on the frame pounding response is illustrated in Fig. 6. The addition of restrainers helps in reducing the frame response in Zone I but increases the stiff frame demand in Zone II. Overall, the effect of restrainers on the frame response is observed only for highly out-of-phase frames and is marginal for other stiffness ratios. For $T_1/T_2=0.32$, the addition of restrainers reduces the stiff frame response by approximately 25% in Zone I for $\kappa=1.0$ and $T_{2\mathrm{eff}}/T_g=0.2$. The flexible frame response is reduced by 23% in Zone III for $\kappa=1.0$ and $T_{2\mathrm{eff}}/T_g=5$. However, the presence of restrainers does not alter the general frame displacement trends due to pounding, thus underlining the importance of the pounding effect over the restrainer effect in the response of bridge frames.

Higher Degrees of Freedom Models to Investigate Two-Sided Impact

The earlier study of two adjacent bridge frames investigated *one-sided* pounding, where impact between bridge decks can occur only on one side of the decks. This is applicable to two-frame bridges where the abutments have failed or become ineffective under strong ground shaking, or where the gap is large enough to preclude interaction. For long multispan bridges, the impact between bridge decks is *two sided* since the interaction of frames

Table 1. Free-Field Ground Motions Listed in Chronological Order of Earthquake Occurrence

No.	Earthquake record	Location[a]	M_s[b]	PGA[c] (g)	T_g[d] (s)
1	1940 Imperial Valley	El Centro*	6.9	0.35	1.00
2	1989 Loma Prieta	Saratoga	7.1	0.47	0.40
3	1989 Loma Prieta	Holister*	7.1	0.37	1.03
4	1992 Landers	Baker Fire	7.5	0.11	1.70
5	1994 Northridge	Sylmar*	6.7	0.83	1.60
6	1994 Northridge	Taff	6.7	0.22	0.90
7	1994 Northridge	Pacoima Dam*	6.7	0.50	0.42
8	1994 Northridge	Lake Hughes	6.7	0.27	0.50
9	1994 Northridge	Lake Obrego Park	6.7	0.45	0.41
10	1995 Kobe	Kobe*	6.9	0.85	0.88
11	1995 Kobe	Osaka	6.9	0.08	1.17

[a] An asterisk indicates used in inelastic analyses.

[b] Magnitude.

[c] PGA.

[d] Characteristic period.

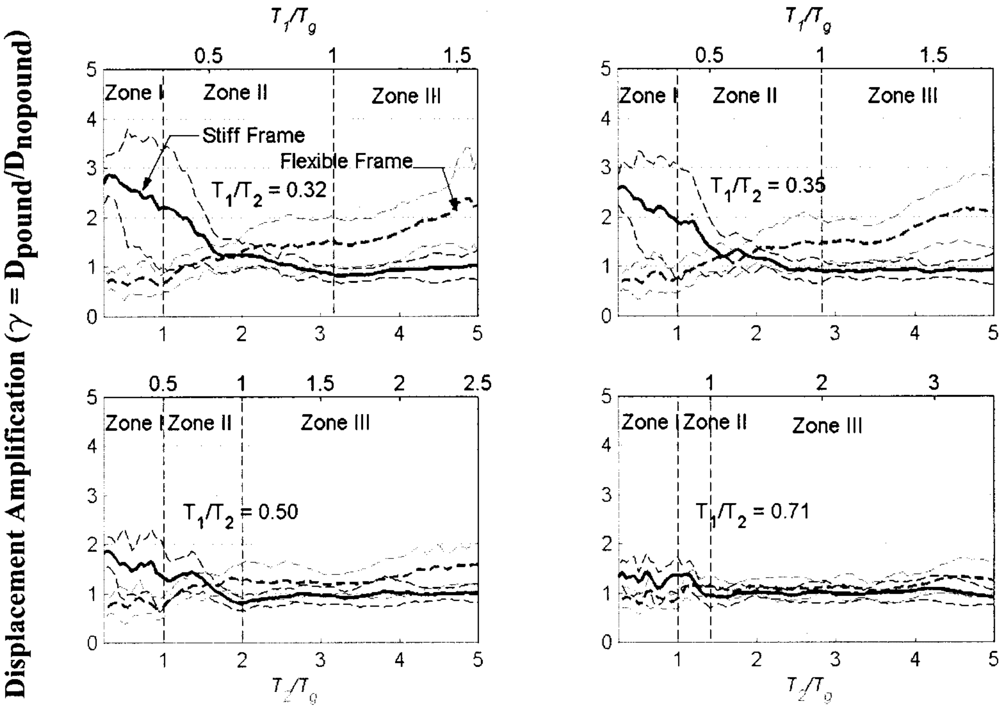

Fig. 4. Mean ±1 standard deviation displacement amplification for elastic frames: 11 earthquake records

can result in impact at both ends of a particular deck. Thus, a multiple DOF (MDOF) model is developed to incorporate the effects of two-sided pounding to investigate the global bridge response.

Each frame is idealized as a single DOF yielding element with mass m_i and a viscous damping coefficient c_i. The inelastic restoring force for each frame (R_i) is obtained from the force–deformation relationship for the frame. The abutments are modeled as link elements with a gap, and initial stiffness K_a capable of resisting only axial forces. The initial stiffness is assumed as 35 kN/mm per mm width of the backwall. Dynamic impact is modeled using the contact element approach. A linear spring of high stiffness is used to ensure small impact duration. The equations of motion describing the bridge response can be written in matrix form as shown

$$\begin{bmatrix} m_1 & & & \\ & \cdots & & \\ & & \cdots & \\ & & & m_4 \end{bmatrix} \begin{Bmatrix} \ddot{u}_1(t) \\ \cdots \\ \cdots \\ \ddot{u}_4(t) \end{Bmatrix} + \begin{bmatrix} c_1 & & & \\ & \cdots & & \\ & & \cdots & \\ & & & c_4 \end{bmatrix} \times \begin{Bmatrix} \dot{u}_1(t) \\ \cdots \\ \cdots \\ \dot{u}_4(t) \end{Bmatrix} + \begin{Bmatrix} R_1(u_1) \\ \cdots \\ \cdots \\ R_4(u_4) \end{Bmatrix} + \begin{Bmatrix} F_{c_1}(u_1-u_2-g_p) \\ \cdots \\ \cdots \\ F_{c_4}(u_3-u_4-g_p) \end{Bmatrix} = -\begin{bmatrix} m_1 \\ \cdots \\ \cdots \\ m_4 \end{bmatrix} \ddot{u}_g(t) \tag{7}$$

where F_c=the contact force due to pounding between the adjacent decks or pounding between deck and end abutment. The solution of this system is obtained numerically using DRAIN-2DX, with 5% modal damping.

Pounding is modeled using compression link elements with a gap of 12.5 mm. An impact spring stiffness of 17,520 kN/mm was determined as effective in limiting the penetration during impact. The effects of restrainers are not considered. The MDOF model is symmetric about the centerline with outer frame stiffness (K_1) and inner frame stiffness (K_2).

Inelastic Response

Inelastic behavior of the frames is investigated to determine the effects of two-sided pounding and the effects of abutments on the distribution of frame displacement demands. The target ductility of each frame when no pounding occurs is $\mu=4$. The frames are assumed to follow a bilinear force–deformation relationship with 5% strain hardening. K_1/K_2 is varied from 2–10 (T_1/T_2 varies from 0.3 to 0.7) and the ground motion effective period ratio $T_{2\mathrm{eff}}/T_g$ is varied as 0.25, 0.5, 1.0, 2.0, and 5.0. Five ground motion records from Table 1 are used as seismic input.

Fig. 7 presents the mean amplification and variability in displacement demand for various stiffness ratios. The mean demand on the stiff frame is increased by 260% and the flexible frame demand is decreased by 50% for $K_1/K_2=10$, in Zone I. Overall trends are similar to those obtained in the two-DOF study. Pounding effects increase for lower T_2/T_g and smaller T_1/T_2 values. The magnitudes of amplification are also similar. Both models exhibit a reduction in frame response when vibrating near the characteristic period (T_g). The effects of pounding are less pronounced as T_1/T_2 increases. However, the trends in Zone II seem

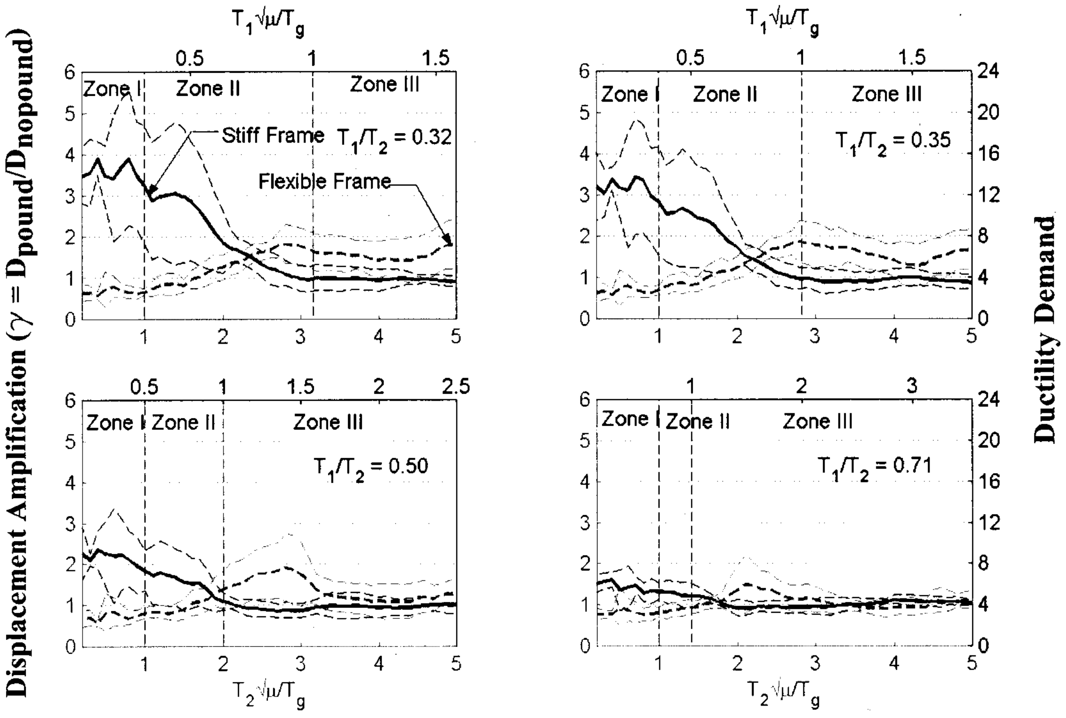

Fig. 5. Mean and variation in displacement demand for inelastic frames ($\kappa=0$); five earthquake records

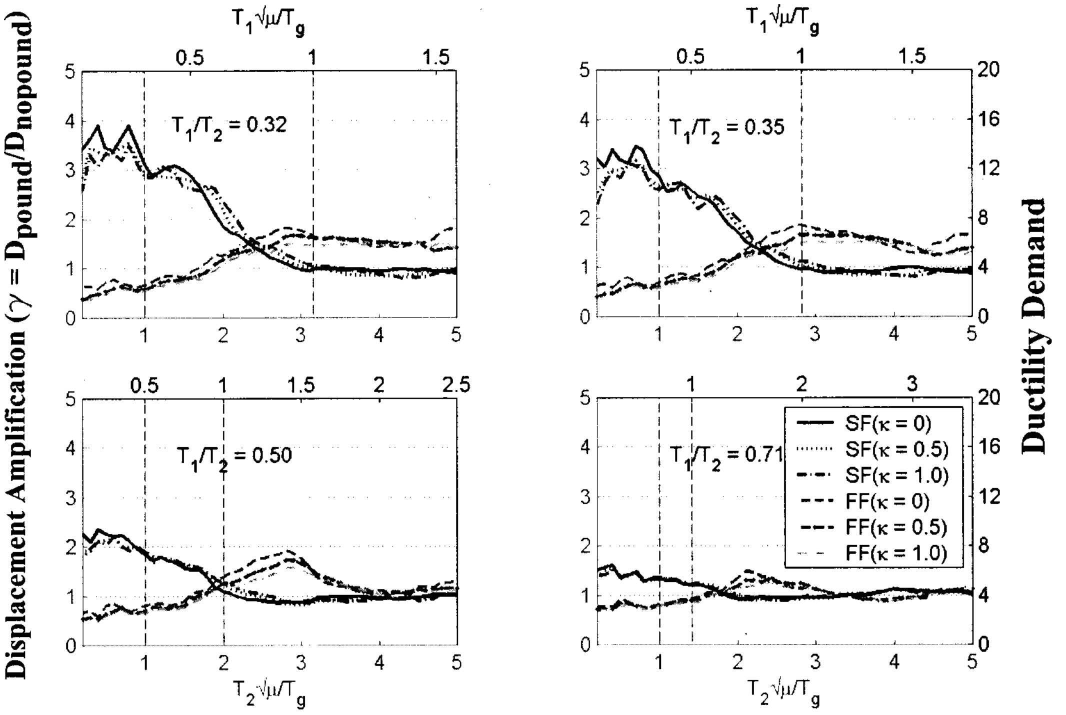

Fig. 6. Mean amplification in displacement demand for five earthquake records: inelastic frames with and without restrainers; target ductility $\mu=4$; SF—stiff frame; FF—flexible frame; restrainer stiffness ratio, $\kappa=0$, 0.5, and 1.0

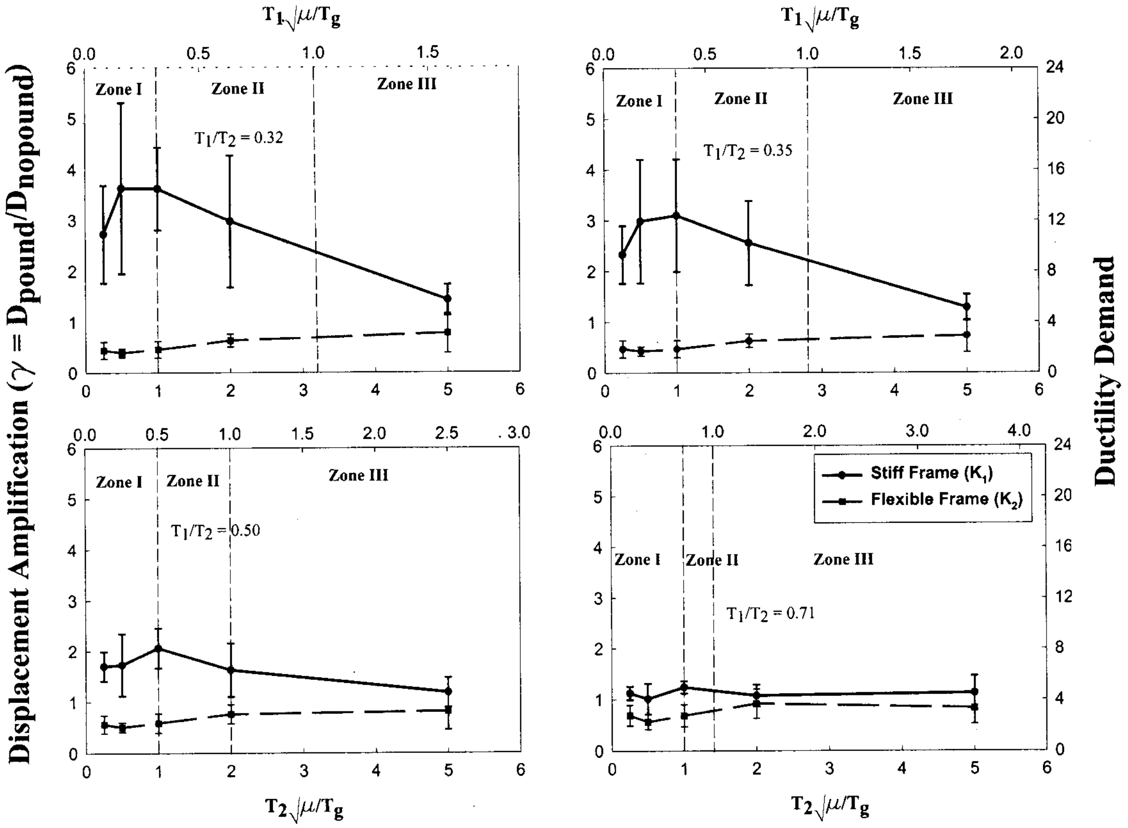

Fig. 7. Two-sided pounding amplification plots; inelastic frames without restrainers (μ=4); 5 earthquake records

to contradict the earlier results observed with the two-DOF stereomechanical model, where both frame responses were amplified due to pounding. This is attributed to the two-sided nature of impact in the higher DOF model. The flexible frame response is mitigated due to interaction with adjacent frames on both sides of the deck.

Evaluation of Caltrans Recommendation through Case Study

To investigate the recommendation suggested by the 1999 Caltrans Seismic Design Criteria for bridge period ratios, a two-dimensional nonlinear model of a typical multiple-frame bridge shown in Fig. 1 is developed. The bridge has four frames connected at three intermediate hinges and has a span of 504 m (1680 ft). The longitudinal frame period ratio of the bridge (T_1/T_2) is 0.43. The superstructure is supported on ten single-column bents. The bents are designed to be nearly rectangular with chamfered ends. The moment capacities and yield moments are obtained through moment-curvature analysis of the column crossections. Pounding is modeled using a contact spring with a stiffness of 17,520 kN/mm. The bridge model is then subjected to the 1994 Sylmar Free Field record, Northridge earthquake, scaled to 0.7 g.

Fig. 8 presents the time history of a typical pier drift ratio for

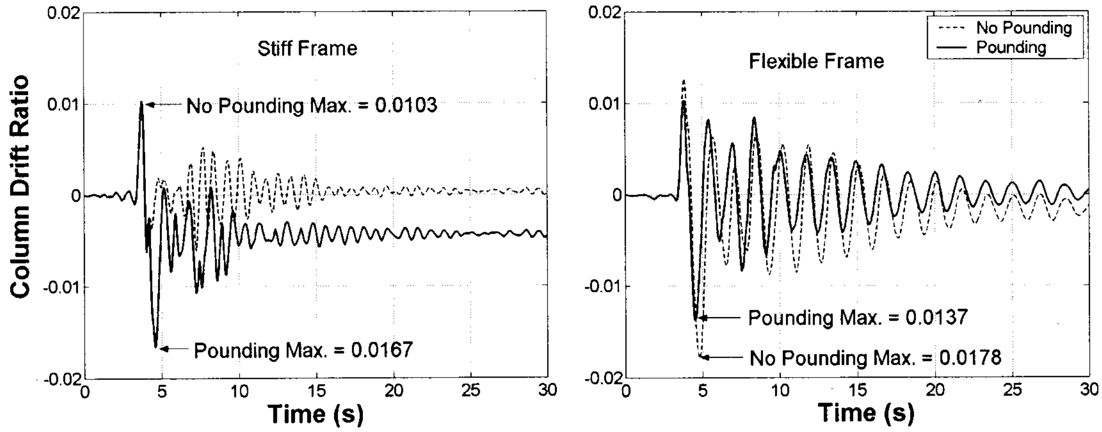

Fig. 8. Time history of column drift ratios; multiple frame bridge with $T_1/T_2=0.43$; Sylmar free field record

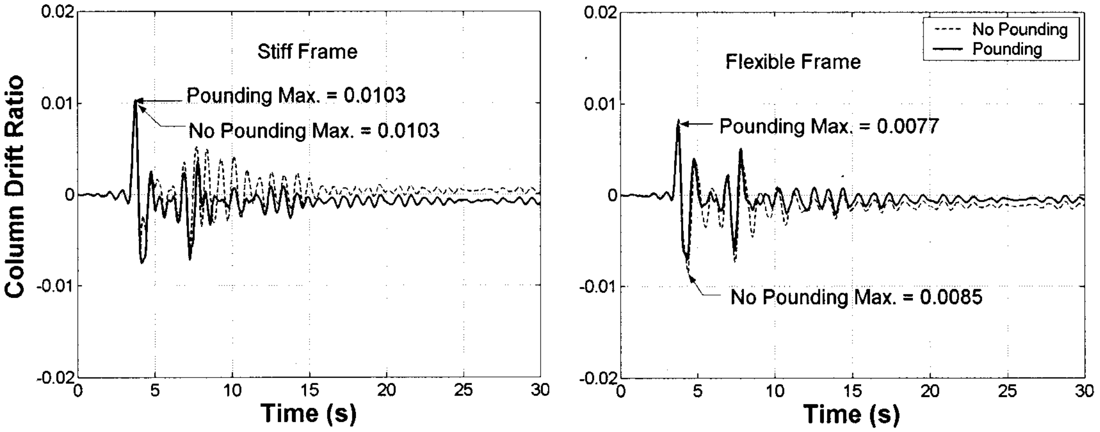

Fig. 9. Time history of column drift ratios; multiple frame bridge with $T_1/T_2=0.72$; Sylmar free field record

the pounding and no-pounding studies. The comparison for the stiff frame shows that pounding increases the drift ratio of pier 1 (Frame 1) from 1.0% when no pounding occurs to 1.7% when the frames are pounding. This corresponds to an increase in the column curvature ductility from 4.3 to 8.0. Conversely, for the flexible frame, pounding reduces the drift ratio of pier 6 (Frame 3) from 1.8% in the no-pounding case to 1.4% when the frames pound. The corresponding reduction in the curvature ductility is from 6.4 to 4.9.

In order to mitigate the pounding effect, the longitudinal period ratio of the bridge is increased, as recommended by Caltrans. Two columns are added to each of the inner frames, thus bringing the frame period ratio to 0.72. The response of this modified bridge to the Sylmar Free Field record is evaluated. A comparison of the no-pounding and pounding responses, shown in Fig. 9, reveals that the effect of pounding on the demands on the frames is considerably reduced. The stiff column drift ratio remains constant at 1.0% ($\mu=4.4$). For the flexible column, the pounding and no-pounding drift ratios differ by a minimal amount, as shown in Fig. 9. This example illustrates the significant effect of the frame period ratio on the pounding response of bridge frames and validates the recommendations by Caltrans for bridge frame period ratios.

Conclusions

This paper investigates the effect that pounding and restrainers have on the global demand of bridge frames in a multiframe bridge. The primary factors affecting the pounding response are identified as the frame stiffness ratio (K_1/K_2) or period ratio (T_1/T_2), and the ground motion effective period ratio ($T_{2\text{eff}}/T_g$). Parametric studies of one-sided pounding using simplified two-DOF models show that pounding is most critical for highly out-of-phase frames. Pounding reduces the frame response when vibrating near the characteristic period of the ground motion (T_g). The amplification in frame response as a function of $T_{2\text{eff}}/T_g$, and T_1/T_2 falls into three regions. In Zone I ($T_{2\text{eff}}/T_g<1$), the stiff frame demand increases and the flexible frame demand decreases due to one-sided pounding. In Zone III ($T_{1\text{eff}}/T_g>1$), the flexible frame pounding response is increased while the stiff frame pounding response decreases. In Zone II ($T_{1\text{eff}}/T_g<1$ and $T_{2\text{eff}}/T_g>1$), pounding slightly increases both frame responses.

Inelastic behavior (frame design ductility, $\mu=4$) shows greater stiff frame amplification in Zone I when compared to the linear case. The yielding of frames also results in smaller response amplification for the flexible frame in Zone III, when compared to elastic behavior. The effect of restrainers on the pounding response of bridge frames is evaluated. The results show that restrainers have very little effect on the demands on bridge frames compared with pounding.

Investigations of two-sided pounding using MDOF models show a favorable postimpact response for the flexible frame and a detrimental effect for the stiff frame demand, for all period ratios. Results from both one-sided and two-sided impact reveal that the response of bridge frames due to pounding is much less pronounced for $K_1/K_2=2.0$ ($T_1/T_2=0.7$) irrespective of the ground motion period ratio, thus validating the recommendations suggested by Caltrans.

Acknowledgments

This study is an extension of previous work supported by Caltrans under Contract No. RTA-59A131. The writers wish to thank Michael Keever for his valuable comments.

Notation

The following symbols are used in this paper:

C = damping matrix;
D_{np} = relative displacement of the hinge when no pounding occurs;
e = coefficient of restitution;
F = restoring force due to restrainers;
F_c = impact force due to pounding;
F_y = yield force of the frame;
g_p = hinge gap;
K_{eff} = effective frame stiffness when inelastic;
K_r = restrainer stiffness;
K_1, K_2 = elastic frame stiffnesses;
M = mass matrix;
m_1, m_2 = frame masses;
R = vector of restoring forces for the frames;
T_{eff} = effective frame period when inelastic;

T_g = characteristic period of the ground motion;
T_1, T_2 = frame periods based on initial stiffness;
u = frame displacement relative to the ground;
$\ddot{u}$ = frame acceleration relative to the ground;
$\dot{u}$ = frame velocity relative to the ground;
$\ddot{u}_g$ = input acceleration;
v_1, v_2 = frame velocities before impact;
v_1', v_2' = frame velocities after impact;
α = gap ratio parameter;
γ = displacement amplification ratio;
κ = normalized restrainer stiffness; and
μ = frame displacement ductility demand.

References

Athanassiadou, C. J., Penelis, G. G., and Kappos A. J. (1994). ‘‘Seismic response of adjacent buildings with similar or different dynamic characteristics.’’ *Earthquake Spectra*, 10(2), 293–317.

Caltrans Seismic Design Criteria. (1999). *design manual-version 1.1*, Calif. Dept. of Transportation, Sacramento, Calif.

DesRoches, R., and Fenves, G. L. (1997). ‘‘New design and analysis procedures for intermediate hinges in multiple-frame bridges.’’ *Rep. No. UCB/EERC-97/12*, Earthquake Engrg. Res. Ctr., University of California, Berkeley, Calif.

Earthquake Engineering Research Institute (EERI). (1995a). ‘‘Northridge earthquake reconnaissance report, Vol. 1.’’ *Rep. No. 95-03*, EERI, Oakland, Calif.

Earthquake Engineering Research Institute (EERI). (1995b). ‘‘They Hyogo-Ken Nanbu earthquake reconnaissance report.’’ *Rep. No. 95-04*, EERI, Oakland, Calif.

Earthquake Engineering Research Institute (EERI). (2001). ‘‘1999 Chi-Chi, Taiwan, Earthquake Reconnaissance Report.’’ *Rep. No. 01-02*, EERI, Oakland, Calif.

Jankowski, R., Wilde, K., and Fuzino, Y. (1998). ‘‘Pounding of superstructure segments in isolated elevated bridge during earthquakes.’’ *Int. J. Earthquake Eng. Struct. Dyn.*, 27, 487–502.

Jankowski, R., Wilde, K., and Fuzino, Y. (1999). ‘‘Reduction of earthquake induced effects of pounding in elevated bridges.’’ *Proc., 2nd World Conf. on Struct. Control*, Vol. 2, Wiley, Chichester, England, 933–939.

Jennings, P. C. (1971). ‘‘Engineering features of the San Fernando Earthquake of February 9, 1971.’’ *Rep. No. EERL-71-02*, Earthquake Engrg. Res. Laboratory, Calif. Inst. of Tech., Pasadena, Calif.

Kawashima, K., and Shoji, G. (2000). ‘‘Effect of shock absorber to mitigate pounding effect between bridge decks.’’ *Proc., Int. Workshop on Mitigation of Seismic Effects on Transportation Struct.*, National Center for Research on Earthquake Engineering, Taipei, Taiwan, R.O.C., 207–218.

Kawashima, K., and Yabe, M. (1996). ‘‘Effectiveness of unseating prevention device with energy dissipation.’’ *Proc., 4th U.S.–Japan Workshop on Earthquake Protective System for Bridges*, Public Works Research Institute, Tsukuba-shi, Japan, 285–307.

Kim, S. H., Lee, S. W., Won, J. H., and Mha, H. S. (2000). ‘‘Dynamic behaviors of bridges under seismic excitations with pounding between adjacent girders.’’ *Proc., 12th World Conf. on Earthquake Engineering*, New Zealand Society for Earthquake Engineering, Upper Hutt, New Zealand.

Ma, X., and Pantelides, C. P. (1988). ‘‘Linear and nonlinear pounding of structural systems.’’ *Comput. Struct.*, 66(1), 79–92.

Maison, B. F., and Kasai, K. (1992). ‘‘Dynamics of pounding when two buildings collide.’’ *Earthquake Eng. Struct. Dyn.*, 21, 771–786.

Malhotra, P. K. (1998). ‘‘Dynamics of seismic pounding at expansion joints of concrete bridges.’’ *J. Eng. Mech.*, 124(7), 794–802.

Maragakis, E., Douglas, B., and Vrontinos, S. (1991). ‘‘Classical formulation of the impact between bridge deck and abutments during strong earthquakes,’’ *Proc., 6th Canadian Conf. on Earthquake Engineering*, Univ. of Toronto Press, Toronto, 205–212.

Miranda, E., and Bertero, V. V. (1994). ‘‘Evaluation of strength reduction factors for earthquake-resistant design.’’ *Earthquake Spectra*, 10(2), 357–379.

New Zealand National Society for Earthquake Engineering (NZNSEE). (1990). ‘‘The Loma Prieta, California, Earthquake of October 17, 1989: Report of the NZNSEE Reconnaissance Team.’’ NZNSEE, Upper Hutt, New Zealand.

Newmark, N. M., (1959). ‘‘A method of computation for structural dynamics.’’ *J. Eng. Mech.* 85, 67–94.

Priestley, M. J. N., Seible, F., and Calvi, G. M. (1995). *Seismic design and retrofit of bridges*, Wiley, New York.

Saiidi, M., and Sozen, M. A. (1979). ‘‘Simple and complex models for nonlinear seismic response of reinforced concrete structures.’’ *Rep. No. UILU-ENG-79-2013*, *Struct. Res. Series No. 465*, Univ. of Illinois, Urbana, Ill.

Trochalakis, P., Eberhard, M., and Stanton, J. (1997). ‘‘Design of seismic restrainers for in-span hinges.’’ *J. Struct. Eng.*, 123(4), 469–478.