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Effects of Pounding at Expansion Joints of Concrete Bridges

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Abstract

This paper presents the results of a study on the effects of pounding at expansion joints of concrete bridges under earthquake ground motions. An engineering approach, rather than continuum mechanics, is emphasized. First, the sensitivity analysis of the gap element stiffness is performed. Second, usefulness of the analysis method for simulation of pounding phenomena is demonstrated. Third, the effects of pounding on the ductility demands measured in terms of the rotation of column ends are investigated. Two-dimensional FE analysis using a bilinear hysterestic model for bridge substructure joints and a nonlinear gap element for the expansion joint is performed on a realistic bridge with an expansion joint. Effects of the primary factors on the ductility demand such as gap sizes and characteristics of earthquake ground motion are investigated through a parametric study. The major conclusions are that pounding effect is generally negligible on the ductility demand for wide practical ranges of gap size and peak ground acceleration, but is potentially significant at the locations of impact.

keyword : impact, bridges, concrete, expansion joint, dynamic structural analysis, ductility

1. Introduction

Pounding at expansion joints (hinges) in long, multispan, concrete bridges has been the source of extensive damage during past earthquakes. Due to Northridge earthquake in 1994, the most significant pounding damage was observed at expansion joints among other similar cases, at the Interstate 5 and State Road 14 interchange located approximately 12 km from the epicenter. In previous studies, the mechanism of pounding was investigated and parametric analyses were carried out to identify the most important dynamic and kinematic parameters associated with the pounding. These studies used either the impact-restitution approach[1,2,3] or the contact-element approach[4,5,6].

If a large spring stiffness in contact-element is used to avoid overlapping between adjacent segments and to ensure small time duration of pounding, it will lead to numerical difficulties and produce unrealistically high pounding forces, depending on analytical approach methods and/or computer codes used. Therefore, a need is required for a more in-depth investigation on this matter to provide an engineering insight to the problems of pounding in bridge structures. In most recent studies on the matter, the bridges were represented by a damped single-degree-of-freedom (SDOF) system, and in some cases supported by a parallel combination of a spring and a hysterestic damper. Although this type of representation for simulating pounding in the bridge structures allows for capturing some dynamic characteristics, it does not take into account all the characteristics of structures that contribute to the pounding especially in the higher modes vibration arising from the entire bridge responding as a system. Since pounding phenomena are dynamically and kinematically complicated, its characteristics and consequences should be examined through finite element analysis which can represent a more realistic bridge.

This study endeavors to develop an engineering approach, rather than the continuum mechanics approach, that can be used for practical evaluation of pounding effect on bridges. This engineering approach utilizes design applicationoriented at finite element codes with gap element capability, and enables one to evaluate the effect of the pounding on the entire bridge as a system. The study particularly focuses on the sensitivity of the substructural response, especially ductility of the column structures, to the gap size and spring stiffness of the gap element.

2. Gap Element and Sensitivity Analysis

2.1 Nonlinear Gap Element

Perhaps one of the most difficult-to-analyze nonlinear behaviors that occur in bridge systems idealized to include gap elements is the closing of a gap between different segments of the bridge. The usual gap element shown in Fig. 1 has the following physical properties: 1) The element cannot develop a force until the opening d_0 is closed; and 2) the element can only develop a compression force. It should be noted that the numerical convergence of the response analysis particularly at the gap element can be very slow if a large elastic stiffness k is used. In order to minimize the difficulty associated with this problem, the stiffness k should not be over 1,000 times the stiffness of the elements adjacent to the gap, according to the author's experience. This kind of dynamic contact problem involving two adjacent structural segments usually does not have a simple and unique solution. In fact, it is impractical to use continuum mechanics analysis in the vicinity of the contact area for local stress and strain evaluation and at the same time to

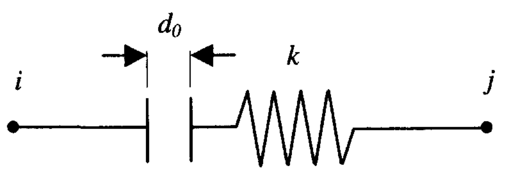


Fig. 1 Gap element

pursue structural dynamic analysis to evaluate the bridge response as a system including, for example, ductility demand at it's column ends. A viable alternative appears to be the deployment of the finite element analysis with gap elements having carefully selected values of stiffness value k.

2.2 Sensitivity Analysis of Gap Element Stiffness

Nonlinear time history analysis is carried out for a wide range of the stiffness value of the gap element with a bridge model. The results are presented in terms of ductility demand at the end of the four columns as shown in Fig. 2.

These results demonstrate that, for this particular example, the ductility demands at the column ends vary as functions of k in such a way that they are more or less constant up to $k = 10^3\,kN/m$ (the first regime), there is a transition regime between $k = 10^3\,kN/m$ and $k = 10^6\,kN/m$ (the second regime) where they appreciably change, and they become constant again beyond $k = 10^6\,kN/m$ (the third regime). The three regimes observed conceivably arise in relation to the value of k relative to the axial stiffness of the colliding girders. The maximum ductility demands observed are not particularly larger under the effect of pounding than under the case of no pounding, although the acceleration response is significantly influenced by the change in the k value. These observations are consistent with the previous results by other researchers[4]. Easy and crisp explanations for them appear unlikely to emerge. Nevertheless attempts are made here to gain some physical insights to this difficult problem. For this purpose, pounding forces are plotted and pounding magnitudes indicated as "area" in Fig. 3 for three values of $k (= 10^3\,kN/m, 10^4\,kN/m$ and $10^5\,kN/m)$ and in Fig. 4 for three values of $k (= 10^6\,kN/m,\ 10^7\,kN/m$ and $10^8\,kN/m)$. Second pounding force or third pounding force in each case is enlarged as a function of time to show more clearly the magnitude, shape and duration associated with this pounding. It is shown that pounding in the third regime occurs almost at the same time for each k, but has different forces, shapes and time duration. However, the pounding magnitudes are almost same as 147.13, 149.57 and 146.64 for the case of the first pounding and 448.39, 430.98 and 408.59 for the case of the third pounding for each k respectively while the magnitudes before convergence are quite different from those values. Thus, as aforementioned, the responses of the displacement will be same once the structures are subjected to the pounding force having a same magnitude. It can be a good reason to explain about the insensitivity, while some conclusions reached by other computer codes might be contradictory because of differences in the cases that they have considered. It is also noted that the value of the pounding force produced by the largest k fluctuate significantly within the duration of each pounding because of high frequency properties.

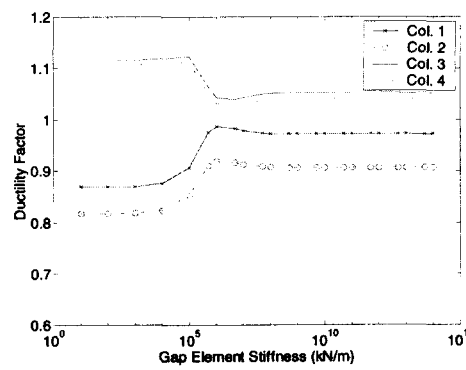


Fig. 2 Ductility factors w.r.t. gap element stiffness

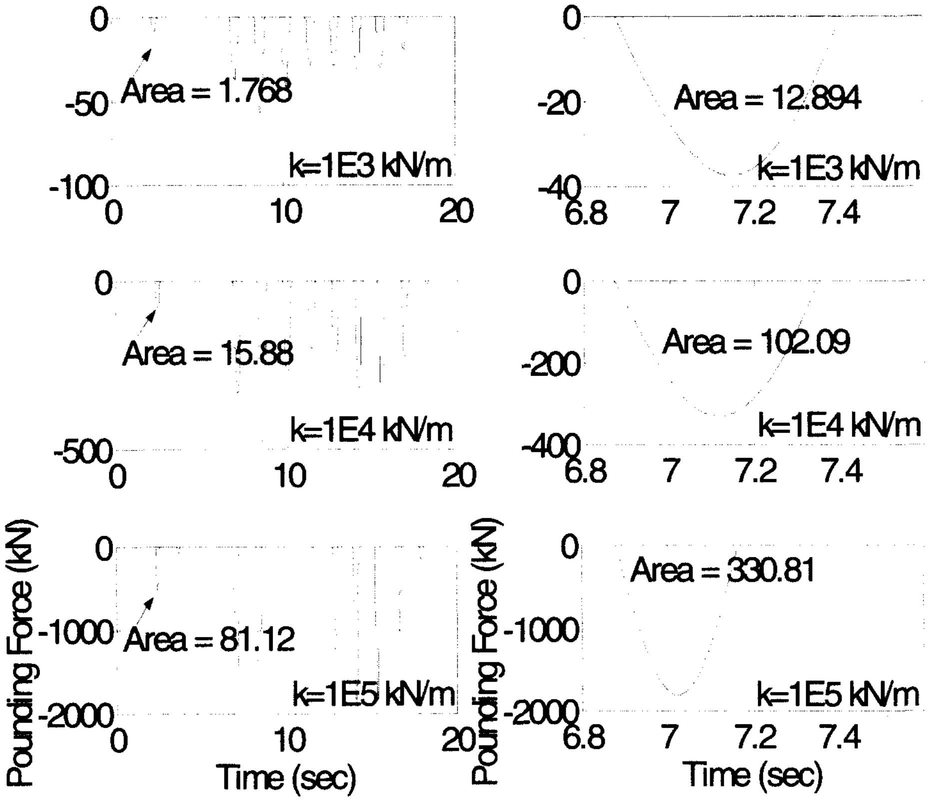


Fig. 3 Pounding forces for three values of *k*

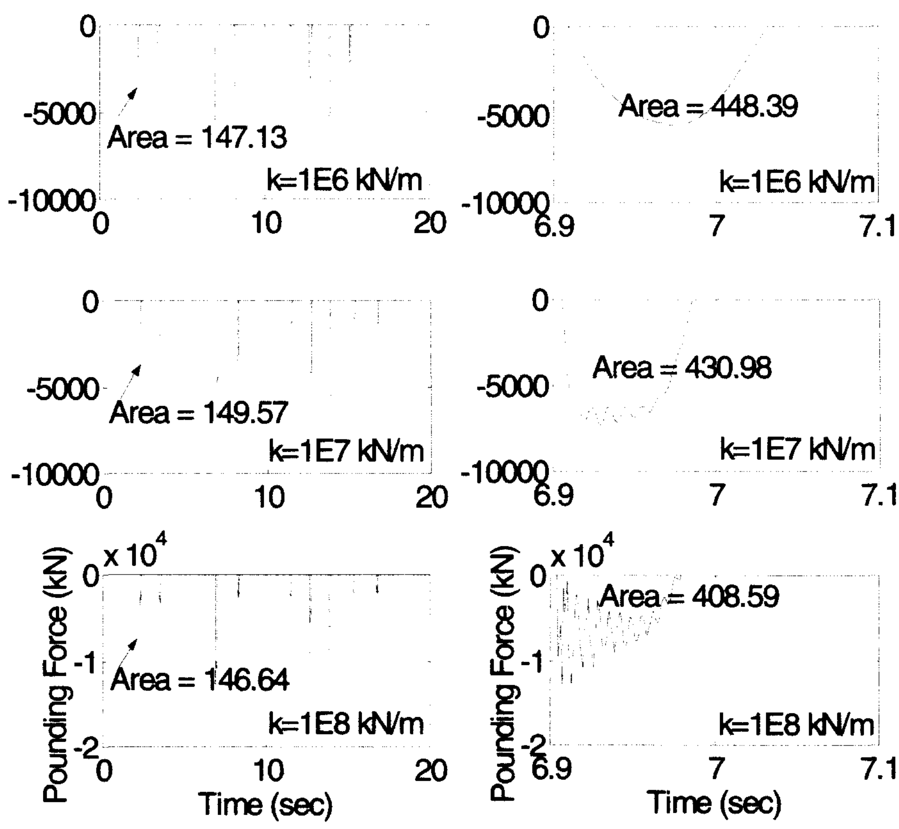


Fig. 4 Pounding forces for three values for *k*

3. Pounding Analysis

3.1 Example Bridge Model and Nonlinearities

A general model representing typical Caltrans' bridges built with expansion joints are considered for the nonlinear time history analysis(7). The nonlinearities included in this study are yielding of columns and pounding of decks at the expansion joint. The *SAP2000/Nonlinear* finite element computer code(8) is utilized for the extensive two-dimensional response analysis of the bridge including the nonlinearities. The El Centro earthquake ground motion considered earlier, is also used here as input for the numerical simulation.

It is typical of a California highway bridge with more than four spans to have expansion joints located nearly at inflection points (i.e., 1/4 to 1/5th of spans). The bridge superstructure consists of reinforced or prestressed concrete box girders. The typical Caltrans' bridge with expansion joints considered in this study is a five span bridge with one expansion joint and equal column height of 21.0 m. The geometry and boundary conditions of the bridge model are shown in Fig. 5. The material and cross-sectional properties of the model as follows: Young's modulus=27.793 GPa, mass density=2.401 Mg/m^3, uniform cross-section area and moment of inertia are respectively 6.701 m^2 and 4.625 m^4 for box girders, while they are 4.670 m^2 and 0.620 m^4 for columns. To reflect the cracked state of a concrete bridge column for the seismic response analysis, an effective moment of inertia(50 % reduction of the gross section moment of inertia) is employed, making the period of the bridge longer. The Column Ductility Program *COLx* is used to model the moment-curvature relationship of plastic hinges for columns.

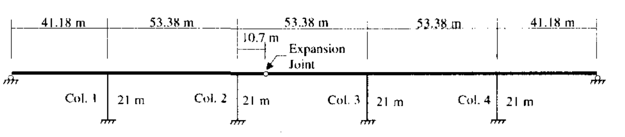


Fig. 5 Elevation of example bridge

The nonlinearities involved in the bridge analytical model are depicted in Fig. 6. The plastic hinge formed in the bridge column is assumed to have bilinear hysterestic characteristics. The expansion joint is constrained in the relative vertical movement, while freely allowing horizontal opening movement and rotation. The closure at the joint, however, is restricted by a gap element when the relative motion of adjacent decks exhausts the initial gap width of 2.54cm (1.0 in).

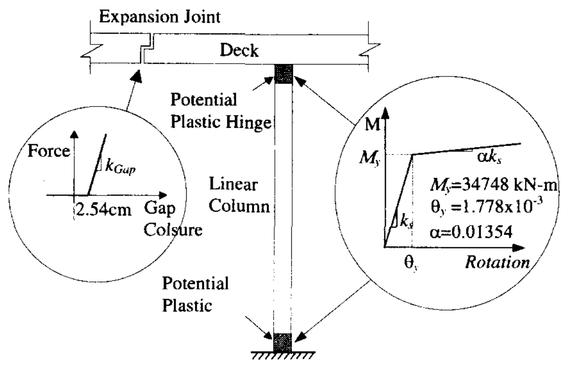


Fig. 6 Nonlinear modeling of bridge

3.2 Numerical Simulation

The results for the case of vibration without pounding were compared to those for the case with pounding in order to highlight how pounding changes the response behaviors. Numerical simulations were carried out under El Centro ground motion excitation as shown Fig. 7(a). Impact force histories are shown in Fig. 8(a). Time histories of accelerations, velocities and displacements of the expansion joint, axial forces of girder and rotations of column ends without and with pounding are plotted in Figs. 7(b), (c), (d), (e) and (f) and Figs. 8(b), (c), (d), (e) and (f).

From these results, it is observed that (1) the pounding takes place eight times with maximum force 6,230 kN; (2) the acceleration and the axial force are affected more by pounding; (3) the separation at the hinge and the peak value of the rotations at column ends are reduced by pounding.

The peak pounding force in Fig. 8(a) is F=6,230 kN and the peak axial force in Fig. 8(e) is F=5,963 kN which is 36 times more than the peak axial force in Fig. 7(e). Thus the axial force generated by pounding is extremely large. However, considering that the cross-sectional area of the box-girder, the axial compressive stress is F/A=0.1 MPa, which is significantly smaller than the compressive strength of concrete (30 MPa). It is not expected, however, that the actual contact between the adjacent segments occurs uniformly over the entire area at the instant of pounding. Thus, it may cause significantly high axial compressive stress locally, leading to some local damage at the contact area at the expansion joint.

The coefficient, which controls the rebound velocities, defined as the ratio of the separation velocity to the approach velocity, is determined from

$$e = \frac{v_2' - v_1'}{v_1 - v_2} \tag{1}$$

where v_1 and v_2 are the velocities of the two masses before pounding, and v_1', v_2' are the velocities after pounding. This value can be calculated from Fig. 8(c). The

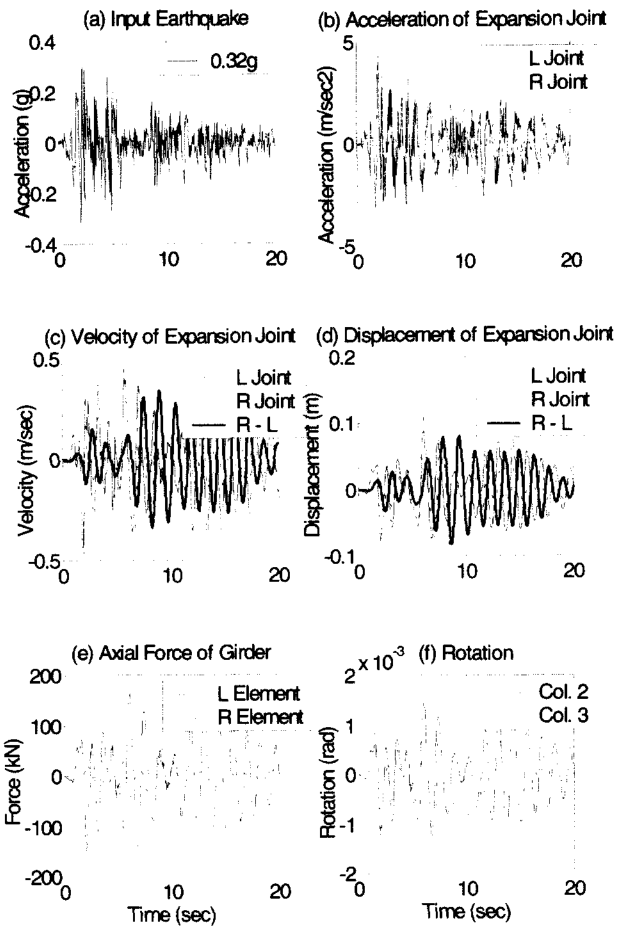


Fig. 7 Responses for the case without pounding

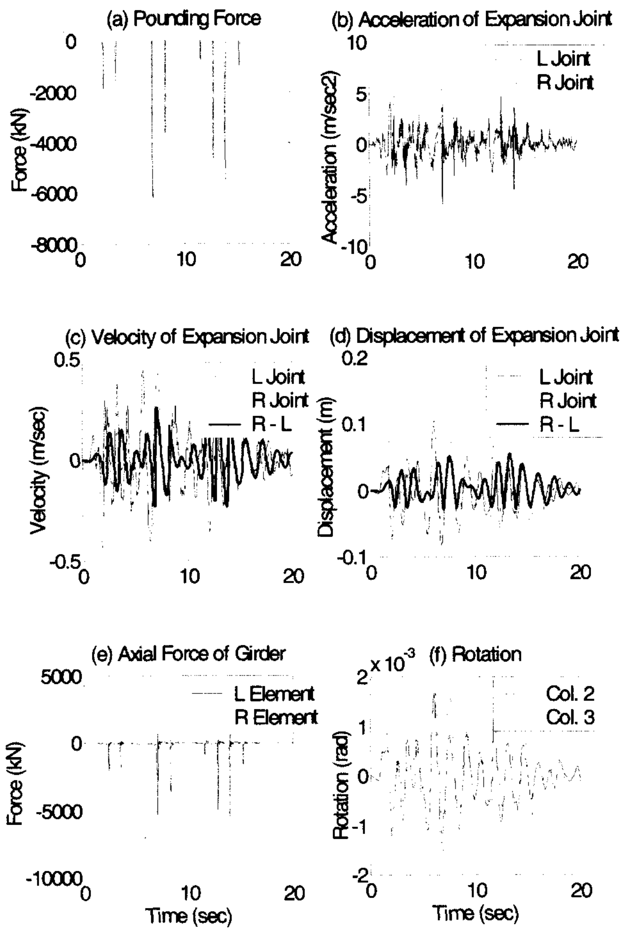


Fig. 8 Responses for the case with pounding

$v_1 = 0.14945 m/\sec$, $v_2 = 0.05377 m/\sec$ at the time 2.26 sec just before pounding and $v_1' = 0.23705 m/\sec$, $v_2' = 0.2966 m/\sec$ at the time 2.38 sec just after pounding lead to $e = 0.62$, meaning 38% energy loss.

The longitudinal separation at the hinge joint in Fig. 7(d) is 83 mm and that in Fig. 8(d) is 55 mm. The adequacy of the bearing seat width can be assessed. Because the maximum separation of 55 mm with pounding is less than the general seat width, it is expected that the left girder will not fall off from the seat in this case, judging from joint configuration shown in Fig. 6. Values of separation in Fig. 8(d) equal to (-) 25.4 mm at times t=2.38, 3.52, 6.98, 8.18, 12.78, 13.90, 15.36, and 16.84 sec corresponding to a complete closure of gap between adjacent segments. Values less than (-) 25.4 mm are explained by that the gap element undergoes axial compression during pounding and results in contraction of length, so that the value can be less than (-) 25.4 mm. The maximum amount beyond (-) 25.4 mm is 3.47 mm in this case.

The rotations of column ends are also shown in Fig. 7(f) and Fig. 8(f) with peak values of 0.00155, 0.00146, 0.00198 and 0.00197 radians for column 1, 2, 3 and 4 respectively for without pounding, while 0.00176, 0.00164, 0.00184 and 0.00182 radians with pounding, although pounding does not happen at the instant of peak value. It is also observed that the rotations of the column 1 and 2 in the stiffer frame increases while those of the column 3 and 4 in the more flexible frame are decreases at the time of pounding.

It should be noted that the bridge has dominant longitudinal free vibration modes with periods of 1.5215 second and 1.3580 second in each isolated frame separated by expansion joint. The frame period ratio is 0.8925 (=1.3580/1.5215). Low frame period ratio means a very stiff frame next to a flexible frame. The results of the previous studies[2,9] indicate that high response amplifications due to pounding occur if the colliding frames are significantly different in period. It is common, however, that bridges have a frame period ratio between 0.8~0.9. Effects of period ratio are thus excluded in this consideration.

4. Effects on Ductility Demand

Numerical simulations were performed for the bridge considered earlier with various gap sizes and Peak Ground Acceleration (PGA) for the four different earthquakes listed in Table 1 in order to investigate effects of poundings on ductility demand. Fig. 9 shows the Fourier amplitude of these earthquakes. Ten different values of gap size d and

PGA p were used in the ranges $10 \le d \le 100mm$ and $0.1 \le p \le 1.0g$ for each earthquake. Note that the horizontal components of the original accelerations were linearly scaled in accordance with the maximum PGA.

Table 1 Earthquake motions used in Analyses

Earthquake	Component	Peak ground acceleration (g)
El centro (1940)	NS	0.32
Loma prieta (1989)	EW	0.13
Northridge (1994)	NS	0.59
Kobe (1995)	NS	0.84

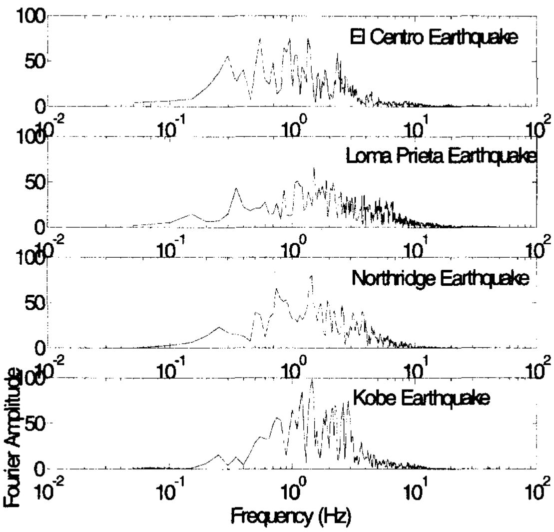
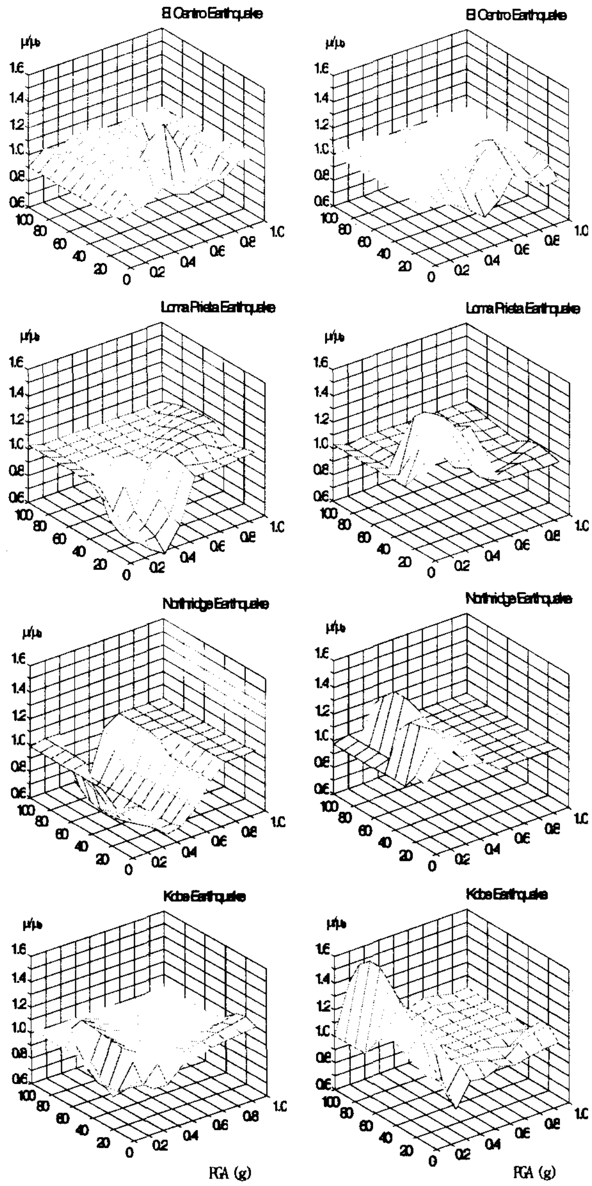
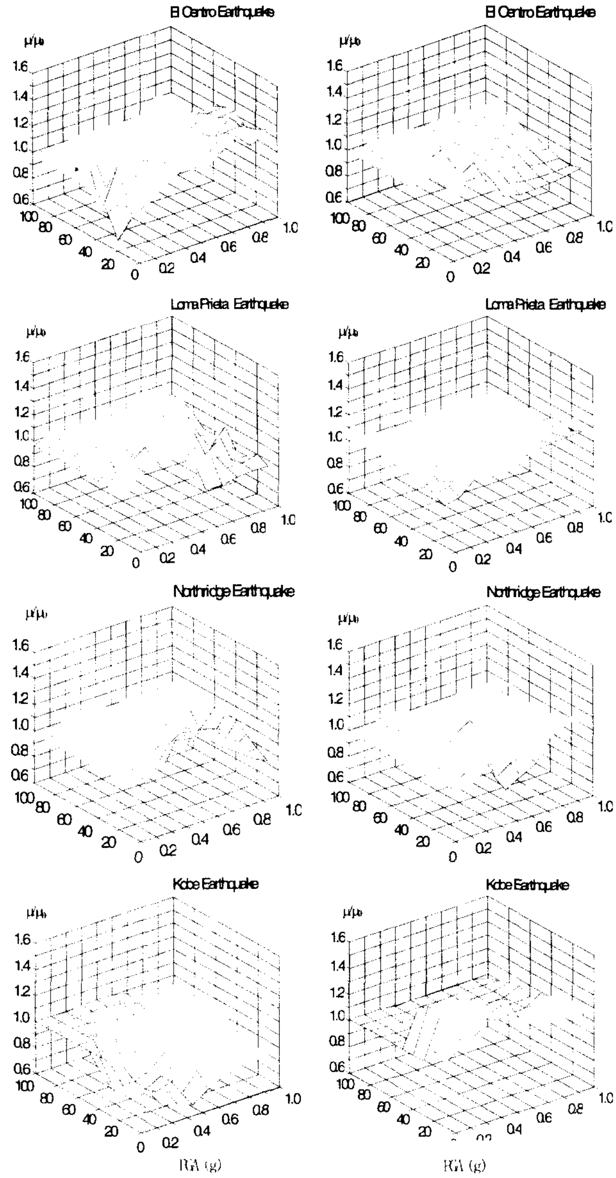


Fig. 9 Fourier amplitude of four earthquakes



(a) Column 2 (b) Column 3

Fig. 10 Ratio of ductility demand w.r.t. gap size and PGA for flexible bridge



(a) Column 2 (b) Column 3

Fig. 11 Ratio of ductility demand w.r.t. gap size and PGA for stiff bridge

The parameter used to describe the (nonlinear) structural response is the ductility demand. The ductility demand is defined as follows: denoting by θ the rotation of the nonlinear spring used to model the plastic zone at each end of every column, and by θ_y the corresponding rotation at the yield point, the ratio θ/θ_y is defined as the ductility demand at the plastic zone. Results are presented as shown Fig. 10 in terms of the ratio μ/μ_0 for the four earthquakes, where μ and μ_0 represent the ductility ratio with and without the pounding effect, respectively. It should be noted that the range of ductility demands for four earthquakes are 0.28~6.41, 0.56~19.35, 0.68~10.63 and 0.73~10.63 in the order of Table 1.

It is observed from Fig. 10 that pounding does not increase the ductility demand very much. Because it is highly unlikely that pounding will take place at the instant of the peak rotation of the column end in such a way that it will further amplify the rotation. The maximum increases of the ratios (μ/μ_0) are 31% under 40mm gap size and 0.6g PGA for El Centro earthquake, 52% under 20mm gap size and 0.3g PGA for Loma Prieta earthquake, 43% under 60mm gap size and 0.3g PGA for El Centro earthquake and 43 % under 60mm gap size and 0.3g PGA. The figure does not suggest the existence of consistent trend in the effect of the gap size and PGA on the ductility demand. They do, however, confirm that the gap size and PGA do have influence on ductility demand. Therefore, in this bridge with an expansion joint, the pounding does not always adversely affect on the column responses with decreasing gap size and increasing PGA.

Another set of pounding analysis was performed with a stiffer bridge to examine the effects of the bridge stiffness on the ductility demands. This bridge has dominant longitudinal free vibration modes with periods of 0.9270 second and 0.8288 second in each isolated frame separated by the expansion joint. The results are plotted in Fig. 11 which exhibits the effect clearly, even though not showing a definite trend.

It is prudent to assume, therefore, that the pounding effect on the ductility demand must be formed under a specific value of gap size and a specific input ground acceleration time history, individually. However, this observation might not always apply, depending on the details of specific bridge characteristic.

5. Conclusions

The review of the past work by many researchers has indicated that it is very difficult to draw general conclusions concerning the effect of pounding due to seismic ground motion on the response of bridge structures. First of all, it is not quite possible to make comparisons between the results of the different studies, as different researchers use different structures, different ways to model their structures, different models for simulating pounding, different methods to analyze the structures, and different quantities to measure the response of the structures. Therefore some of the conclusions reached by these researches might seem contradictory each other because of the different considerations under which their analyses were performed. There is not even an agreement about whether the pounding due to seismic ground motion is beneficial or detrimental to the structural response as a whole. The only common conclusion is that the effect of pounding on the response of the structure is a very complex one, depending on various parameters describing the structures and the characteristics of the ground motion.

This study presented the results of nonlinear analysis of the bridge taking pounding effect into consideration. The nonlinearities of yielding of substructure and pounding in superstructure are included in the analysis. The results indicate that the pounding does indeed affects significantly on the bridge dynamic response but does not increase the displacement responses appreciably. Followings are more specific conclusions based on the results obtained from this study.

(1) The computed displacement responses due to pounding are not very sensitive to the stiffness of the gap element, because they are only affected by the pounding magnitude. If a larger stiffness, thereby resulting in larger impact forces due to pounding, is used, the duration of impact will be shorter, thus leading to a similar pounding magnitude.

(2) Pounding effect with variations in gap width and peak ground acceleration is found to have negligible effect on the ductility demand, but some effect, at the most 62% in this study, within a certain range of conditions.

(3) It should be noted that high response amplifications due to pounding will occur only if the colliding bridge segments separated by an expansion joint are significantly different in period. However it is not expected that this big difference exist in bridge structures but often in buildings. This investigation, therefore, indicates that impact forces generated by pounding are incapable of causing large deformations of bridge columns.

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