PERFORMANCE OF REINFORCED CONCRETE BUILDINGS DURING SEISMIC POUNDING CONSIDERING SOIL-FOUNDATION INTERACTION

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1. Introduction

During earthquakes, adjacent buildings having different dynamic characteristics may vibrate out of phase and collide, if the separation between them is insufficient. Because of rapid increase in urban development and the associated increase in real-estate values, in the past, especially in urban areas many buildings were constructed even up to their property lines. This situation may lead to non-structural and structural damages to the buildings during seismic pounding. The location of impact and magnitude of impact force are highly influenced by the characteristics of input ground motion, geometric configurations and dynamic properties of buildings, soil parameters and gap between the adjacent buildings. Some of the building codes such as IBC 2003 have provided a clause for sufficient separation between adjacent buildings in order to avoid seismic pounding. However, the provision has been removed from IBC 2006. Due to constraints in availability of land and to fulfill functional requirements, adjacent buildings may also be constructed with different floor heights which will give rise to mid-column pounding. In most of the seismic pounding analyses the effects of underlying soil are ignored. The consideration of underlying soil adds extra degrees of freedom at the foundation level and also allows energy dissipation. Hence, it is necessary to include effects of soil on the seismic pounding analysis of buildings.

Jeng and Tzeng (2000) have reported five major types of poundings, viz., mid-column pounding, heavier adjacent building pounding, taller adjacent building pounding, eccentric building pounding and end building pounding. Anagnostopoulos (1988) simulated earthquake induced pounding between adjacent structures by using a spring-damper element where the damping constant is represented in terms of the coefficient of restitution. Jankowski (2005) used a non-linear viscoelastic model to perform more accurate simulations of structural pounding during earthquakes. The analysis results were compared with the results of experiments performed by van Mier et al. (1991) and the characteristics of concrete-to-concrete impact and steel-to-steel impact were also obtained.

Karayannis and Favvata (2005) studied the influence of structural pounding on the ductility requirements and seismic behavior of reinforced concrete structures with equal and non-equal heights, designed according to Eurocode 2 and Eurocode 8. Idealized models with a lumped mass system were considered using the program DRAIN-2DX for the analysis. A parametric study on eccentric pounding of two symmetric buildings conducted by Leibovich et al. (1996) showed the amplification in the response of the buildings due to impact eccentricity and that the effect is not proportional to impact eccentricity. Rahman et al. (2001) highlighted the influence of soil flexibility effects on seismic pounding for adjacent multi-story buildings of differing total heights, by using 2-D structural analysis software RUAUMOKO, for which the discrete model proposed by Mullikan and Karabalis (1998) was used. Soil-foundation interaction is incorporated through the discrete model in the present paper as well. The schematic diagram of the discrete model composed of mass, springs and dampers is shown in Fig. 1.

Three dimensional analysis of buildings causing different types of seismic poundings viz. mid-column pounding (MCP), row building pounding (RBP), mid-column pounding in a row (MCPR), mid-column pounding with heavier adjacent building (MCPHB), and mid-column pounding of eccentrically located buildings (MCPEB), including effects of underlying soil are considered in this research. Numerical results are presented in terms of top floor displacements, interstory displacements, impact forces and normalized story shear.

2. Pounding force and impact element

Either elastic or viscoelastic impact elements are

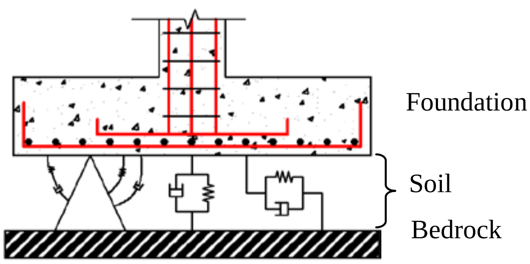


Fig. 1. Discrete model for soil-foundation interaction.

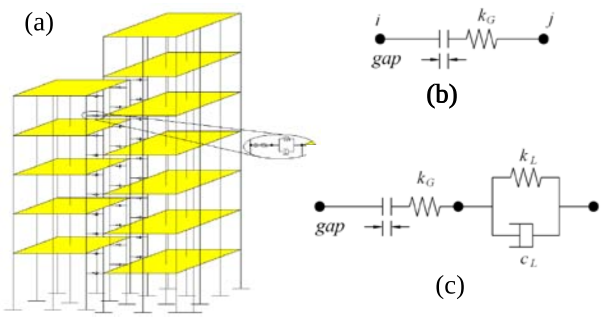


Fig. 2. (a) Buildings connected with impact elements, and (b) Gap element, and (c) Impact element composed of a gap element and a Kelvin-Voigt element.

often used to model the pounding between adjacent structures. To model impact between two colliding structures, the linear spring-damper (Kelvin-Voigt model) element is mostly used. The force in the linear viscoelastic model $F(t)$ during impact is given by

$$F(t) = k_L \delta(t) + c_L \dot{\delta}(t) , \qquad (1)$$

where, $\delta(t)$ is the relative displacement of colliding structural elements, $\dot{\delta}(t)$ is the relative velocity between colliding elements, k_L is the stiffness and c_L is the damping coefficient and is given by

$$c_L = -2 \ln e_r \sqrt{\frac{k_L m_1 m_2}{\left[\pi^2 + (\ln e_r)^2\right](m_1 + m_2)}}, \qquad (2)$$

where, e_r is the coefficient of restitution, m_1 and m_2 are masses of structural members (Anagnostopoulos (1988)). Numerical simulation performed by Jankowski (2005) showed that for concrete-to-concrete impact, $k_L = 93{,}500$ kN/m and $e_r = 0.65$ provides good correlation between experimental results provided by van Mier et al. (1991) and theoretical results. In the present study also, the same values of k_L and e_r are used.

Pounding between buildings is simulated using the impact elements (Fig. 2), which consist of a gap element shown in Fig. 2(b) and a Kelvin-Voigt element. The force transmits from one structure to another only when contact occurs. The force-deformation relationship of the gap element is given by

$$f_G = \begin{cases} k_G[(u_i - u_j) - gap] & if\ u_i - u_j > gap, \\ 0 & if\ u_i - u_j < gap, \end{cases} \qquad (3)$$

where, f_G is the force, k_G is the spring constant, u_i and u_j are the nodal displacements of nodes i and j and gap is the initial gap opening. The stiffness of gap element k_G is considered as 100 k_L to avoid errors in convergence and to ensure that it works nearly rigidly when the gap is closed.

3. Building description and design

Two different configurations of residential buildings are considered, Configuration A: 10-story and 9-story buildings and Configuration B: two 5-story buildings, where the floor plan is the same for both configurations are chosen for MCP. The story heights of the first floor of the 9-story building of Configuration A and the 5-story right building of Configuration B are 1.8 m while all other story heights are 3.6 m. For RBP case, three buildings in a row where a 8-story building is located between two 6-story exterior buildings, which are identical, having story height of 3.6m are considered. A 10-story building located between two identical 9-story buildings is considered for the case MCPR. A story height of 1.8 m is considered at the first floor of the 9-story buildings and rest of the story heights in all the buildings are 3.6 m. Two buildings, a 6-story building having story height 4.5 m at first floor and 3.0 m for the other floors and a 8-story building having story height of 3.0 m are considered for the cases MCPHB and MCPEB. A finite element analysis software SAP2000 is used to analyze the buildings considering 5% damping ratio. Unit weight $\gamma_c = 24\ \mathrm{kN/m^3}$, modulus of elasticity $E_c = 24{,}821\ \mathrm{N/mm^2}$, Poisson's ratio $\nu_c = 0.2$, and characteristic strength $f_c^{'} = 27\ \mathrm{N/mm^2}$ are assumed for concrete and the yield strength of reinforcing steel f_y is assumed to be $414\ \mathrm{kN/m^2}$. Considering live load of $2\ \mathrm{kN/m^2}$, roof load of $1\ \mathrm{kN/m^2}$ and partition load of $1\ \mathrm{kN/m^2}$, the structural components including foundations of the buildings are designed to fulfill the code requirements of ACI 318-02 for which earthquake loads are calculated according to IBC 2003. The buildings are assumed to be located in site class D (stiff soil), seismic use group II and seismic design category A. Square and combined footings are designed assuming silty gravel soil for which allowable soil bearing capacity is taken as $150\ \mathrm{kN/m^2}$.

4. Numerical results and conclusions

The underlying soil is modeled through the discrete model composed of mass, springs and dampers at the foundation level (Figure 1). Their coefficients are obtained by using equations (2.42a)-(2.44) and Tables 2-4 of Wolf (1988) (pg. 32-36), for which soil properties: unit weight $\gamma_s = 16.5\ \mathrm{kN/m^3}$, Poisson's ratio $\nu = 1/3$ and shear modulus $G = 18.75$ MPa are considered and designed footing dimensions with 1.5 m embedment are used. Time history analysis for two near-field earthquakes, 1994 Northridge (Sylmar County Hospital Parking Lot Station, N-S component, PGA = 0.843g,

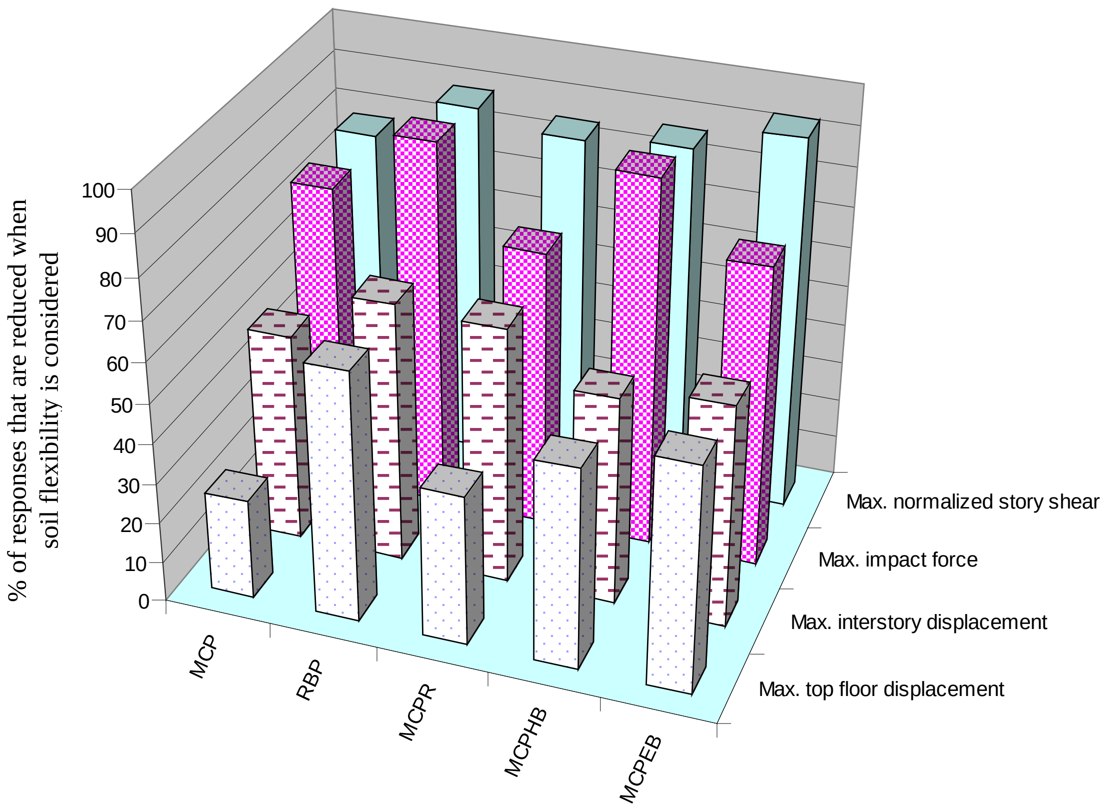
M_w = 6.7) and 1995 Kobe (0 KJMA Station, N-S component, PGA = 0.821g, M_w = 6.9) and two far-field earthquakes, 1940 El Centro (Imperial Valley Irrigation Station, N-S component, PGA = 0.298g, M_w = 7.0) and 1968 Hachinohe (Hachinohe City Station, N-S component, PGA = 0.229g, M_w = 7.9) are conducted using Newmark method with $\beta = 0.25$, $\gamma = 0.5$ and time step $\Delta t = 0.002$ sec.

From Table 1, it can be observed that when soil

Table 1. Response reduction due to soil flexibility

Pounding type/case	Max. impact force	Max. normalized story shear	Max. interstory displacement	Max. top floor displacement
MCP	1 - 74%	24 - 54%	5 - 43%	3 - 34%
RBP	5 - 100%	2 - 40%	5 - 44%	2 - 24%
MCPR	1 - 80%	1 - 58%	1 - 81%	10 - 58%
MCPHB	1 - 66 %	1 - 78%	1 - 45%	14 - 29%
MCPEB	3 - 100%	4 - 61%	1 - 68%	8 - 24%

MCP	–	Mid-column pounding
RBP	–	Row building pounding
MCPR	–	Mid-column pounding in a row
MCPHB	–	Mid-column pounding with heavy adjacent building
MCPEB	–	Mid-column pounding of eccentrically located buildings



MCP	–	Mid-column pounding
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Fig. 3. Percentage of (a) maximum normalized story shear, (b) maximum impact force, (c) maximum interstory displacement, (d) maximum top floor displacement that are reduced when soil flexibility is considered.

flexibility is considered, maximum impact forces are reduced by 5-100% in row building pounding and 3-100% in mid-column pounding of eccentrically located buildings case. 100% reduction indicates that there is no pounding between the buildings when foundation with soil flexibility is provided. Similarly, consideration of soil flexibility reduced maximum normalized story shear by 1-78% in mid-column pounding with heavy adjacent building, maximum interstory displacement by 1-81% and maximum top floor displacement by 10-58% in mid-column pounding in a row.

Figure 3 shows the summary of 5 different types of pounding considered in this study, in terms of percentage of maximum normalized story shear, maximum impact force, maximum interstory displacement and maximum top floor displacement that are reduced when soil flexibility is considered. For all pounding types, maximum normalized story shear forces are highly reduced. Around 90% of maximum normalized story shear forces are reduced in mid-column pounding of eccentrically located buildings case. Similarly, for row building pounding and mid-column pounding with heavy adjacent building cases, around 90% of maximum impact forces are reduced. However, similar trend of reduction is not observed in the cases of maximum interstory displacement and maximum top floor displacement. Only around 60% of the maximum interstory displacements are reduced when flexible soil is considered. In mid-column pounding and mid-column pounding in a row cases, even below 50% of maximum top floor displacements are reduced upon consideration of soil effects. The comparison of results for each case is mainly focused on the effect of soil on the response of buildings.

Therefore, consideration of soil flexibility is effective in reducing maximum impact forces and maximum normalized story shear while it is less effective in reducing maximum interstory displacements and maximum top floor displacements.

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