MID-COLUMN SEISMIC POUNDING OF REINFORCED CONCRETE BUILDINGS IN A ROW CONSIDERING EFFECTS OF SOIL

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ABSTRACT:

Seismic pounding between adjacent non-identical buildings may occur during earthquakes, if the separation between them is insufficient. This paper deals with the seismic pounding between three typical reinforced concrete moment resisting frame buildings in a row, where a 10-story building is located between two identical 9-story buildings, considering the effects of underlying soil on the structural response. The story height of 10-story building is different from those of the two 9-story buildings and this gives rise to mid-column pounding. A finite element analysis software SAP2000 is used to analyze the buildings. The structural components including foundations of the buildings are designed to fulfill the code requirements of ACI 318-02 and IBC 2003. The underlying soil is taken into account through the discrete model at the foundation level and the contacts between the buildings are incorporated through impact elements consisting of a gap element and a Kelvin-Voigt element. Two far-field earthquakes and two near-field earthquakes are used as input motions to investigate the response of buildings. Interstory displacements, impact forces and normalized story shear are computed to evaluate the performance of the buildings. In general, the results of analyses show that the interstory displacements, impact forces and normalized story shear are reduced when underlying soil is considered.

KEYWORDS: impact force, interstory displacement, mid-column pounding, normalized story shear, soil effects.

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 and may also give rise to total collapse of 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; 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. 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-structure interaction is incorporated through the discrete model in the present paper as well. The schematic diagram of the discrete model composed of mass-spring-damper is shown in Fig. 1.

The purpose of this paper is to investigate the effects of soil on mid-column seismic pounding of reinforced concrete buildings in a row, for which the discrete model is used to incorporate soil-foundation interaction. Impact forces, interstory displacements and normalized story shear of the buildings are considered as the parameters to investigate the effects of soil on seismic pounding.

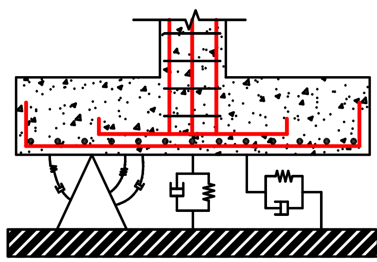

Figure 1 Discrete model for soil-foundation interaction

2. POUNDING FORCE AND IMPACT ELEMENT

Elastic or viscoelastic impact elements are often used to model pounding between adjacent structures, however, Kelvin-Voigt element (i.e. a linear spring-damper element) is mostly used to model impact between two colliding structures. The viscous component of the Kelvin-Voigt element dissipates energy throughout the approach and restitution period, but in reality, most of the energy dissipation takes place during the approach period and minor energy dissipation is observed during restitution period. However, for simplicity, to simulate structural pounding the Kelvin-Voigt element has been widely used. The force in the Kelvin-Voigt element $F(t)$ during impact is given by

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

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

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

where, e_r is the coefficient of restitution, m_1 and m_2 are masses of structural members (Anagnostopoulos (1988)). The 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 addition, Anagnostopoulos (1988), Azevedo and Bento (1996), Mouzakis and Papadrakakis (2004) and Jankowski (2006) have also used $e_r = 0.65$ for concrete to concrete impact. In the present study also $k_L = 93{,}500$ kN/m and $e_r = 0.65$ are used.

To simulate contact of buildings and pounding force, the impact elements are inserted between buildings as shown in Fig. 2(a). The impact element shown in Fig. 2(c) is created by combining a gap element shown in Fig. 2(b) with 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

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

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 error in convergence and to ensure that it works nearly rigidly when the gap is closed.

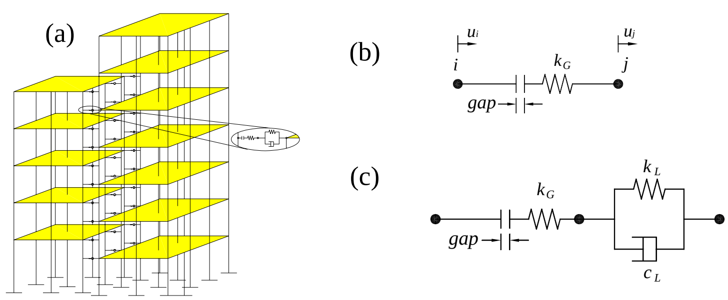

Figure 2 (a) Buildings connected with impact elements; (b) Gap element; and (c) Impact element composed of gap element and Kelvin-Voigt element

3. DESCRIPTION OF BUILDINGS AND DESIGN

For the analysis, a 10-story building located between two identical 9-story buildings is considered (Fig. 3). 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, which gives rise to mid-column pounding during earthquakes. A finite element analysis software SAP2000 is used to analyze the buildings considering 5% damping ratio. Unit weight $\gamma_c = 24$ kN/m^3, modulus of elasticity $E_c = 24821$ N/mm^2, Poisson's ratio $\nu_c = 0.2$, and characteristic strength $f'_c = 27$ N/mm^2 are assumed for concrete and the yield strength of reinforcing steel f_y is assumed to be 414 N/mm^2. Considering live load of 2 kN/m^2, roof load of 1 kN/m^2 and partition load of 1 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 seismic class D, seismic use group II and seismic design category A. The buildings are provided with 150 mm thick slab and 350 mm x 600 mm beams. The layout of the foundations is shown in Fig. 3(b) and the dimensions are given in Appendix A, Table A1.

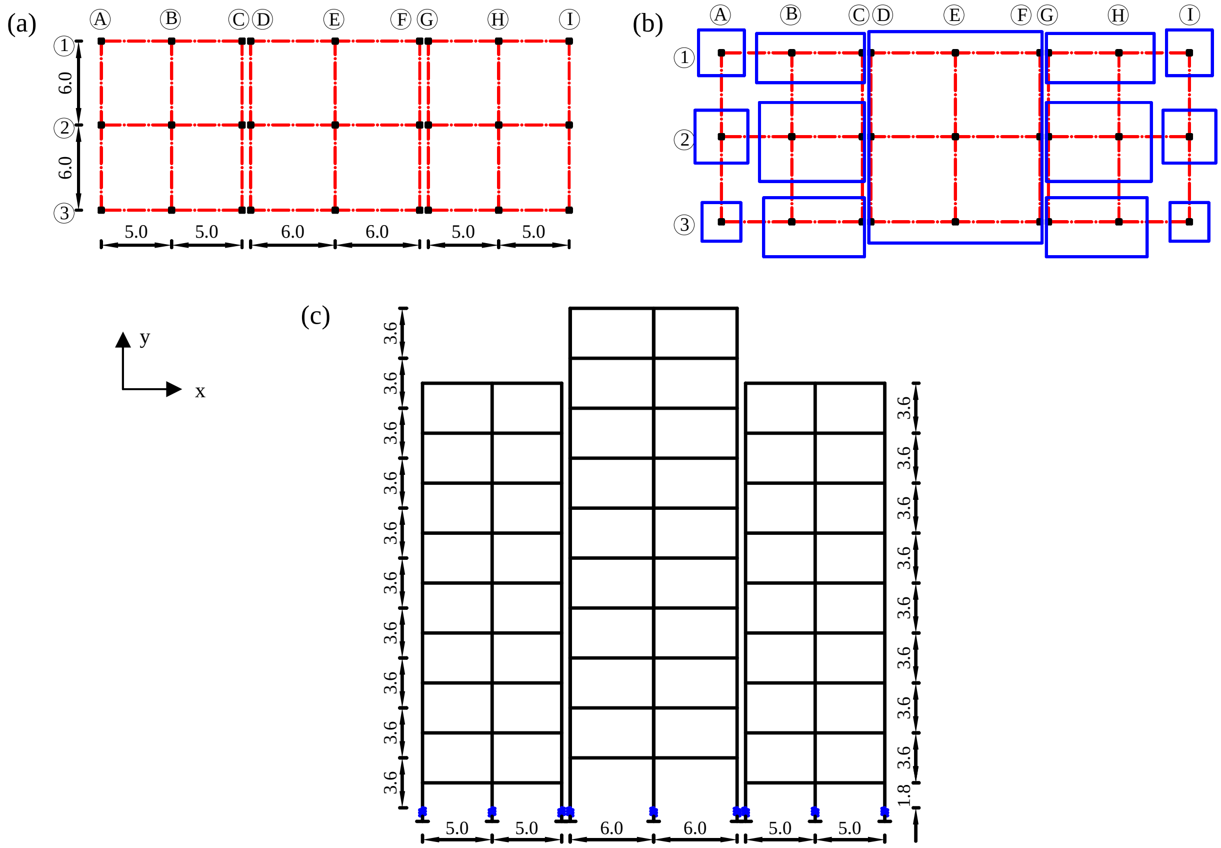

Figure 3 (a) Floor plan, (b) Foundation layout of buildings, (c) Building elevation

4. NUMERICAL ANALYSIS AND RESULTS

The underlying soil is modeled through the discrete model composed of mass-spring-damper at the foundation level (Fig. 2). Their coefficients are obtained using Eqns. (B1)-(B8) and Tables B1 and B2, given in Appendix B, for which soil properties: unit weight $\gamma_s = 16.5\ \text{kN/m}^3$, Poisson's ratio $\nu = 1/3$ and shear modulus $G = 18.75\ \text{MPa}$ are considered and footing dimensions are taken from Appendix A, Table A1 with 1.5 m embedment. The gaps between the buildings are considered as 5 mm. Two far-field earthquakes, 1940 El Centro and 1968 Hachinohe and two near-field earthquakes, 1994 Northridge and 1995 Kobe are used as earthquake inputs along *x*-direction. Newmark method with $\beta = 0.25$, $\gamma = 0.5$ and time step $\Delta t = 0.005$ sec is adopted for time history analysis of buildings.
Interstory displacements of the buildings at columns C2, D2 and G2, for fixed foundation-no pounding, fixed foundation-with pounding and flexible foundation-with pounding cases are shown in Fig. 4. The maximum interstory displacements are observed in all the buildings when there is no seismic pounding. When there is pounding and flexible foundations are considered, compared to fixed foundation cases, interstory displacements are reduced for El Centro, Hachinohe and Kobe earthquakes, however, interstory displacements are increased in the case of Northridge earthquake (Figs. 4(d)-(i)). Figure 5 shows that collision between the buildings, columns C2 and G2 occurs at different time with different magnitudes. Higher magnitude of impact forces are observed in the case of near-field earthquakes. Kobe earthquake has the dominant effects on the buildings as the impact forces due to this earthquake are the largest (Figs. 5(a), 5(c), 5(d)).

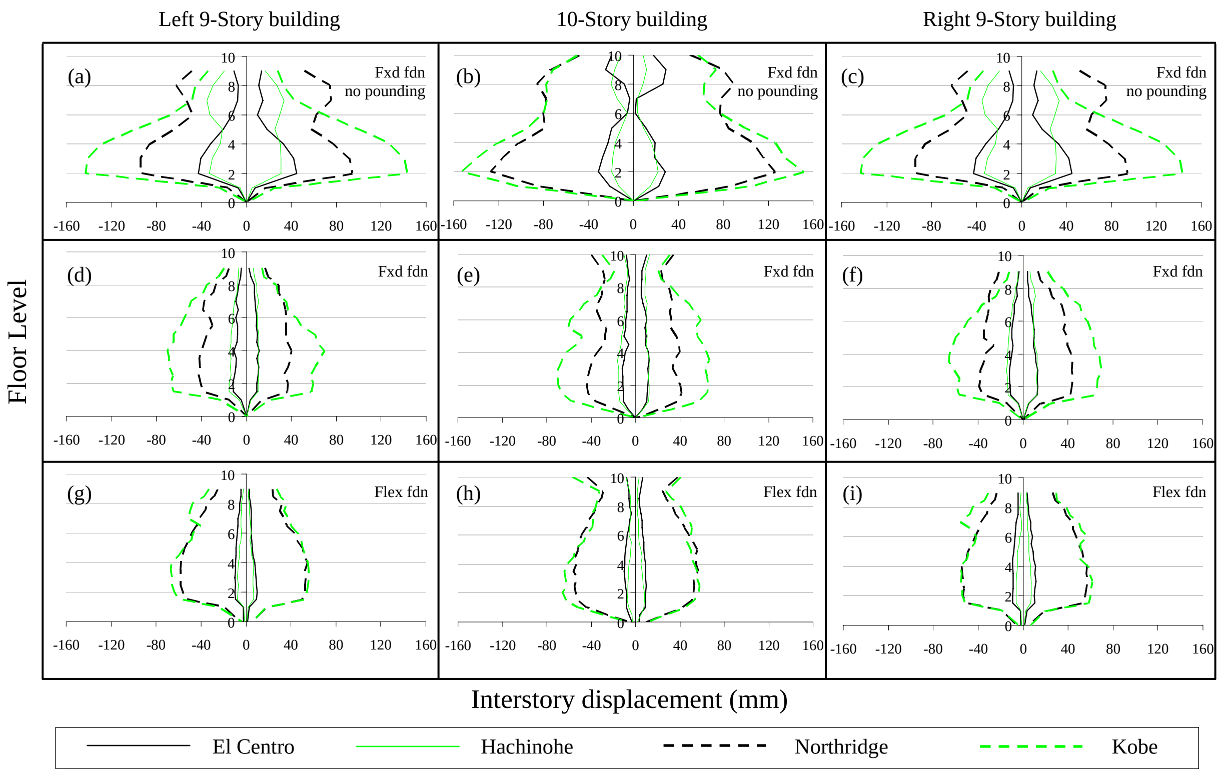

Figure 4 Interstory displacement

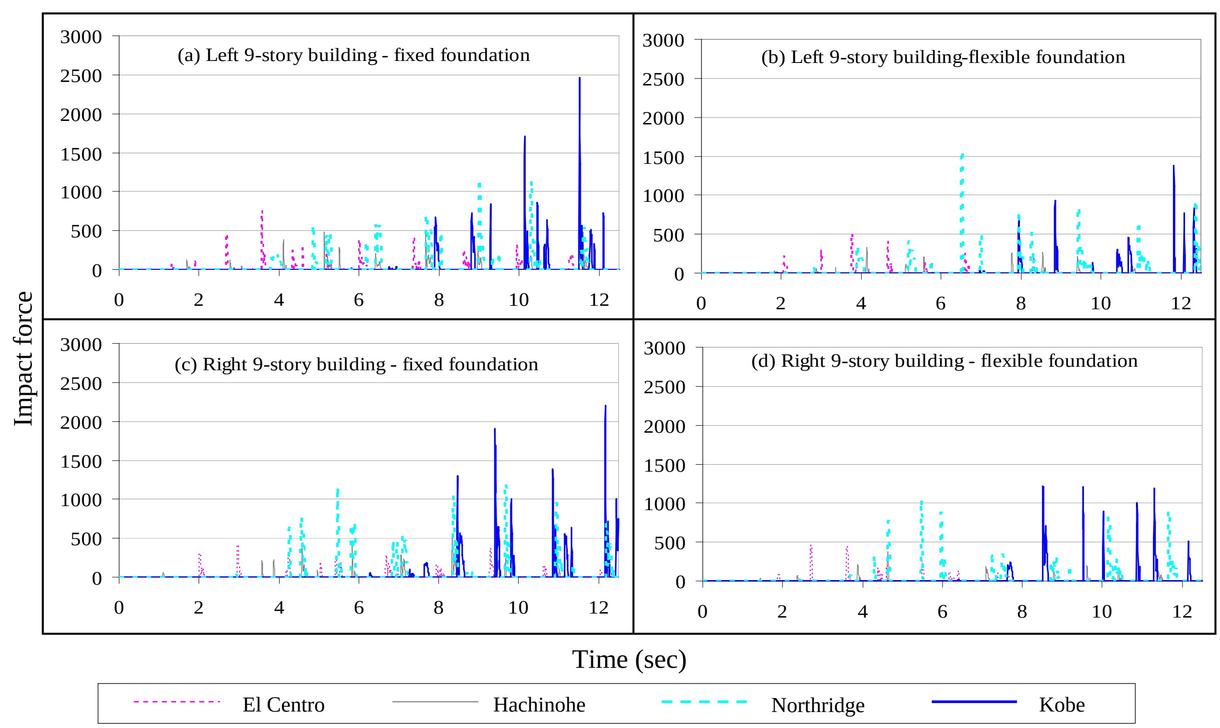

Figure 5 Impact force time history at 9-story building top floor center columns, C2 and G2

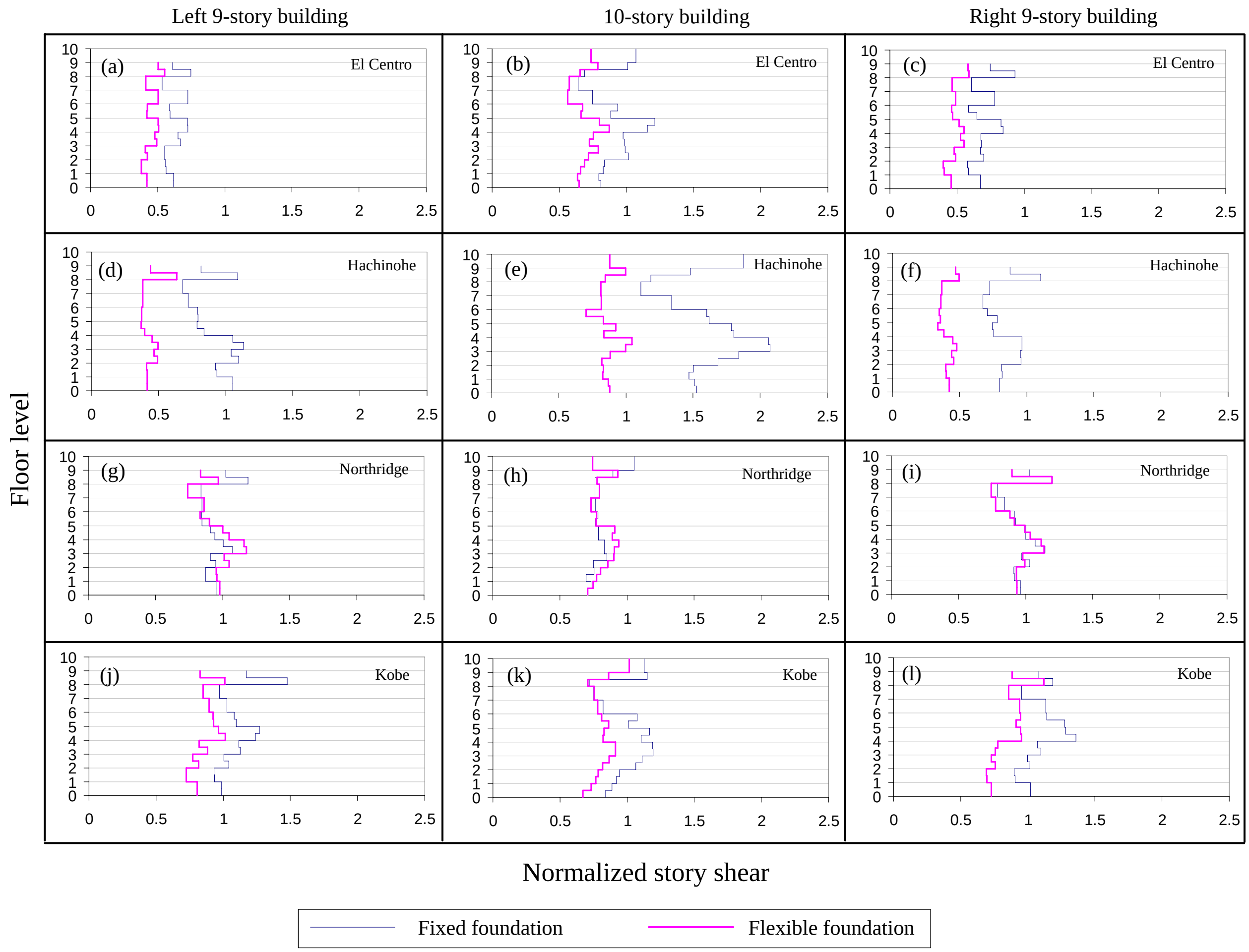

Figure 6 Normalized story shear (a)-(c) El Centro; (d)-(e) Hachinohe; (g)-(i) Northridge; and (j)-(l) Kobe

Normalized story shear, defined as the ratio of maximum story shear resulting from pounding to that of maximum story shear for no pounding case with fixed foundation, are used to express the response of the buildings and are shown in Fig. 6. It can be observed that normalized story shear of the buildings with flexible foundations are less than those of fixed foundations except for Northridge earthquake. Although, at some stories of the buildings under Northridge earthquake, normalized story shear for flexible foundation cases are higher than that for fixed foundation cases, the maximum normalized story shears are observed in the buildings with fixed foundations (Figs. 6(g)-6(l)). Among all of the buildings, the maximum normalized story shear is observed in 10-story building under Hachinohe earthquake. In general, the buildings under consideration are more vulnerable to near-field earthquakes and the consideration of effects of underlying soil is beneficial as the impact forces and peak shear amplification factors are reduced.

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Appendix A:

Table A1 Footing description

<table>
<tr><th>Grid</th><th>A/I</th><th>B/G</th><th>C/H</th><th>D</th><th>E</th><th>F</th></tr>
<tr><td>1</td><td>3.25 x 3.25 x 0.41</td><td colspan="2">7.50 x 3.70 x 0.75</td><td colspan="3" rowspan="3">12.50 x 15.00 x 0.75</td></tr>
<tr><td>2</td><td>3.75 x 3.75 x 0.47</td><td colspan="2">6.30 x 5.60 x 0.75</td></tr>
<tr><td>3</td><td>2.75 x 2.75 x 0.35</td><td colspan="2">7.00 x 4.20 x 0.60</td></tr>
</table>

(All dimensions are in m)

Appendix B:

The coefficients for rectangular footings can be calculated by using Eqns. (B1)-(B8) and Table B1 (Wolf (1988)).

$$K_{hx} = Gb/(2-\nu)*[6.8(l/b)^{0.65} + 2.4][1+\{0.33+1.34/(1+l/b)\}(e/b)^{0.8}] \quad \text{(B1)}$$

$$K_{hy} = Gb/(2-\nu)*[6.8(l/b)^{0.65} + 0.8l/b + 1.6][1+\{0.33+1.34/(1+l/b)\}(e/b)^{0.8}] \quad \text{(B2)}$$

$$K_{v} = Gb/(1-\nu)*[3.1(l/b)^{0.75} + 1.6][1+(0.25+0.25b/l)\}(e/b)^{0.8}] \quad \text{(B3)}$$

$$K_{rx} = Gb^3/(1-\nu)*[3.2l/b + 0.8][1+e/b+\{1.6/(0.35+l/b)\}(e/b)^{2}] \quad \text{(B4)}$$

$$K_{ry} = Gb^3/(1-\nu)*[3.73(l/b)^{2.4} + 0.27][1+e/b+\{1.6/(0.35+(l/b)^4)\}(e/b)^{2}] \quad \text{(B5)}$$

$$K_{t} = Gb^3*[4.25(l/b)^{2.45} + 4.06][1+(1.3+1.32b/l)(e/b)^{0.9}] \quad \text{(B6)}$$

$$K_{hxry} = e/3*K_{hx}, \qquad K_{hyrx} = e/3*K_{hy}, \qquad K = 4G/b \quad \text{(B7)}$$

$$C_0 = b/V_s*K\gamma_0, \qquad C_1 = b/V_s*K\gamma_1, \qquad M_0 = b^2/V_s^2*K\mu_0, \qquad M_1 = b^2/V_s^2*K\mu_1 \quad \text{(B8)}$$

where, G is shear modulus of soil, ν is Poisson's ratio of soil, $V_s = \sqrt{G/\rho}$ is shear wave velocity, $2l$ and $2b$ are length and breadth of rectangular footing, e is embedment, and ρ is soil density.

Table B1 Dimensionless coefficients of discrete model for rectangular foundation (Wolf (1988))

	Dampers		Masses	
	γ_0	γ_1	μ_0	μ_1
Horizontal h_x h_y	$0.75+0.2\left(l/b-1\right)$	-	-	-
Vertical	$0.9+0.4\left(l/b-1\right)^{\frac{2}{3}}$	0.3	-	0.14
Rocking r_x	-	0.45	-	0.34
r_y	-	$0.45+0.23\left(l/b-1\right)$	-	$0.34+0.55\left(l/b-1\right)$
Torsional	-	$0.35+0.12\left(l/b-1\right)$	-	$0.28+0.63\left(l/b-1\right)$

Table B2 Coefficients for 1-D discrete element model for square footing (Mullikan and Karabalis (1998))

Motion	Horizontal	Vertical	Rocking
Mass (inertia) ratio, β	$\frac{m(7-8\nu)}{32\rho r_0^3(1-\nu)}$	$\frac{m(1-\nu)}{4\rho r_0^3}$	$\frac{3m(1-\nu)}{8\rho r_0^5}$
Equivalent radius, r_0	$2a/\sqrt{\pi}$	$2a/\sqrt{\pi}$	$2a/\sqrt[4]{3\pi}$
Virtual soil mass (inertia), m_v	$0.095m/\beta$	$0.27m/\beta$	$0.24m/\beta$
Discrete element model, K_s (static stiffness) and C (damping)	$K_s = 9.2Ga/(2-\nu)$ $C = 0.163Ka/V_s$	$K_s = 4.7Ga/(1-\nu)$ $C = 0.8Ka/V_s$	$K_s = 4.0Ga^3/(1-\nu)$ $C = 0.6Ka/V_s$

where, $2a$ is side of square footing and m is mass of foundation.