SEISMIC POUNDING ANALYSIS OF MULTI-STORY REINFORCED CONCRETE BUILDINGS CONSIDERING EFFECTS OF UNDERLYING SOIL

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1. Introduction

Adjacent buildings having different dynamic characteristics may vibrate out of phase and collide during earthquakes, if the separation between them is insufficient. In the past, especially in urban areas, many buildings were constructed even up to their property lines because of rapid increase in urban development and the associated increase in real-estate values. This may cause non-structural and structural damage to the buildings and may also lead to total collapse of buildings during seismic pounding. The characteristics of input ground motion, geometric configurations and dynamic characteristics of buildings, soil parameters and gap between the adjacent buildings highly influences the location and magnitude of impact. To avoid seismic pounding, some of the building codes such as IBC 2003 have provided a clause to make a provision of sufficient separation between adjacent buildings. However, this clause has not been included in IBC 2006. Due to constraints in availability of land and in order to fulfill functional requirements, adjacent buildings may also be constructed eccentrically, with different floor heights and this may give rise to eccentric and mid-column pounding. Most of the seismic pounding analyses are performed without considering the effects of underlying soil. The consideration of underlying soil leads to an increase in 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. Five major types of poundings; mid-column pounding, heavier adjacent building pounding, taller adjacent building pounding, eccentric building pounding and end building pounding have been reported by Jeng and Tzeng (2000)

Anagnostopoulos (1988) used the spring-damper element in order to simulate earthquake induced pounding between adjacent structures representing the damping constant in terms of the coefficient of restitution. Furthermore, the non-linear viscoelastic model was implemented by Jankowski (2005) for more accurate simulation of structural pounding during earthquakes. The analysis results were compared with the results of experiments performed by van Mier et al. (1991) through which the characteristics of concrete-to-concrete impact and steel-to-steel impact were also obtained.

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. In the present paper too, soil-structure interaction is incorporated through the discrete model. The schematic diagram of the mass-spring-damper is shown in Fig. 1.

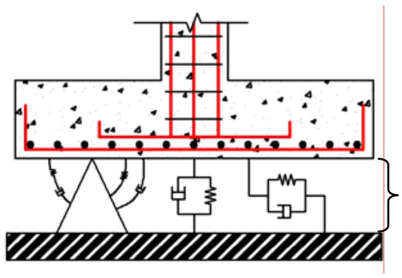


Fig. 1. Discrete model for soil-foundation interaction.

The mass-spring-damper properties are calculated using Equations 2.42a-2.44 and Tables 2-4 of Wolf (1988).

2. Pounding force and impact element

Either elastic or viscoelastic impact elements are 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

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

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.

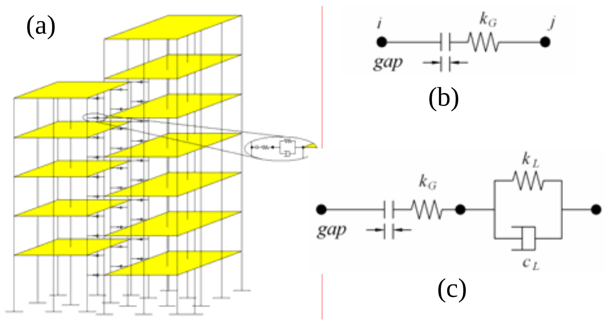


Fig. 2. (a) Buildings connected with impact elements, and (b) Gap element, and (c) Impact element composed of a gap element and a Kelvin-Voigt element.

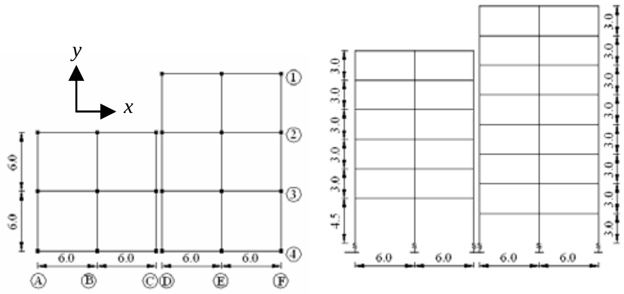
Pounding between buildings is simulated using the impact elements (Fig. 2), which consist of a gap element shown in Fig. 2(b) and a Kelvin-Voigt element. When the gap in the gap element closes, the force transmits from one structure to another. The force-deformation relationship of the gap element is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

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

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 residential 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, located eccentrically as shown in Fig. 3 are considered for the analysis. Concrete with compressive strength f_c' = 27 N/mm^2, unit weight γ_c = 24 kN/mm^3, modulus of elasticity E_c = 24,281 N/mm^2, and Poisson's ratio ν_c = 0.2 and reinforcing steel with yield strength f_y = 414 N/mm^2 are used for analysis and design. Live load of 2 kN/m^2, roof load of 1 kN/m^2 and partition wall load of 1 kN/m^2 are considered and lateral loads are calculated as per IBC 2003 for which site class D, seismic use group II and seismic design category A are considered. The software SAP2000 is used to analyze the buildings considering 5% damping ratio and the structural components including foundations of the buildings are designed to fulfill the code requirements of ACI 318-02. The buildings are provided with 180 mm thick floor slabs which are considered as rigid floor diaphragms and 300 mm x 500 mm beams. The dimensions of footings and column sizes and steel reinforcements are shown in Tables A1 and A2. The arrangement of footings is shown in Fig. 4.



(a) (b)

Fig. 3. 6-story and 8-story buildings: (a) Plan, and (b) Elevation.

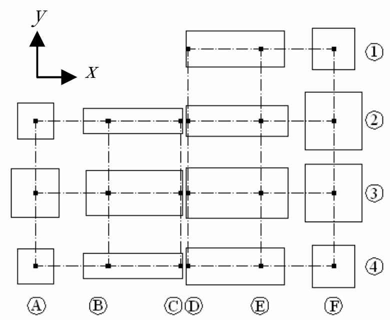


Fig. 4. Foundation arrangement plan.

4. Numerical analysis and results

The designed footing size with 1.5 m embedment and soil properties: density $\rho_s = 16.5$ kN/m^3, Poisson's ratio $\nu = 1/3$ and shear modulus $G = 18.75$ N/mm^2 are used to calculate the coefficients of frequency independent mass-spring-dampers at each footing. 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 considering 50 mm gap between the buildings using Newmark method with $\beta = 0.25$, $\gamma = 0.5$ and time step $\Delta t = 0.002$ sec.

The different magnitudes of impact forces at different times at roof level of 6-story building, column C3, is shown in Fig. 5 where it can be seen that the Kobe earthquake has the dominant effect on both fixed foundation case and flexible foundation case. When a flexible foundation is considered, the reduction in the maximum impact forces at each floor level of 6-story building, column C3 (Fig. 6) can be clearly seen, however, impact force at the 6th floor is significantly increased in the case of Kobe and Hachinohe earthquakes.

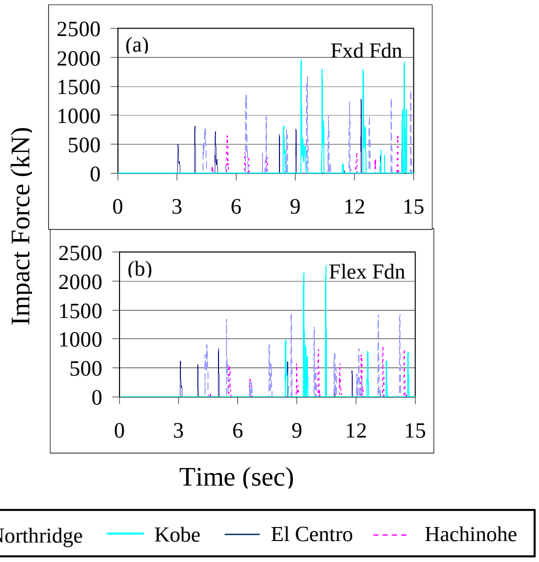


Fig. 5. Impact force time history at roof level of 6-story building column C3: (a) Fixed foundation; and (b) Flexible foundation.

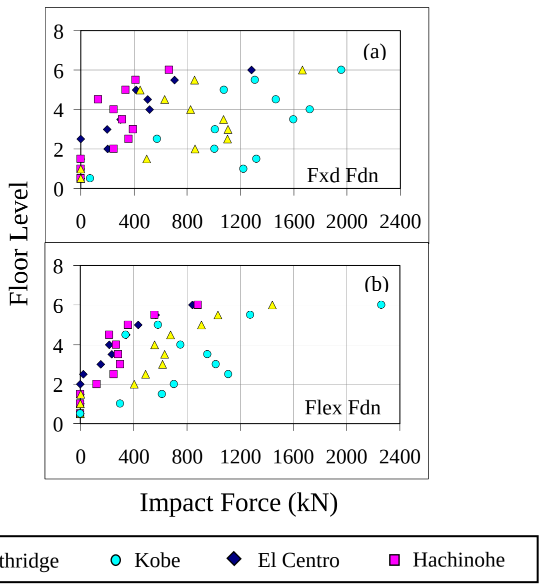


Fig. 6. Maximum impact force at 6-story building column C3: (a) Fixed foundation; and (b) Flexible foundation.

Figure 7 shows the interstory displacements of the buildings for fixed foundation with no pounding, fixed foundation with pounding and flexible foundation with pounding cases. The maximum interstory displacements are observed in no pounding case and the minimum interstory displacements are observed when soil effects are considered. The maximum interstory displacement in 8-story building without pounding (Fig. 7(a)) is due to Northridge earthquake, however, in rest of the cases the maximum interstory displacement is due to the Kobe earthquake. Hence, for both buildings, the near-field earthquakes have a major effect on the interstory displacements.

The responses of the buildings are also expressed in terms of normalized story shear, obtained after normalizing the story shear of the buildings due to pounding by the story shear for fixed foundation no pounding case, and are shown in Fig. 8. The normalized story shear at each floor is higher for fixed foundation case due to considered earthquakes except Hachinohe earthquake (Figs 8(g), (h)). However, the maximum normalized story shear for Hachinohe earthquake is also observed in the case of fixed foundation. The results indicate that in terms of normalized story shear, Kobe earthquake is dominant for fixed foundation case and Hachinohe earthquake is dominant for flexible foundation case.

5. Concluding remarks

This paper shows the importance of considering underlying soil in the study of seismic pounding. Two near-field and two far-field earthquakes are used to analyze two eccentrically located buildings with non-equal story heights which give rise to mid-column pounding. Reduction in impact forces are observed upon considering soil effects and the maximum interstory displacements are observed when there is no pounding. Normalized story shears are also reduced when underlying soil is taken into account. In general, near-field earthquakes have a major effect on the buildings under consideration and soil effects can modify the building response to a great extent.

Different pounding cases considering underlying soil effects, such as row building pounding, mid-column pounding of buildings in a row, mid-column pounding of taller adjacent building, mid-column pounding of eccentrically located buildings have also been studied by the authors. However, only mid-column pounding of eccentrically located reinforced concrete buildings has been presented in this paper and detail of others can be found elsewhere.

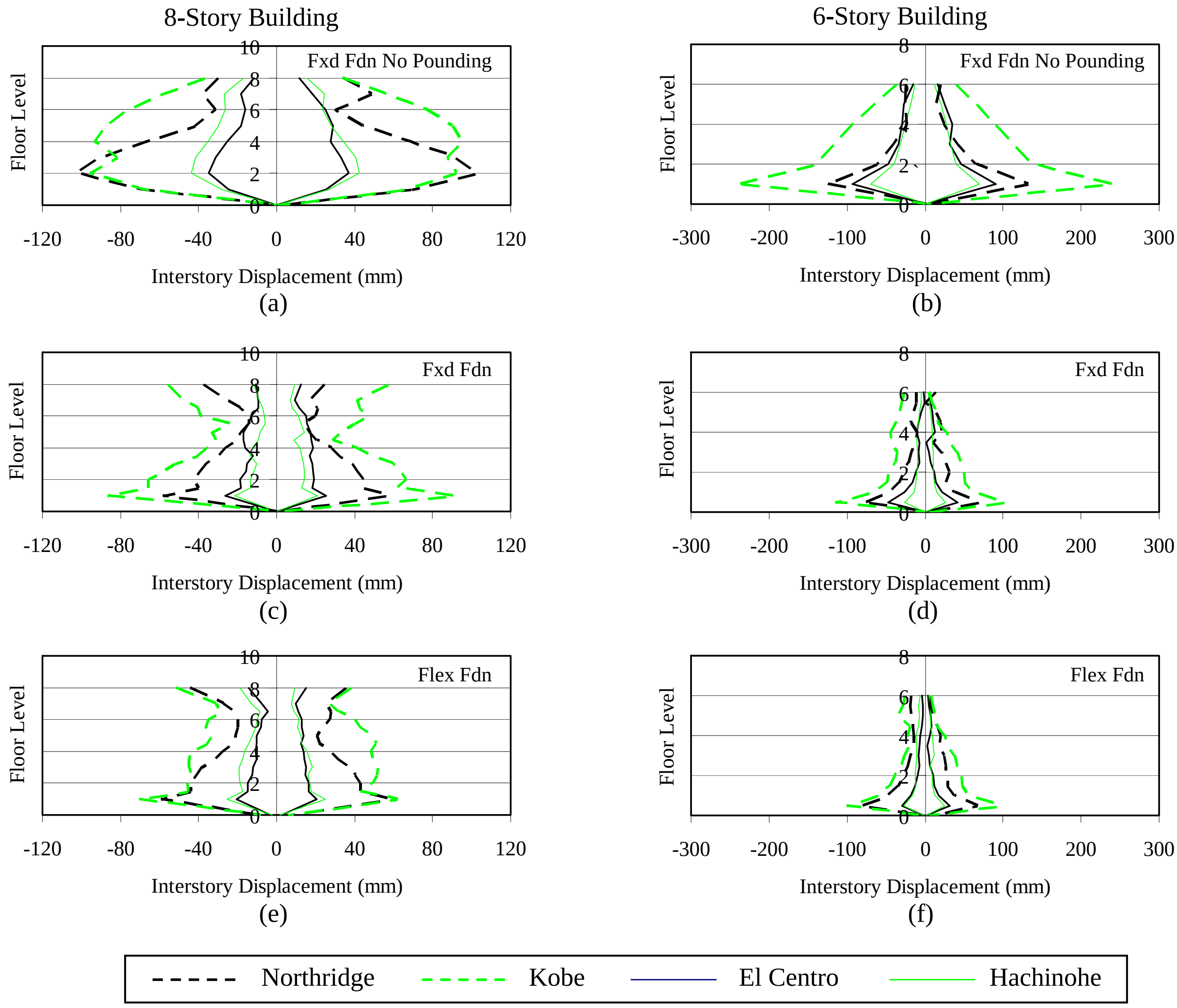


Fig. 7. Maximum interstory displacement of columns C3 and D3: (a) 8-story building, fixed foundation without pounding; (b) 6-story building, fixed foundation without pounding; (c) 8-story building, fixed foundation; (d) 6-story building, fixed foundation; (e) 8-story building, flexible foundation; and (f) 6-story building, flexible foundation.

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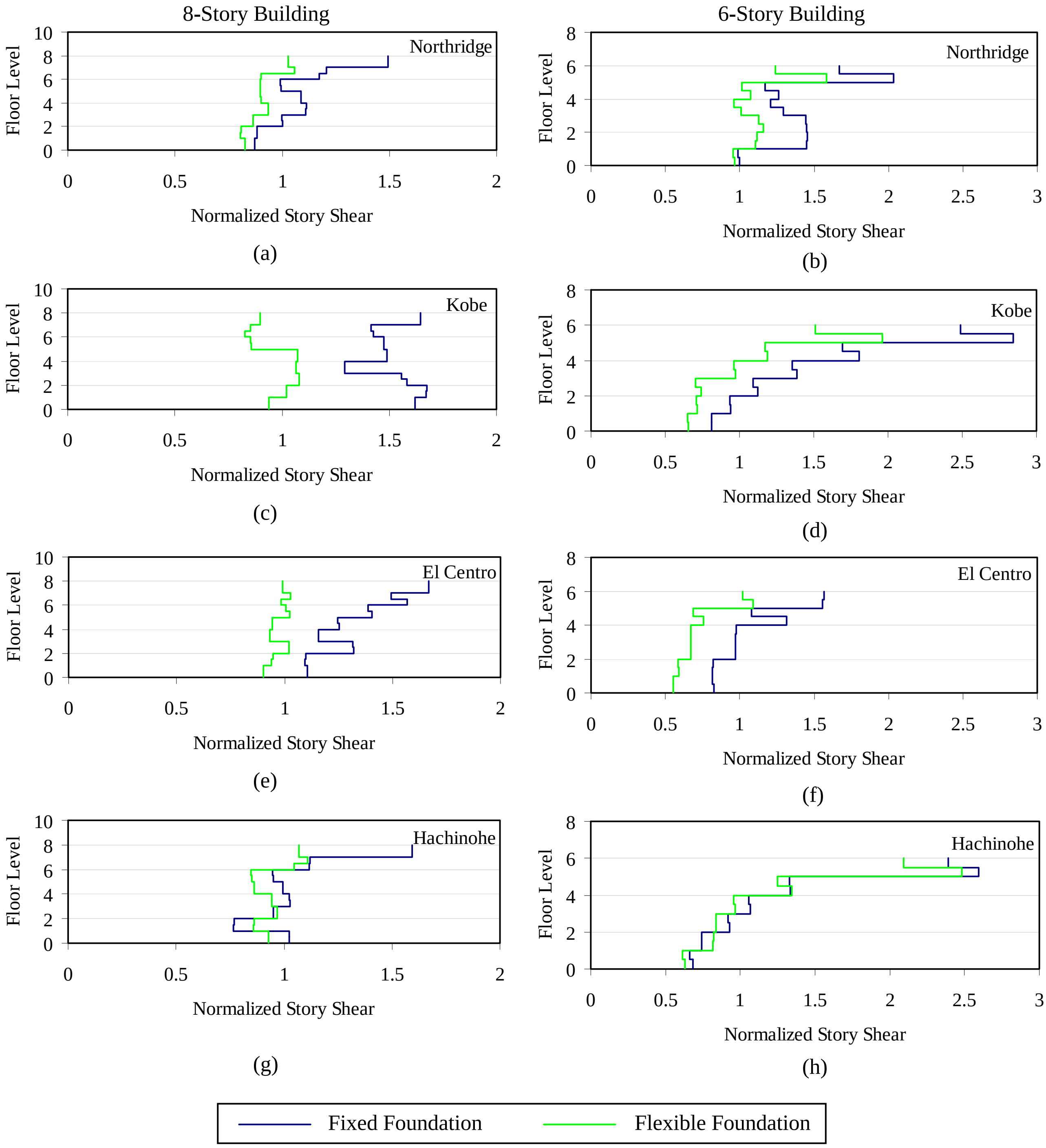

Fig. 8. Normalized story shear: (a), (b) Northridge; (c), (d) Kobe; (e), (f) El Centro; and (g), (h) Hachinohe earthquakes.

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Appendix

Table A1. Footing details.

Grid	A	B	C	D	E	F
1				8.15 x 3.00 x 0.50		3.50 x 3.50 x 0.45
2	3.00 x 3.00 x 0.40	8.20 x 2.10 x 0.40		8.45 x 4.20 x 0.60		4.75 x 4.75 x 0.60
3	4.00 x 4.00 x 0.50	8.00 x 3.70 x 0.60		8.45 x 4.20 x 0.60		4.75 x 4.75 x 0.60
4	3.00 x 3.00 x 0.40	8.20 x 2.10 x 0.40		8.15 x 3.00 x 0.50		3.50 x 3.50 x 0.45

(All dimensions are in m)

Table A2. Column size and main steel reinforcement.

Floor	Grid	A/C	B	D/F	E
1	1			C1 - 4-25Ø + 4-16Ø	C2 - 4-28Ø + 8-20Ø
2				C1 - 4-25Ø + 4-16Ø	C2 - 4-25Ø + 8-20Ø
3				C1 - 4-20Ø + 4-16Ø	C2 - 4-25Ø + 8-16Ø
4-8				C1 - 8-16Ø	16Ø
1	2	C1 - 8-16Ø	C2 - 4-25Ø + 8-16Ø	C2 - 8-25Ø + 4-20Ø	C4 - 8-28Ø + 4-20Ø
2		C1 - 8-16Ø	C2 - 12-16Ø	C2 - 4-25Ø + 8-20Ø	C4 - 4-28Ø + 8-20Ø
3		C1 - 8-16Ø	C2 - 12-16Ø	C2 - 4-20Ø + 8-16Ø	C4 - 12-20Ø
4-6		C1 - 8-16Ø	16Ø	C2 - 8-16Ø	20Ø
7-8				C2 - 8-16Ø	20Ø
1	3	C2 - 8-20Ø	C3 - 12-28Ø	C2 - 8-25Ø + 4-20Ø	C4 - 8-28Ø + 4-20Ø
2		C2 - 8-20Ø	C3 - 8-28Ø + 4-16Ø	C2 - 4-25Ø + 8-20Ø	C4 - 4-28Ø + 8-20Ø
3		C2 - 8-20Ø	C3 - 4-28Ø + 8-16Ø	C2 - 4-20Ø + 8-16Ø	C4 - 12-20Ø
4-6		C2 - 8-20Ø	16Ø	C2 - 8-16Ø	20Ø
7-8				C2 - 8-16Ø	20Ø
1	4	C1 - 8-16Ø	C2 - 4-25Ø + 8-16Ø	C1 - 4-25Ø + 4-16Ø	C2 - 4-28Ø + 8-20Ø
2		C1 - 8-16Ø	C2 - 12-16Ø	C1 - 4-25Ø + 4-16Ø	C2 - 4-25Ø + 8-20Ø
3		C1 - 8-16Ø	C2 - 12-16Ø	C1-25Ø + 4 16Ø	C2 - 4-25Ø + 8-16Ø
4-6		C1 - 8-16Ø	16Ø	C1 - 8-16Ø	16Ø
7-8				C1 - 8-16Ø	16Ø

C1: 360 mm x 360 mm
C2: 450 mm x 450 mm
C3: 500 mm x 500 mm