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Influence of incident angles of earthquakes on inelastic responses of asymmetric-plan structures

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Abstract. This paper presents the influence of incident angles of earthquakes on inelastic dynamic responses of asymmetry single story buildings under seismic ground motions. The dynamic responses such as internal forces and rotational ductility factor are used to evaluate the importance of the incident angles of ground motions in the inelastic range of structural behavior. The base shear and torque (BST) response histories of the resisting elements and of the building are used to prove that the shape of the BST surface of the building can be a practical tool to represent those of all resisting elements. This paper also shows that the different global forces which produce the maximum demands in the resisting elements tend to converge toward a single distribution in a definable intensity range, and this single distribution is related to the resistance distribution of the building.

Keywords: incremental dynamic analysis (IDA); angle of incidence; inelastic dynamic responses; rotational ductility factor; asymmetric-plan

1. Introduction

In recent years, innovative ideas about asymmetric-plan structures have been suggested. De La Llera and Chopra (1995) introduced base shear and torque (BST) surface by using the set of base shear (V) and torque (T) combinations corresponding to the different collapse mechanisms of the building under earthquakes, and their studies show that the in-plan strength distribution and inelastic behavior of the asymmetric single story building under seismic motions can be synthetically represented by BST surfaces in the BST domain. Ghersi and Rossi (2001) examined the influence of bi-directional seismic excitations on the inelastic behavior of irregular one-story systems using resisting elements arranged along two orthogonal directions. Their analysis results showed that the inelastic responses are slightly affected by the simultaneousness of two seismic components although the results have large dispersion. De Stefano and Pintucchi (2002) presented the refined structural model of an in-plan asymmetric building to overcome the limitations of the simplified models and to evaluate the effects of the inelastic interaction in torsionally rigid asymmetric systems considering bi-directional earthquakes. The inelastic interaction between the axial forces and the bi-directional plan forces in the vertical resisting elements decreased the

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rotation of the floor about 20-30 % for the systems with the natural periods larger than 0.2 seconds.

Studies of asymmetric buildings under bi-directional earthquakes have been carried out by many researchers. Marusic and Fajfar (2005) investigated the elastic and inelastic seismic responses of the plan-asymmetric regular multistory steel-frame buildings under bi-directional ground motions, and showed qualitative similarity of the torsional responses in the elastic and inelastic ranges with the exception of the stiff edges in the strong direction of torsionally stiff buildings and in the weak direction of torsionally flexible buildings. The weak direction is defined as the direction applied by the component with the higher peak ground velocity. The response is influenced by the magnitude of the plastic deformation, and the torsional effects are reduced with increasing plastic deformations. Antonio and Ricardo (2007) showed that incident angles of the ground motion impact on several engineering demand parameters of a two-way asymmetric single story structure. They demonstrated that the application of bi-directional ground motions only along the principal axes of asymmetric building underestimates the inelastic peak deformation demands comparing those obtained at other angles of incidence. They also show that overall torsion in a building can be reduced by increasing the degree of inelasticity. Lin and Tsai (2008) proposed 3D modal pushover analysis (3DMPA) which is the procedure of an uncoupled modal response history analysis (UMRHA) using the three degree of freedom (3DOF) modal stick instead of the conventional single degree of freedom (SDOF) modal stick. They showed the effectiveness of the proposed 3DMPA in assessing the seismic response of two-way asymmetric buildings subjected to bi-directional ground excitations through an example analysis of a torsionally flexible asymmetric building.

Stathopoulos and Anagnostopoulos (2004) examined the inelastic seismic torsional response of simple structures with the shear-beam type models as well as with plastic hinge idealization of the single story buildings. Rotational ductility factor and single story shear-beam building models with single or double mass and stiffness eccentricities were used. Rotational ductility factor was mentioned as the basic index of post-elastic response because it can be used as measures of inelastic deformations for both concrete and steel braced frame structures. They also compared normalized displacements and ductility demands of beam between the plastic hinge model and the simplified model. The comparison showed that the effect of eccentricity on the edge displacements at the flexible edge appears to be substantially greater in the simplified model than in the plastic hinge model. Concerning the effects of eccentricity on ductility demands, the simplified model appears to predict some increase at the stiff side and practically no effect at the flexible side. On the other hand, the effects predicted by the plastic hinge model are about the opposite: for the systems examined, an increase in eccentricity appears to reduce rotational ductilities in the frame beams at the stiff side and to cause no change or an increase of such ductilities in the flexible side. This happens in systems with either single or double eccentricity. They extended the study for multistory building (2004) and found that contrary to what the simplified one-story, shear-beam models predict, the so-called flexible side frames exhibit higher ductility demands than the stiff side frames.

From BST surface, investigations by Lucchini *et al.* (2009) focused on the identification of the parameters governing the nonlinear response of single story one-way asymmetric-plan frames with the plastic hinge at the ends of the columns under uni-directional ground motions. Later, Lucchini *et al.* (2011) investigated the torsional response of a two way asymmetric single story building under biaxial excitation based on nonlinear dynamic analyses. Ground motions of increasing intensities, characterized by varying angles of incidence, are used to show the evolution of the seismic behavior with the increase of the inelastic demand. Results of the numerical simulations indicated that the parameters governing the nonlinear response of the asymmetric-plan are associated with the center of resistances (CRs) of system. It is noted that such BST combination corresponds to $V - T_{V\max}$, that is, to the one producing the plastic mechanism of the system that provides the maximum lateral strength $V_{\max}$ in the imposed direction of the seismic excitation. The $V - T_{V\max}$ value can be calculated by evaluating such location on the BST surface (De La Llera and Chopra 1995), defined as the BST interaction surface of all different plastic mechanisms that can develop in the building, corresponding to $V_{\max}$. This point is denoted as the center of resistance CR.

The aim of this study is to identify the parameters governing the nonlinear response of asymmetric-plan structures by investigating the BST surface of the building which is related to the resistance distribution and the resisting elements of the building. The eccentricity ($e_{\max\ \text{total disp},i}$) curves of base shear producing the maximum total displacement in columns are developed with increase of intensity of the ground motions. The results show that, as the responses of the buildings increase from linear to nonlinear range, the global seismic forces (F_x, F_z, M_y) which produce the maximum demands in the resisting elements, can be changed. The distributions of those global seismic forces tend to converge toward a single distribution but these happen only in a definable intensity range.

2. Single story frame for investigation

The structure in this study is one-way asymmetric single story building shown in Fig. 1. The rectangular plan dimensions along the x and z axes and the story height are equal to b, a and h, respectively. The building consists of a rigid diaphragm where the entire story mass is lumped. CM, Ms and I_M denote the center of mass and the translational and rotational masses, respectively. The lateral resistance of the building is provided by six columns symmetrically located in the plan. The plastic hinges subjected to nonlinearities are located at the both ends of each column, and characterized by the same rigid-plastic constitutive law. The yielding point at each i-column is specified by a circular interaction surface such as bending moment (M_x) - bending moment (M_z) - axial force (F_y) interaction surface, and defined by a yielding force value $f_{yielding} = 2M_{yielding}/h$. Yielding forces of the elements in the stiff, mid-stiff and flexible sides are 268, 236 and 204 kN, respectively. Hysteretic behavior with hardening under cyclic loading conditions is adopted. Rayleigh model where the damping matrix is equal to the linear combination of the mass and tangent stiffness matrix, respectively, with equivalent modal damping ratio 2% is considered. The beam elements with hinges in OpenSees are used to construct resisting elements. The dimensions of square sections of columns are different: the dimension of column sections in the stiff, mid-stiff and flexible sides are 50, 45 and 40 cm, respectively, as shown in Fig 2. The different resistance and stiffness distributions of columns are used to define the systems characterized by different nonlinear seismic responses. The CM is coincident with the center of geometry of plan. Center of resistance distribution (CRd) of the system is defined based on the yielding force values of all resisting elements of the model. CRd is not coincident with CM and the center of geometry of plan due to the difference of resistance of columns. The eccentricity between CM and CRd is called resistance eccentricity, and that between CM and center of stiffness (CS) is defined as stiffness eccentricity. In this study, the geometric dimensions of b, a and h are 10, 5 and 3m, respectively. The stiffness of the elements in the stiff, mid-stiff and flexible sides are equal to 14384, 9437 and

5892 kN/m, respectively. While the mass distribution is characterized by values of M and I_M , respectively, are equal to 68.8 kN/(m/s2) and 3000 kN(m/s2).

3. Ground motion

Artificial earthquake with peak ground acceleration (PGA) of 0.154g shown in Fig. 3 is used according to the seismic design guideline of Korea (MOCT 1997, Park *et al.* 2009). Two same ground motions are simultaneously applied in the main and secondary directions. The incident angles of the main and secondary ground motions are α and α + 90 to the x-axis respectively as shown in Fig. 1. The natural period obtained from the numerical model of structure (T_n) is 0.267 (s).

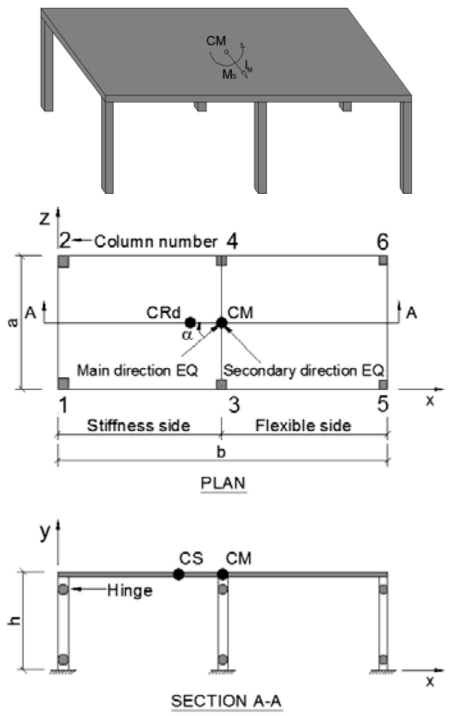


Fig. 1 General configuration of one-way asymmetric single story building

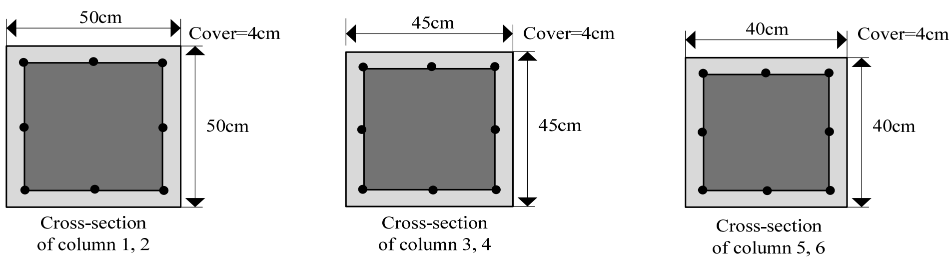


Fig. 2 Cross-section of columns

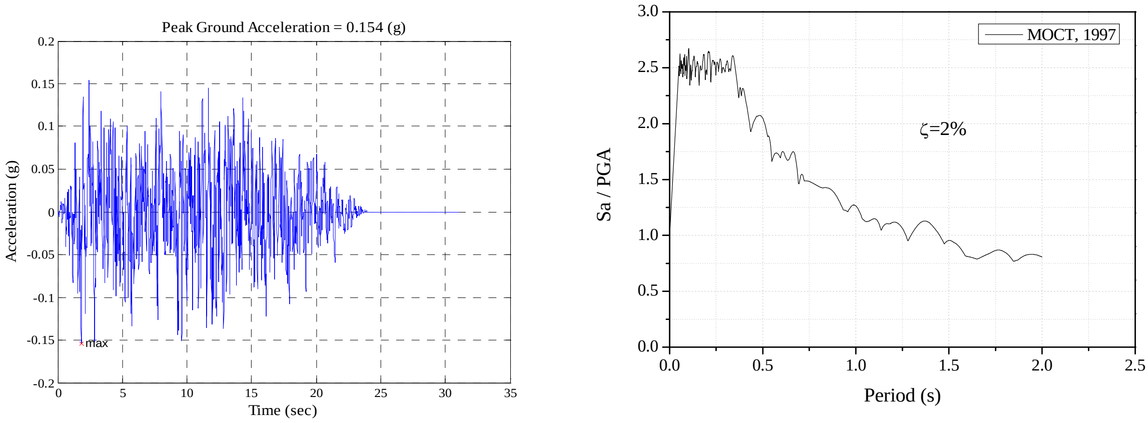


Fig. 3 The artificial earthquake in seismic design guidelines II, Seoul, Korea (MOCT 1997)

4. Methodology

The earthquake excitations are defined by the *x* and *z* components of the acceleration, $\ddot{\mathbf{u}}_{gx}(t)$ and $\ddot{\mathbf{u}}_{gz}(t)$, and the equation of motion of the structure under the earthquake excitation of $\ddot{\mathbf{u}}_g$ is

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{f}_s(\mathbf{u}, \dot{\mathbf{u}}) = -\mathbf{M}\boldsymbol{\iota}\ddot{\mathbf{u}}_g \tag{1}$$

where $\mathbf{u} = \{\mathbf{u}_x \ \ \mathbf{u}_z \ \ \mathbf{u}_\theta\}^T$ is the displacement vector of the diaphragm, $\mathbf{u}_x$ and $\mathbf{u}_z$ are translational displacements and $\mathbf{u}_\theta$ is torsional rotation of the diaphragm produced by the *x* and *z* components of the seismic action. $\ddot{\mathbf{u}}$ and $\dot{\mathbf{u}}$ are acceleration and velocity vectors corresponding to **u**. **M** and **C** are the mass and damping, respectively. The influence vector ***ι*** represents the displacements of the masses resulting from static application of unit ground displacement. The forces $\mathbf{f}_s$ corresponding to displacements **u** are not single-valued and depend on the history of the displacements. Specially, the earthquake excitation $\ddot{\mathbf{u}}_g$ is defined by $\ddot{\mathbf{u}}_g = \{\ddot{\mathbf{u}}_{gx}(t) \ \ \ddot{\mathbf{u}}_{gz}(t) \ \ 0\}^T$ using the *x* and *z* components of the acceleration, $\ddot{\mathbf{u}}_{gx}(t)$ and $\ddot{\mathbf{u}}_{gz}(t)$.

The angles of incidence *α* of the uni-directional ground motion ranging from 0 to 360° with increments of 15° counter clockwise are considered. The maximum responses obtained at orthogonal directions, in which, the structural coordinates are identical to the ground motion coordinates, are compared with those obtained at other angles of incidences. Rotational ductility factor is the basic index of post-elastic response based on the maximum plastic hinge rotation at the top of columns. It is defined as

$$\mu = 1 + \frac{\theta_p}{\theta_y} \tag{2}$$

where θ_p is the maximum rotation of the plastic hinge at the ends of columns and θ_y is a normalizing yield rotation not only for beams but also for columns with only one end yielding. θ_y is typically set equal to $\theta_y = \mathbf{M}_{yielding}\mathbf{L}/6\mathbf{EI}$ where $\mathbf{M}_{yielding}$, **L**, **E**, and **I** are the yielding moment, column length, elastic modulus, and second moment of the cross-section, respectively. The ductility factors obtained also depend on angles of incidence of the ground motions. Therefore, it is necessary to evaluate the ductility demand according to angles of incidence of earthquakes for asymmetric buildings in design process.

All the global seismic forces (F_x, F_z, M_y) which produce the maximum displacement in the columns are considered with the PGA's of 0.5, 0.6, 0.7, 0.8 and 1.2 g. The value of $e_{\text{max total disp},i}$ eccentricity of base shear producing the maximum total displacement in i-column can be obtained from above global seismic force. $e_{\text{max total disp},i}$ eccentricity curve is defined as the curve which connects all the eccentricity points according to angles of incidence. The respective change between $e_{\text{max total disp},i}$ eccentricity curve and the e_r curve defined by resistance distribution of system is examined by the incremental dynamic analysis (IDA) method.

5. Numerical results

5.1 Base shear

Maximum base shears in the x and z directions (F_x and F_z) in the columns of the model under the different incident angles of the ground motions (PGA = 1.2g) are shown in Figs. 4 and 5 and their values are summarized in Tables 1 and 2. The results show that the maximum base shears are not obtained at orthogonal direction in columns. In the elastic range of the structural behavior, it is easy to demonstrate that for a one-way asymmetric single story building subjected to uni-directional ground motion of identical waves, the maximum base shear in x direction obtains at 135^o or 315^o incident angle of the ground motions, while the maximum base shear in the z direction obtains at 45^o or 225^o incident angle of the ground motions. However, the above observation is no longer valid when the dynamic response changes from the elastic range to the inelastic range of the structural behavior. It is apparent from Table 1 that the maximum base shears induce in x direction in column 1 and 2 at 150^o and 120^o incident angles of the seismic motions, respectively. The maximum base shear obtains in the z direction in column 6 at 30^o incident angle of the ground motions as summarized in Table 2. Besides, it is important to notice from Figs. 4 and 5 that the maximum base shears in the columns located in the stiff side (column 1, 2) of the plan are smaller than those obtained in the columns located in the mid-stiff side (column 3, 4) of the plan, which is completely contrasted to the obtained results in the elastic range of the structural behavior with PGA = 0.154g, as shown in Figs. 6 and 7. The above obtained results can be explained that with increasing inelasticity of the structural behavior due to the yielding of some heavier stressed members, the initial one-way asymmetric building approaches to a two-way asymmetric building in some extent, therefore, the maximum responses of the structure attain at another incident angles instead of 135^o, 315^o, 45^o or 225^o incident angles of the ground motions. Simultaneously, the maximum base shears in columns located in the mid-stiff side obtain the larger value in comparison with those located in another sides. Table 1 shows that maximum base shear values in x direction in column 1 obtained at 150^o and 315^o incident angles of the ground motions are 236.57 and 236.45 kN, while those in column 2 obtained at 120^o and 315^o are 238.29 and 236.12 kN, respectively. Although small difference in maximum base shears according to different incident angles of the ground motions is observed, the influence of the incident angles of earthquakes on the inelastic dynamic response of the asymmetric single story building is noticeable.

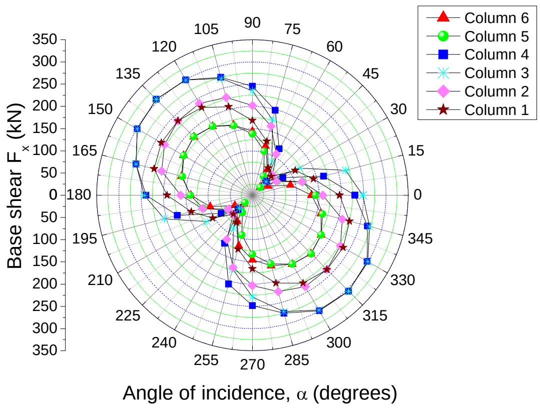


Fig. 4 Maximum base shear F_x in columns of the building (PGA = 1.2g)

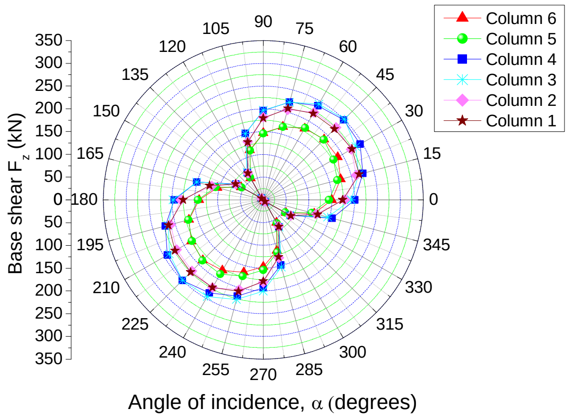


Fig. 5 Maximum base shear F_z in columns of the building (PGA = 1.2g)

Table 1 Maximum base shear F_x (kN) with angles of incidence, PGA = 1.2g

Column number	Orthogonal directions				Non-orthogonal direction		
	0°	90°	180°	270°	120°	150°	315°
Column 1	195.56	168.10	191.70	165.55	227.85	236.57	236.45
Column 2	157.82	202.21	161.14	203.12	238.29	227.86	236.12
Column 3	247.97	234.74	242.77	229.10	299.42	298.55	305.41
Column 4	230.52	245.32	239.06	248.25	299.56	298.89	305.38
Column 5	141.95	137.99	139.37	133.09	179.09	177.98	186.22
Column 6	132.55	143.85	138.51	145.51	179.04	178.80	185.95

Table 2 Maximum base shear F_z (kN) with angles of incidence, PGA = 1.2g

Column number	Orthogonal directions				Non-orthogonal direction		
	0°	90°	180°	270°	30°	45°	225°
Column 1	173.33	179.68	175.15	178.46	224.01	220.28	224.31
Column 2	178.04	181.56	180.42	181.01	221.72	224.87	220.80
Column 3	192.33	196.73	197.47	200.22	233.83	251.06	249.32
Column 4	200.08	196.14	194.94	193.17	244.61	248.60	250.30
Column 5	143.83	145.79	142.99	154.11	177.11	187.07	188.26
Column 6	149.67	147.10	140.54	146.84	188.23	188.11	187.20

5.2 Torsional moment

Maximum torsional moments M_y in the columns under the different incident angles of the ground motions with PGA = 1.2g, are compared in Fig. 8 and Table 3. It is noticed that the maximum torsional moments obtain in all columns at 15° angle of incidence of the ground motions. However, torsional moments are maximum at 45^{o} incident angle with PGA = 0.145g (Fig. 9 and Table 4). The incident angle of the earthquakes, in which, the maximum torsional moment obtained, changes from 45° to 15° when the structural behavior changes from the elastic to inelastic range.

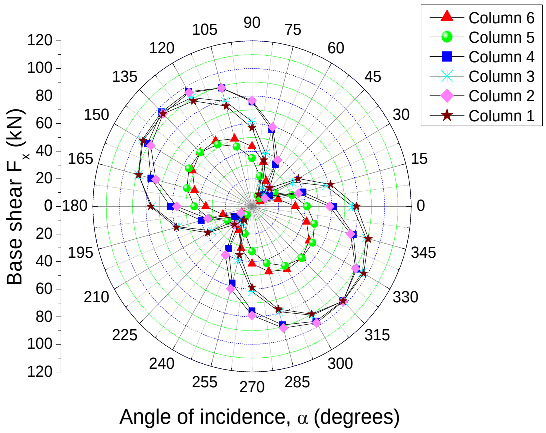


Fig. 6 Maximum base shear F_x in columns of the building (PGA = 0.154g)

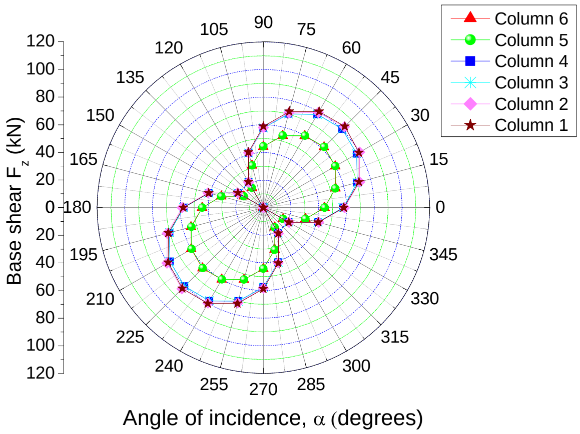


Fig. 7 Maximum base shear F_z in columns of the building (PGA = 154g)

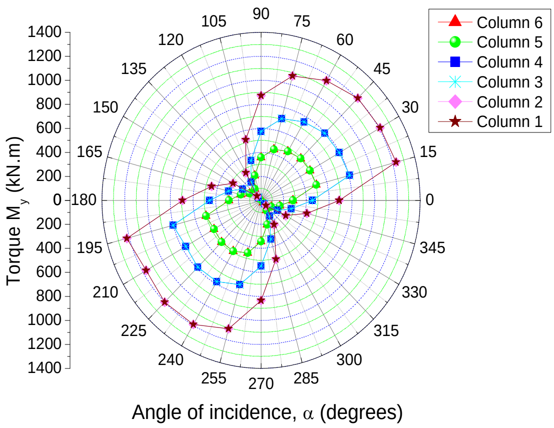


Fig. 8 Maximum torsional moment M_y in columns of the building (PGA = 1.2g)

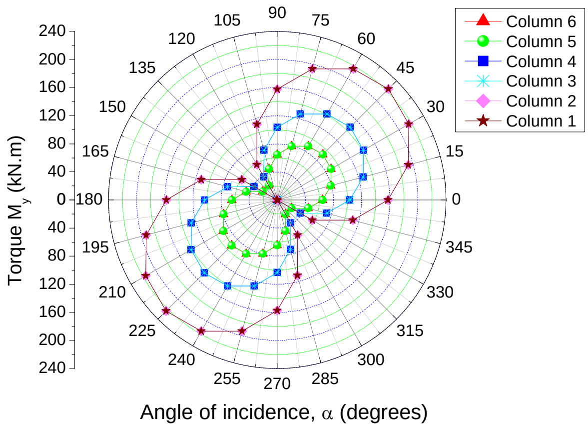


Fig. 9 Maximum torsional moment M_y in columns of the building (PGA=0.154g)

Table 3 Maximum torsional moment M_y (kN.m) with angles of incidence, PGA = 1.2g

Column number	Orthogonal directions				Non-orthogonal direction	
	0°	90°	180°	270°	15°	45°
Column 1	687.36	873.57	691.41	834.05	1230.21	1204.47
Column 2	687.36	873.57	691.41	834.05	1230.21	1204.47
Column 3	450.98	573.15	453.63	547.22	807.14	790.25
Column 4	450.98	573.15	453.63	547.22	807.14	790.25
Column 5	281.53	357.79	283.18	341.60	503.87	493.32
Column 6	281.53	357.79	283.18	341.60	503.87	493.32

Table 4 Maximum torsional moment M_y (kN.m) with angles of incidence, PGA = 0.154g

Column number	Orthogonal directions				Non-orthogonal direction
	0°	90°	180°	270°	45°
Column 1	157.18	157.66	157.58	157.32	223.27
Column 2	157.18	157.66	157.58	157.32	223.27
Column 3	103.13	103.44	103.39	103.22	146.49
Column 4	103.13	103.44	103.39	103.22	146.49
Column 5	64.38	64.57	64.54	64.43	91.45
Column 6	64.38	64.57	64.54	64.43	91.45

Table 5 Maximum rotational ductility factor in columns, PGA = 0.154g

Column number	Orthogonal directions				Non-orthogonal direction
	0°	90°	180°	270°	45°
Column 1+2	1.1957	1.2122	1.2010	1.2082	1.3025
Column 3+4	1.1455	1.1577	1.1494	1.1548	1.2249
Column 5+6	1.1048	1.1137	1.1076	1.1116	1.1621

Table 6 Maximum rotational ductility factor in columns, PGA = 1.2g

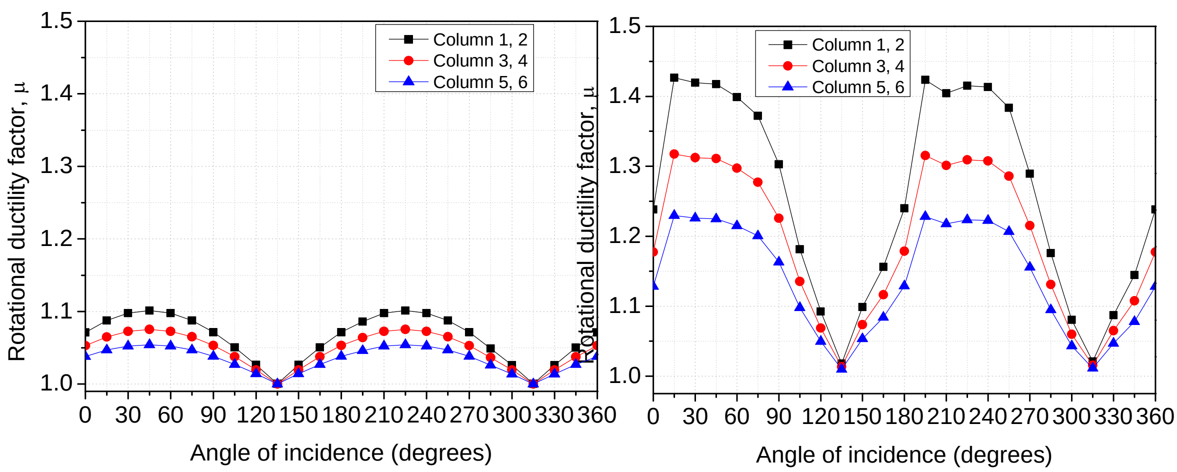
Column number	Orthogonal directions				Non-orthogonal direction	
	0°	90°	180°	270°	45°	15°
Column 1+2	1.2383	1.3028	1.2397	1.2891	1.4175	1.4264
Column 3+4	1.1775	1.2256	1.1785	1.2154	1.3110	1.3177
Column 5+6	1.1283	1.1631	1.1291	1.1557	1.2248	1.2296

5.3 Rotational ductility factor

The rotational ductility factor of buildings under the different incident angles of the seismic ground motions is investigated. The results in Fig. 10(a) and Table 5 show that rotational ductility factor in the elastic range of the structural behavior is the maximum at 45° incident angle of the ground motions with the intensity PGA = 0.154g. The effects of the intensity of ground motions on the rotational ductility factor are also investigated. The results show that the rotational ductility factor increases with the increase of the intensity of ground motion from PGA = 0.154g to PGA = 1.2g. It is noticed from Fig. 10(b) and Table 6 that the incident angle, in which, the rotational ductility factor attained maximum value, changes from 45° to 15° when structural behavior varies from the elastic to inelastic range. The inelasticity of the structural behavior plays an important role in considering the influence of the incident angles of the ground motions on dynamic responses.

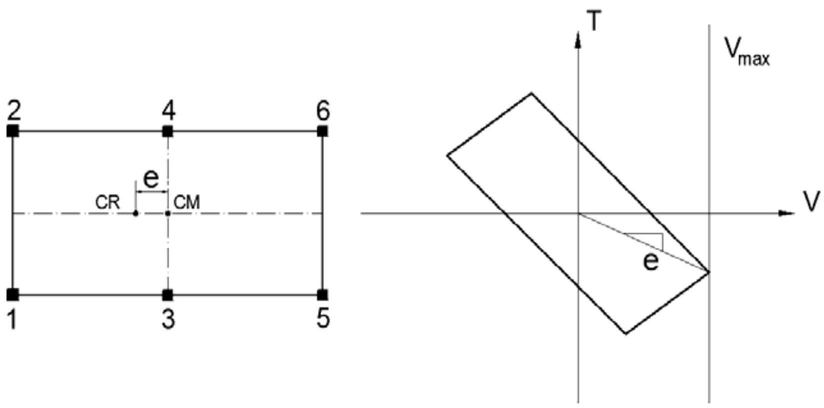
5.4 BST surface of resisting elements

The BST surface of the one-way asymmetric single story building is constructed and shown in Fig. 11. Each *T-V* point of the BST surface corresponds to the collapse mechanism characterized by a resisting force distribution with the resultant located at the distance $e = T/V$ from CM. Among all of the mechanisms, the one which provides the maximum lateral strength V_{max} of the building is associated with the center of resistance, CR. The relationship between each resisting element and resistance distribution of system is also examined using the shape of the BST surface of the building. The BST response histories of the resisting elements and their corresponding boundaries are shown in Fig. 12. The boundary is the idealized curve which is fittest with the BST response histories of the resisting elements. It is clear that the shape of the boundary of the BST surface at each resisting element is very similar to the shape of the BST surface of the building. The results show that the shape of the BST surface of building is characterized by the shapes obtained at resisting elements in this studied structure. It is reasonable that the e_r and $e_{\max\ \mathrm{total\ disp},i}$ curves also have the same shape as shown in Fig. 16. However, other types of studied model still need to be considered in future to clarify the above obtained observation.



(a) PGA = 0.154 g (b) PGA = 1.2 g

Fig. 10 Rotational ductility factor in columns of the building



(a) Location of CR is defined from BST surface (b) BST surface corresponding z direction

Fig. 11 BST surface for buildings characterized by resistance distributions

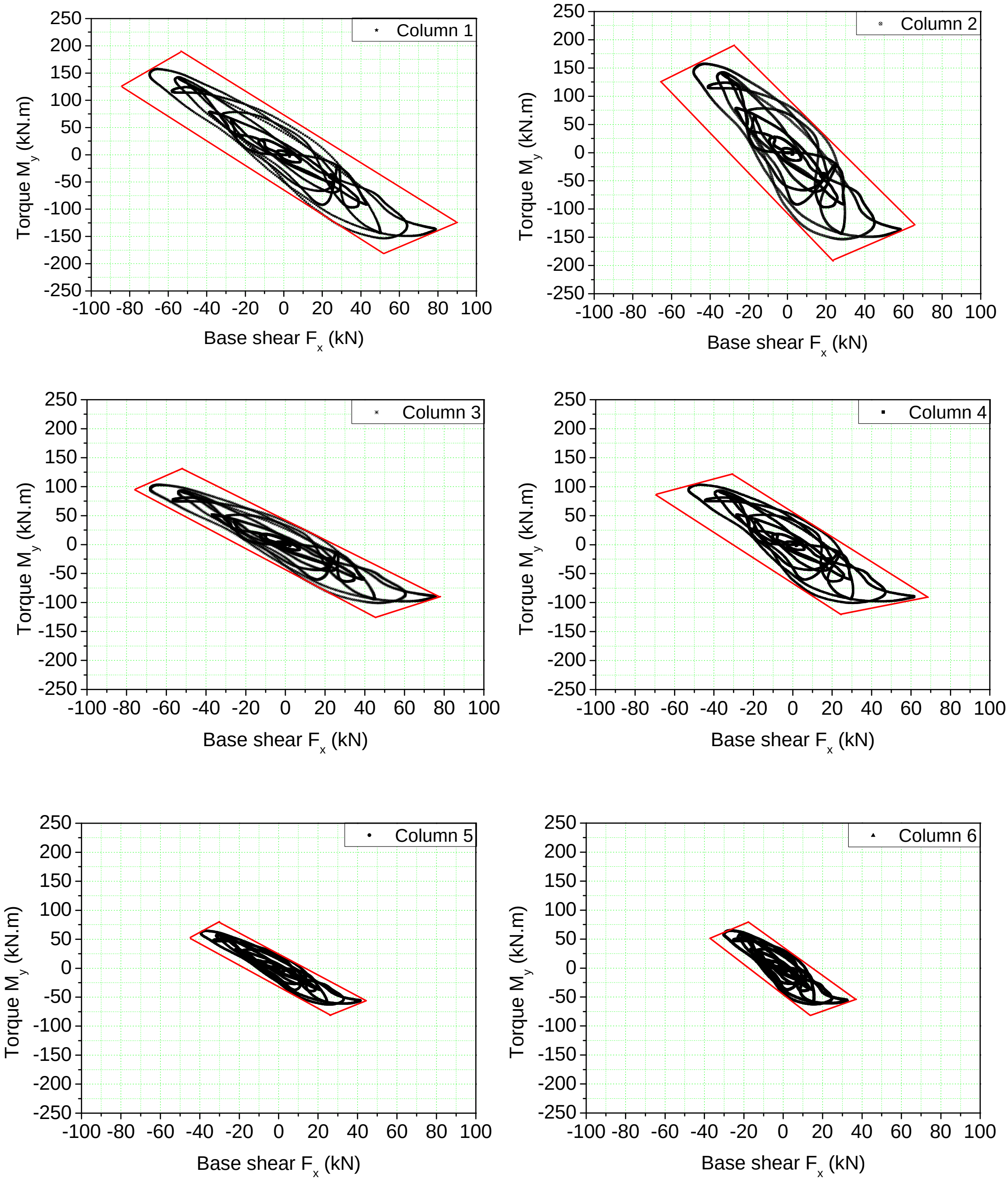


Fig. 12 BST response histories of resisting elements (PGA=0.154g)

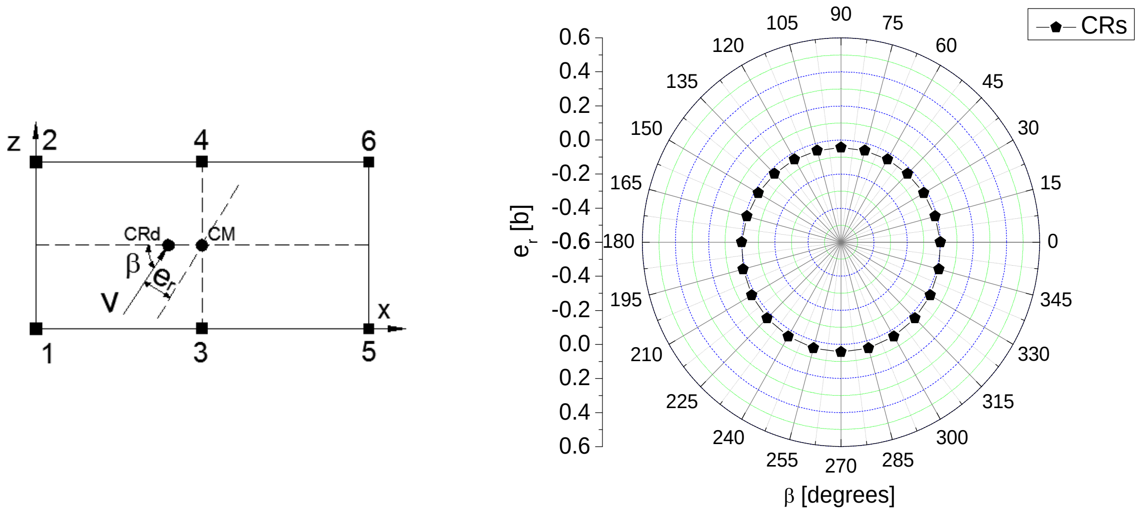


Fig. 13 The e_r curve characterizing the resistance distribution of the system

5.5 Intensity of ground motions

The values (F_x, F_z, M_y) of the BST that produce the maximum total displacements in columns of the structure are reported. Fig. 13 shows the eccentricity (e_r) of the CRs corresponding to the base shear direction ranging from 0 to 360°. The eccentricities are obtained from the combined BST surface with the plastic mechanisms that provide in each fixed β-direction the maximum lateral strength of the building. The location of the CRs can be evaluated through the analysis of the BST surface, i.e., the BST interaction surface of all the different plastic mechanisms in the building (Lucchini *et al.* 2011). Because the eccentricity between CRd and CM in the system is 0.045b, the associated e_r is -0.045b from Fig. 13 by setting β equal to 90°.

If all the α-direction of the excitation is considered, all the global seismic forces producing the maximum total displacements in columns can be evaluated as shown in Fig. 14. The base shear V corresponds to the resultant of the base shears, F_x and F_z in the x and x directions, respectively, is defined as

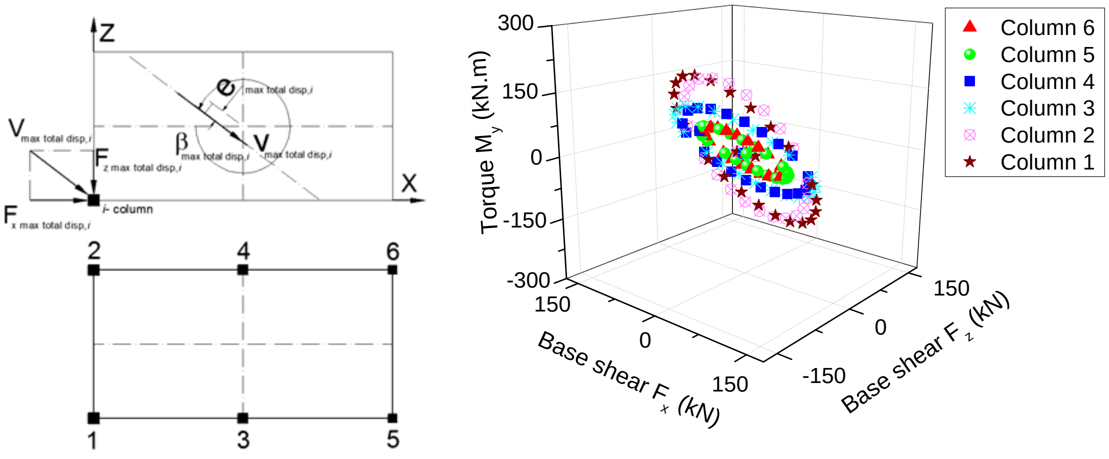
$$V = \sqrt{F_x^2 + F_z^2} \tag{3}$$

Thus, the eccentricity in columns of base shear V in the β direction, which produces the maximum total displacement, is calculated as

$$e_{\text{max total disp},i} = M_y / V_{\text{max total disp},i} = T / V_{\text{max total disp},i} \tag{4}$$

The e_r and $e_{\text{max total disp},i}$ curves are shown in shown in Fig. 15. Although the distribution of eccentricities of the base shears producing the maximum total displacement at each resisting element is different, the trend of distribution is similar to the e_r curve, and the nonlinear seismic responses seem to be basically governed by resistance distribution.

The results of six sets of analyses considering varying incident angles and PGA's of excitations are shown in Fig. 16. The results show that the curves of $e_{\text{max total disp},i}$ converge to the curve of e_r in the range of PGA from 0.154g to 0.6g. However, the $e_{\text{max total disp},i}$ curves tend to diverge in the range of the strong PGA more than 0.7g. The reason is that with the increase of the earthquake intensity, the behavior of structure gets deeper in the nonlinear range so the e_r curve also changes. Moreover, as mentioned earlier, the constitutive model used for the hinges at the ends of the columns exhibit hardening so the e_r curve starts changing as soon as the structure yields. It is concluded that the $e_{\text{max total disp},i}$ curves tend to converge together only in the definable intensity range. The different global seismic forces in this definable intensity range get on the system that produces the maximum demand as the maximum total displacements in resisting elements tend to converge toward a single distribution. This single distribution is related to the resistance distribution of the building.



(a) (b)

Fig. 14 Variation of the base shears location producing the maximum total displacement: (a) Direction $\beta_{\text{max total disp},i}$ and eccentricity $e_{\text{max total disp},i}$ of the base shear producing the maximum total displacement in the i-column; (b) Global seismic forces $(F_x, F_z, M_y)_{\text{max total disp},i}$ producing the maximum displacements in all the columns corresponding to angles of incidence α of ground motions

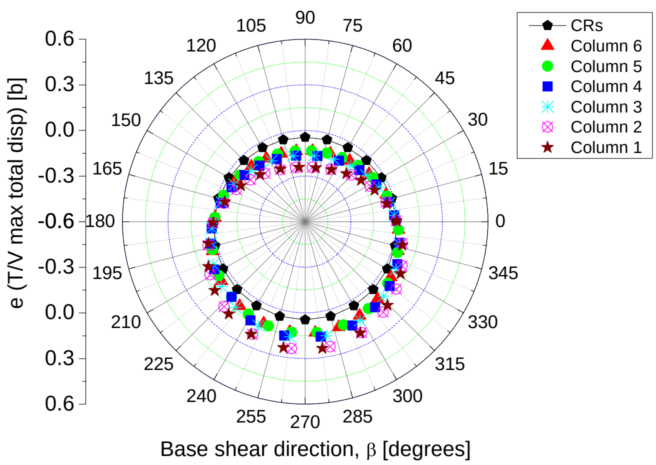


Fig. 15 $e_{\text{max total disp},i}$ and e_r curve

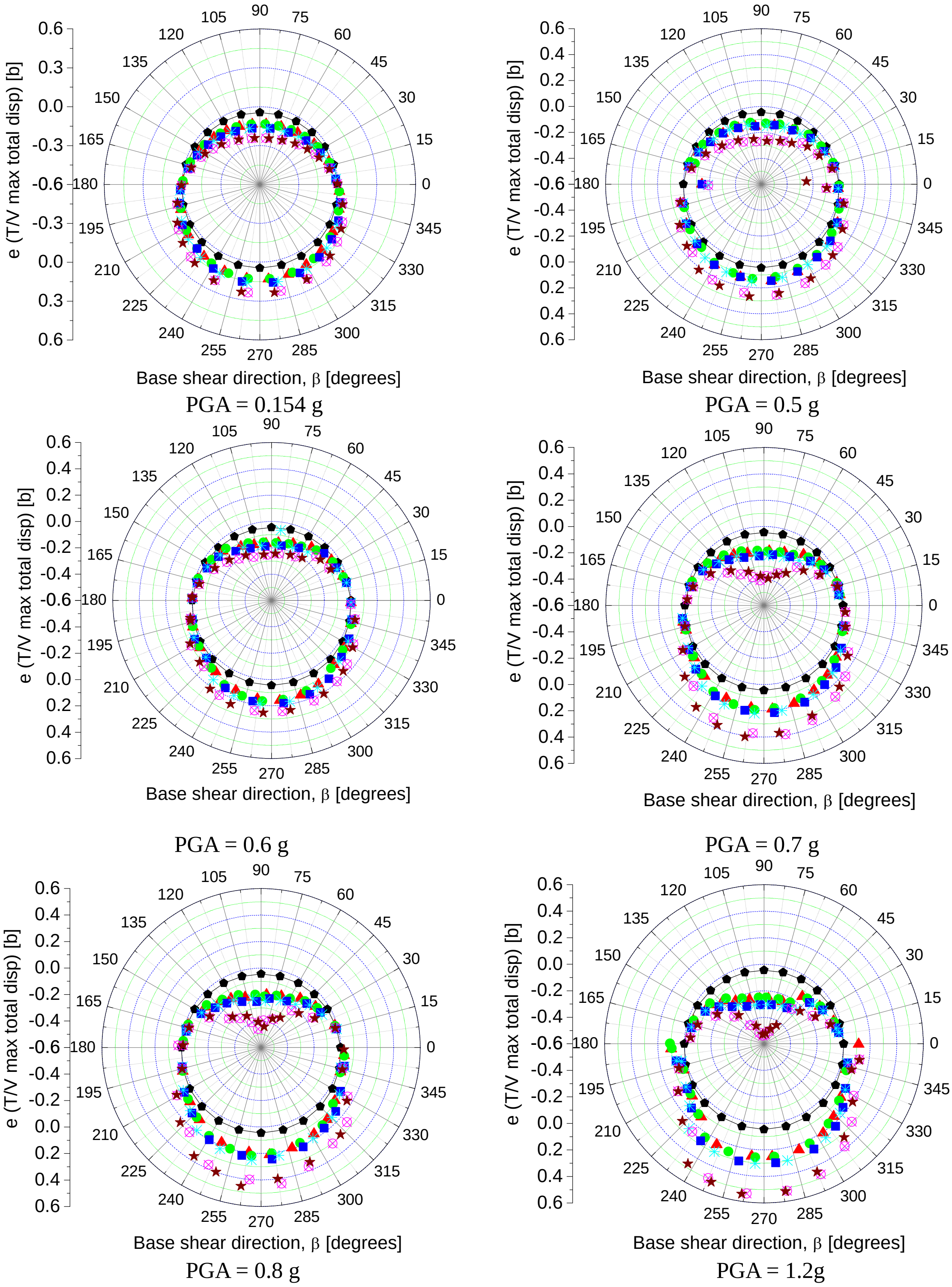


Fig. 16 Global seismic forces $(F_x, F_z, M_y)_{\text{max total disp},i}$ producing the maximum total displacements in columns of the structure for different seismic intensities

6. Conclusions

Inelastic dynamic responses of the one-way asymmetric single story building under the different incident angles of the ground motions are investigated. Sensitivity analyses on the design parameters of the building show that the incident angle of ground motions, in which, the maximum rotational ductility factor as well as the maximum torsional moment attained, changes from 45^o to 15^o when the structural behavior varies from the elastic to inelastic range. The maximum internal forces such as base shears in all columns are not always the maximum value at 45^o, 135^o, 225^o or 315^o incident angles of the ground motions. Because of the increase of inelasticity of the structural behavior induced by the yielding of some heavier stressed members, the initial one-way asymmetric building approaches to a two-way asymmetric building in some extent, therefore, the maximum dynamic responses of the structure attained at another incident angles instead of 135^o, 315^o, 45^o or 225^o incident angles of the ground motions. Moreover, the maximum base shears in columns in the mid-stiff side of the plan are larger than those obtained in the columns in the stiff side of the plan.

The relation between the BST surface of the building characterized by resistance distribution and the BST surfaces of response histories of resisting elements is investigated. The obtained results show that the shape of the BST surface of the system is representative for shapes of BST surfaces of response histories of resisting elements in this studied structure. The different global seismic forces that produce the maximum demands in the resisting elements tend to converge toward a single distribution in the definable intensity range. The curves of $e_{\text{max total disp},i}$ converge to the curve of e_r in the range of PGA from 0.154g to 0.6g, but the $e_{\text{max total disp},i}$ curves tend to diverge in the range of the strong PGA more than 0.7g due to the increase of inelasticity of the structural behavior. When the structural behavior gets deeper in nonlinear range of the structural behavior, the initial one-way asymmetric building approaches to a two-way asymmetric building in some extent or the e_r curve is changed.

The presented results have been obtained from the analysis of a single structure with a single ground motion only. The additional investigations such as analyses with more earthquake records considering different structural schemes are still needed for confirming the generality of the reported conclusions.

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References

Antonio, B.R. and Ricardo, A.M. (2007), "Influence of angle of incidence on seismic demands for inelastic single- storey structures subjected to bi-directional ground motions", *Engineering Structures,* **29**, 2593-2601.

De La Llera, J.C. and Chopra, A.K. (1995), “Understanding the inelastic seismic behavior of asymmetric-plan buildings”, *Earthquake Engineering and Structural Dynamics,* **24**(4), 549-572.
De Stefano, M. and Pintucchi, B. (2002), “A model for analyzing inelastic seismic response of plan irregular building structures”, *15th ASCE Engineering Mechanics Conference,* Comlumbia University, New York, NY, June.
Gersi, A. and Rossi, P.P. (2001), “Influence of bi-directional ground motions on the inelastic response of one-storey in- plan irregular systems”, *Engineering structures,* **23**, 579-591.
Lin, J.L. and Tsai, K.C. (2008), “Seismic analysis of two- way asymmetric building systems under bi-directional seismic ground motions”, *Earthquake Engineering and Structural Dynamic,* **37**, 305-328.
Lucchini, A., Monti, G. and Kunnath, S. (2009), “Seismic behavior of single- story asymmetric-plan buildings under uniaxial excitation”, *Earthquake Engineering and Structural Dynamic,* **38**, 1053-1070.
Lucchini, A., Monti, G. and Kunnath, S. (2011), “Nonlinear response of two- way asymmetric single- story building under biaxial excitation”, *Journal of Structural Engineering ASCE,* **137**(1), 34-40.
Marusic, D. and Fajfar, P. (2005), “On the inelastic seismic response of asymmetric buildings under bi-axial excitation”, *Earthquake Engineering and Structural Dynamic,* **34**, 943-963.
MOCT (1997), “Seismic Design Guideline (II)”, Seoul, Korea.
OpenSees Development Team (1998-2005), OpenSees: Open System for Earthquake Engineering Simulation, Scott, R.F., 1981, Foundation Analysis, Prentice Hall.
Park, D., Dong, Y.K., Chang, K.C. and Byung, S.C. (2009), “Evaluation of liquefaction potential of port structures with earthquake magnitude adjustment”, *Journal of Coastal Research,* 1035-1039.
Stathopoulos, K.G. and Anagnostopoulos, S.A. (2003), “Inelastic earthquake response of single- story asymmetric buildings: an assessment of simplified shear- beam models”, *Earthquake Engineering and Structural Dynamic,* **32**, 1813-1831.
Stathopoulos, K.G. and Anagnostopoulos, S.A. (2004), “Earthquake induced inelastic torsion in asymmetric multistory buildings”, *13th World Conference on Earthquake Engineering,* Vancouver, B.C., Canada, August.

Nomenclature

The following symbols are used in this paper:

$\mathbf{M}$	=	mass matrix
$\mathbf{C}$	=	damping matrix
$\mathbf{K}$	=	stiffness matrix
$\ddot{\mathbf{u}}_g$	=	excitation vector
$\ddot{\mathbf{u}}_{gx}(t)$	=	component of the acceleration in x direction
$\ddot{\mathbf{u}}_{gz}(t)$	=	component of the acceleration in z direction
$\mathbf{u}$	=	displacement vector
ι	=	influence vector
ζ	=	damping ratio
$\mathbf{u}_x$	=	translational displacement in x direction
$\mathbf{u}_z$	=	translational displacement in z direction
$\mathbf{u}_\theta$	=	torsional rotation of the diaphragm
$\ddot{\mathbf{u}}$	=	acceleration vector

$\dot{\mathbf{u}}$	=	velocity vector
μ	=	rotational ductility factor
θ_p	=	maximum rotation of the plastic hinge at the ends of column.
θ_y	=	normalizing yield rotation
$\mathbf{M}_{yielding}$	=	yielding moment
L	=	column length
E	=	elastic modulus
I	=	second moment of the cross-section
V	=	base shear
F_x	=	base shear in x direction
F_z	=	base shear in z direction
T	=	torque
a	=	the width of building
b	=	the length of building
h	=	the height of building
CM	=	center of mass
CR	=	center of resistance
CRd	=	center of resistance distribution of system
CS	=	center of stiffness
Ms	=	translational mass
I_M	=	rotational mass
α	=	angle of incidence
M_y	=	maximum torsional moment
M_x	=	bending moment in x direction
M_z	=	bending moment in z direction
$f_{yielding}$	=	yielding force
F_x	=	maximum base shear in x direction
F_z	=	maximum base shear in z direction
F_y	=	axial force
β	=	base shear direction of building
e_r	=	eccentricities corresponding to the base shear β-direction of building
$\beta_{\text{max total disp},i}$	=	base shear direction producing the maximum total displacement in i-column
$e_{\text{max total disp},i}$	=	eccentricities corresponding to base shear $\beta_{\text{max total disp},i}$-direction in i-column