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Performance of Asymmetric Multistory Shear Buildings with Different Strength Distributions

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Abstract: The experience from the performance of buildings during past earthquakes has shown that asymmetric buildings often sustain more extensive damages as compared to the symmetric buildings. Performance of an asymmetric building can be quantified by responses such as rotation of the floor, the maximum drift of flexible and stiff edges of the building or the ductility demand of the elements on those edges. In this study, the nonlinear dynamic behaviour of a set of five-story asymmetric buildings with different strength distribution is studied to investigate the effect of different strength distribution strategies. To show different responses such as drift, ductility and plastic hinge rotation of the models, fragility curves are used. The results show that among torsionally rigid models studied here, models with a smaller strength eccentricity perform better. However, in general, the optimum strength eccentricity is shown to be a function of the selected damage index. For example, if damage index is represented by the inter-story drift ratio, the appropriate strength distribution is the configuration with a small strength eccentricity. On the other hand, if damage function is represented by ductility demand of the elements, the appropriate strength distribution is the one with strength center between the centers of mass and rigidity. By identifying the more exact configurations of centers of mass and rigidity, they can be utilized as a proper way for reduction of adverse torsional effects in design or rehabilitation of asymmetric buildings. These configurations can also be used as a new reference point for identifying acceptable limits of eccentricity.

Key words: Torsion, fragility, center of rigidity, center of strength, shear wall

INTRODUCTION

Non uniform distributions of mass, stiffness and strength in an asymmetric building cause the building to experience torsional moments and rotational deformations around vertical axes. Rotational deformation causes non uniform distribution demand in Lateral Force Resisting Elements (LFREs). Also, it leads to increased damages in an asymmetric building. The experiences of past earthquakes such as Mexico City 1986 (Chandler *et al*., 1996), confirm these effects. The vulnerability of asymmetric buildings has been addressed by building seismic design codes in the form of special torsional provisions. In these provisions, the design eccentricity is defined as a combination of the stiffness eccentricity and the accidental eccentricity. This eccentricity is used to calculate the design torsional moments. Most of these torsional provisions have been primarily developed based on studies of a single story building in the linear range. Although the stiffness eccentricity is a suitable indicator of torsional responses when the building behaves in the linear range, majority of buildings are designed to behave in the nonlinear range during moderate and large earthquakes. In such a situation, stiffness eccentricity is not a good indicator of building torsional responses. Sadek and Tso (1989) introduced the concept of strength eccentricity. The strength eccentricity is defined as the offset of the center of strength to the center of mass. The center of strength is the center of yield strength of the LFREs. The strength eccentricity is an appropriate indicator for the torsional response of an asymmetric building in the nonlinear range. During a ground motion, a building responds in various ranges of behaviors, from full elastic to elasto-plastic and full plastic. During these excitations some LFREs behave in the linear to nonlinear range while the other LFREs remain in the linear range. Therefore, it can be expected that the torsional responses of a building during moderate and high earthquake intensities depend on both strength and stiffness eccentricities.

In earlier researches, it has been mainly assumed that for every LFRE of a building, its stiffness can be

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calculated based on the dimensions and before assignment of strength to it. Therefore, the element stiffness is assumed to be independent of its strength. These LFRE are called K-type elements. For this type of modeling, the location of center of rigidity can be determined before the design procedure and as such, the design parameter is the location of center of strength. Current design procedures and torsional provisions of seismic codes have been developed based on the cited assumption. Recent researches (Aschhiem, 2002) revealed that for many LFREs such as shear walls and moment resisting frames, stiffness is depended on their strength and will be modified during strength assignment. These LFRE are called D-type elements. For buildings composed of these elements, torsional provisions that assume location of center of resistance can be determined before strength distributions are deficient.

Many researches in the past two decades studied inelastic behavior of asymmetric buildings, mostly focused on single story buildings. Single story models can appropriately show torsional behavior of equi-height asymmetric buildings. In studies on single story buildings before 1997 only K-type elements have been considered. In these researches, assuming that the location of center of strength remains unchanged during design procedure, the proper location of center of strength was examined. For these types of buildings Tso and Ying (1992) suggested that the strength eccentricity should be zero or near to zero in order to reduce the ductility demand on a flexible edge element for buildings that have non-uniform stiffness distribution. Rutenberg *et al*. (1992) and De Stefano *et al*. (1993) tried to find optimum location of center of strength relative to the center of mass and the center of rigidity. They concluded that the best location of center of strength is at the middle of centers of mass and stiffness. Based on the plastic mechanism analysis, Paulay (1997, 2001) considered the behavior of a single story asymmetric structure with D-type elements. He concluded that the current lateral strength distribution of seismic codes is inappropriate. He suggested that an arbitrary strength distribution strategy can be more effective for superior performance of an asymmetric structure in the ultimate limit state. He proposed that an appropriate location of center of strength is some place near the center of mass. Myslimaj and Tso (2002, 2005) studied the effect of different configurations of centers of mass, stiffness and strength on responses of a single story asymmetric structure. They demonstrated that the response of asymmetric structures during the earthquake excitation depends on the location of both the centers of strength and stiffness. They proposed that the best configuration of centers of mass, strength and stiffness is a configuration in which the center of mass is between centers of strength and stiffness. This configuration was termed as the balance configuration. According to their study, the balance configuration will improve the inter-story drift and diaphragm rotational responses of a building. However, it can cause an increase of ductility demand on elements at the stiff side of structure.

To develop a methodology for performance-based design of asymmetric buildings, there is a need for recognition of the proper distribution of the strength between LFREs. Identifying the proper configuration of building centers for a required hazard level and Engineering Demand Parameters (EDPs) associated with various limit states can be a designer guide to improve the behavior of an asymmetric building. Consequently, the main purpose of this study is to investigate performance of different configurations of centers for code-designed one directional asymmetric building models undergoing nonlinear dynamic analysis.

To identify the proper configuration of centers more precisely and to evaluate performance of various sets of centers, five-story building models with different configurations of centers of mass, rigidity and strength are used. Edge displacements, floor rotation, maximum inter-story drift, edge ductility demand and plastic rotation are considered as EDPs. The building models are assumed to consist of D-type elements. These models are subjected to two directional far field ground motions. Subsequently, the maximum values of drift, plastic rotation and ductility responses are evaluated by conducting nonlinear dynamic analysis and the results are presented in the form of different limit states fragility curves. The presented study by using these fragility curves in multistory model tries to more precisely determine the configuration of centers as a useful tool to improve performance of asymmetric multi-story buildings.

MATERIALS AND METHODS

To study the effects of different configurations of centers of mass, stiffness and strength at different levels of hazard, five-story buildings with rigid beams are used. The models consist of rigid diaphragms with dimensions of 20×30 m and three shear walls in each direction with similar properties in the stories. Buildings are symmetric in the x-direction and asymmetric in the y-direction. The asymmetry in the y-direction is produced by changing width and strength of the two edge walls in the y-direction (Fig. 1a, b). Thus, the total strength in all Torsionally Balanced (TB) and Torsinally Unbalanced (TU) models are similar. A symmetric model is also used as a reference TB system. The design gravity loads of the

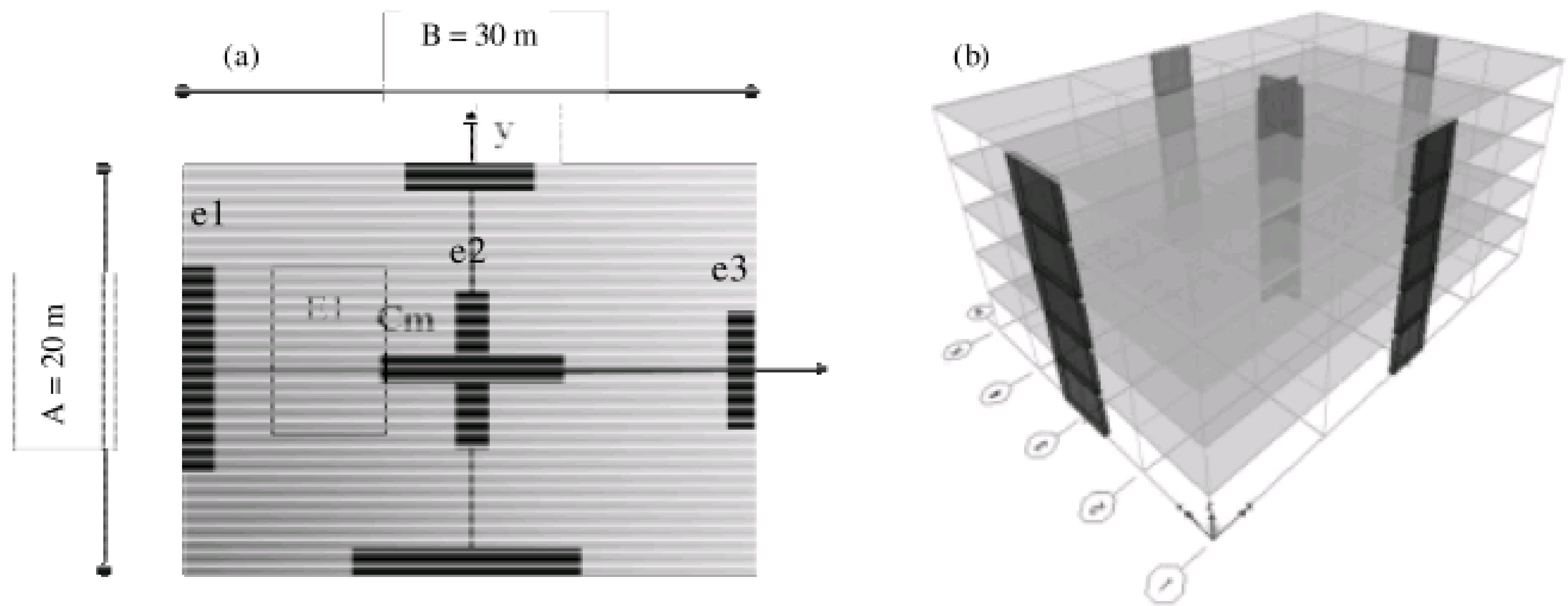

Fig. 1: (a) Plan and (b) 3D view of a TU building model

Table 1: Building models configuration

		Eccentricity (%)			Length (cm)		
Model No.	Description	Strength	Stiffness	Yield	Element 1	Element 2	Element 3
1	Symmetric	0.00	0.00	0.1418	300	300	300
2	Nearly stiffness symmetric	0.14	0.01	0.1418	420	300	180
3	Balance (~0.75 C_v–C_s)	0.11	-0.03	0.1418	420	300	180
4	Balance (~0.5 C_v–C_s)	0.07	-0.07	0.1418	420	300	180
5	Balance (~0.25 C_v–C_m)	0.04	-0.10	0.1418	420	300	180
6	Almost strength symmetric	0.00	-0.13	0.1418	420	300	180
7	De Stefano (~0.25 C_m–C_s)	-0.05	-0.17	0.1418	420	300	180
8	De Stefano (~0.5 C_m–C_s)	-0.14	-0.25	0.1418	420	300	180

TB system have been determined based on the Iranian standard 519 (2006). The design earthquake loads are calculated based on the Iranian Standard 2800 (2005). The design base shear is equal to 320 tones. The lateral strengths of all the TU systems are the same and equal to the lateral strength of the TB system. Lateral strengths of the models in the x or y-directions are the same. To generate TU models, the length of the left and the right walls in the y-direction are changed in a way that the distance between yield center (Center of LFREs yield displacement) of models were equal to 14.29% of the plan width in each story. In all TU systems the shape and geometry of the walls are similar; the length of the right element is 180 cm, left element is 420 cm and the length of all other elements is 300 cm (Table 1).

For producing strength eccentricity, the strength of the left side wall has been increased while the strength of the right side wall has been decreased in such a way that the total strength remains constant.

With these assumptions, the stiffness eccentricity can be calculated from the following equation:

$$e_s / B = \frac{-V_1 l_1 + V_3 l_3}{2 \sum_{i=1}^{3} V_i l_i} \tag{1}$$

where, B is the plan width, V_i and l_i are strength and length of element i, respectively.

In Eq. 1, it is assumed that stiffness of each shear wall is proportional to its strength and inversely related to the length. This assumption is based on the equation for yield displacement of a shear wall element proposed by Priestley and Kowalsky (1998).

Myslimaj and Tso (2002) showed that using different approaches for assigning strength to the two lateral load resisting elements of a building, the distance between centers of strength and stiffness remains almost equal to the distance between centers of mass and yield displacement.

To investigate the effects of different configurations of centers of stiffness and strength, behavior of a TB and seven TU models are examined in this study. All the TU models have similar geometric configurations, element sizes and lateral strengths. The unbalanced conditions of the TU models are created by an asymmetric distribution of strength between the left and right edge elements. Table 1 shows the characteristics of the TB model along with the seven TU models. Model number one is TB, the reference model, with uniform geometry, mass, strength and stiffness distributions in its plan. Model number 2 has its center of stiffness close to its center of mass.

Table 2: Far field earthquake records

Earthquake	Year	Magnitude (M)	Duration (sec)	PGA Y (g)	Site	Distance (km)	Soil
CAPE-MENDOCINO	1992	7.1 m	36	0.229	Shelter Cove Airport	33.80	B
Chi-Chi	1999	7.6 m	35	0.413	TCU047	33.01	B
Compano lucano	1980	6.9 mw	35	0.139	Mercato san servino	48.00	B
Manjil	1990	7.4 mw	25	0.184	Qazvin	49.00	B
Imperial Valley	1979	6.5 m	40	0.169	Cerro Prieto	26.50	B
Izmit	1999	7.6 mw	30	0.208	Gebze-arcelic	38.00	B
Kern county	1952	7.4 mw	25	0.175	Taft	41.00	B
N. Palm Springs	1986	6.0 m	20	0.228	San Jacinto	32.00	B
Northridge	1994	6.7 m	20	0.256	LA - Century	25.40	B
San Fernando	1971	6.6 m	20	0.324	Castaic	24.90	B
Whittier Narrows	1987	6.0 m	20	0.299	Union Oil	25.20	B
Loma Prieta	1989	6.9 m	25	0.233	Golden Gate Bridge	85.10	B
Northridge	1994	6.7m	20	0.404	Westmoreland	29.00	B
Chi-Chi	1999	7.6 m	35	0.204	CHY086	35.43	B
N. Palm Springs	1986	6.0 m	11.185	0.240	Hurkey Creek Park	34.90	B

Model 3 to 5 are the models introduced by Myslimaj and Tso (2002) as having balanced configurations. In all theses models, the center of mass is between centers of stiffness and strength. In model 3, the center of mass is located in an approximate distance of 0.75 C_v-C_s (distance between centers of strength and stiffness) from the center of strength. This distance changes to 0.5 and 0.25 of the distance between C_v and C_s for model 4 and 5, respectively. Model 6 is a model with symmetric strength distribution and unbalanced stiffness distribution. Model 7 and 8 are optimized models, as suggested by De Stefano *et al*. (1993) with the center of strength located between centers of mass (C_m) and stiffness (C_s). For model 7, the distance between C_v and C_m is almost equal to 0.25 of the distance between C_s and C_m, while for model 8 the center of strength is approximately in the middle of the distance between centers of mass and stiffness.

To compare performance of models in a given limit state, a probabilistic approach based on fragility curves has been used. Each point of the fragility curve for an assigned earthquake peak ground acceleration is defined by a relationship as below (De Stefano *et al*., 2004; Reinhorn *et al*., 2001).

$$\text{Fragility} = P[EDP > AC \mid IM] \tag{2}$$

where, IM is the earthquake intensity measure that in this study it is assumed as the peak ground acceleration and AC is corresponding acceptable criteria for the assumed limit state.

A normal statistical distribution is assumed for every Engineering Demand Parameter (EDP) in each specific ground motion intensity. To evaluate the probability of accedence from a specific limit state, the average and standard deviation of each EDP is calculated for the ensemble of 15 earthquake records. Then, using the cumulative distribution function of normal distribution, the probability of exceeding of each EDP from the given limit state is estimated.

Fifteen earthquake ground motion records have been selected for conducting dynamic nonlinear analysis. All the records are scaled to ten different peak ground accelerations from 0.05 to 0.6 g. The characteristics of ground motion records are shown in Table 2.

Dynamic nonlinear analysis of models are done using OPENSEES software (OpenSees, 2005). Five percent damping ratio for the first mode proportional to the mass is included in the analysis.

RESULTS AND DISCUSSION

Using the OPENSEES software, the nonlinear dynamic analysis are performed. All models analyzed for 15 two directional (x, y) farfield ground motions (Table 2). The force-deformation relationship of concrete shear walls assumed to be bi-linear model with post yield stiffness equal to 2% of the elastic stiffness As response parameters, diaphragm rotation, the maximum interstory drift, edge ductility demand and plastic rotation of shear walls are considered.

The interstory drift and plastic rotation of shear walls as obtained from nonlinear dynamic analysis have been compared with limit states recommended by FEMA356 (2000) for each performance level. For the immediate occupancy level, the maximum interstory drift suggested by FEMA 356 (2000) is 0.5% and the plastic hinge rotation is 0.005 radian, while for the life safety level these values are 1% and 0.01 radian and for the collapse prevention level these quantities are 2% and 0.015 radian, respectively.

Rotational responses: Diaphragm rotation is a suitable measure of torsional response of building models. The averages of maximum rotation of diaphragms during earthquake excitation for 15 farfield records are shown in Fig. 2. Models with LFRE in both principle directions of building are subjected to two directional earthquake

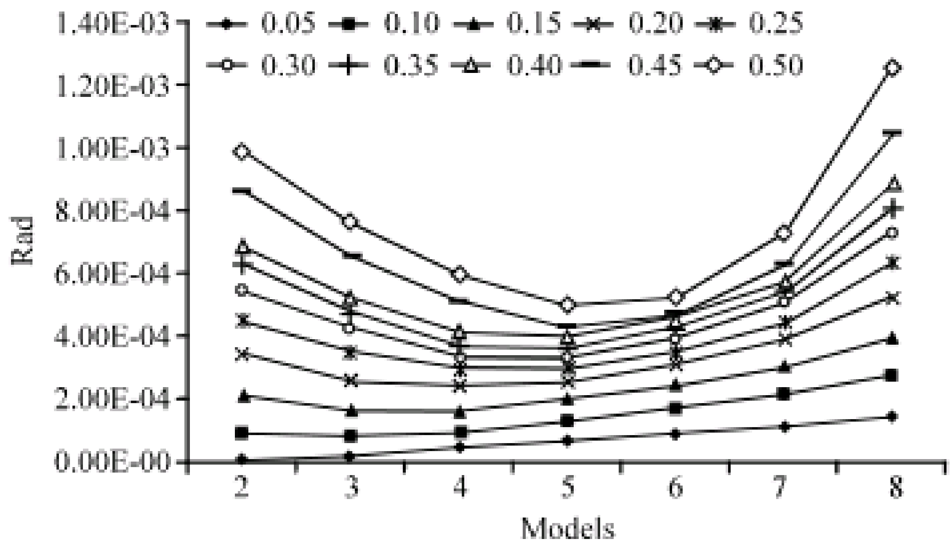

Fig. 2: Average of maximum diaphragm rotation for TU models for PGA 0.05 g till 0.6 g

records. As model 1 is torsionally balanced, it does not have rotational responses and is not represented in the figures.

According to Fig. 2, for PGA equal to 0.05 g where models dominantly respond in the linear range, the rotational response of building model 2 and 3 have the least value in comparison to the other TU models. These two are the models with the least stiffness eccentricity among the TU models. As the earthquake intensity increases, the rotational responses of all models increase. For a PGA equal to 0.1 g where buildings start to behave in the nonlinear range, the difference between three balance models (model 3-5) is clearly significant. For PGAs equal to 0.1 and 0.15 g, models 3 and 4 perform better but with higher earthquake intensities and more nonlinear behavior of models. The model with a smaller strength eccentricity performs better. In general, model 5 behaves better than other TU models. Model 8, in which the center of strength is located approximately in the middle of the center of rigidity and the center of mass, experiences the maximum number of rotational responses. This can be due to the fact that this model has the highest strength and stiffness eccentricity among TU models. Therefore, during earthquake excitations the largest torsional moments in the linear and nonlinear ranges were developed in this model.

The interstory drift: Although, the building diaphragm rotation is a good indicator of the torsional response but it is not a suitable index for building damages during earthquake. For nonstructural and structural damages, the interstory drift is a better indicator of damages. The maximum interstory drift of the eight selected building models are calculated and used to establish the fragility curves for three performance levels, the Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). In Fig. 3, the drift fragility curves of seven TU models are compared with each other and with the TB model (model 1). The drift fragility curves of Fig. 3 shows the CP, LS and IO levels based on the values proposed by FEMA 356 guide line.

As shown in Fig. 3a-c, the probability of accedence for the symmetric model is lower than the TU models. Interstory drift of TU models possesses similar trends to those of the building rotation except that for models with balance configuration, in particular, interstory drift fragility curves are similar. This can be contributed to the fact that these models are translational dominant models. Model 5 (thick curve on Fig. 3) has the least probability of accedence of the assumed limit states. After this model, model 4 for the lower limit state (IO) and model 6 for the higher limit states (CP) perform better.

Plastic rotation: Plastic hinge rotation is the damage measure used in different guidelines such as FEMA 356/357 (2000) and ATC40 (1996) as a main parameter for evaluation of structural members in nonlinear methods. In Fig. 4a-c, plastic rotation fragility curves of building models have been presented for far field ground motion records and IO, LS and CP limit states. Figure 4 shows that model 5 (presented with the thick line) outperforms the other models. Subsequently, model 6 with symmetric strength has the minimum probability of accedence of the assumed limit state among other asymmetric building models.

Between different models, model 2, which is a stiffness symmetric model and model 8 have the maximum probability of accedence of the assumed limit state. The difference between fragility curves for models with balance configuration in the case of plastic rotation increases in comparison to the case of interstory drift. This is due to a higher dependency of plastic rotation to the yield displacement distribution. Consequently, balance configuration that yields a good performance with regard to the displacement and interstory drift for plastic rotation, does not perform well especially for configurations with higher strength eccentricities. The balance model with a higher strength in the left side element (element with less yield displacement) and thus with less strength eccentricity (i.e., model 5), displays the best performance among TU models.

Ductility demand ratio: Ductility demand ratio is a damage parameter that has been widely applied to asymmetric buildings in previous studies. This parameter is related to both displacement and yield displacement of elements. Thus, it can be expected that the corresponding results significantly differ from those of other EDPs.

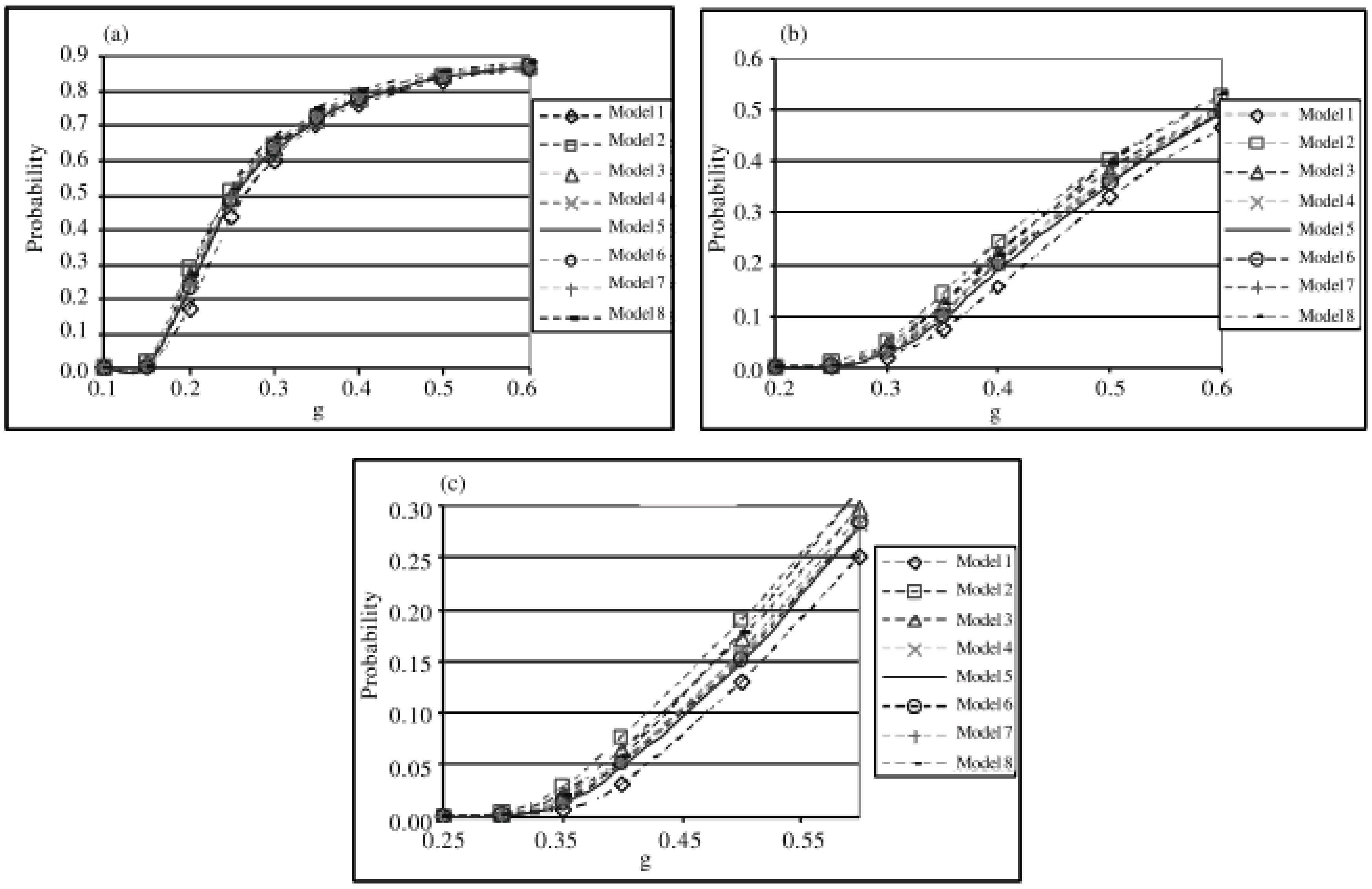

Fig. 3: Drift fragility curves for (a) IO, (b) LS and (c) CP limit states for TB and TU models

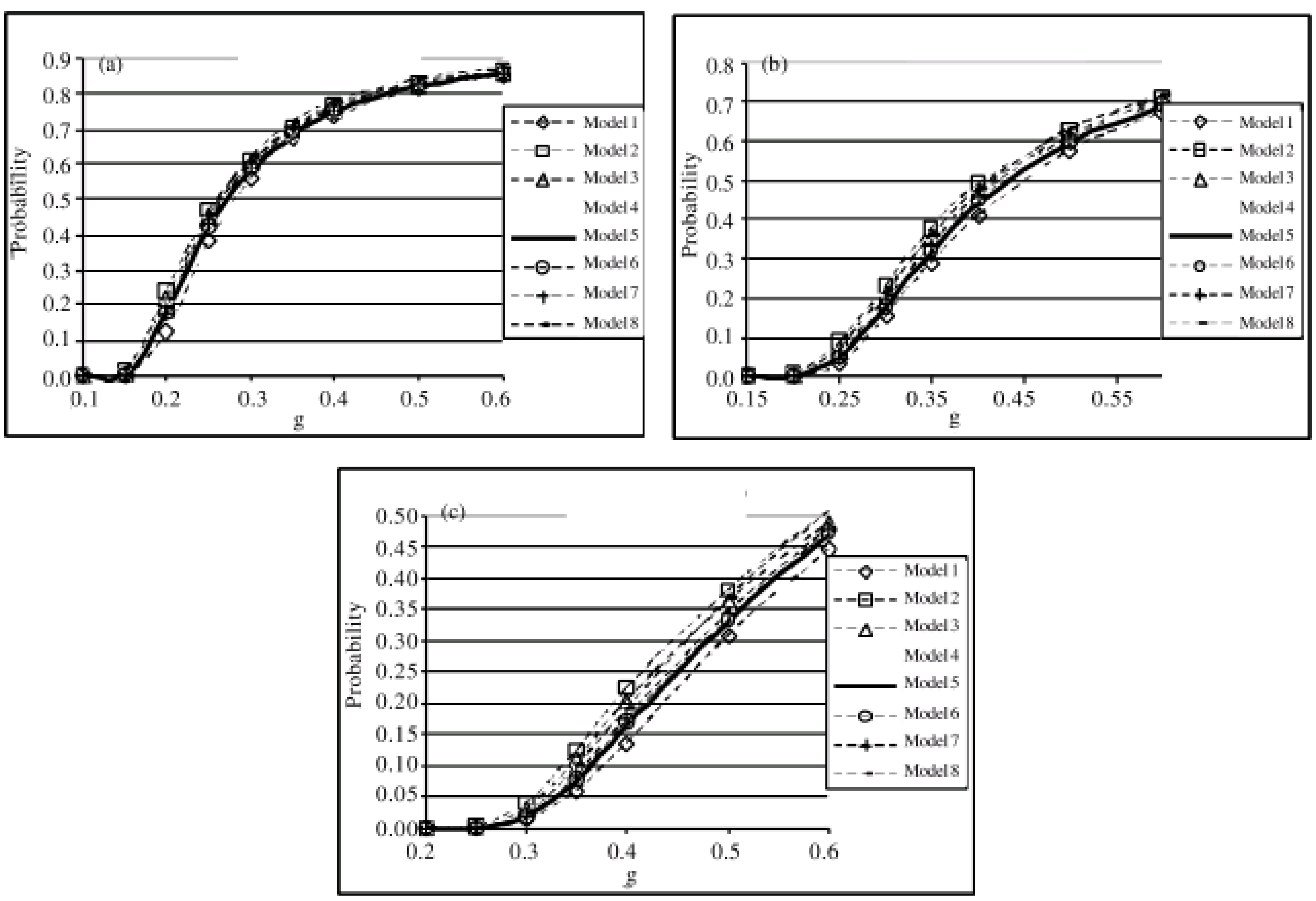

Fig. 4: Plastic rotation fragility curves for (a) IO, (b) LS and (c) CP limit states for TB and TU models

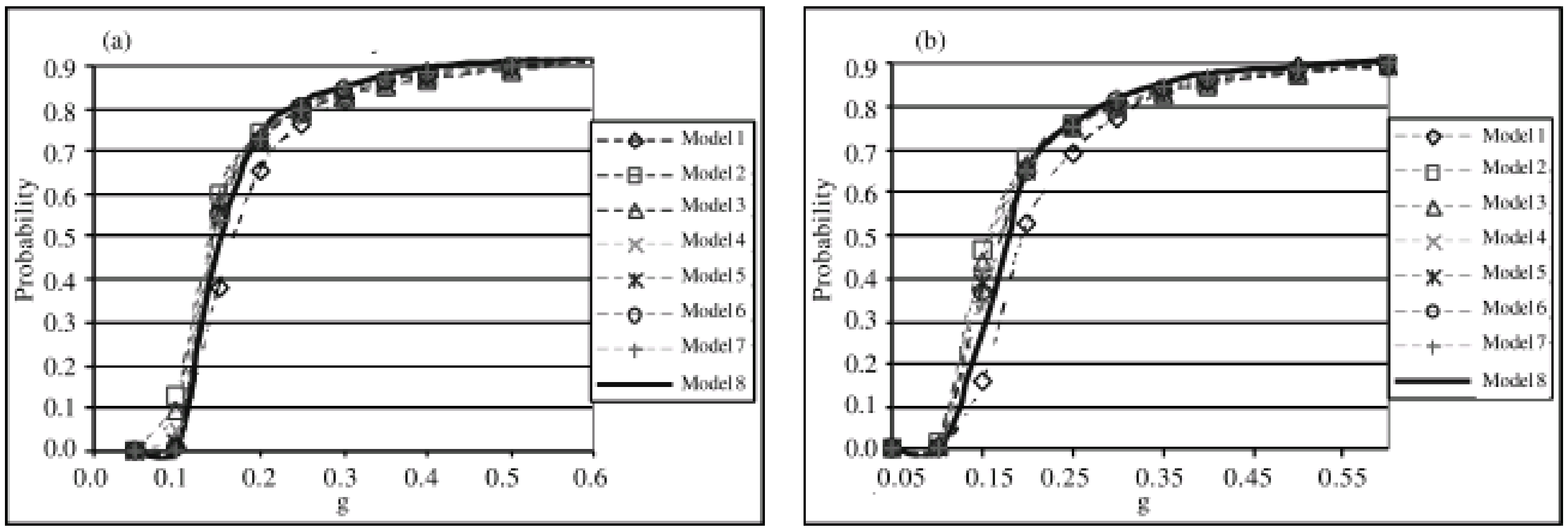

Fig. 5: Ductility demand fragility curves for ductility ratio limit state equal to 7 for TB and TU models, (a) ductility ratio 4.5 and (b) ductility ratio 6

In Fig. 5, ductility demand fragility curves of selected models are shown. In Fig. 5, fragility curves are derived based on models responses to an ensemble of 2D farfield earthquake excitations. The limit state for ductility demand is assumed to be equal to 6 for collapse prevention level 4.5 for the life safety level. Performance of building models for this damage measure parameter differs significantly compared to the interstory drift and plastic rotation of LFREs. In this case, fragility curves of TU models show higher discrepancy and their probability of accedence of assumed limit state in similar levels is larger than the interstory drift and plastic rotation parameter. This is due to a high dependency of this parameter to the yield displacement of LFREs. This dependency reduces the randomness (mid part slope of fragility curves) of ductility fragility curves in comparison to other two EDPs.

In all TU models, the left side element yield displacement is lower than the right side element. Therefore, with an increase of strength in the left side performance of models improves and model 8 performs the best between all TU models. In this model, the center of strength is located in the middle of centers of mass and stiffness. This is the model suggested by earlier researchers for building with K-type elements (Destefano *et al*., 1993, Rutenberg, 1992). Among fragility curves of TU models, this model shows minimum difference with the TB model ductility fragility curves.

CONCLUSIONS

In this study, using fragility representation, the performance of symmetric and asymmetric five story shear models affected by far field ground motions was studied. Diaphragm rotation, interstory drift, plastic rotation and ductility demand were selected as damage measure parameters and fragility curves for each model were generated for two component far field ground motion excitations. Based on the results of this study the following conclusions can be drawn:

- In torsionally stiff buildings, for each specific pattern of strength distribution (each of the 8 models) similar trends of responses were observed for LS and CP limit states. In lower limit states such as IO, the effect of strength distribution on improvement of building performance decreases (less difference in the results of different models) and in higher limit states such as CP, the effect of strength distribution increases
- Performance of different strength distributions and configurations of centers of strength, stiffness and mass differ significantly among different response parameters. For interstory drift and plastic rotation parameters, models with lower diaphragm rotation perform better. However, this situation is not valid for the ductility demand parameter
- Based on results drawn from translational dominant models of this study, for plastic rotation and interstory drift responses, models with balance configuration and strength eccentricity equal to the one forth of C_v-C_s perform better. For the ductility demand, models with center of strength in the middle of centers of mass and stiffness perform better
- For limit states where a moderate to high amount of damage is expected (e.g., LS and CP), the proper configuration of centers are similar to the case of item 4. However, in limit states that a low amount of damage is expected (e.g., IO), models with a low stiffness eccentricity perform better

More exact configurations of centers for each important common demand parameters were identified and studied. These configurations can be used as a proper way to reduce the adverse torsional effect in design or rehabilitation of asymmetric buildings. These configurations can also be used as a new reference point for identifying acceptable code design eccentricity. For each level of eccentricity, using these configurations as the reference, the designer is capable of predicting the effect of torsional responses on performance of an asymmetric building.

NOTATIONS

AC	=	Acceptable criteria
B	=	Plan width
C_m	=	Center of mass
C_P	=	Collapse prevention
C_s	=	Center of stiffness
C_v	=	Center of strength
EDP	=	Engineering Demand Parameters
IM	=	Intensity Measure
IO	=	Immediate Occupancy
l_i	=	Length of element (i)
LFRE	=	Lateral force resisting elements
LS	=	Life Safety
PGA	=	Peak Ground Acceleration
TB	=	Torsionally Balanced
TU	=	Torsionally Unbalanced
V_i	=	Strength of element (I)

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