THE EFFECT OF DIFFERENT STIFFNESS AND STRENGTH DISTRIBUTIONS ON THE SEISMIC PERFORMANCE OF PLAN-ASYMMETRIC SINGLE-STORY RC SHEAR WALL-FRAME BUILDINGS

Hamzeh SHAKIB\(^1\), Sahar MOHAMMADZADEH OSALU\(^2\)

ABSTRACT

The seismic performance of buildings with non-uniform plan distribution of stiffness and strength, may be significantly different compared to symmetric buildings. Previous studies have shown that both distribution and intensity of response parameters in plan-asymmetric buildings are dependent on their stiffness and strength distributions. The locations of strength and stiffness centers relative to mass center provide suitable metrics for strength and stiffness distributions. In this study, the behavior of plan-asymmetric RC shear wall-frame buildings with different locations of strength and stiffness centers respect to the mass center is investigated at various earthquake intensity levels. Diaphragm rotation, lateral displacement and lateral acceleration of buildings are selected as studied responses. For an important class of widely structural elements such as reinforced concrete shear walls and moment resisting frames, stiffness is a strength dependent parameter. These elements are called D-type. In this study, the seismic behavior of asymmetric structures with D-type lateral force resisting elements is investigated. It is concluded that, the effects of strength distribution and configuration of centers of strength, stiffness on each response parameter differs significantly depending on the parameter under study and the earthquake intensity. Also with increasing the earthquake intensity and amount of nonlinear behavior intensity, the sensitivity of displacement and acceleration responses to distribution of stiffness and strength has decreased.

*Keywords: RC shear wall-frame building; Plan-asymmetry; Torsion; Strength distribution*

1. INTRODUCTION

Experiences in past earthquakes such as Mexico City 1985 have shown that asymmetric buildings usually sustained more damage than symmetric buildings due to lateral-torsional coupling of their dynamic response (Esteva 1987). The main causes of asymmetry in structures can be related to irregular stiffness and strength distributions of their lateral load resisting elements in the presence of an approximately uniform distribution of floor mass. The recognition of this sensitivity has led the researchers to concentrate their studies on the earthquake characteristics, evaluation of structural parameters and validity of system models (Kan and Chopra 1977, Kan and Chopra 1981, Tasnimi and Rezazadeh 2012). In the seismic design of structures, one task after the base shear has been established is the distribution of design strength among lateral force-resisting elements (LFRE). In order to mitigate the effect of torsion during the earthquakes, most seismic codes of the world provide design guidelines for strength distribution based on the traditional perception that element stiffness and strength are independent parameters. These LFREs are called K-type elements (Tso and Ying 1992).

\(^1\)Professor, School of Civil & Environmental Engineering , Tarbiat Modares University, Tehran, Iran, shakib@modares.ac.ir
\(^2\)PhD Student, School of Civil & Environmental Engineering , Tarbiat Modares University, Tehran, Iran, sahar.mohammadzade@modares.ac.ir
Researches in the past ten years (e.g., Aschheim 2002) revealed that for many LFREs such as shear walls and moment resisting frames, their stiffness is depended on their strength and will be modified during strength assignment. These LFREs are called D-type elements (Tso and Smith 1999). This implies that the lateral stiffness distribution in a D-type system cannot be defined prior to the assignment of the elements’ strength. Therefore, both stiffness and strength eccentricity are important parameters affecting the seismic response of asymmetric reinforced concrete systems (Priestley et al. 2007).

In traditional approach, it is assumed that stiffness of an LFRE can be estimated independent of its strength. As a result of this assumption, the stiffness distribution is considered as known prior to the allocation of strength. Since the stiffness distribution is as known, the center of stiffness (CS) and the stiffness eccentricity, e_s, which is defined as the distance between the center of mass (CM) and (CS), can be readily determined. For this reason, e_s has become the parameter commonly used in the torsional provisions. Although the studies before 1997 mainly used this approach but it is only proper for LFREs, in which the stiffness and strength are independent.

Another approach has been suggested for stiffness-strength dependent LFREs by Paulay (1997) based on the plastic mechanism analysis, in which he considered the behavior of single story building for this type of element, and concluded that the current lateral strength distribution of seismic code is inappropriate (Myslimaj and Tso 2002). Tso and Myslimaj (2003) studied the effect of different configurations of centers of mass, rigidity, and strength on response of a single-story asymmetric structure. They concluded that both strength and stiffness eccentricities have important roles on torsional response of asymmetric structures. They proposed that the best configuration of center of mass, strength, and stiffness is a configuration in which the center of mass is between centers of strength and stiffness (balance configuration). According to their study, balance configuration will improve the inter-story drift and diaphragm rotational responses of a building but it can cause an increase in ductility demand of elements at the stiff side of the structure. They also investigated the stiffness, strength, mass, and yield displacement eccentricities and their relationship with each other. Based on their studies on an un-symmetric building with two LFREs and center of mass located on the floor center, they concluded that the distance between centers of strength and rigidity is almost equal to yield displacement eccentricity. Shakib and Ghasemi (2007) considered different criteria for minimizing the torsional response of asymmetric structures under near-fault and far-fault excitations. They concluded that in near-fault ground motions, the minimum rotational response, considering the actual behavior method, could be achieved when stiffness and strength centers are located on the opposite sides of the mass center. By increasing pulse period of fault-normal component, displacement ductility demand increases as well. Aziminejad and Moghadam (2009) considered different strength distributions in multi-story asymmetric building and studied the effect of different distribution strategies on building response in the performance-based design approach. They concluded that the model with smaller strength eccentricity performs better. However, in general, optimum strength eccentricity is a function of the selected damage index (Aziminejad and Moghadam 2010).

Most of these studies carried out by employing simple models using shear type LFREs. These models have been used in most of studies carried out on asymmetric buildings since they allow clarifying the influence of governing parameter and deriving effective design criteria (De Stefano and Pintucchi 2002). Stathopoulos and Anagnostopoulos (2003) examined the limitation of a single-story shear building model with K-type element by comparing its results with a more realistic frame model with idealized plastic hinges. They find substantial differences between results of simple shear model with frame model and concluded that simple shear models are not appropriate to assess code provisions for frame type buildings. However, in studies by Perus and Fajfar (2005) on a single-story building containing shear type LFREs and Marusic and Fajfar (2005) on multistory frame types buildings, the findings on the torsional behavior of a single-story shear type building was confirmed by results of multi-story frame type buildings regular in height.

Most of the studies have carried out by employing simple models with shear wall systems. While in most of the realistic systems, the frames are also involved in lateral load resisting. Because of that, in this study the seismic performance of plan-asymmetric RC buildings with shear wall-frame systems has been evaluated. The present study tries to extend some aspects of the previous research to
determine more precisely configuration of centers in asymmetric RC shear wall-frame buildings. In most previous studies, a bi-linear model has been used for modelling of nonlinear behavior of elements. In this study more realistic building models with concentrated plasticity (Plastic hinge) are used by considering all degradation sources including the loading and reloading stiffness, peak-strength and hardening zone stiffness degradation effects in each cycle of response.

2. CHARACTERISTICS OF WALL-FRAME BUILDINGS WITH D-TYPE ELEMENTS

Research concerning the shear wall and rigid connection frames has shown that the yield displacement of the cited LFREs (D-type elements) is a function of their geometric and material characteristics and is not dependent on element strength or axial force.

For cantilever shear wall with different condition of loading the following equation can be used for yield displacement (Paulay 2001):

\[
\Delta_{yw} = \frac{C\varepsilon_y h^2}{l_w}
\]  

(1)

In this equation, \(\varepsilon_y\) is bar yield strain, \(l_w\) is the wall depth and \(C\) is a coefficient related to the shape of lateral force distribution and element section. For a cantilever shear wall with lateral load acting on the top of the wall (as in a single-story building), the value of \(C\) is equal to 0.66.

Also, the yield displacement of the frame at roof \(\Delta_{yf}\) may be taken as:

\[
\Delta_{yf} = \theta_{yf} h
\]  

(2)

Where \(h\) is the height of structure, and \(\theta_{yf}\) is the inter-story yield rotation of the frames. As recommended by Priestley (2007) \(\theta_{yf}\) may be obtained from:

\[
\theta_{yf} = 0.5 \varepsilon_y \frac{l_b}{D_b}
\]  

(3)

Where \(l_b\) is the length of the beam and \(D_b\) its depth.

After calculating yield displacements of elements, the center of yield displacement is calculated according to the distance of each element from the center of mass.

The designer has the freedom to select the relative strengths of the wall and the frame to withstand the demand base shear \(V_{total}\). The strength of the components may be assigned arbitrarily, and this freedom can be exploited to achieve more rational and economical design. Once such assignment is made, the global yield displacement of the wall-frame system can be obtained from:

\[
\Delta_y = \frac{V_{total}}{V_w/\Delta_{yw} + V_f/\Delta_{yf}}
\]  

(4)

Where \(V_w\) and \(V_f\) are the wall and frame strengths, respectively.

After calculation of the yield displacement of each element the stiffness of a LFRE \((K_i)\) is estimated by the following equation:

\[
K_i = \frac{V_i}{\Delta_{yi}}
\]  

(5)
3. STRUCTURAL SYSTEM

A single-story RC shear wall-frame building is considered in this study (Figure 1). The generic building model consists of a rigid diaphragm that is supported by two reinforced concrete shear walls and two moment-frames in y-direction, and two shear walls and four moment-frames in x-direction, as shown in the figure. With the idea that the yield displacements of reinforced concrete systems can be determined from architectural drawings, the yield displacement distribution of the structure is known. The asymmetry of such a distribution is characterized by the location of the center of the yield displacement in relation to mass center. Using plastic mechanism analyses on a number of example structures (Paulay 2001) and focusing on the displacement ductility demand on the elements, it was concluded that, within rational limits, strength can be assigned to the elements in any way that suits the designer’s intentions. A desirable strength distribution leads to establish strength centers that with due attention to the relation of stiffness to strength, different stiffness centers are created.

![Figure 1. Arrangement of resisting elements in (a) plan and (b) three dimension](image)

In this study, Buildings are symmetric in x-direction and asymmetric in y-direction. The asymmetry in the y-direction is produced by changing yield displacement and strength of LFREs in the y-direction. According to Tso and Myshlimaj (2003), a yield displacement distribution-based approach has been used for strength assignment to lateral force-resisting elements. All Asymmetric models have a yield displacement eccentricity equal to 6% of the plan width. Therefore, strength distribution is determined by the following two conditions: (a) the strength eccentricity is related to the yield displacement eccentricity by the relation “$e_r = \beta e_D$”; (b) the radius of gyration for both strength and yield displacement are equal. In the above equation, $e_r$ is the strength eccentricity, $e_D$ is the yield displacement eccentricity and $\beta$ is the parameter which can be chosen by the designer (Tso and
The design gravity load of symmetric model has been considered 150 tonf. The design earthquake load is calculated based on Iranian Standard 2800 (2015). The design base shear is equal to 24 tonf. This value has been distributed between walls and moment frames with the ratio 3 to 1 in each direction.

The lateral strengths of all asymmetric models are the same and equal to the lateral strength of the symmetric model. Lateral strengths of the models in the x or y-directions are the same. To generate asymmetric models the yield displacements of LFREs in the y-direction are changed. To change the yield displacements, changing the yield strain of reinforcements has been used. Table 1 shows the values of stiffness and strength in LRFEs of model in y-direction. In this table $V_{wL}$, $V_{wR}$ and $K_{wL}$, $K_{wR}$ indicate strength of wL and wR and stiffness of wL and wR, respectively. Also $V_{fL}$, $V_{fR}$ and $K_{fL}$, $K_{fR}$ represent strength of frameL and frameR and stiffness of frameL and frameR, respectively.

Shown in Table 2 are general properties of the building models. In this table $e_s$ and $e_r$ indicate stiffness and strength eccentricities and V and K represent total strength and stiffness of the models respectively. In all models the center of mass has been considered at the center of rigid diaphragm. Figure 2 summarizes some characteristics of the symmetric and seven asymmetric models.

**Table 1. The values of stiffness and strength of LFREs in y direction**

<table>
<thead>
<tr>
<th>$\beta$</th>
<th>$V_{wL}$ (tonf)</th>
<th>$V_{wR}$ (tonf)</th>
<th>$V_{fL}$ (tonf)</th>
<th>$V_{fR}$ (tonf)</th>
<th>$K_{wL}$ (tonf/m)</th>
<th>$K_{wR}$ (tonf/m)</th>
<th>$K_{fL}$ (tonf/m)</th>
<th>$K_{fR}$ (tonf/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Symmetric model</td>
<td>9.0</td>
<td>9.0</td>
<td>3.0</td>
<td>3.0</td>
<td>2000.0</td>
<td>2000.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>-0.5</td>
<td>10.2</td>
<td>7.8</td>
<td>3.25</td>
<td>2.75</td>
<td>3090.91</td>
<td>1368.42</td>
<td>130.0</td>
<td>78.57</td>
</tr>
<tr>
<td>-0.25</td>
<td>9.6</td>
<td>8.4</td>
<td>3.13</td>
<td>2.88</td>
<td>2909.09</td>
<td>1473.68</td>
<td>125.0</td>
<td>82.14</td>
</tr>
<tr>
<td>0.0</td>
<td>9.0</td>
<td>9.0</td>
<td>3.0</td>
<td>3.0</td>
<td>2727.27</td>
<td>1578.95</td>
<td>120.0</td>
<td>85.71</td>
</tr>
<tr>
<td>0.25</td>
<td>8.4</td>
<td>9.6</td>
<td>2.88</td>
<td>3.13</td>
<td>2545.45</td>
<td>1684.21</td>
<td>115.0</td>
<td>89.29</td>
</tr>
<tr>
<td>0.5</td>
<td>7.8</td>
<td>10.2</td>
<td>2.75</td>
<td>3.25</td>
<td>2363.64</td>
<td>1789.47</td>
<td>110.0</td>
<td>92.86</td>
</tr>
<tr>
<td>0.75</td>
<td>7.2</td>
<td>10.8</td>
<td>2.63</td>
<td>3.38</td>
<td>2181.82</td>
<td>1894.74</td>
<td>105.0</td>
<td>96.43</td>
</tr>
<tr>
<td>1.0</td>
<td>6.6</td>
<td>11.4</td>
<td>2.50</td>
<td>3.50</td>
<td>2000.0</td>
<td>2000.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
</tbody>
</table>

**Table 2. General properties of building models**

<table>
<thead>
<tr>
<th>$\beta$</th>
<th>$V_{total}$ (tonf)</th>
<th>$K_{total}$ (tonf/m)</th>
<th>$e_s$ (m)</th>
<th>$e_r$ (m)</th>
<th>$T_1$ (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Symmetric model</td>
<td>24.0</td>
<td>4200.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.38</td>
</tr>
<tr>
<td>-0.5</td>
<td>24.0</td>
<td>4668.0</td>
<td>-0.43</td>
<td>-1.17</td>
<td>0.360</td>
</tr>
<tr>
<td>-0.25</td>
<td>24.0</td>
<td>4590.0</td>
<td>-0.21</td>
<td>-0.99</td>
<td>0.363</td>
</tr>
<tr>
<td>0.0</td>
<td>24.0</td>
<td>4512.0</td>
<td>0.00</td>
<td>-0.81</td>
<td>0.366</td>
</tr>
<tr>
<td>0.25</td>
<td>24.0</td>
<td>4434.0</td>
<td>0.21</td>
<td>-0.62</td>
<td>0.369</td>
</tr>
<tr>
<td>0.5</td>
<td>24.0</td>
<td>4356.0</td>
<td>0.43</td>
<td>-0.42</td>
<td>0.372</td>
</tr>
<tr>
<td>0.75</td>
<td>24.0</td>
<td>4278.0</td>
<td>0.64</td>
<td>-0.21</td>
<td>0.376</td>
</tr>
<tr>
<td>1.0</td>
<td>24.0</td>
<td>4200.0</td>
<td>0.85</td>
<td>0.00</td>
<td>0.379</td>
</tr>
</tbody>
</table>
For nonlinear analysis of structures, the ‘concentrated plasticity’ approach (Bozorgnia and Bertero, 2004) is utilized for mathematical modeling. In this approach, the mathematical model of the structure consists of two nonlinear concentrated springs at the elements ends and a linear middle part for all frame elements and one nonlinear concentrated spring at the base and a linear part for all shear walls. As discussed by Ibarra et al. (2005), Haselton (2006) and Zareian and Krawinkler (2007), concentrated plasticity models are used for collapse-state analysis of structures by considering all degradation sources including the loading and reloading stiffness, peak-strength and hardening zone stiffness degradation effects in each cycle of response. Ibarra et al. (2005) proposed a hysteretic model applicable to nonlinear modeling of RC elements to assess their post-peak behavior based on the kinematic hardening rules. The model is known as peak-oriented hysteretic model. In Figure 4, the monotonic backbone curve associated with the model is shown.

4. GROUND MOTIONS

In this study, seven bi-directional far-field ground motions are used for the time history analyses. All records are selected from PEER Ground Motion Database (PEER, 2009). Table 3 shows the specifications of the ground motion records.
Table 3. Specifications of ground motion records

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake Name</th>
<th>Station Name</th>
<th>Magnitude</th>
<th>$S_a(T_{1x})$ (g)</th>
<th>$S_a(T_{1y})$ (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Coalinga-01</td>
<td>Parkfield - Cholame 2WA</td>
<td>6.36</td>
<td>0.2669</td>
<td>0.3496</td>
</tr>
<tr>
<td>2</td>
<td>Morgan Hill</td>
<td>Foster City - APEEL 1</td>
<td>6.19</td>
<td>0.1899</td>
<td>0.1401</td>
</tr>
<tr>
<td>3</td>
<td>Superstition Hills-01</td>
<td>Salton Sea Wildlife Refuge</td>
<td>6.54</td>
<td>0.1818</td>
<td>0.2622</td>
</tr>
<tr>
<td>4</td>
<td>Northridge-01</td>
<td>Pacific Palisades - Sunset</td>
<td>6.69</td>
<td>0.4144</td>
<td>1.0784</td>
</tr>
<tr>
<td>5</td>
<td>Coalinga-01</td>
<td>Parkfield - Fault Zone 1</td>
<td>6.36</td>
<td>0.274</td>
<td>0.3585</td>
</tr>
<tr>
<td>6</td>
<td>Northridge-01</td>
<td>Carson - Water St</td>
<td>6.69</td>
<td>0.205</td>
<td>0.1774</td>
</tr>
<tr>
<td>7</td>
<td>Loma Prieta</td>
<td>SF Intern. Airport</td>
<td>6.93</td>
<td>0.6863</td>
<td>0.8707</td>
</tr>
</tbody>
</table>

5. RESULTS OF ANALYSES

To understand the influence of the design parameter $\beta$ on seismic performance, the response of the asymmetric structures under two directional ground motions will be examined using the OpenSees software. Five percent damping ratio is included in the analysis. In order to investigate the effect of intensity of the records on the seismic performance, all records are scaled to three values of $S_a(T_1)=0.156g$, $S_a(T_1)=0.94g$ and $S_a(T_1)=1.4g$. These values of $S_a(T_1)$ correspond to service level earthquake, standard design level earthquake and a severe earthquake, respectively.

The investigated seismic responses consist of maximum lateral displacement and maximum lateral acceleration of mass center, stiff side and flexible side of the diaphragm in the asymmetric direction and maximum rotation of the diaphragm. For presentation of figures, first the given seismic response is calculated for each ground motion record at each $S_a(T_1)$ and then the average values for seven ground motion records in each $S_a(T_1)$ are obtained.

5.1 Effect of the design parameter $\beta$ on rotation of diaphragm

Diaphragm rotation is a suitable measure of torsional response of building models. The averages of maximum rotation of diaphragms during earthquake excitation for 7 bi-directional records has been shown in Figure 4.

![Figure 4. Variation of average value of Max U0 versus $\beta$ for different $S_a(T_1)$ levels](image)

As a general observation of Figure 4, by increasing $S_a(T_1)$ levels, the rotational response and its sensitivity to the variation of $\beta$ increases.
According to Figure 4, for $S_a(T_1)$ equal to 0.156g where models dominantly respond in the linear range, the rotation response of buildings model with $\beta=0.75$ and $\beta=1.0$ have the least value in comparison to other models. These two are the models with the least stiffness eccentricity among the models.

For $S_a(T_1)$ equal to 0.94g and 1.4g where buildings are more nonlinear behavior rather than $S_a(T_1)= 0.156g$, the minimum rotational response is happened for $\beta=0.25$ in which the centers of strength and stiffness are located at the opposite side of mass center but the value of strength eccentricity ($e_r$) is smaller than the value of stiffness eccentricity ($e_s$). For higher $S_a(T_1)$ levels (1.4g), the rotational response of the model with a minimum stiffness eccentricity is higher than other models.

### 5.2 Effect of the design parameter $\beta$ on lateral displacement of diaphragm

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 inter-story drift is a better indicator. For this reason, maximum displacement of the models is investigate in this part of the paper. Maximum displacement of mass center, left side (stiff side) and right side (flexible side) of the diaphragm and the ratio of these responses to those of symmetric model against different values of $\beta$ parameter has been shown in Figure 5 (a)-(c).

As it can be seen in the figure, for $S_a(T_1)$ equal to 0.156g and 0.94g, a strength distribution shift toward the right would result in an increase in lateral displacement on the left side and a reduction on the right side. However, the lateral displacement of the center is almost insensitive to the strength distributions. for $S_a(T_1)$ equal to 0.156g where models behave mainly in the linear range, the model with $\beta=0.75$ performs better. In this model the centers of strength and stiffness are located at the opposite side of mass center but the value of stiffness eccentricity ($e_s$) is smaller than the value of strength eccentricity ($e_r$).

However, as earthquake intensity increases to $S_a(T_1)=0.94g$ and building models behave in the nonlinear range, the model with $\beta=0.25$ with balanced distribution of stiffness and strength with a lower value of strength eccentricity than the stiffness eccentricity experience less lateral displacement. According to Figure 5, the trends of responses in two graphs for $S_a(T_1)$ equal to 0.156g and 0.94g are similar, but in the case of $S_a(T_1)=1.4g$ with more nonlinear effects the results have an irregularity. With regard to the value of displacement demand of the models (0.17m to 0.2m), the models are expected to experience collapse condition in this range of the lateral displacement. As shown in the figure, in this case the lateral displacement demand would be minimized in $\beta=-0.5$.

It can be resulted according to the values of $U_y/U_{ysym}$ (the ratios of maximum displacement in asymmetric models to symmetric model), that with increasing the earthquake intensity, the sensitivity of the lateral displacement to the variation of stiffness and strength configurations decreases.
(b). $S_a(T_1)=0.94g$

(c). $S_a(T_1)=1.4g$

Figure 5. Variation of average value of Max $U_y$ and $U_y/U_{sym}$ versus $\beta$ for different $S_a(T_1)$ levels

5.3 Effect of the design parameter $\beta$ on lateral acceleration of diaphragm

Since lateral acceleration of buildings is important for secondary systems, it is necessary to find a proper distributions for stiffness and strength of structures for minimizing lateral acceleration.

Shown in Figure 6 (a)-(c) are maximum acceleration of mass center, stiff side and flexible side of the diaphragm and the ratio of these responses to those of symmetric model against different values of $\beta$ parameter.

As shown in the figure, similar to the case of lateral displacement, the sensitivity of the lateral acceleration of mass center to the variation of $\beta$ is very low. However, increasing $\beta$ (a strength distribution shift toward the right) would result in an increase in lateral acceleration on the left side and a reduction on the right side.

For $S_a(T_1)=0.156g$, because of linear behavior of the models, the optimum configuration is the configuration with balanced distribution of stiffness ($\beta=1.0$).
(a). $S_a(T_1)=0.156g$

(b). $S_a(T_1)=0.94g$

(c). $S_a(T_1)=1.4g$

Figure 6. Variation of average value of Max $a_y$ and $a_y/a_{ysym}$ versus $\beta$ for different $S_a(T_1)$ levels

At higher earthquake intensities (0.94g and 1.4g), because of more nonlinear effects, the asymmetric model with $e_r=0.0$ ($\beta=0.0$) performs the best among all examined cases.

As indicated in right plots of the figure, in the case of $S_a(T_1)=0.156g$, with an inappropriate distribution of stiffness and strength, the lateral acceleration of asymmetric model has increased by 35 percent. Also, the lateral acceleration of asymmetric model with $\beta=-0.5$ is higher than $\beta=1.0$. However, with increasing the intensity level of earthquake the lateral acceleration of asymmetric model with $\beta=-0.5$ will be lower than $\beta=1.0$. 
6. CONCLUSION

In this study, the seismic performance of asymmetric single-story RC wall-frame buildings are studied using nonlinear dynamic analyses. Diaphragm rotation, lateral displacement and lateral acceleration were selected as studied parameters. Based on the results of this study the following conclusions can be drawn.

1. The effects of strength distribution and configuration of centers of strength, stiffness on each response parameter differs significantly depending on the parameter under study as well as the earthquake intensity.

2. With increasing the earthquake intensity and amount of nonlinear behavior intensity, the sensitivity of lateral displacement and acceleration responses to parameter $\beta$ (distribution of stiffness and strength) decreases. But the sensitivity of diaphragm rotation increases.

3. For diaphragm rotation and lateral displacement, in lower earthquake intensity, model with the so-called balance configuration, i.e., the mass center is located between the centers of stiffness and strength and with $\beta=0.75$ ($e_s>e_r$), performs better. But in moderate intensities, balance configuration with $\beta=0.25$ ($e_s>e_r$) has the best performance in decreasing these responses. During intensive earthquakes $\beta=0.25$ and $\beta=-0.5$ lead to minimum diaphragm rotation and lateral displacement, respectively.

4. For lateral acceleration, in lower earthquake intensity due to low nonlinear behavior effects, the model with zero stiffness eccentricity ($\beta=1.0$) performs better. However, in higher earthquake intensity the model with zero strength eccentricity ($\beta=0.0$) performs better.

5. Strength distribution and configuration of centers have a minor effect on controlling the response of mass center of structures.

6. As already mentioned, the inter-story drift is a better indicator for nonstructural and structural damages. For this reason and according to the results of this parameter, the models with balance configurations are preferred as better configurations of centers.

7. REFERENCES


