SEISMIC RELIABILITY OF TUNNEL FORM CONCRETE BUILDINGS SUBJECTED TO ACCIDENTAL TORSION: A CASE STUDY

Vahid MOHSENIAN1, Soheil ROSTAMKALAE2, Abdoreza S. MOGHADAM3

ABSTRACT

The results of the eigenvalue analyses of many tunnel form concrete buildings represent primacy of torsional modes to transitional modes leading to torsionally flexible behavior of those buildings. Considering the regularity provisions for tunnel form buildings in plan, it appears that probable eccentricity of mass and stiffness caused by asymmetric distribution of live loads, and construction defects in concreting and curing is the most likely type of irregularity in plan of such buildings. In this study, seismic sensitivity of tunnel form concrete buildings to different configurations of centers of mass and stiffness in plan is evaluated and for each arrangement, the performance level of the building in design basis earthquake (475 years return period) and maximum considered earthquake (2475 years return period) has been determined. The results show that the mass eccentricity has more considerable effects on responses compared to stiffness eccentricity. Moreover, the most critical configuration is the one with centers of mass and stiffness both transferred in one direction. The study also found that buildings with tunnel form system have desirable seismic reliability, as lateral load bearing elements were maintained the immediate occupancy performance level in different configurations of centers of mass and stiffness.

Keywords: Tunnel form system; Mass eccentricity; Stiffness eccentricity; Fragility analysis;

1. INTRODUCTION

According to existing studies and documentation, asymmetric buildings have lower seismic reliability than symmetric buildings. Asymmetric distribution of mass in diaphragms as well as asymmetric distribution of stiffness in lateral load bearing elements is one of the factors of creating irregularity in buildings that can create the distance between the effect point of the resultant of lateral forces due to the input excitation and the effect point of the resultant of resisting forces in lateral load bearing elements. In this case, if the diaphragm is rigid or semi rigid, considerable torsional moments will be produced in the floors. The amount of torsional moments created is affected by the amount of eccentricities of the centers of mass and rigidity as well as their position, and the arrangement of their placement relative to each other. According to Aziminejad and Moghadam (2010), change in the configuration of the centers of rigidity and strength, causes change in the critical values of interstory drifts and ductility demands of the structural elements and the consequent damage to the structural and non-structural elements.

The seismic design regulations to take into account the torsional response due to factors that are not included in the design process of a building necessitate considering a certain amount (usually 5% of the plan in both the main axes and in both the positive and negative directions) of accidental eccentricity of mass. This method not only makes two directional dynamic analyses difficult for engineers, its efficiency is also a question since this value is suggested based on the results of elastic

1M.Sc._University of Science and Culture, Tehran, Iran, mohsenian.vahid@gmail.com
2M.Sc._University of Science and Culture, Tehran, Iran, soheil.rostamkalaee@gmail.com
3Associate Professor_Structural Engineering Research Center, International Institute of Earthquake Engineering and Seismology, Tehran, Iran, moghadam@iiees.ac.ir
and simplified analyses of one-story models (Anagnostopoulos et al. 2015). The Housner and Outinen (1958) studies showed that the equivalent static analysis method and the maximum extracted forces using it are less reliable for designing asymmetric buildings. In addition, it has been shown in these studies that with the increase in the static eccentricity, the accuracy and validity of this method which is currently the most common design method, are significantly reduced. Over time, other researchers have tried to correct this shortcoming (Rutnberg 1992). Thus, a new parameter, "Dynamic Eccentricity" is defined as the ratio of the torsional moment to the base shear of the building, which is derived from the dynamic analysis of building.

The studies by De La Llera and Chopra (1994) show that the difference between the nominal and real values of stiffness of the elements will lead to rigidity asymmetry in the plan and accidental torsion in seismic loading. Myslimaj and Tso (2002) investigated different configurations of centers of mass, rigidity and strength. The results of these studies showed that in the linear range, the center of rigidity has the greatest effect on the behavior of buildings. The results of studies by Dusicka et al. (2000) show the significant impact of the centers of strength and rigidity on the torsional response of buildings. The research carried out by K.Badri et al. (2016) to investigate the effect of mass eccentricity in plan on the safety of buildings under severe earthquakes showed that with increase in the mass eccentricity, collapse probability decreases. As it is seen, the eccentricity of mass and stiffness and the configuration of their centers on the plan have significant impact on the dynamic responses and seismic performance of buildings.

According to the requirement for regularity of tunnel form buildings in plan, it seems that mass eccentricity due to asymmetric distribution of live loads as well as stiffness eccentricity caused by in situ quality control and imperfections of the implementation of concrete and processing, are the most probable types of irregularity in plan of such buildings with this executive method. The results of the eigenvalue analysis indicate the priority of torsional modes on the transitional modes in many conventional buildings with the tunnel form executive system (Balkaya and Kalkan 2003, 2004). This can be attributed to the little torsional stiffness of the building, which is due to the concentration of the structural walls in interior parts of the plan and lack of these elements in the peripheral parts of the building. The dominant torsional behavior in the first mode is one of the most characteristic features of torsionally flexible buildings, in which the ratio of the frequency of the torsional mode to the transitional mode (uncoupled frequency ratio) is less than one. Such buildings usually are more sensitive to eccentricities of mass and stiffness as well as input excitations (Seyedtaghia and Moghadam 2009). The presence of a minimum torsion on floor can strongly affect the reflections of torsionally flexible buildings. Due to the amplification of displacements in the peripheral parts, damages are concentrated in these positions. Therefore, the evaluation of the behavior of these buildings subjected to accidental torsions caused by accidental eccentricity of mass and stiffness is necessary, even if the plan and the arrangement of the load bearing elements is regular and symmetric (Dimova and Alashki 2003).

Despite the widespread use of tunnel form system in housing projects, this system has not been considered as an independent one in the current regulations. The review of previous literature shows that empirical and numerical studies have not been carried out to evaluate the seismic behavior of buildings constructed using the tunnel form technique subjected to accidental torsions. This study is carried out in the domain of nonlinear behavior to study the effect of accidental torsions due to eccentricity of mass and stiffness as well as their various configurations on the responses and seismic behavior of the tunnel form system. In addition to determining the performance level of the studied buildings under the design basis earthquake (DBE, 475 years return period) and the maximum considered earthquake (MCE, 2475 years return period), the responses are compared in both cases with and without the inclusion of accidental torsion.

2. SPECIFICATIONS OF THE STUDIED MODELS

For this study, a plan was used with tunnel form system presented in Figure 1. According to the figure,
the plan is regular and symmetrical in both main directions. Dashed lines represent spandrels over the openings with length and height of 1 and 0.7 meters respectively. Considering the height of the tunnel form buildings built in Iran, ten-story buildings were modeled here. The buildings were assumed to be residential and their location is in the Tehran seismic zone. Height of the floors is 3 meters and the soil type II is considered according to Iranian Standard 2800 (2014) (375 m/s < $V_S$ < 750 m/s). The studied building has been designed on the basis of ACI 318-14 (2014) and using ETABS (CSI, 2015). The behavior factor for the initial design of the building was selected to be 5, based on the usual amount used by the designers for this system. In modeling, for shells and walls, the behavior of shell (simultaneous effect of in-plane and out-of-plane deformation) is considered. The thickness of all walls was 20 cm. Rebars of #8, in every 20 cm away in two longitudinal and vertical directions were used as two layers of reinforcement. The vertical rebars of the walls are #12 in the first four stories. According to Figure 1, diagonal reinforcements were designed for spandrels to ensure ductility and increase the shear strength (Paulay and Binney 1974). The slab thickness is 15 cm. The compressive strength of the concrete and the yielding strength of the rebars are 25 MPa and 400 MPa, respectively. The values of dead load and live load are 640 and 200 Kg/m$^2$, respectively.

![Figure 1. Plan view of the studied buildings, meshing of the elements, and a sample reinforcing of spandrels](image)

3. NONLINEAR MODELING AND DETERMINATION OF THE PARAMETERS

For modeling and analysis in nonlinear range, software PERFORM-3D (CSI, 2016) was used. Shear was considered as a deformation controlled parameter in most of the walls and all spandrels between them. Thus, for walls (except for one meter walls located in axes 2 and 3 which are considered to be moment-control) and spandrels, nonlinear shear behavior and linear flexural behavior are defined (Beheshti-Aval and Mohsenian 2016; Mohsenian et al. 2016).

The criteria used to express the ductility of structural elements are different depending on their behavior. For walls and shear control beams which their ductility is caused by shear failure, drift ($\theta$) and chord rotation ($\gamma$) are selected as criteria, according to Figure 2. The values corresponding to the different performance levels for these elements are described in Table 1 (FEMA356, 2000). In this study, for the modeling of nonlinear shear behavior of elements, according to ASCE/SEI 41-13
(2014), the nominal shear strength of the cross section is considered as the final strength. It should be noted that, with regard to the ratio of the length of the open span to the height of the section (less than 2), the relationship corresponding to deep beams has been used to estimate the nominal shear strength of the spandrels between the walls. Other modeling parameters and acceptance criteria for nonlinear state are taken from the general relationship of load-displacement and the specifications of shear control members (FEMA356, 2000). Walls and the spandrels are modeled using “Shear Wall” element in the software. The other assumptions of this study are the elastic out of plane behavior of the walls, rigid diaphragm for ceilings, rigid joints for the walls, lack of uplift in the foundation, and non-slipping behavior for the rebars in concrete.

![Figure 2. Drift of the wall and rotation of the spandrel (schematic)](image)

<table>
<thead>
<tr>
<th></th>
<th>Immediate Occupancy</th>
<th>Life Safety</th>
<th>Collapse Prevention</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limit States for</td>
<td>0.004</td>
<td>0.006</td>
<td>0.0075</td>
</tr>
<tr>
<td>Drift (θ) and chord</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>rotation (γ) (FEMA356, 2000)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Values are reported in (rad)

4. THE CENTERS OF MASS AND RIGIDITY IN THE FLOORS OF ASYMMETRIC BUILDING

4.1 Mass center

In this study, asymmetric distribution of live loads of the floors is considered to be the main cause of the displacement of the center of mass. According to Iranian Standard 2800 (2014), seismic mass in terms of total dead load and 20% of the live load for floors and roofs are calculated (Equation 1).

\[ W = (\text{Dead Load}) + 0.2 (\text{Live Load}) \]  

(1)

Considering that the building is residential and the contribution of live load in calculating seismic mass, the percentage above 5 would appear to be unreasonable for eccentricity of mass in the floors. Since the diaphragm of the floors is assumed to be rigid, according to the schematic Figure 3, the transitional masses (M) and the moment inertia of mass (I_M0) are assigned to the center of mass of each floor. In order to create mass eccentricity, the center of mass is moved from its initial position (geometric center of plan) to 5% of the plan dimension perpendicular to the direction of analysis. The moment inertia of mass in the new position of the mass center (I_M) is calculated according to the relationship shown in Figure 3. To ensure that changes in the behavior of the building are merely due to the change in the position of the center of mass, value of the mass of the floors (M) is considered.
constant. In order to achieve the worst possible case of torsion, the mass centers of all the floors are moved to one side.

4.2 Center of rigidity

From cylindrical specimens prepared from concrete and cores taken from different elements of buildings, the results of standard tests show that the compressive strength \( f_c \) and the elastic modulus of concrete \( E_0 \) are in fact different from those of the design of buildings. As shown in Figure 3, the difference in the compressive strength of the concrete (and, consequently, its elastic modulus) in the lateral load bearing elements of a floor, is one that can lead to the eccentricity of stiffness. In tunnel form concrete buildings, according to the specific construction method of the system, concreting of walls and slabs of each floor are performed simultaneously and on a large scale. Under such conditions, in situ quality control and consequently possible defects in grading, mix design, vibration, and inappropriate concreting and curing are among the factors that increase the possibility of eccentricity of stiffness in the floors.

In the present study, while maintaining the dimensions of the elements \((L, H, t)\), by decreasing the elastic modulus of the lateral load bearing elements in one side of the plan \((E_1 = E_0 - \Delta E)\), the same amount is added to the elements on the other side. \((E_2 = E_0 + \Delta E)\). Obviously, in this situation, the transitional stiffness of the building remains constant and behavioral difference from the symmetrical state will only results from a change in the position of the center of rigidity. According to Figure 3, if the lateral load is applied to the building in the center of rigidity \((C_S)\), the lateral displacement of the edges of the diaphragm (points 1 and 2) must be the same. Thus, by determining the value of the displacement of the rigidity center \((X_S/B = 5\%)\), the change in the elastic modulus of the elements \((\Delta E)\) is calculated through try and error. The investigations showed that if the displacement of the rigidity center \((X_S)\) is more than 5% of the plan dimension \((B)\), the new elastic modulus for the elements \((E_1, E_2, \ldots)\) would be unreasonable. In order to achieve the worst possible state of torsion, the centers of rigidity of all floors are moved to one side.

![Figure 3. Moving the centers of mass and rigidity in the plan (schematic)](image)

4.3 Studied configurations of centers of mass and rigidity

As stated in the introduction, in addition to the values of the eccentricities, configuration of centers of mass and rigidity, i.e. their placement relative to each other, also affect seismic responses. In this study, the aim was to move the centers to a reasonable value. According to the previous description, the displacement of these centers was considered in one direction along the longitudinal axis and 5% of the plan dimension. Possible configurations for these centers are shown in the Figure 4. In this figure, \(C_M\) and \(C_S\) represent center of mass and rigidity respectively, and the models are named as \(Mi\).
5. STUDY OF VIBRATION FREQUENCIES

In this section, the values of the dead and live loads applied. Meshing of the elements are the same as the values and the intended state for the initial design phase of the buildings. In the combination of gravity and lateral loading, according to Equation 2, the upper limit of gravity load effects is considered. In the symmetric model (M₀), the percentage of the walls in the plan determines that stiffness and strength in the longitudinal direction of the plan (X) is more than the transverse direction (Y). According to Table 2, the order of the transitional modes also confirms this.

\[ Q_G = 1.1 \text{ (Dead Load + Effective Live Load)} \] (2)

As shown, in M₀ model the first mode is only torsional without transitive component. In contrast, the second and third modes are transitional and are respectively along the transverse and longitudinal directions of the plan. In order to create asymmetry in the plan, centers of mass and rigidity have been displaced along longitudinal direction (axis X) and the behavior of the models has been studied only along transverse direction (axis Y). As it is seen in Table 2, the displacement of the centers of mass and rigidity along the longitudinal direction combines the torsion of the building with the transition along the transverse direction. Moving mass center is more effective (compare M₁ with M₃). The greatest increase in the transition rate is about 1.74% and is related to the models with simultaneous eccentricity of the centers of mass and rigidity in same direction (M₂). In models with eccentricity of mass and stiffness, although torsion and transition are coupled in the first mode, the contribution of the torsion is more considerable. Regarding the periods, although changes are negligible, the period of the first vibration mode has increased in all models compared to the base model. In this case, the highest increase is about 2%, which is observed in the model M₂. No changes are made to the longitudinal direction, either in the period or in the effective coefficient of mass. The contribution coefficients of mass of the building in the transitional modes in both the longitudinal and transverse directions show that the assumption of the triangular distribution of earthquake forces in the height of the building and the use of the static method in analysis and design are not satisfactory enough. Based on the results,
spectral analysis or time history analysis is preferable.

6. TIME HISTORY ANALYSIS

In this study, 10 pairs of ground motion records were taken from the PEER Ground Motion Database (http://peer.berkeley.edu/smcat) for time history analysis, in accordance with the soil type of the site (type B in USGS soil classification, 360 m/s < Vs < 800 m/s). Selected records are far field earthquakes and are presented in Table 3. After plotting the spectral response of each pair of ground motion records and comparing them, the principal component was selected based on the larger spectral values in the vibrating frequency range of the building and imposed on the building across the transverse direction. It should be noted that the records are initially scaled so that the PGA is 0.35g (corresponding to DBE) and 0.55g (corresponding to MCE). In the following, for each configuration, the maximum drifts at the center of mass and the edges of the diaphragms are taken and the mean values are compared (Figure 5). In this figure the ratio of the soft edge drifts of the floors to base model drift under DBE (Δ₁) as well as MCE (Δ₂) are shown. It is seen that the Model M₂ has the highest ratio, in which the centers of mass and rigidity are moved together in a same direction. The studies show that responses are more sensitive to mass eccentricity as well as earthquake severity. Moreover, in the models with mass eccentricity (M₂, M₃, and M₄) seismic demand is greater at the edge which is closer to mass center. In model M₁ the edge which is far from the rigidity center has the most displacement.

Table 3. Selected records for time history analysis.

<table>
<thead>
<tr>
<th>Name</th>
<th>Ms</th>
<th>PGA (g)</th>
<th>R* (Km)</th>
<th>Component</th>
<th>Station, Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cape Mendocino</td>
<td>7.1</td>
<td>0.1782</td>
<td>41.97</td>
<td>90</td>
<td>Eureka - Myrtle &amp; West, 1992</td>
</tr>
<tr>
<td>Northridge</td>
<td>6.7</td>
<td>0.2455</td>
<td>23.07</td>
<td>180</td>
<td>Hollywood - Willoughby Ave, 1994</td>
</tr>
<tr>
<td>Northridge</td>
<td>6.7</td>
<td>0.0629</td>
<td>31.69</td>
<td>90</td>
<td>Lake Hughes #4B - Camp Mend, 1994</td>
</tr>
<tr>
<td>Cape Mendocino</td>
<td>7.1</td>
<td>0.1161</td>
<td>19.95</td>
<td>352</td>
<td>Big Tujunga, Angeles Nat F, 1994</td>
</tr>
<tr>
<td>Northridge</td>
<td>6.7</td>
<td>0.2451</td>
<td>19.74</td>
<td>0</td>
<td>Fortuna - Fortuna Blvd, 1992</td>
</tr>
<tr>
<td>Landers</td>
<td>7.4</td>
<td>0.1352</td>
<td>34.86</td>
<td>0</td>
<td>Barstow, 1992</td>
</tr>
<tr>
<td>San Fernando</td>
<td>6.6</td>
<td>0.1103</td>
<td>25.47</td>
<td>90</td>
<td>Pasadena - CIT Athenaeum, 1971</td>
</tr>
<tr>
<td>Hector Mine</td>
<td>7.1</td>
<td>0.3368</td>
<td>11.66</td>
<td>90</td>
<td>Hector, 1999</td>
</tr>
<tr>
<td>Kobe</td>
<td>6.9</td>
<td>0.5093</td>
<td>7.80</td>
<td>0</td>
<td>Nishi-Akashi, 1995</td>
</tr>
<tr>
<td>Friuli, Italy</td>
<td>6.5</td>
<td>0.4169</td>
<td>15.82</td>
<td>0</td>
<td>Tolmezzo, 1976</td>
</tr>
</tbody>
</table>

* Closest Distance to Fault Rupture
Figure 5. The mean of maximum drift at mass center and diaphragm edges

7. PUSHOVER ANALYSIS

The modal lateral load distribution model has been considered. This distribution is proportional to the effective modes in the transvers direction of the plan. The number of vibrational modes is so chosen that at least 90% of the mass of the building contributes to the analysis. In this study, the target displacement of the building was obtained through time history analysis and from the averaging of the largest displacements of the center of mass under excitation of artificial records. In order to better adapt earthquake records to site hazard, artificial records corresponding to the design spectrum were used. Thus, seven artificial records (Figure 6) were extracted by wavelet transformation from the demand spectrum of the region using existing records modification method (Table 3) and applied to the buildings. The demand spectrum is based on Iranian Standard 2800 (2014), for soil type II and DBE hazard level (return period 475 years). The PGA of these records is very close to the PGA of the DBE (0.35g).

Figure 6. The demand spectrum of the region in comparison to the spectra of artificial records

In pushover analysis, the roof drift (ratio of displacement of the mass center to the building’s height) were determined just when the first walls and spandrels reach to the immediate occupancy (IO) performance level. In Figure 7, the building capacity curve with different configurations and the values of drifts, along with the corresponding drift in DBE (target displacement) is showed. In order to better understand the amount of torsion in the building, the drifts of the both edges of the diaphragm in the roof are shown as well. In these figures, W and SP respectively represent the walls and the spandrels. In this analysis, it was found that the asymmetry in plan changes the position of the first elements that reach to the level of failure and also changes the distribution of damages from the symmetric form that exists for model M₀, to an asymmetric distribution. Asymmetry in the plan leads to a slight drop in the building capacity curve. In models with eccentricity of mass, the torsion in the
roof is considerable. The greater seismic demand in the spandrels than the shear walls results in the early damages in these elements relative to the walls, so spandrels are the first vulnerable structural elements of the building. As it is seen, eccentricity of mass and stiffness do not reduce the performance level of the building.
8. COLLAPSE PROBABILITY ANALYSIS OF THE BUILDING BASED ON ENGINEERING DEMAND

For certain levels of intensity, it is possible to determine the probability of reaching the building's response to the levels corresponding to the different performance levels. In this approach, the variation of the seismic intensity in the analysis will determine the limit state for damage in the building. This process is considered in this study and the steps are described as follow. In each model, the maximum value of the building's response is taken under each record scaled to a specified PGA. In the next step, assuming that the natural logarithm of the resulting values has a normal distribution, after calculating the parameters of mean ($\mu$) and standard deviation ($\delta$) for the values taken at this level of intensity, a probability density function ($F(x)$) is extracted. According to the schematic Figure 8, by substituting a value for $X_0$ as the corresponding response to a damage state, the surface below the curve of the probability density function from $-\infty$ to $X_0$ shows the reliability of the building. That is, at this level of intensity (PGA=cte.), with the probability of $P$, the response of the building does not reach the value of $X_0$ and will not experience the performance level. Obviously, the difference between $P$ and the unit value will result in a probability of exceeding this damage state. For the studied building, under hazard levels of DBE and MCE the maximum lateral drift and the chord rotation were considered as the response in walls and spandrels, respectively. The amount of $P$ is extracted in the form of Figure 9 according to the described process, for the performance level of IO and life safety (LS) (for spandrels only).
Figure 9. The reliability of the elements at DBE and MCE hazard levels

As it is seen, the eccentricity of mass and stiffness leads to slightly increase the probability of exceedance of walls and spandrels from the IO performance level. The study of fragility curves indicates that in earthquake with PGA of 0.35g (corresponding to DBE) in all models, the probability that the walls reach the IO performance level is almost zero. At the same level of intensity and in all models, the probability of reaching the spandrels to the aforementioned performance level is less than 2%. In earthquake with PGA of 0.55g (corresponding to MCE) in all models, the probability that the walls reach the IO performance level is less than 8%. At this level of intensity and in all models, the probability of reaching the spandrels to the IO performance level is less than 40%.

8. CONCLUSION

The results of this study in the range of examined models and assumptions are as follows:
1- The greatest effect on the period and the mass participation of the building are related to the models with the simultaneous eccentricity of the mass and stiffness in a same direction.
2- The seismic response of the building is affected by the eccentricity of mass and stiffness as well as the severity of the earthquake. Meanwhile, responses are more sensitive to mass eccentricity.
3- In the worst Model (M₂), the maximum drifts of the building in DBE and MCE hazard levels are estimated to be approximately 2.36 and 2.09 times the maximum drifts in symmetric mode.
4- For high intensity earthquakes and models with mass eccentricity, torsion at the upper floors of the building is significant. Accordingly, the mass center of roof is not a suitable control point for displacement provisions. So, using the drift of mass center as a damage index is not recommended for these buildings.
5- The damage modes and the location of the beginning of the failure in the building elements are heavily influenced by the configuration of centers of mass and rigidity.
6- For the high levels of intensity (MCE) and in the most critical case, asymmetry increases the probability of reaching the walls and spandrels to the IO performance level by about 4 and 12 percent, respectively, relative to the symmetrical model.
7- Considering the walls as the gravity and lateral load bearing main elements in this system, it can be concluded that the buildings maintain the IO performance level at both DBE and MCE hazard levels.
8- It seems that accidental torsion due to the asymmetric distribution of mass and stiffness in plan, as well as the various configurations of centers of mass and rigidity, do not have considerable effect on the performance level of tunnel form buildings under DBE hazard level.
9- It seems that the redundancy and excessive strength in this system, in the face of accidental torsions caused by asymmetric distribution of mass and stiffness, provide considerable reliability and desirable seismic performance for the building.

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PEER Ground Motion Database, Pacific Earthquake Engineering Research Center, Web Site: http://peer.berkeley.edu/peer_ground_motion_database

