Seismic Performance of Torsionally Stiff and Flexible Single Story Buildings Designed Based on Iranian Seismic Code (Standard 2800)

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Abstract:

This paper examines differences in performances of a range of torsionally stiff and flexible single story buildings designed with the provisions of Iranian Standard 2800. Seismic nonlinear dynamic time history behavior of eight building models subjected to seven horizontal bi-directional design spectra compatible ground motions are investigated. These models cover a wide range of very to torsionally stiff to very flexible buildings. Response parameters are element ductility demand and building story drift ratio. These criteria are appropriate indices for structural and nonstructural damages, respectively. This investigation shows that the linear static analysis of building code such as Iranian Standard 2800 is not generally adequate for structures with very low torsional stiffness.

Keywords: Torsion, Torsionally stiff, torsionally flexible, Standard 2800, Non-linear dynamic analysis, Story drift ratio, Performance level.

1. Introduction

The assessment of structural performance during past earthquakes have indicated that irregularity due to asymmetric mass, stiffness and strength distribution is one of the main reasons of structural vulnerability. Such plan-wise irregular structures are divided into two classes with low or high “torsional to translational modal frequencies ratio (ΩR)” that are named torsionally flexible and stiff structures, respectively. The farthest and the nearest edge of building from the center of rigidity are named flexible and stiff edges, respectively. In torsionally flexible buildings (ΩR<1), both edges generally experience more displacement compared to their symmetric building counterpart. In torsionally stiff structures (ΩR≥1), flexible and stiff edges, respectively experience more and less displacement compared to their symmetric building counterpart [1].

In this paper, an overview of torsional provisions and their frameworks in the 2005 NBCC [2] and torsional provisions in Standard 2800 (2005) [3] are presented first, then the capability of these torsional provisions in Standard 2800 (2005) to control the effect of torsion in torsionally flexible and stiff structures are evaluated. [3]

2. Background

Humar et al. (2003) investigated torsional provisions in the 2005 editions of the NBCC using models similar to Fig. 1. In this model, the building floor is assumed to be infinitely rigid in its own plane. The entire mass of the structure is distributed at the floor level. The origin of the coordinate system is assumed to be at the mass center, denoted by CM. Lateral resisting elements are shown as hatched lines. The center of stiffness, which in the elastic range is also the center of resisting forces CR, is eccentric with respect to CM and lies at a distance from CM

The frameworks of the NBCC are based on equaling displacements in static analysis with dynamic analysis for flexible and stiff edges in order to derive formulas to be used in static analysis [1].

The 2005 NBCC restrict the use of the equivalent static load method of design to buildings that are relatively stiff in torsion. An alternative measure of torsional stiffness is being proposed. In the 2005 NBCC, a building with a rigid diaphragm will be considered torsionally sensitive if a ratio B exceeds 1.7.

Parameter B is determined by calculating the ratio Bx for each level x, and independently for each orthogonal direction, according to the following equation:
Fig. 1. General configuration of the models

$$B_x = \frac{\delta_{\text{max}}}{\delta_{\text{ave}}}$$  \hspace{1cm} (1)

Where $\delta_{\text{max}}$ is the maximum story displacement at the extreme points of the structure at level $x$ in the direction of the earthquake induced by the equivalent static forces acting at a distance $\pm 0.1b$ from the centers of mass at each floor; $\delta_{\text{ave}}$ is the average of the displacements of the extreme points of the structure at level $x$ produced by the above forces, and $b$ is the dimension of floor $x$ perpendicular to the direction of earthquakes. Ratio $B$ is then taken as the maximum of all values of $B_x$ in both orthogonal directions.

Determination of $\delta_{\text{max}}$ and $\delta_{\text{ave}}$ requires that a three-dimensional (3D) static analysis of the structure be carried out. The 2005 NBCC require that a dynamic analysis be carried out for determining the design forces whenever $B$ exceeds 1.7.

For buildings where the equivalent static load method of design is permitted, the 2005 NBCC specify the following values for the design eccentricities:

$$e_{d1} = e + 0.1b$$  \hspace{1cm} (2)
$$e_{d2} = e - 0.1b$$  \hspace{1cm} (3)

Standard 2800 (2005) applies seismic forces in mass center for equivalent static analysis and changes mass center location equal to accidental eccentricity. This standard simulates the effect of uncertainty in mass, stiffness distribution and earthquake rotational component through the accidental torsion. This standard defines amplification factor ($A_t$) that is multiplied by the accidental torsion for torsionally flexible structures in order to decrease ductility demand in critical edges through increasing the strength and stiffness for lateral resisting elements.

In the following parts of this paper, building models that cover a large variation of frequency ratios are introduced first. Then these models subjected to seven recorded events with pairs of appropriate horizontal ground-motion time history components are analyzed. This research shows that Standard 2800 (2005) is not always satisfactory and the application of static analyses for torsionally flexible structures does not lead to structural safety.

3. Building models characteristics

Fig. 2 shows the architectural plan of models considered in this study to cover a wide range of torsional to lateral stiffness ratios. Dimensions of plan are $42m \times 10m$ and the story height is $3m$. These models are able to represent stiff and flexible torsional behavior. The lateral stiffness of models is symmetric in the $y$ direction. The structural system in two directions is concentric braced frame ($CBF$). Applying different configurations of braces in the $x$ direction, eight models that cover the required range for frequency ratio are produced. Each model has brace in A, B and C axes in the $y$ direction. In the first four models, stiffness distributions in the $x$ direction are symmetric too.

Fig. 3 shows the locations of braces, schematically.

Models are considered residential buildings with rigid floors located in high seismic zone. Loads on structures are assumed based on Iranian national building code (2004) [4]. Accidental torsion is $0.05b$, where $b$ is the dimension of the building model perpendicular to the direction of the earthquake. Using linear static analyses, the buildings are designed based on UBC97 design provisions. Locations of braces in the $x$ direction are used to identify each models such as $O^O^X^X^O^O$, where letters O and X refer to frames without and with braces, respectively. Letter N shows the number of O or X.

Fig. 2. The plan of the models.
frames. For all models, configurations of braces in the y direction are the same. According to Standard 2800 (2005) torsional irregularity shall be considered to exist when the maximum story drift computed, including accidental torsion at one end of the structure transverse to an axis, is more than 1.2 times the average of the story drifts of the two ends of the structure. If this rule is not satisfied, \( A_j \) factor should be calculated.

\[
A_j = \left( \frac{\delta_{\text{max}}}{1.2\delta_{\text{ave}}} \right)^2, \quad 1 \leq A_j \leq 3
\]

Where:

\( \delta_{\text{max}} \) = the average of the displacements at the extreme points of the structure at Level J.

\( \delta_{\text{ave}} \) = the maximum displacement at Level J.

According to the provisions of the 2005 NBCC, Table 1 shows that for torsionally stiff structures (models A and B) equivalent static analysis is allowed marginally (maximum of \( B \) is equal to 1.77), but for torsionally flexible structures, equivalent static analysis is not allowed (maximum of \( B \) is equal to 3.74 in model D).

Table 1 shows fundamental period, rotational to translational frequencies ratio \( (\Omega_R) \), parameter \( B \), whether static analysis is allowed or not also parameter \( A_j \) for the eight selected models. According to Table 1, models A and B are almost torsionally stiff while the others are torsionally flexible structures.

Finally, in according to Standard 2800 (2005) [3], Table 1 shows that for torsionally stiff structures (models A and B) amplification factor can be neglected, but for torsionally flexible structures, amplification factor is necessary (more than 1). Based on new amplified eccentricities, models are analyzed again. The designed models are used in nonlinear bi-directional dynamic time history analyses subjected to seven pairs of horizontal ground motion components.

![Fig. 3. Locations of braces in the eight selected models; models A to D with symmetric stiffness distribution in the x direction; models E to H with asymmetric stiffness distribution in the x direction; in all models the stiffness distribution in the y direction is symmetric](image)

<table>
<thead>
<tr>
<th>Model</th>
<th>( T ) for direction</th>
<th>( \Omega_R ) for direction</th>
<th>( B )</th>
<th>static analysis</th>
<th>( A_j )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( X )</td>
<td>( Y )</td>
<td>( \Omega_{\text{max}} )</td>
<td>( \Omega_{\text{ave}} )</td>
<td>( \Omega_{\text{ave}}^* )</td>
</tr>
<tr>
<td>A-X'</td>
<td>0.13</td>
<td>0.20</td>
<td>1.12</td>
<td>1.69</td>
<td>1.41</td>
</tr>
<tr>
<td>B-OX'O</td>
<td>0.15</td>
<td>0.20</td>
<td>0.86</td>
<td>1.12</td>
<td>0.99</td>
</tr>
<tr>
<td>C-O'X'O'</td>
<td>0.18</td>
<td>0.20</td>
<td>0.63</td>
<td>0.67</td>
<td>0.65</td>
</tr>
<tr>
<td>D-O'X'O'</td>
<td>0.23</td>
<td>0.20</td>
<td>0.49</td>
<td>0.41</td>
<td>0.45</td>
</tr>
<tr>
<td>E-O'X'O'</td>
<td>0.56</td>
<td>0.14</td>
<td>0.52</td>
<td>0.44</td>
<td>0.48</td>
</tr>
<tr>
<td>F-O'X'O</td>
<td>0.53</td>
<td>0.18</td>
<td>0.57</td>
<td>0.54</td>
<td>0.56</td>
</tr>
<tr>
<td>G-O'X'O</td>
<td>0.39</td>
<td>0.20</td>
<td>0.63</td>
<td>0.73</td>
<td>0.68</td>
</tr>
<tr>
<td>H-O'X'O</td>
<td>0.13</td>
<td>0.20</td>
<td>0.75</td>
<td>0.91</td>
<td>0.83</td>
</tr>
</tbody>
</table>

**Note:** *\( \Omega_R \) is average value for two directions*
Table 2. The selected records and their characteristics

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake</th>
<th>Station</th>
<th>Date</th>
<th>Magnitude</th>
<th>Distance from source (km)</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Northridge</td>
<td>24087 Arleta Nordhoff Fire Sta</td>
<td>1994/01/17</td>
<td>6.7</td>
<td>9.2</td>
<td>0.344</td>
</tr>
<tr>
<td>2</td>
<td>Northridge</td>
<td>24000 LA – Obregon park</td>
<td>1994/01/17</td>
<td>6.7</td>
<td>37.9</td>
<td>0.355</td>
</tr>
<tr>
<td>3</td>
<td>Northridge</td>
<td>90014 Beverly Hills -12520 Malibu</td>
<td>1994/01/17</td>
<td>6.7</td>
<td>20.8</td>
<td>0.314</td>
</tr>
<tr>
<td>4</td>
<td>Victoria</td>
<td>6604 Cerro Prieto Mexico</td>
<td>1980/06/9</td>
<td>-</td>
<td>-</td>
<td>0.304</td>
</tr>
<tr>
<td>5</td>
<td>N.Palm Springs</td>
<td>12149 Desert Hot Springs</td>
<td>1986/7/8</td>
<td>6</td>
<td>8</td>
<td>0.331</td>
</tr>
<tr>
<td>6</td>
<td>N.Palm Springs</td>
<td>5071 Morongo Valley</td>
<td>1986/7/8</td>
<td>6</td>
<td>10.1</td>
<td>0.395</td>
</tr>
<tr>
<td>7</td>
<td>Whittier Narrows</td>
<td>24000 LA – Obregon park</td>
<td>1987/10/4</td>
<td>5.3</td>
<td>-</td>
<td>0.374</td>
</tr>
</tbody>
</table>

4. Nonlinear dynamic time history analyses

For a better probabilistic evaluation, seven events recorded on soil profile type II based on Standard 2800 (2005) and high seismic zone are selected from Pacific Earthquake Engineering Research Center Database (PEER) [5]. Table 2 shows the selected records and their characteristics. For each pair of horizontal ground motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of the scaled horizontal components is constructed. The motions are scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake in the period range of 0.2 to 1.5 seconds. Two components of each pair of time histories shall be applied simultaneously to the models. Fig. 4 shows the SRSS spectra and its average. There are satisfactory compatibility of frequency content between the scaled average value of the SRSS spectra and 1.4 times the design spectra of Standard 2800 (2005) in the period range of interest (Fig. 5). SAP2000 (2008) is used for conducting nonlinear dynamic time history analyses [6].
5. Dynamic nonlinear analyses assumptions

Some of the main modeling and analyses assumptions are:
1- The effect of accidental eccentricity is considered by change in mass distribution in such a way that the center of mass displaces by 0.05\(b\), where \(b\) is the dimension of the building model in the direction of the structural eccentricity.
2- The walls are assumed isolated from the frames, thus infill effects have been neglected.
3- The proportional damping is assumed with damping ratio equal to 5 percent at two periods of 0.1 and 1 second.
4- Plastic hinge model considering axial load-moment interaction (PMM) is assigned to the middle and two edge columns and axial plastic hinge model (\(P\)) is assigned to braces based on FEMA356 [7]. Fig. 6 shows force-displacement relation for brace, schematically.
5- Analysis type is nonlinear direct integration time history.
6- The time step in analysis is considered as 0.0025 second, for convergence matters.
7- Geometric \(P-\Delta\) nonlinear effect is considered.
8- Each analysis is performed initially for gravity load, then seismic analysis is started from the state at the end of previous analysis. The parameters of interest shall be calculated for each time history analysis. Because seven time-history analyses are performed, the average values of the response parameter of interest should be used.

Response parameters considered here are ductility demand in plastic hinge and story drift ratio. The story drift ratio is calculated in both flexible and stiff edges of each orthogonal direction.

6. Nonlinear analyses results

For torsionally flexible or stiff “symmetric” models (A to D) in the x direction, in accordance with Fig. 7, the provisions of Standard 2800 (2005) [3] have adequate efficiency in limiting story drift ratio for critical edges (to a value less than 0.68\%).

For models E and F in the x direction, story drift ratios (2.95\%, 2.58\%) are greater than drift limit. These models (E and F) are representative of very low torsional stiffness structures (\(\Omega_{RX}\) are equal to 0.52 and 0.57, respectively) and distribution of their resisting elements are asymmetric in plan. Therefore, the efficiency of the provisions of Standard 2800 (2005) for drift limit in such models are questionable. It is concluded that drift has been increased with decreasing parameter \(\Omega_{RX}\).

In considering drift, these provisions are only adequate for models G and H with \(\Omega_{RX}>0.63\) (maximum drift of 1.28\% in model G).

Average maximum story drift ratios for the flexible and stiff edges in the y direction are shown in Fig. 8. All the structures are symmetric in the y direction. Conclusion is similar to those obtained from Fig. 7 for symmetric cases. Therefore, the provisions of Standard 2800 (2005) [3] have adequate efficiency in limiting drift for critical edges (maximum drift of 0.62\% in model F).

Fig. 7. Average maximum story drift ratios for the flexible and stiff edges in the x direction

Fig. 8. Average maximum story drift ratios for the flexible and stiff edges in the y direction

Fig. 9 presents overall evaluation of the frameworks and code provisions in limiting drift of all edges at each direction. It is realized that structural behavior in the x direction is generally dominant with respect to overall structural behavior. Therefore, the conclusions in Fig. 7 are also valid here.

Fig. 10 shows the average maximum floor rotations for the eight models. In models with symmetric mass and stiffness distribution (A to D), floor rotation caused by accidental torsion is small (less than 0.51x10\(^{-3}\) radian), but for models with asymmetric stiffness distribution, floor rotation is larger and it is increasing, with decrease in...
parameter $\Omega_R$. Model E, with respect to brace configurations, has minimum frequency ratio and its floor rotation is maximum ($2.30\times10^{-3}$ radian).

Fig. 9. Average maximum story drift ratios of all edges and each direction

Fig. 10. Average maximum floor rotations for the eight selected models

Fig. 11 shows average maximum ductility demand ($\mu$) in plastic hinges for the eight selected models. The ductility demand is defined here by subtracting elastic deformation from plastic deformation divided by yield deformation. Fig. 11 shows for symmetric models (A to D), ductility demand is less than 2, but for asymmetric models (E to H), ductility demand is less than 4. Standard 2800 (2005) considers life safety performance level ($\mu<6$) for a residential building. It is concluded that ductility demand in the selected models, especially torsionally stiff models, is less than acceptable limit. Therefore, the provisions of Standard 2800 (2005) have appropriate efficiency in limiting ductility demand.

7. Conclusions

It should be emphasized that the results of this study are valid only within the framework of presented modeling and analysis assumptions such as: soil profile type, earthquake and excitation angle, behavior of material, analysis type, and capability of software, sizes of steel shapes, geometry and the number of models.

The main results of this study are as follows:

1- For symmetric models, including both torsionally flexible and stiff structures, the provisions of Standard 2800 (2005) for static analysis have adequate efficiency to limit story drift ratio for critical edges. Also for asymmetric cases with low torsional stiffness those provisions are suitable. But for asymmetric cases with very low torsional stiffness, the efficiency of the provisions is questionable. It is concluded that drift has increased with decrease in parameter $\Omega_R$.

2- For symmetric models, floor rotation caused by accidental torsion is small, but for asymmetric models, floor rotation is larger and it is increasing with decrease in parameter $\Omega_R$.

3- It is concluded that ductility demand in the selected models, especially torsionally stiff models, and is less than its limitation. Therefore, the provisions of Standard 2800 (2005) have appropriate efficiency to limit ductility demand.

As a final conclusion, parameter $\Omega_R$ in the provisions of Standard 2800 (2005), generally increases the stiffness of structure and therefore, seismic forces due to the shape of design spectra in Standard 2800 (2005) for torsionally flexible structures. In this study the provisions of Standard 2800 (2005) were able to limit ductility demand, but they did not limit drift to the allowable level for very low torsional structure with $\Omega_R<0.68$. Based on these observations four strategies can be considered in future studies:

1- Limit the use of linear static analysis for structures with at least a minimum frequency ratio ($\Omega_R$).

2- With appropriate design, limit frequency ratio ($\Omega_R$) of the building to mentioned bounds.

3- Proposes modified formulas for increasing the structural stiffness in order to reduce drift.

4- Reduces $R$ factor (i.e. force modification factor related to ductility).

Acknowledgments

This study was conducted to evaluate efficiency the provisions of Standard 2800. The authors gratefully acknowledge the contribution of the members of the International Institute of Earthquake Engineering and Seismology, IIEES and Qazvin Islamic Azad University, QIAU, through critical comments and suggestions.
Fig. 11. Average maximum ductility demand for the eight selected models.

References


[5] PEER Strong Motion Database, peer.berkeley.edu website
