A CASE STUDY ON THE SEISMIC PERFORMANCE ASSESSMENT OF THE HIGH-RISE SETBACK TOWER IN ACCORDANCE WITH TBI GUIDELINES

Kamyar Kildashti *, Rasoul Mirghaderi
School of Civil Engineering, University of Tehran, Tehran, Iran.
*Email: kildashti@ut.ac.ir

ABSTRACT

This paper describes the three-dimensional seismic vulnerability assessment of a high-rise steel moment-frame setback tower, designed and detailed per the 2010 ASCE7 and 2010 AISC-341. The performance evaluations are conducted based on nonlinear history analyses under a set of multi-directional strong ground motion records which are scaled to maximum considered earthquake spectrum in accordance with ASCE41-13. Plastic hinge monotonic and hysteretic behavior for prequalified steel reduced beam section connections are obtained from values presented in TBI Guidelines and PEER/ATC72. Hysteretic behaviors are adjusted based on FEMA P440A recommendations to incorporate stiffness and strength degradation in analyses. Plastic rotation demands of the order of 2-4% of a radian and inter-story drift/residual demands close to the order of 0.03/0.01 are created on the basis of median of the records. These values almost coincide to performance at or near ‘collapse prevention’ reported in TBI Guidelines. This performance level clarifies the accuracy of response modification coefficient presented in ASCE7-10 for special moment resisting frames. The well-distributed yield pattern along building’s height implies the superior contribution of lateral force-resisting system to control dynamic instability.

KEYWORDS

Performance-based design, high-rise building, moment resisting frame, nonlinear history analysis.

INTRODUCTION

Nowadays Performance-Based Seismic Design Procedures (Tall Building Initiatives, TBI, 2010) are introduced for high-rise buildings as recommended alternatives to the prescriptive strategies for seismic design of new buildings encompassed in standards such as ASCE7-10 (2010). The main reasons why the alternatives have been developed are generally related to either height limits or uncovered seismic-force-resisting systems in accordance with the Building Codes. Elaborated structural and earthquake knowledge about selection and scaling of ground motions, reliable mathematical modelling and nonlinear history analyses, are the prerequisites for proper implementation of TBI guidelines. In TBI guidelines, acceptable criteria is determined according to two hazard levels including service level earthquake (SLE) and maximum considered earthquake (MCE). In SLE, the initiation of repairable damage is considered to obtain structural limit states, while in MCE, the onset of considerable strength and stiffness degradations in structural components are presumed with low probability of overall or local collapse.

Design and acceptable seismic performance of a concrete core-wall high-rise building was evaluated by Yang et al. (2012) based on two alternative approaches in accordance with International Building Code (IBC 2006) and TBI guidelines. The results indicated appropriate performance of the structural system. Different design procedures in terms of code-compliant design and performance-based design for a 40-story buckling-restrained brace frame were compared by Jones and Zareian (2013). The structural performance objectives are assessed on the basis of inter-story drift ratio (IDR) exceedance from allowable values. Performance-based evaluation of ultra-high-rise building designed beyond code-specified provisions was studied by Wei et al. (2012) and the results demonstrated that performance design are reliable to predict structural response in severe earthquakes.

In this paper seismic performance of a case study high-rise moment-frame with setback irregularity is assessed based on the TBI recommended criteria for MCE hazard level. The major objective of this assessment is to validate collapse safety margin as declared by ASCE7-10. This objective is gained by using nonlinear history analyses to estimate the response of the high-rise tower to a set of ground motions that are scaled to MCE shaking as reported in TBI guidelines. Despite the fact that this evaluation does not offer quantifiable margin against collapse it demonstrates under selected ground motions forces and deformations are not beyond acceptable limits.
A CASE STUDY BUILDING DESCRIPTION

Selected building is comprised of two similar legs with approximate height of 240 meters (50 story) connected together in top stories (Figure 1 (a)). These legs are architecturally designed for residential and hotel occupancy called ‘Apartment Part’ and ‘Hotel Part’, respectively. This building was architecturally designed by ATKINS Group with typical story height of 4 meter, while ground and podium floors have level height of 6 meter. Because the building is located in earthquake-prone region, site-specific investigations are carried out to obtain design spectrum. The structural designs were carried out by engineering research group knowledgeable in seismic design of high-rise buildings. Response spectrum analysis (RSA) on the basis of design-based spectrum is conducted on three dimensional elastic model of the building using commercial program CSI-ETABS (2013). The three dimensional view of the structural model is shown in Figure 1 (b). In Figure 2 (a), typical floor plan of the building is illustrated. Design parameters according to ASCE7-10 are adjusted to obtain minimum gravity and lateral loads appropriate for the building design. Special moment resisting frames (SMRFs) are adopted to resist against both gravity and lateral loads. The arrangement of moment resisting connections in one portion of the floor plan is depicted in Figure 2 (b). Gravity load bearing system is comprised of composite slab including metal deck, corrugated sheets of 1 mm, and concrete slab of 140 mm thickness supported by steel joists connected to beams or columns. Beam sections were sized with 1-shape built-up sections while column sections were proportioned with H-shape built-up sections in accordance with Load Resistance Factor Design (LRFD) of ANSI/AISC 360-10 (2010) and seismic detailing is provided by requirements developed in ANSI/AISC341-10 (2010). Prequalified Reduced Beam Section (RBS) connections are detailed in accordance with ANSI/AISC341-10 to resist against lateral loads. All requirements such as strong column- weak beam and panel zone strength are accounted during design process.

It is noteworthy that structural concrete walls are arranged through first five stories of the building to reduce flexural demands on steel column base plates. As can be observed in Figure 1 (a), a structure called ‘Top Hat’ is located on top of the apartment and hotel parts of the building due to the architectural reasons. The structure has to be continuous in the ‘Top Hat’ area to accommodate façade requirements in the finishing of the architectural form. ASTM-A36, Grade 36 and ASTM-A572, Grade 50 steel is used for the beams and columns in the building, respectively. It is assumed that the nominal yield stress is 248.21 Mpa and 344.73 Mpa for A36 and A572, respectively. Additionally, the nominal ultimate stress is 399.90 Mpa and 448.15 Mpa for A36 and A572, respectively. The compressive strength of concrete is set to 30 Mpa for structural walls and floor slabs.

SELECTION OF GROUND MOTION RECORDS

Ground motions for nonlinear response history analysis (NLRHA) are selected from PEER NGA database (2014). These records, as reported in Table 1, include classes from moderate earthquakes (Mw=6.5) to very large earthquakes (Mw=7.9). Several requirements for selecting and scaling of ground motions based on the recommendations reported in TBI guidelines including; controlling of seismic hazard conditions, compliance with site conditions and matching with the target spectrum (MCE spectrum) are taken into account. According to ASCE41-13 (2013), spectral-matching is adjusted to match frequency contents of accelerograms in which the response spectrum is within predefined limits of the MCE spectrum over the defined period range 0.2T₀ to 1.5T₀, where T₀ is the fundamental period of vibration (Figure 3(a)). Spectral matching is recommended by TBI for tall buildings rather than amplitude matching procedure as a result of reducing dispersion of response values. In

![Figure 1 (a) Architectural rendering (b) 3-D ETABS model](image)
Figure 3 (b), the dispersion in the results is depicted by dashed lines with one and two times of standard deviation (SD) around median values.

![Figure 3 (b)](image)

Table 1 Selected ground motion records

<table>
<thead>
<tr>
<th>Record Seq. No.</th>
<th>Event</th>
<th>Year</th>
<th>Station</th>
<th>Magnitude (Mw)</th>
<th>Mechanism</th>
<th>Rrup (km)</th>
<th>Vs(30) (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RSN143</td>
<td>Tabas, Iran</td>
<td>1978</td>
<td>Tabas</td>
<td>7.4</td>
<td>Reverse</td>
<td>2.05</td>
<td>767</td>
</tr>
<tr>
<td>RSN182</td>
<td>Imperial Valley-06</td>
<td>1979</td>
<td>El Centro Array #7</td>
<td>6.5</td>
<td>Strike Slip</td>
<td>0.56</td>
<td>211</td>
</tr>
<tr>
<td>RSN184</td>
<td>Imperial Valley-06</td>
<td>1979</td>
<td>El Centro Differential Array</td>
<td>6.5</td>
<td>Strike Slip</td>
<td>5.09</td>
<td>202</td>
</tr>
<tr>
<td>RSN802</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>Saratoga - Aloha Ave</td>
<td>6.9</td>
<td>Reverse Oblique</td>
<td>8.50</td>
<td>381</td>
</tr>
<tr>
<td>RSN838</td>
<td>Landers</td>
<td>1992</td>
<td>Barstow</td>
<td>7.3</td>
<td>Strike Slip</td>
<td>34.86</td>
<td>371</td>
</tr>
<tr>
<td>RSN879</td>
<td>Landers</td>
<td>1992</td>
<td>Lucerne</td>
<td>7.3</td>
<td>Strike Slip</td>
<td>2.19</td>
<td>1369</td>
</tr>
<tr>
<td>RSN1114</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>Port Island</td>
<td>6.9</td>
<td>Strike Slip</td>
<td>3.31</td>
<td>198</td>
</tr>
<tr>
<td>RSN1161</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>Gebze</td>
<td>7.5</td>
<td>Strike Slip</td>
<td>10.92</td>
<td>792</td>
</tr>
<tr>
<td>RSN1176</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>Yarmica</td>
<td>7.5</td>
<td>Strike Slip</td>
<td>4.83</td>
<td>297</td>
</tr>
<tr>
<td>RSN1501</td>
<td>Chi-Chi Taiwan</td>
<td>1999</td>
<td>TCU063</td>
<td>7.6</td>
<td>Reverse Oblique</td>
<td>9.78</td>
<td>476</td>
</tr>
<tr>
<td>RSN1510</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TCU075</td>
<td>7.6</td>
<td>Reverse Oblique</td>
<td>0.89</td>
<td>573</td>
</tr>
<tr>
<td>RSN1602</td>
<td>Duzce, Turkey</td>
<td>1999</td>
<td>Bolu</td>
<td>7.1</td>
<td>Strike Slip</td>
<td>12.04</td>
<td>294</td>
</tr>
<tr>
<td>RSN2114</td>
<td>Denali, Alaska</td>
<td>2002</td>
<td>TAPS, Pump Station #10</td>
<td>7.9</td>
<td>Strike Slip</td>
<td>2.74</td>
<td>329</td>
</tr>
<tr>
<td>RSN4040</td>
<td>Bam, Iran</td>
<td>2003</td>
<td>Bam</td>
<td>6.6</td>
<td>Strike Slip</td>
<td>1.70</td>
<td>487</td>
</tr>
<tr>
<td>RSN8164</td>
<td>Duzce, Turkey</td>
<td>1999</td>
<td>IRIGM</td>
<td>7.1</td>
<td>Strike Slip</td>
<td>2.65</td>
<td>690</td>
</tr>
</tbody>
</table>

EXPLANATION OF THE ANALYTICAL MODEL

CSI-SAP2000 (2010), a general purpose finite element program, are utilized to simulate analytical model of the structure for NLRHA. The software is well-equipped to solve nonlinear equations of motion in structural systems with extensive number of degrees of freedom. One-way load transfer from floors to beam elements is considered and the floor slab contribution in stiffness and strength of the structure is excluded. Leaning column with no lateral stiffness is modeled to capture p-Delta effects during lateral displacement. Load combination for p-Delta effects are considered by 1.00 times dead loads plus 0.25 live loads according to TBI guidelines. Stiffness and mass proportional Rayleigh damping are specified as 2.5% for the periods at 6 and 1 second.
Figure 3 Spectrum-matched ground motion records (a) spectral-matched record set (b) comparison between MCE spectrum and median, median+1SD, and median+2SD

As stated previously, RBS connections are adopted to provide required stiffness and strength against lateral load. Fiber-hinge elements are utilized to model flexural hinging in the RBS region nearby column face. To capture all important deterioration modes regarding lateral torsional buckling modes in steel components, artificial limits are imposed on stress-strain curves of each fiber model, as stated in TBI guidelines. Nonlinear parameters for modeling inelastic springs in beams are obtained from recommendations reported in PEER/ATC72 (2010) and FEMA P440A. Following equations are implemented to obtain backbone curve modeling parameters based on material and geometric characteristics of beam cross sections:

\[
\theta_p = 0.19 \left( \frac{h}{t_w} \right)^{-0.314} \cdot \left( \frac{b_f}{2t_f} \right)^{-0.10} \cdot \left( \frac{L_b}{r_y} \right)^{-0.1185} \cdot \left( \frac{L}{d} \right)^{0.113} \cdot \left( \frac{d}{0.5334} \right)^{-0.76} \cdot \left( \frac{F_y}{344.83} \right)^{-0.07}
\]  

(1)

\[
\theta_{pc} = 9.62 \left( \frac{h}{t_w} \right)^{-0.513} \cdot \left( \frac{b_f}{2t_f} \right)^{-0.863} \cdot \left( \frac{L_b}{r_y} \right)^{-0.108} \cdot \left( \frac{F_y}{344.83} \right)^{-0.36}
\]  

(2)

\[
\Lambda = \frac{E_t}{M_y} = 592. \left( \frac{h}{t_w} \right)^{-1.138} \cdot \left( \frac{b_f}{2t_f} \right)^{-0.632} \cdot \left( \frac{L_b}{r_y} \right)^{-0.205} \cdot \left( \frac{F_y}{344.83} \right)^{-0.391}
\]  

(3)

where \( h/t_w \): ratio of depth to web thickness of the cross section, \( L_b/r_y \): ratio of unbraced length to radius of gyration about the weak axis of the cross section, \( b_f/2t_f \): ratio of flange width to thickness of the cross section, \( L/d \): ratio of shear span to depth of the cross section, \( F_y \): yield strength of the flange in Mpa, \( \theta_p \): pre-capping plastic rotation for beams with RBS connections, \( \theta_{pc} \): post-capping plastic rotation for beams with RBS connections, \( A \): reference cumulative plastic rotation for beams with RBS connections, \( E_t \): reference hysteretic energy dissipation capacity, and \( M_y \): yield moment strength. The process of obtaining a backbone curve using the parameters defined above is illustrated in Figure 4 (a).

To validate analytical model with experimental results, an experimental test of steel RBS moment connection which was reported in literature (Uang et al. 2012) is taken into account. Standard loading protocol are considered, shown in Figure 4 (b). A typical comparison between experimental results and analytical model is shown in Figure 4 (c). As can be observed, the analytical result matches the experimental result reasonably well in the medium to large hinge rotation.

RESULTS AND DISCUSSIONS

According to TBI guidelines both global structural response including IDRs and residual drift ratios (RDRs) and local deformation-controlled actions in each component shall not exceed acceptance criteria while analytical model of the building are subjected to ground motions scaled to MCE level.
Peak transient story drift

TBI declares that the mean of the absolute values of the maximum transient drift ratios from the set of analyses in each story level shall not exceed 3%. Additionally, the absolute value of the maximum story drift ratio from the set of analyses shall not exceed 4.5%. The heightwise profile of the absolute values of mean/maximum of maximum IDR histories obtained from NLRHAs in orthogonal directions (X, Y directions) along with SRSS values for the ‘Apartment Part’ and ‘Hotel Part’ are shown in Figure 5 (a)/(b) and Figure 6 (a)/(b), respectively. It is noteworthy that the SRSS values are calculated by the following relationship (Magliulo & Ramasco (2007)):

$$SRSS(E_x(t_i), E_y(t_i)) = \sqrt{(E_x(t_i))^2 + (E_y(t_i))^2} \tag{4}$$

where $E_x(t_i), E_y(t_i)$ are demands in the X and Y directions at the i th instant of the time history. As can be observed, the maximum of maximum (Max-Max) IDRs do not exceed the value of 4.5% and the mean of maximum (Mean-Max) IDRs do not exceed the value of 3% according to the TBI guidelines. The scattering of the results are more significant in lower stories and in some parts of upper stories (around floor 39). In lower stories, concentration of dispersed IDRs is attributed to p-delta effects while sudden jump of IDRs in upper stories is associated with the higher mode and the ‘Top Hat’ effects in combination. On the other hand, the dispersion of the IDRs in intermediate stories is almost uniform, indicating less influence of ground motion frequency contents on the IDR variations.

In Figure 5 (a) the results of drifts for the tallest frame in the building (selected frame in Figure 1 (a)) in the radial direction are also compared to those obtained from displacements at X and Y directions. As can be observed, the differences between interstory drifts obtained from the tallest frame in radial direction and
those obtained from plan displacements in X and Y directions are attributed to the participation of other frames in the lateral response.

The heightwise distribution of the absolute values of Mean-Max/Max-Max RDRs obtained from NLRHAs in orthogonal directions (X, Y directions) along with SRSS values for the ‘Apartment Part’ and ‘Hotel Part’ are shown in Figure 7 (a)/(b) and Figure 8 (a)/(b), respectively. Values of permanent drifts corresponding to Mean-Max and Max-Max RDRs are justifiably less than the limiting values reported in TBI guidelines. Similar to the IDR results, mean of the maximum RDRs is more scattered near both lower and upper stories as observed in Figure 7 (c), Figure 8 (c). The result again accentuates p-delta and higher mode effects as two dominant modes of response for the building under study.

**Residual story drift**

RDRs may cause excessive post-earthquake displacements in the building and postpone the immediate operation after earthquake events. According to TBI guidelines, the mean of the absolute values of RDRs from s set of ground motions scaled to MCE level shall not exceed 1% in each story level, while this limiting value reach to 1.5% for maximum of the absolute values of RDRs in any analyses. The heightwise distribution of the absolute values of Mean-Max/Max-Max RDRs obtained from NLRHAs in orthogonal directions (X, Y directions) along with SRSS values for the ‘Apartment Part’ and ‘Hotel Part’ are shown in Figure 7 (a)/(b) and Figure 8 (a)/(b), respectively. Values of permanent drifts corresponding to Mean-Max and Max-Max RDRs are justifiably less than the limiting values reported in TBI guidelines. Similar to the IDR results, mean of the maximum RDRs is more scattered near both lower and upper stories as observed in Figure 7 (c), Figure 8 (c). The result again accentuates p-delta and higher mode effects as two dominant modes of response for the building under study.

**Plastic rotation acceptance criteria**

According to the requirements in TBI, ultimate beam plastic hinge rotation for beams in moment frames can be obtained from Chapter 3 of PEER/ATC72. Ultimate beam hinge rotation in the building is determined on the basis of the following equation as stated in Figure 4 (a):

\[
\theta_u = 1.5(\theta_p + \theta_y)
\]  

(5)
Beam geometries and material characteristics in the building prove that the ultimate plastic rotation for different beams is around 4%, as a result, this value is accepted as an acceptable criterion. In Figure 9 (a), the heightwise profile of Mean-Max beam end plastic rotations for ‘Apartment Part’ and ‘Hotel Part’ is illustrated. As can be observed end beam rotation almost satisfies TBI acceptance limit along the height. Furthermore, the dispersions of the Mean-Max plastic rotations for apartment and hotel parts of the building are shown in Figure 9 (b), Figure 9 (c), respectively. As can be seen, there is not much scattered results along the height except in the limited number of stories. Additionally, Mean-Max plastic rotations are not necessarily mimic the same heightwise pattern demonstrated for the Mean-max of IDRs or RDRs.

**CONCLUSIONS**

In this paper the capability of code-specified requirements, ASCE7-10, for proportioning and detailing of a case study high-rise moment-frame tower to resist against ground motions scaled to MCE hazard level was investigated. Analytical modelling of the structure was simulated based on the recommended parameters in TBI guidelines, Peer/ATC72, and FEMA P440A to incorporate stiffness and strength deterioration in nonlinear history analysis. Acceptance performance criteria corresponding to mean/maximum IDRs, RDRs and end beam plastic rotations were obtained from the recommendations reported in TBI guidelines. The results demonstrated that all the limiting criteria declared by TBI guidelines to provide safety margin against collapse were justifiably satisfied. Dominant modes of response during nonlinear history analyses were governed by p-delta effects and higher mode effects in lower stories and upper stories, respectively. Heightwise profile of plastic hinge rotations at beam ends was rather uniform that indicated spreading of plasticity through different stories.

**REFERENCES**


PEER NGA-West2 Data Base (2014). *Pacific Earthquake Engineering Research (PEER)*, University of California, Berkeley. Available from: http://ngawest2.berkeley.edu/


