EVALUATION OF SEISMIC DEMAND OF COLUMNS AND BEAMS IN TWO-STORY X SPECIAL CONCENTRICALLY BRACED FRAMES

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ABSTRACT

Special Concentrically Braced Frames (SCBFs) are widely used as seismic-force resisting system. Design basis of SCBFs is intended to assure that the desired nonlinear behavior is achieved through restricting inelastic deformation in braces. Following capacity design approach of seismic design provisions, other framing elements including columns and beams should be stronger than the braces. Columns are the most essential members in buildings to support gravity loads, resist lateral forces and provide stability of the structure. In order to determine force demands in columns, Nonlinear Response History Analysis (NRHA) should be done. The primary objective of current study is to evaluate the prescribed methods of AISC 341 for seismic demand of columns and beams in SCBFs. Four archetype 2 to 8-story frames with two-story X-bracing configurations have been studied. The numerical models of the SCBFs were developed in OpenSees software, in which simplified discrete component models including nonlinear geometric effect for simulating brace buckling, fracture in braces and nonlinear constitutive law for steel are considered. The modeling has been verified with results obtained by tests performed on three-story single-bay frames. NRHA is implemented to find the seismic force demands of columns and beams for 18 scaled near-field and far-field records. Seismic demands of columns and beams are compared with design provisions of AISC 341. The NRHA results show that the prescribed elastic analysis of SCBFs are generally conservative for axial load demand in columns and flexural moment demand in beams, but elastic analysis cannot capture shear and moment demand in columns.

Keywords: Seismic Demand; Nonlinear Response History Analysis; Capacity Design; SCBF; Column

1. INTRODUCTION

Special Concentrically Braced Frames (SCBFs) are widely used as the seismic resisting system in buildings all around the world. These frames are able to provide high strength and stiffness and have serviceable performance during smaller, more frequent earthquakes. During severe infrequent earthquakes, they are expected to deform inelastically and dissipate the imparted energy through tensile yielding and post-buckling deformation of the bracings. Design basis of SCBFs is intended to assure that the desired inelastic deformation is achieved. According to ASCE 7 (ASCE, 2010) approach, seismic hazard zones are specified as Seismic Design Category (SDC) based on mapped spectral acceleration parameters (SDS and SD1) and occupancy risk categories. In this regard, ASCE 7 requires employing SCBF system for SDC D, E, or F which is equivalent to zone 3 and 4 of BHRC (BHRC, 2014) in Iran. Similarly, in the latest revision of the Iranian seismic design code, employing SCBF is prescribed for most buildings of high seismicity zones.

Columns are the most essential members in buildings to resist the lateral forces and provide stability of the structure in addition to supporting gravity loads. For design of the columns in SCBFs, Seismic Provisions for Structural Steel Buildings AISC 341 (AISC, 2010a) and 4th ed. of Iranian National Building Codes for Structural Steel Design INBC 10 (INBC, 2012), stipulated that the required strength of columns, beams and connections in SCBF shall be based on the load combinations that include the amplified seismic load. In determining the amplified seismic load, as depicted in Figure 1, the effect of...

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horizontal seismic forces, shall be taken as the larger force determined from two following analyses: (1) an analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension, and (2) an analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength. AISC 341, section F2.3 mentioned that the required strength of columns need not exceed the forces determined from nonlinear analysis. It is also stated in Section D1.4 of AISC 341-10 that all lateral seismic resisting columns must be designed against axial forces obtained from load combinations that include the amplified seismic load using over-strength factor. In order to determine the upper bound of axial force demands in columns, a nonlinear response history analysis should be done.

For better understanding and improving the seismic performance of Concentrically Braced Frames (CBFs), numbers of experimental and analytical studies have been performed. The experimental investigations include the testing of component members and full systems and provide the basis of the development of the SCBF design requirements. Various simulation methods were suggested to represent the seismic behavior of CBFs based on earlier experimental research. The typical failure modes experienced by SCBF buildings due to earthquake excitation include damage to braces, brace-to-framing connections, columns and base plates has been studied (i.e. Kato et al, 1987; Osteraas and Krawinkler, 1989; Tremblay et al, 1995; Kelly et al, 2000). Much experimental and analytical investigation has been conducted on the performance of braces (i.e. Tremblay et al, 2003; Yang and Mahin, 2005). The performance of gussets has been investigated by some researchers (i.e. Astaneh-Asl, 1982; Roeder et al, 2011). Some studies have been conducted on seismic behavior of concentrically braced frames (i.e. Uriz et al, 2008; Tremblay, 2008). Recently, research of SCBF buildings has focused on the experimental and numerical performance of braces, brace-to-framing connections and systems (i.e. Lehman and Roeder, 2008; Chen, 2010; Hsiao et al, 2012; Hsiao et al, 2013; Karamanci et al, 2014).

The primary objective of the research in this paper is to evaluate the aforementioned prescribed methods of AISC 341 and INBC 10 for seismic demand of columns and beams in SCBFs. Furthermore, accuracy and reliability of these methods is investigated. Some archetypes 2, 4, 6 and 8-story X type have been selected. They were designed based on AISC 341. The numerical models of the SCBFs were developed in OpenSees structural analysis software. Analytical modeling has been verified by results obtained by test performed by Lumpkin in University of Washington (2009). Nonlinear Dynamic Analysis is implemented to find the axial force, shear force and bending moment demands of columns and bending moment demand in beams.
2. NON-LINEAR DYNAMIC ANALYSIS PROCEDURE

Non-linear dynamic analysis performed using OpenSees Software (Open System for Earthquake Engineering Simulation, n.d.). Modeling was done by text file based on TCL which is to string based command language. Pre-defined steel material named “Steel02” where is based on Giuffre–Menegotto–Pinto hysteretic model, assigned to structural elements. Low cycle fatigue effect considered for steel material. Fiber cross sections were employed, which enable the creation of the various steel cross sections with the assumption of plane strain compatibility. For discretizing of sections flanges and webs of sections are gridded and divided to some small rectangular parts as shown in Figure 2.

![Discretized sections with fiber type finite elements method: (a) HSS; (b) Wide Flange](image)

2.1 Modeling of Braces

Braces are expected to act as fuse elements during seismic excitations and absorb and dissipate imparted earthquake energy with non-linear buckling behavior. To provide buckling potential in braces, an initial imperfection should be considered. A parabolic curvature with a lateral displacement equal to 1/500 of brace length in mid-span was chosen for any brace. Each brace divided into 16 segments with 4 integration points. As shown in Figure 2(a), the HSS tube was discretized using 2×8 fibers along the thickness in each flange and 2×16 fibers along the thickness in each web.

2.2 Modeling of Beams and Columns

Rigid zones of beam and columns modeled as elastic beam-column elements with cross section area and moment of inertia equal to 10 times of relevant section. For beams and columns, 3 and 4 integration point was chosen, respectively. As shown in Figure 2(b), the wide flange sections were discretized using 2×8 fibers along the thickness in each flange and 2×16 fibers along the thickness in web.

2.3 Modeling of Gusset Plate Connections

For simplifying the brace to frame connection through gusset plate, rotational spring model proposed by Hsiao et al (2013) was used. In this model a non-linear spring was modeled at the physical ends of braces. Rotational stiffness and yield moment of spring are defined as follows:

\[ K_{rotational}^{cal} = \frac{E}{L_{ave}} \left( \frac{W_w t^3}{12} \right) \]

\[ M_y = \frac{W_w t^2}{6} F_{y, gusset} \]

In these equations \( K_{rotational}^{cal} \) is rotational stiffness and \( M_y \) is yield moment of the spring. \( E \) and \( F_{y, gusset} \) are elastic modulus and yield stress of gusset plate material, respectively. As presented in Figure 3, \( W_w \), \( L_{ave} \) and \( t \) are effective Whitmore width, average effective length and thickness of gusset plates, respectively.
2.4 Verification of Non-Linear Dynamic Modeling

Non-linear model has been verified using experimental data results of a three story frame which was tested by National Center for Research on Earthquake Engineering (NCREE) in Taiwan, and the Universities of Washington (Lumpkin, 2009), as shown in Figure 4.

The hysteretic shear-drift curves of each story compared for evaluation accuracy of numerical simulation. For example, Figure 5 shows the results of experiment and analytical model of the first story. The numerical model can adequately simulate behavior of tested CBF system.
3. DESCRIPTION AND DESIGN OF MODEL FRAMES

In this study, 4 two-dimensional frames of an office building with 2, 4, 6 and 8 stories were modeled. These frames have five bays with equal spacing of 6 m each. The story height was 4 m for 1st story and 3.5 m for other stories. Figure 6 shows typical building plan and relevant elevation of the four frames. The lateral resisting system of the frames were special concentrically braced frames with double story-X bracing configuration that brace the building in 2 non-adjacent bays. For each floor, similar loading was considered as dead load=5.0 kN/m², partition load=1.0 kN/m² and live load=2.5 kN/m². American wide flange sections were selected for beam and columns and HSS sections were used for braces. Material of all sections are ASTM-A992 with $F_y=350$ MPa and $F_u=450$ MPa.

![Figure 6. Typical plan and elevations of model frames](image)

These office buildings are considered to be located in Tehran with seismic parameters $SD_1=0.52$ and $SD_s=1.11$ based on probable seismic hazard analysis of Iran available in Iran hazard website (Iran Seismic Hazard Probabilistic Analysis, n.d.).

3.1 Linear Static Analysis

Linear Static Analysis prescribed by Seismic Provisions for Structural Steel Buildings AISC 341 was done using software ETABS ver. 9.7.4.

3.2 Design of Main Members of Frames

Columns, beams and braces were designed by LRFD method according to AISC 360 (AISC, 2010b) and AISC 341 (AISC, 2010a). First, braces size was determined for general load combination addressed in ASCE 7 (ASCE, 2010). The required strength of beams and columns was determined based on amplified seismic loads regarding to the capacity design procedure of ASCE 341 described before.

3.3 Design of Connections

According to section 2.3, dimensions of gusset plate connections are required to determine the rigid zone of beam-column connection and the rotational spring behavior which is assigned in non-linear model. For this purpose, all connections were designed based on amplified seismic loads.

3.4 Ground Motion Records

For dynamic response analysis, 18 records were selected from records used in SAC projects (FEMA, 2008) to cover a range of frequency content, time length, amplitude, distance and fault mechanism where 11 records were Far-Field and 7 were Near-Field. Selected records are listed in Table 1. Ground excitation data was obtained from PEER website (PEER Ground Motion Database, n.d.).
Table 1. Summary of earthquake events and recording station data

<table>
<thead>
<tr>
<th>Record Type</th>
<th>ID #</th>
<th>M</th>
<th>Year</th>
<th>Name</th>
<th>Recording Station Name</th>
<th>Owner</th>
</tr>
</thead>
<tbody>
<tr>
<td>Far Field</td>
<td>1</td>
<td>7.6</td>
<td>1999</td>
<td>Chi-Chi, Taiwan</td>
<td>TCU045</td>
<td>CWB</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.5</td>
<td>1976</td>
<td>Friuli, Italy</td>
<td>Tolmezzo</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>7.1</td>
<td>1999</td>
<td>Hector Mine</td>
<td>Hector</td>
<td>SCSN</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6.5</td>
<td>1979</td>
<td>Imperial Valley</td>
<td>El Centro Array #11</td>
<td>USGS</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>6.9</td>
<td>1995</td>
<td>Kobe, Japan</td>
<td>Nishi-Akashi</td>
<td>CUE</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>7.5</td>
<td>1999</td>
<td>Kocaeli, Turkey</td>
<td>Arcelik</td>
<td>KOERI</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>7.3</td>
<td>1992</td>
<td>Landers</td>
<td>Coolwater</td>
<td>SCE</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>6.9</td>
<td>1989</td>
<td>Loma Prieta</td>
<td>Capitola</td>
<td>CDMG</td>
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<tr>
<td></td>
<td>9</td>
<td>7.4</td>
<td>1990</td>
<td>Manjil, Iran</td>
<td>Abbar</td>
<td>BHRC</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>6.7</td>
<td>1994</td>
<td>Northridge</td>
<td>Beverly Hills - Mulhol</td>
<td>USC</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>6.6</td>
<td>1971</td>
<td>San Fernando</td>
<td>LA - Hollywood Stor</td>
<td>CDMG</td>
</tr>
<tr>
<td>Near Field</td>
<td>12</td>
<td>6.7</td>
<td>1992</td>
<td>Erzican, Turkey</td>
<td>Erzincan</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>13</td>
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<td>1979</td>
<td>Imperial Valley-</td>
<td>El Centro Array #7</td>
<td>USGS</td>
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<td>6.9</td>
<td>1989</td>
<td>Loma Prieta</td>
<td>Corralitos</td>
<td>CDMG</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>6.9</td>
<td>1989</td>
<td>Loma Prieta</td>
<td>Saratoga - Aloha</td>
<td>CDMG</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>6.8</td>
<td>1985</td>
<td>Nahanni, Canada</td>
<td>Site 1</td>
<td>--</td>
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<tr>
<td></td>
<td>17</td>
<td>6.7</td>
<td>1994</td>
<td>Northridge-01</td>
<td>Northridge - Saticoy</td>
<td>USC</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>6.7</td>
<td>1994</td>
<td>Northridge-01</td>
<td>Sylmar - Olive View</td>
<td>CDMG</td>
</tr>
</tbody>
</table>

3.5 Scaling of Ground Motions

The accelerograms were scaled regarding to procedure mentioned in chapter 16 of ASCE7 (ASCE, 2010) so that the average 5% damping spectrum of all scaled records in the range of 0.2T to 1.5T is not less than corresponding values in the standard design spectrum, where T is the fundamental period of the structure. For instance, Figure 7 shows the spectrum of scaled records for 4 story frame. It should be noted that the lower period bound governed the average scaled records.

![Figure 7. Response spectrum of scaled records for 4 story frame](image-url)
4. RESULTS

Findings from Linear Static Analysis (LSA) and Non-Linear Dynamic Analysis (NDA) are presented in this section. Distribution diagrams for axial compression, shear and bending moment demands of columns and beams related to 2, 4, 6 and 8 story frames are shown in Figures 8 to 12. The notations used in diagrams and texts are introduced in Table 2. For the NDA, the average results of 18 scale ground motion in addition to average plus and minus standard deviation are plotted. The nominal capacity of beam and column members are computed in accordance with AISC 360 (AISC 2010b) provisions for axial force, shear force and bending moment. The frame actions are presented in terms of demand capacity ratio.

Table 2. Notations used in diagrams of results and texts

<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSA1</td>
<td>Linear Static Analysis, LRFD General Combination</td>
</tr>
<tr>
<td>LSA2</td>
<td>Linear Static Analysis, LRFD Amplified Combination using overstrength factor of 2</td>
</tr>
<tr>
<td>LSA3</td>
<td>Linear Static Analysis, Amplified Seismic Loading with expected strength of braces (Yielding + Buckling)</td>
</tr>
<tr>
<td>LSA4</td>
<td>Linear Static Analysis, Amplified Seismic Loading with expected strength of braces (Yielding + Post-Buckling)</td>
</tr>
<tr>
<td>LSA5</td>
<td>Linear Static Analysis, LRFD Amplified Combination with Tension Only Braces</td>
</tr>
<tr>
<td>NDA</td>
<td>Non-Linear Dynamic Analysis, Average Response</td>
</tr>
<tr>
<td>NDA-</td>
<td>Non-Linear Dynamic Analysis, Average - Standard Deviation of Responses</td>
</tr>
<tr>
<td>NDA+</td>
<td>Non-Linear Dynamic Analysis, Average + Standard Deviation of Responses</td>
</tr>
<tr>
<td>D/C</td>
<td>Demand per Nominal Capacity Ratio</td>
</tr>
</tbody>
</table>

4.1 Evaluation of Axial Force Demands of Columns

Figure 8 shows the height-wise distribution curves of axial compression force demand capacity ratios (D/C) for 2, 4, 6 and 8 story frames. Linear Static Analysis based on design code shows lower demands than Non-Linear Analysis, although, demands in upper stories is close together. Axial tension D/C ratios for 2, 4, 6 and 8 story frames are presented in Figure 9.

4.2 Evaluation of Shear Force Demands of Columns

Distribution diagrams of shear force D/C ratios for 2, 4, 6 and 8 story frames, are presented in Figure 10. Whereas, in typical linear modeling of concentrically braced frames, connections of braces and beams generally are modeled as hinge, thus in this vertical truss behavior, only axial force is imposed in columns. But, in realistic non-linear modeling, dynamic response analysis shows significant shear force demand in columns that is not considered in design code provisions. For average NDA results, however, maximum shear force D/C ratio of columns can reach around 0.4 in mid-stories of braced frames.

4.3 Evaluation of Bending Moment Demands of Columns

Figure 11 illustrates distribution diagrams of maximum bending moment D/C ratios for 2, 4, 6 and 8 story frames. Similar to shear force responses, considerable bending moment demands in columns can be observed in non-linear dynamic history response analysis, which is not considered in linear analysis and corresponding code-based design provisions. The maximum bending moment D/C ratio of columns in average NDA results is 0.92, 0.58, 0.94, and 0.88 for 2, 4, 6, and 8 story braced frames, respectively. This high moment D/C ratio of columns in combination with axial force means columns in SCBF are vulnerable to significant damage at design-level earthquake events.
Figure 8. Height-wise distribution of axial compression force D/C ratio for columns: (a) 2-story; (b) 4-story; (c) 6-story; (d) 8-story

Figure 9. Height-wise distribution of axial tension force D/C ratio for columns: (a) 2-story; (b) 4-story; (c) 6-story; (d) 8-story
Figure 10. Height-wise distribution of shear force D/C ratio for columns:
(a) 2-story; (b) 4-story; (c) 6-story; (d) 8-story

Figure 11. Height-wise distribution of bending moment D/C ratio for columns:
(a) 2 story; (b) 4 story; (c) 6 story; (d) 8 story
4.4 Evaluation of Bending Moment Demands of Beams

Distribution diagrams of maximum bending moment D/C ratios of beams for 2, 4, 6 and 8 story frames are presented in Figure 12. Generally, bending moment D/C ratio of beams is within acceptable range.

5. CONCLUSIONS

In this paper, response history analysis results of realistic non-linear model of SCBF have been presented to evaluate seismic demands of columns and beams. Numerical model results conformed adequately to experimental results of a full scale 3-story braced frame. Analysis reveals that axial compression and tension force demands comply with required strength specified in design code provisions and procedure of code, especially for columns in bottom stories, is conservative. But, linear modeling of CBFs is not able to estimate shear forces and bending moments in columns properly. Non-linear dynamic analysis shows significant shear and bending moment demands in columns. Since large shear and bending moment demands occur in columns, more investigation should be done for understanding the real seismic behavior of columns. This considerable shear and bending demand in column can mainly be attributed to post buckling behavior of compression braces. Bending moment demand of beams in NDA is satisfactory less than code-base linear analysis results.

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