Assessment Of Response Reduction Factor For Asymmetric RC Frame Building

Kruti J. Tamboli1, J A. Amin2

1 Student, Dept. of Civil Engineering, Sardar Vallabhbhai Patel Institute of Technology, Vasad-388306, Gujarat, India
2 Associate Professor, Dept. of Civil Engineering, Sardar Vallabhbhai Patel Institute of Technology, Vasad-388306, Gujarat, India

Abstract — Response reduction factor is the factor by which the actual base shear force should be reduced, to obtain the design lateral force during design basic earthquake (DBE) shaking. The R factors in many developing countries are often adopted from the well developed seismic design codes used in the United States or Europe. These R factors provide false representation for the structural practices applied in developing countries and thus considered unrealistic. So there is a need to come up with realistic R factors for various structural systems used in such countries. In the present study efforts are made to evaluate the response reduction factor and ductility of asymmetric RC braced frame using nonlinear static pushover analysis. The types of the frame considered in this study are asymmetric RC frame with X bracing at alternate bay, shear wall at alternate bay.

Keywords- asymmetric RC frame building; bracing; ductility; pushover analysis; response reduction factor

I. INTRODUCTION

The earthquake is a phenomenon that releases high amount of energy in a short time through the earth. Structures designed to resist moderate and frequently occurring earthquakes must have sufficient stiffness and strength to control deflection and prevent any possible collapse. In other words, a structure not only should dissipate a considerable amount of imported energy by ductile behavior, but also it should be able to control the deformations and transfer the force to foundation through enough lateral stiffness in ground motions. Braced frame systems offer an attractive solution to satisfy multiple design objectives. Their elastic properties provide the stiffness and strength needed to achieve operational performance objectives, which are primarily defined by the performance of non-structural elements. In present times buildings are often constructed with irregularities such as soft stories, torsional irregularities, vertical and plan irregularities. Past earthquake studies shows that the most of the RC buildings having these types of irregularities were severely damaged under the seismic ground motion. Torsional effects may significantly modify the seismic response of buildings and causes collapse of structure. These effects generate due to different reasons like non uniform distribution of mass, stiffness and strength needed to achieve torsional components of the ground movement. These types of buildings undergo coupled lateral and torsional motion during earthquake. This particular behavior under earthquake excitation usually leads to classify such type of buildings asymmetric systems. In many buildings the center of resistance does not coincide with the center of the mass. By reducing the distance between the center of mass and stiffness, torsional effects should be minimized. The choice of the stiffness characteristics of structures is an important part in the design process. Difficulties related to the study of their seismic response can be identified using pushover analysis. Current force-based design procedure adopted by most seismic design codes allows the seismic design of building structures to be based on static or dynamic analyses of elastic models of the structure using elastic design spectra. The codes anticipate that structures will undergo inelastic deformatons under strong seismic events; hence, such inelastic behaviour is usually incorporated into the design by dividing the elastic spectra by a factor, R, which reduces the spectrum from its original elastic demand level to a design level. The most important factors determining response reduction factors are the structural ductility and overstrength capacity.

II. LITERATURE REVIEW

Maheri and Akhbari [1] evaluated the seismic behavior factor for steel X-braced and knee-braced RC buildings from inelastic pushover analyses of brace-frame systems of different heights and configurations and tentative R values are proposed for steel-braced moment-resisting RC frame dual systems for different ductility demands. Charles et al. [2] evaluate Finite element (FE) analyses to understand the nonlinear, cyclic behavior of multi-story X-braced frames and their gusset plate connections and developed the FE model. The results show that the design and detailing of the gusset plate has a significant impact on the seismic performance of the frame. The results also suggest that floor slabs, gusset plate stiffeners and framing member sizes affect the frame performance and must be considered in the analysis and design of the system. Kadid and Yahiaoui [3] investigate the seismic behavior of RC buildings strengthened with different types of steel braces, X-braced, inverted V braced, ZX braced, and Zipper braced. Static non linear pushover analysis has been conducted to estimate the capacity of three story and six story buildings with different brace-frame systems and different cross sections for the braces. It is found that adding braces enhances the global capacity of the buildings in terms of strength, deformation and ductility compared to the case with no bracing.
Maheri et al. [4] studied the pushover experiments conducted on scaled models of ductile RC frames, directly braced by steel X and knee braces. Test results indicate that the yield capacity and the strength capacity of a ductile RC frame can be increased and its global displacements can be decreased to the desired levels by directly adding either an X- or a knee-bracing system to the frame. Desai et al. [5] presented experimental tests for four full-scale frames with y-bracings of different geometries and cross sections conducted at BHRC1 structural engineering laboratory. The results show that out-of-plane buckling with single curvature in braces can be replaced by in-plane, double curvature buckling through appropriate detailing of cross sections and connections. Maheri and Hadjipour [6] discussed results of static tests on full-scale connections for internally braced reinforced concrete frames are. Three types of brace/RC frame connections, suitable for the seismic retrofitting of existing RC frames and the seismic design of frames under construction, are investigated. It is concluded that steel brace/RC frame connections can be designed successfully by combining the appropriate current provisions from the codes of practice for steel and concrete structures.

III. CONCEPT OF RESPONSE REDUCTION FACTOR

The code provision allows the structure to be damaged in the case of severe shaking. Hence, the structure is designed for seismic force much lesser than that expected under strong earthquakes, if the structure were to remain linearly elastic. Thus, the Indian seismic standard IS 1893 provides for a realistic force for an elastic structure and then divides that force by 2R. In other words, the term R gives an indication of the level of over strength and ductility that a structure is expected to have. Thus, the structure can be designed for much lower force than is implied by the strong shaking by considering the following factors, which will prevent the collapse of the structure:

1. Over strength factor (R_s)
2. Redundancy factor (R_R)
3. Ductility factor (R_μ)

Over strength factor accounts for the yielding of a structure at load higher than the design load due to various Partial safety factors, strain hardening, oversized members, confinement of concrete. Non-structural elements also contribute to the over strength.

\[ R_s = \frac{V_o}{V_d} \]

Where, \( V_o \) = base shear at the roof displacement corresponding to the limiting state of response,
\( V_d \) = design base shear according to IS: 1893 (Part 1) 2002

Redundancy factor depends on the number of vertical framing participate in seismic resistance. Yielding at one location in the structure does not imply yielding of the structure as a whole. Hence the load distribution, due to redundancy of the structure, provides additional safety margin. Ductility is the capacity of material/structure to absorb energy by deforming into the inelastic range.

<table>
<thead>
<tr>
<th>Lines of vertical seismic framing</th>
<th>Draft redundancy factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.71</td>
</tr>
<tr>
<td>3</td>
<td>0.86</td>
</tr>
<tr>
<td>4</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Ductility factor is a ratio ultimate displacement/ code specified to the yield displacement. In present study equation derived by Miranda and Beretero is used to find the ductility factor \( R_\mu \).

\[ R_\mu = (\mu - 1/\Phi) + 1 \]

Where:

For rock site:
\[ \Phi = 1/((10T - \mu T) - (1e^{1.5(ln(25 - 0.6)^2/2T)}) \]

For alluvium site:
\[ \Phi = 1/((12T - \mu T) - (2e^{2(ln(250.2)/5T)})) \]

For soft soil site:
\[ \Phi = 1/((T_g/3T) - (3T_g e^{3(ln(15T_g)/0.25)^2}/)) \]

Where, \( T_g \) is the predominant period of the ground motion.
IV. FRAME CONSIDERED IN THE STUDY

In the present study a typical 4-story RC frame has been taken for the analysis. The floor plan and elevation of the building is shown in Fig:2. The building parameters are defined as, building plan dimension 18m x 9m, Concrete Grade – M25, Steel Grade – fy 415 MPa. Slab Thickness – 150mm, height of each storey – 3.2m. Live load is taken as 2 kN/m² at floor level and no liv load at roof level. It is assumed that the building is located in zone-IV and soil type is medium.

Table 2. Beam size and reinforcement detail

<table>
<thead>
<tr>
<th>Beam</th>
<th>Beam size (mm)</th>
<th>Top reinforcement</th>
<th>Bottom reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; story</td>
<td>300x550</td>
<td>4 bars of 20mm dia.</td>
<td>4 bars of 20mm dia.</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; story</td>
<td>300x550</td>
<td>4 bars of 18mm dia.</td>
<td>4 bars of 18mm dia.</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; story</td>
<td>300x550</td>
<td>4 bars of 16mm dia.</td>
<td>4 bars of 16mm dia.</td>
</tr>
<tr>
<td>4&lt;sup&gt;th&lt;/sup&gt; story</td>
<td>300x550</td>
<td>4 bars of 12mm dia.</td>
<td>4 bars of 12mm dia.</td>
</tr>
</tbody>
</table>
Table 3. Column size and reinforcement detail

<table>
<thead>
<tr>
<th>Column</th>
<th>Column size (mm)</th>
<th>Top reinforcement</th>
<th>Bottom reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st story</td>
<td>375x375</td>
<td>4 bars of 16mm dia.</td>
<td>4 bars of 16mm dia.</td>
</tr>
<tr>
<td>2nd story</td>
<td>375x375</td>
<td>3 bars of 16mm dia.</td>
<td>3 bars of 16mm dia.</td>
</tr>
<tr>
<td>3rd story</td>
<td>375x375</td>
<td>2 bars of 16mm dia.</td>
<td>2 bars of 16mm dia.</td>
</tr>
<tr>
<td>4th story</td>
<td>375x375</td>
<td>2 bars of 12mm dia.</td>
<td>2 bars of 12mm dia.</td>
</tr>
</tbody>
</table>

The calculated design seismic base shear and corresponding lateral forces on the frame as per IS: 1893, are shown in Table 4.

Table 4. Storey forces for 4 – storey building

<table>
<thead>
<tr>
<th>Floor level</th>
<th>W_i (KN)</th>
<th>h_i (m)</th>
<th>W_i h_i^2</th>
<th>Q_i (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>1788.535</td>
<td>12.8</td>
<td>293034.06</td>
<td>259.64</td>
</tr>
<tr>
<td>3rd floor</td>
<td>2875.61</td>
<td>19.6</td>
<td>265016.21</td>
<td>234.81</td>
</tr>
<tr>
<td>2nd floor</td>
<td>2875.61</td>
<td>6.4</td>
<td>117784.98</td>
<td>104.36</td>
</tr>
<tr>
<td>1st floor</td>
<td>2875.61</td>
<td>3.2</td>
<td>29446.21</td>
<td>26.09</td>
</tr>
<tr>
<td>∑W_i h_i</td>
<td></td>
<td></td>
<td>705281.49</td>
<td></td>
</tr>
</tbody>
</table>

Where \( A_h = \) design horizontal seismic coefficient for the structure, and may be calculated using, \( A_h = (Z/2)x(I/R)x(S_a/g) \)

Where, \( Z = \) Zone Factor = 0.24 (from Table 2 of IS 1893 (Part 1):2002)
\( I = \) Importance Factor = 1 (from Table 6 of IS 1893 (Part 1):2002)
\( R = \) Response Reduction Factor = 5 (from Table 7 of IS 1893 (Part 1):2002)
\( S_a/g = \) average Response acceleration coefficient for medium soil = 2.5 (from Figure IS1893 (Part1):2002)
Therefore \( A_h = 0.06 \)

This design base shear is distributed along the length of the building as follows: \( Q_i = ( \frac{V_i}{B_i} \frac{W_i h_i^2}{(\sum W_i h_i^2)} ) \)

Different RC frame configurations considered in the present study are:

4.2. Bare frame
4.3. Asymmetric RC frame with X-bracing in alternate bay
4.4. Asymmetric RC frame with Shear wall in alternate bay

4.1. Analytical modeling of Reinforced concrete members
Plasticity in RC members was assumed to be lumped at probable locations. The program defined plastic hinge properties of SAP2000 Program as per FEMA356 provisions were used to take into account the material nonlinearity. Plastic hinges that generally develop are those corresponding to flexure and shear. Flexural hinge properties involve axial force, bending moment interaction (P-M) as the failure envelop and bending moment rotation (M-θ) as the corresponding load deformation relation; shear hinge properties involve shear force-shear deformation relation (V-Δ). The different inelastic stages were suggested on pushover curves to facilitate the description of behavior of different members of the RC frame building at various stages: (1) \( P_{CG} \): formation of plastic hinge in the ground-storey columns; (2) \( P_{BG} \): formation of plastic hinges in ground-storey beams; (3) \( P_{C1} \): formation of plastic hinge in the first-storey columns; (4) \( P_{B1} \): formation of plastic hinges in first storey beams; (5) \( P_{C2} \): formation of plastic hinge in the second-storey columns; (6) \( P_{B2} \): formation of plastic hinges in second-storey beams; (7) \( P_{C3} \): formation of plastic hinge in the third-storey columns; (8) \( P_{B3} \): formation of plastic hinges in third-storey beams;

4.2. Bare frame
Bare frame represents the currently used common practice of not including the strength and stiffness of masonry in analysis and design procedure. The capacity curve obtained from the nonlinear pushover analysis of the bare RC frame building is shown in Figure 3. Linear behavior was observed in different member of RC frame up to a base shear of 13% of seismic weight and up to a roof displacement corresponding to 0.3% drifts. Non-linearity was observed to be well distributed along the height of the frame. Failure of the frame was found to take place due to flexure failure of open ground storey columns at the ultimate lateral load corresponding to 17% of seismic weight and lateral drift of 1.08%.
Figure 3. Pushover curve and plastic hinge formation details for bare frame

4.3. Asymmetric RC frame with X-bracing in alternate bay
From capacity curve shown in Figure 4, it was observed that first inelastic activity was observed at lateral load corresponding to 35% of seismic weight and 0.05% drift. The ultimate strength of frame was found to be at 68% of seismic weight and 0.15% of lateral drift. Plastic hinges were found to develop in all the ground as well as upper storey members. The value of β and μ are 9.31 and 3.24 accordingly for this type of frame.

Figure 4. Pushover curve and plastic hinge formation details for asymmetric RC bracing in alternate bay

4.4. Asymmetric RC frame with Shear wall in alternate bay
In this case the frame was strengthened by providing RC shear wall in only central bay of all stories. The remaining two bays of all stories were left open and the masonry walls in central bays of all stories were replaced with RC shear walls. RC shear walls were modeled as 4-noded nonlinear layered shell elements with thickness as 190 mm. Concrete and reinforcement were modeled as membrane layers. 0.25% reinforcement ratio was used in shear wall. First inelastic activity was observed at lateral load corresponding to 60% of seismic weight and 0.06% drift (Figure 5). The ultimate strength of frame was found to be at 77% of seismic weight and 0.1% of lateral drift. The value of β and μ are 7.30 and 3.65 accordingly for this type of frame.
V. EVALUATION OF RESPONSES REDUCTION FACTOR AND DUCTILITY FACTOR

The calculation for evaluation of response reduction factor as per ATC-19 for RC frame seismic zone-IV is shown below. The calculations of ‘R’ factor for other frames are not shown here due to paucity of space.

**Estimation of strength factor:**

Maximum Base Shear (from pushover curve)

\[ V_o = 1652 \text{ kN} \]

Design Base shear (as per EQ calculation)

\[ V_d = 624.92\text{kN} \]

Using equation for strength factor, given in ATC – 19

\[ R_s = \frac{V_o}{V_d} = \frac{1652}{624.92} = 2.64 \]

**Estimation of ductility factor:**

Maximum drift capacity (0.004 x H), \( \Delta m = 51.2 \text{ m} \)

Yield drift (from pushover curve), \( \Delta y = 29 \text{ mm} \)

Using equation for displacement ductility ratio, given in ATC – 19

\[ \mu = \frac{\Delta m}{\Delta y} = \frac{51.2}{29} = 1.76 \]

Equation for ductility factor, derived by Miranda and Bertero [7]

\[ R_\mu = \left( \mu - 1 / \Phi \right) + 1 \]

\[ \Phi \text{ for medium soil:} \]

\[ \Phi = 1 + \left[ \frac{1}{(12 \cdot T - \mu T)} - \frac{1}{(2 \cdot 5T)} \right] \cdot e^{-2(\ln T - 0.2)^2} \]

\[ T = 0.34\text{seconds (From analysis)} \]

\[ \Phi = 1.24 \]

\[ R_\mu = 1.61 \]

**Estimation of redundancy factor:**

The value of redundancy factor as suggested in ATC-19 is summaries in Table-1.

**Estimation of response reduction factor R:**

\[ R = R_s \times R_\mu \times R_R \]

\[ R = 2.64 \times 1.61 \times 1 \]

\[ R = 4.27 \]

Table 5 shows the value of R-factor and its key component of considered RC frame in this study. The values of the response reduction factor are considerably affected by the types and arrangements of the bracing systems. Provision of bracing in alternate bays increases the values of responses reduction factor nearly 2.81 times as compared to the bare RC frame respectively. Provision of shear wall in alternate bays increases the values of responses reduction factor nearly 2.94 times as compared to the bare RC frame respectively.
VI. CONCLUSIONS

In this study the lateral strength and response reduction factor (R) of 4-storey asymmetric RC braced frame are elevated using nonlinear static pushover analysis. The significant outcomes of the present study are summarized as follows:

- Large increase in lateral strength and stiffness of the RC braced frame was observed as compared to bare RC frame.
- Yield lateral force for the asymmetric RC frame with X-bracing at alternate bays was found to be 35% of seismic weight, and it was about 2.69 times more than that observed in the case of the bare RC frame respectively.
- Ultimate lateral force for the asymmetric RC frame with X-bracing at alternate bays was found to be 68% of seismic weight, and it was about 2.69 times more than that observed in the case of the bare RC frame respectively.
- Yield lateral force for the RC frame with shear wall at alternate bays was found to be 60% of seismic weight, and it was about 4.61 times more than that observed in the case of the bare RC frame respectively.
- Ultimate lateral force for the RC frame with shear wall at alternate bays was found to be 77% of seismic weight, and it was about 4.52 times more than that observed in the case of the bare RC frame respectively.
- The response reduction factor of asymmetric RC frame is considerably affected by the arrangements of the bracing systems. Provision of bracing/shear wall in alternate bays increases the values of responses reduction factor nearly 2.81 to 2.94 times respectively as compared to the bare RC frame respectively.

REFERENCES


Table 5. Values of R and μ

<table>
<thead>
<tr>
<th>Frame configuration</th>
<th>R_S</th>
<th>R_D</th>
<th>R_G</th>
<th>R</th>
<th>μ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare frame</td>
<td>2.64</td>
<td>1.61</td>
<td>1</td>
<td>4.27</td>
<td>1.76</td>
</tr>
<tr>
<td>RC bracing in alternate bay</td>
<td>6.34</td>
<td>1.46</td>
<td>1</td>
<td>9.31</td>
<td>3.24</td>
</tr>
<tr>
<td>Shear wall in alternate bay</td>
<td>5.29</td>
<td>1.37</td>
<td>1</td>
<td>7.30</td>
<td>3.65</td>
</tr>
</tbody>
</table>