Abstract

Increasing population in recent decades has made the necessity of quick low cost construction unavoidable. Meanwhile the use of prefabricated concrete frame with prefabricated concrete shear walls due to better quality of performance and reduction of manufacturing time is a good option for public multiple floor buildings. These structures in addition to resistance to seismic loads have more flexibility in methods of construction. There is little information about the exact behavior of prefabricated concrete frame with shear walls under earthquake and dynamic loads. Hence much research has been done by various researchers on the seismic behavior of these structures in the form of experimental and analytical models. Today, the main section of the seismic design of buildings is done based on equivalent static force method and calculating earthquake force of design from earthquake linear spectrum by applying a reduction coefficient called behavior coefficient of structure that embrace philosophy of design. So the necessity of determining behavior coefficient with respect to its importance in seismic design of structures seems essential. The behavior coefficient of structure is a coefficient that includes inflexible function of structure and indicates strength and hidden ductility of structure in inflexible stage.

In this study various types of pre-fabricated concrete frames together with pre-fabricated 4 and 8-storey shear walls with 1 and 3 bays are studied. The effect of two kinds of uniform and triangular loading on behavior factor has also been taken into consideration. Non-linear static analysis method (Push-over) has been used in order to determine the behavior factor.

On the whole, the results have shown that by increasing the bays and building storeys the structural behavior coefficient was increased.

KEYWORDS: prefabricated concrete; shear wall; ductility; behavior coefficient; push-over
1. INTRODUCTION

Components built somewhere other than the main site of the building and then are installed in the main site called Prefabricated concrete. Main reasons for using prefabricated concrete can be summarized in the following (Hass 1983):

- Being economical for performance and cost of consumed materials (formwork, scaffolding),
- Reducing construction time,
- Higher quality control,

Besides the above benefits one should also note the following problems (Hartland 1975):

- Connections between elements require careful attention. A glance at the damage caused by past earthquakes indicate that a high percentage of precast concrete structures ever made, doesn’t have sufficient and acceptable resistance to earthquakes so that the main reason is inappropriate behavior of moment resisting connections,
- Increasing problems of storage, transportation and installation (if the number of prefabricated components is large),

In seismic zones, one of the principles for designing structures is adequate lateral stiffness of structure for controlling inter-story drift to prevent damage to nonstructural elements. Also, during slight to moderate earthquakes, structural elements should remain elastic and have adequate resistance. During severe earthquakes also structure should have sufficient ductility to withstand large (inelastic) displacements so that the structure doesn’t fail (having ductile behavior). This tolerance should be such that no failure occurs. Accordingly, in structures designed to resist lateral forces, providing stiffness, strength and energy absorption capability are considered by designers simultaneously.

In prefabricated concrete frame with prefabricated shear wall system, the walls are connected together by a series of horizontal joints, and vertical joints are used where the walls are connected to columns. The horizontal joints are those lying between panels horizontally, and actually connect the upper part of a panel to the lower part of another. Vertical joints are also used to connect the walls to the columns laterally.

Nowadays, the most important part of seismic design of structures is carried out based on equivalent static method and calculation of seismic forces from linear spectrum of an earthquake, with application of a structural reduction factor, which contains the design philosophy. The reduction factor is a coefficient that contains inelastic behaviour and shows hidden resistance and ductility of structures in the inelastic zone. With attention of behavior factor signification in structural seismic design seems the determination of this factor is necessary.

Numerous researches have been made by various researchers about precast concrete walls vibratory behavior in the basis of experimental tests and analytical models. Some of theses researches are described hereunder:

In the year 2007 Bora and his colleague performed some researches on the wall connections to foundation in precast wall structures. Thin walls are usually discreted in the connected place to foundation, but in the presented method, the uplift force increase is prevented in the connected place causing the thin walls or hollow-core used as the shear wall to stand the seismic force. Main criteria to prevent brittle fracture in this way, the ductility and efficiency of wasting more energy of connection (Bora et al. 2007).

In the year 2008 Hashemi studied the made structures behavior coefficient with precast concrete large panels, and considered the effects of concrete resistance, change of number of bays and storeys as well as the effects of lateral loading on behavior coefficient. The results showed that the concrete resistance has little effect on behavior coefficient. On the whole, by increasing the number of bays and storeys, the
ductility will be decreased and consequently the force decrease coefficient \( R_\mu \) will be decreased, but by increasing the extra resistance coefficient \( R_S \) the behavior coefficient will be increased (Hashemi 2008).

In the year 2009 Aaleti and Stiharan made some researches on precast walls behavior. In this research they used from some pinned connection for between wall connections, and consequently by using from a simple analysis method, the wall behavior against the lateral load, depth of neutral axis and changing the form of pinned connections obtained in the form of function of roof drift (Aaleti and Sritharan 2009).

2. SEISMIC DESIGN BASEMENTS

Nowadays, the most important part of seismic design of structures is carried out based on equivalent static method and calculation of seismic forces from linear spectrum of an earthquake.

Knowledge of initial period and damping values are insufficient to define the seismic force intensity for a system exhibiting inelastic behaviour. To obtain the base-shear force, a standard approach is in use, based on the definition of a force reduction factor. Force reduction factor of MDOF consist of three parts:

\[
R = R_\mu \times R_S \times Y
\]

Where \( R_\mu \) is the ductility factor that is the same as SDOF systems, \( R_S \) is the over-strength factor that defined as the ratio of the actual to the design lateral strength, and \( Y \) is the allowable stress factor that be used to reduce base shear force at the point that the first plastic hinge accures, to the design force. Pushover diagram can be obtained by using static push over analysis for special structures. This diagram shows base shear force against increasing of roof displacement (Unag 1991). Figure 1 shows a sample pushover diagram.

In fact, because of ductility and inelastic behaviour of structures, elastic force \( V_e \) can be reduced to \( V_y \). So, ductility factor is defined as:

\[
R_\mu = \frac{V_e}{V_y}
\]

The ductility factor represents the minimum reduction coefficient corresponding to a specific level of ductility. The relationship between displacement ductility and ductility-dependent reduction factor has been the subject of considerable research. In this research, the equation that proposed by Miranda and Bertero for rock site have been used (Miranda et al. 1994).

Figure 1: pushover diagram (Hashemi 2008).
The equation for reduction factor introduced by Miranda and Bertero was obtained considering 124 ground motions recorded on a wide range of soil conditions. The soil conditions were classified as rock, alluvium and very soft sites characterized by low shear wave velocity. A 5% of critical damping was assumed. It is given as:

\[ R_\mu = \frac{\mu - 1}{\phi} + 1 \geq 1 \]  

(3)

\[ \mu = \frac{\Delta_{\text{max}}}{\Delta_y} \]  

(4)

Where \( \phi \) depends on \( \mu \) and \( T \), assumes different formulations for rock, alluvium and soft sites. For rock site it is as shown below:

\[ \phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} \exp\left[-1.5(\ln(T) - 0.6)^2\right] \]  

(5)

Previous research on the performance of buildings during severe earthquakes indicated that structural over strength plays a very important role in protecting buildings from collapse. The over strength factor (\( R_s \)) may be defined as the ratio of the actual to the design lateral strength:

\[ R_s = \frac{V_y}{V_S} \]  

(6)

Quantification of the actual over strength can be employed to reduce the forces used in the design, hence leading to more economical structures (Tasnimi and Massumi 1999). These include: the difference between the actual and the design material strength, load factors and multiple load cases.

For allowable stress design method, design codes reduce \( V_s \) to design force \( V_w \). This reduction is considered with the allowable stress factor (\( Y \)) as following:

\[ Y = \frac{V_s}{V_w} \]  

(7)

In the case of allowable stress designing of steel and concrete structures, the \( Y \) factor value could be in the range 1.4 - 1.5, but for ultimate design method the value of 1.0 is considered (Tasnimi and Massumi 1999).

3. VARIOUS HORIZONTAL AND VERTICAL JOINTS USED

Figure 2 shows the joints which were used. Figure (2 - a) shows RS horizontal joint presented by Soudki et al. (Soudki et al. 1995). Figure (2 - b) shows the vertical joint which has been suggested by Chakrabarti et al. was used (Chakrabarti et al. 1988).

The behavior of these joints has been shown in the Table 1.
Figure 2: Vertical and horizontal connections (Soudki et al. 1995; Chakrabarti et al. 1988).

Table 1: Parametric values of force-deformation curves for two types of connections (Soudki et al. 1995; Chakrabarti et al. 1988).

<table>
<thead>
<tr>
<th></th>
<th>RS</th>
<th>Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td>E (kg/cm²)</td>
<td>130460</td>
<td>750000</td>
</tr>
<tr>
<td>A (cm²)</td>
<td>4.91</td>
<td>1</td>
</tr>
<tr>
<td>K₀/K₀</td>
<td>0.24</td>
<td>0.38</td>
</tr>
<tr>
<td>F₀ (kg)</td>
<td>22420</td>
<td>4500</td>
</tr>
<tr>
<td>Fₚ (kg)</td>
<td>28540</td>
<td>19950</td>
</tr>
<tr>
<td>F₀/Fₚ</td>
<td>0.6</td>
<td>0.1</td>
</tr>
<tr>
<td>D₀</td>
<td>0.075</td>
<td>0.06</td>
</tr>
<tr>
<td>Dₗ</td>
<td>0.12</td>
<td>0.07</td>
</tr>
<tr>
<td>Dₑ</td>
<td>0.17</td>
<td>0.1</td>
</tr>
<tr>
<td>Dₙ</td>
<td>0.18</td>
<td>0.2</td>
</tr>
</tbody>
</table>

E = Youngs modulus, A = Bar area
4. STUDIED MODELS

The studied models in this research consist of 1 and 3-bay frames in 4 and 8 stories. In the 3-bay frames, the shear wall is once in the middle bay and the other time in the two side-bays. In this study, it is assumed that investigated structures have no torque and out of plane buckling, so they were modeled as 2D. Each bay is 6 meters with a height of 3.2 meters for each story. The obtained results from structures analysis, the values of various parameters of behavior factor were obtained.

Table 2 is the results relating to one and three bays frames and with height of 4 storeys which the behavior coefficient has been compared in two uniform and triangular loading status.

Table 3 is the results of non-linear analysis of one and three bays frames with 8 storeys. It is worth mentioning these behavior coefficients have been presented for designing with ultimate strength method.

Table 2: obtained Parameters from non-linear static analysis of 4-storey frame.

<table>
<thead>
<tr>
<th>Uniform loading</th>
<th>Number of bays</th>
<th>Number of shear wall</th>
<th>$T_e$</th>
<th>$\Delta y$</th>
<th>$\Delta_{max}$</th>
<th>$\mu$</th>
<th>$R_\mu$</th>
<th>$R_S$</th>
<th>$R = R_S \times R_\mu$</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.19</td>
<td>2.75</td>
<td>17.92</td>
<td>6.4</td>
<td>3.2</td>
<td>2.2</td>
<td>7.04</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>0.2</td>
<td>3.17</td>
<td>19.33</td>
<td>6.09</td>
<td>3.24</td>
<td>2.6</td>
<td>8.42</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>0.155</td>
<td>1.64</td>
<td>12.03</td>
<td>7.17</td>
<td>2.88</td>
<td>1.65</td>
<td>4.75</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Triangular loading</th>
<th>Number of bays</th>
<th>Number of shear wall</th>
<th>$T_e$</th>
<th>$\Delta y$</th>
<th>$\Delta_{max}$</th>
<th>$\mu$</th>
<th>$R_\mu$</th>
<th>$R_S$</th>
<th>$R = R_S \times R_\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.192</td>
<td>2.8</td>
<td>17.92</td>
<td>6.38</td>
<td>3.21</td>
<td>2.3</td>
<td>7.38</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>0.2</td>
<td>3.14</td>
<td>18.3</td>
<td>5.83</td>
<td>3.12</td>
<td>2.53</td>
<td>7.89</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>0.155</td>
<td>1.74</td>
<td>12.03</td>
<td>6.56</td>
<td>2.93</td>
<td>1.78</td>
<td>5.22</td>
<td></td>
</tr>
</tbody>
</table>

Note: $T_e$ in terms of second and $\Delta$ in terms of millimetre
Table 3: obtained Parameters from non-linear static analysis of 8-storey frame.

<table>
<thead>
<tr>
<th>Number of shear wall</th>
<th>Uniform loading</th>
<th>Triangular loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bays</td>
<td>$T_e$</td>
<td>$\Delta_y$</td>
</tr>
<tr>
<td>1 1</td>
<td>0.535</td>
<td>18.97</td>
</tr>
<tr>
<td>3 1</td>
<td>0.464</td>
<td>40.96</td>
</tr>
<tr>
<td>3 2</td>
<td>0.405</td>
<td>19.1</td>
</tr>
<tr>
<td>Number of shear wall</td>
<td>$T_e$</td>
<td>$\Delta_y$</td>
</tr>
<tr>
<td>1 1</td>
<td>0.536</td>
<td>8.13</td>
</tr>
<tr>
<td>3 1</td>
<td>0.45</td>
<td>35.33</td>
</tr>
<tr>
<td>3 2</td>
<td>0.381</td>
<td>15.1</td>
</tr>
</tbody>
</table>

Note: $T_e$ in terms of second and $\Delta$ in terms of millimeter.

5. CONCLUSION

In most codes, the behavior coefficient has been presented on the basis of structural system in the form of a fixed number. While this coefficient depends on different factors including ductility of structure, period, number of bays and structure height. In these codes, on the basis of one kind of structural system, a unique behavior coefficient has been presented for all period limits. It is impossible to provide a distinguished level of safety coefficient for all structures.

Having regard to the presented behavior coefficients in this paper, by increasing the number of bays and storeys, the behavior coefficient will be increased. The change of loading will have a few effect on the behavior coefficient. Increasing the number of bays with shear walls causses the decrease of behavior coefficient.

REFERENCES


