An Evaluation on Overstrength Factors of Reinforced Concrete Buildings

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ABSTRACT:

On August 17, 1999, the moment magnitude (Mw) 7.4 Kocaeli earthquake which was considered to be one of the most catastrophic events to have ravaged a highly industrialized province since the 1923 Tokyo earthquake, struck the Marmara Region of northwestern Turkey. Subsequently, on November 12, 1999 the Mw 7.2 Düzce earthquake hit the same region. Most of the collapsed and damaged structures were typically four-to-eight-story reinforced concrete buildings (RCBs). Due to the calamitous consequences of these earthquakes, Turkish Earthquake Code-2007 (TEC-07) and lastly Turkish Earthquake Code-2016 (TEC-16) draft version were accepted. One of the essential change in TEC-16 is earthquake load reduction factor, Rₐ and one of its parameters, overstrength factor (Ω) which depends on different factors and the most important of them is ductility (µ) that is also one of the most crucial keys to design earthquake resistant RCBs. This study mainly focuses on investigating of Ω values for three different stories (3, 6 and 9) of RCBs. Nonlinear dynamic time history analyses were performed according to TEC-16 by generating 3D finite element models (FEMs) under a set of strong ground motions. Using the output of the analyses, Ω values were calculated for each of 3D FEMs. Finally, the results obtained from the analyses were compared to TEC-16 Ω values and discussed at the end of the analyses.

KEYWORDS: Overstrength Factor, Nonlinear Dynamic Time History Analyses, Ductility, Reinforced Concrete Buildings, Strong Ground Motions, Turkish Earthquake Code 2016-Draft Version

1. INTRODUCTION

During major earthquakes in recent years, many reinforced concrete buildings (RCBs) suffered from local failures and total collapse in Turkey. Therefore, in the last two decades, earthquake codes have undergone radical changes. In 2016, Turkish Earthquake Code-2016 (TEC-16) draft version was offered. One of the most important changes in TEC-16 is earthquake load reduction factor, Rₐ and its mentioned parameter overstrength factor (Ω). Different Ω values are given for every type of building structural systems according to structural system behavior factors, R and allowed total building height classes, (BHC) in TEC-2016 draft version [1].

Moreover, in TEC-16, some formulas are given in order to calculate Ω. Although all of these given values and equations below help to calculate Ω values and in reality, those given values for Ω may not be the same for each building types. Because of different parameters that can also affect ductility, µ Eq. (1) like buildings material and section properties and etc., every individual building must have its own unique Ω value.

\[ \mu_\Delta = \frac{\Delta_{\text{max}}}{\Delta_y} \]  (1)
\[ R = R_\mu \times \Omega = \frac{V_e}{V_d} \]  (2)
\[ R_\mu = \frac{V_e}{V_y}, \; \Omega = \frac{V_y}{V_d} \]  (3)
\[ R_{\mu} \approx \mu_{\Delta} \quad (4) \]
\[ R_a = \Omega + (R/I - \Omega) (T/T_B), T \leq T_B \quad (5) \]
\[ R_a = R/I, \ T > T_B \quad (6) \]

\( R_{\mu} \) is a strength reduction factor, structures with periods bigger than 0.5 sec \( R_{\mu} \) can be considered as equal to structural ductility \( (\mu_{\Delta}) \) in Eq. (4) and earthquake load reduction factor, \( R_a \) is shown in Eq. (5) and Eq. (6).

\( V_e \) is the maximum seismic demand for elastic response, \( V_y \) is the base shear corresponding to the maximum inelastic displacement also known as actual strength, \( V_d \) is the design base shear Eq. (3). The relationships between the force reduction factor \( (R) \), structural overstrength \( (\Omega) \), and the ductility reduction factor \( (R_{\mu}) \) are shown Fig.1. [7].

Past experience and observation of building behavior following earthquakes has shown that a structure can be economically designed for a fraction of the estimated elastic seismic design forces, while maintaining the basic life safety performance objective. This design philosophy implies that structural inelastic behavior (and damage is expected. This reduction in design seismic force is effected through the use of force reduction factor, \( R \). The intent of the \( R \) factor is to simplify the structural design process such that only linearly elastic static analysis is needed for most building design.

While some deformation-controlled members, detailed to provide ductility, are expected to deform inelastically, force-controlled members that are designed to remain elastic would experience a significantly higher seismic force level than that predicted based on actual design seismic forces. To account for this effect, the code uses a seismic force amplification factor, \( \Omega \), such that the realistic seismic force in these force-controlled members can be conveniently calculated from the elastic design seismic forces. To control drift or to check deformation capacity in some deformation-controlled members, a similar approach is also adopted.

![Figure 1. The relationships between the force reduction factor, \( R \), structural overstrength, \( \Omega \), and the ductility reduction factor, \( R_{\mu} \) [7].](image)

The typical response envelope relating force to deformation is shown in Figure 1 and can be established from either testing or a pushover analysis. The structure first responds elastically, which is then followed by an inelastic response as the lateral forces are increased. A series of plastic hinges form throughout the structure, leading to a yielding mechanism at the strength level \( V_y \).
The design method follows a simplified procedure. Based on the fundamental linear elastic period of the structure, a base shear, $V_e$, is then reduced by a factor, $R$, to establish a design seismic force level $V_d$ beyond which elastic analysis is not valid. To estimate internal forces that develop in force-controlled members for capacity design, the corresponding forces at the design seismic force level, $V_d$, are then amplified by a system overstrength factor, $\Omega$.

The 3D pushover analysis and nonlinear dynamic time history analysis (NDTHA) are performed in order to calculate $V_e$ and $V_d$. There are various studies in the literature that regarding to calculating not only $\Omega$, but also $R_a$ and other parameters related to ductility, $\mu$ [6, 7, 8, 11]. These studies are mainly focused on nonlinear static analysis and nonlinear dynamic time history analysis. The time history analysis is the most reliable analysis method among all the nonlinear analysis methodologies.

However, its easy application comparing to time history analysis, nonlinear static analysis is still being carried out by many researchers. In this study, pushover capacity curve (Base Shear-Top Displacement) is plotted by carrying out nonlinear static analysis (NSA) on 3-, 6-, and 9-stories RCBs. In addition to NSA, maximum displacements and base shear capacities are plotted by using NDTHA.

2. ANALYTICAL STUDY

2.1. Details of 3-, 6-, and 9-Story Reinforced Concrete Buildings and Scaled Ground Motions

In this study, three buildings with 3-, 6-, and 9-stories in which seismic loads are fully resisted by reinforced concrete moment frames, are designed based on TEC-16 and Turkish Building Code (TBC-500) [2] requirements and 3D finite element models (FEMs) for each building are modelled by using Sap2000 V18 [3]. The concrete used in the structural system has a compressive strength of 25 MPa and a steel yield strength of 420 MPa. The system is designed with 4 spans in the X-direction and 3 spans in the Y-direction. The span lengths are 5 m and 4 m for X and Y direction respectively. The typical story height is 3 m for each model and the columns are assumed to be fixed to the ground. Floor plan and elevation of 3-, 6-, and 9-stories RCBs are shown in Fig. 2.

![Figure 2. Floor plan and elevation of 3-, 6-, and 9-stories of RCBs](image)
Fundamental periods of vibration for the 3-, 6-, and 9-stories of RCBs are found after a modal analysis and shown in Table 1.

Table 1. Fundamental periods of vibration for the 3- 6-, and 9-stories

<table>
<thead>
<tr>
<th>Story</th>
<th>Period (T, sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-Story</td>
<td>0.24071</td>
</tr>
<tr>
<td>6-Story</td>
<td>0.50226</td>
</tr>
<tr>
<td>9-Story</td>
<td>0.75198</td>
</tr>
</tbody>
</table>

The nonlinear dynamic analysis of three RCBs 3D models was carried out using 11 strong ground motions listed in Table 2 after they were all scaled to the design spectrum corresponding to earthquake ground motion level with probability of exceeding 10% in 50 years with return period of 475 years which is also defined as standard design earthquake ground motion in TEC-2016 [1].

Table 2. Selected strong ground motions for nonlinear dynamic analysis

<table>
<thead>
<tr>
<th>Number</th>
<th>Event</th>
<th>Year</th>
<th>Scale Factor</th>
<th>Station</th>
<th>Mag</th>
<th>Mechanism</th>
<th>Rjb(km)</th>
<th>Rrup(km)</th>
<th>Vs30(m/s)</th>
<th>Lowest useable freq (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Duzce, Turkey</td>
<td>1999</td>
<td>3.2129</td>
<td>Lamont 1060</td>
<td>7.14</td>
<td>strike slip</td>
<td>25.78</td>
<td>25.88</td>
<td>782</td>
<td>0.075</td>
</tr>
<tr>
<td>2</td>
<td>Duzce, Turkey</td>
<td>1999</td>
<td>1.7618</td>
<td>Mudurnu</td>
<td>7.14</td>
<td>strike slip</td>
<td>34.3</td>
<td>34.3</td>
<td>535.24</td>
<td>0.1</td>
</tr>
<tr>
<td>3</td>
<td>Hector, Mine</td>
<td>1999</td>
<td>0.7151</td>
<td>Hector</td>
<td>7.13</td>
<td>strike slip</td>
<td>10.35</td>
<td>11.66</td>
<td>726</td>
<td>0.0375</td>
</tr>
<tr>
<td>4</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>3.8127</td>
<td>Chihaya</td>
<td>6.9</td>
<td>strike slip</td>
<td>49.91</td>
<td>49.91</td>
<td>609</td>
<td>0.1</td>
</tr>
<tr>
<td>5</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>4.489</td>
<td>OKA</td>
<td>6.9</td>
<td>strike slip</td>
<td>86.93</td>
<td>86.94</td>
<td>609</td>
<td>0.0625</td>
</tr>
<tr>
<td>6</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>2.8911</td>
<td>TOT</td>
<td>6.9</td>
<td>strike slip</td>
<td>119.64</td>
<td>119.64</td>
<td>609</td>
<td>0.0625</td>
</tr>
<tr>
<td>7</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>0.7599</td>
<td>Izmit</td>
<td>7.51</td>
<td>strike slip</td>
<td>3.62</td>
<td>7.21</td>
<td>811</td>
<td>0.125</td>
</tr>
<tr>
<td>8</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>0.7693</td>
<td>Gebze</td>
<td>7.51</td>
<td>strike slip</td>
<td>7.57</td>
<td>10.92</td>
<td>792</td>
<td>0.1</td>
</tr>
<tr>
<td>9</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>1.1425</td>
<td>Arcelik</td>
<td>7.51</td>
<td>strike slip</td>
<td>10.56</td>
<td>13.49</td>
<td>523</td>
<td>0.0875</td>
</tr>
<tr>
<td>10</td>
<td>Helena, Montana-01</td>
<td>1935</td>
<td>2.7602</td>
<td>Carroll College</td>
<td>6</td>
<td>strike slip</td>
<td>2.07</td>
<td>2.86</td>
<td>593.35</td>
<td>0.1625</td>
</tr>
<tr>
<td>11</td>
<td>Imperial Valley-06</td>
<td>1979</td>
<td>0.9314</td>
<td>Cerro Prieto</td>
<td>6.53</td>
<td>strike slip</td>
<td>15.19</td>
<td>15.19</td>
<td>471.53</td>
<td>0.1125</td>
</tr>
</tbody>
</table>

All buildings were designed for a local soil class ZB in downtown Bostancı, Istanbul where $S_c$ and $S_1$ are found 1.007g and 0.274g respectively [5]. By using target spectrum, scale factors for every selected strong ground motions are found [4] and plotted over the design spectra (target spectrum) Fig. 3.
2.2. Analyses Results

In all analyses, the geometric and material nonlinearities are considered. First, the 3D pushover analysis is carried out for three different models in X and Y directions. The pushover capacity curves are shown below for 3-, 6-, and 9-stories RCBs respectively in Fig. 4. Secondly, NDTHA is carried out and base shear capacities are calculated for each direction.

The overstrength, which is specified as member or structural capacity, is usually defined using $\Omega$, which can be defined as the ratio of maximum base shear in actual behavior to first significant yield in structure. By using Eq. (3) overstrength factors for each FEM are calculated and shown in Table 3 and the differences between the base shears obtained using NSA and NDTHA are also shown.

Several studies have been carried out in the past pointed out the considerable differences between these approaches yielding different results [8, 10, 14]. One of the main reasons is being due to the fact that NSA cannot take into account the effects of energy content, duration and the frequency content of a strong ground motion. In addition, it’s well known that as the building height increases, the results between two approaches differs significantly because of the importance of higher-mode effect. Furthermore, the cyclic loading effect may be the reason for different top displacement values between NSA and NDTHA methods shown in Fig. 4.
and Fig. 6. [14]. The building with less displacement in the NDTHA method is subjected to more earthquake effects than the NSA method [10].

### Table 3. Overstrength factors

<table>
<thead>
<tr>
<th>Analysis Type</th>
<th>Parameters</th>
<th>3-Story</th>
<th>6-Story</th>
<th>9-Story</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Global Direction</td>
<td>X</td>
<td>Y</td>
<td>X</td>
</tr>
<tr>
<td>Pushover Analysis</td>
<td>Design base shear, Vd (kN)</td>
<td>1279.5</td>
<td>1223</td>
<td>2240</td>
</tr>
<tr>
<td></td>
<td>Base shear capacity, Vy (kN)</td>
<td>3215</td>
<td>3137.5</td>
<td>5272.2</td>
</tr>
<tr>
<td></td>
<td>Overstrength factor, (\Omega = \frac{V_y}{V_d})</td>
<td>2.51</td>
<td>2.56</td>
<td>2.35</td>
</tr>
<tr>
<td>Nonlinear Dynamic</td>
<td>Design base shear, Vd (kN)</td>
<td>1905</td>
<td>1235</td>
<td>1890</td>
</tr>
<tr>
<td>Analysis</td>
<td>Base shear capacity, Vy (kN)</td>
<td>5850</td>
<td>3778</td>
<td>5686</td>
</tr>
<tr>
<td></td>
<td>Overstrength factor, (\Omega = \frac{V_y}{V_d})</td>
<td>3.07</td>
<td>3.05</td>
<td>2.94</td>
</tr>
</tbody>
</table>

According to nonlinear dynamic time history analysis results, maximum base shear capacities and maximum top displacements for all 3D models in the direction of X and Y are given below in Fig. 5 and Fig. 6.

**Figure 5a. Maximum Base Shear in the direction of X and Y for 3-story RCB**

**Figure 5b. Maximum Base Shear in the direction of X and Y for 6-story RCB**

**Figure 5c. Maximum Base Shear in the direction of X and Y for 9-story RCB**
3. CONCLUSIONS

Many reinforced concrete buildings (RCBs) in Turkey suffered from major failure during recent earthquakes and that lead to significant changes in Turkish Earthquake Codes. Many seismic codes permit a reduction in design loads, taking advantage of the fact that the structures possess significant reserve strength (that is $\Omega$) and capacity to dissipate energy (ductility, $\mu$) [13].
In this study, overstrength factors, $\Omega$ and its parameters $V_y$ and $V_d$ for RCBs are calculated by carrying out nonlinear static analysis (Pushover analysis) and nonlinear dynamic time history analysis (NDTHA). The main outcomes of this study and ideas for future studies can be summarized as follows:

1. Overstrength factor ($\Omega$) in TEC-16 draft code, is given 3 for the buildings which can resist seismic loads by reinforced concrete moment frames. The results by carrying out pushover analysis which are presented in Table 3 show that $\Omega$ values for 3- and 9-story RCBs are found to be about 2.5. The overstrength factor decreases when the ductility of the frame decreases and for that reason, 6-story RCB, $\Omega$ value was found to be smaller than 2.5 in both X and Y directions which is lower than the other FEMs.

2. Nonlinear dynamic time history analysis results demonstrate that $\Omega$ values are compatible with the given values in TEC-16.

3. For a future study, according to TEC-16, different types of earthquake ground motion levels (DD-1, DD-2, DD-3, DD-4) and local soil classes (ZA, ZB, ZC, ZD, ZE and ZF) can be used to generate different target spectrums and then carry out nonlinear dynamic analysis by using earthquake records that are scaled according to their individual target spectrums. Moreover, this study can be expanded by analyzing different types of structural systems.

REFERENCES