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The Nonlinear Behavior of Low Rise RC Buildings With Common Structural Deficiencies

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Abstract
The nonlinear behavior of existing low rise reinforced concrete buildings with structural deficiencies was examined in this study. A set of push over analyses has been done for hypothetically defined four representative different types of widely used frames with four, five and six stories. The structural deficiencies such as the low compressive strength of concrete in existing buildings, low percentage of reinforcement in column sections, lack of proper lap splicing in critical regions of columns and beams in existing buildings are all grouped and have been taken into consideration in push over analyses. Also, possible structural changes such as alterations in the orientations of columns, having additional stories on top of the existing building and changing the demand of live loads on the existing structure are idealized for representative frames to obtain load-top deflection curves. All graphs achieved can be utilized in quick assessment work to evaluate earthquake load resisting capacities of the huge amount of existing low rise similar reinforced concrete structures constructed in earthquake prone areas.

Introduction
Huge number of low rise reinforced concrete buildings have been constructed with structural deficiencies, which cause substantial decrement in the expected safety against earthquakes. These buildings have demonstrated more or less similar behavior when they were subjected to Erzincan 1992 or Dinar 1995 or Adana-Ceyhan 1998 and Gölcük 1999 Earthquakes mainly because of the common structural deficiencies and features. There can be found millions of structures built in the same way due to local construction habits so there is no reason to expect better structural performance from these buildings. The assessment and the rehabilitation works carried out after these earthquakes have been utilized for the identification of structural deficiencies and for the definitions of the representative buildings. And then it was attempted to find out what would be the possible behavior first, if these buildings were properly designed according to the valid structural codes in the period of constructions, such as the 1975 Turkish Earthquake Code (TEC, 1975). For that purpose a set of nonlinear 2D analyses
has been carried out to see the possible failure modes, the overstrength factors, the overall displacement ductility of representative frames and the ultimate lateral load resisting capacities. Another two sets of nonlinear 2D analyses have been performed to see the effects of the common structural deficiencies like the lack of the expected compressive strength of concrete and longitudinal reinforcement of lateral load resisting vertical elements, on the general behavior of the frames. In addition to that several individual analyses have been done for the structural deficiencies such as lap splice problems, the lack of required ductility of columns due to extra stories added or changing demand of live loads and/or changing the orientation of columns etc.

It becomes extremely important to estimate the possible structural behavior of low rise RC buildings with certain common deficiencies because of the huge amount of buildings which exist in seismically active regions of the country. Since detailed assessment techniques are expensive and need relatively long time, quick assessment techniques considering structural and material characteristics of local buildings gain relative importance.

In order to provide more parametric base for a new assessment technique proposed by Karadogan et al. (2003), a group of 4, 5, 6 story representative buildings with selected possible structural deficiencies has been modeled and a set of nonlinear analyses carried out to observe the structural behavior till failure and ultimate loads are achieved.

2D Nonlinear Analysis of Representative RC Frames

The nonlinear analyses of frame buildings have been completed using two different computer programs; Namely, EPARCS ‘Elasto-plastic Analyses of Reinforced Concrete Structures’ (Girgin, 1996) and SAP2000 based on plastic hinge hypothesis to have reference push over curves.

Assumptions

i. The vertical dead and live loads considered, are kept constant while the intensity of lateral loads is increased keeping their triangular distribution over the height of the building unchanged,

ii. It is assumed that the expected structural damage on beams will not be as important as column damages,

iii. The lateral load which corresponds to the first plastic hinge on any one of the columns will be referred as $P_e$, elastic limit load,

iv. The lateral load which corresponds to the first column plastic hinge exceeding rotation capacity will be referred as $P_f$, the failure load,

v. $P_{L1}$ indicates the smallest lateral load level which corresponds to anyone of the possible mechanism, such as story mechanism,

vi. 3D structural systems can be idealized by 2D frames.

Typical plan, selected column orientations and the representative 2D frames with four, five and six stories are all given in Figure 1, 2, 3, respectively. It has been assumed that the frames of axis #2 shown in Figure 1, for each possible hypothetical structure have similar nonlinear behavior with the overall 3D structure.
The basic assumptions related to the material characteristics and structural feature of representative frames are as follows,

i. Concrete compressive strength and reinforcement tensile strength are 18 MPa and 420 MPa, respectively,

ii. The representative structures are resting on firm soils,

iii. All requirements of Turkish Earthquake Code 1975 (TEC1975) and the Turkish Standard for Design and Construction of Reinforced Concrete Structures (TS500, 1984) have been satisfied during the design stage,

iv. Strength safety factors of materials are taken as unity,

v. Gross sectional flexural rigidities are employed in nonlinear analyses,

vi. Ultimate strain for concrete and steel are taken \( \varepsilon_{cu} = -0.003 \) and \( \varepsilon_{su} = 0.01 \) respectively.
The common structural deficiencies which were considered in push-over analyses of representative structures are summarised below:

i. The experimental works performed on concrete cylinder cores indicated that the compressive concrete strength, $f_{ck}$ is around 10±2.8 MPa, (Karadogan et al., 2003). Therefore design concrete compressive strength of 18 MPa has been reduced to 8 MPa in order to observe the effects of strength deficiencies in nonlinear structural behavior.

ii. Not only the low concrete compressive strength but also simultaneously insufficient longitudinal reinforcement has been selected as a major parameter in the comparative parametric work. A set of analyses with $f_{ck}=8$ MPa and reinforcement ratio $\rho = 0.008$ has been carried out.

iii. It has been assumed that insufficient lap splicing problem is a systematic problem for all columns. This insufficiency can be considered as the loss of moment capacities of critical sections of columns. Some of the nonlinear analyses have been repeated reducing the existing moment capacities of columns by 50% and 10%

iv. Another lap splice insufficiency has been observed especially at the exterior support sections of beams. In order to reflect this common deficiency into the nonlinear analysis the positive moment capacity of the support sections has been reduced 50% systematically.

v. The original orientation of columns used in design stage has been changed in the construction stage without modifying the design. Column orientation given in Figure.2 a-type has been changed to b, c, d type keeping all other parameters same.

In addition to the five groups of mathematical models used to represent the structural deficiencies listed above, the following three models have been developed for three additional cases:

i. A set of design has been completed considering the only vertical loads satisfying all the associated code requirements. These 48 representative frames have been analyzed under increasing lateral loads and constant vertical loads to obtain the push-over curves and to observe the possible failure modes.

ii. Regardless the original design assumptions of 4, 5 and 6 story frames, an additional story has been placed to represent one of the unusual illegal actions observed in practice. Push-over analyses have been carried out for 5, 6 and 7 story new frames respectively all the additional stories have the same features with the top stories of original frames.

iii. Changing the demand of live loads on the existing structure is a possible structural deficiency frequently observed in existing low rise RC buildings. It is obvious that the possible increment in mass of structure may have changes in dynamic characteristics and on the intensity of earthquake loads acting on the structures. The necessary changes in models have been done, and new push-over analyses have been performed to have better understanding for in structural behavior.

The Parametric Work
All hypothetical frames with and without structural deficiencies have been analyzed considering material nonlinearity only so that the elastic limit, $P_e$, the failure load, $P_G$, the ultimate load $P_{L1}$, and the corresponding top displacements $\delta_e$, $\delta_G$ and $\delta_{L1}$ have been obtained respectively. The nondimensionalized push-over curves achieved at the end of these analyses are present below basically in three groups, Figure 4, 5, 6.

The push-over curves which correspond to the properly designed frames are shown in Figure 4 where one can observe that there is a certain amount of overstrength capacity for these frames which can be approximately as 1.40. Also it can be roughly said that elastic limit, $P_e$, which corresponds to the first plastic hinge on columns is around 1.15 times higher than the expected lateral earthquake force. The average overall displacement ductility observed is around 1.5 which is smaller than the proposed value $R=4$ specified in Turkish Earthquake Code 1997 (TEC, 1997) for nonductile frame structures.

![Concrete Compressive Strength 18MPa](image)

**Concrete Compressive Strength 18MPa**

Reinforcement is satisfying all the requirements of TEC1975 (CASE A)

As soon as concrete compressive strength drops down to the order of 8 MPa below the proposed strength the average failure load parameter and elastic limit becomes 0.66 and 0.55 respectively, Figure 5. It is observed in this case that the ductility level remains around 1.50. If the longitudinal reinforcement ratio drops down 20% and if the concrete compressive strength is around 8 MPa, drastic changes in lateral load resisting capacity is observed depending on the type of beams used in the model. The results of nonlinear analyses indicate that the nondimensional failure load capacity for the cases of I-IV type beams is around 0.22 and for the case of II-III type beams the same ratio is around 0.41. It has been also observed that the elastic limit, $P_e$, for the case of I-IV type beams is exceeded at very early stage of lateral load increments. In other words those structures have practically no earthquake resisting capacity. The same threshold for the case of II-III type beams is around 0.24, Figure 6.

The average values for $P_e$, $P_G$ and $P_{L1}$ which corresponds to the properly designed cases A, to the case of low concrete compressive strength, B, and to the worse cases which have not only the low concrete compressive strength deficiency but also the insufficient
longitudinal reinforcement, C, respectively are all tabulated in Table 1 for the sake of comparisons.

The order of the effect of lap splice deficiency on ultimate load capacity of typical frames with II-type beams has been exemplified on three selected frame and the results are given in Figure 7. For this purpose the necessary push-over analyses have been repeated with lower moment capacities at critical column sections of all stories where lap splice deficiencies have been systematically observed in the field. The bending moment capacities of bottom sections of all columns have been reduced to 50% and 10% of the original design. 50% and 90% decrement in moment capacities of critical
columns draw back the ultimate load capacity to the level of 1.1 and 0.85 respectively. Also it has been observed in the later case that the lateral stiffness of the original structure was lowered down substantially due to the early appearances of plastic hinges on columns.

Table 1. Critical points on push-over curves

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_e$</td>
<td>1.15</td>
<td>0.52</td>
<td>0.24</td>
</tr>
<tr>
<td>$\delta_e/H(x10^{-3})$</td>
<td>1.93</td>
<td>1.01</td>
<td>0.41</td>
</tr>
<tr>
<td>$P_0$</td>
<td>1.41</td>
<td>0.66</td>
<td>0.41</td>
</tr>
<tr>
<td>$\delta_p/H(x10^{-3})$</td>
<td>2.89</td>
<td>1.43</td>
<td>0.81</td>
</tr>
<tr>
<td>$P_{\text{l1}}$</td>
<td>1.49</td>
<td>0.80</td>
<td>0.54</td>
</tr>
<tr>
<td>$\delta_p/H(x10^{-3})$</td>
<td>66.16</td>
<td>28.19</td>
<td>1.59</td>
</tr>
</tbody>
</table>

CASE A: Concrete compressive strength 18MPa and reinforcement is satisfying all requirement of TEC1975
CASE B: Reinforcing is satisfying all requirement of TEC1975, low concrete compressive strength, 8MPa
CASE C: Longitudinal reinforcement ratio is 20% smaller than minimum code requirement for columns and concrete compressive strength is low, 8MPa

Lap splice deficiencies observed around the support section of beams have been roughly represented by reducing positive moment capacity 50% in several frames. The results achieved at the end of nonlinear analyses are represented in Figure 8 by push-over curves which indicate reduction in lateral load capacities and stiffness and increment in ultimate displacements. Nondimensional lateral load capacity in this case is around 0.95.

The upper and lower bunches of curves presented in Figure 9 put together all the pushover curves based on the column orientations given in Figure 2 a-d and b-c respectively. It should be kept in mind that the original design based on the column orientation given in Figure 2a has been referred prior to nonlinear analyses with different column orientations. The average values of ultimate loads observed in Figure 1 are 1.40 and 0.90 for upper and lower bunch of push-over curves.

The push-over curves presented in Figure 10 belong to the frames which were designed originally only according to the vertical loads satisfying all the associated code requirements. Although the lateral earthquake loads were not considered in this design stage, frames have certain amount of lateral load resisting capacities. Roughly, it can be said that nondimensional lateral load carrying capacity is around 1.0, which means that these frames are supposed to resist the earthquake loads defined according to the TEC-1975. It should not be forgotten that the ductility requirements of that code is not satisfied by the existing philosophy. In other words, since the existing ductility ratio observed in Figure 10 is smaller than the expected value 4, real earthquake loads to be imparted to these frames will be higher than the lateral loads that look like to be resisted. It is also obvious that real damage will be higher than the expected level.

The additional stories constructed on the top of the existing frames, regardless of the present design, cause decrement not only in the lateral load resisting capacity of frames but also the overall ductility of the structure, Figure 11.
The effect of increment of occupant load and/or importance factor on the pushover curves is presented in Figure 12. This is considered as a deficiency since some of the buildings constructed originally as residential buildings are modified to become schools or small factory buildings for which the occupancy loads and importance factor are substantially different than the same factors for residential buildings.

Figure 7. Insufficient Lap Splicing at Critical Sections of Columns
Figure 8. Insufficient Lap Splicing at Support Sections of Beams
Figure 9. Alterations in The Orientation of Columns
Figure 10. Samples Designed Considering Only The Vertical Loads
Figure 11. Samples with Additional Story
Figure 12. Samples Considering Increased Live Loads

Idealization of Capacity Curves
Although the column orientations have a certain amount of influence on the overall behavior of representative frames regardless of the deficiencies mentioned in previous paragraphs, it is possible to have probabilistic average capacity curves as presented in Figure 13, (Bayazit and Oğuz, 1998). The similar idealized capacity curves of representative frames with different deficiencies are given together in Figure 14.

Figure 13. Idealized Capacity Curves

Figure 14. Idealized Capacity Curves of Samples with Other Deficiencies

Conclusions
Some of the conclusions achieved at the end of the parametric work presented above are listed in the following paragraphs;

48 RC frames which were properly designed for vertical and lateral loads demonstrate an average overstrength factor 1.4 and overall displacement ductility 1.50 which is suggested as 4 in local engineering practice. It should be kept in mind that the nonlinear analyses carried out are based on gross cross sectional stiffness. If the concrete compressive strength is reduced to the levels frequently observed on the field, the average overstrength factor drops down 50% and ductility ratio remains around 1.50. If both concrete compressive strength and the column reinforcement ratios are reduced, the overstrength factor drops down 65%.

Some of the other common deficiencies such as lap splicing problems for columns and especially for exterior beams, changes in column orientations, column axial load due to added stories are all effective at roughly 30% of the lateral load resisting capacity of the frames.

The average bilinear load–displacement curves can be utilized for quick assessment methods. Before the reported results are generalized, more nonlinear analyses for new types of structures should better be carried out.

References


