EVALUATION OF STRENGTH REDUCTION FACTOR FOR EXISTING MID-RISE RC BUILDINGS

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ABSTRACT :

Many earthquake prone countries in the world have significant amount of existing deficient buildings to be evaluated for seismic actions. Although nonlinear methods are more preferable for assessment of existing buildings, most of the practicing engineers are unfamiliar to these methods. Therefore, linear methods seem to be in use in the near future for assessment of great number of deficient existing buildings in a reasonable time. In linear methods, nonlinear behavior is taken into account by a single parameter: strength reduction factor (R). This study evaluates the use of R factors for different ductility levels, performance levels and soil types.

KEYWORDS: Existing buildings, Ductility, Mid-rise, Performance level, Reinforced concrete, R factors

1. INTRODUCTION

Although nonlinear methods capture the real behavior of structures better than linear ones, their use is somewhat limited due to additional modeling work. Most of practicing civil engineers in many countries are lack of the required knowledge about nonlinear principles. Therefore, elastic methods do not seem to be fully replaced by nonlinear ones in the near future. In elastic methods, nonlinear behavior is taken into account by a single parameter: “strength reduction factor (R)”. Thus this parameter has a critical role for the proper evaluation of buildings. In building codes, some values of the R factors are given for certain classes of structures but they are meant to be for new construction. Such R factors may not be suitable for evaluation of existing buildings with elastic methods. This study evaluates R to be used for existing mid-rise reinforced concrete buildings which are thought to be the major portion of the building stock under risk in developing countries. The buildings reflect existing deficiencies in the building stock. Turkey is selected to represent the developing countries. Eleven 4-story and eleven 7-story buildings are modeled with features common in Turkish building stock. All buildings are modeled with two different transverse steel amounts accounting for seismic detailing to evaluate buildings with different ductility levels. Capacity curves of the building models are obtained by nonlinear static analyses. Displacement capacities at Immediate Occupancy, Life Safety and Collapse Prevention performance levels are determined. The “Equivalent” Single-Degree-Of-Freedom system models obtained from capacity curves are subjected to acceleration records of 83 different earthquakes (approximately 20 records for each of the USGS site classification) to determine seismic demands. Based on the obtained data R values for different performance levels are suggested for existing buildings. This study is useful for better understanding of R factors and proper elastic modeling of existing buildings for seismic assessment.

2. STRENGTH REDUCTION FACTOR

Because of the economical and functional reasons, nearly all buildings are built to behave nonlinearly during the design seismic event. The static lateral force method accounts for nonlinear response of the structures by the use of “strength reduction factors” (R), which is also known as “response modification factor” (ATC 1978; UBC 1997; FEMA 1997; FEMA 2000b; IBC 2000; TEC 2007). The elastic base shear demand ratio obtained from 5% damped acceleration response spectrum (C_e) are divided by R factor to greatly reduce the force demands to obtain design base shear force ratio (C_b) (Eqn. 2.1).

$$C_b = \frac{C_e}{R} \tag{2.1}$$

Strength reduction factor is the most important factor in design or evaluation of the buildings using elastic methods since no other parameter affects the base shear demand as much as R. Therefore proper selection of R factor has a key role in proper seismic assessment.

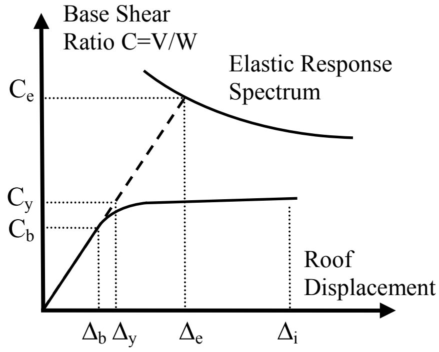

Figure 1 Typical base shear ratio-roof displacement relationship

2.1. Components of R Factor

Since the reasons to use R factor are based on different phenomenon, the R factor has different components. Based on experimental data Whittaker et. al., Uang and Bertero, described R factor as the product of three factors that accounted for over strength, ductility and damping (Uang and Bertero 1986; Whittaker et al. 1987):

$$R = R_o R_\mu R_\xi \tag{2.2}$$

In Eqn. 2.2 R_o is over strength factor, R_μ is ductility factor and R_ξ is damping factor. Using data from earthquake simulation tests, the over strength factor is calculated to be yield base shear ratio (C_y) divided by design base shear ratio (C_b), ductility factor is the base shear ratio required for elastic response (C_e) divided by yield base shear ratio (C_y), and the damping factor was set to unity. In a more recent study a new formulation for R is proposed (ATC 1995a);

$$R = R_o R_\mu R_R \tag{2.3}$$

The formulation given in Eqn. 2.3 is same as the Eqn. 2.2 (since R_ξ is 1.0) except the redundancy factor (R_R). This factor is intended to reflect the effects of redundancy of the structure such as structural indeterminacy and improved reliability due to multiple lines of load carrying mechanisms. Some research can be found in literature about redundancy factor (ATC 1995a; ATC 1995b; Whittaker et al. 1999; Wang and Wen 2000; Husain and Tsopelas 2004; Tsopelas and Husain 2004). As seen above different formulations of R factor has been proposed in literature. This study concentrates on the over strength and ductility factors and assumes R is the product of these factors as given in Eqn. 2.2. Although many studies are available about R factors, no research has been done concerning R factors for different performance levels and soil classifications. This paper focuses on the over strength and ductility factors for the existing buildings with different ductility classes and for different performance levels, and variation of these factors according to different soil classification.

3. BUILDING MODELS

The major portion of the building stock of many developing countries are consists of deficient mid-rise reinforced concrete buildings. In scope of the study existing mid-rise reinforced concrete buildings below code requirements are investigated. Two sets of RC buildings 4-story and 7-story are selected to represent mid-rise buildings located in the high seismicity region of Turkey, eleven buildings in each set. The selected buildings are typical beam-column RC frame buildings with no shear walls. Since the majority of buildings in Turkey were constructed according to 1975 Turkish Earthquake Code, the 4- and 7-story buildings are designed according this code considering both gravity and seismic loads (a design ground acceleration of 0.4 g and soil class Z3 that is similar to class C soil of FEMA-356 is assumed (FEMA 2000a). Material properties are assumed to be 16 MPa for the concrete compressive strength and 220 MPa for the yield strength of both longitudinal and transverse reinforcement. Strain-hardening of longitudinal reinforcement has been taken into account and the ultimate strength of the reinforcement is taken as 330 MPa. One of the important deficiencies in the existing building stock is insufficient amount of transverse reinforcement. The transverse reinforcement amount may be considered to represent construction and workmanship quality or compliance to the code, since closer spacing of transverse reinforcement shows that the structure has ductile detailing and is code compliant and/or has better construction and workmanship quality. Two different spacings are considered as 100 mm and 200 mm to investigate R factors of the buildings with different ductility classes.

3.1. Modeling Approach

Nonlinear static analyses have been performed using SAP2000 Nonlinear Version 8 that is a general-purpose structural analysis program (SAP2000). Three-dimensional model of each structure is created in SAP2000 to carry out nonlinear static analysis. Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. SAP2000 implements the plastic hinge properties described in FEMA-356 (or ATC-40) (FEMA, 2000a; ATC-40, 1996). As shown in Figure 2, five points labeled A, B, C, D, and E define force-deformation behavior of a plastic hinge.

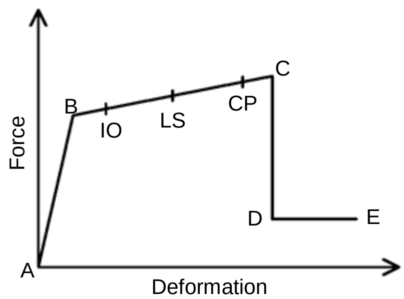

Figure 2 Force-Deformation relationship of a typical plastic hinge

The definition of user-defined hinge properties requires moment–curvature analysis of each element. Modified Kent and Park model (Scott et al., 1982) for unconfined and confined concrete and typical steel stress–strain model with strain hardening (Mander, 1984) for steel are implemented in moment–curvature analyses. The points B and C on Fig. 2 are related to yield and ultimate curvatures. The point B is obtained from SAP2000 using approximate component initial effective stiffness values as per TEC-2006; 0.4EI for beams and values depending on axial load level for columns: 0.4EI for $N/(A_c \text{ x } f_c) \leq 0.1$ and 0.8EI for $N/(A_c \text{ x } f_c) \geq 0.4$. f_c is concrete compressive strength, N is axial load, A_c is area of section. For the $N/(A_c x f_c)$ values between 0.1 and 0.4 linear interpolation is made.

The ultimate curvature is defined as the smallest of the curvatures corresponding to (1) a reduced moment equal to 80% of maximum moment, determined from the moment-curvature analysis, (2) the extreme compression fiber reaching the ultimate concrete compressive strain as determined using the simple relation provided by Priestley et al. (Priestley et al., 1996), given in Eqn. 3.1, and (3) the longitudinal steel reaching a tensile strain of 50% of ultimate strain capacity that corresponds to the monotonic fracture strain. Ultimate concrete compressive strain is given as:

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}} \tag{3.1}$$

where ε_{cu} is the ultimate concrete compressive strain, ε_{su} is the steel strain at maximum tensile stress, ρ_s is the volumetric ratio of confining steel, f_{yh} is the yield strength of transverse reinforcement, and f_{cc} is the peak confined concrete compressive strength. The input required for SAP2000 is moment-rotation relationship instead of moment-curvature. Also, moment rotation data have been reduced to five-point input that brings some inevitable simplifications. Plastic hinge length is used to obtain ultimate rotation values from the ultimate curvatures. Several plastic hinge lengths have been proposed in the literature (Park and Paulay, 1975; Priestley et al, 1996). In this study plastic hinge length definition given in Eqn. 3.2 which is proposed by Priestley et al. is used.

$$L_p = 0.08L + 0.022 f_{yh} d_{bl} \geq 0.044 f_{yh} d_{bl} \tag{3.2}$$

In Eqn. 3.2, L_p is the plastic hinge length, L is the distance from the critical section of the plastic hinge to the point of contraflexure, d_{bl} is the diameter of longitudinal reinforcement.

Following the calculation of the ultimate rotation capacity of an element, acceptance criteria are defined as labeled IO, LS, and CP on Fig. 2. IO, LS, and CP stand for Immediate Occupancy, Life Safety, and Collapse Prevention, respectively. This study defines these three points corresponding to 10%, 60%, and 90% use of plastic hinge deformation capacity. In existing reinforced concrete buildings, especially with low concrete strength and/or insufficient amount of transverse steel, shear failures of members should be taken into consideration. For this purpose, shear hinges are introduced for beams and columns. Because of brittle failure of concrete in shear, no ductility is considered for this type of hinges. Shear hinge properties are defined such that when the shear force in the member reaches its strength, member fails immediately. The shear strength of each member is calculated according to TS 500 that is similar to UBC (TS 500, 2000; UBC, 1997).

Effect of infill walls are modeled through diagonal struts as suggested in TEC-2006 and FEMA-356. Nonlinear behavior of infill walls is reflected by assigned axial load hinges on diagonal struts whose characteristics are determined as given in FEMA-356. Material properties are taken from TEC-2006 to reflect characteristics of infill walls in Turkey; 1000 MPa, 1 MPa and 0.15 MPa were assumed as modulus of elasticity, compressive strength and shear strength values, respectively. Range of some important properties of the building models is given in Table 3.1. Further information about building models and behavior can be found in the study by Inel et al. (Inel et al., 2008).

Table 3.1 Natural period, weight and strength coefficient ranges of 4- and 7-story buildings

N	**Period Range (s)**	**Seismic Weight Range (kN)**	**Yield Base Shear Ratio (Vy/W)**
4	0.47-1.10	8456-10163	0.11-0.25
7	0.74-1.32	2912-20277	0.11-0.18

3.2. Nonlinear Static Analyses

In order to obtain capacity curves and displacement ductility (maximum roof displacement at which performance criteria still satisfied over yield roof displacement) values of the building models, nonlinear static analyses are carried out using SAP2000. The applied lateral forces applied at center of mass were proportional to the product of mass and the first mode shape amplitude at each story level under consideration. P-Delta effects were taken into account. Example capacity curves are provided in Fig. 3 for one of the 4-story models for 100 and 200 mm transverse reinforcement spacing. The vertical axis plots shear strength coefficient that is the base shear normalized by seismic building weight. The horizontal axis plots global displacement drift that is lateral displacement of building at the roof level normalized by building height. The figure indicates significant effect of transverse reinforcement spacing on displacement capacity.

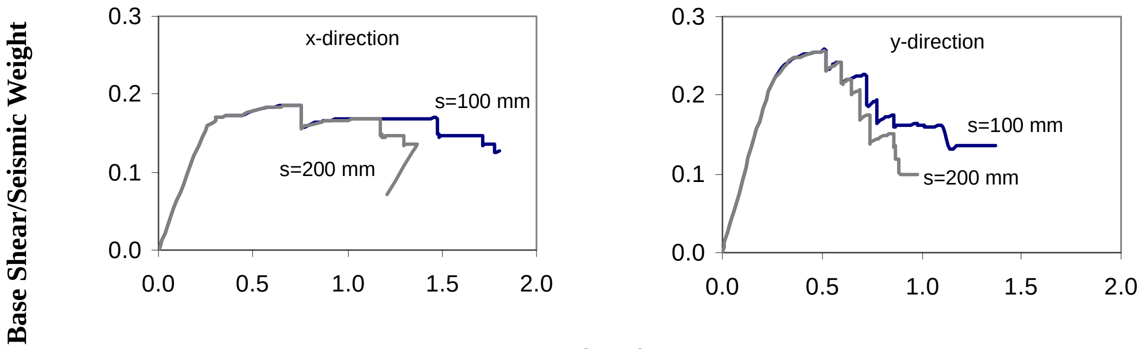

Figure 3 Example capacity curves of a typical 4-story building for 100 and 200mm transverse reinforcement spacing.

3.3. Performance Evaluation

Performance evaluation of the investigated buildings is conducted using recently published Turkish Earthquake Code (2006). Three levels, Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) are considered as specified in this code and several other international guidelines such as FEMA-356, ATC-40. Criteria given in the code for three performance levels are listed in Table 3.2.

Table 3.2 Performance levels and criteria provided in Turkish Earthquake Code (2006)

Performance Level	**Performance Criteria**
Immediate Occupancy (IO)	1. There shall not be any column or shear walls beyond IO level. 2. The ratio of beams in IO-LS region shall not exceed 10% in any story. 3. There shall not be any beams beyond LS.
Life Safety (LS)	1. In any story, the shear carried by columns or shear walls in LS-CP region shall not exceed 20% of story shear. This ratio can be taken as 40% for roof story. 2. In any story, the shear carried by columns or shear walls yielded at both ends shall not exceed 30% of story shear. 3. The ratio of beams in LS-CP region shall not exceed 20% in any story.
Collapse Prevention (CP)	1. In any story, the shear carried by columns or shear walls beyond CP region shall not exceed 20% of story shear. This ratio can be taken as 40% for roof story. 2. In any story, the shear carried by columns or shear walls yielded at both ends shall not exceed 30% of story shear. 3. The ratio of beams beyond CP region shall not exceed 20% in any story.

4. NONLINEAR RESPONSE HISTORY ANALYSES

The capacity curve of each building obtained from pushover analysis was approximated with a bilinear curve using guidelines given in ATC-40 and FEMA-440 and reduced to equivalent SDOF systems (ATC-40, 1996; FEMA, 20005). Then these SDOF systems are subjected to nonlinear response history analysis by using ground motion record sets. USGS site classification based on the average shear wave velocity to a depth of 30 m is used for soil site classification of the selected records.

Table 3.3 Average values for some properties of used ground motion records

Soil Type	Number of records	Magnitude	PGA (g)	PGV (m/s)	PGD (m)
A	20	7.00	0.40	0.30	0.11
B	23	6.71	0.39	0.36	0.11
C	20	7.02	0.40	0.43	0.19
D	20	7.05	0.26	0.36	0.20

Four site classifications include 83 different records, approximately 20 records for each soil type. Soil type A is the stiffest soil type with highest shear wave velocity and D is the weakest soil with the lowest shear wave velocity. All earthquake records are taken from PEER website (http://peer.berkeley.edu/smcat/search.html). Average values for some properties of selected ground motion records are given in Table 3.3.

5. ANALYSES RESULTS

Displacement capacity of the buildings are evaluated for IO, LS and CP performance levels using nonlinear static analyses and criteria given in TEC 2006. Displacement ductilities are calculated dividing displacement capacities by yield displacement. Using response history analyses with the given displacement ductilities, R_μ of the building models are determined. Total of 88 capacity curves (eleven 4- and eleven 7 story buildings, two principal direction and 2 transverse reinforcement spacing) are analyzed for 83 acceleration records. Table 3.4 lists average values for yield base shear strength ratio (C_y), design base shear ratio (C_b), over strength factor (R_o), and ductility ratio (R_μ) for different performance levels, soil type and transverse reinforcement spacing. Note that due to contribution of walls to the lateral strength, C_y values given in table may seem to be high for existing pre-modern code buildings.

Table 3.4 Average C_y, C_b values and R factors for different soil type, spacing and performance level

					R_μ						**Recommended R (R_o R_μ) Values**					
# of story	**C_y**	**C_b**	**R_o**	**Soil Type**	**IO**		**LS**		**CP**		**IO**		**LS**		**CP**	
					s100	**s200**	**s100**	**s200**	**s100**	**s200**	**s100**	**s200**	**s100**	**s200**	**s100**	**s200**
4	0.16	0.1	1.45	**A**	1.18	1.15	2.36	1.62	3.59	2.81	1.60		3.60	2.40	5.40	4.20
				B	1.19	1.15	2.59	1.69	4.02	3.04						
				C	1.19	1.15	2.55	1.69	3.83	2.96						
				D	1.20	1.15	2.42	1.67	3.61	2.82						
7	0.14	0.1	1.57	**A**	1.10	1.03	1.63	1.34	2.51	1.92			2.70	2.10	4.40	3.20
				B	1.11	1.03	1.78	1.39	2.96	2.15						
				C	1.11	1.03	1.80	1.40	3.04	2.16						
				D	1.10	1.03	1.65	1.34	2.66	1.98						

Base shear demands are calculated according to TEC 2006. Note that since the yield strength of the buildings is not significantly affected by amount of transverse reinforcement, C_y is given independent of transverse reinforcement. The change of C_y, R_o and R_μ and R values for CP performance level and 100 mm transverse reinforcement spacing with building period are given in Fig. 4. Similar trend is observed for all the other performance levels and transverse reinforcement spacing.

6. DISCUSSION OF RESULTS

Total of 88 capacity curves of 22 buildings (eleven 4- and eleven 7 story buildings, two principal direction and 2 transverse reinforcement spacing) are used to evaluate R factors for existing buildings. Three different performance levels and 83 ground motion records of four different soil types are considered. Based on 21912 nonlinear response history results the following observations are made.

1- Average yield base shear strength ratio of 4-story buildings are higher than that of 7-story ones. It is observed that C_y values become lower as the period gets higher (Fig.4), as observed by other studies (Akkar et al.,2005; Ozcebe 2004).

2- Although 4-story buildings have higher yield base shear strength ratio, average over strength factor of 7-story buildings are higher than 4-story ones (Table 3.4). This can be explained by lower design base shear demands of 7-story buildings with higher periods. In scope of the building models of this study R_o values increase with increasing period (Fig. 4).

3- All average ductility factors of 7-story buildings are lower than corresponding ductility factors of 4-story buildings (Table 3.4). This shows that as the number of story or period increases the ductility and ductility factors decreases (Fig. 4) as pointed out by other researchers (Akkar et al.,2005; Ozcebe 2004) and indicated by observed wide spread damage of higher buildings compared to lower ones after earthquakes (Dogangun 2004; Sezen et al., 2003; Sucuoglu and Yilmaz, 2000).
4- Even if R_o values are higher for 7-story buildings, R factors are lower than that of 4-story buildings, because higher R_o values are not enough to compensate the insufficiency in ductility factors. R also seems to be decreasing with increasing period. (Table 3.4, Fig. 4).
5- The effect of transverse reinforcement amount on the ductility ratio is significant for LS and CP levels (Table 3.4). The difference in ductility ratio of 100 mm and 200 mm transverse reinforcement spacing is up to 33% depending on performance levels and number of stories.
6- Soil type may affect R values as observed by other researchers (Miranda and Bertero, 1994). Up to 20% difference in the average values of R_μ is observed (Table 3.4).
7- Recommended R values for evaluation of existing buildings are provided in Table 3.4 for general use.

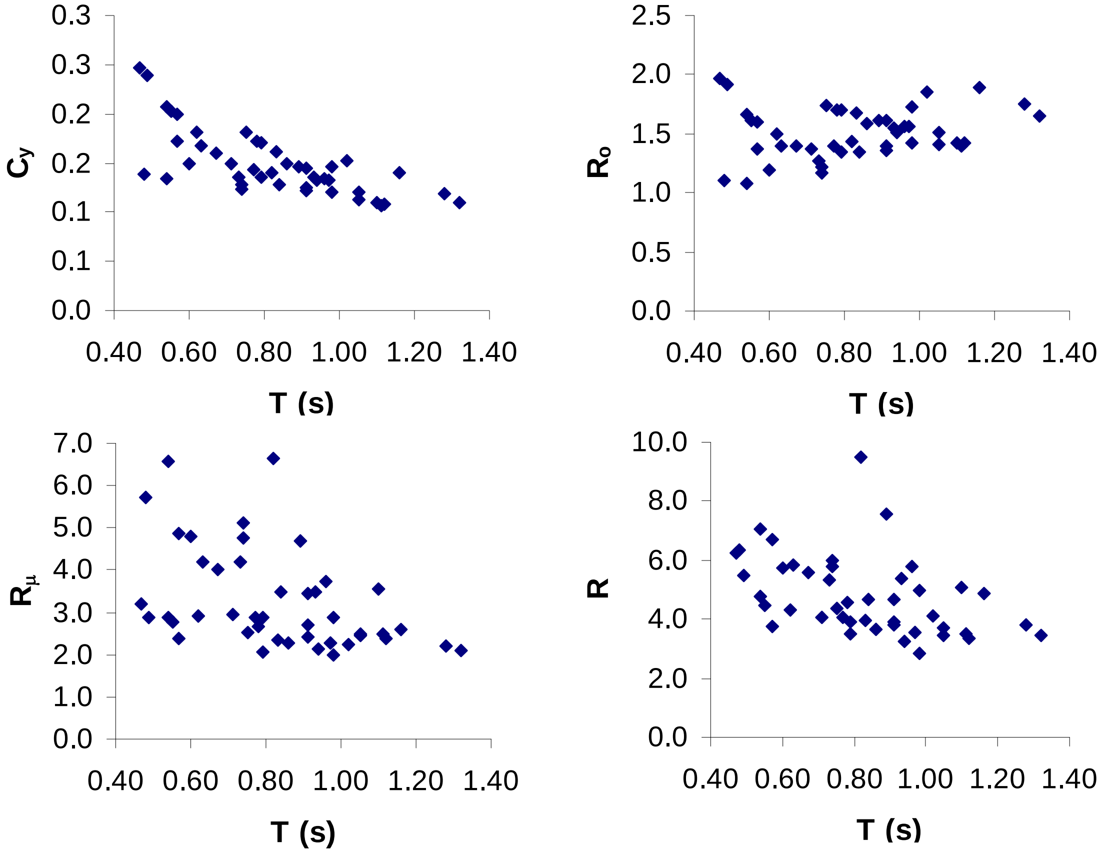

Figure 4 Relationship of C_y, R_o, R_μ and R with period (for Collapse Prevention and s100 mm)

ACKNOWLEDGEMENTS

The authors acknowledge support provided by Scientific and Technical Research Council of Turkey (TUBITAK) under Project No: 107M569 and partial support of Pamukkale University Research Fund Unit (PAU-BAP) to attend the conference.

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