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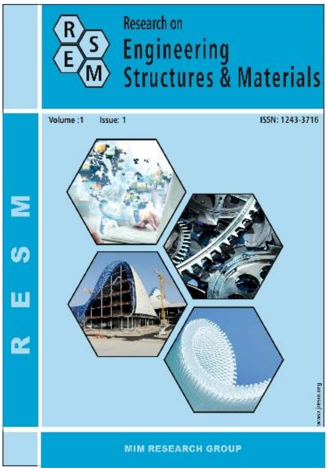

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Research Article

Strength reduction factors for existing mid-rise RC buildings for different performance levels

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Abstract

Many earthquake prone countries 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 behaviour is taken into account by a single parameter: strength reduction factor (R) which is used to greatly reduce the elastic force demand accounting for the nonlinear behaviour. This study evaluates the use of R factors for different ductility and performance levels of buildings with respect to different soil site class. It is observed that the R factors: decrease with increasing periods, are more sensitive for higher performance levels, may change more than 30% with respect to number of story or transverse reinforcement amount, and may change 20% depending on the site class. However, effect of site class is generally smaller and a clear trend is not observed. Exemplary R values for different ductility, performance levels and number of stories are provided in the study.

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1. Introduction

Although nonlinear methods capture the real behaviour of structures better than linear ones, their use is somewhat limited due to additional modelling 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 close future. In elastic methods, nonlinear behaviour is taken into account by a single parameter: "strength reduction factor (R)" [1]. 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 [2-5]. Such R factors may not be suitable for evaluation of deficient existing buildings with elastic methods. This study examines R factors 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. Turkey is selected to represent the developing countries. Eleven 4-story and eleven 7-story buildings are modelled with features common in Turkish building stock. The buildings reflect existing deficiencies in the building stock [6]. All buildings are modelled 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 (IO), Life Safety (LS) and Collapse Prevention (CP) performance

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levels are determined. The "Equivalent" Single-Degree-Of-Freedom (SDOF) 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 [7]. Based on the obtained data, R values for different performance levels are suggested for existing buildings. This study is useful for better understanding R factors and proper elastic modelling 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" [4, 5, 8, 9]. 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) (Fig. 1, Eq. 1).

$$C_b = \frac{C_e}{R} \tag{1}$$

The commentary to the NEHRP Recommended Provisions describes the R factor as "an empirical response modification (reduction) factor intended to account for both damping and ductility inherent in a structural system at displacements great enough to approach the maximum displacement of the system [10]. 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.

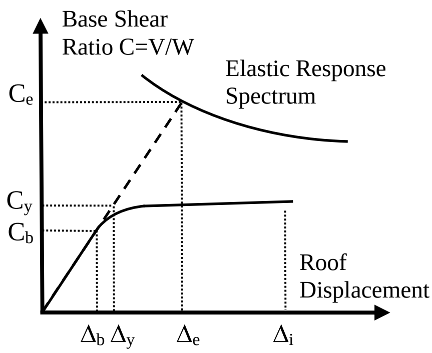


Fig. 1 Typical base shear ratio-roof displacement relationship

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 Uang and Bertero [11], Whittaker et. al. [12] described R factor as the product of three factors that accounted for over strength, ductility and damping:

$$R = R_o R_\mu R_\xi \tag{2}$$

In Eq. 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 (R_o) 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 later document a formulation for R is proposed [9] as:

$$R = R_o R_\mu R_R \tag{3}$$

The formulation given in Eq. 3 is same as the Eq. 2 (when 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.

The proposed formulations do not specifically address the effects of plan and vertical irregularity in framing systems. Whittaker et al. [1] states that irregularity could be addressed by reducing the response modification factor by a regularity factor. EC8 reduces the behaviour factors by 20% for irregular buildings [3].

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 Eq. 3 with R_R is equal to unity.

Although many studies are available about R factors, limited research has been done concerning R factors for different performance levels and soil site classifications. As reported by other researchers, local construction practices significantly affect R values [1]. Therefore, additional studies are important to better establish these factors for regions with varying construction practices. 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 site classification.

2.1.1 Over Strength Factor

The yield lateral strength of a building (V_y) generally exceeds its design lateral strength (V_b) because components are likely to be designed with capacities significantly greater than the design requirements, material strengths generally exceed specified nominal strengths, and displacement and detailing requirements often force the use of stronger components than that necessary for strength alone. For a structural system, over strength factor (R_o) vary as a function of seismic zone and building period [13]. Buildings located in different seismic zones will have different values of over strength due to the different ratio of gravity loads to seismic loads which results in values depending on seismic zones for the strength factor. Differences in local construction practices may also significantly affect the strength factor value [1].

Freeman [14] estimated strength factors of approximately 2.8 and 4.8 for four story and seven-story reinforced concrete moment frames, respectively. Uang and Maarouf [15] analyzed a six-story reinforced concrete perimeter moment frame building shaken by the 1989 Loma Prieta earthquake. The strength factor reported for the building is 1.9. Hwang and Shinozuka [16] studied a four-story reinforced concrete intermediate moment frame building located in seismic zone 2 as per the Uniform Building Code. The building has determined to have a strength factor of 2.2 if no limits to the damage to the structural system is imposed. Whittaker et al. [1] stated that the scatter in the reported values for the strength factor is significant.

For multistory multibay frames, overstrength factor is specified in EC8 as 1.30 [3]. ASCE7 gives an overstrength ratio of 3.0 for RC framed structures [2]. However, this ratio is intended for finding design forces in the members. Therefore, it is given as higher for conservative purposes and does not conform to the logic of overstrength factor in scope of the study.

Mondal et al. [17] investigated the R factors of RC moment frame buildings designed and detailed following the Indian standards for ductile detailing at two performance levels. They reported overstrength factors of 2.70, 2.64, 2.39 and 2.26 for 2, 4, 8 and 12 story buildings based on fundamental mode shape lateral load distribution.

2.1.2 Ductility Factor

The ductility factor (R_μ) is a measure of the global nonlinear response of a structural system due to its plastic deformation capacity. It is measured as the ratio of the base shear considering an elastic response (V_e) / base shear considering an inelastic response (V_y). It is assumed that the elastic force demand on the system (V_e) can be reduced by the factor R_μ owing to the inelastic displacement capacity available with the system. In the last decades, significant studies have been conducted to investigate the ductility factor based on SDOF systems [18-22]. These relationships are based on statistical behaviour of inelastic SDOF systems under strong motion (with 5% damping) on rock or stiff soil.

EC8 gives behaviour factor as 3.0 and 4.5 times the overstrength ratio for medium and high ductility class buildings, respectively [3]. Therefore, these values may be assumed as suggested ductility factors. Mondal et al. [17] in the above mentioned study determined 2.01 and 2.05 ductility factors for 4 and 8 story RC buildings for a limit state based on ATC 40 Structural Stability level [23]. For another limit state based on element plastic rotation capacities, the figures are 2.32 and 2.51, respectively.

Nishanth et al. [13] examined the response reduction factors for moment resisting RC frames designed according to Indian codes. When their reported values for different seismic zones are averaged, the ductility factors for high ductility 4, 7, 10, 13, 16 story buildings are 1.76, 3.03, 3.73, 3.53, 2.95, respectively. Corresponding values for buildings with moderate ductility are 1.73, 2.71, 3.11, 2.75, 2.20. They concluded that the values of R as given by the codes are of higher degree.

2.1.3 Damping Factor

The damping factor (R_ξ) accounts for the effect of supplementary viscous damping and is mainly significant for structures with special energy dissipating devices. Without these devices, the damping factor is generally assigned a value equal to unity and not considered in the determination of the response reduction factor for the force-based evaluation [1, 24]. Since there are no energy dissipating devices in the considered models the R_ξ is taken as 1.0 in scope of the study.

2.1.4 Redundancy Factor

A redundant seismic structural system is composed of multiple vertical lines of lateral load resisting frames, which transfers earthquake-induced forces to the foundation. As there are generally multiple lines of frames in RC structural systems, they usually fell in the category of redundant structural systems. For redundant systems, the lateral load is shared by different frames depending on the relative stiffness and strength characteristics of each frame. Frames aligned in the same direction, form a redundant parallel system. Therefore, the reliability of the system will be higher than the reliability of individual frames of the structural system. If the frames are identical, the reliability will be equal to that of a single frame as they will fail simultaneously. If the frame properties are uncorrelated, the resulting system reliability will be higher. Four lines of strength and stiffness compatible vertical seismic framing in each principal direction of a building have been recommended as the minimum necessary for adequate redundancy [25, 26]. Whittaker et al. [1] gives exemplary values for R_R as 0.71, 0.86 and 1.0 for systems with 2, 3 and 4 lines of vertical seismic framing, respectively. As there are more than 4 lines of framing in each direction of the considered models and due to the suggestion of ASCE7, R_R is taken as 1.0 in this study [2]. More information can be found in literature about redundancy factor [27-31].

3. Building Models

The major portion of the building stock in many developing countries are consists of deficient mid-rise reinforced concrete buildings [6]. In scope of the study, existing mid-rise reinforced concrete buildings inferior to 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 (design ground acceleration of 0.4g). There are eleven buildings in each set. The selected buildings are typical beam-column RC frame buildings with no shear walls (Fig. 2). Since the majority of buildings in Turkey were constructed according to 1975 Turkish Earthquake Code [32], the 4- and 7-story buildings are designed according this code considering both gravity and seismic loads. Soil site class Z3 that is similar to class C soil of FEMA 356 (2000) is assumed [33]. 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 [6]. Strain-hardening of longitudinal reinforcement has been taken into account and the ultimate strength of the reinforcement is taken as 330 MPa.

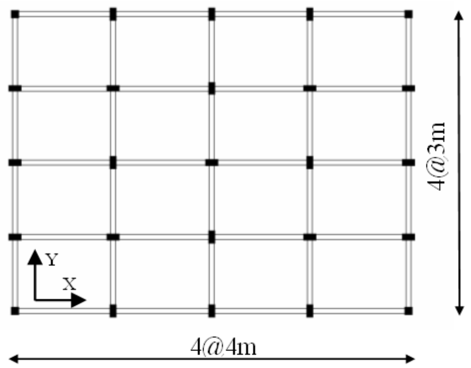


Fig. 2 Plan view of reference 4- and 7- story buildings

One of the important deficiencies in the existing building stock is the 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 [34]. Two different spacings are considered as 100 mm and 200 mm to investigate R factors of the buildings with different ductility classes [6]. More info about building models can be found at the study by Inel et al. [35].

3.1 Modelling Approach

Nonlinear static analyses have been performed using SAP2000 Nonlinear that is a general-purpose structural analysis program [36]. Three-dimensional model of each structure is created in SAP2000 to carry out nonlinear static analysis. Beam and column elements are modelled 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 ATC 40 [23] or FEMA 356 [33]. As shown in Fig. 3, five points labelled A, B, C, D, and E define force-deformation behaviour of a plastic hinge.

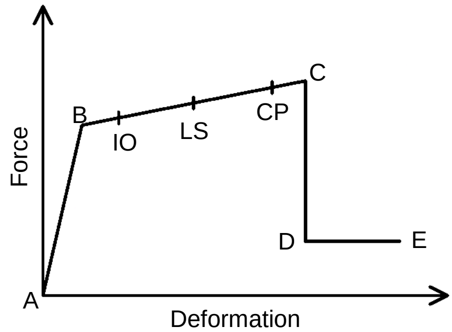


Fig. 3 Force-Deformation relationship of a typical plastic hinge

The definition of user-defined hinge properties requires moment–curvature analysis of each element. Modified Kent and Park model [37] for unconfined and confined concrete and typical steel stress–strain model with strain hardening (Mander [38]) 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 2007 [4]; 0.4EI for beams and values depending on axial load level for columns as given in Eq. 4.

$$\begin{array}{ll} 0.4EI & \mathit{for} \quad \mathrm{N/(A_c \ x \ f_c)} \leq 0.1 \\ (0.4 + 4/3 \cdot (\mathrm{N/(A_c \ x \ f_c)} - 0.1))\mathrm{EI} & \mathit{for}\ 0.1 < \mathrm{N/(A_c \ x \ f_c)} < 0.4 \\ 0.8EI & \mathit{for} \quad \mathrm{N/(A_c \ x \ f_c)} \leq 0.4 \end{array} \tag{4}$$

fc is concrete compressive strength, N is axial load, A_c is area of section. For the $N/(A_c \times f_c)$ 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 relation provided by Priestley et al. [39], given in Eq. 5, 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:

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}} \tag{5}$$

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 [39, 40]. In this study plastic hinge length definition given in Eq. 6 which is proposed by Priestley et al. [39] is used.

$$L_p = 0.08L + 0.022\, f_{yh} d_{bl} \geq 0.044\, f_{yh} d_{bl} \tag{6}$$

In Eq. 6, 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 determination of the ultimate rotation capacity of an element, acceptance criteria are defined as labelled 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 [41]. 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 [42] that is similar to UBC 1997 [5].

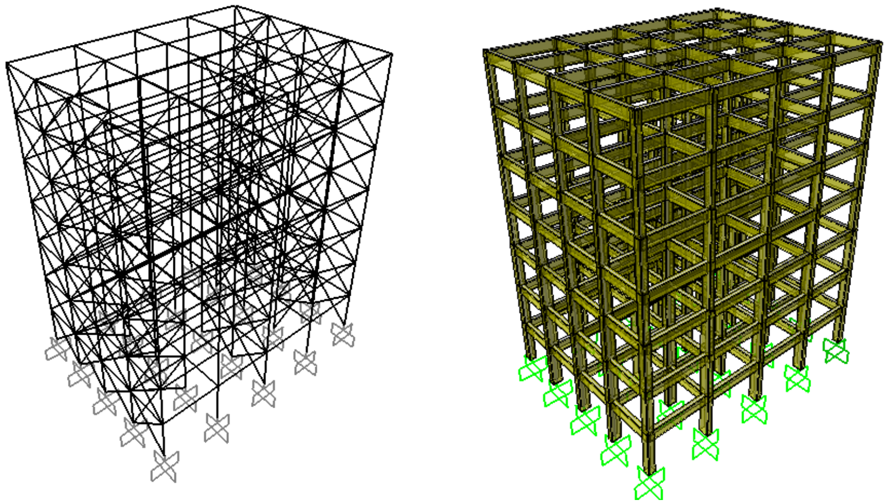
Effect of infill walls are modelled through diagonal struts as suggested in TEC 2007 [4] and FEMA 356 [33]. Nonlinear behaviour of infill walls is reflected by assigned axial load plastic hinges on diagonal struts whose characteristics are determined as given in FEMA 356 [33]. Material properties are taken from TEC 2007 [4] 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.

Reference 7 story model is composed of 200 nodes, 483 RC frame members, 168 infill wall elements and 910 plastic hinges. Reference 4 story model is composed of 125 nodes, 276 RC frame members, 96 infill wall elements and 520 plastic hinges. The given number of nodes are for the points where different members are connected. Note that the software further divides the frames to increase accuracy, if needed.

Range of some important properties of the building models is given in Table 1. Further information about building models and behaviour can be found in the study by Inel et al. [35]. Views of the reference 7 story models are given in Fig. 4. The infill wall elements are not included to in Fig. 4b decrease the complexity of the view.

Table 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 (V_y/W)**
4	0.47-1.10	8456-10163	0.11-0.25
7	0.74-1.32	2912-20277	0.11-0.18



a) Wire frame view of the model b) View of the model with 3D members (infill walls not visible)

Fig. 4 Views of the 7 story model

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. The 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. 5 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.

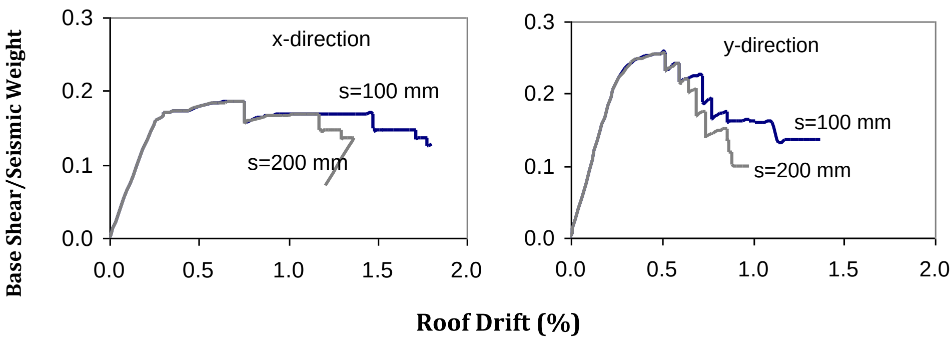


Fig. 5 Example capacity curves of reference 4-story building for 100 and 200mm transverse reinforcement spacing.

3.3 Performance Evaluation

Performance evaluation of the investigated buildings is conducted using Turkish Earthquake Code [4]. Three levels, Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) are considered as specified in this code and several other international guidelines [23, 33]. Criteria given in the code for three performance levels are listed in Table 2.

Table 2 Performance levels and criteria provided in Turkish Earthquake Code [4]

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 [23] and FEMA 440 [43] and reduced to equivalent SDOF systems. Then these SDOF systems are subjected to nonlinear response history analysis by using ground motion record sets. Earthquake records with different characteristics may significantly affect the results of analyses [44, 45]. In this study USGS soil site classification based on the average shear wave velocity to a depth of 30 m is used [7]. Site class A is the stiffest type with a shear wave velocity higher than 750 m/s, and D is the weakest site with a shear wave velocity lower than 180 m/s. The site B has a shear wave velocity between 750 m/s and 360 m/s whereas site C has a shear wave velocity between 360 m/s and 180 m/s. Four site classifications include 83 different records, approximately 20 records for each site class. All earthquake records are taken from PEER website [46]. Average values for some properties of selected ground motion records are given in Table 3.

Table 3 Average values for some properties of used ground motion records

Site class	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

5. Analyses Results

Displacement capacities of the buildings are evaluated for IO, LS and CP performance levels using nonlinear static analyses and criteria given in TEC 2007. 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 directions and 2 transverse reinforcement spacing) are analyzed for 83 acceleration records. Table 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, site class 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 4 Average C_y, C_b values and R factors for different site class, spacing, performance level

Story	C_y	C_b	R_o	Site Class	R_μ IO s1	R_μ IO s2	R_μ LS s1	R_μ LS s2	R_μ CP s1	R_μ CP s2	Exemplary R (R_o.R_μ) Values IO s1	IO s2	LS s1	LS s2	CP s1	CP s2
4	0.16	0.11	1.45	A	1.18	1.15	2.36	1.62	3.59	2.81	1.7		3.6	2.4	5.5	4.2
				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	0.09	1.57	A	1.10	1.03	1.63	1.34	2.51	1.92	1.7	1.6	2.7	2.1	4.4	3.2
				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						

Note: s1= s100 mm, s2= s200 mm

Base shear demands are calculated according to TEC 2007 [4]. 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. 6. Similar trends are observed for all the other performance levels and transverse reinforcement spacing.

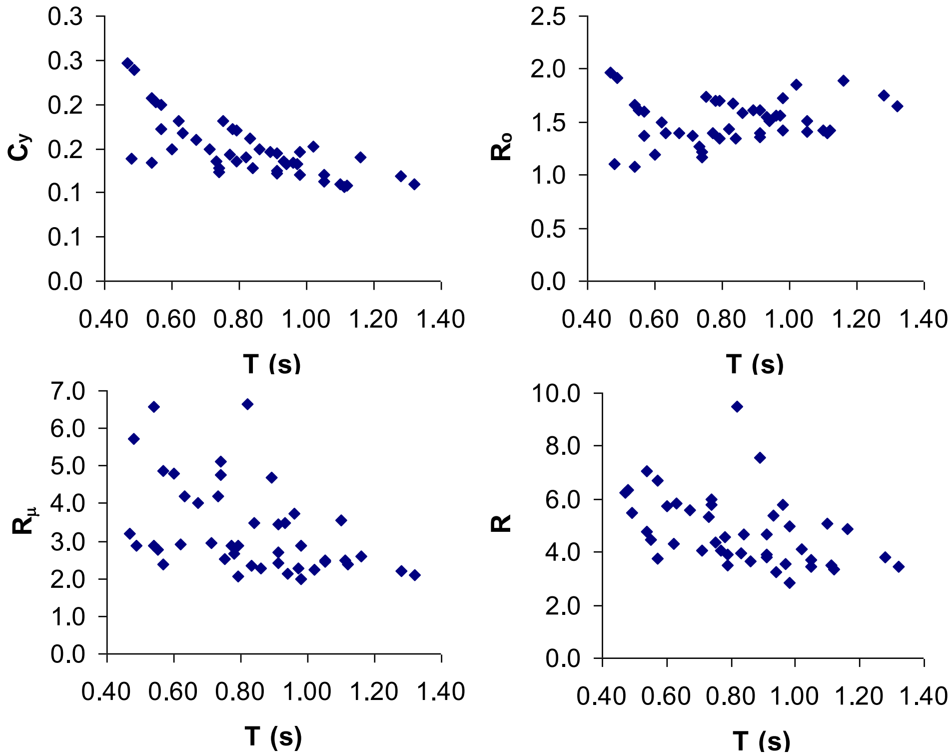


Fig. 6 Relationship of C_y, R_o, R_μ and R with period (for Collapse Prevention and s100 mm)

5. Conclusions

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

- 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. 6), as observed by other studies [47-49].
- 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 4). This can be explained by lower design base shear demands of 7-story buildings with higher periods located at descending branch of code spectrums. In scope of the building models of this study R_o values seem to scatter around 1.5 (Fig. 6). It is notable that 1.5 is the value implicitly used for overstrength in TEC 2007 Section 2.5 [4].
- All average ductility factors of 7-story buildings are lower than corresponding ductility factors of 4-story buildings (Table 4). This shows that as the number of story or period increases the ductility and ductility factors decreases (Fig. 6) as pointed out by other researchers [47, 48] and indicated by observed wide spread damage of higher buildings compared to lower ones after earthquakes [50-53].

- 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 4, Fig. 6).
- IO level is just at the beginning of inelastic behaviour. Since R_μ is a factor regarding inelasticity, it does not change by the amount of transverse reinforcement for IO level.
- The effect of transverse reinforcement amount on the ductility ratio is significant for LS and CP levels (Table 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. This is in compliance with the findings of an experimental study by Rizwan et al. [54], which reports reductions of R values by 40 to 60% due to construction defects.
- Even if it is not accounted in the codes, or in many other studies in literature, site class may affect R values as observed by other researchers [22]. The dependence is more evident for higher performance levels (CP). Up to 20% difference in the average values of $R\mu$ is observed (Table 4). However, in general the differences are smaller and a clear trend is not observed.
- Examples of R values for evaluation of existing buildings are provided in Table 4 for general use.

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