The Effects of Material Properties on Building Performance

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ABSTRACT

In recent earthquakes during the last two decades, severe damages have been occurred on the existing buildings in Turkey. Destructive earthquakes revealed that the existing building stock in urban regions is significantly vulnerable to seismic hazard. A large number of residential buildings located in regions of high seismicity require performance evaluation before the next big earthquake hits the region. In many earthquake resistant codes, several procedures are proposed to determine the building performance. The investigations on the damaged buildings show that material strengths are very important parameters on the building performance. In this study, material strengths' effects on the building performance were investigated by using a nonlinear elastic analysis method.

INTRODUCTION

Turkey is an earthquake country located in a highly active seismic zone. Reinforced concrete (RC) frame structural system is frequently used in regions of high seismic risk. Recent earthquakes in Turkey, including the Kocaeli 1999, Duzce 1999 and Van 2011 Earthquakes, revealed that the reinforced concrete buildings did not perform well and a number of RC buildings collapsed causing life and economic losses. The major seismic deficiencies in these buildings were lack of ductility, poor material quality and workmanship, inadequate design and structural systems.

In the literature many studies are given related to the earthquakes in Turkey, examining the observed structural damages and damage reasons, the performance of structures and structural deficiencies etc. [1-5].

During the last few years, the nonlinear analysis procedures have been developing for estimating the seismic demands of buildings. When evaluating the seismic demands of buildings, engineers are more likely to adopt pushover analysis instead of the more complicated non-linear time history analysis [6]. Pushover analysis is a simplified nonlinear static procedure in which earthquake loads are applied incrementally to the buildings up to a plastic collapse mechanism is developed. The use of the pushover analysis is recommended for low-rise buildings where the effects of higher modes are not significant. The pushover analysis is restricted considering a single-mode response. In order to take into account the effects of the higher modes, some researchers have focused on developing of modal pushover analysis [7, 8].

In this study, the effect of inadequate material strength on the building performance was investigated according to Turkish Earthquake Code (TEC) 2007 that has similarities with FEMA 356 [9] guidelines. Structural Analysis Program (SAP 2000) [10] was used to obtain the seismic performance of the residential buildings. In the analyses, residential buildings with different numbers of stories were considered. The results of analyses were given in figures comparatively.

DAMAGE LIMITS AND DAMAGE STATES IN STRUCTURAL MEMBERS

First step of the analysis of existing buildings is to collect information on the structure. In TEC 2007 [11], the information collected on existing buildings is classified with respect to the scope of data and the type of building system. These levels are *"limited"*, *"moderate"* and *"comprehensive"*. Knowledge factors are applied to the calculated member capacities, which are 0.75 for the limited, 0.90 for the moderate, and 1.0 for the comprehensive knowledge levels, respectively.

Generally, structural members can be classified as "ductile" or "brittle" with respect to their mode of failure in determining the damage limits. In TEC 2007 [11], three damage limits are defined at the cross section level for ductile members. These are minimum damage limit (MN), safety limit (SF) and collapse limit (CL) as shown in Figure 1. The corresponding damage states are also given in the same figure. MN defines the onset of significant post-elastic behavior at a critical cross section. Brittle members are not permitted to exceed this limit. A member damage state is determined by its critical cross section with the most severe damage state.

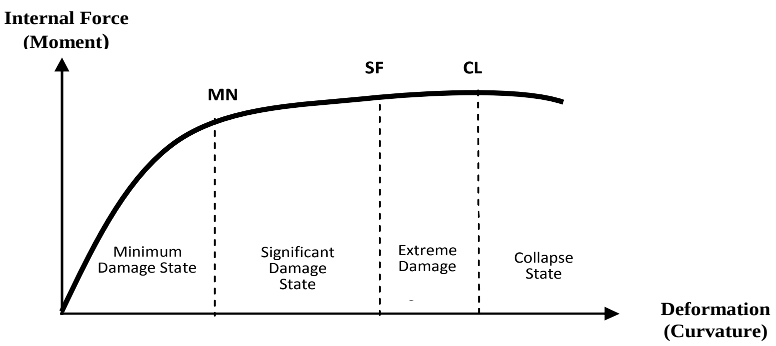


Figure 1 Damage limits and damage states in a ductile member

nonlınear procedure

Nonlinear structural analysis can be classified in two paths: one is nonlinear time history analysis, and the other one is nonlinear pushover analysis. Nonlinear time history analysis is accepted as the most accurate and the most reliable one [12-14]. But due to its difficulty in the applications, pushover analysis becomes more popular for engineers.

In TEC 2007, the incremental equivalent static lateral force analysis and incremental modal response spectrum analysis or multi-mode pushover analysis can be employed for performance assessment of existing buildings. Incremental equivalent static lateral force analysis is limited to 8-story buildings with total height not exceeding 25 m, and not possessing torsional irregularity.

Nonlinear flexural behavior in frame members are confined to plastic hinges, where the plastic hinge length L_p is assumed to be half of the section depth ($L_p = h/2$). Pre-yield linear behavior of concrete sections is represented by cracked sections, which is $0.40EI_o$ for beams and varies between $(0.40\text{-}0.80)EI_o$ for columns where EI_o is the gross sectional flexural rigidity. Strain hardening in the plastic range may be ignored, provided that the plastic deformation vector remains normal to the yield surface.

Incremental equivalent static lateral force analysis

In nonlinear static analysis, lateral forces are increased until the earthquake displacement demand is reached. Internal member forces and plastic deformations are calculated at the demand level. A capacity diagram is obtained from the incremental analysis which is expressed in the “base shear force - roof displacement” plane. Then the coordinates of this plane is transformed into “modal response acceleration versus modal response displacement” as shown in Figure 2 below.

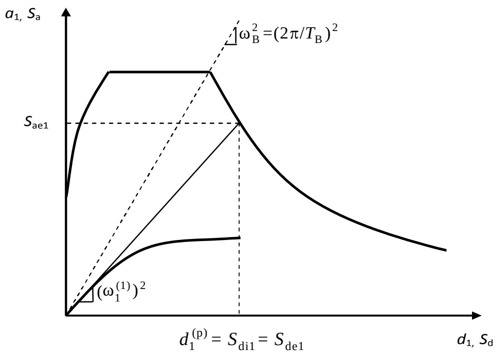


Figure 2 Capacity diagram and the displacement demand in the modal acceleration-displacement plane

The modal displacement demand d_1 is equal to the inelastic displacement demand S_{di1}, which is in turn equal to the modal linear elastic displacement demand S_{de1} when $(\omega_1^{(1)})^2 \leq \omega_B^2$ as shown in Figure 2. When $(\omega_1^{(1)})^2 > \omega_B^2$

$$d_1^{(p)} = S_{di1} = S_{de1} \tag{1}$$

where

$$C_{R1} = \frac{1 + (R_{y1} - 1)\, T_B / T_1^{(1)}}{R_{y1}} \geq 1 \qquad (2)$$

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \qquad (3)$$

Building displacements, internal deformations and forces can be calculated at the modal displacement demand d_1 by appropriate transformations by using the first mode properties. The plastic rotations obtained at the member's plastic hinge locations are then used for calculating the plastic curvature demands at these critical sections.

$$\phi_p = \frac{\theta_p}{L_p} \qquad (4)$$

$$L_p = 0.5\ h \qquad (5)$$

$$\phi_t = \phi_y + \phi_p \qquad (6)$$

Performance assessment of reinforced concrete members

Concrete compressive strain and steel tensile strain demands at the plastic regions are calculated with the help of the moment-curvature diagrams at the plastic curvature level in Eq. (4). Moment-curvature diagrams of the critical sections are obtained by applying appropriate stress-strain rules for concrete and steel. Finally, the calculated strain demands are compared with the damage limits given below to determine the member damage states in view of Figure 1.

- Concrete and steel strain limits at the fibers of a cross section for minimum damage limit (MN)

$$(\varepsilon_{cu})_{MN} = 0.0035 \qquad (\varepsilon_s)_{MN} = 0.0010$$

- Concrete and steel strain limits at the fibers of a cross section for safety limit (SF)

$$(\varepsilon_{cg})_{SF} = 0.0035 + 0.01(\rho_s / \rho_{sm}) \leq 0.0135 \qquad (\varepsilon_s)_{SF} = 0.0040$$

- Concrete and steel strain limits at the fibers of a cross section for collapse limit (CL)

$$(\varepsilon_{cg})_{CL} = 0.004 + 0.014(\rho_s / \rho_{sm}) \leq 0.018 \qquad (\varepsilon_s)_{CL} = 0.0060$$

In the expressions, ε_{cu} is the concrete strain at the outer fiber, ε_{cg} is the concrete strain at the outer fiber of the confined core, ε_s is the steel strain and (ρ_s/ρ_{sm}) is the ratio of existing confinement reinforcement at the section to the confinement required by the Code.

BUILDING EARTHQUAKE PERFORMANCE

In TEC 2007, four performance levels were defined to determine building performance of RC buildings. Building earthquake performance level is determined after establishing the member damage states, as explained above. The rules for determining building performance are given below for each performance level [11, 15]:

Immediate Occupancy (IO): In any story, in the direction of the applied earthquake loads, not more than 10% of beams are in significant damage state whereas all other structural members are in the minimum damage state.

Life Safety (LS): In any story, in the direction of the applied earthquake loads, not more than 30% of beams are in extreme damage state. Shear carried by those columns in the extreme damage state should be less than 20% of the story shear at each story. All other structural members are in minimum or significant damage states.

Collapse Prevention (CP): In any story, in the direction of the applied earthquake loads, not more than 20% of beams are in collapse state whereas all other structural members are in minimum, significant or extreme damage states. Shear carried by those columns in the collapse state should be less than 30% of the story shear at each story. Furthermore, such columns should not lead to a stability loss. Occupancy of the building should not be permitted.

Collapse (C): If the building fails to satisfy any of the above performance levels, it is accepted as in the collapse state. Occupancy of the building should not be permitted.

TARGET PERFORMANCE LEVELS FOR BUILDINGS

The reference design spectrum in the TEC 2007 has 10% probability of exceeding in 50 years. Based on TEC 2007, it is estimated that the spectral ordinates for 50% probability of exceeding in 50 years are half of the reference spectrum whereas the ordinates for 2% probability of exceeding in 50 years are 1.5 times that of the reference spectrum. Accordingly, the target performance levels of retrofitted buildings are summarized in Table 1. Since residential buildings are examined in this study, Life safety performance level is selected as target performance level.

Table 1: Target performance levels for buildings under different earthquake intensities

Type of Building Use	**Probability of Exceeding**		
	50% in 50 years	10% in 50 years	2% in 50 years
Emergency facilities (hospitals, etc.)	-	IO	LS
Buildings with long duration occupancy (schools, etc.)	-	IO	LS
Theatres, concert halls, sports arenas, congress centers	IO	LS	-
Buildings containing toxic materials	-	IO	CP
Others (residential, commercial, etc.)	-	LS	-

NUMERICAL ANALYSES

Numerical analyses were performed to investigate the effects of some structural parameters on the seismic performance of buildings. For this reason, 32 buildings having different characteristics were selected for analysis. Three-dimensional models of each of the 32 buildings created in SAP 2000 [10] were subjected to pushover analysis to determine the performance level. The analyses were carried out for these buildings by varying the number of storeys, column sizes, steel yield strength and concrete compressive strength. Gravity and seismic loads were considered, assuming a design ground acceleration of 0.4g and soil class Z4. RC building models with 2, 3, 4 and 5 storeys were created to represent the low- and mid-rise buildings located in the high seismic regions of Turkey. The span number of both x and y directions in the models was selected as 4. The axis in each model is 4 meters. Typical floor height is 3.0 m. The plan and 3D views of selected buildings are given in Figure 3 and Figure 4, respectively.

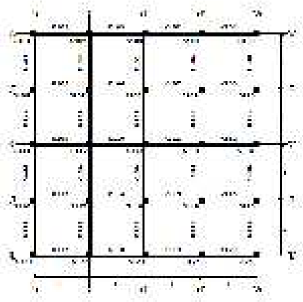


Figure 3. Layout of RC framed building

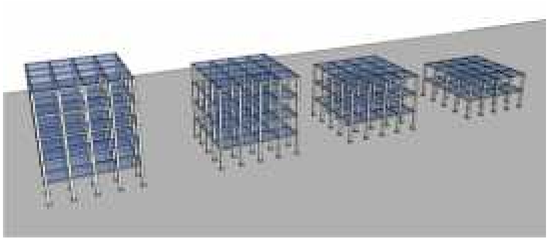


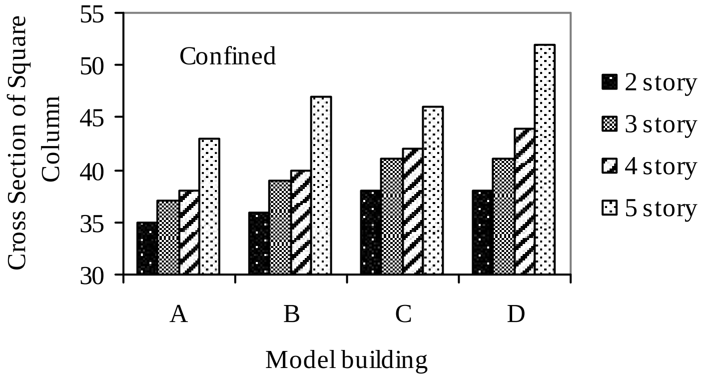
Figure 4. 3D view of RC framed building

The properties of model buildings are given in Table 2. The compressive strength of concrete (f_c) was considered as 20 MPa and 10 MPa. The yield strength of the steel (f_y) was selected as 420 MPa and 220 MPa for longitudinal and transverse reinforcements. Transverse reinforcement spacing values for confined and unconfined column and beam cross sections are considered as 100 mm and 200 mm, respectively.

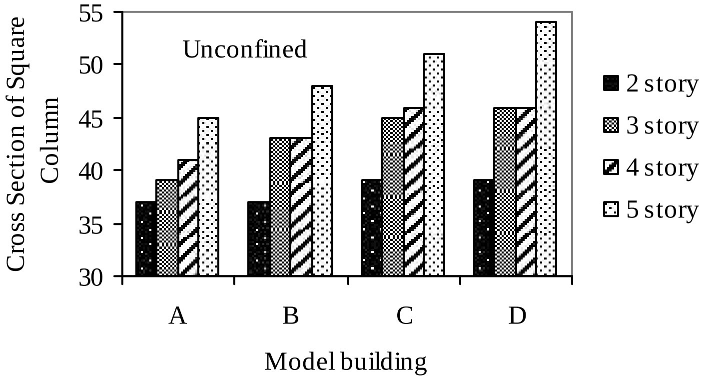
Table 2: The properties of model buildings

Model building	f_c (MPa)	f_y (MPa)	Confined (C)/ Unconfined (UC)	Number of storey
A	20	S420	C and UC	2,3,4 and 5
B	10	S420	C and UC	2,3,4 and 5
C	20	S220	C and UC	2,3,4 and 5
D	10	S220	C and UC	2,3,4 and 5

All beams in the buildings have the same cross-section of 250 mm / 600 mm. In the first model, the cross section of columns is selected as 300 mm × 300 mm. Performance analysis of the model building was carried out by using SAP 2000 program. If the performance level of the building does not satisfy Life Safety level, the cross section of the column is increased step by step, equally in both directions (i.e. 10 mm). This process is repeated until the performance level of the building satisfies the life safety. The required dimensions of column cross sections obtained for all model buildings are given in Figure 5.



(a)



(b)

Figure 5. The dimensions of the column cross sections obtained for all model buildings

The analysis results show that the required unconfined column area by the TEC 2007 is greater than the confined column area and the performance of the building varies depending on the steel and the concrete strengths, whereas the steel strength is more effective parameter than the concrete compressive strength. The effect of the concrete strength on the building performance is very limited in the nonlinear numerical analysis. The effect of the material increases, as the number of storeys increases. In the case of the unconfined column, the required cross sectional area of the columns increases. It is worth to mention that, the destructive earthquakes have shown that the low concrete strength has important role on the poor performance due to lack of ductile behavior of structural elements, such as loss of bond, brittle shear failures and brittle compressive failures. This fact cannot be seen here, due to the assumption made in the numerical analysis, such as perfect bond.

Conclusion

Seismic analysis of the model buildings is performed according to TEC 2007, where the seismic performance evaluation is similar to that of FEMA 356. The results of the analysis show that the steel yield strength is one of the most important parameters affecting the performance level of the buildings. The analysis yield that the effect of the concrete strength on the building performance level less pronounced. However, this is not exactly true. Because, destructive earthquakes revealed that the concrete strength is very important parameter on the building performance, it leads to brittle behavior of the structural systems.

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