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# Effect of rapid screening parameters on seismic performance of RC buildings

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**Abstract.** This study investigates the effects of soft story, short columns, heavy overhangs, pounding, and construction and workmanship quality parameters on seismic response of reinforced concrete buildings through nonlinear static and dynamic procedures. The accounted parameters are selected for their common use in rapid screening of RC buildings. The 4- and 7-story buildings designed according to pre-modern codes are used to reflect majority of the existing building stock. The relative penalty scores are employed in this study to evaluate relative importance of certain irregularities in the existing rapid seismic assessment procedures. Comparison of relative scores for the irregularities considered in this study show that the overall trend is similar. The relatively small differences may be accounted for regional construction practices. It is concluded that initial-phase seismic assessment procedures based on architectural features yield in somewhat similar results independent of their bases. However, the differences in the scores emphasize the proper selection of the method based on the regional structure characteristics.

**Keywords:** existing buildings; irregularity; mid-rise; seismic assessment; pounding; reinforced concrete

## 1. Introduction

In the last decades, industrialization and rapid population growth has brought an urgent need for new accommodation supply in developing countries (Building Census 2000, 2001). Man y buildings were constructed as quickly as possible regardless of any care for quality and safety considerations. Most of such buildings do not conform requirements of modern code detailing (Otani 1997, Tama 2012, Inel *et al*. 2013, Ozmen *et al*. 2013, Bilgin 2015, Çırak 2015, Inel and Meral 2016). Economic losses and casualties during recent devastating earthquakes in Turkey revealed deficiencies of existing buildings regarding their seismic performances like in many earthquake prone countries. Hence, seismic vulnerability assessment of existing buildings is essential to take preventive measures and reduce potential damage to structures and loss of human lives during possible future earthquakes (Gunes 2015, Lim *et al*. 2016).

Due to huge number of vulnerable buildings in Turkey or in earthquake prone countries with similar construction practice, identification of defective buildings with inspection and detailed analysis is a difficult, expensive, and time consuming task. This situation directs the civil engineering profession to find reliable, economical, and yet simple methods to identify the safety level of structures. Several procedures have been proposed to predict seismic vulnerability of existing buildings (FEMA-154 1988, FEMA-154 2002, Japan Seismic Index (Ohkubo 1991), FEMA-310 1998, Ozcebe 2004, Yakut 2004)

The seismic vulnerability assessment procedures can generally be classified into three stages: walk-down, preliminary, and final evaluation stages (Ozcebe 2004). The walk-down and preliminary evaluation stages are most widely used procedures when quick assessment of huge number of buildings is point of interest. In these stages, typical parameters considered for vulnerability assessment include site classification, number of stories, existence of soft story, short columns, heavy overhangs, potential pounding possibility, and lateral load resistance of the building.

Many of the rapid seismic evaluation procedures use some scoring to identify the safety levels of the buildings (FEMA-154 1988, Ozcebe 2004, Yakut 2004). Certain variables are added to or subtracted from an initial value or this initial value is multiplied by some other variables reflecting the properties of the building under consideration. Then according to final result some judgment on the safety level of buildings are made. Assigning values for these variables is a crucial step on establishment of the evaluation method. Inevitably some properties of the building, usually irregularities, have to be reduced to a single value. For irregularities this value should reflect the effect of irregularity on the seismic performance of the structure.

This study aims to examine the effects of soft story, short columns, heavy overhangs, pounding, and transverse steel amount parameters on seismic response of reinforced concrete buildings through nonlinear static and dynamic procedures. Since modelling of pounding with nonlinear static procedures is not possible, nonlinear dynamic time history analysis is used to evaluate pounding effect on

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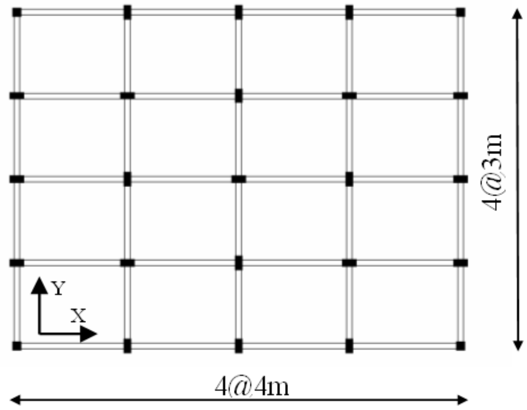



Fig. 1 Plan view of reference 4- and 7- story buildings

seismic response. Transverse reinforcement amount is considered to represent construction and workmanship quality or compliance to the code. Since closer spacing of transverse reinforcement shows that the structure is code compliant and/or has better construction and workmanship quality. Main goal of this paper is to evaluate whether the effects of aforementioned parameters are reflected in scoring of existing methods, consistently. The scope of current study is limited to low- and mid-rise reinforced concrete buildings because the high-rise buildings require specific and detailed evaluation rather than the use of conventional seismic vulnerability assessment procedures. Four procedures that are based on scoring have been considered to carry out the study (FEMA-154 1988, FEMA-154 2002, Ozcebe 2004 and Yakut 2004).

## 2. Methodology

Certain irregularities considered in seismic assessment procedures (e.g., FEMA-154 1988 and 2002, Ozcebe 2004, Yakut 2004) are selected to analytically evaluate their relative importance on the seismic response of reinforced concrete buildings. The relative penalty scores are used to compare the methods with different attributes. It is worth to note that all parameters considered in seismic assessment procedures are not suitable for the analytical study. Hence, soft story, short columns, heavy overhangs, pounding, and construction and workmanship quality parameters are selected for study.

Two RC buildings with 4-story and 7-story are selected to represent reference mid-rise buildings located in the high seismicity region of Turkey. The selected buildings are typical beam-column RC frame buildings with no shear walls. Both buildings have the same plan view as shown in Fig. 1. The beam dimensions are 200x500 mm and 250x600 mm in the 4-story and 7-story buildings, respectively. The first story columns of 4-story buildings are 300x300 mm for corners and 500x250 mm for the others. The longitudinal reinforcement ratio is between 1.0% and 1.5%. The first story columns of 7-story buildings are 400x400 mm for corners and 600x300 mm for the others. The longitudinal reinforcement ratio is between 1.1% and 1.5%.

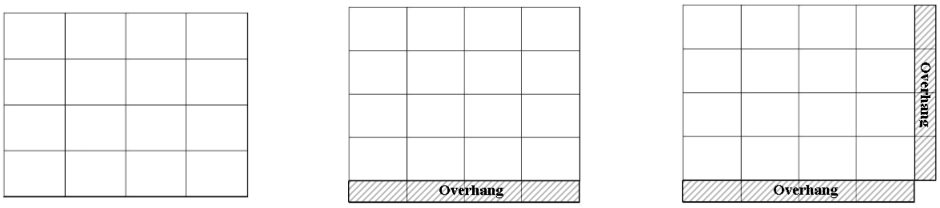
Since the majority of existing deficient buildings were constructed according to 1975 Turkish Earthquake Code, the 4- and 7-story buildings are designed according this code considering both gravity and seismic loads (a design ground acceleration of 0.4 g and soil class Z3 that is similar to class C soil of FEMA-356 (2000) is assumed). The reference buildings have been modified in order to eliminate any vertical or plan irregularities (soft story, short columns, heavy overhangs etc.) to form a basis for evaluation. The older type buildings in Turkey are typically not regular as shown in the Fig. 1. 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. The concrete used in old buildings in Turkey is generally referred as “B225” in blueprints. B225 corresponds to a concrete with 16-18 MPa compressive strength. Authors assumed 16 MPa design strength and divide it by a material factor of 1.5 (TS500 2000). Therefore the in-place concrete strength is assumed as 10.67 in the study. Further reduction in concrete strength is not preferred as it may suppress the effect of considered irregularities. Strain-hardening of longitudinal reinforcement has been taken into account and the ultimate strength of the reinforcement is taken as 330 MPa.

Each of the 4- and 7-story buildings is modified to have one of the aforementioned irregularities. For investigation of the irregularities’ effects on the seismic behaviour, the main strategy is comparison between capacity curves of regular and irregular buildings obtained through non-linear static analysis. However, the nonlinear static analysis does not reflect the effect of pounding. Thus, pounding effect is investigated by using nonlinear time history analysis and seismic energy demands to be absorbed by the structural members.

For nonlinear static analysis, buildings are modelled for two different transverse reinforcement spacing as 100 mm and 200 mm for each irregularity to further investigate the relation between construction and workmanship quality and irregularities. Total of 20 building models, 10 for 4-story and 10 for 7-story, are used for analysis. For pounding analysis, an interior frame is considered for the 4- and 7-story buildings. In order to investigate the effect of pounding, the behaviour of the frame under different cases of pounding is compared to the single frame behaviour. Total of 96 non-linear time history analysis is carried out for 12 different cases of the frames with 8 different ground motion records of past earthquakes.

## 3. Reference buildings & irregularities

The 4- and 7-story regular frame buildings are 16 m by 12 m in plan. They have 4@4 m bays along *X* direction and 4@3 m bays along *Y* direction (Fig. 1). Typical floor height is 2.8 m. The buildings are symmetrical about both horizontal and vertical axes to minimize any irregularity effects other than the subject of study. Also the stronger axes of columns are equally distributed in both axes to prevent any insufficiency in one direction because of orientation of columns. The column and beam dimensions used in this study are typical frame element proportions in the existing building stock in Turkey (Ozmen *et al*. 2015).




(a) Regular frame plan (b) One sided overhang (c) Two sided overhang

Fig. 2 Building models plan views with and without overhangs

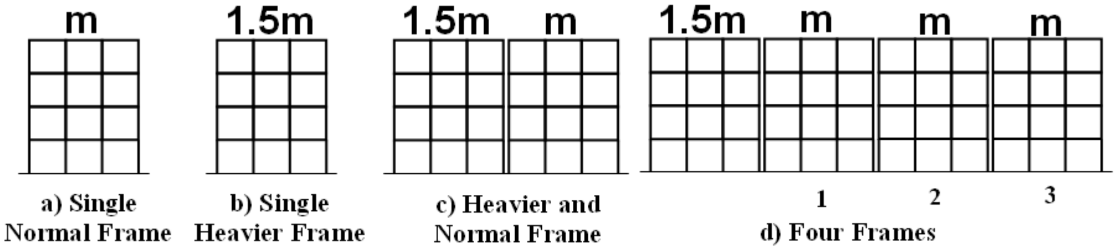



Fig. 3 Investigated cases of pounding

The vertical loads consist of live and dead loads of slabs, wall loads on beams and dead loads of columns and beams. Dead load on slabs is taken as 4.7 kN/m$^2$ for normal floors and 4.5 kN/m$^2$ for roof which also accounts for the weight of isolation and covering system. Live loads on the slabs are taken as 2 kN/m$^2$ for normal floors and 1.5 kN/m$^2$ for roof as indicated in TS 498 (1987). All beams have 6 kN/m dead load accounting for partition walls on them. First mode periods of 4-story building are 0.57 and 0.56 sec in *X* and *Y* directions, respectively while that of 7-story building are 0.78 sec in both *X* and *Y* directions. Note that the story weights consist of dead loads and the assumed portion of live loads by code at the time of earthquake (30% of live load for residential buildings).

Soft story most frequently occurs due to lower stiffness of first floor of buildings. In many cases because of commercial reasons the first story may have a greater height than the upper ones. In this study, first floor height of soft story model is increased to 4 m instead of 2.8 m as in the regular buildings (Ozmen *et al*. 2015).

Short columns are formed due to semi-infilled frames, band windows, semi-buried basement or mid-story beams at the stairway shafts in buildings. In this study, 30-cm length band windows are assumed to cause short columns. All perimeter columns are assumed to be short columns in order to avoid significant torsional behaviour and to have a symmetrical building.

Heavy overhangs shift buildings' mass centre upwards and takes it apart from centre of rigidity. Thus it has negative effects on seismic behaviour. Past earthquakes revealed that buildings with heavy overhangs are more susceptible to damage (Santiago *et al*. 2003, Yon *et al*. 2015). In this study, two cases are modelled: overhangs at one side and overhangs at two cross sides of a building (Fig. 2). For this purpose 1.5 m overhangs are attached to the regular buildings sides. The wall loadings are relocated on the beams surrounding the overhang portion.

In order to investigate pounding effect, non-linear time history analysis is carried out for different cases of the 4- and 7-story frames as illustrated in Fig. 3. First mode periods are 0.45 and 0.61 sec for 4- and 7-story frames, respectively.

In Fig. 3, first case (Case a) is the single frame alone for being the reference for pounding investigation since no pounding effect exists in that case. Second case (Case b) is the single frame with each of its floors is 1.5 times heavier than the single frame. This heavier form of single frame is introduced for the investigation of the pounding effect on the heavier frame. Since the pounding is a phenomenon between two or more neighbouring buildings located at the same region under same architectural rules by the regional authority (municipalities, etc.), the buildings usually have similar number of story and thus similar structural features. If two buildings have exactly the same structural properties there will be no pounding. Because neighbouring buildings with different number of story are considered as a separate case, the mass of the floors is only parameter considered to reflect difference between neighbouring frames having the same number of story in current study. Due to the mentioned reason, the biggest practical difference is taken as 50% in mass meaning approximately 22% difference in their natural periods. Third case (Case c) is the simplest form of pounding between two buildings one heavier than the other. Last case (Case d) is the case intended for the modelling of a more critical form of pounding, because of the chance of all the buildings in the row may hit the ones at the end together, known as "end pounding" (Inel *et al*. 2013, Licari *et al*. 2015). Although only the 4-story frames are illustrated, all cases of analyses are repeated for 7-story frames as well.

In view of the fact that the positive and negative acceleration properties of earthquake records are not same, buildings can have the maximum response either at negative or positive displacement region. In some cases one side can be very dominant. Therefore the locations of the buildings are important for the pounding response. In order to

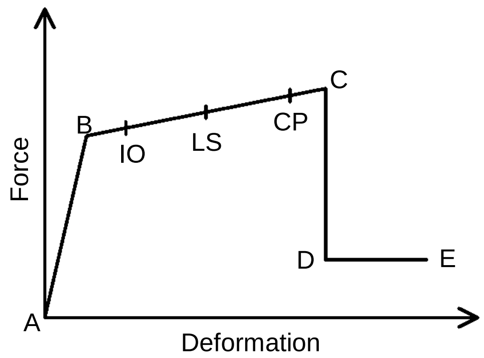


Fig. 4 Force-Deformation relationship of a typical plastic hinge

overcome this problem for the cases c and d, additional non-linear time history analyses are carried out in the reverse order of the frames and the arithmetical average of these analyses results are considered assuming same chance of occurrence of negative and positive side stronger earthquakes.

## 4. Modelling approach

Nonlinear static analyses have been performed using SAP2000 Nonlinear that is a general-purpose structural analysis program (SAP2000). Nonlinear dynamic analyses have been carried out using ZEUS (Elnashai *et al.* 2006), a structural analysis program for the linear and non-linear analysis of the building type structures implementing spread elasticity assumption, taking into account both material and geometric nonlinearity. 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 FEMA-356 (2000) or ATC 40 (1996). As shown in Fig. 4, five points labelled A, B, C, D, and E define force-deformation behaviour of a 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. 4 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; 0.4EI for beams and values depending on axial load level for columns: 0.4EI for $N/(A_c \times f_c) \leq 0.1$ and 0.8EI for $N/(A_c \times f_c) \geq 0.4$. fc is concrete compressive strength, N is axial load, Ac 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.* (1996), given in Eq. (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

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh} \varepsilon_{su}}{f_{cc}} \tag{1}$$

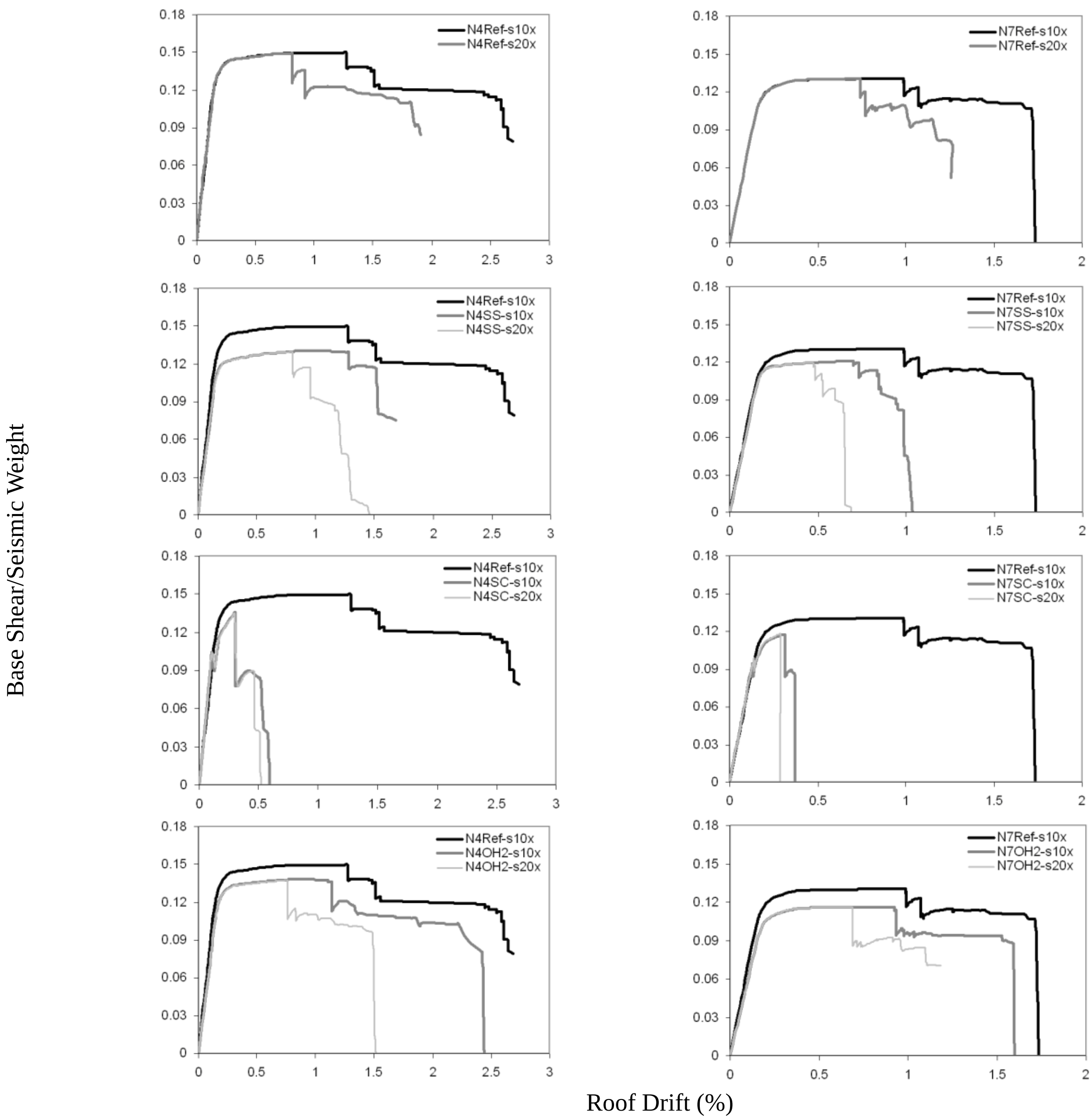
where $\varepsilon_{cu}$ is the ultimate concrete compressive strain, $\varepsilon_{su}$ is the steel strain at maximum tensile stress, $\rho_s$ is the volumetric ratio of confining steel, $f_{yh}$ is the yield strength of transverse reinforcement, and $f_{cc}$ is the peak confined concrete compressive 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, Priestly *et al.* 1996). In this study plastic hinge length definition given in Eq. (2) which is proposed by Priestley *et al.* (1996) is used.

$$L_p = 0.08L + 0.022 f_{yh} d_{bl} \geq 0.044 f_{yh} d_{bl} \tag{2}$$

In Eq. (2), $Lp$ 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. 4. 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 (Inel *et al.* 2008). 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 (2000) that is similar to UBC (1997).

For pounding non-linear time history analyses are performed by ZEUS (Elnashai *et al.* 2006) using linear acceleration assumption by Newmark Integration Scheme. The interior frames are modelled in ZEUS by defining the material, section and member properties. The program determines the moment-curvature relation for the sections using steel, cover and core concrete properties and areas. The most distinguished feature of the program is the used “spread plasticity assumption” of non-linear region through structural members instead of widely used simplification “lumped plasticity approach”. Program establishes the moment-rotation relation by determining the yielded sections along the members and a more accurate analysis can be made. The geometrical nonlinearity and P-delta effects are taken into account by the program. Because pounding is a phenomenon related with demand instead of




(a) 4 story pushover curve (x)

(b) 7 story pushover curve (x)

Fig. 5 Pushover curves for x axis of four and seven story buildings

capacity, no ultimate deformation limits are considered for the structural elements. The pounding effect between buildings is modelled by using gap elements that are infinitely rigid in compression and have zero stiffness for tension. These elements are introduced at each floor level between neighbouring buildings.

## 5. Nonlinear analyses

### *5.1 Pushover analyses*

The 4- and 7-story buildings are firstly subjected to gravity loads and then lateral loading. Gravity loads are in place during lateral loading. In all cases, lateral forces are applied monotonically in a step-by-step nonlinear static analysis. The applied lateral forces are proportional to the product of mass and the first mode shape amplitude at each story level under consideration and applied at centre of mass. *P*-Delta effects are taken into account. Although the first mode shape is used in this study, a non-modal shape vector such as an inverted triangular or a rectangular shape may be used for the lateral load pattern.

Capacity curves are plotted as base shear normalized with seismic weight versus roof displacement normalized by building height. Fig. 5 shows capacity curves of 4- and 7-story buildings in *x*-axis direction for regular and irregular cases. Capacity curves in y axis have similar trend. In Fig. 5, “Ref.” stands for the regular reference building with no irregularity, “SS” stands for soft story, “SC” for short column, “OH1”and “OH2” stand for overhang(s) on one or two sides, respectively. The number after “N” is the number of story and after “s” is the spacing between transverse reinforcement in centimetres (cm).

Table 1 Used Acceleration records and corresponding scale factors

| No | Type | Identifier | Location | Mag. | PGA (g) | Distance (km) | Scale Factor 4-story | Scale Factor 7-story |
|---|---|---|---|---|---|---|---|---|
| 1 | Forward Directivity (FD) | LN92LUCN | Landers | 7.5 | 0.733 | 42 | 0.797 | 0.767 |
| 2 | | LP89SARA | Loma Prieta | 7.1 | 0.504 | 28 | 1.210 | 2.010 |
| 3 | | NR94NWHL | Northridge | 6.7 | 0.589 | 19 | 0.840 | 1.040 |
| 4 | Long Duration (LD) | CH85LLEO | Central Chile | 7.8 | 0.711 | 60 | 1.428 | 2.213 |
| 5 | | IV40ELCN | Imperial Valley | 6.3 | 0.348 | 12 | 1.840 | 2.120 |
| 6 | | LN92JOSH | Landers | 7.5 | 0.274 | 15 | 2.280 | 3.040 |
| 7 | Short Duration (SD) | IV79ARR7 | Imperial Valley | 6.6 | 0.333 | 27 | 1.150 | 1.640 |
| 8 | | BB92CIVC | Big Bear | 6.6 | 0.544 | 12 | 4.430 | 5.050 |

### 5.2 Nonlinear time history analyses

In nonlinear time history analyses the use of acceleration records having different characteristics is an important issue. Because using acceleration records with same features can exaggerate or underestimate some kind of behaviour (Özdemir and Bayhan 2015). In order to overcome this problem acceleration records given in Table 1 are selected among the ones used in FEMA-307 (1998) having different type and attributes as illustrated. The categorization for Short Duration (SD) and Long Duration (LD) is intended to discriminate broadly between records for which the duration of inelastic response is short or long. Ground motions recorded near a rupturing fault may contain relatively large velocity pulses if the fault rupture progresses toward the recording station. Motions selected for the forward directivity (FD) category were identified as containing near-field pulses. For further details FEMA-307 (1998) may be examined.

The acceleration records are scaled to have a maximum roof displacement of 2% of building height for single frames (Case a) by trial-error method to get a comparable and consistent response between the results of the nonlinear time history analyses. 2% drift limit is the design limit for the RC buildings per TEC 2007. The scale factors are also given in Table 1 for the 4- and 7-story frames. As the spacing of lateral reinforcement only affects the capacity rather than the displacement demand, the scale factors are for both of the different transverse reinforcement spacing buildings.

## 6. Effects of irregularities

Capacity curves of buildings are influenced by irregularity parameters in various levels either in strength or deformation terms or both. Rather than relying on solely strength or deformation terms, absorbed energy is used to compare regular and irregular forms of buildings. Such an approach is also useful for reducing the effect of irregularity to a single parameter similar to the penalty scores provided in the several rapid seismic assessment procedures.

Absorbed energy is calculated using the area under the capacity curve as force multiplied by displacement and normalized with mass (Akbas *et al*. 2009, Vimala and Kumar 2016). This quantity is termed as "absorbed unit energy". The effect of each irregularity is determined through the ratio of absorbed unit energy of irregular building to that of regular one. The calculated energy ratios using capacity curves are listed in Table 2. The absorbed energy ratio of short column case is given for 16 short columns out of total 25 columns in the 4- and 7-story buildings. Considering band windows are on one side of building and having minimum two bays at that side, presence of minimum three short columns exist. Then penalty score for short columns is modified with a factor of 3/16 for single short column case.

Effect of irregularities based on absorbed unit energy is summarized in Table 3 after simplification of numbers. Note that the numbers in the table show the absorbed energy ratio of building with irregularity to that of regular building. Transverse reinforcement spacing effect may simply be considered as multiplier for other irregularities to

Table 2 Energy per mass ratios for the given irregularity and reference buildings

| Building No | Irregularity | Axes | *s* (cm) | 4-story $E/E_{ref}$ | 7-story $E/_{Eref}$ |
|---|---|---|---|---|---|
| 1 | Reference | *X* | 10 | 1.00 | 1.00 |
| 2 | | | 20 | 0.68 | 0.69 |
| 3 | | *Y* | 10 | 1.00 | 1.00 |
| 4 | | | 20 | 0.71 | 0.55 |
| 5 | Soft story | *X* | 10 | 0.56 | 0.52 |
| 6 | | | 20 | 0.39 | 0.32 |
| 7 | | *Y* | 10 | 0.48 | 0.49 |
| 8 | | | 20 | 0.36 | 0.28 |
| 9 | Short Column | *X* | 10 | 0.84 | 0.84 |
| 10 | | | 20 | 0.84 | 0.84 |
| 11 | | *Y* | 10 | 0.84 | 0.84 |
| 12 | | | 20 | 0.84 | 0.83 |
| 13 | Overhang 1 | *X* | 10 | 0.91 | 0.88 |
| 14 | | | 20 | 0.61 | 0.61 |
| 15 | | *Y* | 10 | 0.80 | 0.70 |
| 16 | | | 20 | 0.49 | 0.48 |
| 17 | Overhang 2 | *X* | 10 | 0.81 | 0.80 |
| 18 | | | 20 | 0.48 | 0.55 |
| 19 | | *Y* | 10 | 0.77 | 0.68 |
| 20 | | | 20 | 0.48 | 0.44 |

Table 3 Effect of irregularities suggested in the current study

| Irregularity | 4-story | 7-story |
|---|---|---|
| Reference Building | 1.00 | 1.00 |
| Construction and Workmanship Quality (If poor) | 0.70 | 0.60 |
| Soft Story | 0.55 | 0.50 |
| Short Column | 0.85 | 0.85 |
| Overhangs at one side | 0.85 | 0.85 |
| Overhangs at two sides | 0.75 | 0.75 |

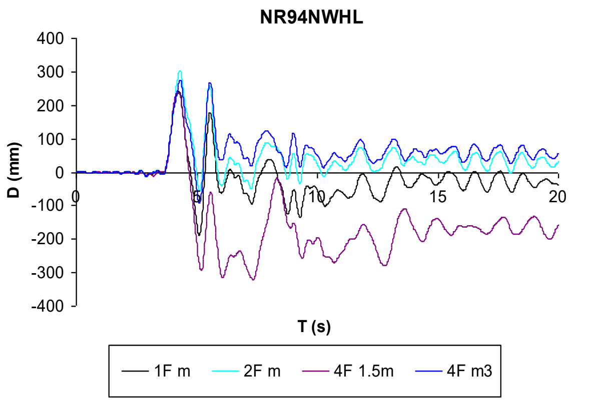



Fig. 6 Displacement vs. time graph for all the cases subjected to NR94NWHL

Table 4 Energy demand ratios for pounding

| Case | 4-story | 7-story |
|---|---|---|
| 2F 1.5 m | 1.020 | 0.995 |
| 2F m | 1.038 | 1.085 |
| 4F 1.5 m | 1.056 | 1.002 |
| 4F m1 | 0.943 | 1.048 |
| 4F m2 | 0.957 | 1.046 |
| 4F m3 | 1.035 | 1.061 |

reflect construction and workmanship quality.

Pounding is a dynamic phenomenon related with demand rather than capacity. Absorbed energy demand for pounding cases is determined using time history results to be consistent with other irregularity parameters. An example pounding effect is plotted in Fig. 6 for the 4-story building subjected to Northridge Earthquake recorded at Newhall station (NR94NWHL) ground motion. Average of eight ground motions for each case is given in Table 4. The number before "F" stands for the number of neighbouring frames, and the number before "m" is the weight ratio of the frame that of reference frame, the number after "m" shows the location of the frame in Case d, which is illustrated in Fig. 3 under each frame. Since each case in Table 4 stands for different orientation of a building for pounding, maximum values are chosen as 1.056 and 1.085 for 4- and 7-story frames, respectively. This means that pounding causes 5.6% and 8.5% increase in energy demands to be absorbed by the 4- and 7-story buildings, respectively.

Several seismic assessment procedures recommend relative figures indicating the influence of the architectural features and the construction and workmanship quality on the seismic vulnerability (FEMA-154 1988 and 2002, Ozcebe 2004, Yakut 2004). These procedures are selected in this study to analytically evaluate relative importance of certain irregularities. The relative penalty scores employed in this study have been normalized to determine relative significance as provided in Table 5. Note that the weighting coefficients given in Table 5 represent the relativity among the coefficients, thus they add up to unity.

The study by Middle East Technical University (METU) (Ozcebe 2004) uses irregularity features as given in Table 5. Penalty scores given for 4- and 7-story buildings are

Table 5 Relative weight of irregularity coefficients in the previous and current study

| Feature | Weighting Coefficients | | | | |
|---|---|---|---|---|---|
| | FEMA-154 (1988) | FEMA-154 (2002) | Ozcebe (2004) | Yakut (2004) | Current Study |
| Soft Story | 0.31 | 0.44 | 0.38 | 0.39 | 0.38 |
| Short Columns | 0.15 | | 0.11 | 0.15 | 0.12 |
| Heavy Overhangs | 0.15 | 0.15 | 0.22 | 0.16 | 0.16 |
| Pounding | 0.08 | | 0.07 | --- | 0.06 |
| Construction and Workmanship Quality | 0.31 | --- | 0.22 | 0.30 | 0.28 |

averaged. While FEMA-154 (1988) provides penalty scores for soft story, short column, and pounding with the same term; heavy cladding and post-benchmark year features are considered to represent heavy overhangs and construction and workmanship quality, respectively. The updated FEMA-154 (2002) decreased the number of parameters by grouping similar parameters under the same feature. Soft story and short columns are considered as vertical irregularity. Heavy overhangs, construction and workmanship quality is assumed to be in plan irregularity and post-benchmark year, respectively. Pounding is not included in the updated FEMA-154 (2002). Yakut (2004) provides penalty scores for soft story and short columns as named in Table 5 while heavy overhangs is represented through plan irregularity. Pounding is not taken into consideration. Construction and workmanship quality is given as a separate parameter depending on other irregularity parameters and is taken as consistently.

The effect of irregularities differs in construction and workmanship quality and soft story between 4- and 7-story buildings. The negative effect of quality increases as the number of story increases in buildings, consistent with the observed damage in Duzce after 1999 Duzce Earthquake (Ozcebe 2004). Since the differences are in the range of 10-15%, penalty scores can be simplified independent of number of stories for low- and mid-rise RC buildings. Relative penalty scores (total adds up to unity) are shown in Table 5.

## 7. Conclusions

This study examines the effects of soft story, short columns, heavy overhangs, pounding, and construction and workmanship quality parameters on seismic response of reinforced concrete buildings through nonlinear static and dynamic procedures. The 4- and 7-story buildings designed according to pre-modern code are used to reflect majority of the existing building stock. The relative penalty scores are employed in this study to evaluate relative importance of certain irregularities in the existing seismic assessment procedures.

Penalty scores for seismic assessment procedures are either based on solely engineering judgment, statistics of damaged buildings during an earthquake and engineering judgment, or geometric and material properties of components comprising lateral load resisting structural system. Although each procedure considered in this study may have a different basis, relative penalty scores have similarities. The observations can be summarized as:

- Relative penalty score of soft story and short columns is in the range 0.45-0.55. Studies that reflect construction practice in Turkey (Ozcebe 2004, Yakut 2004 and current study) penalize soft story higher than FEMA-154 (1988, 2002). Since The METU study (Ozcebe 2004) is based on statistical data of damaged buildings, it can be concluded that soft story is more common and is noticeably destructive during an earthquake in Turkey.
- Short column penalty score is approximately the same for all procedures.
- Except the METU study (Ozcebe 2004), the heavy overhang penalty scores are the same. This is mainly due to construction practice in Turkey. In cases of overhang, because of architectural reasons the beam connecting the columns at the side with an overhang is not constructed. This results in an additional irregularity of frame discontinuity. This negative effect is included in the penalty score for heavy overhang for the statistically based study (Ozcebe 2004).
- Pounding effect has similar and the least score in the studies that consider this effect. This may be explained by low chance of occurrence of pounding having a detrimental effect. However, pounding of buildings with different story levels can cause significant damage.
- Penalty score of construction and workmanship quality has some differences. FEMA-154 (2002) gives the highest score. This can be explained by exclusion of pounding and due to lumping soft story and short columns together that results in consideration of less number of parameters. The METU study gives the least score in which buildings are classified in three groups as good, average, and poor. In Table 5, relative score is given as the difference between average and poor building by assuming the reference buildings being average. When the difference is considered for good and poor cases, the penalty score doubles. This increases the relative penalty score for construction and workmanship quality.

Comparison of relative scores for the irregularities considered in this study shows that the overall trend is similar. The relatively small differences may be accounted for regional construction practices. This analytical study concludes that initial-phase seismic assessment procedures based on architectural features yield in closer results independent of their bases. The relative scores obtained from the analytical investigation have also a good agreement with the assessment procedures considered for comparisons. However, the differences in the scores emphasize the proper selection of the method based on the regional structure characteristics.

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