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Shear stress indicator to predict seismic performance of residential RC buildings

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Abstract. A large number of residential buildings in regions subjected to severe earthquakes do not have enough load carrying capacity. The most of them have been constructed without receiving any structural engineering attention. It is practically almost impossible to perform detailed experimental evaluation and analytical analysis for each building to determine their seismic vulnerability, because of time and cost constraints. This fact points to a need for a simple evaluation method that focuses on selection of buildings which do not have the life safety performance level by adopting the main requirements given in the seismic codes. This paper deals with seismic assessment of existing reinforced concrete residential buildings and contains an alternative simplified procedure for seismic evaluation of buildings. Accuracy of the proposed procedure is examined by taking into account existing 250 buildings. When the results of the proposed procedure are compared with those of the detailed analyses, it can be seen that the results are quite compatible. It is seen that the accuracy of the proposed procedure is about 80% according to the detailed analysis results of existing buildings. This accuracy percentage indicates that the proposed procedure in this paper can be easily applied to existing buildings to predict their seismic performance level as a first approach before implementing the detailed and complex analyses.

Keywords: buildings; confined concrete; earthquake engineering; reinforced concrete buildings; seismic evaluation of existing buildings

1. Introduction

In the past two decades a lot of countries have experienced several moderate to severe earthquakes. There are a lot of literatures which are related to the earthquake effects on the reinforced concrete (RC) buildings, which are presented some comprehensive experiences after severe earthquakes, and examining issues such as observed structural damage, causes of damage, performance of structures, structural deficiencies etc. (Dogangun 2004, Sezen *et al.* 2003, Yakut *et al.* 2005, Inel *et al*. 2008, Yon *et al.* 2013, 2015a, Inel and Meral 2016, Bikçe and Çelik 2016).

In earthquake regions, there are a large number of residential buildings which have very low level of seismic safety. Since the most of these buildings have been constructed without controlling by structural engineers and their number is quite large, there is a need for a simple evaluation method that focuses on determination of buildings which do not satisfy "the life safety performance level" required by earthquake codes. To scrutinize seismic vulnerability of buildings, experimental and theoretical studies must be done on existing buildings (Desprez *et al.* 2015, Yon *et al.* 2015b, Onat *et al.* 2015, Oncu and Yon 2016, Onat *et al.* 2016). These issues are too hard and needs long time and have high costs. In many earthquake codes, a series of procedures have been proposed to determine the seismic performance level and the seismic vulnerability of the existing buildings. These procedures require detailed structural and seismic engineering know-how, and detailed experimental material tests and complex structural analyses. They have been implemented successfully for these cases, when number of buildings are limited, financial fund is available and slight acceptable damages exist. When the last three conditions are not satisfied-which is often the case-, there is a need for a simple evaluation method to predict the seismic vulnerability of buildings. Recently, the needs for developments of practical techniques on prediction of seismic vulnerability of existing buildings are on the increase.

To evaluate seismic safety assessment of RC frame-wall buildings, a method have been presented by Akbay and Aktan (1991). In this method, shear stiffness model is experimentally developed for distribution of shear stresses along the reinforced concrete shear walls. Shear wall specimens are tested to obtain their shear strength and critical moment-to-shear ratios.

Ozmen *et al.* (2014) have investigated effects of parameters which are important for seismic performances of RC buildings. Evaluations on the effect of the parameters for different performance levels and seismic loadings have been presented in the paper. Seismic performances of the models have been determined for different performance levels and seismic loading conditions.

Korkmaz *et al.* (2015) have investigated the earthquake

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performances of reinforced concrete (R/C) residential buildings in Turkey and damage parameters observed from previous earthquakes have been defined as structural parameters. Different types of frame buildings have been modeled. Parameters have been taken into account as the number of stories, column sizes, and reinforcement and concrete strength. Different R/C buildings were analyzed for performance evaluation. The influence of parameters has been investigated on the structural performance of the buildings.

Kunnath *et al.* (1995) have evaluated the seismic performance of non-ductile RC frame buildings in regions of low to moderate seismicity. Several significant aspects of non-ductile detailing are modeled using rational simplifications of expected member behavior at critical sections to facilitate a complete inelastic time history analysis of the structural system. Discontinuous positive flexural reinforcement, lack of joint shear reinforcement, and inadequate transverse reinforcement for column core confinement are investigated by carrying out analysis. Seismic evaluations of three-, six-, and nine story buildings are carried out under low- to moderate earthquake excitations. The essential parameters of the response are presented with a point of view to identify the vulnerability of such buildings to a potential seismic design event.

Hassan and Sozen (1997) have presented a simplified method of ranking reinforced concrete, low-rise (up to 5 stories), and monolithic buildings according to their vulnerability to seismic damage. The proposed ranking process requires only structural dimensions as the input and is based on effective wall and column indices plotted in a two-dimensional form. The process is tested by using a group of buildings that suffered various levels of damage during the Erzincan Earthquake of 1992.

Ozmen (2013) has investigated performances of the rapid evaluation methods to estimate seismic damage by examining the correlation between the rapid evaluation method scores and the quantified damage states after the Simav earthquake. A total of 144 reinforced concrete buildings have been examined by considering the properties of structural system. It is concluded that the estimations with the rapid evaluation methods may diverge from the actual scene after an earthquake.

Bilgin (2013) has carried out seismic fragility assessment of reinforced concrete buildings. Lateral stiffness, strength and displacement capacities of the selected buildings have been determined by nonlinear static analyses. The inelastic dynamic characteristics of the buildings have been investigated by using a set of 100 strong ground motion records. The results have revealed that the effect of concrete and detailing quality on Immediate Occupancy performance level is more limited and less critical as the ground motion intensity increases.

A procedure to determine the seismic vulnerability of existing building structures have presented by Gülkan and Sozen (1999). In this method, a rationalization for ranking RC frame buildings with masonry infill walls with regard to seismic vulnerability is presented. The method essentially requires only the dimensions of the structure as input and is expressed in terms of where its attributes are located in a two-dimensional plot of masonry wall and column percentages. It is shown that increasing drift at the ground story is attained by decreasing either attribute. It is shown that a more robust estimate of the contribution of the filler wall to frame stiffness should be based on the compression-tension strength of its mortar rather than the elastic modulus, either of the masonry or mortar.

Matamoros *et al.* (2004) have proposed a simplified procedure to proportion earthquake-resistant RC structures without irregularities. The flat rate method is used to assess the vulnerability of low- and medium-rise RC existing buildings to earthquakes in a simple manner. The method is based on the concept that the maximum expected roof drift of a building is proportional to the ratio of total mass to stiffness of the lateral load resisting system.

Yakut (2004) has proposed a preliminary procedure to assess rapidly the likely seismic performance of existing RC buildings. In this procedure, Capacity Index is computed considering the orientation, size and material properties of the components comprising the lateral load resisting structural system. The procedure has been tested and calibrated based on the data compiled from damage surveys conducted after the earthquakes that occurred within the last decade in Turkey.

Albayrak *et al.* (2015) have presented the techniques of the rapid street screening procedure for seismic failure risk assessment in buildings. Risk assessment criteria is considered as the age of building, number of stories, existence of soft story, short column, heavy overhangs, pounding affect, topographic effects, visual building construction quality and earthquake zone. Each building is classified as high risk, moderate risk and low risk by calculating performance risk score of buildings.

Bianco and Granati (2015) have presented a method to carry out a simplified and fast non-linear static seismic assessment of an existing RC building. This method is based on the evaluation of the control point displacement. The proposed procedure is applied to some RC buildings in literature, and whose results are retrieved among the latest scientific publications.

Gaudio *et al.* (2015) have proposed a simplified analytical method based on Shear-Type assumption for the seismic fragility assessment of RC buildings. Presence of infill walls is considered in this method. The influence of parameters in predicting seismic fragility, such as the number of stories and the age of construction, is investigated. The results of the study are compared with statistical data about the characteristics of building stock.

Dya and Oreta (2015) investigated preliminary risk assessment of existing buildings which have the soft story. The buildings are analyzed by using nonlinear static pushover method to determine the seismic performance of the building. The study has been found that one of the primary concerns in vertical irregularities is the localization of seismic demand.

The purpose of this paper is to investigate feasibility of shear stress as an indicator to predict the seismic safety assessment of existing RC residential buildings. Seismic performances of the buildings are evaluated according to Turkish Earthquake Code 2007 (TEC 2007). The presented procedure is an alternative approach for seismic evaluation of existing buildings. Although the presented procedure is

Table 1 Concrete and steel strain limits at the fibers of a cross section for different damage limits

Damage Level	Concrete Strain Limit	Steel Strain Limit
Concrete and steel strain limits at the fibers of a cross section for minimum damage limit (MN)	$(\varepsilon_c)_{MN}$=0.0035	$(\varepsilon_s)_{MN}$=0.01
Concrete and steel strain limits at the fibers of a cross section for safety limit (SL)	$(\varepsilon_c)_{SL}=0.0035+0.01(\rho_s/\rho_{sm})\leq 0.0135$	$(\varepsilon_s)_{SL}$=0.04
Concrete and steel strain limits at the fibers of a cross section for collapse limit (CL)	$(\varepsilon_c)_{CL}=0.004+0.014(\rho_s/\rho_{sm})\leq 0.018$	$(\varepsilon_s)_{CL}$=0.06

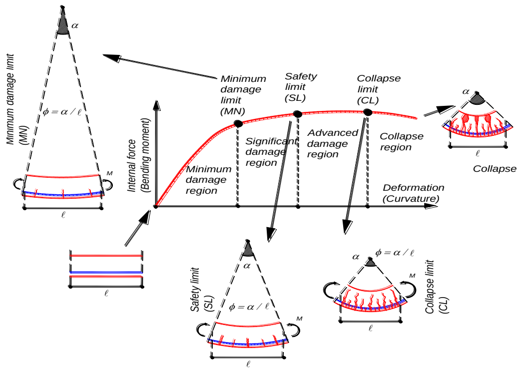


Fig. 1 Damage limits and damage states in a ductile member (Celep 2014)

simple, it can be said that the procedure has an acceptable level of accuracy when compared to the uncertainties in existing buildings. Seismic performance of RC buildings are evaluated by employing "Shear Stress Indicator" (SSI) which is computed by considering the only column dimensions of structural system. By using the presented procedure, it can be easily and rapidly determined, whether life safety performance level of the buildings is satisfied or not. However, there will be always buildings in the gray zone for which the detailed analysis methods given in codes will be applied and engineering skills will be needed about seismic performance level of the buildings in gray zone.

2. Seismic performance of RC buildings

First step of the performance analysis of existing buildings is to collect information on these structures. The information collected on existing buildings is classified with respect to the scope of data and a type of load bearing 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 (TEC 2007).

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. But due to its difficulty in the applications, pushover analysis is more popular than nonlinear time history analysis for engineers (Tekeli *et al.* 2013). 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 (single mode pushover method) is used in the numerical analyses. In nonlinear static analysis, lateral forces are increased until the earthquake displacement demand is reached. The base shear force versus roof displacement curves of buildings are obtained by using plastic hinges at the both ends of the beams and columns. The plastic hinge length L_p is assumed to be half of the section depth ($L_p=h/2$) (TEC 2007). 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. Moment-curvature diagrams of the critical sections are obtained by applying appropriate stress-strain rules for concrete and steel. Unconfined and confined concrete models developed by Mander *et al.* (1988) are used in analyses. The calculated strain demands are compared with the damage limits. Concrete and steel strain limits at the fibers of a cross section for minimum damage limit (MN), safety limit (SL), and collapse limit (CL) are given in Table 1. In the expressions, ε_c is the concrete strain at the outer fiber, 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.

Generally, structural members can be classified as "ductile" or "brittle" with respect to their mode of failure in determining the damage limits. Three damage limits are defined at the cross section level for ductile members. These are minimum damage limit (MN), safety limit (SL) and collapse limit (CL) as shown in Fig. 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.

Building earthquake performance levels are determined after establishing the member damage states. Four performance levels are defined for RC buildings. Since residential buildings are examined in this study, the buildings are expected to satisfy the life safety performance level under the design spectrum obtained for 10% probability of exceeding in 50 years. The rules for determining building performance are given below for each performance level (TEC 2007):

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 advanced damage state. Shear carried by those columns in the advanced 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 advanced 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 such building should not be permitted.

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

3. Determination of shear stress indicator

A simplified procedure is proposed to evaluate the seismic safety of low-rise buildings in this study. The proposed procedure is recommended for low- to mid-rise RC frame buildings without shear walls, and aims to develop a simplified procedure to assess the seismic performance of existing RC buildings by considering the related rules given in the TEC 2007 which has similar performance evaluation requirements to those of FEMA 356. For the numerical treatment of the procedure, only RC frame system buildings are considered. The procedure assumes that the ground story is the critical story and the performance of the columns at this story governs the seismic performance safety level of the building. When the building has a basement having periphery RC wall, the ground story is still the critical story. However, when the basement does not have periphery RC walls, then the critical story should be assumed as the basement. It is quite possible that in some cases it can be difficult to determine the critical story depending on the discontinuity of periphery walls. In such cases, the following two stories can be considered to find the critical story (Tekeli *et al.* 2014). The procedure focuses on determining of a Shear Stress Indicator (SSI) value for the critical story which is the ground story in general. The SSI value is defined as a ratio of the elastic seismic story shear to the total column cross section area at the ground story, which can be seen in Eq. (1) as reciprocal of the average shear stress as well.

$$\mathrm{SSI} = \frac{V_t}{A_c} \tag{1}$$

Where A_c is cross-section area of columns. According to TEC 2007, the elastic base shear V_t can be calculated as

$$V_t = A_0 \times \mathrm{I} \times \mathrm{S(T)} \times \mathrm{W} \tag{2}$$

Where A_o is the effective ground acceleration coefficient, I is the building importance factor, $S(T)$ is the spectrum coefficient and W is the total weight of building, which include the dead load and the live load with a participation factor being 0.3 for the residential buildings. In this study, in order to cover wide range of buildings, A_o, I and $S(T)$ are taken into account as 0.4, 1.0 and 2.5 respectively. A_0=0.4 corresponds to the first seismicity zone in Turkey. Furthermore $S(T)$=2.5 corresponds to low and moderately high buildings. The SSI value is defined as given in Eq. (3) by arranging Eq. (1),

$$\mathrm{SSI} = \frac{\mathrm{W}}{A_c} \tag{3}$$

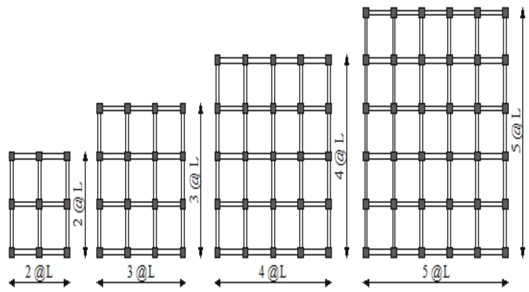
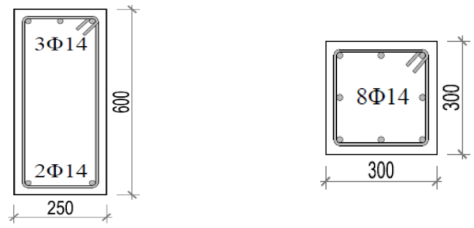


Fig. 2 Structural layouts of the RC building models

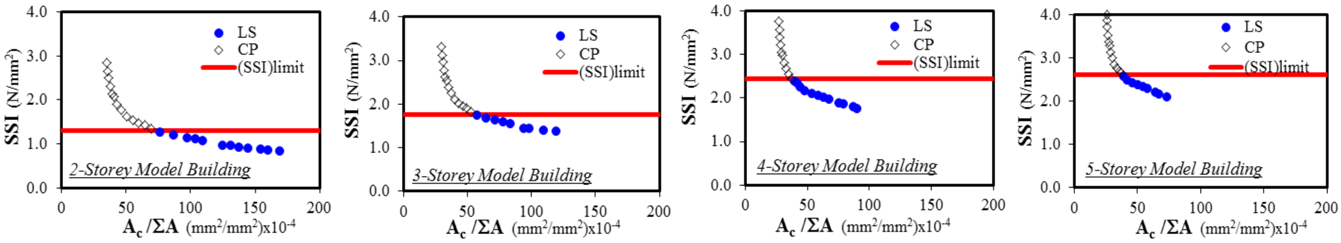


(a) (b)

Fig. 3 Configurations of longitudinal reinforcement of the beam and column

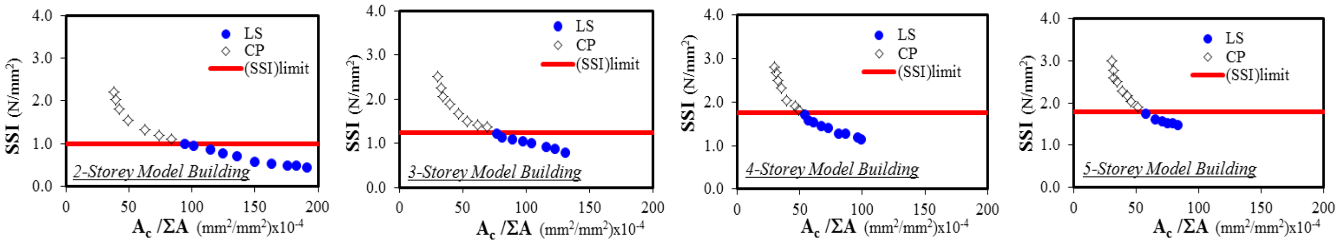
In order to decide whether the building satisfies the life safety performance level or not, the evaluated SSI value is compared with the limiting value of SSI ($\mathrm{SSI}_{\mathrm{limit}}$). When the SSI value of the building is smaller than the corresponding $\mathrm{SSI}_{\mathrm{limit}}$ value, it is concluded that the building satisfies the life safety performance level. Otherwise, it is assumed that the building does not satisfy the life safety performance level. The values of $\mathrm{SSI}_{\mathrm{limit}}$ are determined by using the results of the large number of analyses of the selected buildings. As, it is well-known, the seismic safety of a building depends on the structural configuration, on the layout of the columns and the beams having regular frame system in two directions and their proportions. Furthermore, it depends on the seismic zone, on soil type as well as on number of the stories. All these parameters are taken into account in evaluation of the limiting values of the SSI. In some codes (AIJ 1992) and studies (Mollick 1995, Otani 2000, Tekeli *et al.* 2013, 2014) by considering average shear stress value, a procedure is proposed by using similar assumptions. For example, the average shear stress value is bounded by Mollick (1995) as 1.2 N/mm^2. The average value is taken into account as independent from number of story and material classes. The most important advantage of the proposed procedure in this study is to identify limit values depending on the number of story and material classes.

Numerous analyses are performed to investigate the effects of various structural parameters on the seismic performance of buildings and to determine their effects on the limiting values of the SSI in the proposed procedure. In fact, for this reason, approximately 150 buildings having different characteristics were selected for analysis, and three-dimensional models of each of the buildings are developed and their analysis are performed by using SAP 2000 software adopting the pushover analysis to determine their performance levels. These analyses are carried out for



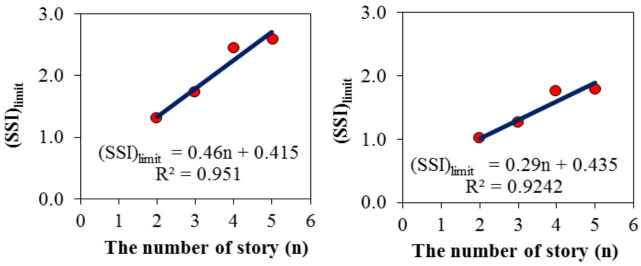
(a) (b) (c) (d)

Fig. 4 The limiting values of SSI (SSI_{limit}) for the RC building models (Material A)



(a) (b) (c) (d)

Fig. 5 The limiting values of SSI (SSI_{limit}) for the RC building models (Material B)



(a) “Material A” (b) “Material B”

Fig. 6 The relationship between SSI_{limit} and the number of story

Table 2 The SSI_{limit} values obtained from the idealized relationship

The number of story	$(SSI)_{limit}$	
	“Material A”	“Material B”
2	1.3	1.0
3	1.8	1.3
4	2.3	1.6
5	2.7	1.9
6	3.2	2.2

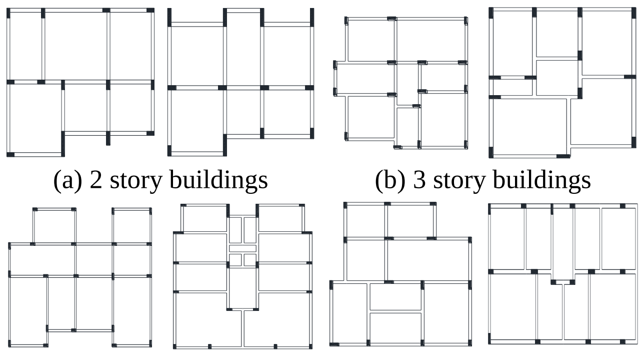
the buildings having the selected structural configuration by varying the number of storey, the column sections, number of spans in two directions, concrete strength, steel yield strength, spacing of confinement (transverse) reinforcements. Gravity and seismic loads are considered by assuming the design ground acceleration of 0.4g (first seismic zone) and the soil class C according to FEMA 356. RC building models having 2, 3, 4 and 5 stories are developed to represent low- and mid-rise buildings located in the high seismicity regions of Turkey. The span numbers of the structural model in both *x* and *y* directions are selected as 2, 3, 4 and 5 having a length of 4.0 m. The buildings have symmetrical structural layout in plan with respect to the both *x* and *y* axes to adopt a building without any irregularity. The structural configurations of the selected buildings are given in Fig. 2.

Performance level of the selected model buildings are determined for two different cases, such as, concrete strength of 20 MPa and 10 MPa, steel yield strength of 420 MPa and 220 MPa, spacing of transverse reinforcement of 100 mm and 200 mm, respectively. The second case is defined as “Material B”, while the first case is defined as “Material A” in the study. All beams in the buildings have the same cross-section of 250 mm×600 mm. Performance analysis of the building was carried out by using SAP 2000 program for different cross section of column.

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. The cross sections of the columns are increased step by step, equally in both directions (i.e., 10 mm). This process is repeated for different steps of the analyses to show the behavior and variation. Longitudinal reinforcement ratios vary between 1% and 1.5 % in all columns. Configuration of longitudinal reinforcements of the columns having 300 mm×300 mm sizes as an example and beams are given in Fig. 3.

The average results obtained from the analyses are given in Figs. 4 and 5, respectively. SSI_{limit} values are determined in the graphics where Σ*A* and A_c corresponds to total floor area of building and total of cross-section areas of all columns at the critical story, respectively. As it is seen, the SSI_{limit} is different for each building considered having various concrete strength, span and number of stories.

Fig. 6 shows idealized relationship between SSI_{limit} value and the number of story (*n*) depending on material classes. The SSI_{limit} values obtained from the idealized relationship are given in Table 2 based on the number of story and material classes.



(a) 2 story buildings (b) 3 story buildings

(c) 4 story buildings (d) 5 story buildings

Fig. 7 Structural layouts of some of the selected existing buildings

4. Application of procedure and numerical solutions

Accuracy of the proposed procedure is examined on existing 250 RC buildings which are randomly selected from the different cities in Turkey. Although the results presented above are limited for the buildings located in the first seismic zones and on the soil class C according to FEMA 356, the results can be extended to the other seismic zone and soil types as well by extending the numerical analysis. The performances of the selected 250 different buildings are investigated by considering the detailed rules given in the TEC 2007 and by considering the proposed procedure. The results obtained from these analyses are given in figures, comparatively. The analyses are carried out by considering the design parameters of the building which are obtained from their blueprint drawings. Structural configurations of some of the selected buildings are given in Fig. 7. The structural properties of the selected buildings are given in Table 3.

The obtained results from analyses are given for two different cases. In the first case, it assumed that the concrete and reinforcement strengths are 20 MPa and 420 MPa, respectively considering confinement exists, that is transverse reinforcement (stirrup) spacing is 100 mm (Material A). In the other case, the material strengths are assumed to be 10 MPa and 220 MPa, respectively without confinement, where stirrup spacing is equal to 200 mm (Material B).

The validity of the proposed procedure is given in Figs. 8 and 9 for "Material A" and "Material B", respectively. When the results of the proposed procedure are compared with those of the detailed analysis results of existing buildings, it can be seen that the results are quite compatible. The accuracy of the proposed procedure for analysis of existing buildings is determined to be around 80%.

5. Conclusions

In this study, a simplified procedure is presented for preliminary seismic vulnerability of existing RC residential buildings having frame type structural system. The procedure uses the dimensions of structural members only,

Table 3 The structural properties of the selected some buildings

Building ID	The number of storey	T_1 (sn)	h_i (m)	$A_{total\ floor\ area}$ (m^2)	$W_{building}$ (kN)	A_c (m^2)
1	2	0.23	2.9	183	1997	2.00
2	2	0.23	3.0	218	2789	2.35
3	2	0.21	2.8	155	2383	2.41
4	2	0.49	4.5	973	9167	3.85
5	3	0.29	3.0	3119	23959	16.64
6	3	0.28	3.0	318	2602	2.16
7	3	0.33	3.0	353	4178	2.81
8	3	0.38	3.0	404	4238	2.30
9	4	0.40	2.9	457	4332	1.95
10	4	0.39	2.9	338	4227	3.00
11	4	0.38	2.9	650	6536	2.81
12	4	0.51	2.9	540	5180	1.88
13	5	0.41	2.9	428	5460	2.27
14	5	0.41	2.9	861	8712	3.63
15	5	0.57	3.0	628	6524	2.30
16	5	0.86	3.0	720	7930	2.35
17	6	0.52	3.0	766	8886	4.00
18	6	0.45	3.0	2228	26317	10.72
19	6	0.76	2.9	653	6836	1.50
20	6	0.75	3.0	1280	12468	3.45

Table 4 The performance levels obtained with proposed procedure and SAP 2000 program for the selected some buildings

Building ID	The number of storey	Perf. Level (Sap 2000)	Proposed Procedure			
			SSI=W/A_c (N/mm^2)	$(SSI)_{limit}$	Perf. Level	Compatible (C) Incompatible (IC)
1	2	LS	1.0	1.3	LS	C
2	2	LS	1.2	1.3	LS	C
3	2	LS	1.0	1.3	LS	C
4	2	CP	2.4	1.3	CP	C
5	3	LS	1.4	1.8	LS	C
6	3	LS	1.2	1.8	LS	C
7	3	CP	1.5	1.8	LS	IC
8	3	CP	1.8	1.8	CP	C
9	4	LS	2.2	2.3	LS	C
10	4	LS	1.4	2.3	LS	C
11	4	LS	2.3	2.3	CP	IC
12	4	CP	2.8	2.3	CP	C
13	5	LS	2.4	2.7	LS	C
14	5	LS	2.4	2.7	LS	C
15	5	CP	2.8	2.7	CP	C
16	5	CP	3.4	2.7	CP	C
17	6	LS	2.2	3.2	LS	C
18	6	LS	2.5	3.2	LS	C
19	6	CP	4.6	3.2	CP	C
20	6	CP	3.6	3.2	CP	C

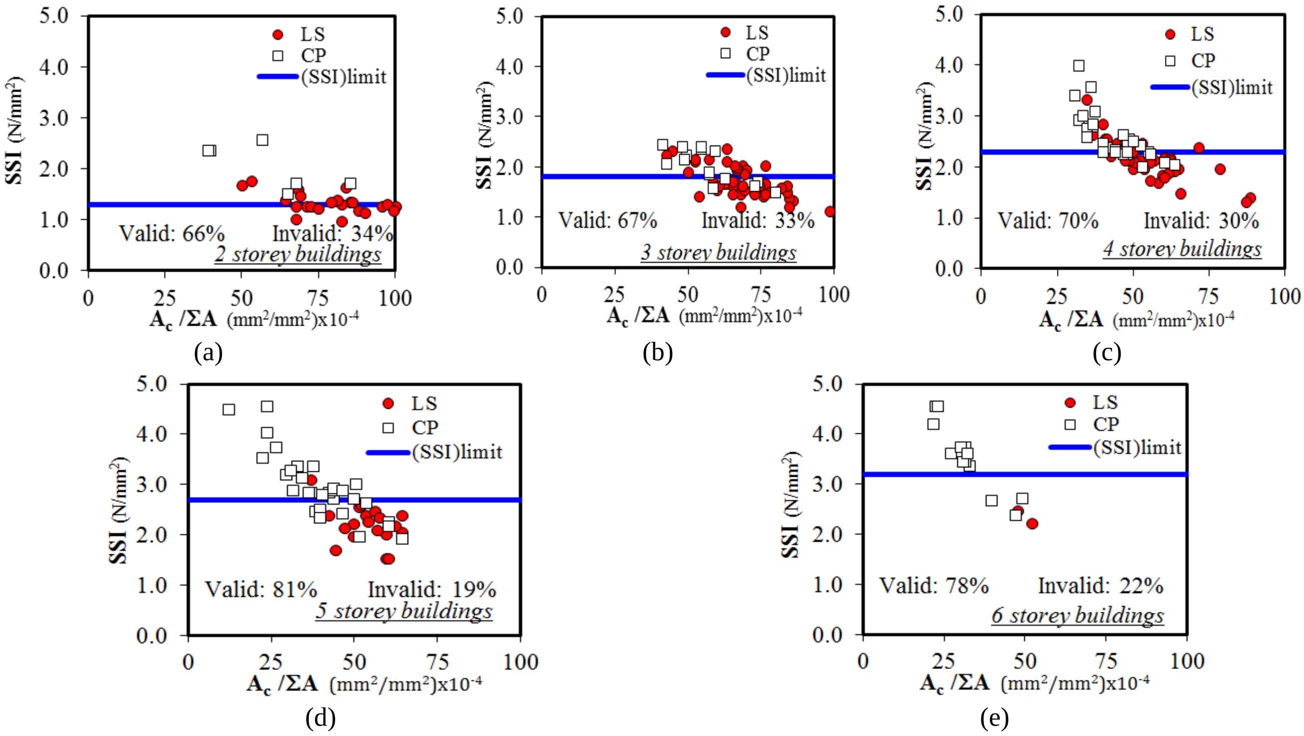


Fig. 8 Validity of proposed procedure for existing buildings (Material A)

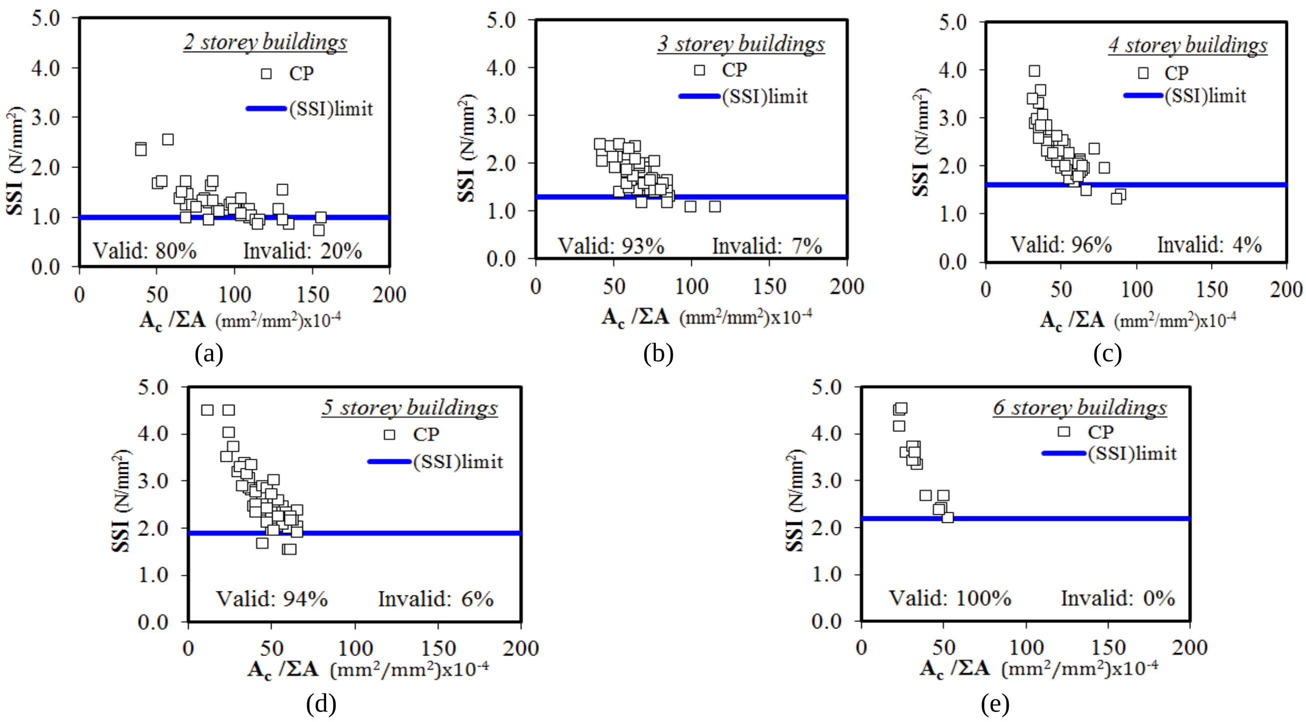


Fig. 9 Validity of proposed procedure for existing buildings (Material B)

and it is calculated based on shear stresses. In this study, the shear stress is called as the Stress Shear Indicator (SSI), which is obtained as a ratio of the seismic story shear force to the total cross-sectional areas of all columns at the ground (critical) story. In the proposed procedure, the SSI value of an existing building is compared to the limiting values of SSI (SSI_{limit}), which is evaluated by the performance assessment of numerous model buildings by using the pushover analysis given in TEC 2007. The procedure requires a minimum level of information and computation and it is very appealing because practicing engineers need simple ways to do quick seismic assessment on the performance level of buildings. The following conclusions can be derived from this study:

1. The results of the analyses show that the numbers of stories, floor areas of buildings, cross sectional area of

columns and the material strength are significant parameters in identifying the seismic vulnerability of RC buildings.

2. SSI_{limit} values are determined according to the number of stories, material strengths, amount of transverse reinforcement. From the results of SSI analyses, it is seen that as the number of stories increase, SSI_{limit} values increase.

3. SSI_{limit} values are obtained for strengths of two different materials and amount of transverse reinforcement. They change between 1.3 and 3.2 for “Material A”, while the values change between 1.0 and 2.2 for “Material B”. As the strengths of materials decrease, the SSI_{limit} values also decrease. The dimensions of RC column cross-sections need to be increased to satisfy the LS performance level of RC building with low strength of materials.

4. The correlations between the number of story and SSI_{limit} values are obtained as 0.95 and 0.92 from the analyses for “Material A” and “Material B” respectively. The corresponding SSI_{limit} values can be determined from the correlation.

5. The accuracy of the procedure is investigated by applying the proposed procedure in this paper on a large number of selected existing buildings and by comparing the results of this procedure with the results of other analysis methods. It can be seen that the results of the proposed procedure are quite compatible.

6. The accuracy of the proposed procedure is determined between 66% and 81% for “Material A” as depending on the number of story. The accuracy is determined between 80% and 100% for “Material B”. The proposed procedure gives good accuracy especially in the analyses of RC buildings with low material quality.

7. These percentages indicate that the proposed procedure can be applied to the existing buildings to predict their performance level. The procedure is recommended for predicting performance levels of low- to mid-rise RC buildings.

8. The proposed procedure provides an acceptable estimation about whether the existing buildings meet the LS performance level as defined in the TEC 2007 or not. The procedure attempts to give approximate and satisfactory results on performance assessment of framed RC buildings, so this procedure can be used as a first approach before implementing the detailed and complex analyses of existing buildings.

9. Although the results are limited for the framed buildings located in the first seismic zone and on the soil class C, the procedure can easily be applied and extend to the other regions with a few minor modifications.

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