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Seismic response of substandard RC frame buildings in consideration of staircases

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Abstract. During the seismic performance assessment of existing buildings, staircases are generally not taken into account as structural members but as dead load. Staircases, as secondary structural members, not only serve for connecting successive floors but also provide considerable amount of strength and stiffness to the building which can modify its seismic behaviour considerably. In this parametric study, the influence of staircases on the seismic response of substandard RC frame buildings which differ in number of storey and span, presence of staircase and its position has been examined. Modal Analyses and bi-directional Non-Linear Time History Analyses (NLTHA) were conducted to compare several engineering demand parameters (EDPs) such as inter-storey drift ratio (ISDR), floor accelerations, modal properties, member shear forces and plastic hinge distribution. Additionally, short column effect, variation in shear forces of columns that are attached to the staircase slab, failure and deformation in staircase models have also been investigated. As the staircase was considered in the analytical model, a different damage pattern can be developed especially in the structural components close to staircase.

Keywords: substandard; RC frame; staircase; short column; seismic

1. Introduction

Buildings that do not satisfy the provisions of the modern seismic design codes are called the substandard buildings, which constitute a considerable portion of the existing building stock in the underdeveloped and developing countries (Tunaboyu and Avşar 2017). The deficiencies of substandard buildings mainly arise from poor material quality and workmanship, insufficient and improper reinforcement detailing, structural irregularities and inadequacies in structural configuration violating capacity design principles such as strong column-weak beam phenomenon and most importantly, designing the buildings by considering gravity loads only. Such buildings did not perform satisfactorily in the past damaging earthquakes and caused loss of life and property (Yılmaz and Avşar 2013). Due to the lack of a proper audit and qualified workmanship in substandard RC buildings, the mechanical properties of materials are usually unsatisfactory and well below the code requirements. Demolition and reconstruction of the whole substandard building stock may not always be an economical solution. Instead, some of the substandard buildings should be retrofitted to satisfy the seismic performance requirements of codes. The key point for determining whether a building needs retrofitting or not and which parts of a building need to be retrofitted is to conduct a seismic performance assessment analysis with a realistic analysis model for the investigated building.

In the preceding studies, it is known that the realistic modeling of the staircase components in a structural analysis model plays a crucial role in the accuracy of the analysis results. Cosenza *et al*. (2008) performed nonlinear pushover analysis on a typical building frame with different models of staircase, and as a result, they stated that shear failure becomes dominant in the squat column and slabs and precedes the ductile flexural failure. Jiang *et al*. (2012) stated that the staircases act as the first line of defense in earthquakes and therefore yielding and failure takes place in staircases first. Yuan *et al*. (2013) performed elasto-plastic time history analyses for 18 RC structure models with and without staircases. They observed that, for frames away from the staircases, the internal forces of the frame members in the models with staircases are smaller than for the models without staircases. For this reason, they stated that it is inaccurate to design these frames merely based on the internal forces resulting from the models with staircases, because a re-distribution of the internal forces will occur when the staircases are damaged. Hongling *et al*. (2013), analysed building models with and without considering staircase and observed that horizontal bracing effect develops due to the presence of staircase. They concluded that the period of vibration as well as inter-storey displacement decreases, whereas, base shear increases due to staircase. Moreover, it was also reported that the location of the staircase can induce torsion effects and change the internal force distribution. Other than the analytical studies, Li and Mosalam (2013) presented the effect of staircase on the building response based on the site investigations conducted after the 2008 Wenchuan Earthquake. The

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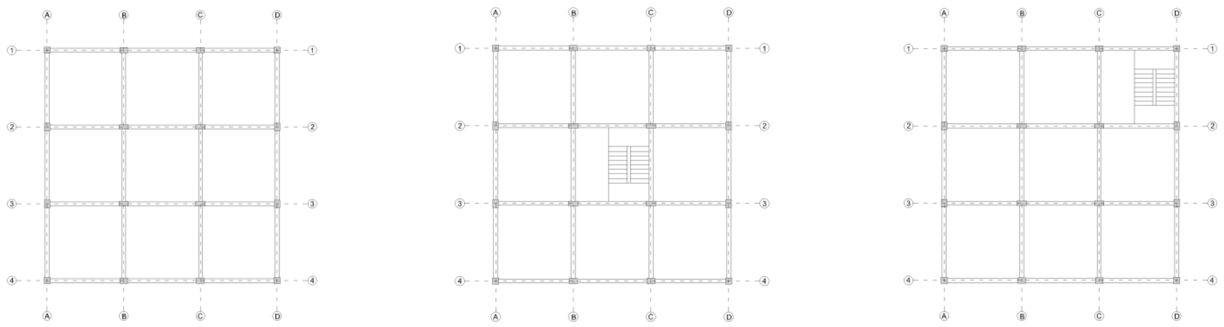
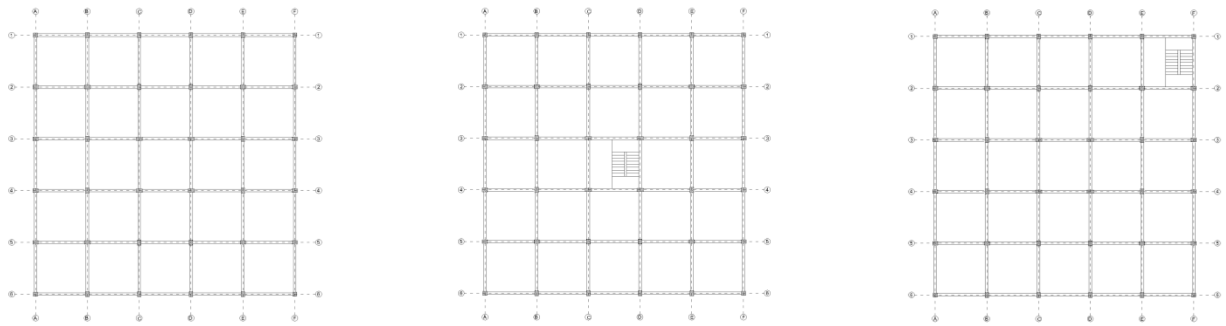


(a) (b) (c)

Fig. 1 Storey layout of 3 spanned (a) non-staircased (b) centric staircased (c) eccentric staircased models



(a) (b) (c)

Fig. 2 Storey layout of 5 spanned (a) non-staircased (b) centric staircased (c) eccentric staircased models

authors emphasized that, unexpected seismic response occurs when the interaction between the staircase and the primary structural system was not taken into account during the design stage. Damages observed in the main structural components as in the form of short columns and short beams which were caused by neglected interaction between primary load carrying members and staircases. Xu and Li (2012), Singh and Choudhury (2012), Tegos *et al*. (2013), Zaid *et al*. (2013), Cao *et al*. (2014) remarked that staircases have an impact on the distribution of the internal forces and may cause an increased damage in the landing beams and columns. Onkar *et al*. (2015) modelled six storey RC buildings having varying concrete quality and designed them without considering staircase. The building models were subjected to non-linear pushover analysis and a superior ductile performance was achieved. After the inclusion of staircases, it has been observed that the superior performance of the building models has been drastically reduced. They concluded that ignoring the contribution of staircase in structural modeling and design can lead to excessive seismic damage compared to non-staircased model and even collapse can take place under a seismic event.

When compared with the previous studies, in this paper, it is the first time a parametric study was conducted on substandard RC building models with and without staircases. The main contribution of this study is to underline the effects of staircases on the seismic behaviour of substandard RC buildings especially with the emphasis on the formation of short columns. By this way, the necessity of considering staircases in the analytical model of substandard RC buildings will be highlighted in order to reproduce their actual seismic response required to achieve a realistic seismic performance assessment results for such seismically vulnerable structures. Substandard models with three different number of stories and two different number of spans were designed without considering staircases and earthquake loads. In order to obtain a more realistic substandard building action, it was meant to stimulate strong beam weak column phenomenon. To do so, relatively stronger beams were considered relative to the columns. As the first step, analytical models of all buildings were generated in SAP2000, (2016) as structural frame system without any staircases. Next, staircase components were included in the analytical models with the consideration of different staircase locations, namely centric and eccentric. Modal analyses and nonlinear time-history analyses were performed under selected and scaled recorded ground motion records for each of the generated analytical models. The analyses results were used to compare several engineering demand parameters (EDPs) such as inter-storey drift ratio (ISDR), floor accelerations, modal properties, member shear forces and plastic hinge distribution. Additionally, short column effect, variation in shear forces of columns that are attached to the staircase slab, failure and deformation in staircase models have also been investigated.

2. Description of buildings

A parametric study was conducted to investigate the effect of staircases on structural configurations by varying the number of spans and stories of RC framed buildings.

Table 1 Column cross-sectional and reinforcement details of the substandard frames

Building Nomenclature	Cross-Sectional Dimensions (cm) (H/B)	Longitudinal Bars	Stirrups (cm)
3×3×3	35/40	10Ø 14	Ø 8/20
	45/35	10Ø 14	Ø 8/20
	50/35	12Ø 14	Ø 8/20
3×5×5	35/40	10Ø 14	Ø 8/20
	45/35	10Ø 14	Ø 8/20
	50/35	12Ø 14	Ø 8/20
5×3×3	40/40	12Ø 14	Ø 8/20
	35/50	12Ø 14	Ø 8/20
	60/35	12Ø 14	Ø 8/20
5×5×5	40/40	12Ø 14	Ø 8/20
	35/50	12Ø 14	Ø 8/20
	60/35	14Ø 14	Ø 8/20
8×3×3	45/45	12Ø 14	Ø 8/20
	40/60	16Ø 14	Ø 8/20
	75/40	20Ø 14	Ø 8/20
8×5×5	45/45	12Ø 14	Ø 8/20
	40/60	16Ø 14	Ø 8/20
	75/40	20Ø 14	Ø 8/20

For this purpose, RC frames were designed by considering 2 different span numbers (3 and 5 spans) and 3 different storey numbers (3, 5, and 8 stories). Each span length is 5 meters and each storey height is 3 meters in all models. In addition, presence of staircase as well as its position was also considered as a variable in the parametric study. The storey layout plans of the 3-spanned and 5-spanned models are given in Fig. 1 and Fig. 2, respectively. All the beams were assumed to have a height of 60cm and width of 25 cm with 5Ø 14 (7.69 cm^2) top reinforcement and 3Ø 12 (3.39 cm^2) bottom reinforcement. The cross-sectional and reinforcement details of the column members of the substandard RC frames employed in this parametric study is presented in Table 1. Even though geometrical irregularity is one of the most common types of structural irregularities, the geometry of these layout plans were determined to be regular in order to highlight the effect of staircases on substandard structures by eliminating the effects caused by geometrical irregularity. Dog-legged staircase, which is one of the most common staircase types in RC buildings, having landings at each floor as well as at the mid-height of each storey was employed in the analytical models. The width of the inclined flights and landings are 1.2 meters and their thickness is 12 centimeters. 7Ø 14 longitudinal reinforcements were considered along the flight running direction together with Ø 8 transverse reinforcement with a spacing of 150 mm.

The cross-sectional properties of columns and beams such as geometric dimensions and reinforcement details, which are described in a statistical study conducted by Inel *et al*. (2010) on structural properties of existing low and mid-rise reinforced concrete building stock in Turkey, were adopted in the analytical model. The longitudinal reinforcement percentage of the columns inspected by Inel *et al*. (2010) are approximately 1% which is the percentage

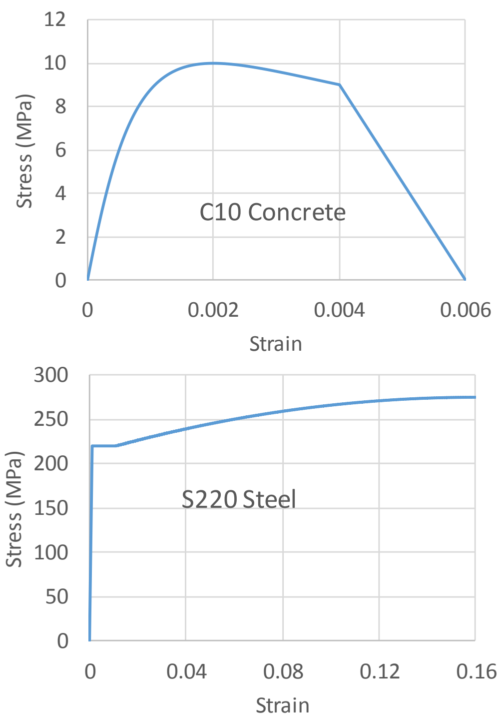
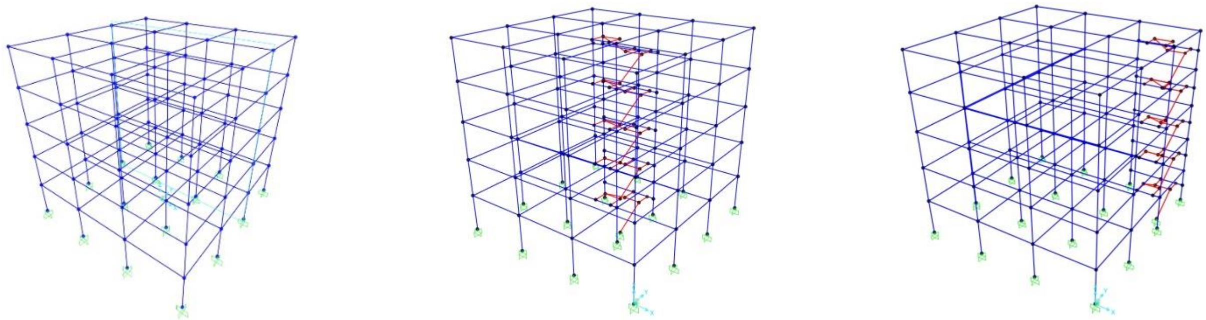


Fig. 3 Material models employed for C10 concrete and S220 reinforcement steel

used in the columns of this study. The confinement bar spacing of RC sections was kept constant and considered as 20 cm which is nonconforming to code-based earthquake resistant detailing and is also well within the statistical results for confinement bar spacing as reported by Inel *et al.* (2010). Additionally, Bal *et al.* (2008) conducted a statistical study on detailed assessment of structural characteristics of Turkish RC building stock for loss assessment models. Based on the outcomes of this study, poor material properties were selected for the investigated substandard buildings in the present study. Therefore, mechanical properties of C10 grade concrete and S220 grade reinforcement steel were employed in the definition of material properties and these properties are also within the statistical results of the study presented by Bal *et al*. (2008). It is also worth noting that, Thermou and Psaltakis (2017) employed similar material properties for their study on substandard RC structures. The material models employed for concrete and reinforcement steel to be used in the analytical solution are presented in Fig. 3.

After defining the sectional and material properties in the analytical models of the substandard frames, all models were analyzed under gravity loads only and sections were designed by considering minimum limitations enforced by TS 500 (2000), which defines the requirements for design and construction of reinforced concrete structures.

The building nomenclature is specified such that; Storey Number×Span Number (in *X* direction)×Span Number (in *Y* direction)_staircase information (such as non-staircased models are labelled as “non” (e.g., 5×3×3_non), centric staircased models are labelled as “cent” (e.g., 8×5×5_cent) and eccentric staircased models are labelled as “ecc” (e.g., 3×3×3_ecc)).



(a) (b) (c)

Fig. 4 Columns, beams and staircase elements were modelled as frame members; (a) 5×3×3_non, (b) 5×3×3_cent and (c) 5×3×3_cor

3. Analytical modelling

The columns, beams and staircase elements were modelled as frame members as shown in Fig. 4. Floor slabs were not modelled, yet the dead load and live load acting on them have been taken into account as triangular tributary loads on the supporting beams. In addition to the self-weight of the structural components, dead load of infill walls (7.5 kN/m), dead load of slabs (5 kN/m^2) and live load (2 kN/m^2) were considered. Additionally, rigid diaphragm constraints were assigned to the nodes of each floor level and to the nodes of each landing slabs of the staircases at the mid-story. A fully restraint support condition was assumed at the bottom of columns and flight of the staircase at the ground storey.

In order to model the non-linear behavior of the RC sections, moment-curvature analyses were conducted in XTRACT (2007) software. The nonlinear material properties and damage limits are defined as per TEC (2007). For the conducted parametric study, instead of considering each local damage such as bar slipping, bar buckling, etc. as outlined by Di Sarno *et al*. (2017), code specified strain based limit states were adopted in the definition of plastic hinges.

For columns, staircase flights and staircase landing beams at the mid-story, flexural P-M2-M3 hinges, for beams flexural M3 hinges were assigned to their both ends. Owing to the rigid diaphragm constraint at the landing slabs of the staircases at the mid-story, flexural M3 hinges were assigned to the landing beams. For this purpose, lumped plasticity was assigned at the maximum moment regions of the RC components for the nonlinear analyses. A representative force-displacement relationship employed in the plastic hinges is presented in Fig. 5 with specific performance points. Accordingly, points B and C corresponds to the yield and ultimate capacity of the hinge, respectively. Whereas, points D and E are the initial and ultimate displacement capacities after attaining the ultimate strength. Hinge properties of RC components were determined based on their section geometry and material properties of reinforcement steel and confined and unconfined concrete.

In Fig. 5, there are also three different performance points defined between points B and C which represent the damage limits according to TEC (2007). These points are

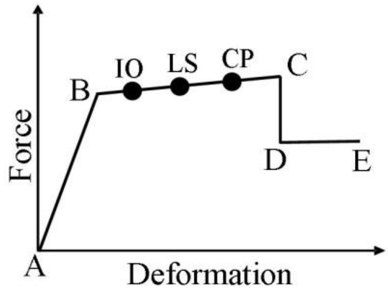


Fig. 5 Hinge properties, force vs. displacement curve

IO (immediate occupancy), LS (life safety) and CP (collapse prevention). The damage limits of these performance points were specified based on the equations (see Eqs. (1) to (3)) given in TEC (2007);

a) For Immediate Occupancy (IO), upper bounds of the unconfined concrete compressive strain (ε_{cu}) in the outermost fiber of the RC section and the reinforcement steel strain (ε_s)

$$(\varepsilon_{cu})_{IO} = 0.0035 \quad ; \quad (\varepsilon_s)_{IO} = 0.010 \tag{1}$$

(b) For Life Safety (LS), upper bounds of the confined concrete strain (ε_{cg}) in the outermost concrete fiber within the hoop and the reinforcement steel strain

$$(\varepsilon_{cg})_{LS} = 0.0035 + 0.01(\rho_s/\rho_{sm}) \leq 0.0135; \quad (\varepsilon_s)_{LS} = 0.040 \tag{2}$$

(c) For Collapse Prevention (CP), upper bounds of the confined concrete strain (ε_{cg}) in the outermost concrete fiber within the hoop and the reinforcement steel strain

$$(\varepsilon_{cg})_{CP} = 0.004 + 0.014(\rho_s/\rho_{sm}) \leq 0.018; \quad (\varepsilon_s)_{CP} = 0.060 \tag{3}$$

In Eqs. (2)-(3), ρ_s is the available amount of code compliant hoop and ρ_{sm} is the minimum required code compliant hoop in the RC section. Since transverse reinforcements of existing substandard RC sections have generally 90-degree hooks (Yurdakul and Avsar 2016), which should be 135-degree as per TEC (2007), ρ_s is assigned to be zero for all RC sections.

After performing cross-sectional analysis in XTRACT, (2007) the corresponding moment-curvature curves are calculated for each RC section. The obtained moment-curvature data was idealized by a bilinear approximation

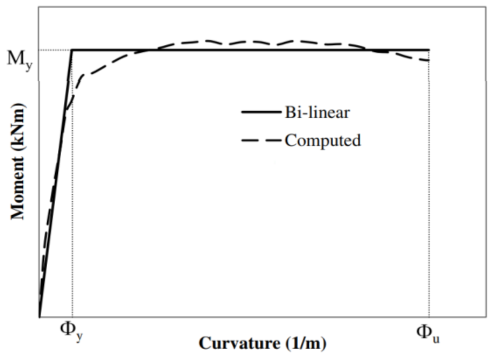


Fig. 6 Bilinear representation of moment-curvature data

according to FEMA 356 as shown in Fig. 6. The yield and ultimate rotations to be used in the bilinear moment-rotation relation were determined by Eqs. (4) and (5) respectively.

$$\theta_y = \frac{\Phi_y x L}{3} \tag{4}$$

$$\theta_u = \theta_y + \left(\Phi_u - \Phi_y\right) x L_p \tag{5}$$

Φ_y and Φ_u are the yield and ultimate curvature, respectively, as shown in Fig. 6. L is the distance from the column critical section of the plastic hinge to the point of contraflexure, and L_p is the plastic hinge length, which is the half-length of the cross-sectional dimension in the loading direction as per TEC (2007).

Other than the deformation controlled flexural hinges, force-controlled shear hinges were assigned at the mid-point of all frames for the purpose of considering possible brittle type of shear damage. Especially, the short columns due to the staircase landings at the mid stories became the most vulnerable members for shear failure. Although the hysteretic model adopted for the assigned shear hinges can affect the seismic response as emphasized by Del Vecchio *et al*. (2015), in order to minimize the computational demand for the conducted parametric study, force-controlled shear hinges without any hysteretic behavior were adopted to identify whether the corresponding member attained its shear capacity or not. The shear capacities of all RC components were calculated in accordance with Eq. (6) based on the Turkish Standards for Reinforced Concrete Design (TS-500) and assigned to the corresponding force-controlled shear hinges.

$$V_r = 0.52\, f_{ct} b_w\, d\, (1 + \gamma\, N_d / A_c) + \frac{A_{sw}}{s} f_{yw}\, d \tag{6}$$

Where, f_{ct} is tensile strength of concrete, b_w and d are the width and depth of the RC section, respectively, N_d is the design axial load, γ is a factor for the sense of axial load (0.07 for compression and -0.3 for tension), A_c is the cross-sectional area of the member, A_{sw} is the amount of transverse reinforcement, s is the spacing between transverse reinforcement bars, f_{ywd} is the yield strength of transverse reinforcement.

4. Ground motion selection and scaling

A number of representative earthquake ground motion records need to be selected and scaled to perform a series of bi-directional NLTHA. In accordance with TEC (2007), it is stated that at least three or seven earthquake ground motions shall be selected with certain limitations. If three ground motion pairs are used in the analysis, the maximum response will be considered as a final result. However, if seven or more records are used, the average value of the response parameter will be considered. The requirements given below should be satisfied during the selection and scaling process of earthquake ground motions as per TEC (2007);

- Duration of the strong motion shall be longer than 5 times the fundamental period of the building and at the same time longer than 15 seconds.
- Average of the spectral acceleration of the selected ground motions at zero period should be more than $A_o g$, where A_o is the effective ground motion acceleration coefficient for the corresponding seismic zone and g is the gravitational acceleration.
- Average of the spectral accelerations of the selected acceleration records for 5% damping ratio shall not be less than 90% of the elastic design acceleration

Table 2 General characteristics of the selected earthquake ground motion pairs (PEER)

Name	Year	Station	*Mw*	*R* (km)	Component	PGA (g)	PGV (cm/s)	PGD (cm)	*Vs* (m/s)	Scale Factor
Kocaeli	1999	Düzce	7.5	13.6	180	0.312	58.9	44.2	276.0	1
					270	0.358	46.4	17.6		
Morgan Hill	1984	Gilroy #3	6.2	13.0	0	0.194	11.2	2.3	349.9	1.3
					90	0.200	12.7	3.4		
Düzce	1999	Bolu	7.1	12.0	0	0.728	56.4	23.1	326.0	1.3
					90	0.822	62.1	13.6		
Landers	1992	North Palm Springs	7.3	26.8	0	0.136	11.0	5.0	345.4	1
					90	0.134	14.5	5.6		
Imperial Valley	1979	Westmorland Fire	6.5	15.2	90	0.074	21.3	16.6	193.7	1
					180	0.110	21.9	10.0		
Superstition	1987	Parachute Test site	6.5	0.9	225	0.455	112.1	54.0	348.7	1.3
					315	0.377	43.9	15.3		
Kobe	1995	Shin Osaka	6.9	19.1	0	0.243	37.8	8.6	256.0	1
					90	0.212	27.9	7.6		

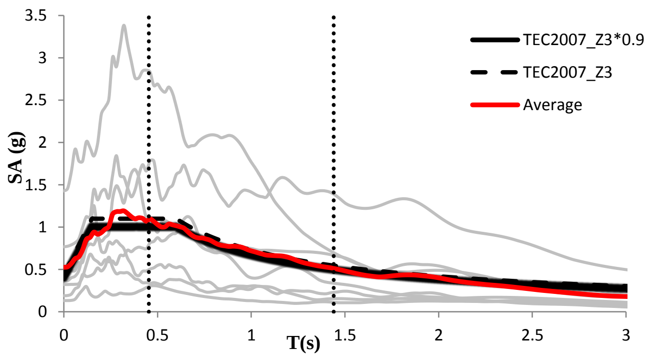


Fig. 7 SRSS of response spectrum curves of the selected earthquake ground motions and design spectrum

spectrum, in the period range between $0.2T_1$ and $2T_1$ where T_1 is the fundamental period of the building in the earthquake direction considered.

According to the above-mentioned criteria in TEC 2007, seven bi-directional earthquake ground motion records were selected from strong ground motion database of PEER and then scaled. Therefore, the average values of the analyses results of the seven earthquake ground motion records were considered. Characteristics of the selected ground motions and employed scaling factors are presented in Table 2. The square root of the sum of squares (SRSS) of the response spectra of the scaled ground motions, their average and design spectrum curve for Z3 soil class are given in Fig. 7.

5. Results and discussion

In this study, even if the analyses were conducted in biaxial direction, the main scope is to examine the effect of staircases in the direction which is parallel to flight running direction of the stairs, and for this study, it corresponds to *Y* direction of the global axes as shown in Fig. 4. *X* direction is out of the scope of this study as Yuan *et al*. (2013) proposed in their study that, the effect of staircases can be neglected in the direction perpendicular to the flight running direction since the increase of the lateral stiffness in this direction is less influential compared to the Y-direction.

5.1 Modal analysis results

Modal analyses were conducted to investigate the effect of staircases on the modal properties of the RC buildings. The results and the differences between non-staircased and staircased (both centric and eccentric) cases were presented in Table 3 in terms of modal participating mass ratio (MPMR) of the first mode and first mode period (The first mode is the translational mode in *Y* direction.).

From the modal analysis results, it can be inferred that as the staircases are included to the mathematical model, MPMR of the mode-1 (translation in *Y*-direction) decreases especially for the RC buildings with more spans.

Eccentrically placed staircases have a greater influence in the reduction of MPMRs. This can be attributed to the increase in the contribution of the torsional mode to the translational displacement. On the other hand, staircases

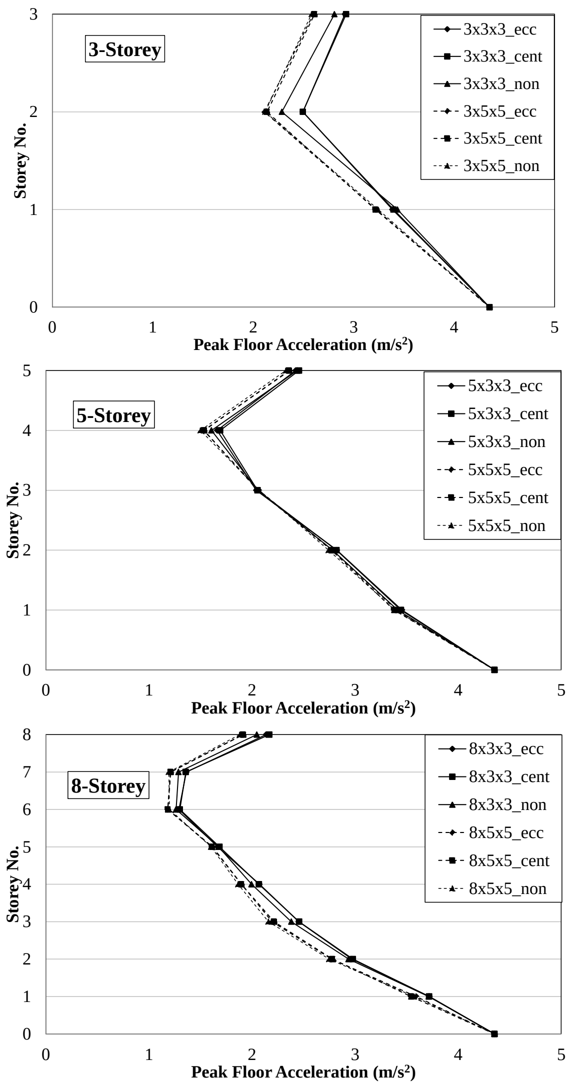


Fig. 8 Average peak floor accelerations of all models

decreased the natural vibration period of the buildings owing to their local stiffening effect. As the number of spans and stories increase, natural vibration periods of the models increased. Effect of staircases on the first mode vibration period of the buildings is not that much influential as in the case of MPMR. The reduction in the first mode period due to the inclusion of staircase is relatively more for the buildings with less number of stories and span numbers. Eccentrically placed staircases caused less decrement in natural vibration period with respect to centric staircases.

5.2 Nonlinear time-history analysis results

5.2.1 Peak floor accelerations

The peak floor acceleration is an important engineering demand parameter for the comfort of the residents of the buildings. Also, it is a significant parameter in the determination of seismic force imposed on the non-structural components on the floors.

The plotted results shown in Fig. 8 were obtained from SRSS of both *X* and *Y* directions and are the average results of all selected earthquake ground motions.

Table 3 Modal participating mass ratios (MPMR) and first mode vibration periods (T_1) of the models

Span No.	Non	Eccentric		Centric		Non	Eccentric		Centric	
	MPMR	MPMR	%Diff.	MPMR	%Diff.	T_1 (s)	T_1 (s)	%Diff.	T_1 (s)	%Diff.
3 Storey Models										
3×3	0.879	0.820	6.7	0.871	0.9	0.642	0.612	4.7	0.610	5.0
5×5	0.878	0.695	20.8	0.867	1.2	0.667	0.657	1.5	0.652	2.2
5 Storey Models										
3×3	0.844	0.818	3.1	0.841	0.3	1.024	0.984	3.9	0.973	5.0
5×5	0.842	0.672	20.2	0.830	1.5	1.028	1.017	1.1	1.008	1.9
8 Storey Models										
3×3	0.816	0.796	2.4	0.815	0.2	1.397	1.360	2.6	1.350	3.4
5×5	0.817	0.742	9.2	0.815	0.2	1.440	1.427	0.90	1.419	1.5

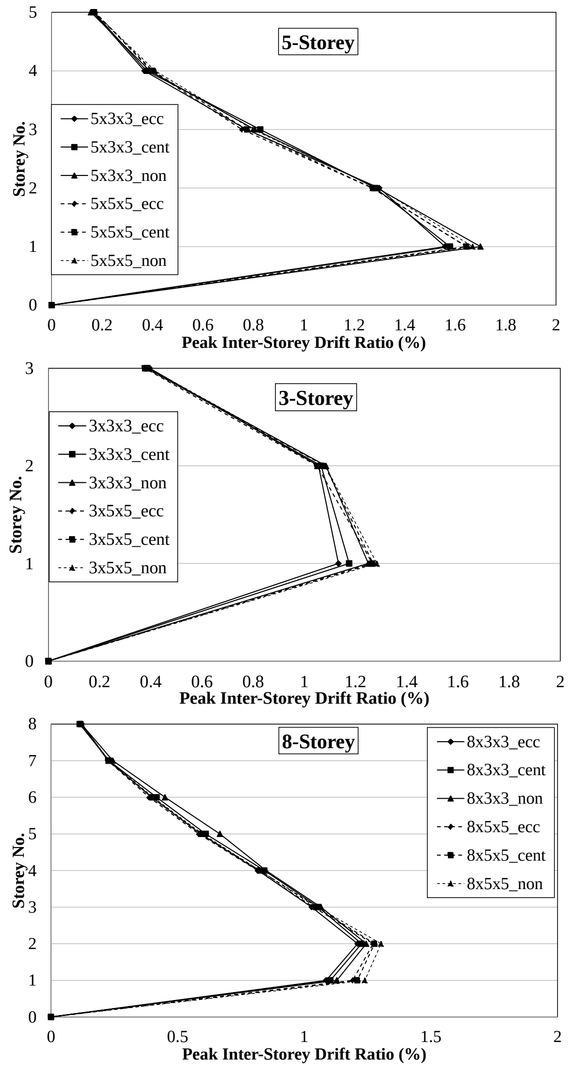


Fig. 9 Average ISDR of all models

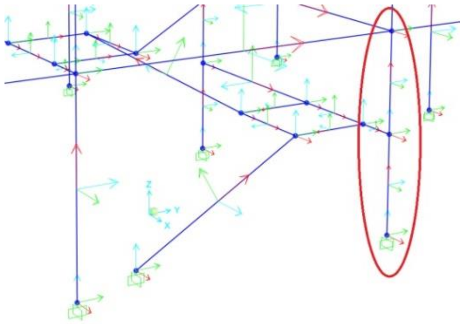
It is important to emphasize that, the variation in the peak floor acceleration is more pronounced for the taller buildings. 5-spanned buildings were exposed to relatively less floor accelerations than their 3-spanned counterparts having the same storey number. It was also observed that the effect of the staircase and its position in the building did not influence the average results of the peak floor acceleration considerably.

5.2.2 Inter-storey drift ratios (ISDR)

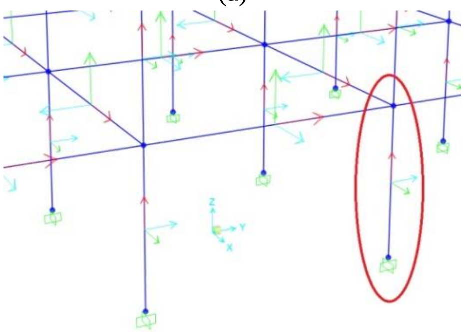
Inter-storey drift ratio (ISDR) is an important indicator for seismic damage of both structural and non-structural components. It is the ratio of the relative displacement between the successive stories with respect to the corresponding storey height. In order to limit the secondary effects due to large displacements and to protect the secondary components in the building, modern seismic testing codes limit the max ISDR. Inter-storey drift ratio of each model was determined by dividing the relative drift of two consecutive floors with the storey height. In this study, ISDR has only been calculated for Y direction which is parallel to staircase flight running direction. In Fig. 9, the average ISDR results were presented. Similar to the peak floor acceleration results, the effect of staircase on the global response of RC buildings in terms of ISDR is negligible when the average results are compared (Fig. 9). Although the difference is insignificant, it can be indicated that the non-staircased models have the largest average ISDR values compared to the staircased counterparts, which can be attributed to increased stiffness induced by the staircases. Since the imposed seismic demands on the structural components of lower stories are greater compared to the higher stories, max ISDR values were obtained in the lower stories.

5.2.3 Shear force demands in staircase supporting columns

In this section, shear force demands in the columns that are supporting the staircase were investigated for both 2-2 and 3-3 local axes of the columns in centric staircased and non staircased cases. The reference columns chosen for analyses are shown in Fig. 10. In this figure, blue arrows stand for 3-3 local axis which is parallel to flight running direction, while green arrows stand for 2-2 local axis. In Fig. 11, the shear diagrams of the core frame of a staircased model (on the left) and of a non-staircased model (on the right) are presented. From the figure, it can be seen that the presence of a staircase causes a leap in the shear force demand at the intersection of staircase components and columns. The sudden increase in the column shear demand can be attributed to the “short column” phenomenon and this may cause the exceedance of the shear force capacity of the corresponding column. When the shear force capacity of a member is exceeded, the shear hinge assigned on that



(a)



(b)

Fig. 10 The selected reference column of (a) centric staircased models (b) non-staircased models

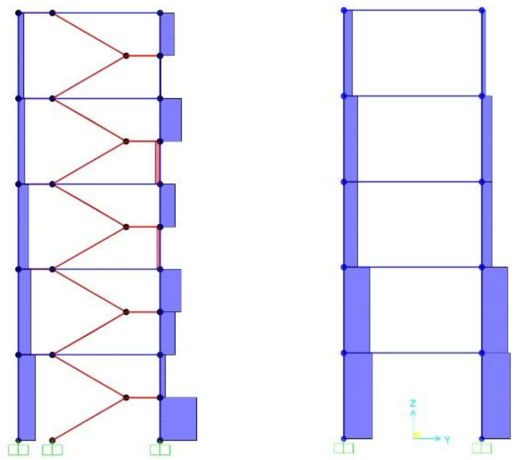


Fig. 11 Distribution of shear force along short-column (on the left) and reference column (on the right) during t=5.58 s of Superstition Hills Earthquake

member will be activated for identifying the shear failure. Based on the NLTHA results, the exceedance of shear force carrying capacity has only been occurred for staircase supporting columns. For that reason, shear force demands of the reference columns which are shown in Fig. 10 were compared in Fig. 12. The continuous red lines in Fig. 12 represent the shear force capacity of the reference columns which was calculated in accordance with Eq. (4) as per TS500. The dotted brown lines in Fig. 12 corresponds the shear capacity obtained by Eq. (5), which is the shear strength model proposed by ACI 318. Del Vecchio *et al.* (2017) concluded that the ACI318 model can be used to assess the shear strength of existing RC members with a significant underestimation. However, when the variability in the material and geometrical properties of the existing substandard RC buildings are of concern, the underestimated shear capacity of RC components are considered to be on the conservative side.

$$V_r = 0.17\left(1 + \frac{N_d}{14A_c}\right)\sqrt{f_c}b_w d + \frac{A_{sw}}{s} f_{yw}\, d \qquad (5)$$

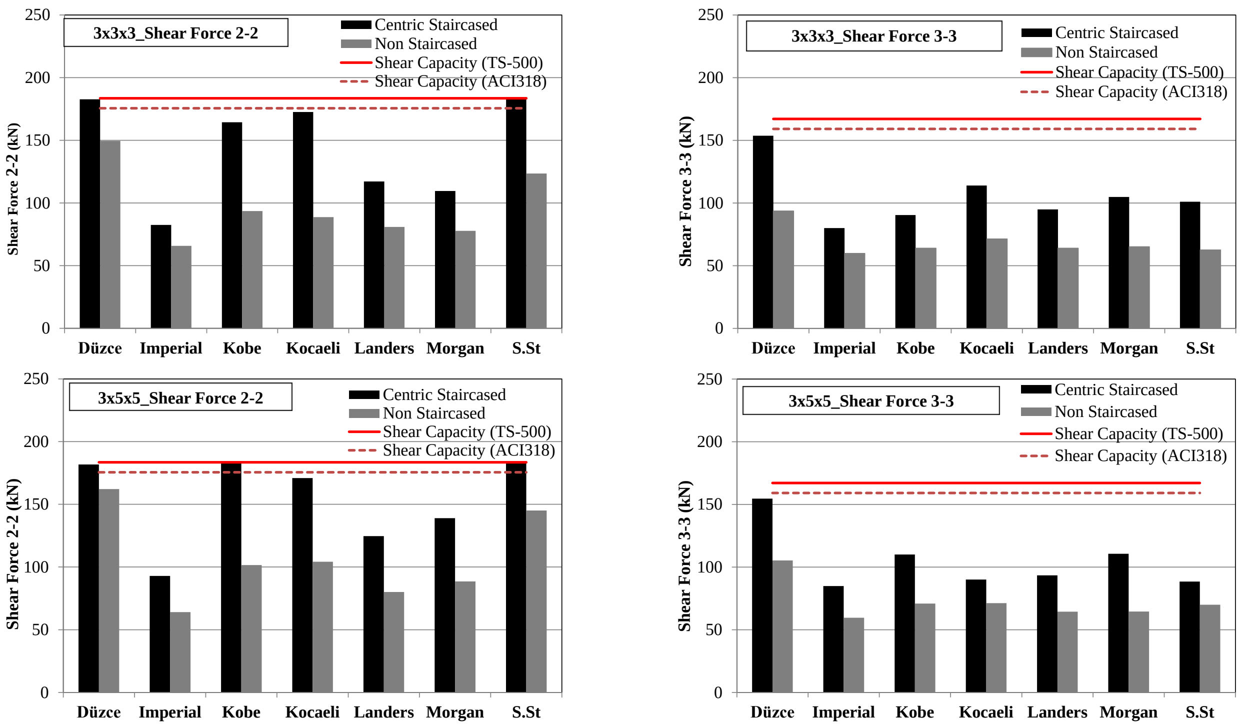


Fig. 12 Comparison of shear force demands and capacity of the reference columns

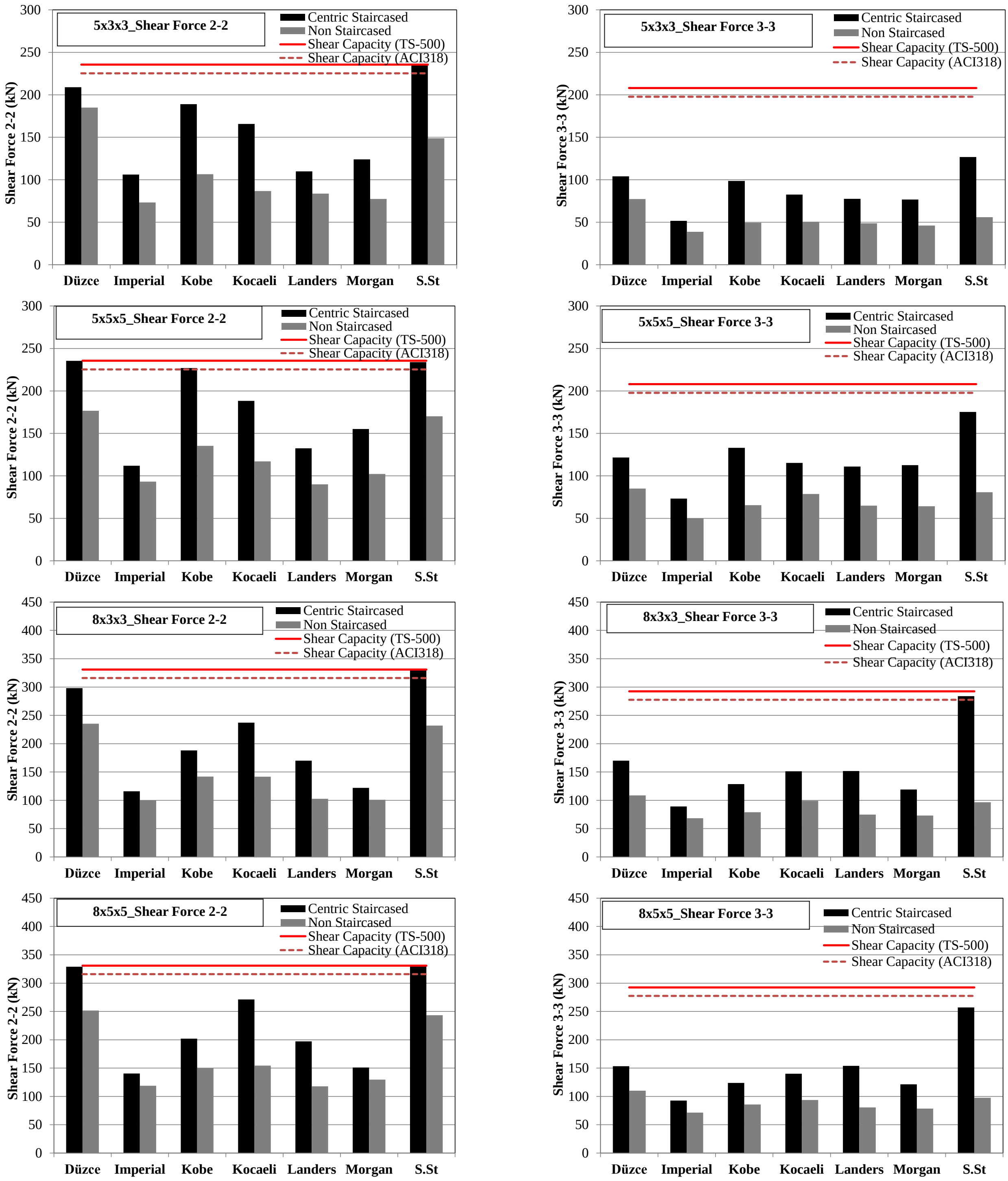


Fig. 12 Continued

Based on the shear force demand results, the study showed that, for each earthquake ground motion analysis, the selected column is exposed to higher shear demands due to the presence of staircase which was an expected outcome due to the formation of short column with the presence of staircase. It was observed that staircases can cause column shear failure, which can even lead to total collapse of the building. Moreover, consideration of staircase in the analytical models led to the exceedance of shear capacity of the reference columns of all substandard models and this was observed especially for Superstition Hills and Düzce Earthquake records, which have imposed more seismic demands on the buildings. This outcome can be attributed to their high energy content. Their relatively higher PGV values are the indicators for their high energy content.

5.2.4 Effect of staircase and its position on the base storey shear force

Base shear force is the total shear force imposed on the basement columns. Calculating base shear forces gives an

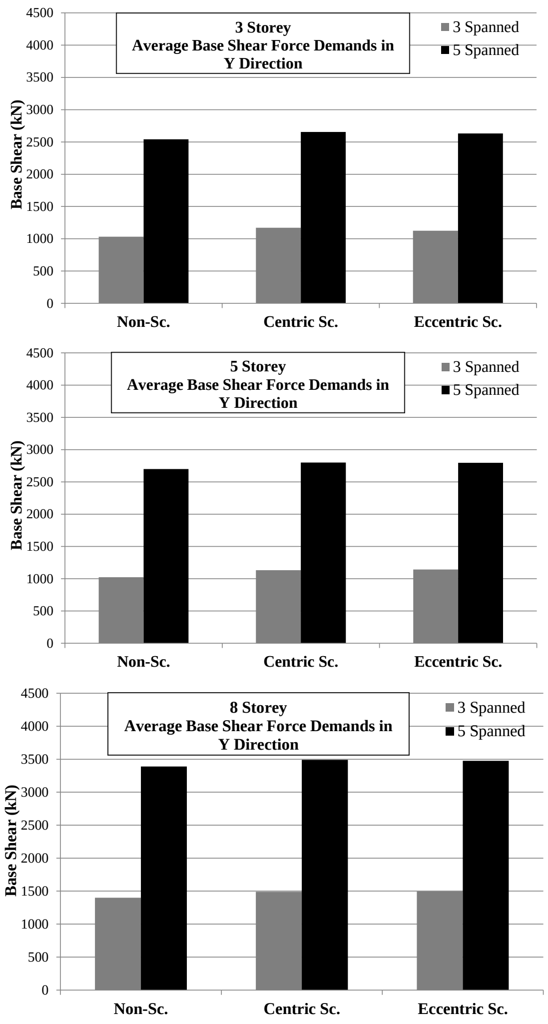


Fig. 13 Base shear force demands of all models

insight about the total seismic force acting on the building. In this study, base shear forces have only been calculated for *Y* direction which is parallel to staircase flight running direction. In Fig. 13, the average base shear force of each model is presented.

Analyses results of the base shear force demands show that inclusion of staircases increases the average base shear forces yet, the increment in the base shear demand is not that much significant. Additionally, the position of staircase has no influence on the base shear force. It can be easily seen that, as the number of spans increases, the base shear force increases as well due to the increased mass of the building. For instance, 5×5 span buildings have more than twice as much base shear demand as 3×3 span buildings have. Finally, the base shear demand increases as the number of storey increases. However, the effect of number of storey on the building base shear is less influential than the effect of number of spans.

5.2.5 Effect of staircase on the plastic hinge distribution

In this section, the distribution of plastic hinges is examined under the Superstition Earthquake ground motion record, which is one of the most damaging earthquakes

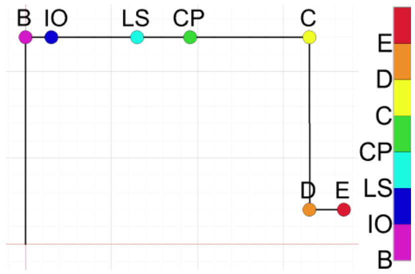


Fig. 14 Performance points on the plastic hinge with a representative force-deformation relationship

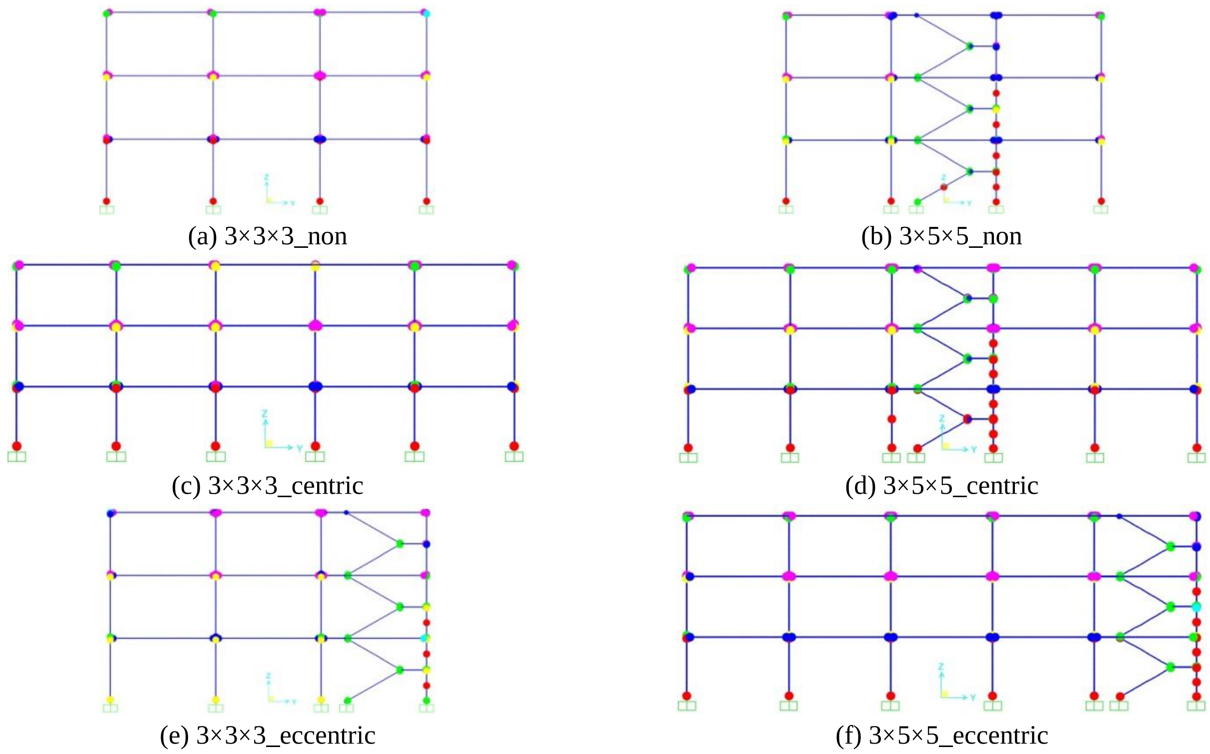
among the other selected earthquakes due to its high spectral acceleration values and high PGV value. Damage state on the plastic hinges is evaluated according to the coloured scale shown in Fig. 14 (on the right). These colours refer to the performance points of a hinge which is shown as an example in Fig. 14 (on the left). By this way, the effect of staircase on the distribution of both flexural and shear hinges is investigated. In the coloured scale, "B" stands for the beginning of inelastic behavior, "IO" stands for "immediate occupancy", "LS" stands for "life safety" and "CP" stands for "collapse prevention" damage limit states.

The distributions of the plastic hinges are presented in Figs. 15-16-17 for the 3-, 5- and 8-story building models, respectively. The plastic hinge distributions were taken from the final state of the nonlinear time-history analyses under Superstition Earthquake ground motion record.

Hinge distributions show that the maximum damage by means of flexural plastic hinges occurred at the bottom ends of the first floor columns of the buildings without staircase. Also, damage level on the columns is higher with respect to the beams of the substandard models as expected due to strong beam-weak column phenomenon. It is obvious that staircase insertion in the analytical models caused shear hinges to develop due to the short column effect by the interaction between mid-floor staircase landing beams and columns. Yet, no shear damage has been observed on non-staircased models. There is a considerable difference between the damage patterns of the staircased and non-staircased models. The columns close to staircase are most likely exposed to an increased level of seismic damage than the other structural components in the building.

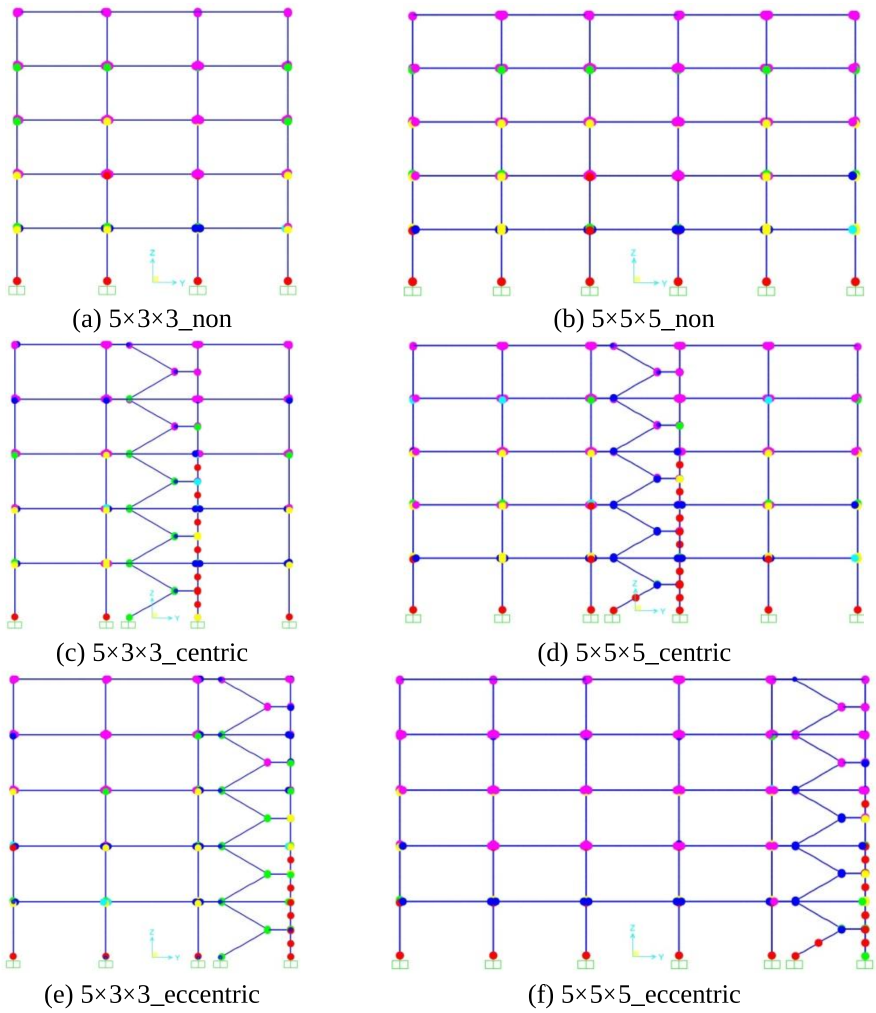
6. Conclusions

In this parametric study, the impact of staircases on the seismic performance of substandard RC buildings was investigated. Specimens were prepared which differ in number of storey, number of spans, presence of staircase as well as its position in the plan. After the section dimensions and reinforcement details were specified to be representative of the substandard RC buildings, their analytical models were developed by SAP2000 in order to examine the impact of staircases in terms of inter-storey drift ratio, floor accelerations, modal properties, member shear forces, base shear force, plastic hinge distribution and formation of short columns. For this purpose, modal



(a) 3×3×3_non
(b) 3×5×5_non
(c) 3×3×3_centric
(d) 3×5×5_centric
(e) 3×3×3_eccentric
(f) 3×5×5_eccentric

Fig. 15 Distribution of plastic hinges on 3 storey models



(a) 5×3×3_non
(b) 5×5×5_non
(c) 5×3×3_centric
(d) 5×5×5_centric
(e) 5×3×3_eccentric
(f) 5×5×5_eccentric

Fig. 16 Distribution of plastic hinges on 5 storey models

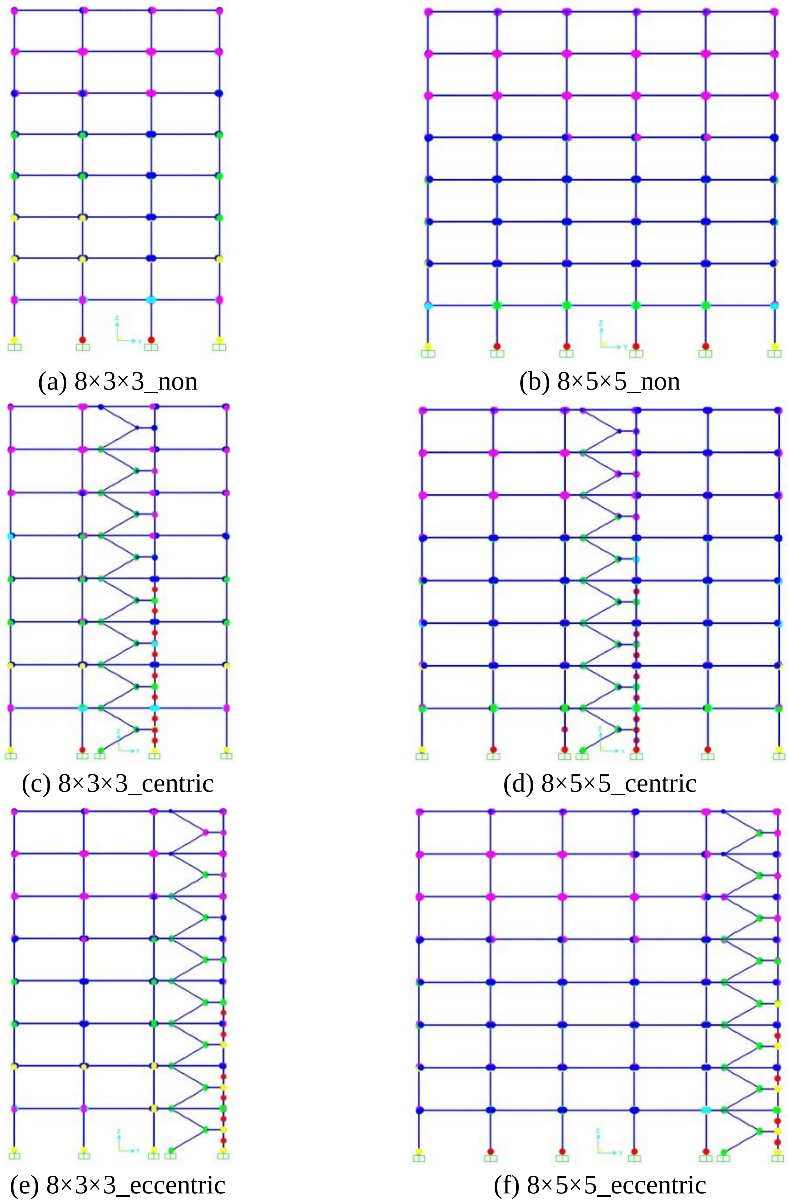


Fig. 17 Distribution of plastic hinges on 8 storey models

analyses and nonlinear time-history analyses were conducted. Displacement controlled flexural hinges and force controlled shear hinges were assigned on the RC members to reproduce their nonlinear response under seismic actions.

According to the data obtained, following conclusions can be drawn;

• Staircases alter the modal properties of the buildings such as modal participating mass ratio (MPMR) and natural vibration period. Variation in MPMR due to the inclusion of staircases is more pronounced compared to the variation in natural vibration period. MPMR decreases due to the structural irregularity induced by staircases and natural vibration period decreases due to local stiffening effect of staircases. Moreover, eccentrically placed staircases have a greater effect on decreasing the MPMR by causing the non-uniform distribution of stiffness in building layout. In addition, eccentrically placed staircases caused less decrement in natural vibration period with respect to centric staircases.

• Inclusion of staircases increases base shear force demand of the substandard RC buildings. Furthermore, their base shear force demand increases proportionally as the buildings get taller and heavier.

• Effect of staircase on the global seismic response of substandard buildings was not that much significant in terms of average peak floor acceleration and ISDR. Although the average ISDR value of non-staircased models are slightly greater than their staircased counterparts, the difference is negligible. However, the effect of staircase was explicitly observed in the base shear force and critical column shear force distribution.

• The presence of staircase has a significant influence on the plastic hinge distribution of the structural components. Especially, more plastic hinges were developed in the structural members close to staircase. Besides, only the flexural damage was observed in the non-staircased models, but shear failure was also observed in the staircased models, especially in the columns close to staircase due to the formation of short columns.

• Presence of staircases can change the damage pattern considerably for the substandard RC buildings under seismic actions. Therefore, staircase components should be taken into account to reproduce their seismic response more realistically, especially in the seismic performance assessment calculations.

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