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Time-History Analysis of Reinforced Concrete Frame Buildings with Soft Storeys

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Abstract This research study investigates the change in dynamic characteristics of reinforced concrete moment-resisting frame buildings without and with fully infill walls. In addition, building models with partially infill walls have also been investigated. A set of different building models have been developed to perform the analysis as (1) bare frame (without infill walls), (2) frame with fully infill walls, (3) frame models with infill panels and soft storey located at base level, 3rd storey level, 6th storey level, 9th storey level, and 12th storey level. The equivalent diagonal strut method has been utilized in order to account for the stiffness and structural action of the masonry infill panels. Dynamic time history, using two ground motion records from near and far-fault regions, has been used to perform the seismic analysis of the considered model configurations. The selected two ground motion records have been scaled to meet the expected peak ground acceleration in Cairo zone. The two ground excitations are applied separately in two orthogonal directions. The structural software package ETABS has been used in developing the building models and performing the simulation analysis. Some selected numerical simulation results in terms of storey shear forces, lateral deflections, interstory drift ratios and overturning moments at each storey level are obtained for all the considered configurations and presented in comparative way. Based on the obtained time-history results, it has been found that the dynamic storey responses for bare frame model significantly differ from the responses obtained for both fully infill and partially infill frame models.

Keywords Time-history analysis · Masonry infill walls · Single diagonal strut · Soft storey

1 Introduction

A large number of moment-resisting frame buildings have been or are being constructed. In addition, more are being planned to be constructed all over the world. These types of buildings have various social and functional uses such as parking garages, reception lobbies and any other open air spaces which have no infill masonry walls and called soft or weak storey. Although multi-storey reinforced concrete buildings with open spaces are highly vulnerable to collapse under the effect of lateral earthquake loads, they have become an unavoidable feature for the most of the newly constructed reinforced concrete framed buildings. This may be due to the essential needs of such open spaces particularly in big cities with limitations in land availability. Figure 1 shows a cross section through a frame building with soft storey. These types of buildings are generally designed considering walls as non-structural elements without regard to the masonry infill wall action.

Some of the modern seismic codes and the conventional practices as well neglect the effect of masonry infill wall based on the assumption that it may lead to some conservative results [1]. Some other codes (e.g. IS 1893–2002 [2], IBC-2003 [3]) provide a factor to magnify the induced straining

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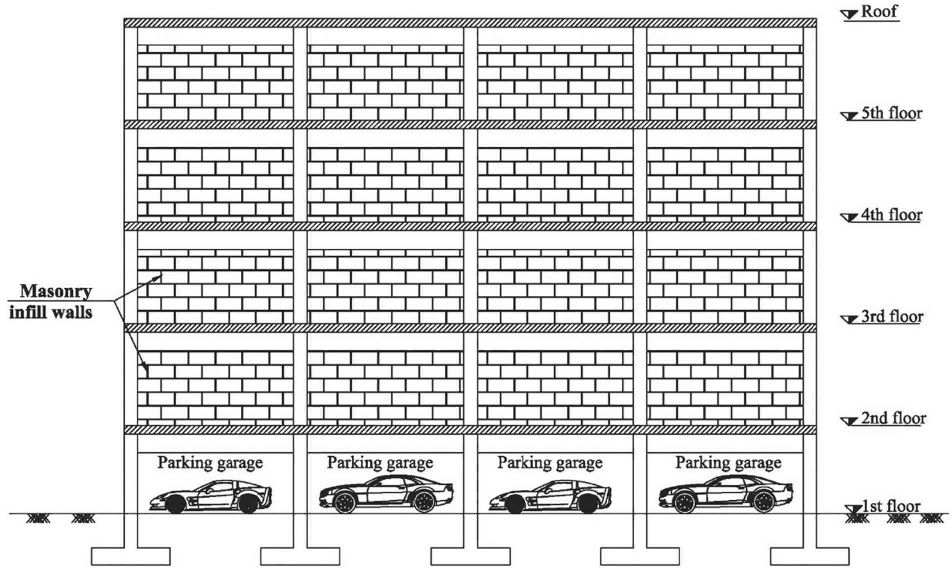
Fig. 1 Cross section through a frame building with soft storey



actions in terms of bending moments and shear forces. In order to propose a magnification factor for the induced shear at base for a building with soft storey, Scarlet [4] performed an analysis based on two extreme situations in which uniform structures as well as rigid structures with soft storey have been used.

Most of structural design practices in Egypt treat masonry infill walls as non-structural elements, and consequently the contribution from stiffness and strength of such elements to the building is neglected during the analysis. Actually, the presence of such infill walls significantly changes the frame action behaviour and results in changing lateral load transfer mechanism.

A building with an open storey or sometimes called soft storey is the one that has a stiffness discontinuity due to the significant flexibility of the open storey compared with the adjacent storeys. Several codes defined the stiffness discontinuity in a building storey as the one with lateral stiffness less than 70% of the lateral stiffness of the storey above or less than 80% of the average stiffness of the three storeys above [3,5,6].

Dolsek and Fajfar [7] attempted to explain the reason behind occurrence of soft storey effect in uniformly infilled frames as well as when this phenomenon occurs. Structural models designed according to the Eurocode 2008 together with structures designed with limited strength and ductility according to previous codes have been utilized to perform the analysis. In 2002, Demir and Sivri [8] studied the seismic response of reinforced concrete structures with different configurations of masonry infill in order to show the effects of non-structural masonry infill walls on the induced building's response. The results of the conducted elastic analysis demonstrated that the presence of non-structural masonry infill significantly modifies the overall seismic response of the studied framed building structures.

Performance of a number of configurations of multi-storey reinforced concrete frame models as bare frame, masonry infill and masonry infill with soft storey at ground floor under earthquake loads has been investigated [9]. Kabir and Shadan [10] developed a finite element model of a 3D-panel building system using the ABAQUS software package to investigate the effect of presence of a soft storey on seismic performance of such building systems. Results verified numerically that the 3D-panel system has considerable resistance under the applied ground motion records.

Several methods of analysis in terms of linear and nonlinear have been utilized to deeply understand the behaviour of building structures with masonry infill actions. Hirde and Ganga [11] employed the pushover analysis to discuss the seismic performance of a twenty storey reinforced concrete building with soft storey located at different levels along with a soft storey at ground level. The conducted study indicated the formation of plastic hinges in columns at ground open storey. From safe design point of view, this is not acceptable criterion. Karwar and Londhe [12] conducted a comparative study in order to investigate the seismic response behaviour of reinforced concrete framed building models with and without masonry infill action. The nonlinear static analysis has been used to perform the response analysis in terms of shear at base, displacement and the performance point. Setia and Sharma [13] employed the equivalent static analysis to perform response evaluation of reinforced concrete buildings with soft storey. Five different models with shear wall in x-direction as well as in z-direction used in the analysis and developed by the structural software package namely STAA Pro.

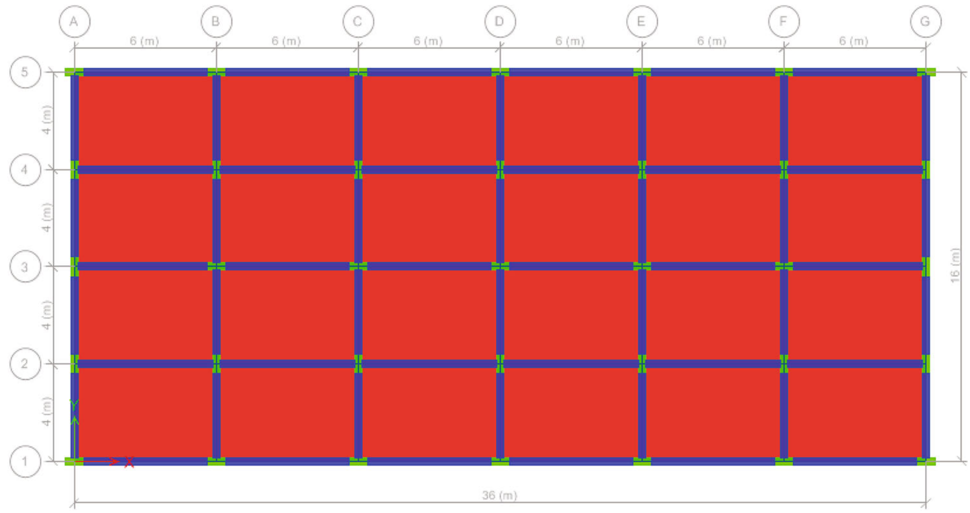


Fig. 2 Typical plan floor of six bays four bays thirteen storeys frame building

The effect of seismic level on the response of masonry infilled structures subjected to severe earthquake records has been investigated [14]. In addition, the effect of existence of a soft storey on the design strategy has also been considered in the analysis. Agrawal [15] analysed the performance of masonry infilled building structures with and without openings. The effect of variation of opening percentage on the lateral stiffness of infilled building model has been analysed as well. A trial to investigate the seismic response of base-isolated building with soft storey has been carried out by Pinarbasi and Konstantinidis [16]. The effect of soft storey flexibility on the corresponding building with fixed-base has also been investigated and compared with the isolated-base building case.

In this paper, the dynamic response time-history of reinforced concrete moment-resisting frame buildings to near-fault records of El Centro (1940) and far-fault records of Loma Prieta (1989) has been considered. Several building models, including fully infill frame model and frame models with infill panels and soft storey at base level, 3rd storey level, 6th storey level, 9th storey level and 12th level, have been developed for analysis purposes. Since buildings without inclusion of masonry infill action can behave differently than buildings with inclusion of such action, a bare frame building model has also been considered in the analysis.

2 Building Models

In order to seismically investigate frame buildings without and with fully infill walls as well as frame buildings with open soft storeys, a twelve storey reinforced concrete moment-resisting frame building is considered. The considered building has a width of 16 m divided into 4 bays and length of 36 m divided into six bays as well (see Fig. 2). The associated storey height considered is of 3 m.

Table 1 Dimensions and reinforcement of building elements

Structural element	Dimensions (mm)	Reinforcement
Beams	300 × 600	6 Φ 16
Columns	300 × 900	4 Φ 25 + 24 Φ 22

Different building models have been developed in order to meet the cases considered in the study. Bare frame model, fully infill walls model and partially infill walls models due to the existence of soft storeys at different levels have been created. Due to the symmetrical view of the considered frame model, the effect of torsional response has been avoided. The designed reinforced concrete horizontal elements in terms of beams have been set to be of 300 mm × 600 mm. The vertical elements in terms of columns have been found to be of cross sections 300 mm × 900 mm without reduction in dimensions throughout the building height. Table 1 presents dimensions and reinforcement details of the designed building elements. The columns orientation as can be seen in the Fig. 2. ETABS software package is used to perform the dynamic analysis following the Egyptian Code for loads (Figs. 3, 4).

3 Modelling of Masonry Infill Walls

Two methods have been proposed in order to properly simulate the behaviour of masonry infill walls, namely the micro-model method (see for example, Ref. [17]) and the macro-model method which has been introduced in 1960 by Polyakov [18]. Although the micro-model method is producing the better results and can be used for understanding local and global response, it is rarely used due to its complexity in generating the model and the computational costs. The macro-model method, also called the equivalent diagonal strut method, has been developed to study the global response

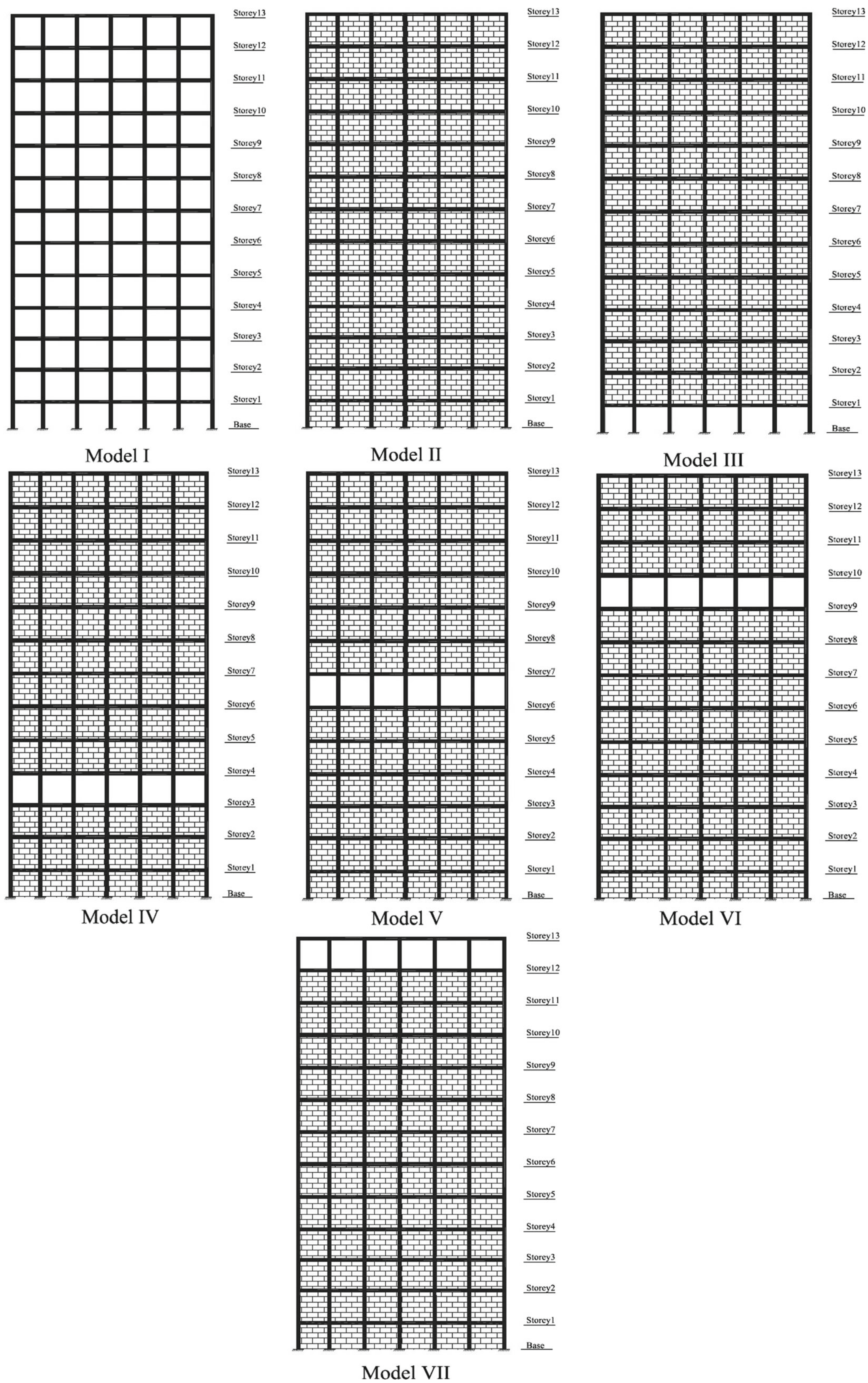


Fig. 3 Frame building models

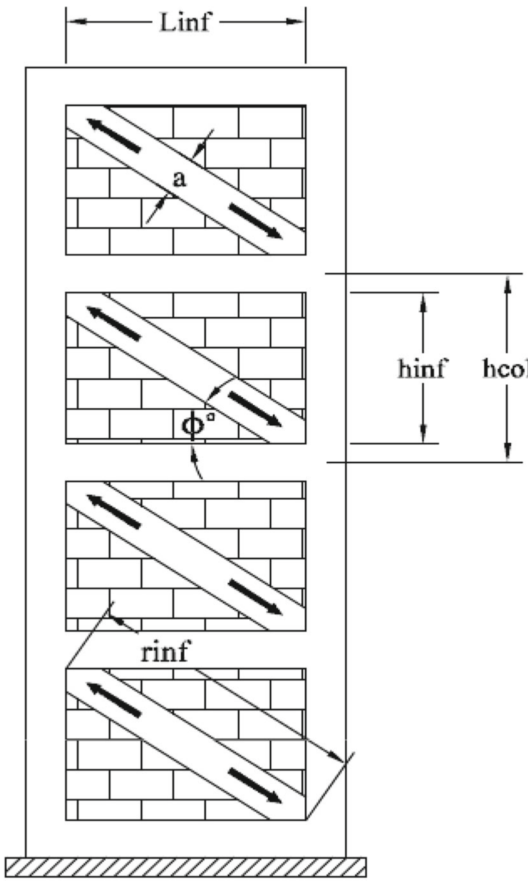


Fig. 4 Equivalent diagonal compressive strut action

of masonry infill frame buildings. The main disadvantage of the equivalent diagonal strut method is the deficiency in modelling the openings accurately. However, there are some advances in considering openings in walls where some number of struts can be used in order to accommodate the effect of openings [19]. In the current study, walls are modelled as panel elements without any opening. Requirements of FEMA 356 [20] will be followed to model the masonry infill walls. According to FEMA 356, masonry infill walls prior to cracking is modelled with an equivalent diagonal compression strut of width a. The thickness and modulus of elasticity of the strut are same as those of the represented infill panel.

The thickness of the strut can be written in terms of the column height h_{col} between centrelines of beams and the length of panel L as:

$$a = 0.175(\lambda_1\, h_{\mathrm{col}})^{-0.4} r_{\mathrm{inf}} \tag{1}$$

where the value of diagonal length of infill panel r_{inf} can be calculated according to Eq. (2)

$$r_{\mathrm{inf}}\sqrt{(L_{\mathrm{inf}})^2 + (h_{\mathrm{inf}})^2} \tag{2}$$

The Coefficient λ_1 which is used to determine equivalent width of infill strut can be calculated as a function of the infill panel height h_{inf}, moduli of elasticity of both frame materials E_{fe} and material of infill panel E_{me}, columns moment of inertia I_{col}, infill panel length L_{inf} and thickness t_{inf}, according to Eq. (3):

$$\lambda_1 = \left[\frac{E_{\mathrm{me}}\, t_{\mathrm{inf}} \sin 2\phi^\circ}{4E_{\mathrm{fe}}\, I_{\mathrm{col}}\, h_{\mathrm{inf}}}\right]^{\frac{1}{4}} \tag{3}$$

4 Time-History Analysis Method

Equivalent static force method, as a representative to linear static analysis, is the simplest technique for performing linear dynamic analysis. This simple method requires less computational efforts and follows formulations given in the codes of practice. However, it is applicable for specific types of building structures with regular shapes and limited heights as well (see Ref. [21]). In addition, response spectrum analysis as a linear dynamic method is quite accurate than the equivalent static one [22]. The time-history analysis, as a nonlinear dynamic analysis, is the best technique to evaluate structural response under earthquake excitations described by ground acceleration records. Dynamic earthquake loads incrementally affect the structure with time intervals Δt, and the governing equations of motion are solved using a step-by-step integration procedure which is the most powerful technique for nonlinear analysis. The response is evaluated for a series of short time increment. The general equation of motion is:

$$M\ddot{U}(t) + C\dot{U}(t) + KU(t) = -M\ddot{U}_{\mathrm{g}}(t) \tag{4}$$

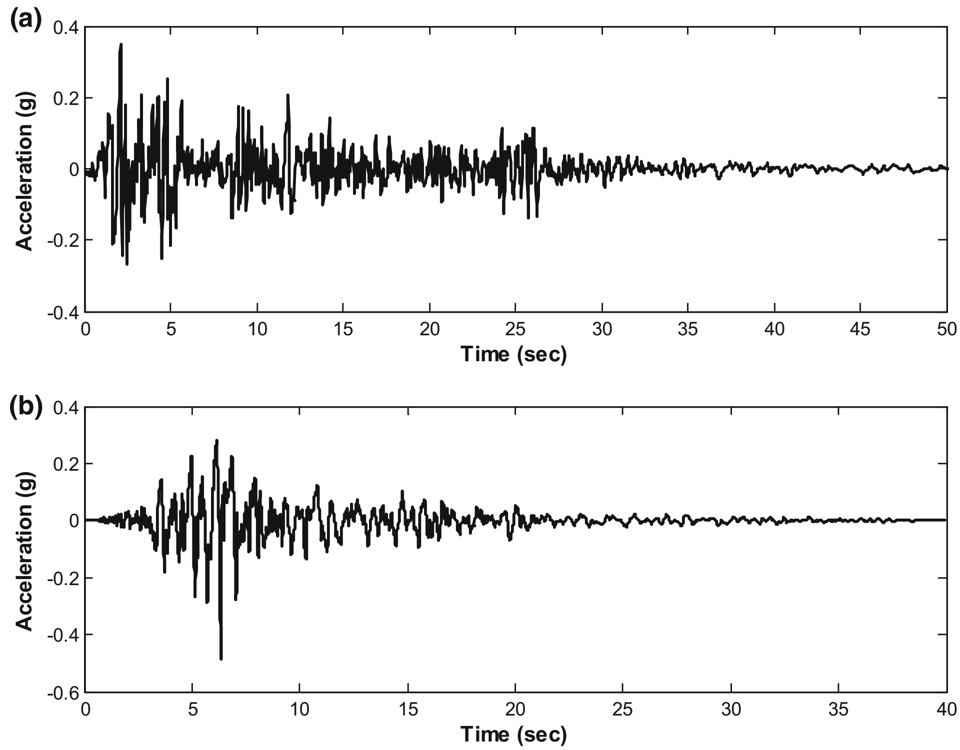
Where M, C and K are the mass, damping and stiffness matrices, respectively. The symbols $U, \dot{U}$ and $\ddot{U}$, respectively, denote displacement, velocity and accelerations vectors. $\ddot{U}_g$ is the ground acceleration vector. The aforementioned vectors and matrices can be calculated for one dimensional element by defining the proper interpolation function [23]. The equation of motion stated in Eq. (4) can be expressed in the incremental form as:

$$M\Delta\ddot{U}(t) + C(t)\Delta\dot{U}(t) + K(t)\Delta U(t) = \Delta P(t) \tag{5}$$

where $\Delta U(t)$, $\Delta\dot{U}(t)$ and $\Delta\ddot{U}(t)$, respectively, denote the incremental vectors of displacements, velocities and accelerations. The vector $\Delta P(t)$ is the incremental vector of external earthquake load.

Although the time intervals should be short enough to give an accurate representation of such a rapid varying function of time at the conventional methods, the method proposed by Chen and Robinson [24] removed the limitation on the size

Fig. 5 **a** El Centro and **b** Loma Prieta time history records



of time intervals and allows longer time intervals. Another techniques for solving incrementally the equations of motion are based on the explicit and implicit Runge–Kutta methods and can be found in Refs. [25,26]. Two different ground excitations have been selected to perform the dynamic analysis of the current study. One of these two records has been taken from the near-fault region of El Centro (1940) with site source distance of about 8 km. The second has been taken from the far-fault regions of Loma Prieta (1989) with site source distance of about 22 km. Figure 5 provides the acceleration time histories for the two earthquake ground motions used in the current analysis. The ground motions records are obtained from the PEER Strong Motion Database (http://peer.berkeley.edu/smcat/).

4.1 Nonlinear Analysis

Unfortunately, the linear elastic analysis cannot provide the structural designer with the complete picture about the performance of the structure when subjected to a ground motion record. Nonlinear analysis in terms of material and geometrical nonlinearities is considered as a reliable structural analysis capable of simulating the proper behaviour of the material and the deformation of structural elements under the applied dynamic loads. When the materials move within the yield strength limits, then the behaviour of such materials follows a linear trend. However, for the case the materials exceed the elastic limit or the yield strength, permanent deformations, cracks, beam rotations and energy dissipations in the form of inelastic and strain energy occur. Geometric nonlinearities refer to nonlinearities in kinematic quantities such as the strain-displacement relations in solids. Large deformations usually result in nonlinear strain- and curvature-displacement relations. All equilibrium equations are written in the deformed configuration of the structure. This may require a large amount of iteration. Although large displacement and large rotation effects are modelled, all strains are assumed to be small. The lateral deformations are more pronounced under dynamic loads. In the geometric nonlinearity, as the deflection of structural element gets increase the element starts to lose its stability. Due to the $P - \Delta$ effect, the applied force follows the deformed member and creates further more instability very quickly. In the structural computer program, some information in terms of stress–strain curve for concrete and steel and the limit states has to be added to the computer model in order to perform nonlinear analysis. In addition the $P - \Delta$ effect for the vertical elements has to be defined as well.

The performed nonlinear TH analyses herein employed Takeda hysteretic model. This model is considered as one of the best models that follow hysteretic rules for describing the nonlinear relation between the applied force and the corresponding deformation of the structural members. The schematic representation of the Force–displacement relationships of Takeda hysteretic model is presented in Fig. 6a.

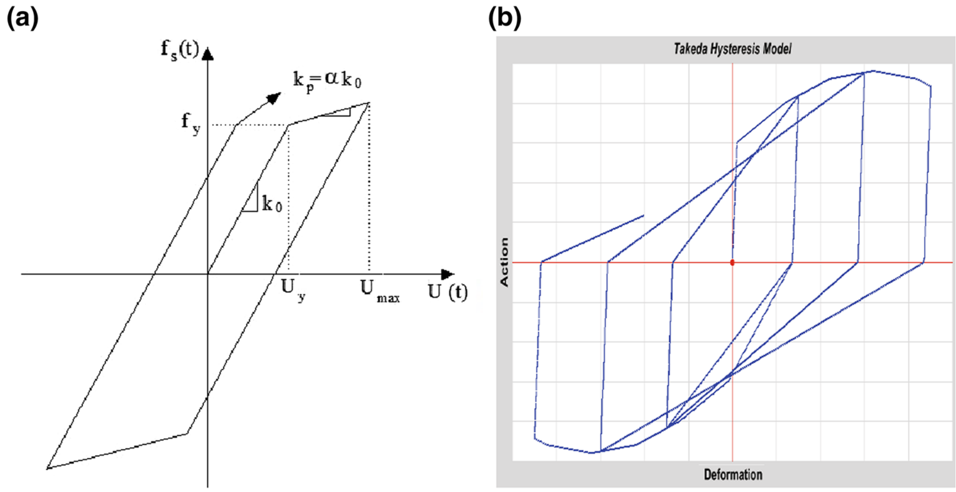


Fig. 6 Force–deformation relationship, **a** schematic representation, **b** developed in ETABS

Table 2 Shows the fundamental natural periods calculated by the generalized stiffness method and the corresponding ones calculated by dynamic analysis using ETABS

Model no	ETABS dynamic analysis		Generalized stiffness method	
	Longitudinal direction	Transverse direction	Longitudinal direction	Transverse direction
Fundamental natural period (s)				
I	2.064	2.17	2.34	2.21
II	0.852	0.88	1.00	1.05
III	0.946	1.02	0.89	0.94
IV	0.98	0.98	1.05	1.09
V	0.939	0.944	0.99	1.03
VI	0.889	0.91	0.94	0.98
VII	0.85	0.89	0.90	0.96

By the way, in ETABS structural package, this model is the default one for the structural elements of the building model (see Fig. 1b). The computer model starts with the initial stiffness k_0 of the building (see Fig. 6), and then the model is loaded incrementally till reaching the linearity limits. As the building model exceeds the elastic limits and reaches the post yield stiffness k_p, it hits the nonlinear zone and the iteration process starts to calculate strains, deflections and stiffness. U_{max} and U_y define the peak and yield displacements of the concrete elements. Here f_y is the yield force.

5 Model Validation

In order to validate the results of the developed models, the generalized stiffness method has been utilized [27]. The generalized stiffness method mainly depends on the equivalent lamped mass model. The method has been employed to calculate the fundamental natural periods and then compare with those values calculated by developed models using ETABS. Due to space limitations, only Table 2 with the calculated results from both dynamic analysis and the generalized stiffness method are presented. A more detailed review on how to apply the generalized stiffness method to calculate the fundamental natural period can be found in Ref. [27]. As it can be seen from the presented results in Table 2, the calculated values employing the generalized stiffness method show slight difference in comparable with those obtained employing the ETABS dynamic analysis.

6 Numerical Results and Discussion

Time-history analysis provides the structural response of the considered building models over time during and after the application of the seismic load. A twelve storey reinforced concrete framed building modelled as (i) bare frame (i.e. considering masonry walls as non-structural elements), (ii) fully infill frame building with masonry wall considered as structural elements and (iii) partially infill frame building with soft or open storey located separately at base, 3rd, 6th, 9th, and 12th floor levels is considered for time-history analysis under two real ground motion records. El Centro as a near-fault motion has been selected to perform the analysis. The far-fault records namely Loma Prieta have been also selected. In order to model the masonry infill action, the most widely used single-strut model is employed where it is simple and evidently most suitable for large structures. Properties of the used infill materials in terms of modulus of elasticity, unit weight and poisons ratio are 5500 MPa, 20.0 kN/m^3, 0.15, respectively. It is worth noting that masses of infill walls have been considered during the dynamic analysis of all the developed models. The dynamic time-history analysis of the building models considered is performed using the dynamic analysis software ETABS. The seismic loads produced by the structural package ETABS correspond to the data records of the El Centro and Loma Prieta earthquakes with peak ground accelerations of 0.34 and 0.48 g, respectively. A damping ratio of 5% has been associated for all the models during the analysis. The dynamic analysis software ETABS enables the user to apply the ground excitations separately in two orthogonal directions. Storey shear forces and storey moments which are considered as the most useful responses used for earthquake resistant design strategy are obtained along the height of the building models and presented in a comparative way for all the developed models. Storey displacements

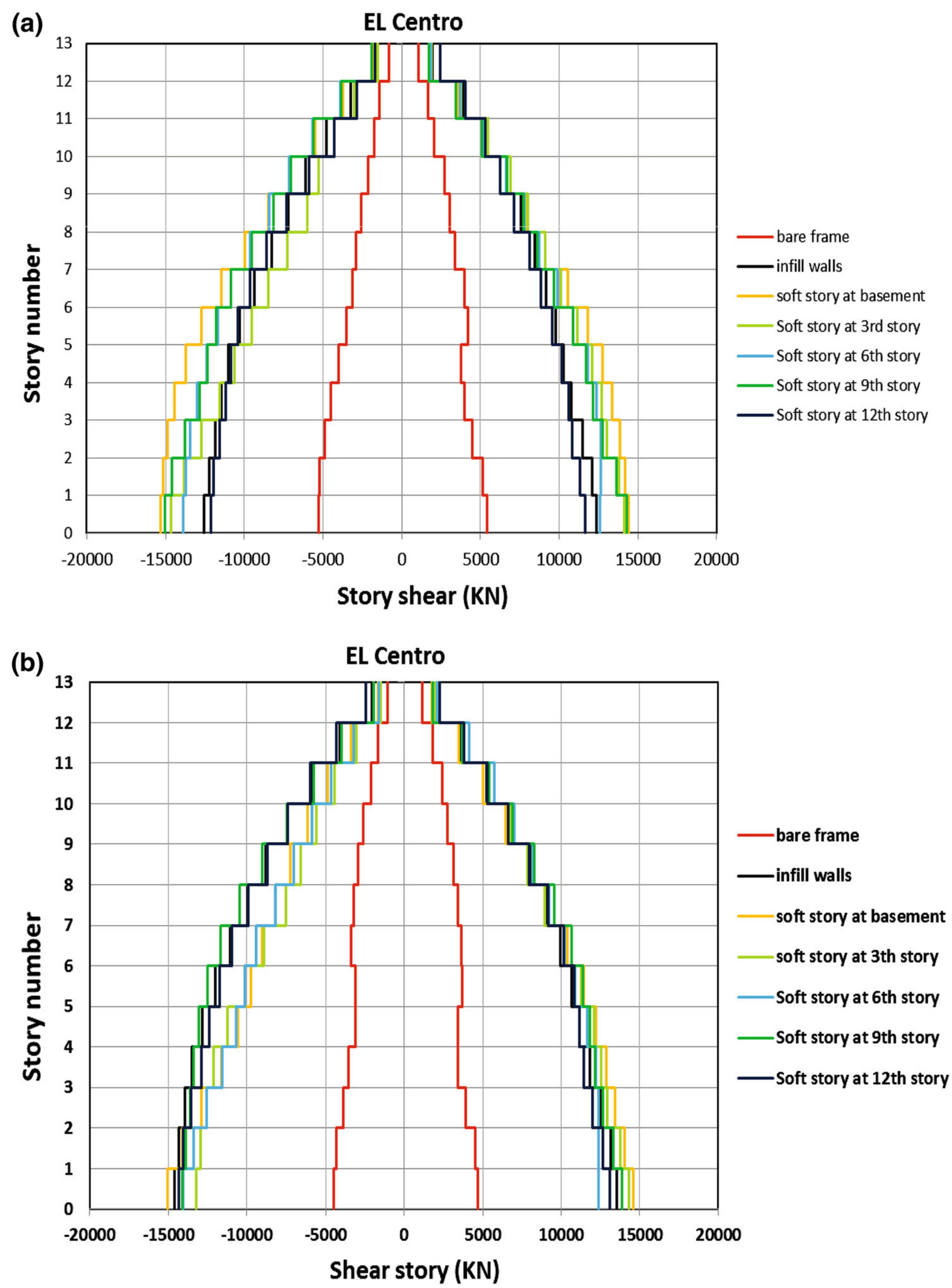


Fig. 7 Induced storey shear forces under the El Centro earthquake records for **a** *x*-direction loading, **b** *y*-direction loading

which is a measure of the building deflection are also presented following the same manner. The predicted storey drifts which can be defined as the measured displacement between two consecutive stories normalized by storey height are presented as well. Figures 6 through 13 present the obtained results for the aforementioned responses under the El Centro and Loma Prieta records for both x and y-directions.

Distribution of storey shear forces due to the applied lateral load patterns is presented in Figs. 6 and 7 for the considered building models under El Centro and Loma Prieta ground motion records applied in both x and y-directions, respectively. The plotted curves clearly show a significant difference between the cases of considering masonry infill walls and the case of bare frame in which modelling of masonry infill is ignored. As shown in these figures, storey shear results of bare frame model show the lowest values among all other models considered in this study. The difference between storey shear values for the bare frame case and the other cases of masonry infill is highly pronounced at lower storeys under the El Centro and Loma Prieta earthquake records. However, for the models with masonry infill and soft storeys assigned at different levels, the captured storey shear forces show slight differences along the height of the models under the El Centro earthquake for both x and y-directions of loading (see Fig. 7). However, under the Loma Prieta records, the soft storey level affects the induced storey shear values especially at lower storeys. Regardless the direction of loading and under the near-fault motion El Centro, it has also been noticed that the maximum shear at base is associated with the masonry infill model with soft storey at bottom level (see Fig. 7). For the far-fault motion Loma Prieta, the induced maximum shear at base has been assigned for the masonry infill model with soft

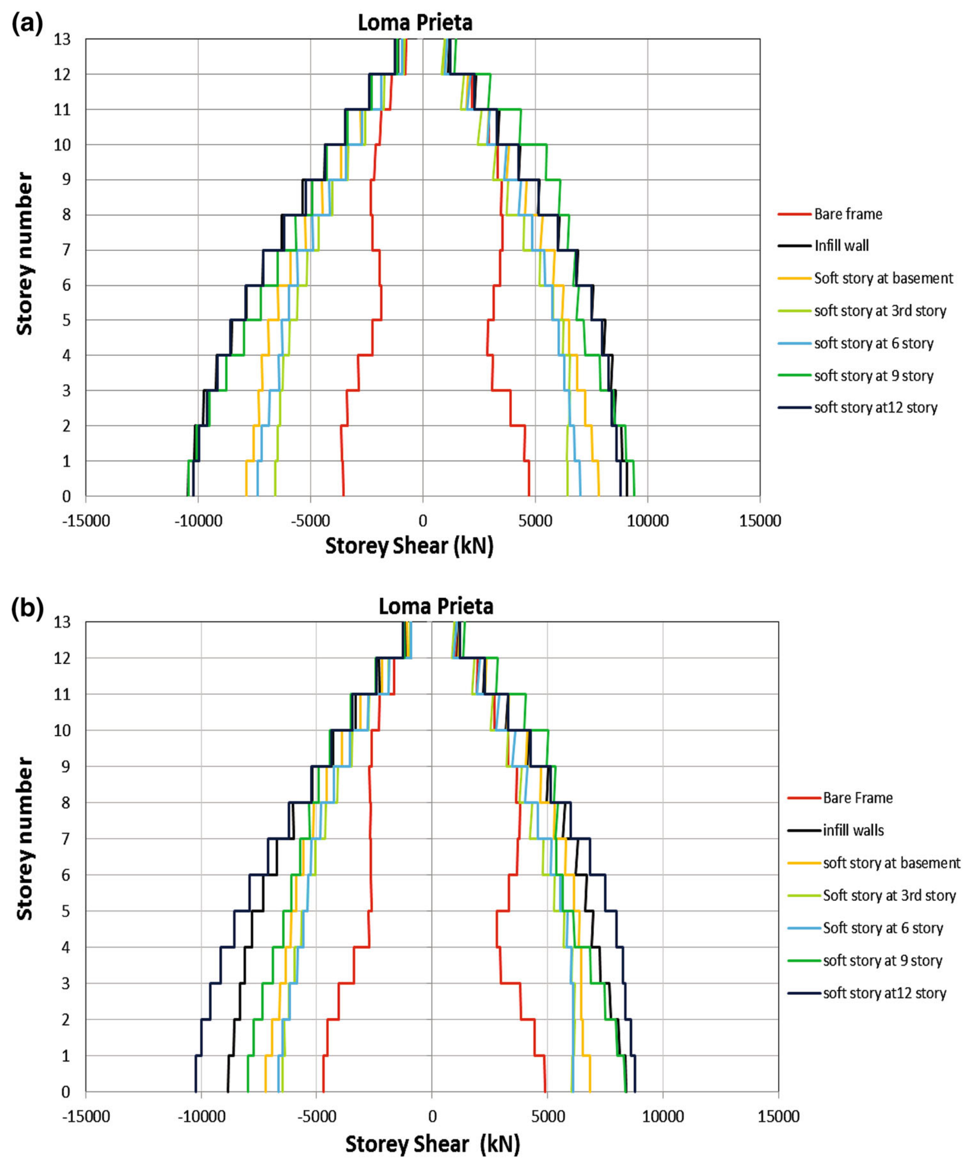


Fig. 8 Induced storey shear forces under the Loma Prieta earthquake records for **a** x-direction loading, **b** y-direction loading

storey at top (see Fig. 7). Since earthquake resistant design considers the shear at base as a governing parameter, the ignorance of masonry infill action underestimates the values of shear at bases and may lead to unsafe design. For both bare and infill frames, the generated storey shear forces under the El Centro earthquake records for both x and y-directions are in general larger than those induced under the Loma Prieta records for the same loading directions (compare Figs. 7 and 8). Masonry infill action magnifies the storey shear values with about 2.5 and 1.5 times as compared to bare frame case under the considered El Centro and Loma Prieta earthquake motions, respectively. The obvious explanation for this can be due to the El Centro records are characterized as near-fault records, while the Loma Prieta records are of far-fault characteristics. From the plotted curves for storey shear forces under the El Centro records, it has been noticed that the existence of a soft storey at the lowest level controls the outer range of the calculated storey shear forces in both x and y-directions. More specifically, with the application of near-fault records of the El Centro earthquake load in x-direction, the infill model with soft storey at base has been found to induce peak positive and negative values of 14,444 kN and −15329 kN, respectively. Similar to x-direction, the infill model with soft storey at base produced outer range values of 14,584 kN and 15061 kN during the application of El Centro records in y-direction. Contrary to the near-fault

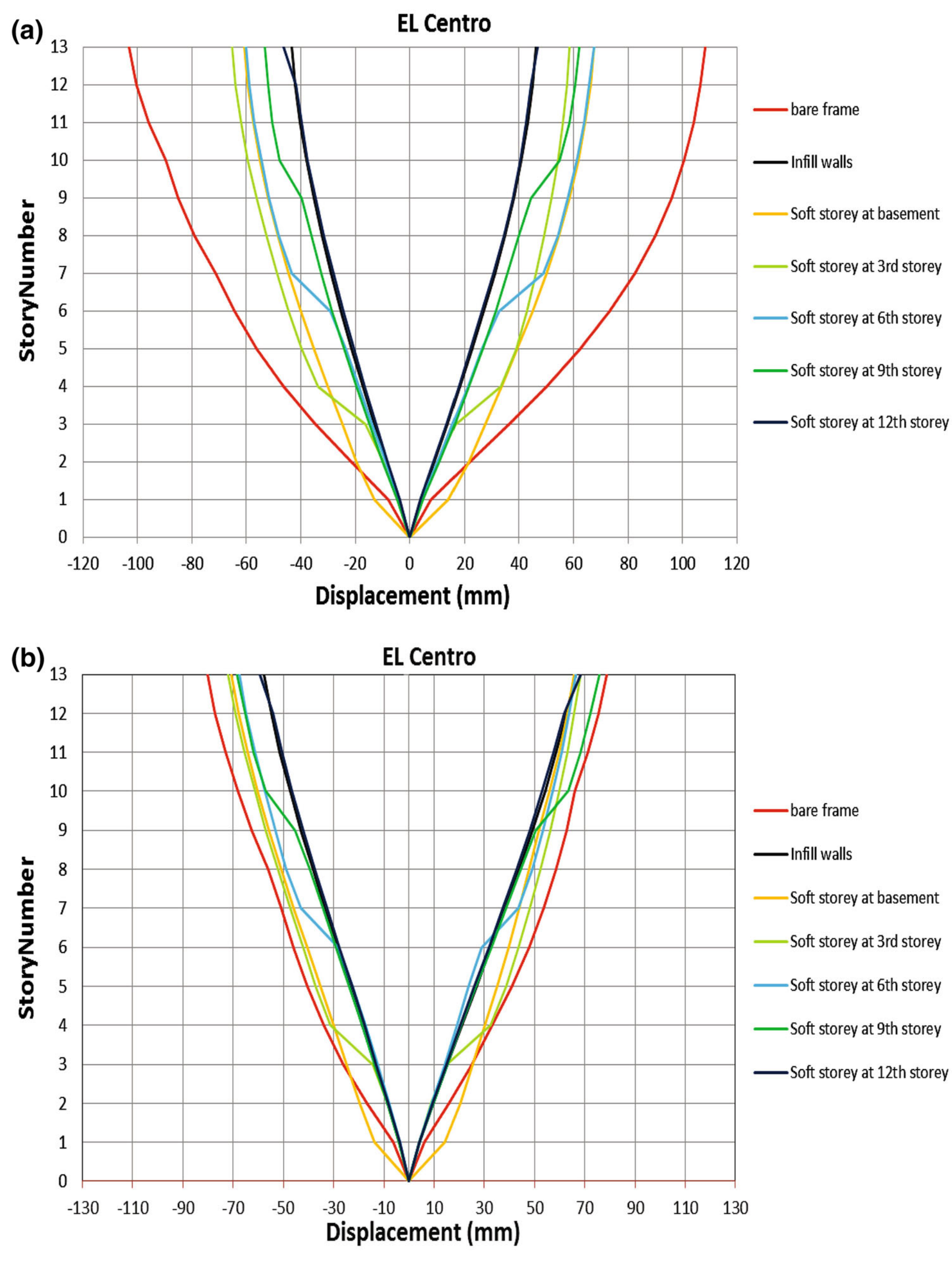


Fig. 9 Induced storey displacements under the El Centro earthquake records for **a** x-direction loading, **b** y-direction loading

records, the application of far-fault motion to excite the models clearly indicates that the existence of soft storey at higher levels produced peak values of shear forces at base control the outer range of the plotted curves. Under the applied far-fault records of Loma Prieta earthquake in x-direction, it has been found that the infill model with soft storey at the ninth floor induces peak positive and negative shear values at base of 9386 and −10448 kN respectively. These captured values represent the range of the plotted curves of storey shear in x-direction. With the application of the earthquake load in y-direction, it has been found that the infill model with soft storey at the twelfth floor produces the peak positive and negative values of 8490 and −8894 kN, respectively, to form the outer range of the plotted curves in y-direction. It is worth noting that the bare frame model produced the inner range of storey shear values regardless the direction of loading as well as the type of the applied ground motion records either near-fault or far-fault records.

Peak displacement patterns of the 12-storey bare frame building model and fully infill building model as well as the building model with soft storeys at different levels under the El Centro and the Loma Prieta earthquake records are presented in Figs. 9 and 10, respectively. The two earthquake records are applied in two orthogonal directions. As shown in this figures, the existence of soft storey causes a sudden change in the obtained peak displacements. This abrupt change leads to an increase in storey displacements just after passing the soft storey level which is highly pro-

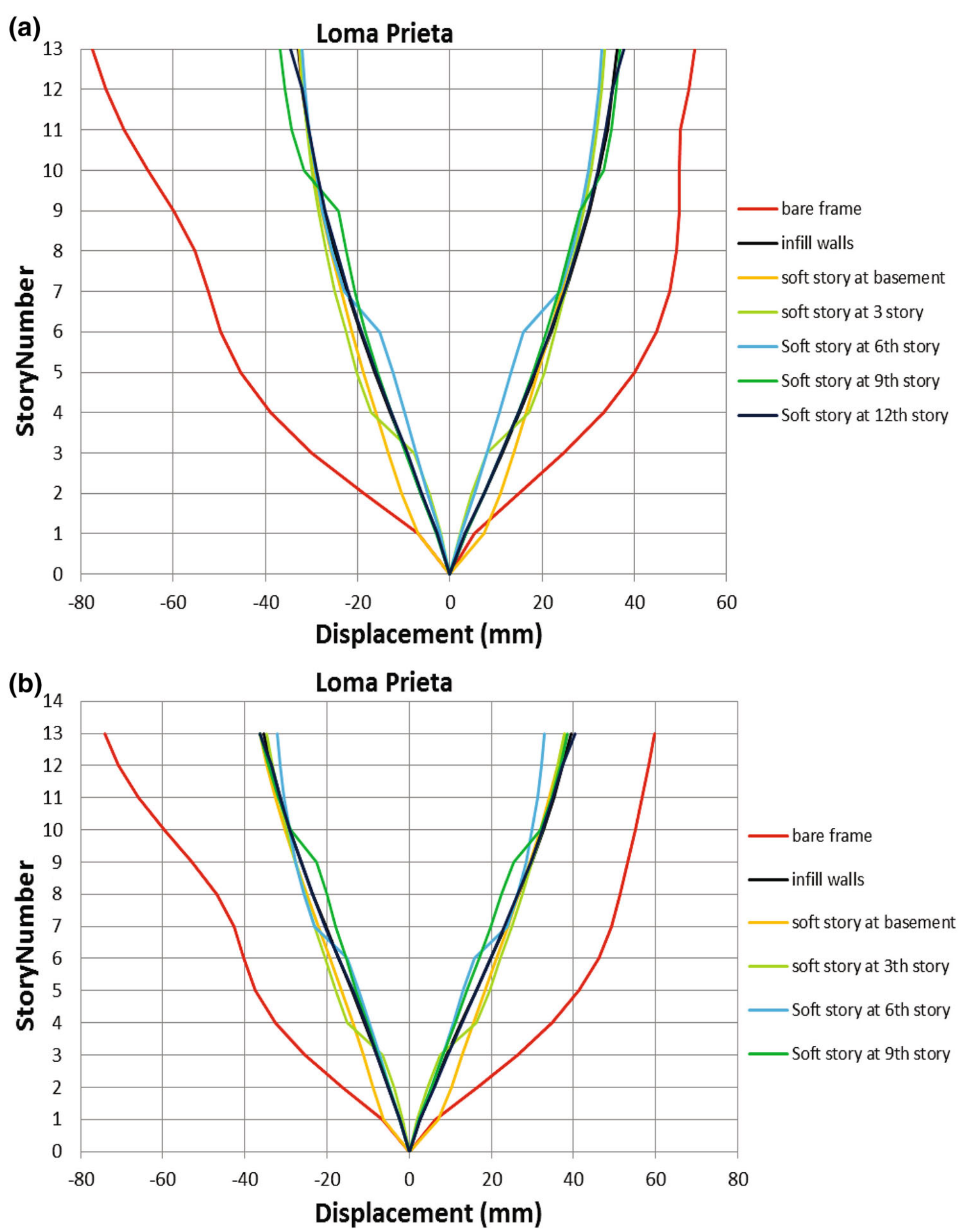
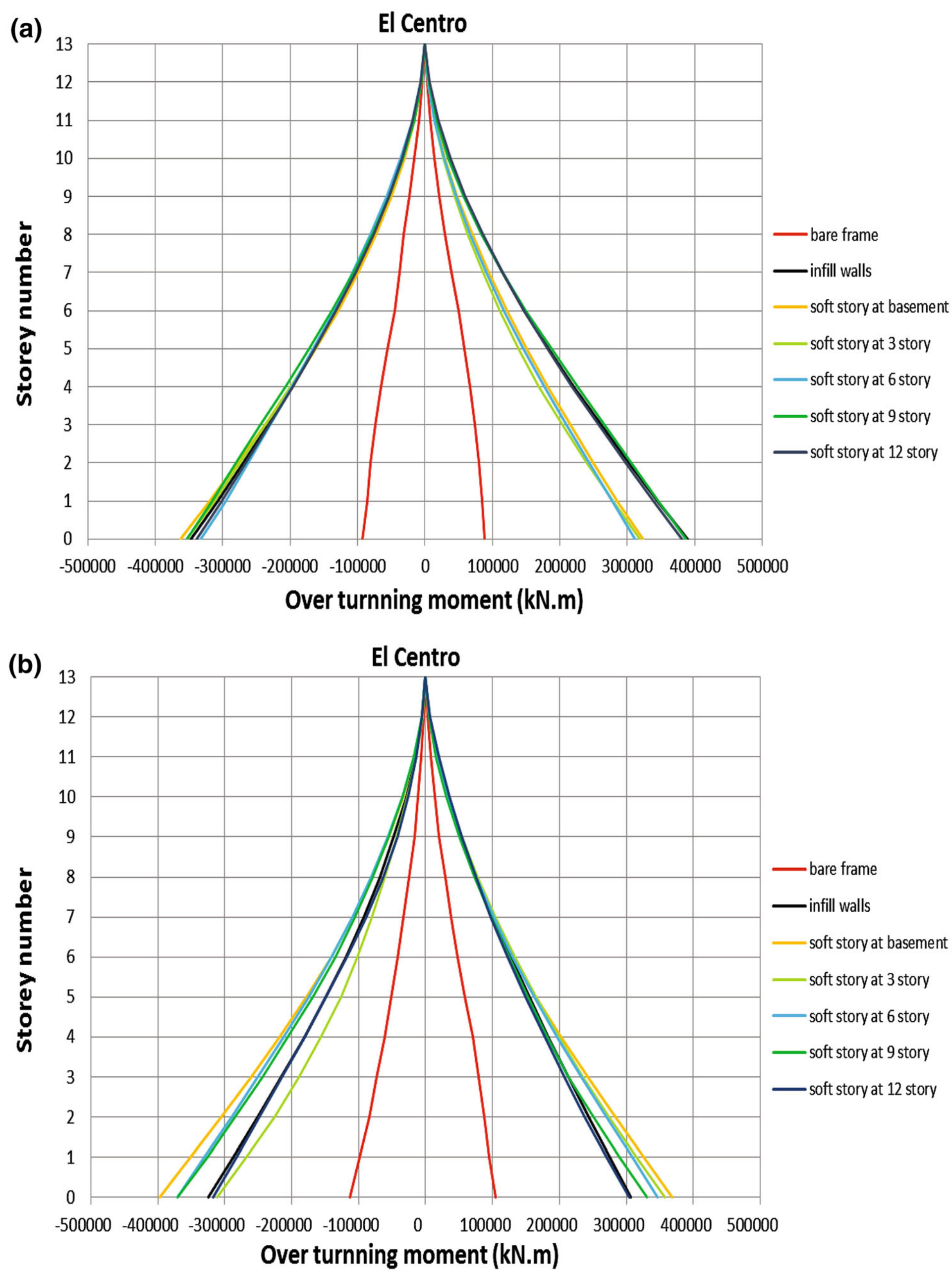


Fig. 10 Induced storey displacements under the Loma Prieta earthquake records for **a** x-direction loading, **b** y-direction loading

nounced under the El Centro records. The bare frame model produces higher peak storey displacements as compared to the masonry infill building frame models without and with soft storeys under both the El Centro and Loma Prieta earthquakes. This can be due to infill frame building systems with and without soft storeys have higher stiffness than the bare frame building model under the applied dynamic lateral load. This added stiffness to the infill system is due to the presence of masonry infill walls. In contrast, the infill wall model without soft storey produces the lowest storey displacements under both the El Centro and Loma Prieta records for the considered two directions of loading. If the induced lateral deflections due to the existence of soft storey become too large, the resulting $P - \Delta$ effect may lead to an instability to the building structure and potentially results in collapse. For the masonry infill model without and with soft storeys, slight differences between the induced peak storey displacements can be seen from the plotted curves under the Loma Prieta ground motion records applied in both x and y-directions. In general, at the upper storeys of the structural models considered and under the two earthquake records in terms of El Centro and Loma Prieta, greater lateral displacements are allocated as compared to the induced deflections of the lower storeys regardless the loading direction.

Fig. 11 Induced storey moments under the El Centro earthquake records for **a** x-direction loading, **b** y-direction loading



Results of the storey moment patterns for the bare frame model, model with infill walls and models with soft storeys at 3rd, 6th, 9th and 12th floor levels under El Centro ground motion records are presented in Fig. 11 considering two directions of loading. The corresponding results under the Loma Prieta earthquake records are presented in Fig. 12. Comparing Figs. 11 with 12, it can be noticed that the induced peak moments under the Loma Prieta earthquake vary significantly as the level of the soft storey changes especially at lower storeys. However, under the El Centro Earthquake records, the variation in the obtained peak storey moments seems to be insignificant and shows slight change with the variation of the soft storey level. Under both excitation records and irrespective the loading direction, the bare frame building model induces the lower storey moments. Contrary to the bare frame model, the full masonry infill building model induces storey moments of higher values as compared to the other models. This can be seen clearly when using Loma Prieta earthquake to excite the building models. Under the El Centro earthquake records, the value of induced peak moment considering x-direction of loading shows good agreement with the one obtained considering y-direction of loading. However, under the Loma Prieta earthquake load applied in x-direction, the peak moment value obtained is higher than the one obtained one in y-direction. Consequently, a magnification factor for the moments has to be considered in the design stages. The reason behind the higher storey moments in case of fully infilled models in comparison

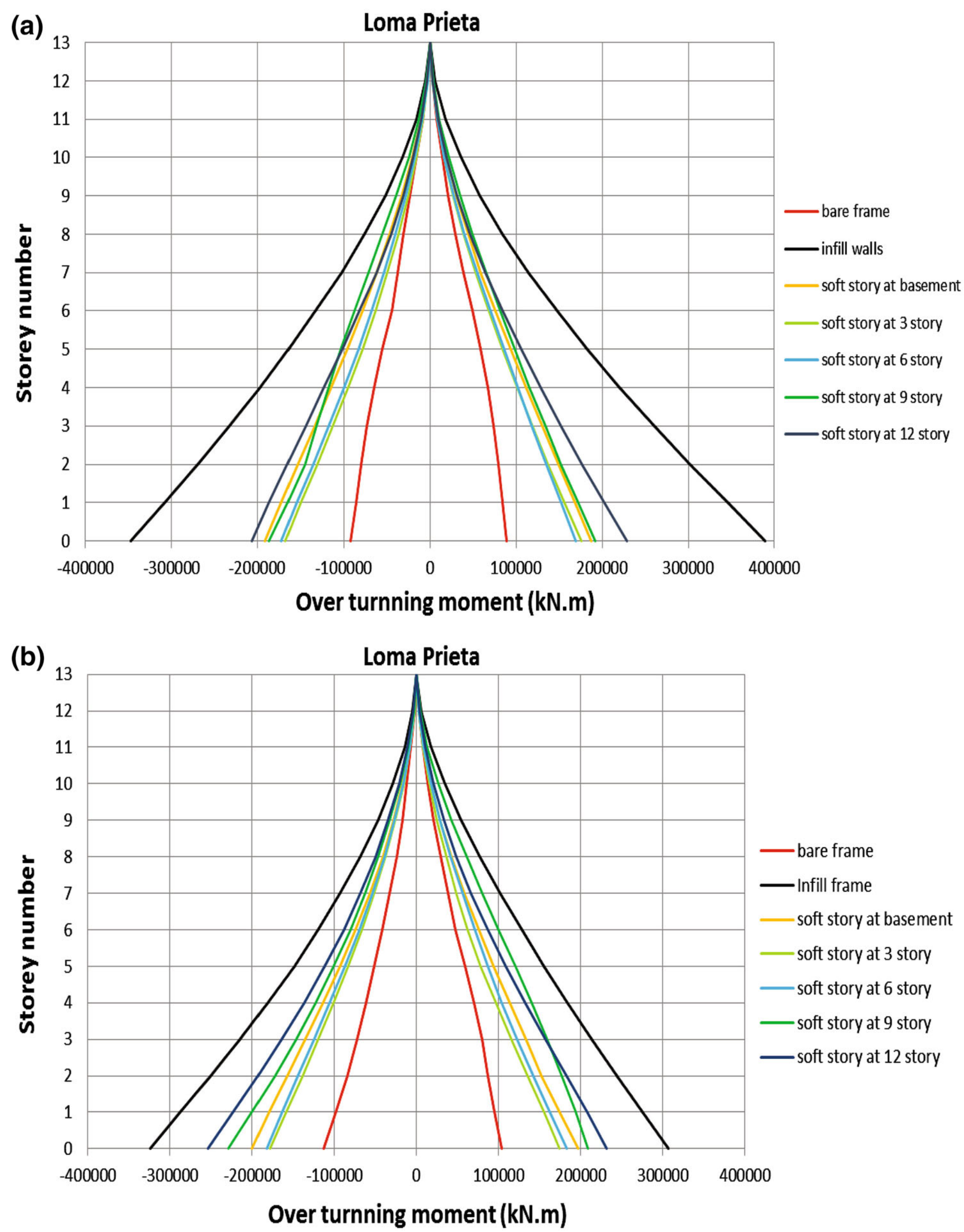
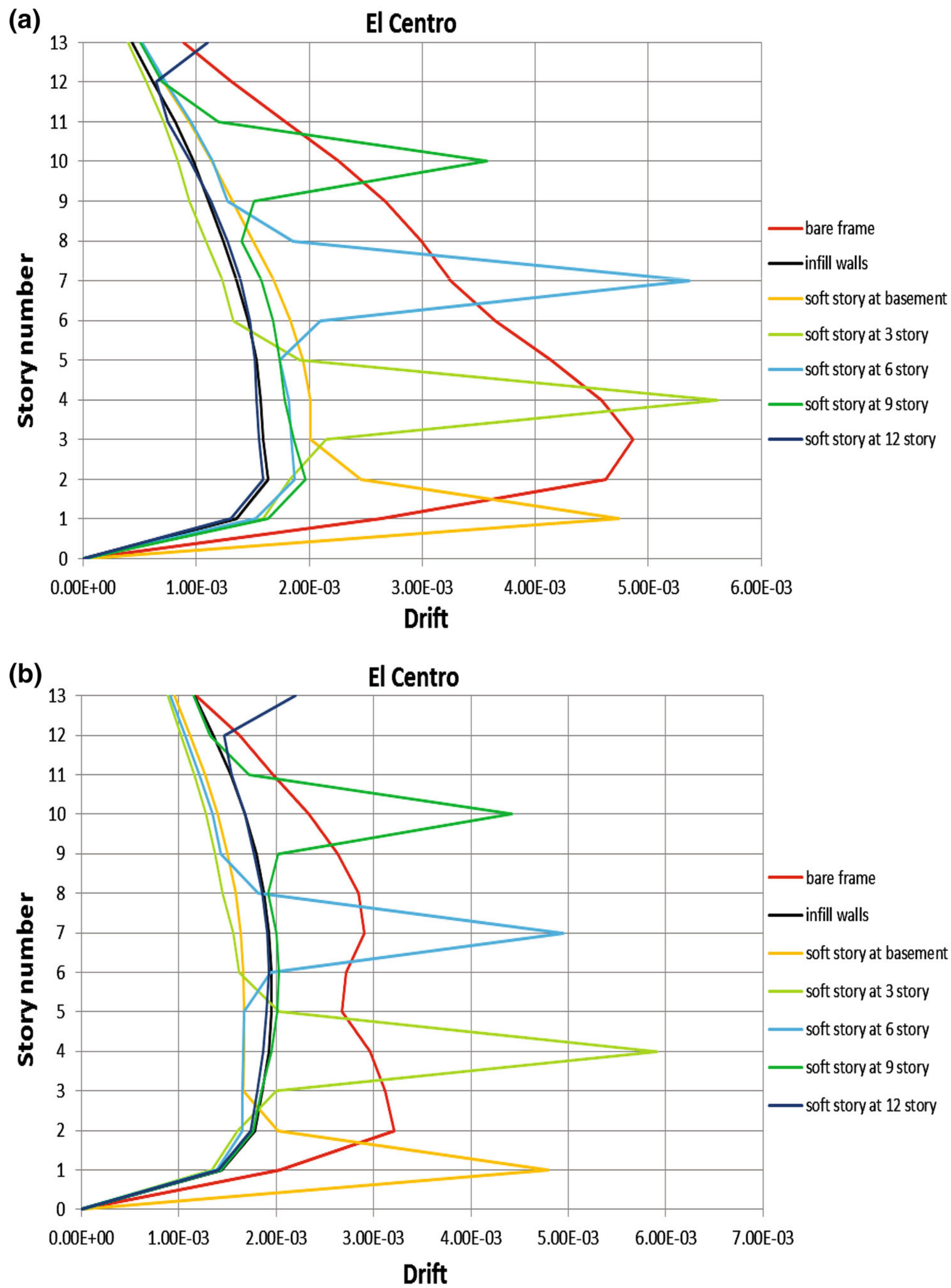


Fig. 12 Induced storey moments under the Loma Prieta earthquake records for **a** x-direction loading, **b** y-direction loading

to the other models can be due to the increase in stiffness of the fully infill model which is assumed to be the summation of the bare frame model stiffness and the infill wall stiffness. The increase in stiffness leads to an increase in the induced straining action in terms of storey moments.

Figures 13 and 14 show the results of maximum storey drift ratios of 12-storey structure under the El Centro and Loma Prieta ground motion records. These obtained results demonstrate the differences among the drift profiles of the building structure modelled as bare frame, fully infilled building model and infilled building models with soft storeys. As it can be seen from the figures, the bare frame building model has drift ratios of higher values than those associated with the considered fully infill frame building model under the near-fault El Centro records and the far-fault Loma Prieta records. In addition and as it can be seen from the figures, the captured values of the drift ratios at the specified soft storeys of the infill frame building models exceed those values obtained considering the building structure modelled as bare frame under the applied two ground motions. However, this increase in drift ratio can be more pronounced under near-fault El Centro records as compared to the far-fault Loma Prieta records regardless the direction of loading.

Fig. 13 Induced storey drifts under the El Centro earthquake records for **a** *x*-direction loading, **b** *y*-direction loading



Compared to the fully infilled frame building model, the infill frame models with soft storeys have sudden increase at the specified soft storey level. This observed trend has been noticed for all the considered soft storey models under the two ground motion records considered. This can be due to the building frame models with soft storeys suffer from strength and stiffness discontinuities of those above and below storeys.

The unexpected movements of building structure under lateral seismic actions due to soft storeys can be highly significant. These may lead to pounding between adjacent buildings or structurally separate portions of the same building that do not have adequate separation distance in between. These collisions between structural elements can result in increase in floor acceleration as well as significant localized damage between these structural elements.

From seismic design point of view, these excessive lateral movements and drifts due to existence of a soft storey can significantly affect the structural elements even if these elements are part of the lateral forces resisting system.

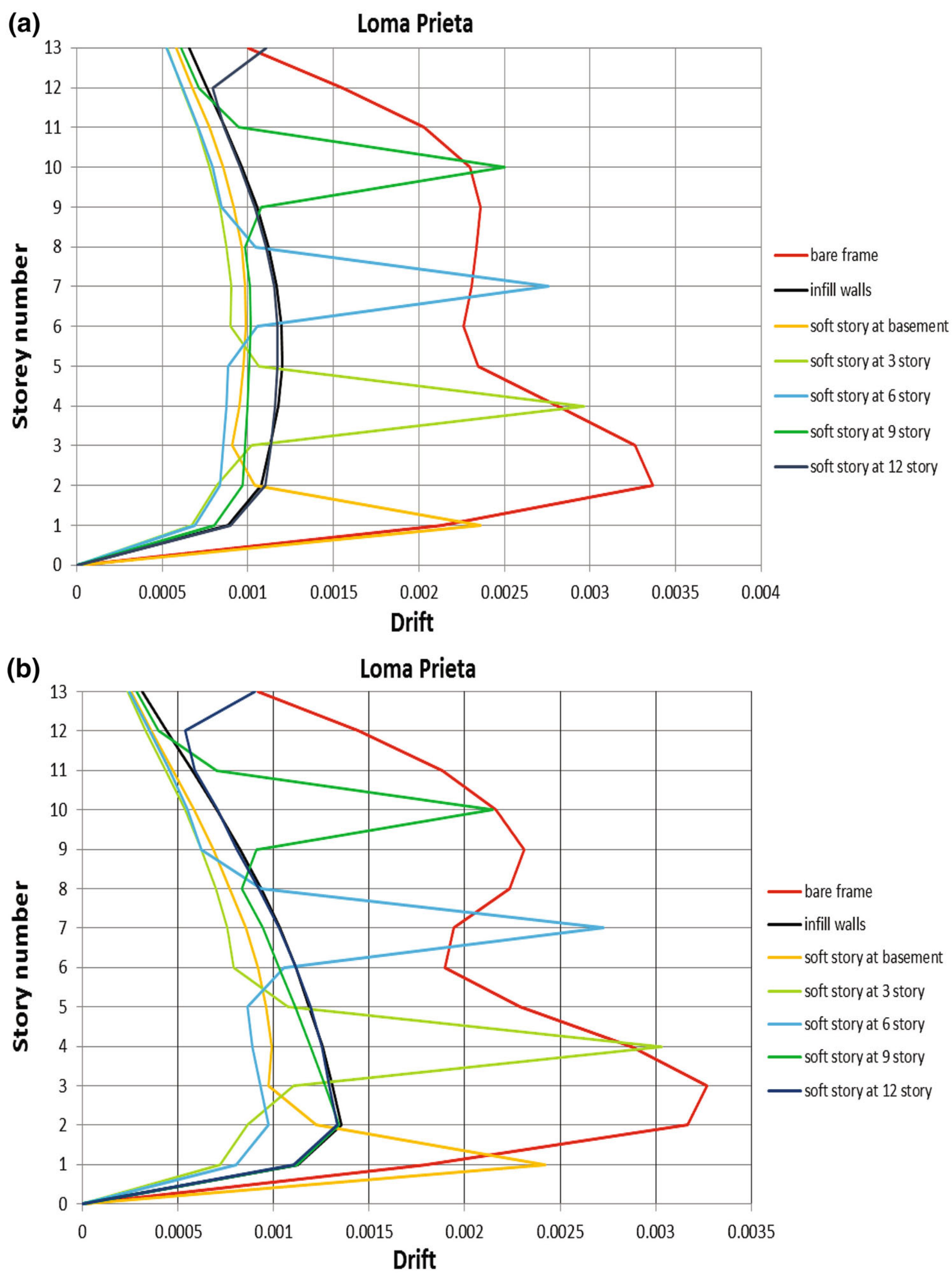


Fig. 14 Induced storey drifts under the Loma Prieta earthquake records for **a** *x*-direction loading, **b** *y*-direction loading

7 Conclusions

The current research study has been carried out on reinforced concrete framed buildings fully as well as partially infilled under seismic loads. Dynamic time-history analysis has been performed employing two ground motions from near and far-fault regions. The influence of infill wall action on the seismic performance of the developed structural models with soft storey has been investigated. The effect of variation of soft storey level has been studied as well. The following results summarize the main findings of the considered different scenarios of the structural models.

1. The masonry infill action has a significant influence on the global performance of the building structure where the induced structural responses for bare frame case do significantly vary with the different configurations associated with fully or partially masonry infill walls under either near-fault or far-fault earthquake loads.
2. Considering masonry infill action reduces the induced storey displacements as compared to the bare frame case. However, the induced storey moments and storey shear forces increase with the incorporation of masonry infill action.
3. Masonry infill walls enhance the seismic performance of the building structure during earthquake excitations in terms of displacement control, storey drifts and lateral stiffness.
4. The level of soft storey has a significant role on the induced storey shear forces under the far-fault Loma Prieta earthquake especially at lower storeys. However, under El Centro earthquake, the level of soft storey seems to be slightly significant.
5. Contrary to the induced storey shear forces, the induced storey displacements and moments are significantly affected by the variations of soft storey level under near-fault motion. However, under far-fault motion, these responses seem to be unaffected with the variation of the soft storey level.
6. Compared to the fully infilled frame building model, the infill frame models with soft storeys have sudden increase in the obtained responses at the specified soft storey levels regardless direction of loading and the type of the applied earthquake records as well.
7. Although the masonry infill action decreases the values of induced storey drift as compared to the bare frame case, the existence of a soft storey at a specified level highly magnifies storey drift at that level with values exceed those associated with the bare frame case.
8. The National Building Codes should provide a magnification factor for the storey response in terms of storey shear forces and overturning moments. On the other hand, in case of ignoring masonry infill actions, a reduction factor for storey displacements should be provided.

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