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Soft story retrofit of low-rise braced buildings by equivalent moment-resisting frames

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Abstract. Soft-story buildings have bottom stories much less rigid than the top stories and are susceptible to earthquake damage. Therefore, the seismic design specifications need strict design considerations in such cases. In this paper, a four-story building was investigated as a case study and the effects of X-braces elimination in its lower stories studied. In addition, the possibility of replacement of the X-braces in soft-stories with equivalent moment resisting frame inspected in two different phases. In first phase, the stiffness of X-braces and equivalent moment-resisting frames evaluated using classic equations. In final phase, diagonals removed from the lowest story to develop a soft-story and replaced with moment resisting frames. Then, the seismic stiffness variation of moment-resisting frame evaluated using nonlinear static and dynamic analyses. The results show that substitution of braced frames with an equivalent moment-resisting frame of the same stiffness increases story drift and reduces energy absorption capacity. However, it is enough to consider the needs of building codes, even using equivalent moment resisting frame instead of X-Braces, to avoid soft-story stiffness irregularity in seismic design of buildings. Besides, soft-story development in the second story may be more critical under strong ground excitations, because of interaction of adjacent stories.

Keywords: ductility; dynamic analysis; earthquake/seismic analysis; energy dissipation; non-linear analysis; retrofit/rehabilitation; stability/instability; steel structures

1. Introduction

Soft-story buildings, in which first stories are much less rigid than the other stories above, are susceptible to earthquake damage because of large, non-reinforced openings on their first stories. These openings often created because of parking spaces, large windows and lobbies in residential and commercial buildings. Soft-story may also occur because of abrupt changes in number of infill walls between stories, which are usually not considered as a part of load bearing system (Inel and Ozmen 2008). Without proper design, such buildings cannot withstand the lateral forces generated by earthquakes. Therefore, the upper floors pancake down on top of folded first floor and crush everything underneath. Under large earthquake excitations, soft-story buildings behave like an inverted pendulum with the ductility demand concentrated at the soft-story (Wibowo *et al.* 2010). As a result, the columns of the soft-story collapse because of excessive demand capacities.

Moehle and Alarcone (1986) studied the first researches on soft-story. They employed both analytical and experimental tools to control the behavior of reinforced concrete structures with a hybrid system of moment-resisting frame and shear wall. They noticed the shear walls of one of the models on the first story cracked and the ductility demand of the members at the discontinuity locations increased up to 4 to 5 times of the early shear wall (without crack). Valmudsson and Nau (1997) studied the effect of the changes in the stiffness and strength of the first story in buildings of 5, 10, and 20 stories. They remarked that reducing the stiffness of the first story by 20 percent increases the story drift between 20 to 40 percent. Also, simultaneous drop of both stiffness and strength of the first story by 30 percent, increases the ductility demand between 2.2 and 3 times. Yoshimura (1997) analyzed a reinforced concrete building with a soft first story which collapsed by the 1995 Hyogoken-Nanbu Earthquake. He clarified if the soft first story fails, its collapse could be unavoidable even for a base-shear strength of 60% of the total weight. Chintanapakdee and Chopra (2004) studied the effect of the stiffness and strength irregularities on the drift and deformation of a twelve-story building under twenty earthquake records. They employed strong column-weak beam principle in the design of the structure. The occurance of soft (or weak) story increased the drift of that story and decreased the drift of other stories in comparison with the primary regular structure. Trung and Lee 2008) studied the effect of irregularity in elevation of a twenty-story building with a moment-resisting frame system. They made irregularities in the stories 1, 1 to 3, 9 to 12, 18 to 20, and

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20. They found that reduced stiffness of stories led to excessive drift in that stories and decreased the drift in adjacent stories. In addition, reduction of the stiffness is more severe in the middle and bottom stories of the building. Ghale-Noei and Golkari (2011) studied the effect of brace removing because of architectural limits for providing big entrances or large windows in commercial parts of steel buildings. They found that formation of plastic hinges and soft-story damages is much severe in the middle stories. Tena-Colunga (2010) proposed two procedures for determination of the soft-first-story irregularity condition in buildings. He offered the soft-first-story irregularity condition controlled by a big drop of the lateral shear stiffness of one or more resisting frames within a given story. He clarified that this definition is better than building code rules. The building codes need a significant decrease of the lateral shear stiffness of all resisting frames within a given story for soft-story evaluation. Guevara-Perez (2012) studied the soft-story and weak-story buildings and made some remarks to avoid collapse of these buildings during earthquakes.

Tremblay and Tirca (2003) mitigated soft-story response of multistory buildings using zipper concentrically braced steel frames. They used traditional capacity design principles and aimed to keep the zipper column in elastic region. Therefore, the complete plastic mechanism developed in the structure and prevented the story mechanism due to concentration of inelastic demand in height of structure. Hejazi *et al.* (2011) studied effect of soft-story on structural response of high rise buildings and highlighted the importance of avoiding sudden changes in lateral stiffness and strength. They tried to add various bracing arrangements to reduce soft-story effect on seismic response of building.

Haque and Amanat (2009) studied behavior of masonry infilled reinforced concrete frames using response spectrum and equivalent static methods. Study of the sway characteristics showed that the columns of open ground floor demanded significantly higher flexibility and ductility. Therefore, conventional equivalent static force method is incapable of predicting these behaviors resulting in significant under-design of the columns of open ground floor which led to the collapse of many such buildings in the past earthquakes. Guney and Aydin (2012) showed the contribution of infill walls to the building response during earthquake. Therefore, different type of configuration of infill walls modeled and analyzed by the Finite Element Method. They found that existence of infill walls causes, less shear forces on the frame columns. However, in the case of infilled frame with a soft ground story, the shear forces acting on columns are considerably higher than bare frame shear forces. Lee *et al.* (2011) studied the seismic responses of a 1:5-scale five-story reinforced concrete building model with a high irregularity of weak story, soft story, and torsion simultaneously at the ground story. They found that the lateral resistance and stiffness of the critical columns and walls increased or decreased significantly with the large variation of acting axial forces caused by the high bidirectional overturning moments and rocking phenomena under the bi-directional excitation. Favvata *et al.* (2013) studied the seismic performance of reinforced concrete (RC) frame structures with irregularities leading to soft first floor using capacity assessment procedures. They investigated soft first story effect in different cases and found that an increase of the demands observed for inter-story drift at the first floor level due to the considered irregularities.

Pang *et al.* (2012) proposed a 3D collapse analysis model, named as “pancake”, for soft-story in light-frame wood buildings. Pirizadeh and Shakib (2013) studied both regular and irregular buildings using the Incremental Dynamic Analysis (IDA) method. They looked into a regular 10-story building and some irregular structures. They changed stiffness, mass, strength and combination of strength and stiffness in stories. They clarified the changes in the bottom half of the structure reduces the story displacement capacity and increases the chance of failure. Dya and Oretaa (2015) studied the behavior of soft-story buildings using pushover analyses and found seismic demand concentration in the soft-story. They remarked that any properly designed building will be able to withstand seismic excitations without incurring notable damage.

Sahoo and Rai (2013) evaluated two seismic strengthening techniques to improve the seismic performance of the existing non-ductile reinforced concrete columns in a soft ground story. The techniques were related to column retrofit using partial steel jacketing (or caging) and using aluminum shear links to absorb energy. They found the second method was more effective to control the drift response of the soft-story. Mo and Chang (1995) proposed base isolation in soft-first-story buildings. They fitted shear walls with Teflon sliders at first story, while the remaining first story columns designed with reduced yielding stress. Miyamoto and Scholl (1996) employed viscous dampers for seismic rehabilitation of non-ductile soft-story concrete structure of a hotel. They proved that installation of viscous dampers and moment frames at the first story lessens the drifts at all levels to the wanted performance. Also, the characteristics of Hydraulic Mass Control System (HMCS) on soft-first-story structures studied based on experiments and its control mechanism revealed (Hui 1994). Jennings *et al.* (2014) investigated retrofit of wood structures by application of Shape Memory Alloy (SMA) steel in Scissor-jack braces and supported analytical models using full scale hybrid tests.

The National Building Code of India (2005), China (2002), and ASCE-7 (2010) defined the Stiffness-soft-story irregularity. The lateral stiffness of irregular building is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above. However, The Turkish Earthquake Code (2007) defines it with different procedure which does not use the qualitative equations. The vertical regular building shall satisfy Eq. (1).

$$\eta_{ki} = \left(\Delta_i / h_i\right)_{avr} / \left(\Delta_{i+1} / h_{i+1}\right)_{avr} < 2 \qquad (1)$$

where, η_{ki} is the drift of story i and calculated using the equivalent static analysis with an eccentricity of 5% for columns and bearing walls, Δ_i and Δ_{i+1} are lateral displacements of stories subjected to earthquake loading, h_i and h_{i+1} are the heights of stories i and $i+1$, respectively.

The similar procedure considered in EUROCODE-8 (2003). It specifies that the lateral load resisting system, including moment-resisting frame or bracing, must be continuous and without any discontinuity from the foundation to the top of the building. In addition, the stiffness and mass of the building shall be constant in its height or reduced gradually without sudden variation of the stiffness and mass in the stories of the building.

In this research, the effect of the growth of the soft-story in a four-story steel building with special X-bracing studied using classical formulas and analytical tools. The building designed using the AISC360 specification for steel buildings (2010) and AISC341 seismic provisions (2010). The behavior of the building studied in two phases after removing the braces from the stories in the first and second levels. Then, the braces in the soft-story substituted with the beam-to-column moment-resisting connections and the stiffness of that story increased gradually using bigger column and beam sections. The behavior of each frame studied separately using the nonlinear static analysis (pushover) method. The possibility of substitution of the braces with equivalent moment-resisting frames in the soft-story studied. The behavior of each frame also studied under four sever earthquake records of Chi-Chi-Taiwan, Tabas-Iran, San Fernando-USA, and Northridge-USA using the nonlinear time history analyses. The current definitions of seismic provisions for soft-story stiffness in buildings corroborated and the possibility of using moment-resisting frames for retrofit of soft-stories discussed.

2. Classical stiffness theory for frames

The classical equations of solid mechanics and the equilibrium equations of the frames used to evaluate the shear stiffness of the stories in X-braced and moment-resisting frames.

2.1 X-braced frame

The early condition of the braced frame before loading and its deflected shape after loading depicted in Fig. 1. The rigid beam considered as part of a rigid floor. In addition, the hinged connections assumed for all members of the braced frame to evaluate its stiffness in shear.

The stiffness of the braced frame used for determinations of the lateral force capacity and lateral displacement of the frame. When the lateral force applied statically and increased gradually, the diagonal in compression first reaches to its buckling capacity. The equilibrium of free body diagram needs the force in the brace in tension is equal to the force in the brace in compression. Therefore, assuming short diagonals for braces, the equivalent lateral capacity at the onset of buckling of the brace in compression will be equal to

$$F_b = 2F_y.0.658^{\frac{F_y}{F_e}}.A_b.\cos\theta \tag{2}$$

where F_y is yielding stress, θ is the angle of inclination of

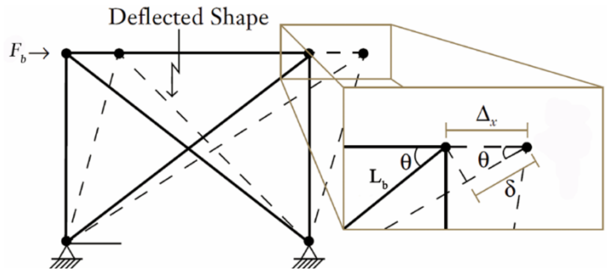


Fig. 1 Behavior of the braced frame under lateral loads

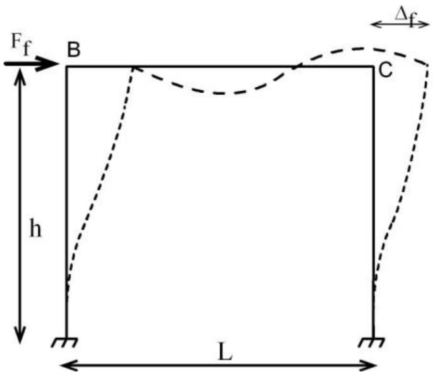


Fig. 2 Behavior of the moment frame under earthquake loading

the braces to the horizon, F_e is Euler's elastic critical buckling stress, and A_b is cross-sectional area of the braces. The lateral displacement of the braced frame calculated by the classical equations of solid mechanics in Eq. (3) from projecting longitudinal deflection of the brace in tension.

$$\Delta_b = \frac{F_b.L_b}{E.A_b.\cos\theta} = \frac{F_y 0.658^{\frac{F_y}{F_e}}.L_b}{E.\cos\theta} \tag{3}$$

where E is the modulus of elasticity of steel and L_b is the brace length.

The early stiffness of the X-braced frame calculated using $F=K.\Delta$ relation.

$$K_b = \frac{F_b}{\Delta_b} = \frac{2.E.A_b.\cos^2\theta}{L_b} \tag{4}$$

The stiffness of the braced frame is calculated by replacing $\cos\theta = L / L_b$ in Eq. (4).

$$K_b = \frac{2EA_bL^2}{L_b^{\ 3}} \tag{5}$$

where L is the braced frame span.

2.2 Moment resisting frame

A typical one-story moment-resisting frame depicted in Fig. 2. All connections assumed as rigid and the axial deformations neglected in columns for calculation of the lateral stiffness of moment resisting frame.

The slope-deflection formulas, equilibrium equations of the bending moments in the joints of the frame, and equality of the slopes at the beam to column joints at *B* and *C* used for evaluation of parameters. Therefore, the horizontal reaction force at the base of columns is equal to the lateral capacity of moment resisting frame in Eq. (6).

$$F_f = \frac{2E.I_c}{h^2}(6\theta_B - \frac{12\Delta_f}{h}) \tag{6}$$

where *L* is frame span, *h* is column height, and I_c is the moment of inertia of the column. Rotation of joint *B* calculated by

$$\theta_B = \frac{3\Delta_f I_c}{h^2\left(\frac{2I_c}{h} + \frac{3I_b}{L}\right)} \tag{7}$$

where I_b is the moment of inertia of the beam.

The lateral displacement of moment-resisting frame calculated as,

$$\Delta_f = \frac{F_f h^3(2I_c L + 3I_b h)}{12EI_c(I_c L + 6I_b h)} \tag{8}$$

Similar to braced frames, the elastic stiffness of the moment-resisting frame calculated by dividing Eq. (6) to Eq. (8).

$$K_f = \frac{12EI_c(I_c L + 6I_b h)}{h^3(2I_c L + 3I_b h)} \tag{9}$$

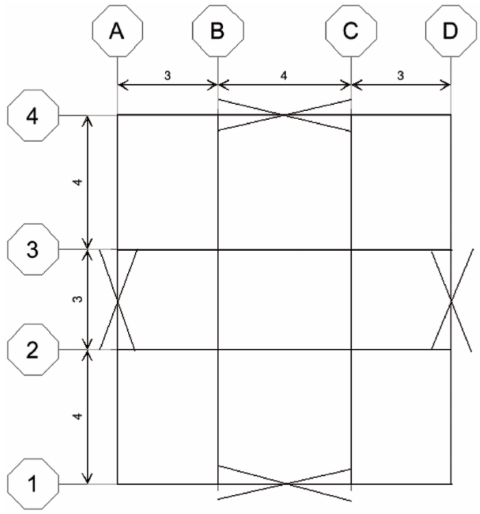
Alternatively, there is a user-friendly form of Eq. (9) in Eq. (10) (Silva and Badie 2008).

$$K_f = \frac{24EI_c}{h^3}\left(\frac{6\alpha + \beta}{6\alpha + 4\beta}\right) \qquad \alpha = \frac{I_b}{I_c} \quad \beta = \frac{L}{h} \tag{10}$$

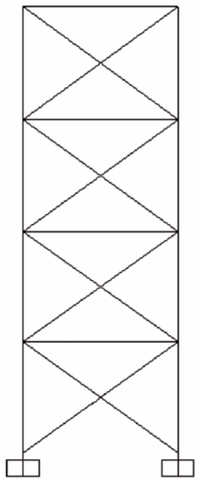
3. Design of frames

3.1 General design of buildings

A four-story building designed using Special concentrically braced frames (see Fig. 3). One of the braced frames on axis 1 (between axes *B* and *C*) selected and studied. The ASCE7-10 code used to evaluate the seismic design parameters for seismic design category D. Response modification coefficient (R_u), over-strength factor (Ω_0) and deflection amplification factor (C_d) considered equal to 6, 2 and 5 from Table 12.2.1 of ASCE7-10, respectively. The heights of the stories are 3.2 meter and concrete slab used for floors. European profiles of HEB, IPE and double channel shapes used for the design of columns, beams, and braces, respectively. The braces designed according to requirements of especial concentrically braced frames in AISC 341 (2010). Columns and braces selected as seismically compact sections. The beam to column connections considered as hinge (moment free) connections. Therefore, the braces designed for total story shear both in tension and compression. Braces' slenderness ratio limited to $KL/r \le 4\sqrt{E/F_y}$, where E is modulus of elasticity of steel, Fy is yield strength of steel, K is slenderness ratio (equal to 0.5 and 0.7 for in-plane and out-of-plane directions), L is free length of diagonals and r is gyration radius of braces section. The columns also checked for amplified earthquake forces using Load Combinations with over-strength Factor in section 12.4.3.2 of ASCE7-10. Equivalent static and scaled response spectrum analysis methods used for analysis and primary seismic design of frames.



(a) Typical building plan



(b) braced frame

Fig. 3 Plan and frame of specimens

3.2 Braced frame

The frames classified into two groups, named as F4A and F4B. The F4A group included continuous X-braces at

Table 1 Design of the braced frames

Story	Column	Beam	Braces	
			F4A	F4B
1	HEB260	IPE240	2UNP100	2UNP100
2	HEB200	IPE240	2UNP80	2UNP100
3	HEB160	IPE240	2UNP80	2UNP80
4	HEB100	IPE240	2UNP80	2UNP80

all stories. The size of members selected as close as possible to earthquake demand capacities. During the nonlinear dynamic analyses, some of the braces at the second story damaged severely by the superimposed forces because of removing the braces of the first story and the analyses became unstable. Therefore, a new group of braces introduced and named as F4B. In this group, the braces of the second story strengthened and selected similarly to the braces at the first story. It means the stiffness of the first two stories became equal. So, the only difference between the groups F4A and F4B is the size of bracing cross sections in the second story. The properties of the braced frames, F4A and F4B, depicted in Table 1.

3.3 Soft-story in first story

The story stiffness reduced and soft-story formed after removing the braces at first story in frame F4A. The brace at the soft-story replaced with a moment-resisting frame to study the replacement possibility. Therefore, the hinge beam-to-column connections of the story replaced with rigid ones. It is coming to be realized for high seismic applications where story drifts of 2-2.5% must be accommodated, frame distortion cannot be ignored (Thornton and Muir 2009). But in general, structural engineering practice, the beam to column connections considered as hinge connections. Therefore, special detailing needed to avoid distortional forces from moment connections due to gusset plates. In order to change this type of hinge connections to rigid ones, it may be used top and bottom plates on flanges. Of course, other type of detailing may be used for different details to convert them to rigid moment resisting connections.

After converting hinge connections to rigid ones, by increasing the size of beams and columns of the soft-story, the story stiffness increased gradually and continued until 70 percent of the second story stiffness (as needed by most of the building codes). In addition, the seismic performance of the moment resisting frame studied for stiffness's more than 70 percent of the second story.

According to section 12.14.4.2.2 of ASCE7-10 for vertical combinations of framing systems, different seismic force-resisting systems are permitted to be used in different stories. The value of R used in a given direction shall not be greater than the least value of any of the systems used in that direction. Therefore, the behaviour factor of Specially Concentrically Braced Frame used for design of all members in these frames.

The strong column-weak beam principle considered in the design of moment resisting frames in the first story. The

Table 2 Design of the braced frames

Frame class	Column	Beam	Relative stiffness (%) (1st/2nd)
F4A10 (F4B8)	HEB260	IPE240	10 (8)
F4A25 (F4B20)	HEB360	IPE300	25 (20)
F4A35 (F4B28)	HEB400	IPE330	35 (28)
F4A53 (F4B42)	HEB450	IPE400	53 (42)
F4A72 (F4B59)	HEB500	IPE450	72 (59)
F4A82 (F4B66)	HEB500	IPE500	82 (66)
F4A97 (F4B77)	HEB550	IPE500	97 (77)
F4A124 (F4B99)	HEB600	IPE550	124 (99)

Table 3 Properties of designed moment-resisting frames in the second story

Frame	Column	Cover Plates on Flanges	Beam	Relative stiffness percent (2nd/3rd)
F4C8	HEB260	-----------	IPE240	8
F4C28	HEB360	PL220X22	IPE400	28
F4C40	HEB400	PL220X30	IPE450	40
F4C50	HEB450	PL220X50	IPE450	50
F4C70	HEB500	PL220X70	IPE500	70
F4C79	HEB550	PL220X70	IPE550	79
F4C88	HEB600	PL220X70	IPE600	88

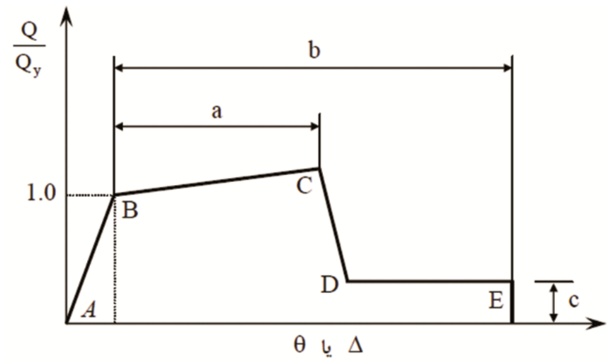


Fig. 4 Force-displacement curve (FEMA-356 2000)

properties of the designed moment resisting frames listed in Table 2. In name of the frames in column 1 of Table 2, the number after *F* shows the number of stories. Also, the last number (after letter "A") represents the percentage ratio of the first story stiffness to that of the second story. The properties of the beams and columns in the top stories are similar to the main braced frame. The numbers inside parentheses related to frame class F4B with similar bracings in first and second stories. Therefore, the stiffness ratio of the first story to the second story in the frames of the class F4B is less than that of class F4A.

3.4 Soft-story in second story

The similar procedure for soft-story at the first story followed to study the effects of brace elimination in the second story. The story stiffness compared with the third story and a soft-story formed by removing the braces in the second story. Then, the beam-to-column connections at the second story converted to rigid connections and the stiffness of moment-resisting frame in the second story increases gradually. The properties of replaced moment resisting frames in soft-story in the second story (class F4C) summarized in Table 3. The last number in name of frames in Column 1 of Table 3 represents the percentage ratios of the second story stiffness to the third story. Since the local strengthening of soft-story studied in this research, the columns strengthened using steel cover plates on flanges.

4. Analytical modelling

A two-dimensional frame used for nonlinear analyses. Plastic hinges provided at both ends of columns and beams and center of diagonals and defined using FEMA356 (2000) specifications. The fixed end connections in moment-resisting frames considered as rigid and their inelastic behavior are not considered. The plastic hinges created automatically by software and checked manually. The Force-displacement curve defined as shown in Fig. 4.

The behavior of member is linear between points *A* and *B*. Point *B* is the start of nonlinear behavior. The region between points *B* and *C* named as strain hardening and has plastic behavior with slope equal to three percent of first stiffness. The strength reduces between points *C* and *D*. The behavior is again nonlinear after point *D* (with reduced strength). Finally, the member fails in point *E*.

In nonlinear analyses, the gravity loads applied first and continued by applying the lateral loads.

5. Seismic loading

5.1 Nonlinear static analysis (pushover)

The lateral displacement applied incrementally, and the nonlinear static analysis performed in each step. The stiffness adjusted based on the deformed geometry in each step. In addition, the internal force of the members at the hinge points defined as the elastic-plastic behavior of materials according to FEMA 356 (2000) specifications. The loads increased gradually until the structure became mechanism or reached target displacement.

5.2 Nonlinear dynamic analysis (time history)

The nonlinear time history analysis is one of the most accurate analysis methods. The inelastic demands of structures evaluated accurately under earthquake records. It is a step-by-step analysis for estimating the dynamic response of structures under earthquake records. The form of solved equations in this analysis is

$$M.\ddot{U}(t) + C.\dot{U}(t) + K.U(t) = r(t) \tag{11}$$

where *K* is stiffness matrix, *C* is damping matrix, M is the diagonal mass matrix, *U(t)* is displacement vector, $\dot{U}(t)$ is velocity vector, $\ddot{U}(t)$ is acceleration vector and *r*(*t*) is the vector of applied loads on the structure.

The earthquake records shall represent the real motion of the ground at the site of construction as far as possible.

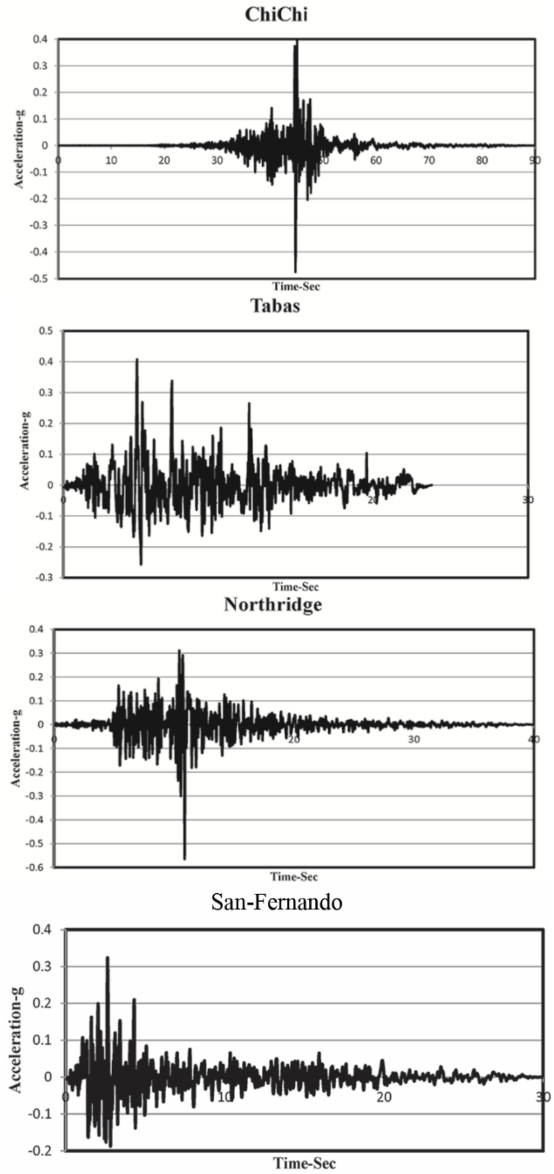
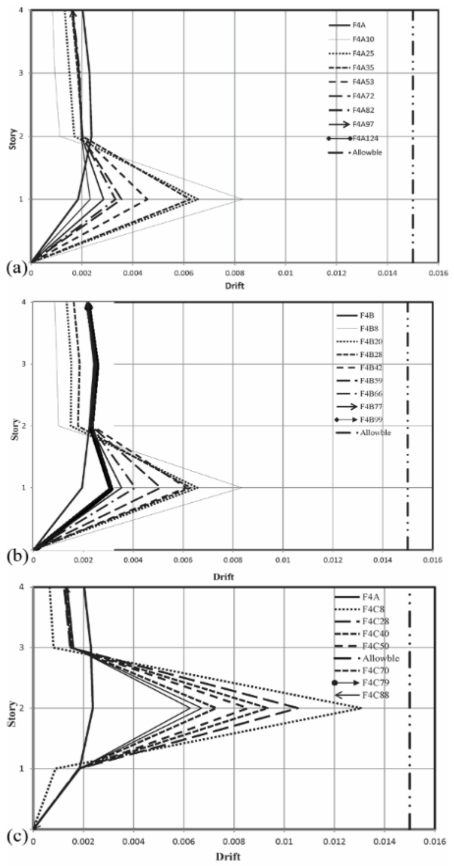


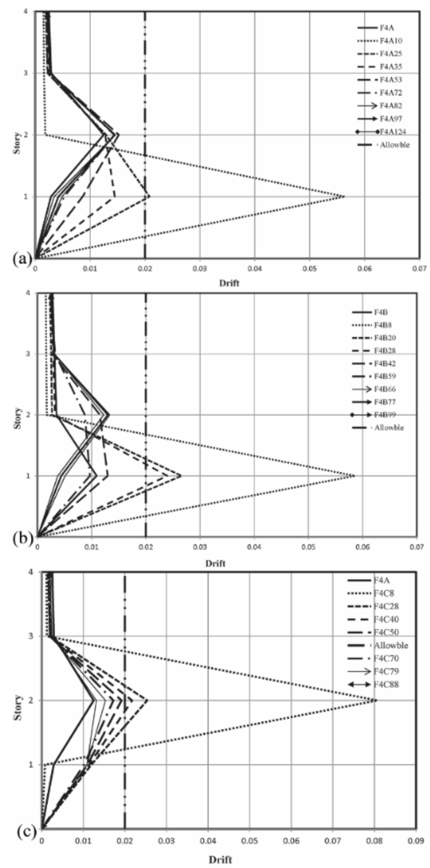
Fig. 5 Selected earthquake records for nonlinear dynamic analyses

Table 4 Design of the braced frames

Earthquake		Data	MW	P.G.A
Chi-Chi	N	1999/09/20	7.62	0.512 g
	W			0.474 g
Tabas	LN	1978/09/16	7.4	0.328 g
	TR			0.406 g
Northridge	090	1994/1/17	6.7	0.355 g
	360			0.563 g
San Fernando	021	1971/02/09	6.6	0.324 g
	291			0.268 g



Transient drifts



Permanent drifts

Fig. 6 Comparison of transient and permanent drifts for the frames of groups; (a) F4A, (b) F4B, and (c) F4C

Therefore, the requirements of ASCE7 (2010) satisfied for selection and scaling of records. Earthquake records of "Chi-Chi-Taiwan (1999), Tabas-Iran (1978), Northridge-USA (in 1994), and San Fernando-USA (1971), selected from the PEER database (see Table 4 and Fig. 5).

6. Seismic performance

6.1 Nonlinear static analysis

6.1.1 Transient drift

Drifts compared in two forms of transient and permanent according to FEMA 356 (2000) to evaluate the ability of the system to satisfy the design requirements based on the selected performance level. The performance level of the structure selected to have minimum human life loss in severe earthquakes, and keep the stability of building without any major structural damages in mild and moderate earthquakes. Therefore, the "life-safety" level used for buildings.

Transient drift defined as the maximum relative lateral displacement of stories, which predicted to develop in the building during the design earthquake. According to the FEMA 356 (2000), the allowable transient drift of the system is equal to 0.015. The transient drifts for each frame in comparison with the allowable value showed in Fig. 6. The transient drift of the soft-story is less than the allowable values. In addition, as expected, the increase in soft-story-stiffness using heavier columns and rigid beam-to-column connections reduces the story drift. It is remarkable that more stiffness of moment resisting frame if compared with the complete bracing frame, cannot lead to smaller transient drifts.

6.1.2 Permanent drift

Permanent drift defined as the maximum relative lateral displacement of stories from nonlinear analysis of frames. It often defined as the step after the target displacement point in the nonlinear static analysis. The target displacement serves as an estimate of the global displacement of the structure is expected to experience in a design earthquake. It is the roof displacement at the center of mass of the structure. According to the FEMA 356 (2000), the permanent allowable displacement of the X-Braced frame is equal to 0.02.

The permanent drifts of the frames of groups F4A, F4B and F4C at the target displacement and its comparison with the allowable values depicted in Fig. 6. According to This Figure, removing braces at the first and second stories leads to notable drift increase in the story. In addition, drifts reduce to the allowable value by the stiffness increase of 25, 20, and 40 percent in the frames of groups F4A, F4B and F4C, respectively. Also, the soft-story drifts were equal to early braced frame drifts, if the stiffness of groups F4A, F4B and F4C increased to 72, 66 and 88 percent, respectively.

6.1.3 Ductility

Ductility defined as the ability of a structure to withstandlarge deformations beyond the yield point without

Table 5 Ductility of the frames under nonlinear static analyses

Frame	μ	Frame	μ	Frame	μ
F4A	2.3	F4B	2.34	F4A	2.3
F4A10	3.65	F4B8	3.62	F4C8	3.47
F4A25	2.54	F4B20	2.43	F4C28	2.52
F4A35	2.45	F4B28	2.21	F4C40	2.48
F4A53	2.52	F4B42	1.95	F4C50	2.47
F4A72	2.59	F4B59	1.94	F4C70	2.4
F4A82	2.57	F4B66	1.92	F4C79	2.3
F4A97	2.53	F4B77	2	F4C88	2.43
F4A124	2.49	F4B99	1.88	----	----

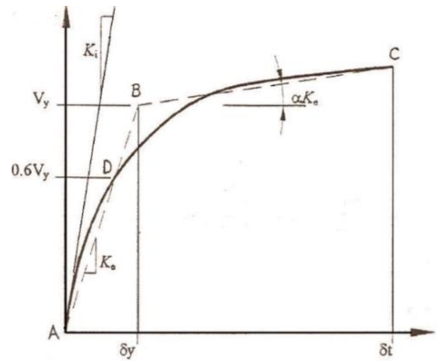


Fig. 7 pushover curve and equivalent bilinear diagram (ATC-40 1996)

failure (Eurocode-8 2003) and determined as

$$\mu = \frac{\Delta u}{\Delta y} \tag{12}$$

where Δ_u is the maximum displacement of pushover curve and Δ_y is the yield displacement of the frame.

Each frame has its own target displacement that calculated according to FEMA 356 (2000). The ductility of each frame calculated using the nonlinear static analysis and assumption of the final displacement equal to the target displacement (see Table 5). It found the ductility demand increased by 1.5 times that of the braced frame by replacement of braces in frame F4A with rigid beam-to-column connections and formation of soft-story at the second story. While the ductility demand increased by 1.6 times that of the braced frame by replacing braces in the first story of frame F4A with rigid beam-to-column connections. Frames F4A10 and F4C8 refer to moment resisting frame after removing braces in first and second stories, respectively.

Further stiffening of the soft-story using rigid beam-to-column connections and strengthening the members leads to gradually ductility decrease to reach the ductility of the original braced frame.

6.1.4 Energy absorption capacity

The energy absorption capacity calculated by measuring the area under the force-displacement curve, or the equivalent bilinear model. The equivalent bilinear model defined using two lines with a slope of K_e and αK_e for linear and nonlinear regions, respectively (see Fig. 7). The project of the intersection point of the lines on the vertical axis (base shear) is equal to V_y (ATC-40 1996).

Table 6 Energy absorption capacity of frames up to the target displacement

Frame	ED	Frame	ED	Frame	ED
F4A	89850	F4B	88620	F4A	89850
F4A10	269190	F4B8	268938	F4C8	303404
F4A25	164444	F4B20	144844	F4C28	136976
F4A35	152503	F4B28	138127	F4C40	132635
F4A53	119119	F4B42	145585	F4C50	128859
F4A72	102862	F4B59	123484	F4C70	124892
F4A82	100673	F4B66	117890	F4C79	116451
F4A97	94793	F4B77	116753	F4C88	103805
F4A124	88840	F4B99	105387	----	----

Dissipated energy in an inelastic system evaluated according to the area under the equivalent bilinear curve by Eq. (13) (ATC-40 1996).

$$E_D = 4\left(a_y d_{pi} - d_y a_{pi}\right) \tag{13}$$

where, a_y and d_y are the yield force and matching displacement of the structure; a_{pi} and d_{pi} are the maximum force and displacement of the push-over curve, respectively. Each frame has its own target displacement. The energy absorption capacity of each frame up to the target displacement point is given in Table 6. In this table, elimination of the braces and development of a soft-story at the second story increases the energy absorption demand up to 3.42 times that of the braced frame.

Comparison of the energy absorption capacities for the frames of series F4A and F4B with F4C shows the energy absorption demand in the frame with a soft-story at the second story, and the frames with a soft-story at the first story, are 3.42 and 3.00 times that in the braced frame, respectively. Therefore, brace elimination at the second story is more critical than the first story.

As shown in Table 6, increasing the stiffness of soft-story at the first story in models of series F4A and F4B, and soft-story at the second story in models of series F4C, decreases the energy absorption demand gradually and tends to that in the braced frame.

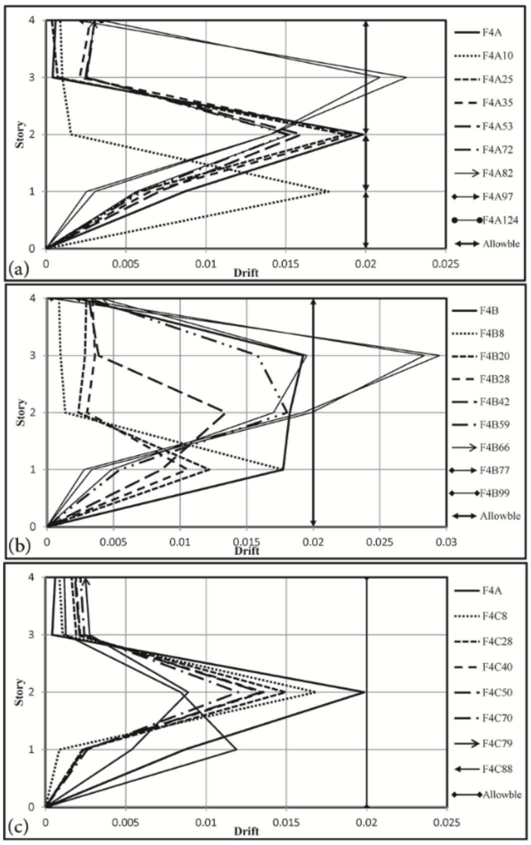
6.2 Nonlinear dynamic analysis

6.2.1 Maximum story drift

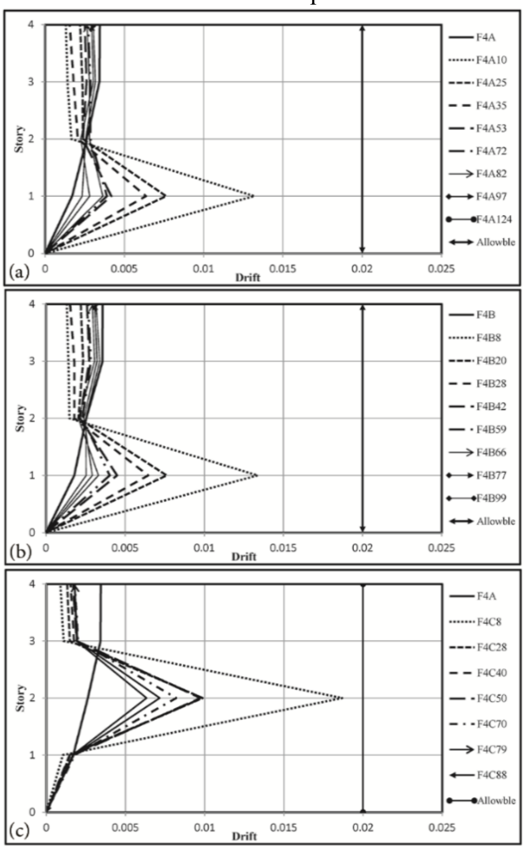
The maximum story drifts compared under all four-earthquake records (see Fig. 8-9).

Chi-Chi Earthquake

The natural period of frame F4B and the dominant period of Chi-Chi earthquake are equal to 0.439 and 0.44, respectively (see Fig. 8). Therefore, the closeness of structure and earthquake period causes resonance. Resonance has less effect in frame F4A with a period equal



Chi-Chi earthquake



Tabas earthquake

Fig. 8 Maximum drifts of stories under Chi-Chi and Tabas earthquakes for frames of groups; (a) F4A, (b) F4B, and (c) F4C

to 0.447 because of more difference between the dominant period of the earthquake record and the natural period of the frame. In addition, increasing the stiffness of the frames of series F4A decreased the drift of the first story. Considering a limit of 0.02 for drifts of the stories, based on FEMA 356 (2000) for the braced frames, drifts of the third stories exceeded the allowable values in frames with the stiffness of over 70 percent. This may be because of the absence of formation of any plastic hinges in the frame with soft-story at the first and second story or the resonance.

Increasing the second-story stiffness in frame F4B by using larger braces decreases the second-story displacements. In addition, the maximum drift of stories got closer to each other. However, the displacement values were still significant due to the resonance.

By increasing the stiffness of moment resisting frame in the second story to more than 80 percent (in frames of series F4C), the drift of the first story was a little more than the initial braced frame. But, the drift of the second story decreased significantly.

The drifts of the third and fourth stories in frames F4A, F4B, and F4C with similar stories at the initial braced frame compared in Fig. 8. It is clear that removing braces not only affects the drift of adjacent stories but also affects non-adjacent stories because of interaction behavior of near stories.

Tabas Earthquake

The analysis results of removing diagonals at the first and second stories depicted in Fig. 8 for F4A and F4B frames, respectively. The soft-story drifts increased to 7.7 and 7.2 times in the first and second stories of frames F4A10 and F4B8, respectively. Then, the story drift decreased by increasing the stiffness of the soft-story. But, never reached the drift of the initial braced frame.

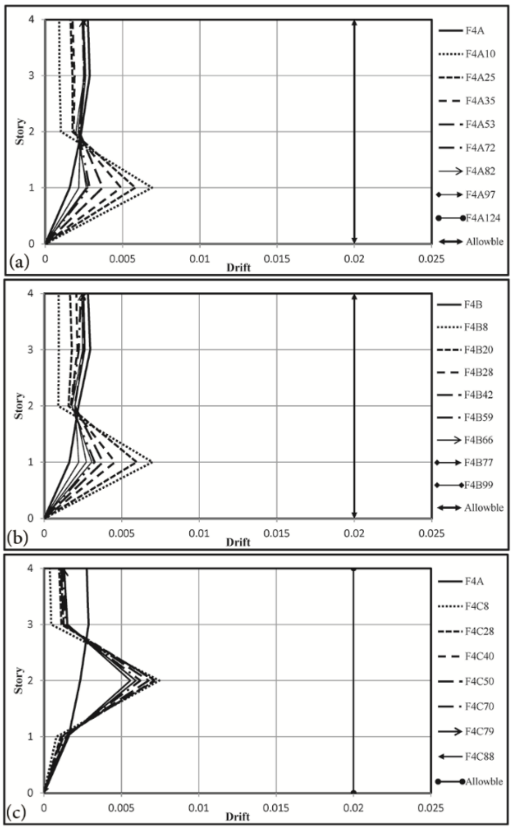
The drift of the second story (the soft-story) reduced to half of that for F4C8 frame by increasing the stiffness of the frames by 50 percent in frame F4C50. The increase in soft-story stiffness by more than 70 percent led to significant decrease in its drift, but did not reach the corresponding drift in the complete braced frame (F4A). In addition, removing braces at soft-stories decreased the drifts of the near and non-adjacent stories because of the concentration of earthquake energy in the soft-story.

Northridge Earthquake

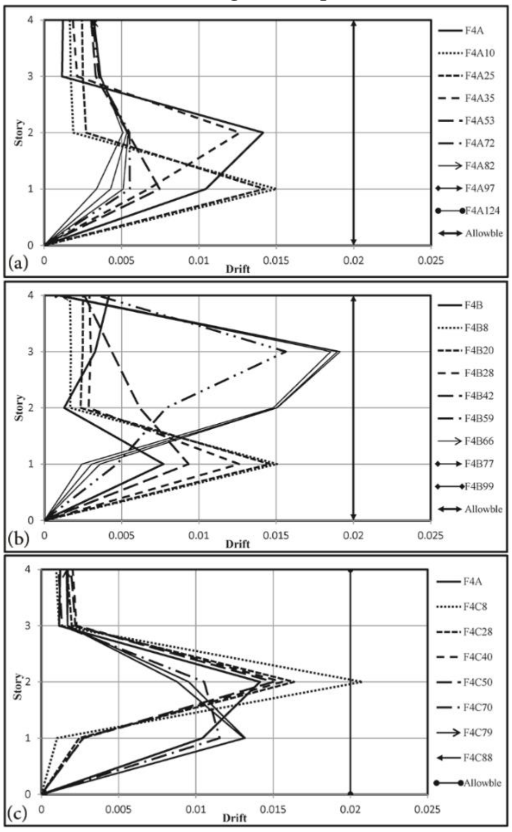
The effects of removing the braces at the first or second stories under Northridge earthquake depicted in Fig. 9. The drifts of soft-stories in the first and second stories increased to 4.3 and 3.2 times, respectively. Similar to Tabas earthquake, removing braces in the soft-stories decreased the drifts at other stories, even less than drifts in the complete braced frame.

San Fernando Earthquake

The braces at the first or second stories removed and analysed under San Fernando earthquake (see Fig. 9). The corresponding drifts in soft-stories increased to 2.0 and 1.5 times that for the complete braced frame, respectively, and even exceeded the allowable drift limit. The soft-stories' drift decreased significantly by increasing the stiffness of the soft-stories up to 70 percent. After that, the plastic hinges formed in near stories. As a result, drifts of the near



Northridge earthquake



San Fernando earthquake

Fig. 9 Maximum drifts of stories under Northridge and San Fernando earthquakes for frames of groups; (a) F4A, (b) F4B, and (c) F4C

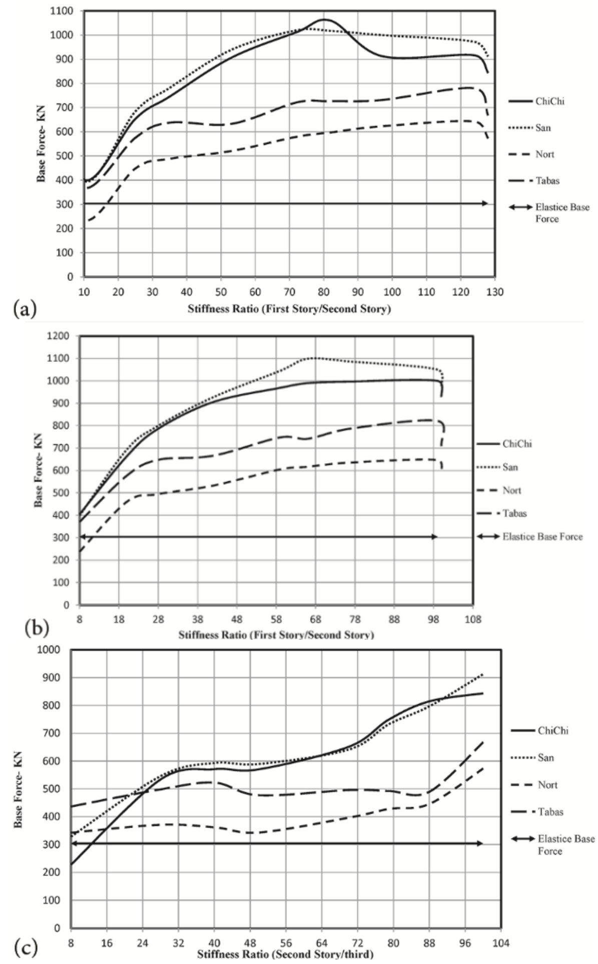


Fig. 10 Comparison of the base shears under earthquake records for frames of groups; (a) F4A, (b) F4B, and (c) F4C

stories became more than that for the complete braced frame. Also, the drifts of non-adjacent stories were slightly more than that for the complete braced frame.

6.2.2 Maximum base shear

The base shear of the frames under selected earthquake records of Chi-Chi, San Fernando, Tabas, and Northridge compared in Fig. 10. The maximum story shear capacity decreased in all records after removing braces in the soft-story. Particularly, it was less than the calculated base shear using equivalent static analysis in Northridge record with the soft-story in the first story and in Chi-Chi record with the soft-story in the second story.

The base shear capacity increased by stiffness growth of the first story to 70 percent of that in the second story under two records of Chi-Chi and San Fernando. It decreased with a mild slope for frames with larger stiffnesses. Similarly, the maximum base shear remained unchanged by increasing the stiffness of the first story up to 80 percent of that in the second story under two records of Northridge and Tabas. It did not increase in frames with larger stiffnesss.

Increasing the relative stiffness of the second story similar to that of the third story in frame class F4C by 28 percent (F4C28) increased the base shear by 2.35, 1.66, 1.14, and 1.08 times under Chi-Chi, San Fernando, Tabas, Northridge records, respectively, compared to frame F4C8. But, it was less than the equivalent base shear in the complete braced frame.

7. Conclusions

In this research, four story steel frames designed and analyzed under nonlinear static and dynamic analyses and the effects of removing braces in lower stories studied. The replacement of X-braces in soft-stories with moment resisting frames, having different stiffnesses investigated as a retrofit procedure. The “soft-story” developed in the first or second stories by removing diagonals. The simple frame converted to a moment resisting frame by replacing beam-to-column connections of the soft-story with rigid connections. The nonlinear static and dynamic analyses employed to study seismic performance of converted moment resisting frames with different stiffnesses.

Stiffening of the soft-story using rigid beam-to-column connections and strengthening the members decreased ductility gradually to reach the ductility of the original braced frame. In addition, the energy absorption demand gradually decreased and tended to that in the braced frame. The nonlinear dynamic analyses showed that removing braces and development of soft-story in the second story was slightly more critical than that in the first story due to the interaction of near stories.

The base shear of studied moment resisting frames increased gradually by increasing the stiffness of moment frame. But, it remained almost constant at stiffnesses higher than 70 percent of that in the upper story. Therefore, the seismic provisions of building codes are sufficient for seismic design of soft stories in low-rise buildings. However, the drift of equivalent moment resisting frame with the similar stiffness of the complete braced frame was more than the braced frame.

According to findings of this research, the soft-story at the braced frame of low-rise buildings may be retrofitted using equivalent moment resisting frame with a minimum stiffness of 70 percent of the stiffness of the complete braced frame.

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