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Effect of Earthquake characteristics on seismic progressive collapse potential in steel moment resisting frame

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Abstract. According to the definition, progressive collapse could occur due to the initial partial failure of the structural members which by spreading to the adjacent members, could result in partial or overall collapse of the structure. Up to now, most researchers have investigated the progressive collapse due to explosion, fire or impact loads. But new research has shown that the seismic load could also be a factor for initiation of the progressive collapse. In this research, the progressive collapse capacity for the 5 and 15-story steel special moment resisting frames using push-down nonlinear static analysis, and nonlinear dynamic analysis under the gravity loads specified in the GSA Guidelines, were studied. After identifying the critical members, in order to investigate the seismic progressive collapse, the 5-story steel special moment resisting frame was analyzed by the nonlinear time history analysis under the effect of earthquakes with different characteristics. In order to account for the initial damage, one of the critical columns was weakened at the initiation of the earthquake or its Peak Ground Acceleration (PGA). The results of progressive collapse analyses showed that the potential of progressive collapse is considerably dependent upon location of the removed column and the number of stories, also the results of seismic progressive collapse showed that the dynamic response of column removal under the seismic load is completely dependent on earthquake characteristics like Arias intensity, PGA and earthquake frequency contents.

Keywords: progressive collapse; push-down nonlinear static analysis; nonlinear dynamic analysis; seismic load; earthquake frequency content; arias intensity

1. Introduction

According to the definition, progressive collapse is extension of initial local failure due to an external factor, which has caused initiation of failure from one element to the other, and ultimately results in total collapse of structure or a large portion of it. The abnormal loads which could result into the progressive collapse include airplane crash, error in design or construction, fire, gas explosion, hazardous material, crash of vehicles, bomb explosion etc. which because of low probability of occurrence are not considered in the design of structures (NIST 2007). The first collapse which attracted attention of the researchers towards the nature of the problem and its features was the collapse of 22 story Ronan Point apartment in London. Though it had not many fatalities but it was important from the historical point of view, and it was the turning point in the research activities related to the progressive collapse. But, after the terrorist event of 11 September 2001, more researchers have begun studying this issue.

Recently some methods have been presented for enhancement of the strength, ductility and continuity of the existing and new buildings under the progressive collapse by GSA (2003) and DOD (2009) Guidelines. The Alternate Path Method was utilized by these Guidelines. According to this method, if the structure has enough load paths the initial failure of the members would not extend to the other members and the local damage becomes limited. The Alternate Path Method is a threat -independent method and does not consider the reasons for initiation and extension of the collapses, but instead takes into the account the structure response after the column removal. In this method the analysis is performed in 4 steps: 1) removal of the column and performing the analysis, 2) checking the allowable values for the elements, 3) if the allowable values are not satisfied for a certain member it would be removed and in the next analysis the re-distribution to the adjacent elements is permitted 4) the above steps are iterated till no member exceeds the allowable determined values. The analysis by Alternate Path Method, which is mentioned in the Guidelines is performed by 4 methods: 1-linear static, 2-linear dynamic, 3-nonlinear static, 4-nonlinear dynamic.

Kim and Kim (2009) assessed the steel moment resisting frames strength against the progressive collapse. They used the Alternate Path Method, proposed in GSA and DOD Guidelines, they showed that the results considerably change due to changes in the applied load, location of the removed column and number of the stories. Also they found that the linear method presents more conservative results than nonlinear analyses. Fu (2009) investigated a 20-story building using the finite element method against the progressive collapse. In this research nonlinearity of the material and the geometric nonlinearity were assumed in the

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procedure. For simulation of progressive collapse one of the columns was removed. They stated that dynamic response of the structure is basically dependent on the amount of absorbed energy by the structure after sudden removal of the column. Also it became clear that the structure response was considerably dependent upon the location of the removed column. Khandelwal *et al*. (2009) studied the potential of progressive collapse in the steel braced frames, they noted that the frames with EBF bracing had better performance in the progressive collapse with respect to the frames with CBF bracing. An optimized novel design method also presented by Haddadi *et al*. (2016) for steel frames subjected to progressive collapse. Kim *et al*. (2011) showed that adding rotational frictional dampers to the structural system, in addition to reducing structural response during the earthquake, would increase the resisting capacity of the structure against the progressive collapse. The progressive collapse capacity of steel moment-resisting frames with different number of floors was investigated using the alternate path method by Tavakoli and Kiakojouri (2013), then three suggestions are made for assessment of structural robustness. Most progressive collapse analyses are performed using the Alternate Path Method. This method is not dependent on the cause of collapse initiation, but in recent years further research works have been conducted on the progressive collapse, which incorporate cause of the collapse initiation, like progressive collapse due to fire (Usmani *et al*. 2009, Sun *et al*. 2012, Tavakoli and Kiakojouri 2015), Seismic progressive collapse (Parsaeifard and Nateghi-A 2012, Tavakoli and Rashidi 2013, Karimiyan *et al*. 2013, Tavakoli and Kiakojouri 2012, Karimiyan *et al*. 2014, Tavakoli and Akbarpoor 2014) and progressive collapse due to blast load (Almusallam *et al*. 2010, Tavakoli and Kiakojouri 2013).

Stratosk (2007) categorized progressive collapse into 4 classes and 6 types. He explained the possible mechanism for the event of progressive collapse in the structures.

Although up to now, many research works have been performed on the progressive collapse under the gravity loads But it is only in recent years that the topic of seismic progressive collapse has attracted much attention. Strong earthquakes induce large lateral loads in the members and form extra stresses in them. Consequently it might cause weakness in one or more load bearing structural members which through re-distribution of loads in the failed elements would cause extension of damage in the intact members and formation of the progressive collapse. Wibo and Lau (2009) demonstrated that each abnormal loading which causes local damage and large deformations and considerable stress concentrations in the structural members, could be a cause of the progressive collapse initiation. For this reason they stated that the seismic load also, could through local failures and stress concentration in special members, cause initiation of progressive collapse, also they stated that analysis of the seismic progressive collapse could be performed by modification in the available analysis methods for the progressive collapse.

Also by detailed study on the possible mechanisms of progressive collapse and types of collapses due to the earthquake, it could be concluded that some of these failures could occur during the earthquakes. Also due to such factors as instantaneous failure of the infilled frames, existence of the short column, torsion in the building or weakness in design or construction during the earthquakes, some special members may undergo damage earlier and thus cause re-distribution of the loads to the adjacent members which leads to the progressive collapse initiation.

Lu *et al*. (2013) studied the possible loss mechanisms in the high rise buildings due to the severe earthquakes. The results of their study showed that due to various factors in the severe earthquakes, the structural members may undergo Buckling or by falling of the higher stories onto the lower ones cause collapse of the entire structure. This type of the progressive collapse during the earthquake is known as the pancake-type collapse. Lu *et al*. (2013) also assessed the damage mechanisms associated with severe earthquakes, they showed that the most important type of damage in the high rise structures could be the pancake-type progressive collapse. They also showed that due to various earthquakes, such factors as crushing of the concrete shear wall and other factors cause transfer of extra loads to other structural elements and extension of damage to some of the vertical load bearing elements. Therefore it could be concluded that even a case of re-distributed progressive collapse of the zipper-type could occur. Even though the underlying mechanisms of seismic progressive collapse event are complicated and may a combination of multiple collapse types occur during the earthquake. Tavakoli and Rashidi (2013) studied potential of the progressive collapse for the steel moment resisting frame under lateral load for various column removal locations. For this purpose, 2D and 3D pushover analyses were performed. The results showed that by increase in the number of spans and stories, the structure capacity to resist against progressive collapse under lateral loads, increases. Karimian *et al*. (2013) have investigated the effect of eccentricity and lack of symmetry in the plan, on potential of the progressive collapse under the seismic loads. For this purpose, ordinary RC moment resisting frames with various levels of eccentricity were analyzed by the time history method.

Tavakoli *et al*. (2015) investigated the effect of using base isolation on performance of concrete moment resisting frames against the progressive collapse, under the gravity loads and Seismic load. They found that using the base isolation has not a significant effect on increase in the structure strength against the progressive collapse under the gravity loads, but causes that damages be localized and prevents their extension to other intact spans under the seismic loads.

PGA, frequency content and Arias intensity are among the most important characteristics of ground motions. Therefore one could expect that structural dynamic response of the structures under various earthquakes be completely different. Lu *et al*. (2013) showed that earthquakes with different frequency contents induce different modes and mechanisms of damage in the structural members. Takewaki *et al*. (2011) evaluated performance of the structures against Pacific Coast Earthquake in the year 2011 in Japan. They stated that while there are advanced seismic design codes in Japan, the high rise buildings have suffered considerable damages. This is due to the high predominant period of earthquake and closeness to the fundamental period of the tall buildings. Cakir (2012) has evaluated the effect of earthquake frequency content on the seismic behavior of the cantilever retaining walls. They found that dynamic behavior of the cantilever retaining wall is considerably dependent on the characteristics of the earthquake record and soil and structure interaction.

Previous research works haves shown that the progressive collapse could occur under the effect of seismic loads. Therefore in recent years much attention has been attracted to the seismic progressive collapse, but most of the researcher have concentrated on the simulation of seismic loads using the static analysis method while both the progressive collapse and earthquake have dynamic nature. On the other hand earthquakes possess various characteristics and these could not be taken into the account through static analysis. Also column weakening at the PGA of the earthquakes is not possible using the static methods, therefore there is need for further investigation of the seismic progressive collapse under the nonlinear time history analysis. In this article first the potential of progressive collapse of 5 and 15-story moment resisting frames in different position of column removal under the gravity loads specified by the GSA Guidelines is investigated. Then the effects of different characteristics of the earthquakes on the seismic progressive collapse for critical locations of column removal have been evaluated.

2. Descriptions of modeling

2.1 Building modeling

In this article in order to investigate the progressive collapse potential, 2 Dimensional 5 and 15-story steel special moment resisting frames were considered. The plan has four spans with 5 meter in two directions. The height of all stories were assumed to be equal to 3.3 m. Fig. 1 and Fig. 2 show the plan and elevation of the studied buildings.The sections of beams and columns were designed by using Sap2000 software and Iranian building codes and the equivalent static method in a way that could bear the gravity and seismic loads and satisfy the seismic criteria of stress ratio and the drift allowable limits. The properties of the used sections are shown in Tables 1-2. The site is located in the area with high seismicity and type 3 soil according to the Iranian seismic code 2800. All beam-to-column connections were assumed to be rigid.

To perform the required analyses corresponding to the progressive collapse, the finite element software package ABAQUS VER. 6.13 was utilized. For modeling of the beams and columns the BEAM element in the Elements library of ABAQUS was used. The BEAM elements in the ABAQUS software are beam-column elements and the axial, shear and torsional deformation are allowed in them. The effects of mesh size are studied and the appropriate mesh size which could maintain accuracy of the modeling was selected. In the dynamic analysis a Rayleigh damping Equal to 5% and proportional to the mass and stiffness was taken in modes which have the largest effect on the structural response. The properties of used steel materials are given in Table 3. In the ABAQUS software the elastic zone is defined by Young Modulus and Poisson ratio. In the ABAQUS software, the plastic portion is determined by the true stress and logarithmic plastic strain. The materials behave linearly and elastically up to the yield stress. After this stage they enter the strain hardening stage till reaching the ultimate stress. The plastic properties are shown in Fig. 3 (Tavakoli and Kiakojouri 2014).

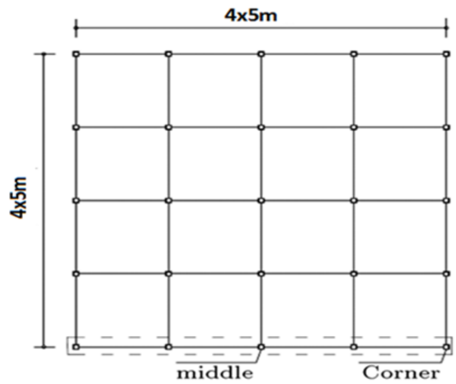
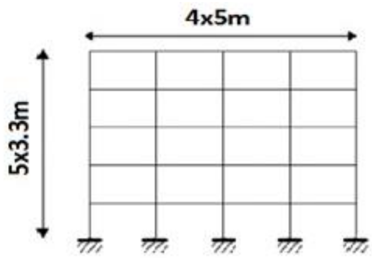
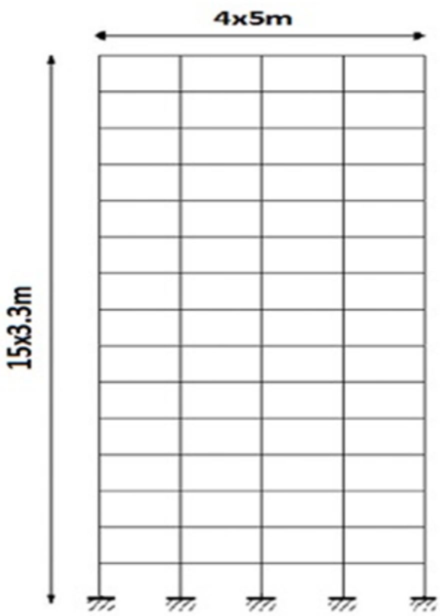


Fig. 1 Typical plan



(a) Elevation of 5-story



(b) Elevation of 15-story

Fig. 2 Elevation of model structures

Table 1 Member size of 5-story analysis model structures

Story number	beams	columns
1-4	W 12×19	Box 35×35×1.0
5	W 10×17	Box 25×25×1.0

Table 2 Member size of 15-story analysis model structures

Story number	beams	columns
1-5	W18×35	Box 55×55×2.0
6-10	W18×35	Box 45×45×1.5
11-13	W12×19	Box 40×40×1.5
14	W10×17	Box 35×35×1.0
15	W10×17	Box 35×35×1.0

Table 3 Properties of steel material

Material	Modules of Elasticity [Gpa]	Poisson coefficient	Density [kg/m^3]	Yielding Stress [MPa]
Steel	210	0.3	7850	240

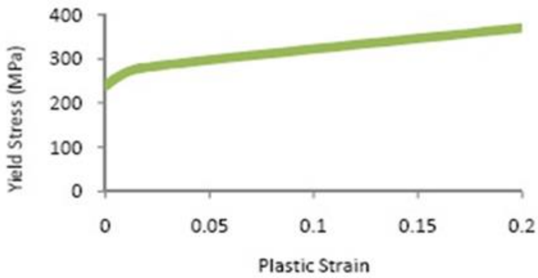


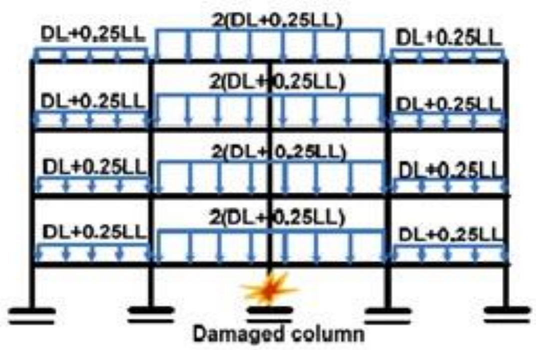
Fig. 3 Plastic property

3. Methodology of analysis

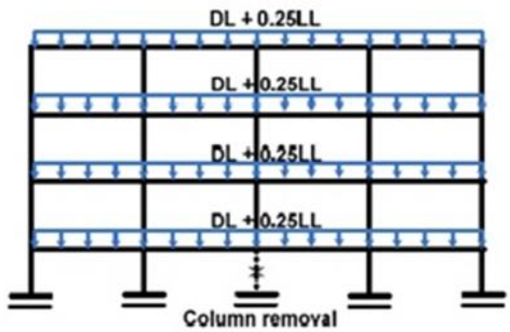
In this paper, first in order to investigate the potential of progressive collapse under the gravity loads, and under different column removal scenarios, the push-down nonlinear static and nonlinear dynamic analyses were performed. The various column removal cases examined in this paper are explained in Table 4. The gravity loads in these analyses were considered according to GSA 2003 guidelines. For conducting push-down nonlinear static analysis the loading: 2 (Dead Load+0.25×Live Load) was applied on the removed spans and the above loading without magnification factor 2 was applied on the other spans (as shown in Fig. 4(a)). In the push-down static analyses, the progressive collapse resisting capacity is determined based on the load factor. In this method, the gravity loads were increased step by step till the vertical displacement of removed column location reached 20 cm (Tsai and Lin 2008). At each step of analysis, the load factor is the ratio of the applied equivalent gravity load to the specified load in the GSA guideline. Also to perform dynamic analyses the gravity load: (Dead Load+0.25×Live Load) was applied to all spans uniformly (as shown in Fig. 4(b)). In the nonlinear dynamic analyses in order to apply the effects of dynamic column removal the axial force, shear force and bending moment of the column before removal were calculated then the column was replaced by concentrated loads equivalent to the member forces. For simulation of sudden removal of column, the concentrated equivalent load after 7 second were suddenly removed according to the short time specified by the GSA Guidelines as shown in Fig. 5 (as a work of Kim *et al*. 2011).

Table 4 Column removal analysis cases

Case	frame	Story	Column
1	5 story	First	Middle
2	15 story	First	Middle
3	5 story	First	Corner
4	15 story	First	Corner
5	5story	Fourth	Corner
6	15story	Tenth	Corner



(a) Nonlinear static method (GSA 2003)



(b) Nonlinear dynamic method (GSA 2003)

Fig. 4 Applying gravity loads in progressive collapse analysis as GSA 2003

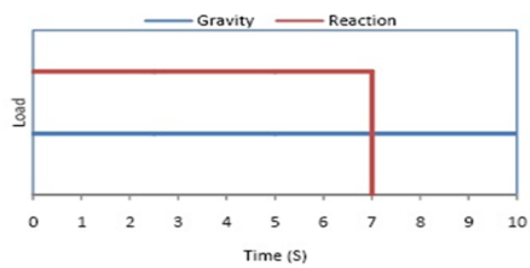


Fig. 5 Function of column removal in dynamic analysis

For conducting the seismic progressive collapse analyses the same gravity loads specified by GSA code were applied for dynamic analyses. In addition to the gravity load, the seismic loads were applied in the form of acceleration history to the base of the frames. For simulation of critical columns removal under the pre-seismic or mainshock it was assumed that the column load bearing capacity at the beginning of the earthquake or at PGA reaches 20% of the initial value, because usually due to weakness in construction or design or other factors, during the earthquake the columns are not completely removed. This method presents a more realistic analysis of the seismic progressive collapse. Also for investigation of the effect of earthquake characteristics in the seismic progressive collapse, earthquakes with different characteristics were assumed. The characteristics of the used earthquakes in the seismic analyses are given in Tables 5-8.

Table 5-Group A Earthquakes

Record	Station	Magnitude	Shear Wave Velocity (m/s)	Arias Intensity-Scaled to 0.35 g (m/s)	Arias Intensity-Scaled to 0.5 g (m/s)	PGA/PGV	Predominant Period(sec)-Mean Period(sec)
Superstation hill 1987	El centro imp	6.54	192.05	1.05	2.14	0.78	0.46 0.97
Taiwan SMART1986	SMART1 O08	7.3	357.43	2.42	4.94	0.76	0.42 1.01
Chichi-taiwan 1999	Chy-088	6.2	318.52	3.30	6.74	0.76	0.44 0.98
Coalinga-1983	Parkfield	6.36	178.27	3.88	7.92	0.78	0.48 1.02

Table 6-Group B Earthquakes

Record	Station	Magnitude	Shear Wave Velocity (m/s)	Arias Intensity-Scaled to 0.35g(m/s)	Arias Intensity-Scaled to 0.7g(m/s)	PGA/PGV	Predominant Period(sec)-Mean Period(sec)
Chalfant valley-020.97	Mcgee creek surface	6.19	359.23	1.13	4.55	3.88	0.06 0. 15
Taiwan SMART1(33)	SMART1 I04	5.8	314.88	1.13	4.54	2.05	0.18 0.31
Taiwan SMART1(5)	SMART1-M07	5.9	327.61	1.12	4.48	1.02	0.34 0.6
Northen - Calif-03	Ferndale City Hall	6.5	219.31	1.14	4.59	0.79	0.8 1.06

Table 7-Group C Earthquakes

Record	Station	Magnitude	Shear Wave Velocity (m/s)	Arias Intensity-Scaled to 0.35 g (m/s)	Arias Intensity-Scaled to 0.7 g (m/s)	PGA/PGV	Predominant Period(sec)-Mean Period(sec)
Lytle creek	LA Hollywood-Stor FF	5.33	316.46	1.18	4.75	2.18	0.24 0.29
Northridge	Carson - catskill ave	6.69	305.14	1.20	4.80	1.45	0.4 0.52
Taiwan smart 1	Smart 1 c00	6.32	309.41	1.18	4.74	0.57	0.8 1.1

Table 8-Group D Earthquakes

Record	Station	Magnitude	Shear Wave Velocity (m/s)	Arias Intensity-Scaled to 0.35g(m/s)	Arias Intensity-Scaled to 0.7g(m/s)	PGA/PGV	Predominant Period(sec)-Mean Period(sec)
Mammoth Lakes	Mammoth Elem school	4.85	350.54	1.66	6.64	2.6	0.12 0.2
Humbolt Bay	Ferndale City Hall	5.8	219.31	1.64	6.54	1.05	0.42 0.59
Taiwan Smart1	Smart 1 O01	6.32	267.67	1.7	6.86	0.45	0.66 1.04

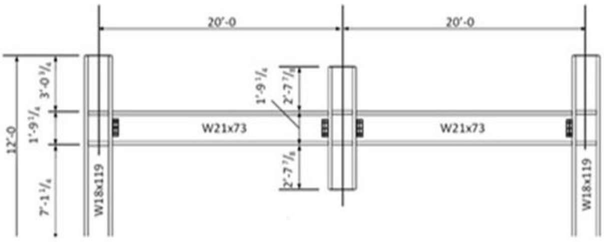


Fig. 6 Experimental configuration utilized in NIST testing program

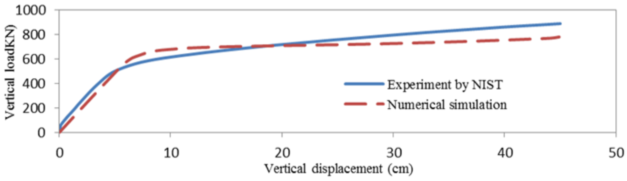


Fig. 7 Comparison of NIST experiments with numerical simulation

In order to validate the modeling, one 2D Experimental steel model was selected. In this Experimental model, Sadak *et al*. (2010), put a 2D steel frame under the column removal scenario and using the hydraulic ram they applied the load until occurrence of the collapse. This Experimental model was used in this article for validation (as shown in Fig. 6). This Experimental model was modeled in the ABAQUS software and the load-vertical displacement (push-down) diagram was compared to the Experimental results (as shown in Fig. 7). The difference between the Experimental and numerical results varies from 0-11% which demonstrates good compatibility between the numerical simulation and the Experimental results.

4. Progressive collapse-resisting capacity

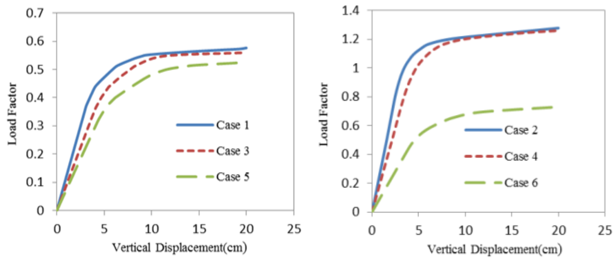
4.1 Push-down nonlinear static analysis

In order to investigate structure resistance against the progressive collapse, push-down nonlinear static analysis was conducted using the displacement-control method and maximum displacement of 20 cm at the location of column removal. In this method the gravity loads were increased till the vertical displacement of 20 cm was attained at the location of column removal. As shown in Figs. 8(a)-(b) due to greater number of load transfer paths of damaged column, by the removal of the interior column, the load factor is larger than the time where the corner column was removed. Also by comparing the load factors for the 5 and 15 -story frames, it could be concluded that increase in the number of stories for similar reasons causes increase in the load factor and consequently increase in the resistance against the progressive collapse.

4.2 Dynamic removal of column

In this section, the maximum rotation of beams for different cases of column removal, were assessed under nonlinear dynamic analysis, by applying the specified vertical load in the GSA Guidelines. The maximum rotations of beams for different locations of column removal are shown in Table 9. By increase in the number of stories, due to increase in the structural Redundancy and participation of more elements to create alternate load paths, the structural response decreased. Also the corresponding values of beam rotations in the structures in which the corner column was removed, increased with respect to those with interior column removal, because there is lower number of alternate paths for transfer of removed column loads after removal of the corner column with respect to the interior column. Also the rotation values are higher with column removal at the 10th story with respect to the column removal at the first story. By Comparing the rotations values with the allowable values in GSA it could be concluded that the special moment resisting frames with 5 and 15 stories resist against the progressive collapse under the gravity load, because the allowable rotation value in the GSA is equal to 0.21 radians and in no case neither in the 5-story nor in the 15-story frames, the rotations did exceed the allowable rotation values.

Also, the 5-story frame is more susceptible to the progressive collapse than the 15-story frame. moreover in both 5 and 15- story frames, for the corner columns, the probability of progressive collapse occurrence is higher due to the fewer available paths for transfer of damaged column load. The results of nonlinear dynamic analysis are consistent with those of the push-down nonlinear static analysis. With respect to the push-down static and column removal dynamic analyses, it became clear that while the 5 and 15- story steel special moment resisting frame systems are safe against the progressive collapse under the gravity loads, but for some column removal locations, larger



(a) 5 story (b) 15 story

Fig. 8 Push down Load-displacement curves of 5 and 15 story in the removal of the corner and middle columns

Table 9 Maximum rotation of beam in removal span for different cases

Case	Maximum rotation in removal span(rad)
1	0.045
2	0.0071
3	0.057
4	0.0078
5	0.089
6	0.028

rotations are obtained. Cases 3 and 5 which depict the location of removed corner columns at the first and fourth stories of the 5-story frame have the lowest load factor and largest rotations. Therefore the progressive collapse analyses under the seismic loads were conducted for these two special cases.

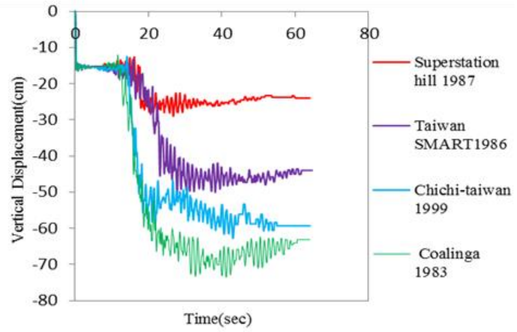
4.3 Seismic progressive collapse

As mentioned before, the seismic load also could cause initiation and progress of damages. On the other hand both the earthquake and progressive collapse have dynamic nature and for more precise examination of the structure behavior against the seismic progressive collapse there is need for time history nonlinear dynamic analysis.

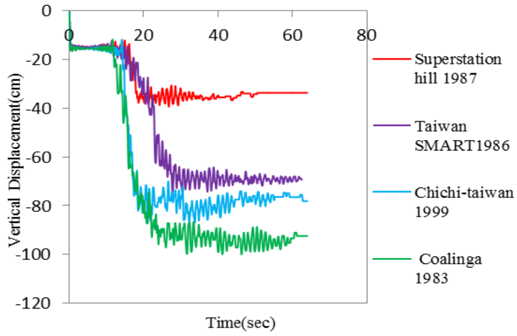
In the previous section the potential of progressive collapse was investigated for different locations of column removal and the critical locations of column removal under the gravity loads were identified. In this section the effect of different earthquake characteristics on the seismic progressive collapse is assessed. For this purpose first it was assumed that due to weakness in construction or design or other Factors and under the pre-seismic event or previous events, the intended critical column became weak so that its load bearing capacity was reduced to its 20% value. Then at the final section it was assumed that the intended column is weakened during the earthquake at PGA.In this paper, in the seismic progressive collapse results, the words “removed column” are used to refer to the “weakened column”.

4.3.1 Column removal at the earthquake initiation

PGA, Arias intensity and frequency content are among important characteristics of the earthquakes. In this section the effect of Arias intensity on the vertical displacement of the location of column removal was investigated. Arias intensity is defined as the time-integral of the square of the ground acceleration. Also according to the empirical definition, arias intensity is an earthquake parameter in which the effects of distance from the fault and earthquake magnitude and some other factors are considered in it simultaneously (Travasarou *et al*. 2003, Gómez-Bernal *et al*. 2012). In this section in order to study the effects of Arias intensity on the seismic progressive collapse, earthquakes with completely similar frequency contents of type 3 soil (according to the Iranian 2800 code) were scaled to the two equal PGAs of 0.35 g and 0.5 g. As shown in Figs. 9(a)-(b), the horizontal vibration of the earthquake caused increase in the vertical displacement of the structure at the location of column removal with respect to the column removal under the gravity load alone, According to Fig. 9(a), where the column is removed prior to the initiation of earthquake and only under the gravity load, the vertical displacement at the column removal location is about 17 cm, but in continuation by addition of the seismic load, the vertical displacement exceeds 17 cm which also demonstrates the effect of seismic load upon increased displacement under the gravity load only, the main reason for it, is the flexural yielding of the beam due to the column removal. The horizontal forces of earthquakes cause increase in the internal forces of the members and due to the



(a) scaled to 0.35 g



(b) scaled to 0.5 g

Fig. 9 Comparison of Vertical displacement at the point of column removal for case 5 under the effect of group A earthquakes

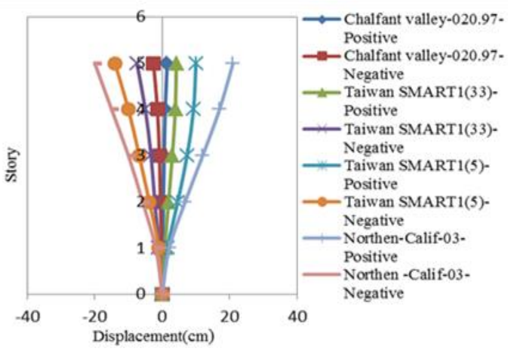
yielding of the beam cause increase in the vertical displacement of the beam. Figs. 9(a)-(b), also show that Arias intensity is among the most influential factors in increase in the vertical displacement at the column removal location. By increase in Arias intensity, the input energy of the structure increases and for this reason the ultimate vertical displacement increases at the column removal location. Also the results show that the earthquakes with higher PGA did not always induce higher vertical displacements because peak ground accelerations mainly affect response amplitude and Arias intensity reveals the ground motion intensity. For example vertical displacement at the corner column removal location of the forth story for Superstation Hill Earthquake after scaling to 0.5 g, reached 33 cm, but for Taiwan Smart 1986 Earthquake after scaling to 0.35 g reached 43 cm, therefore it could be concluded that the effect of Arias intensity on the seismic progressive collapse, is more than PGA.

The frequency content of the earthquakes and natural period of structures are considerably effective on the dynamic response of the structures. The frequency content of the earthquakes is defined by various methods of which the most important are based on the predominant period and the PGA/PGV ratio and other parametrs (Kramer 1996, Tso *et al*. 1992). Earthquake frequency content can be classified based on PGA/PGV ratio:high PGA/PGV ratio when PGA/PGV>1.2, intermediate PGA/PGV ratio when 1.2>PGA/PGV>0.8 and low PGA/PGV ratio when PGA/PGV<0.8 (Kianoush and Ghaemmaghami 2011). In this section, potential of progressive collapse for the case of corner column removal at the first and fourth stories of the

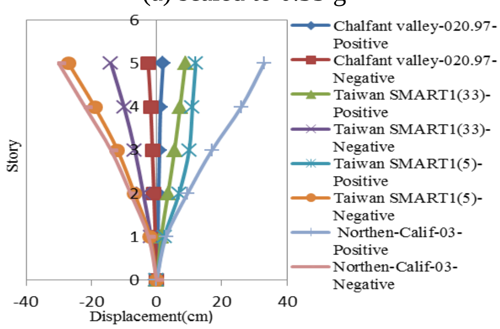
5-story frame at the beginning of the earthquakes with different frequency contents but identical Arias intensity and PGA, were assessed. The analyses results for 3 different groups of earthquakes (group B, C, D) with identical Arias intensity and PGA, were controlled. As the beams at each location of the column removal have inherent stiffness and natural vibration special to them, naturally it is expected that the earthquakes with different frequency contents but identical Arias intensity and PGA, would have different effects on the progressive collapse under the seismic loads.

In this section first for each group of the earthquakes, the absolute horizontal displacement of the stories were compared. Then, comparison was made between vertical displacements of column removal location considering different frequency contents of the earthquakes for the two critical column removal locations.

As shown in Figs. 10, 11 and 12, by increase in the predominant period of the earthquakes, the horizontal displacement responses of the stories increased too. For more accuracy, the presented results were iterated for 3 groups of earthquakes. The earthquakes of groups B, C and D which are earthquakes with identical Arias intensity and PGA but different frequency contents, were applied on the 5-story special moment resisting frame at the column removal location of the 4th story. For example Figs. 10(a)-(b) show that The Northern Calif earthquake which was an earthquake with high predominant period and PGA/PGV<0.8, induced largest horizontal displacement in the frames after removal of the corner column of the 4th story and the Chalfant Valley earthquake with the minimum

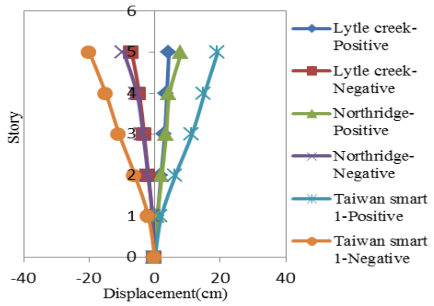


(a) scaled to 0.35 g

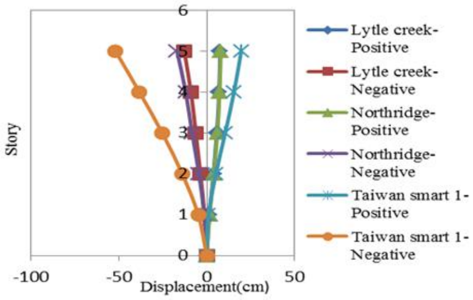


(b) scaled to 0.7 g

Fig. 10 Comparison of lateral displacement of the frames for case 5 under the effect of group B earthquakes

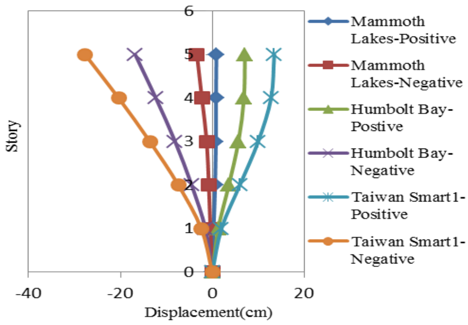


(a) scaled to 0.35 g

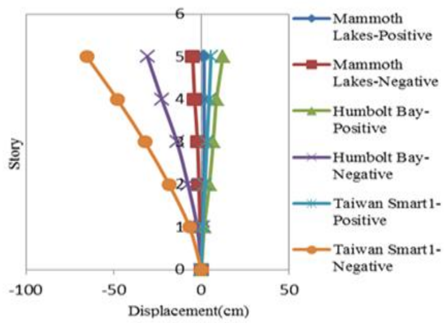


(b) scaled to 0.7 g

Fig. 11 Comparison of lateral displacement of the frames for case 5 under the effect of group C earthquakes



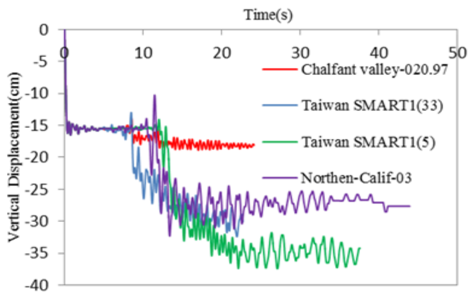
(a) scaled to 0.35 g



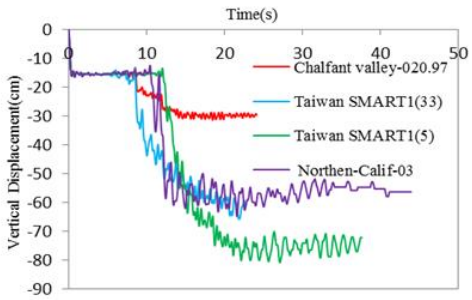
(b) scaled to 0.7 g

Fig. 12 Comparison of lateral displacement of the frames for case 5 under the effect of group D earthquakes

predominant period and PGA/PGV>1.2, induced smallest horizontal displacement in the frames. Also in groups C and D, the Taiwan Smart 1 earthquakes which had the largest predominant period, the greatest horizontal displacement was induced in the 5-story moment resisting frame at the corner column removal of the 4th story. Therefore it could

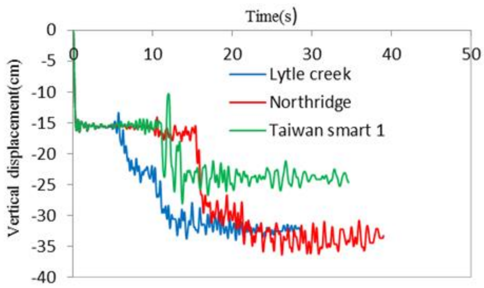


(a) scaled to 0.35 g

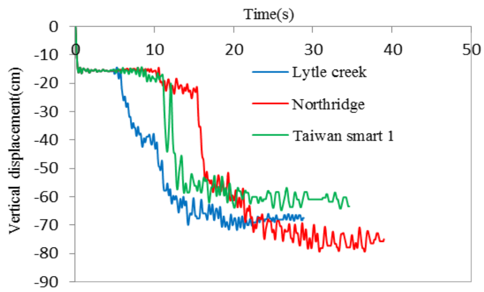


(b) scaled to 0.7 g

Fig. 13 Comparison of Vertical displacement at the point of column removal for case 5 under the effect of group B earthquakes



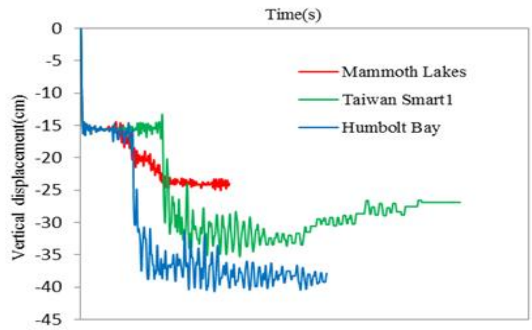
(a) scaled to 0.35 g



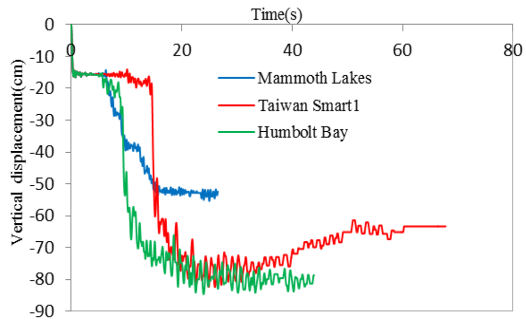
(b) scaled to 0.7 g

Fig. 14 Comparison of Vertical displacement at the point of column removal for case 5 under the effect of group C earthquakes

be concluded that with increase in the predominant period of the earthquakes, the horizontal displacements for all stories of the 5-story frame also increase, so the horizontal displacements of the stories after column removal, are considerably dependent on the frequency content of the earthquakes.



(a) scaled to 0.35 g

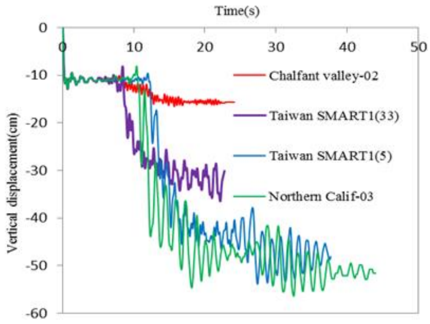


(b) scaled to 0.7 g

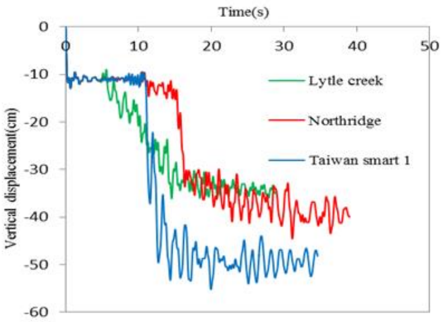
Fig. 15 Comparison of Vertical displacement at the point of column removal for case 5 under the effect of group D earthquakes

Figs. 13, 14 and 15 show that the earthquakes with different predominant periods, induce different vertical displacements during the seismic progressive collapse at the corner column location of the 4th story. When the predominant period of the earthquake is low, the natural period of vibration of the beam in vertical directions are low, too. Generally, Earthquakes induce additional back and forth forces and moments in the beams and by yielding of the beam, the vertical displacement increases. On the other hand at each column removal location, the beam has the flexural stiffness and natural frequency special to it at the vertical direction after column removal (like cantilever beam). As seen in the figures, the earthquakes with different frequency contents and identical Arias intensity and PGA, induced different ultimate vertical displacements after column removal. Contrary to the earthquakes with high periods which induced largest horizontal displacements, the Taiwan Smart 1 (5) and Northridge and Humbolt bay earthquakes with mean predominant period and 0.8<PGA/PGV<1.2 ratio or close to these ratios, induced largest vertical displacements at the corner column removal location of the 4th story, which is due to the greater consistency between the beam stiffness and the natural frequency of beams at the vertical direction after the column removal with earthquake frequency content. Therefore it could be concluded that for the corner column removal of the 4th story in the 5-story special moment resisting frame, the greatest potential of the progressive collapse event is induced by the earthquakes with mean predominant periods.

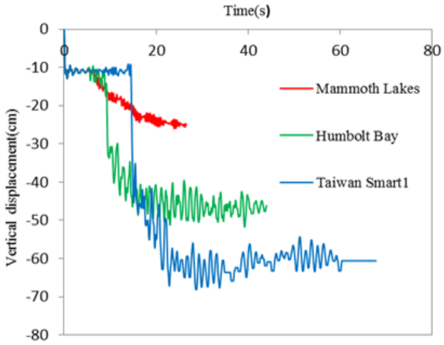
Also by increase in the PGA from 0.35 g to 0.7 g due to



(a) Goroup B



(b) Goroup C



(c) Goroup D

Fig. 16 Comparison of Vertical displacement at the point of column removal for case 3 under the effect of group B,C and D earthquakes(all earthquake scaled to 0.7 g)

increase in the arias intensity, the vertical displacement increased. Also the earthquakes with higher PGAs (0.7 g), the difference between vertical displacements for earthquakes with different frequency contents, increased. Therefore the main conditions for higher effectiveness of the earthquake frequency content in the seismic progressive collapse are the significant amounts of the PGA and the Arias intensity of the earthquakes.

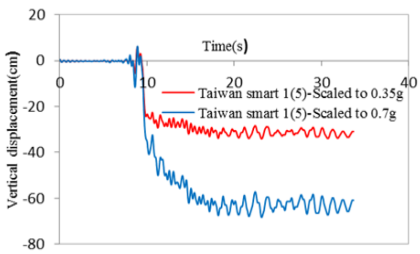
Therefore it could be stated that in column removal at the initiation of the earthquake, the ultimate displacement of the column removal location, is affected by the frequency content of the earthquake (predominant period or PGA/PGV ratio), flexural stiffness of the beam and the natural frequency of the beam after column removal. On the other hand for each column removal location at the first and fourth stories, the natural frequency of the beam after column removal is different. The period of vibration of the beams(or natural frequency of beam) after column removal at each location of the removed column is dependent on the beams dimensions and the amount of loads that they bear, so it is expected that beams in different column removal locations, exhibit different performances against the earthquakes with different frequency contents. For the case of column removal of the first story of 5-story special moment resisting frame, as shown in Figs. 16(a)-(b)-(c), for each 3 groups of the earthquakes with different predominant periods and identical Arias intensities, the earthquakes with high predominant periods and low PGA/PGV ratio, induced maximum vertical displacement. Therefore for removal of the first story column in the special moment resisting frame in this research, the earthquakes with high predominant period are more consistent with natural frequency of beam and flexural stiffness of the beam after column removal. While concerning the corner column removal of the 4th story, the earthquakes with mean predominant periods and PGA/PGV ratios, induced maximum vertical displacements. Therefore the vertical displacement at the column removal location, in addition to the earthquake frequency content, depends upon the column removal location and the beam stiffness at the location of removed column. Therefore considering the number of stories, type of structural system and different locations of column removal, it is predicted that the results of seismic progressive collapse would be different.

4.3.2 Column removal at PGA of the earthquake

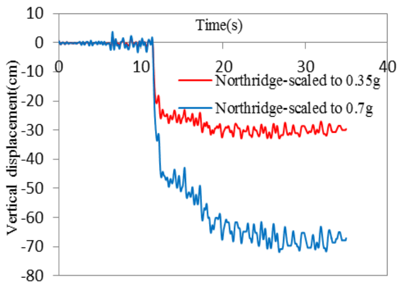
For a more realistic simulation of the seismic progressive collapse, the column should be removed at the strong ground motion especially at PGA. In this case, a more precise simulation of the progressive collapse under the seismic load is presented. Here, sudden displacement at the column removal location, could attain higher values with respect to the previous case where the column was removed at the initiation of the earthquake. Sudden displacement at the moment of column removal is important regarding that, if in a short time, suddenly a significant displacement occurs, impact force is exerted on the structure which could cause failure in the nut and bolt or welded connections or in a shorter time, the plastic hinges rotations exceed the allowable limits. In this case, the possibility of zipper-type or pancake-type (which is more dependent on sudden impact) progressive collapse, could be investigated.

In the previous section, the effects of various factors for column removal at the initiation of the earthquake were investigated. The amount of vertical displacement of column removal location at the initiation of the earthquake and without considering the effect of earthquake on sudden displacement of removal location was about 17 cm for the corner column removal of the fourth story (as shown in Fig. 9). But if the column is removed at the PGA, the sudden vertical displacement of removal location, with respect to removal from the initiation, increases more than 15 cm and by increase in the PGA, the increased value also rises. This shows that sudden displacement at the moment of removal depends on the force which is exerted suddenly and in the form of impact on the structure.

Therefore the PGA of the earthquake which has smaller

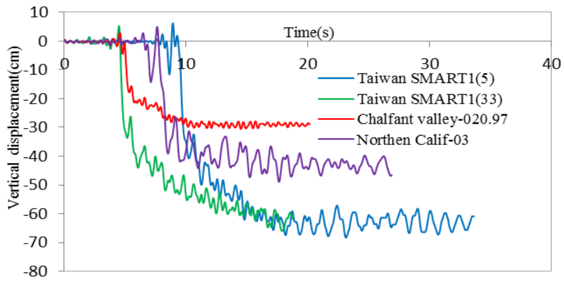


(a) Taiwan smart1(5) earthquake

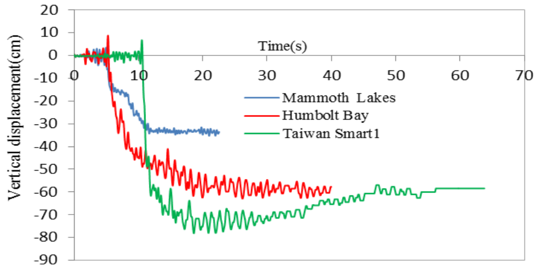


(b) Northridge earthquake

Fig. 17 Comparison of sudden Vertical displacement at the point of column removal when the column removed at the PGA for case 5 under the effect of Taiwan smart1(5) and Northridge earthquakes



(a) Group B



(b) Group D

Fig. 18 Comparison of sudden Vertical displacement at the point of column removal when the column removed at the PGA for case 5 under the effect of group B and D earthquakes

effect on the ultimate sudden displacement (in the case of column removal at the initiation) with respect to the Arias intensity and frequency content is completely effective on increase in the sudden displacement. Fig. 17 demonstrate increase in the sudden displacement at the removal location and at the PGA with respect to removal at the initiation. Also these figures indicate the effect of increase in PGA from 0.35 g to 0.7 g on the sudden displacement at the removal location.

Further investigations revealed that in addition to the PGA, other factors are influential in increase in the sudden vertical displacement at the column removal location. Regarding the way the beam vibrates in cases where columns are removed at the initiation of the earthquake, the vertical vibration amplitude is smaller in earthquakes with lower predominant periods with respect to those with higher predominant periods. Therefore in the earthquakes with high or mean predominant periods, if the column is removed at PGA, the possibility of intense sudden vertical displacement at the column removal location is higher. As shown in Fig. 18(a) The Northern Calif and Taiwan Smart 1(5) Earthquakes which have high or mean predominant periods and low PGA/PGV ratios, induced greater sudden displacements at the column removal location and at the PGA of the earthquakes, with respect to the earthquakes with lower predominant periods. The results were also repeated for group D. Regarding Fig. 18(b) it was observed that sudden displacement at the column removal location for Taiwan Smart 1 Earthquake which is an earthquake with high predominant period and low PGA/PGV ratio, was about 5 times of Mammoth lake Earthquake which is an earthquake with low predominant period and high PGA/PGV ratio. Therefore in earthquakes with high predominant periods, the possibility of intense sudden displacement at the column removal location at the PGA of the earthquake is higher.

In recent years, researchers have turned to implementing energy methods for further understanding the progressive collapse phenomenon. Energy method is an appropriate method for investigation of progressive collapse in other structural elements. Therefore, further investigation of seismic progressive collapse using the energy criterion to focus on the extension of failure to other elements, is considered to be essential.

5. Conclusions

In the present research, first the performance of 5 and 15- story moment resisting frames were investigated for different column removal locations against progressive collapse under the gravity loads. For this purpose the push-down nonlinear static and nonlinear dynamic analyses were conducted and the results obtained were compared to the allowable values in the GSA Guidelines. Then for critical locations of column removal, the effects of earthquake characteristics like Arias intensity, PGA and frequency content of the earthquake on the progressive collapse under the seismic load was investigated. The obtained results are as follows:

- The 5 and 15-story steel special moment resisting frames designed against the progressive collapse due to the gravity load are safe, because for different column removal locations, the responses did not exceed the allowable values in the GSA Guidelines.
- By comparison of potential of the progressive collapse for different column removal locations, it could be

concluded that by removal of corner columns, the potential of progressive collapse is higher with respect to the interior columns, because there are lower number of elements for load re-distribution corresponding to the removed column. By similar reasons, with column removal at higher stories also the potential of progressive collapse is higher than the lower stories. Also the 15-story frame has lower potential of the progressive collapse with respect to the 5-story frame.

• After removal of the column in the seismic progressive collapse, the earthquake caused increase in the vertical displacement at the column removal location which was due to the flexural yielding of the beam after column removal and increase in the members' internal forces due to the earthquake. Also the earthquakes themselves could be the cause of progressive collapse initiation.

• In the seismic progressive collapse analyses, in the case of column removal at the initiation of the earthquake, for the earthquakes with identical frequency content which were scaled to identical PGAs, The potential of seismic progressive collapse is dependent upon Arias intensity. By increase in Arias intensity, the vertical displacement of column removal location increased and the probability of seismic progressive collapse increased.

• The ultimate vertical displacement for the case of corner column removal at the initiation of the earthquake is largely dependent on Arias intensity and the input earthquake energy. The earthquakes with higher PGA but lower Arias intensities may induce smaller vertical displacements and vice versa.

• For different locations of column removal at the 5-story special moment resisting frame, with identical Arias intensity and PGA, the seismic progressive collapse is completely dependent on the earthquake frequency content, the natural frequency of the beam after column removal and its flexural stiffness at the column removal location.

• By comparison of vertical displacements at the column removal location for earthquakes with different frequency contents, it could be concluded that by increase in the PGA value and Arias intensity, the difference in vertical displacements for earthquakes with different frequency contents is increased. In other words, one of the conditions for effectiveness of the earthquake frequency content in the seismic progressive collapse is the significance of PGA and Arias intensity of the earthquakes.

• By column removal at the PGA of the earthquake, the sudden displacement at the column removal location could be magnified with respect to removal at the earthquake initiation. The amount of magnification depends on such factors as PGA value and the earthquake frequency content. By increase in PGA, the sudden displacement at the removal location increased. Also in the earthquakes with higher predominant periods, the possibility of intense sudden displacement at the column removal location is higher at the PGA of the earthquakes than the earthquakes with lower predominant periods.

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