Vulnerability Evaluation in steel moment and asymmetric approach to progressive collapse

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Abstract: Usually buildings are designed for ordinary loads like dead, live, wind and earthquake loads. Notwithstanding there are other loads like error in design, error in construction, fire, explosion, accidental over loads, hazardous material, vehicle crashes, bomb explosion etc. which may rarely take place, but in case of occurrence could result in uncompensable losses with the progressive collapse approach. Various methods have been suggested for reducing the probability of progressive collapse risk in the structures, among them the Alternate Path Method for preventing the progressive collapse seems to be the most logical and comprehensive one. In this method removal of a main and critical member is investigated and the structure for determining the effect of this removal is analyzed. By removal of a column in a structure, other members are subjected to large displacements and rotations. In the previous research, as asymmetric moment resisting frames have not been much studied, in this research a sample of asymmetric steel frame with moment resisting system is modeled utilizing GSA [1]and DOD[2] codes in the OpenSees software and then using the Alternate Path Method (APM), the progressive collapse is controlled and the induced forces in the adjacent members together with structure stability are compared for each case. The obtained results show that vulnerability rate of symmetrical steel moment resisting frames is different in comparison with the asymmetrical ones and the linear and nonlinear dynamic analyses were performed for comparison purpose and it was observed that in comparison to linear analysis according to DoD, the nonlinear static analysis, yielded larger structural response and the results varied based on such variables like the applied load, column removal location or the story height at which column removal takes place.

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1. Introduction

Vulnerability and gradual failure include a set of failures which result into partial or overall collapse of the structure. In the best method for reducing the potential of gradual failure published by NIST [3], the risk of unnatural loads which could result in gradual failure are classified as follows: airplane impact, design error, fire, explosion, accidental overload, hazardous material, vehicle crashes, bomb blast, etc.

As the probability of occurrence of these risks is low, they are not considered in the design. Most of them have features of short term effects and result into dynamic reactions. In the united states the General Services Administration (GSA) and Department of Defense (DoD) have provided guidelines and information concerning procedures for resisting gradual failure of building structures. Among various methods for design against gradual failure, the guidelines generally have proposed the Alternate Path Method. In this procedure, the structure is designed in a way that if a member encounters failure, alternate paths be available for bearing the loads so that overall failure does not occur. This method has the advantage of being simple and straight forward. In the most common procedure for application of this design method, the building structures should be able to lose each of the columns without any collapse occurrence in them.

The analysis process proposed By the guideline for Alternate Path Method, are the linear static (LS), linear dynamic (LD), nonlinear static (NS) and nonlinear dynamic (ND) procedures, which were also suggested by seismological analysis and also design for FEMA274 structures. William and Kaewkulchai [4] investigated the analysis procedures through analysis of 2D frames.

They found that the linear static method should result into conservative results because it could not reflect the dynamic effect resulted from the sudden removal of the columns. The aim of this research is assessment of the collapse potential of steel moment resisting frames, and the results of step-by-step linear analysis as proposed by GSA 2003 and DoD 2005 guidelines are compared to the results of nonlinear dynamic analysis. The effect of such parameters like column removal location and number of the stories also were investigated, [1] and [2].

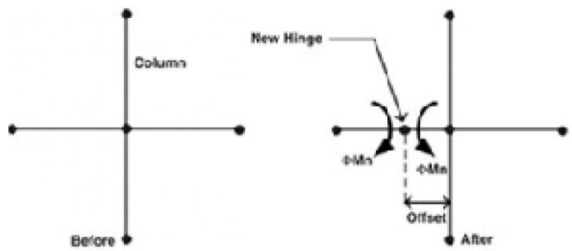


Fig.1- Hinges modeling

Table 1-Acceptable criterion for gradual collapse according to GSA 2003

Component	Ductility	Rotation(rad)
Steel Beams	20	0.21
Steel Columns(tension controls)	20	0.21
Steel Columns(compression controls)	1	-

2.moment resisting frame being studied (model geometry)

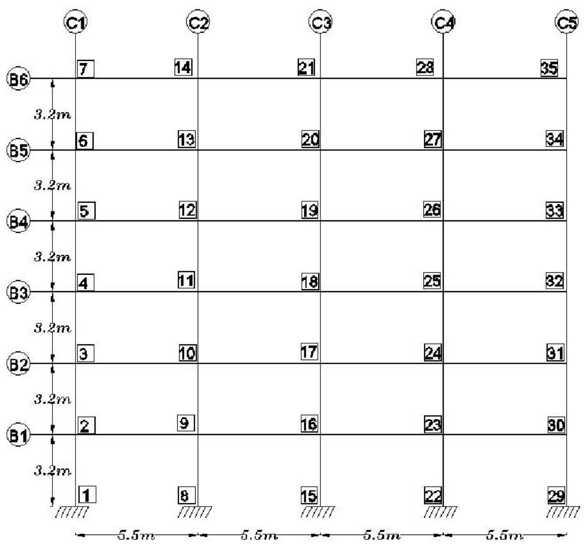


Fig.2-Original model with equal spans

In this article in order to determine vulnerability and resistance of moment resisting frames against the building progressive collapse, 5 different models comprised of 6 stories and 4 spans were designed in which section properties and span lengths are similar for each 5 models, also the total height, except for the original model, was equal in all 1-4 models and are assumed as depicted in Figs. 2-4. The height of typical floors in Fig. 21 is 3.2 m and in other frames the ground floor height is 4.0 m and the typical floors height is taken equal to 3.2 m. The clear distance between columns in Figs. 2-3 is taken equal to 5.5 m and in other figures is asymmetric. Among other properties of the structures is that column –to-beam connections are fully fixed also the supports are taken as fixed. All frames are assumed to be ordinary moment resisting frames which are modeled in OpenSees software. According to GSA 2003 guideline, the limit values for DCR for the girders is equal to 3 and for the columns is 2 which is based on the width -to -thickness ratio [4].

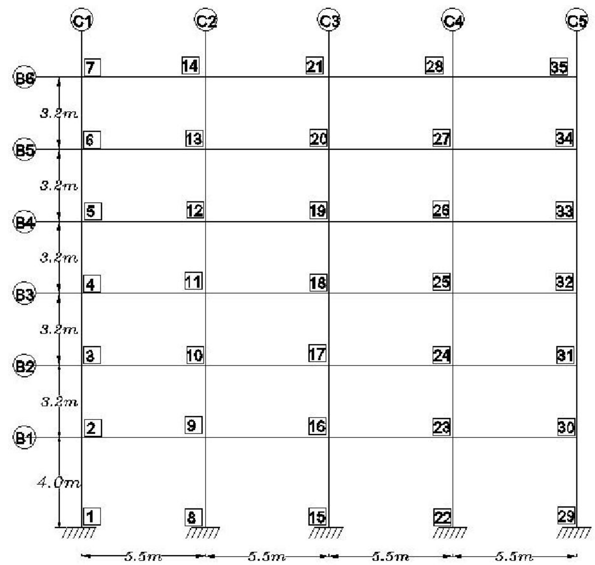


Fig.3-Model "1"- with equal spans and asymmetric height

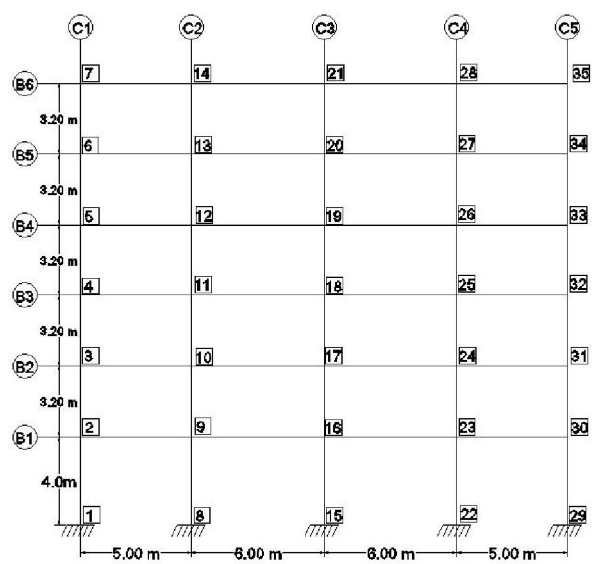


Fig.4-Model "2" with exterior asymmetric spans

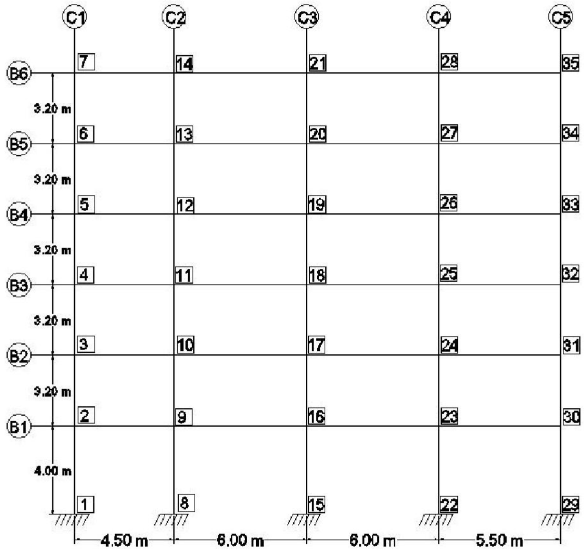


Fig.5-Model "3" with exterior asymmetric spans

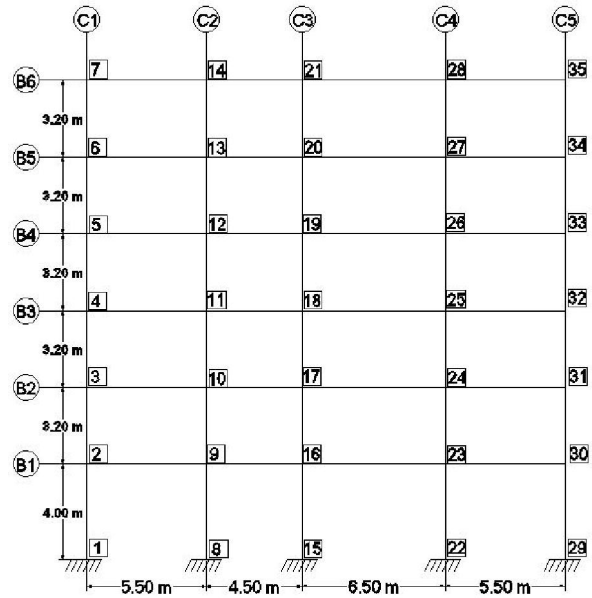


Fig.6-Model "4" with exterior and interior asymmetric spans

Table 2- Values of the applied loads on the structure

Earthquake	Station	PGV (cm/Sec)	PGA (g)	PGD (cm)	Magnitude (M)	Step Time Δ	Year of	Type of soil
Imper ial Valley	Elcentro Array	91	0.367	62	6.5	0.005	1979	III

3. Applied loads on the structure

Table 3-Properties of the used accelerograms

Materials	Amounts	Unit
Mass per unit volume	**M =785**	$\frac{kg}{m^3}$
Weight per unit volume	**W = 7850**	$\frac{kg}{m^3}$
Modulus of elasticity	**E = 2.0 ×10^6**	$\frac{kg}{cm^2}$
Poisson's ratio	**= 0.3 υ**	----
Steel yield stress	**F_y = 2400**	$\frac{kg}{cm^2}$
Ultimate strength steel	**F_u = 3600**	$\frac{kg}{cm^2}$

The load cases in fact indicate the nature of applied loads on the structure, In this frame 3 types of load cases which albeit numerous load cases could be defined, the nature of load cases studied in this frame include: Dead load, Live load, Earthquake load (EQ), the load values applied on the structure are according to the following table [5].

Table 4-properties of the steel section

Load	Story	Dead load	Live load	Unit
Gravity load	first floor – five floor	460	200	$\frac{kg}{m}$
	Sixth floor (Roof)	520	150	

4. Used accelerograms

According to the Code, those records which are used in determination of effect of ground motion should represent the real ground motion at the site of building construction. AS no accelerogram is capable of producing responses that are compatible to the response spectrums within the period ranges, it is necessary to select and use a number of naturally scaled accelerograms. In this research attempt is made that the near fault records be used. According to the definition made by Mohraz on the near fault field, when the distance from the station is less than 15 km, the record is selected and applied on the structure. The selected accelerogram is taken from the PEER site. In this research the records are chosen from among different near fault occurred earthquakes [5].

5. Material properties

In these frame just one type of steel material is used for the sections in the structure and the properties of this material are according to the following table [5].

6. Sections properties

The sections of beams and columns are taken from the Stahl library of sections and the IPE section is used for both sections and details of these sections are given in the following table.

Table5- Used sections in the designed frames

Eleman	Axis	Story	Section
Column	All Story	Lateral (COL1)	2IPE360
	All Story	Middle (COL2)	2IPE400
	All Story	Middle (COL3)	2IPE400
	All Story	Middle (COL4)	2IPE360
	All Story	Lateral (COL5)	2IPE300
Beam	Story 1	All Mouth	IPE330
	Story 2		IPE300
	Story 3, 4		IPE270
	Story 5, 6		IPE240

7. Analysis procedure

1.7. acceptance criterion for gradual failure

GSA 2003propsed use of Demand Capacity Ratio (DCR) that is the ratio of the acting force and the strength of the structural component as a criterion or determination of failure of the main structural components through the following linear analysis procedure [6]:

$$DCR = \frac{Q_{UD}}{Q_{CE}}. \quad (1)$$

In which QUD is the demand acting force in the building component (bending moment, axial force, shear etc.) and QCE is the unfactored ultimate capacity of the building component (bending moment, axial force, shear etc.)

2.7. Step by step procedure of linear static analysis

The step by step procedure for performing LS analysis proposed in GSA 2003 is as follows [1].

Step 1:

Removal of a column from the considered location and conducting linear static analysis using the following gravity load imposed on the span between two columns from which the column is removed:

(2) DL +0.25LL 2

In which DL and LL indicate the Dead load and live load, respectively.

Step 2:

Examine DCR in each building component. If the DCR of a member exceeds the shear acceptance criterion, that member would not be accepted. If the DCR of a member end exceeds the bending acceptance criterion, a hinge is inserted at the member end location as shown in Fig.1

Step 3:

At each inserted hinge, equal bending moments but with opposite signs are applied corresponding to the expected flexural strength (foreseen) of that member, (nominal strength multiplied by the over-strength factor equal to 1.1), as shown in Fig. 1

Step 4:

The stages 1-4 are repeated as long as the DCR corresponding to each member has not exceeded the limit state. If the moments are again redistributed throughout the entire building and the DCR values likewise exceed, in regions outside of the allowable failure which are defined in the guideline, the structure will be considered to have a high potential for progressive collapse. DoD2005 has proposed a similar procedure for the Alternate Path Method except the increase in applied load Eq (3), acceptance criterion and allowable collapse region

$$2(1.2DL + 0.5LL)+ 0.2\ WL \quad (3)$$

In which WL is the wind load (surcharge due to wind effects)

3.7. Applied loads for dynamic and static analyses

For the static analysis both GSA2003 and DoD2005 used the dynamic amplification factor equal to 2 for load combination. DoD guideline proposes a gravity load greater than the value proposed by GSA guideline. The Wind Load is included in the DoD load combination [1] and [2].

For dynamic analysis both guidelines have proposed the use of dynamic amplification factor. For conducting dynamic analysis, the applied axial load on the column is calculated before the column removal. Then the column is replaced by the point loads equivalent to the forces of its own components. For simulation of the sudden column removal, the component forces are removed after elapse of a certain amount of time as shown in Fig. 7 in which variables M,P and D depict the axial, shear and bending moment forces, respectively and W depicts the vertical load. In this research, the forces are linearly incremented for 5 seconds till reach their ultimate values and remain about 2 seconds without change till the system reach equilibrium state and the upward forces are suddenly removed for 7 seconds to simulate the dynamic effect of sudden column removal [7].

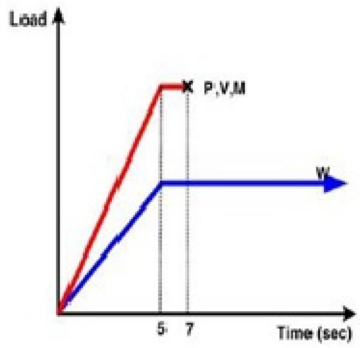


Fig.7-Applying loads for dynamic analysis procedure.

8. Analytical modeling

Numerical analysis of the exterior frames was performed using the OpenSees program [7]. In the material model In the material model, the proportion of loading was not considered in this study since the behavior after sudden column removal was not fast enough to include the proportion effect. As the dynamic behavior caused by sudden column removal was not involved with load reversal as in structures subjected to earthquake load, use of complicated hysteretic model was not necessary. Damping ratio was assumed to be 5% of the critical damping, which is usually

adopted for analysis of structures undergoing large deformation. The progressive collapse analyses were carried out by removing a column in various locations in accordance with the GSA 2003 and DoD 2005 guidelines [8] and [9].

9. Analysis of progressive collapse

In this research in order to determine the strength of designed moment resisting frames which are also according to the requirements of UFC code [10]. in each of 5 models, columns are removed from a certain location of the structure. Considering that the exterior frames have 4 spans and respecting that structures are symmetric in Figs. 2 and 3, At the first story, the exterior and interior columns each are separately and suddenly removed from the structure and the corresponding response of the structure in confronting this event has been investigated. Overall, 3 different scenarios have been studied in this study which for the main models and (1) all 3 columns were removed and for models (2) -(4) also the most critical column was selected and removed. The meaning of progressive collapse which is generally used is this assumption that due to the factors which cause progressive collapse, the column is damaged and removed from the structure and the structure behavior is investigated in this condition. In Figs. 8, 9 and 10, the column removal scenario which is used in the AP method for assessing structure against the progressive collapse, it is shown that this event is real and it is demonstrated at each of the blast tests and the consequent events. In Table 6, various removal scenarios and the corresponding members have been shown.

Table 6: Cases of progressive collapse analysis for the 6-story model

Model	Eleman	Floor	Scenario
6 Story	C1	Ground	1
	C2	Ground	2
	C3	Ground	3

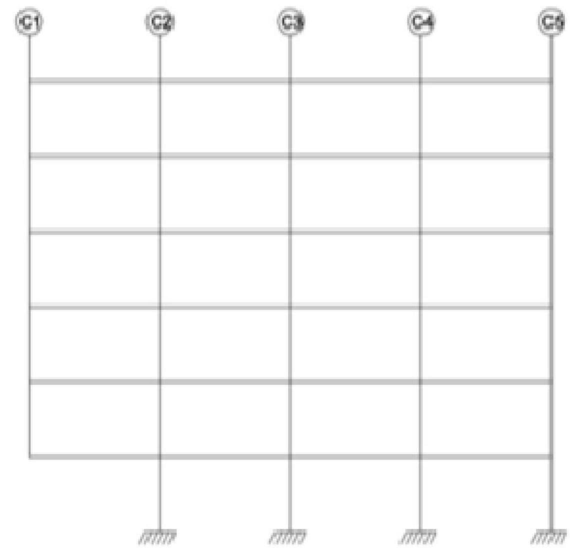


Fig.8 : First scenario

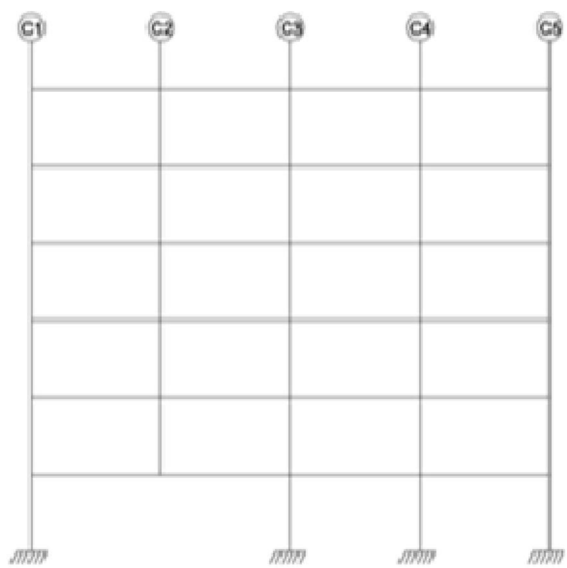


Fig.9: Second scenario

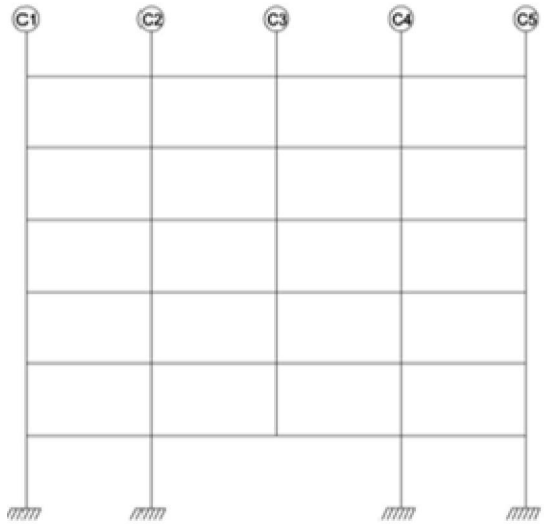


Fig.10: Third scenario

First, two models i.e. the main model and model no. 1 with equal span lengths and different heights of the first floor, were investigated. In these two models, 3 different scenarios were studied and after column removal the results of corresponding displacements and accelerations were compared and examined and the maximum displacements and accelerations values corresponding to the floors are given in Table 9. The computational period and frequencies of all models are also given in tables 7 and 8, respectively.

Table 7: computational period (seconds)

Mood Story	Story 1	Story 2	Story 3	Story 4	Story 5	Story 6
Main model	2.283	0.769	0.413	0.261	0.194	0.163
Model 1	2.419	0.821	0.440	0.276	0.202	0.166
Model 2	2.420	0.821	0.440	0.277	0.203	0.168
Model 3	2.417	0.820	0.440	0.276	0.203	0.168
Model 4	2.408	0.818	0.439	0.275	0.201	0.166

Table 8: Frequency (radians/seconds)

Mood Story	Story 1	Story 2	Story 3	Story 4	Story 5	Story 6
Main model	2.753	8.173	15.232	24.086	32.369	38.448
Model 1	2.598	7.650	14.283	22.795	31.077	37.753
Model 2	2.597	7.652	14.275	22.711	30.884	37.458
Model 3	2.560	7.662	14.290	22.724	30.892	37.462
Model 4	2.610	7.678	14.316	22.856	31.187	37.845

In models (1)-(4) also the critical column of each structure is selected and the corresponding displacements and accelerations of the floors were studied and investigated. The maximum displacements and accelerations values of the critical column together with corresponding diagrams are given in Table 10 and Figs. 11-19.

Table 9: Maximum values of displacement and acceleration in the main model and model 1 (first, second and third scenarios)

Story	Model	Scenario	Max Displacement (cm)	Max Acceleration (cm/s^2)
Story 1	Main model	C1	21.153	2.73981
	Model 1	C1	23.1589	2.73988
	Main model	C2	23.139	2.74013
	Model 1	C2	23.1604	2.74062
	Main model	C3	22.3544	2.74013
	Model 1	C3	23.1604	2.74062

10. Diagrams of floors displacements and accelerations for the main model and models 1-4.

Table 10: Maximum values of displacement and acceleration in models 1, 2, 3 and 4 at the critical condition

Story	Model	Critical Column	Max Displacement (cm)	Max Acceleration (cm/s^2)
Story 1	Model 1	C2	23.1604	2.74062
	Model 2	C3	22.7612	2.63150
	Model 3	C3	22.14	2.58023
	Model 4	C3	23.3358	2.58287

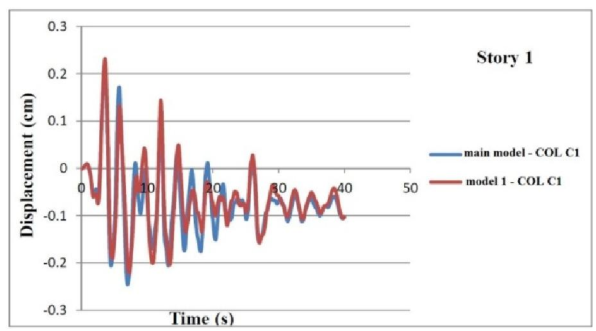


Fig.11- Displacement of the main model and model 1-first scenario

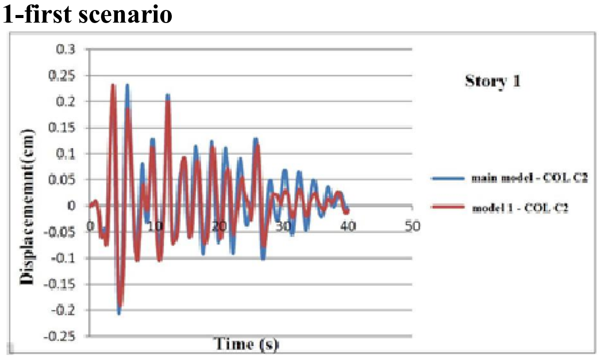
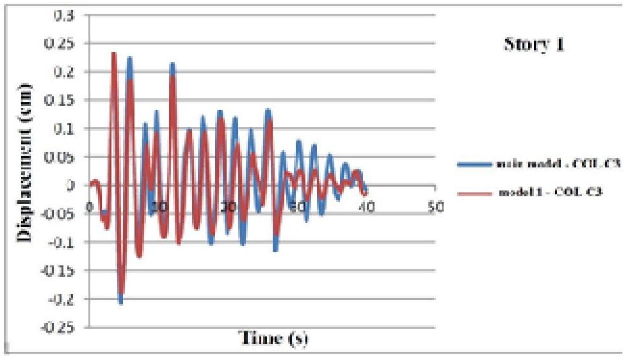
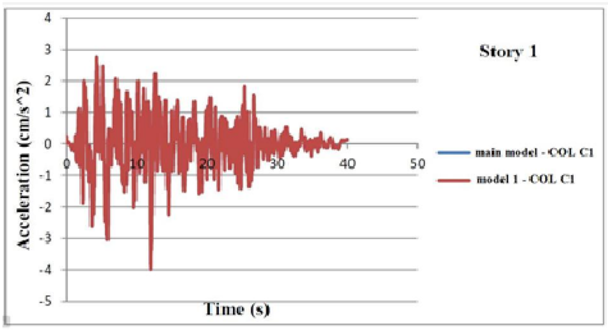
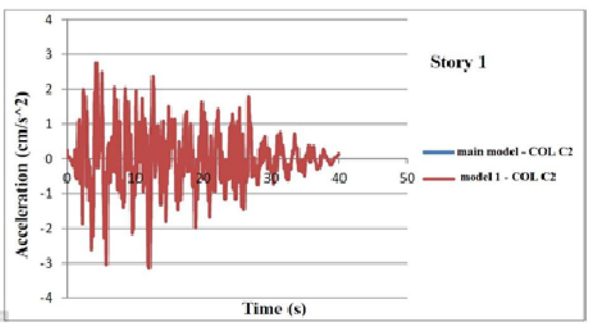


Fig.12 -Displacement of the main model and model 1-second scenario







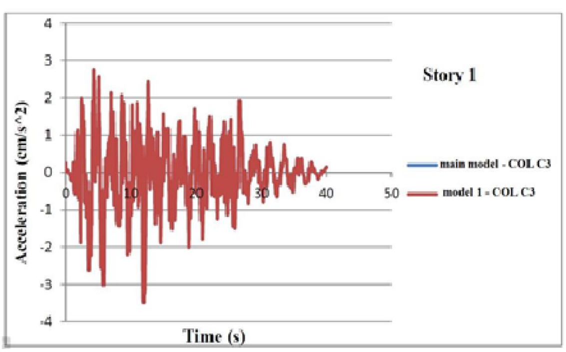
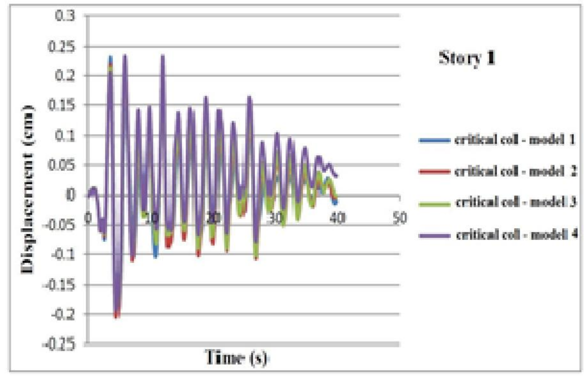
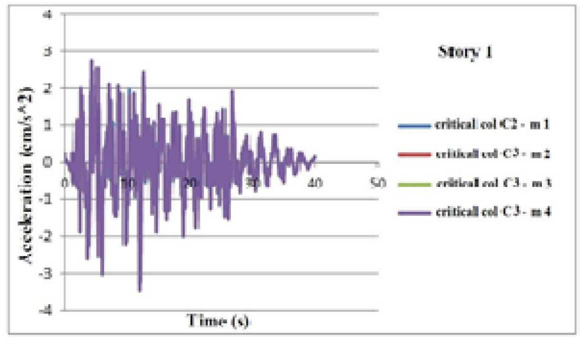
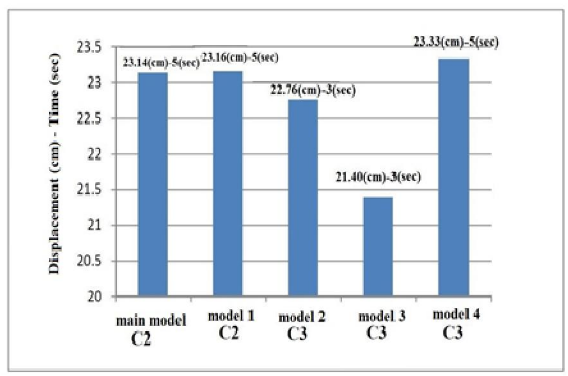


Fig.16 - Acceleration of the main model and model 1-third scenario







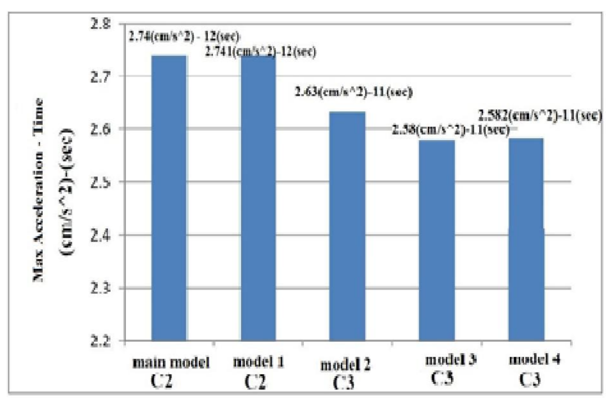


Fig.20- Investigating acceleration of models 1-4-critical column

11. Analysis results

In this paper, vulnerability of asymmetric steel moment resisting frames was assessed and different models in height and asymmetric spans were modelled and analyzed via OpenSess software. Then, progressive collapse was controlled using alternate path method (APM), and forces created in adjacent members and structural stability was compared in each status. Following analyses and results can be interpreted through examining table 7, 8, 9 and 10 and diagram 19 and 20 respectively related to maximum displacement-time and maximum acceleration-time in 5 models.

1. According to non-linear static analysis, frames have great potential for vulnerability to progressive collapse.

2. Increasing the height in model (1) compared to the original one, period of lower floors, especially first floor, increases a few tenths because mass of the floor increases by rising the height. This value increases up to hundredths in higher floors; but due to asymmetric spans, period of structure declines a few hundredths in models (1) to (4) and even up to thousandths in higher floors.

3. Rising the height of first floor as 1 m in model (1) and the original one, there was no significant change in maximum displacement and acceleration values; it indicated that increase in floor height did not have much impact on floor displacement and acceleration.

4. According to study on maximum displacement-time graphs of models (1) to (4), the more spans become asymmetrical and irregularity takes place in spans, the more displacement occurs in those models.

5. Investigation into vulnerability of models in critical states demonstrates which irregularity shows best behavior that according to diagrams 19 and 20, model (3) had better behavior in terms of maximum displacement and maximum acceleration in comparison with other models; it shows that irregularity in side spans may have less impact than middle ones.

6. Joints have been formed in all spans and even the ones in which columns have not been removed; and the structure can resistant against progressive collapse by reinforcing a limited number of members.

Obtained results are recommended as a practical achievement in designing steel moment frames in regard with architectural limitations in locating columns to reduce vulnerability.

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