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NUMERICAL MODELING OF INFILLS IN ASYMMETRIC STEEL MOMENT FRAMES FOR THEIR DYNAMIC ANALYSIS WITH PROGRESSIVE COLLAPSE APPROACH

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Abstract. *Nowadays investigation of progressive collapse prevention in case of important existing and new buildings due to terrorist attacks, design or construction mistakes as well as explosion seems to be necessary. In previous studies, experimental and numerical works has been carried out on the effect of infilled frames on steel moment frames; but In previous work on progressive collapse of building system, the main concern has been proposing a substitute load transfer path for the deleted path, and no attention has been paid to the effect of infills in nonlinear dynamic behavior of the structural system. The aim of this study is proposing an appropriate procedure for numerical modeling of infills in steel asymmetric moment frames in progressive collapse analysis. The proposed procedure has been applied to a series of 6 story moment frames with different span size, once without the infills and once more with them. Introduced in terms of equivalent compressive bar, strength of masonry infill is numerically simulated by FEMA-356-Commentrary for Seismic Rehabilitation. The employed software in this research was OpenSees For verification of the proposed numerical modeling procedure results of previous experimental studies have been used. In this paper, endurance time method in loading and dynamic analysis has been chosen to assess progressive collapse. Various scenarios have been considered for column removal to trigger the progressive collapse. Results of dynamic analysis show that the amount of vertical deflection of the studied frames with contribution of infills is significantly less than those obtained without infills.*

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1 INTRODUCTION

Nowadays structural frames are filled by masonry materials such as brick in terms of interior or exterior walls which have a high stiffness and in-plane strength. Given great complexity of wall numerical models and analyses of the frames using convenient approaches (like time history analysis), engineers actually ignore the impact of infills in most of their engineering analyses and designs, which would result in a high error in realistic behavioral estimation of the frames. Previous studies and observations show that the infills significantly affect the behavior of structures and if it is ignored, a considerable error would happen in estimation of real structural performance and behaviors such as stiffness and lateral strength, ductility and energy dissipation capacity. Most of investigations into progressive collapse of buildings have been associated with analyses on existing buildings; they have been focused only on sudden removal of structural elements and rarely noticed the effect of infills on asymmetric frames. Important structures which are prone to progressive collapse should be designed in such a way that if any member is removed, substitute paths can transfer loads and load-bearing members around the removed element possess extra capacity to withstand forces without overall collapse. When a vertical load-bearing member is removed, beams demonstrate chain reactions against vertical loads; and applied loads are primarily tolerated by vertical component of axial forces in beams. A few researches have been accomplished to propose effective solutions for reducing the potential of progressive collapse. Dynamic analysis of the impact of brick infills particularly on asymmetric steel frames is one of topics scarcely considered in regard to progressive collapse, which has been assessed in this study via endurance time method that is a new approach of loading and dynamic analysis in progressive collapse.

2-Literature review

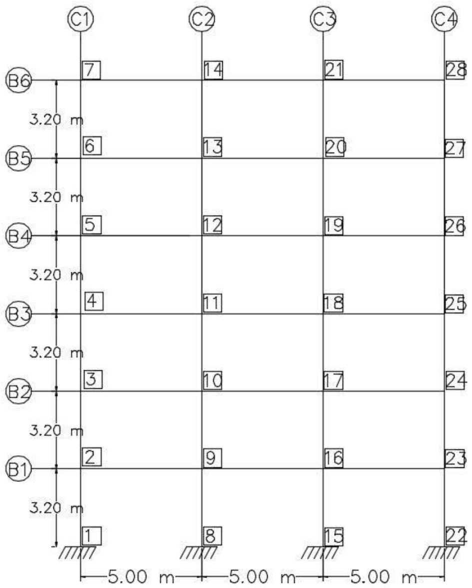
The role of masonry infills in withstanding lateral loads was first revealed when Empire State building was constructed in New York (1931). Diagonal cracks were observed in some masonry partitions of 29th and 41st floors, which were caused by a storm at a wind speed over 90 mph [1]. In another analysis on behavior of steel structures survived Manjil earthquake in 1990 in Iran, significant role of the panels in absorbing a considerable proportion of seismic loads became evident. Participation of infills in lateral loading has led to serious damages in walls and various failure modes such as diagonal cracks and corner crushing have been observed [2].

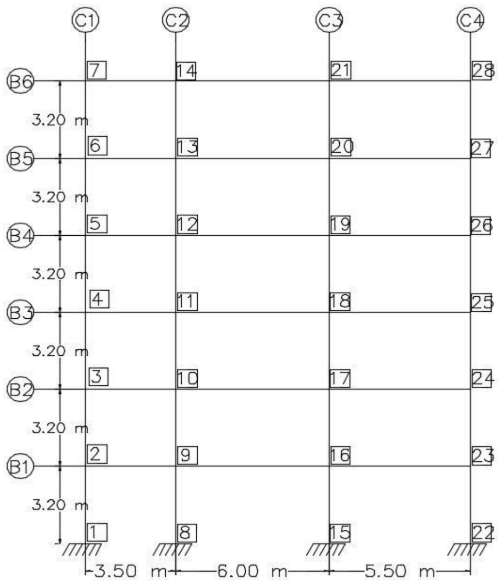
Since 1950s up to now, experimental and analytical researches have been done on behavior of infilled frames to identify different effects of infills on frames under the action of lateral loads. Given complexity and sensitivity of the subject, numerous researchers have investigated into the matter so that brief explanations could be found in reports by Moghaddam and Dawling (1987) [3], Abrams (1994) [4], Fardis (1996) [5], Crisafulli (2000) [6] and FEMA 306 and FEMA 307 commentaries (1998) [7, 8]. Generally, the works can be classified into three groups: experimental researches, analytical-numerical studies in order to model infills, and evaluation of infilled structures.

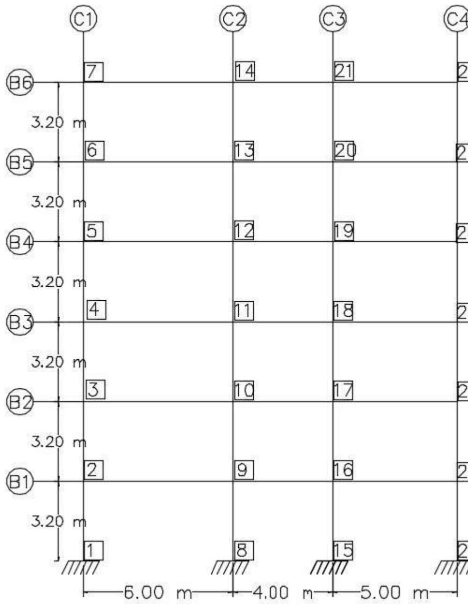
In regard to recent investigations into structural frames with infills, Kocak and Yildirim (2011) studied the effect of infills on frequency but they did not discussed about progressive collapse and asymmetry in models. In 2011, Ricci and Verderame assessed reinforced concrete frames and the impact of infills but they did not consider progressive collapse. Numerous studies and investigations have been carried out on infills, particularly in symmetric concrete frames, but asymmetric steel frames have been rarely noticed. In the latest research in 2016, progressive collapse of asymmetric steel frames was explored by Nezamisavojbolaqi and Shafiee [9,10]. According to literature review, it can be summarized that the impact of infills on asymmetric steel moment frames was studied in this paper because it has not been properly considered yet.

3- Numerical modeling of a steel moment frame and selecting earthquake records

In this article, 12 modes of a 6-story steel moment frame have been modeled in 3 spans with the same floor height on each story; the effect of infills has been considered in 6 models and withdrawn in the other 6 ones. Span and height asymmetry of frames has been considered in accordance with ordinary spans and designed based on AISC 360-10 using LRFD[1] by Etabs. In all of these frames, it has been mainly assumed that light masonry walls just withstand lateral loads, while steel frames are designed to resist both vertical and lateral loads. All



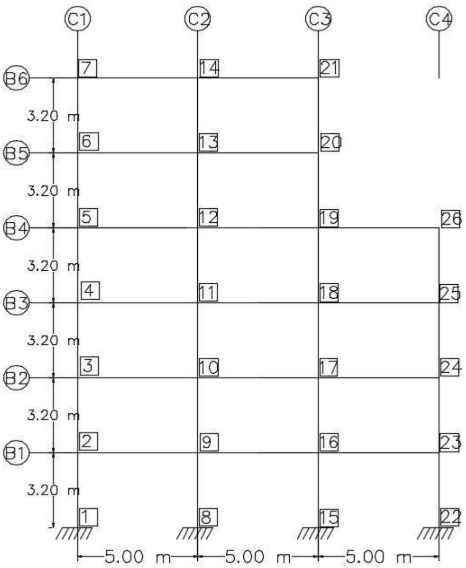


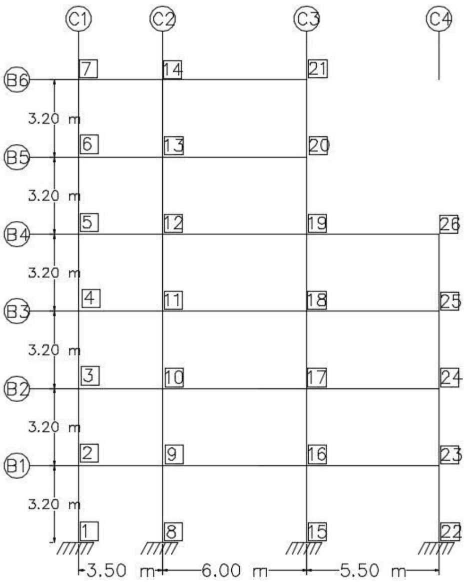


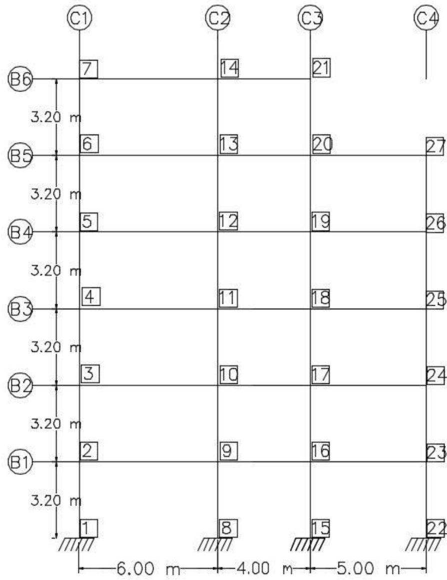
frames are modeled as common moment frames by OpenSEES[11]. The selected models are illustrated as below, regardless of infills.

Figure 1. Model 1 with symmetric spans   Figure 2. Model 2 with asymmetric spans   Figure 3. Model 3 with asymmetric

[1] Load and Resistance Factor Design







spans
Figure 4. Model 4 with symmetric spans and asymmetric heights   Figure 5. Model 5 with asymmetric spans and heights, regardless of infills
Figure 6. Model 6 with asymmetric spans and heights

All models are 3.2 m in height which is the same as common floor heights in residential buildings and offices. The first story has a height of 3.2 m and columns with fixed supports.

4. Sections

All columns and beams as been chosen from IPE sections made of steel st-37 with density of 7850 kg/m^3.

Table 1. Specifications of steel sections

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

5.Loading

In fact, loading states indicate characteristics of the loads applied to the structure; three loading states have been introduced for the frame, although it is possible to consider numerous loading states in a structure. In this study, Load types are as follows: dead loads (DEAD), live loads (LIVE), and earthquake loads (EQ). The amounts of loads applied to the structure have been listed in the table below [12]. All frames have been equally loaded as follows:

Table 2. The amounts of loads applied to the structure

Load	Story	Dead load	Live load	Unit
Gravity load	first floor – five floor	460	200	-
	Sixth floor ((Roof	520	150	

6.The weight of brick infills:

Assuming the same height in all frames, variable beam lengths, and constant thickness of whole walls, infills are of the same geometrical properties in all frames:

Thickness of walls (t): 20 cm; Height of walls (b): 3.20 m; Length of walls (l): varied based on beam spans.

The walls are made of clay brick and cement-sand mortar with density of 1300 kg/m3 and 2700 kg/m3, respectively. The density of wall should be determined to calculate its weight. Since the masonry wall consists of two types materials with different unit weights including clay brick or blocks and cement-sand mortar, it is assumed that each basic infill panel element has a unit volume (wall = 1 V infill) and a brick is 10 times as great as mortar in volume; so that total density of the wall can be determined. Given the assumptions and considerations above, weight and distributed dead load of the wall has been calculated as follows ($W_{\text{infill wall}}$ = 740 kg/m):

7. Applied accelerograms

According to the regulations, records used to determine the effect of ground movements should reflect actual ground movements in construction site as far as possible. Since no accelerogram can lonely generate the response adapted to response spectrum in all frequencies, it is necessary to choose and employ a set of scaled natural accelerograms. In this study, it has been attempted to utilize a near-field earthquake. According to obvious definition of near-field earthquake specifying a distance up to 15 km from the station, the record has been chosen and applied to the structure. The selected accelerogram has been provided by the PEER website. In this paper, various records were selected from near-fault earthquakes. To carry out nonlinear dynamic analyses, 7 earthquake records of different conditions such as geotechnical properties of soil, faulting mechanism, distribution pattern of seismic waves, etc. have been selected, which adapted to seismic design response spectrum of the frames.

Table 3. Specifications of the earthquake records

No	Earthquake	Station	PGV	PGA	PGD	Magnitude

			(cm/Sec)	(g)	(cm)	(M)
1	Imperial Valley	Elcentro Array	10.7	0.205	9.16	7.2
2	Tabas , Iran	Bajestan 69	5.7	0.029	6.16	7.7
3	Kobe	FUK 0	1.7	0.01	0.67	6.8
4	Chi-Chi, Taiwan	CHY080	49.0	0.724	27.82	7.6
5	Bam , Iran	Baft 2003	2.17	32	0.49	6.6
6	Northridge	24278Castaic - Old Ridge Route	12.4	0.217	1.94	6.7
7	LOMAP/CAP-UP	Capitola 47125	19.4	0.541	2.6	6.9

8. The validated model

An experimental research by Jinkoo Kim et al has been used to validate the results of selected models by OpenSEES, which was about progressive collapse of steel frames with various bracing systems caused by sudden removal of a column in a braced span. To validate the results of simulations in this study, the behavior of braced frame in case of sudden removal of a column in a braced span has been reassessed using Chevron brace configurations through a nonlinear static method. The frame has been loaded according to ASCE7-05 and designed using IBC2006. The seismic design has been accomplished based on spectral accelerations of Ss and S1 equal to 1.5 and 0.6, respectively. Dead and live loads applied in the study have been equal to 430 kg/m and 240 kg/m, respectively. The frame has 4 stories with each one 3.1 in height. It has 4 spans with each one 6.1 m in length. ASTM A992 steel has been employed to fabricate the beams and columns. The braces have been made of A500-46 steel hollow sections. Specifications of the frame studied by Jinkoo Kim et al have been considered in this paper. The structure has been loaded based on the loading composition proposed by GSA in terms of DL + 0.25LL. Structural designs and strength assessment of the structure against progressive collapse has been done by SAP2000.

The frame and removed column have been demonstrated in figure 7 and load factor-displacement graph at the point above removed column in main structure designed by Jinkoo Kim has been illustrated in figure 8. According to the graph in figure 8, maximum load factor equaled 2 and the structure was then confronted a considerable loss of capacity.

Figure 7. The position of removed column in the frame

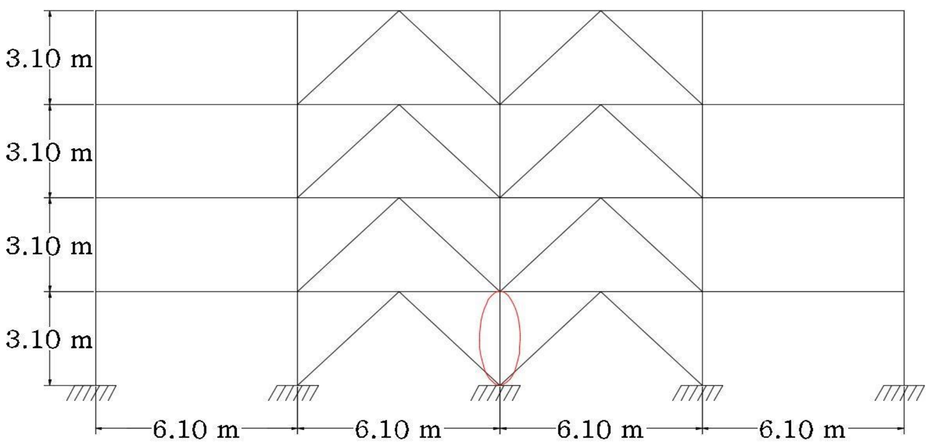


Figure 8. Load factor-displacement graph by Jinkoo Kim

Figure 9. Load factor-displacement graph by

OpenSEES

Figure 9 shows load factor-displacement graph at the point above removed column in the structure modeled via OpenSEEs, which has been revalidated by the author. According to the figure, maximum load factor applied to the structure was 2.05 that properly adapted to the results presented by Jinkoo Kim.

9. Analysis procedure

9-1. 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 [13]:

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

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.)

9-2. Step by step procedure of linear static analysis

The step by step procedure for performing LS analysis proposed in GSA 2003 is as follows [14].

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 overstrength factor equal to 1.1).

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[15] 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 (3)

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

9-3. 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 [15].
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. 10) 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 [16].

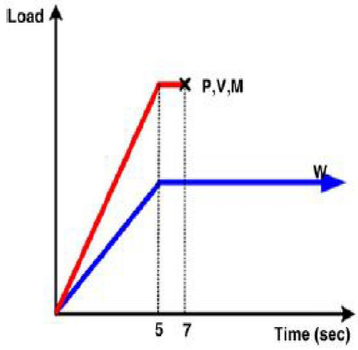


Fig.10-Applying loads for dynamic analysis procedure.

10. Dynamic analysis of models

When a set of records were selected and scaled for each frame, the historical time analysis was accomplished via OpenSEES. A viscous dissipation of 5% (ξ=5%) as a normal value was considered in all frames and various wall layouts. Each frame with various wall layouts was affected by the set of records in different intensities and their various responses were measured in regard to drift and lateral displacement of floors, base shear and forces in elements. Given that the frames were modeled 2-dimensionally, just one of horizontal components of records was used in the analysis. In addition, interactions between soil and the structure were ignored but P-Δ effects were considered in the analysis.[17,18]
Exterior frames of the modeled structure were numerically analyzed via OpenSEES. In this study, loading ratio was not applied to modeled materials because the structural behavior after sudden removal of the column was not fast enough to consider the effect of loading ratio. Since the dynamic behavior was caused by sudden removal of a column, it was not included in cyclic loading of structures under seismic loads. It was not necessary to use a complex hysteretic model. A 5% dissipation rate was described as critical dissipation; the value is usually adopted to analyze structures exposed to great displacements. The gradual (progressive) collapse analysis was carried out at various points through the removal of a column, in accordance with GSA2003 and DoD2005 [19,20].

11. Progressive collapse analysis

To determine strength of moment frames designed according to UFC [21], columns were removed at different points in each of 12 models in this study. Given that surrounding frames had 4 spans according to (Figure 11), individual exterior and interior columns were suddenly removed from the structure in first floor regarding progressive collapse pattern and resultant structural response was investigated. In this study, three different scenarios were generally assessed based on which three columns were removed in all 12 models and the most critical column was identified in regard to the results. Progressive collapse scenario commonly used refers to the assumption due to which columns are destroyed and removed from the structure by progressive collapse factors and structural behavior is studied in that situation. (Figure 11) and (Table 4) show a column removal scenario applied in AP design method to evaluate the structure against progressive collapse. It refers to a real incident and the explosion and consequences have been proved through experiments. Table 6 represents various scenarios about removal of a member.

Table 4. States of progressive collapse analysis in 12 models on 6 floors

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

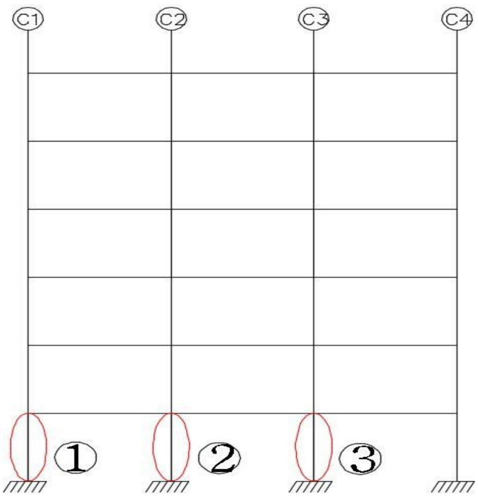


Figure 11. The column removal on ground floor in various scenarios

Entire 12 models were studied based on three various scenarios and resultant displacement and corresponding acceleration were compared and assessed after the column removal and maximum displacement and acceleration of floors were also listed in Table 7. Calculated period and frequency of all models were represented in Table 5 and 6, respectively.

Mood Story	Story 1	Story 2	Story 3	Story 4	Story 5	Story 6	Mood Story	Story 1	Story 2	Story 3	Story 4	Story 5
Model 1	2.257	0.79	0.439	0.289	0.212	0.205	Model 7	0.562	0.195	0.185	0.128	0.108
Model 2	2.255	0.789	0.435	0.288	0.218	0.212	Model 8	0.582	0.202	0.187	0.140	0.116
Model 3	2.247	0.787	0.438	0.290	0.213	0.203	Model 9	0.566	0.196	0.184	0.128	0.114
Model 4	2.029	0.812	0.443	0.284	0.221	0.202	Model 10	0.501	0.198	0.175	0.120	0.103

Model 5	1.995	0.792	0.436	0.279	0.219	0.194	Model 11	0.513	0.204	0.175	0.130	0.111
Model 6	2.109	0.759	0.444	0.303	0.218	0.201	Model 12	0.529	0.188	0.1760.176	0.127	0.105

Table 7. Calculated period (s)

Table 8. Frequency (rad/s)

Mood Story	Story 1	Story 2	Story 3	Story 4	Story 5	Story 6	Mood Story	Story 1	Story 2	Story 3	Story 4	Stor
Model 1	2.782	7.945	14.300	21.690	29.540	30.527	Model 7	11.167	32.177	33.830	48.759	57.
Model 2	2.785	8.000	14.413	21.773	28.771	29.601	Model 8	10.788	31.143	33.590	44.933	54.
Model 3	2.795	7.974	14.333	21.700	29.531	30.902	Model 9	11.092	32.009	34.129	44.966	54.
Model 4	3.095	7.737	14.175	22.050	28.491	31.104	Model 10	12.532	31.719	35.820	52.453	60.
Model 5	3.149	7.924	14.404	22.502	28.639	32.321	Model 11	12.244	30.730	35.886	48.518	56.
Model 6	2.979	8.282	14.163	20.716	28.981	31.196	Model 12	11.866	33.460	35.732	49.629	59.

Critical columns of each structure were chosen in models (1) to (12) and the results of displacement and acceleration of floors were explored and maximum displacement, critical column acceleration and relevant graphs were presented in Figures 12-27, according to Table 8.

Table 9. Maximum displacement and acceleration in models (scenario (1), (2) and (3)) on 1st floor

Model	Scenario	Max Displacement (cm)	Max Acceleration (cm/s^2)	Model	Scenario	Max Displacement (cm)	Max Acceleration (cm/s^2)
Model 1	C1	0.0370	1.675	Model 7	C1	-0.0009	4.448
	C2	0.0338	1.666		C2	0.0118	1.478
	C3	0.0326	1.664		C3	0.0951	1.559
Model 2	C1	0.0348	1.713	Model 8	C1	0.0008	3.246
	C2	0.0406	1.734		C2	0.0106	1.216
	C3	0.0350	1.729		C3	0.0113	1.888
Model 3	C1	0.0305	1.670	Model 9	C1	0.0106	1.410
	C2	0.0299	1.576		C2	0.0106	1.320
	C3	0.0318	1.535		C3	0.0086	1.541
Model 4	C1	0.0617	1.615	Model 10	C1	-0.0022	2.381
	C2	0.0386	1.560		C2	0.0060	1.725
	C3	0.0442	1.532		C3	0.0061	1.619
Model 5	C1	0.1605	1.819	Model 11	C1	0.0081	6.330
	C2	0.0403	1.624		C2	0.0068	1.530
	C3	0.0451	1.664		C3	0.0077	1.398
Model 6	C1	0.0315	1.665	Model 12	C1	0.0033	2.228
	C2	0.0353	1.601		C2	0.0023	1.601

	C3	0.0371	1.603		C3	0.0029	3.139

Table 10. Maximum displacement and acceleration in critical mode of models on 1st floor

Model	Critical Column	Max Displacement (cm)	Max Acceleration (cm/s^2)	Model	Critical Column	Max Displacement (cm)	Max Acceleration (cm/s^2)
Model 1	C2	0.0338	1.666	Model 7	C2	0.0118	1.478
Model 2	C3	0.0350	1.729	Model 8	C3	0.0113	1.888
Model 3	C2	0.0299	1.576	Model 9	C2	0.0106	1.320
Model 4	C2	0.0386	1.560	Model 10	C2	0.0060	1.725
Model 5	C3	0.0451	1.664	Model 11	C3	0.0077	1.398
Model 6	C2	0.0353	1.601	Model 12	C2	0.0023	1.601

Displacement-acceleration graphs

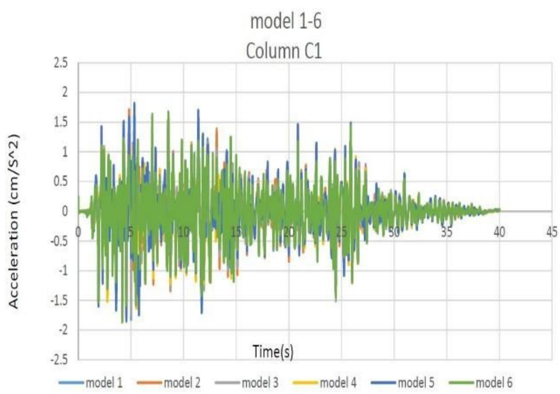


Figure 12. Acceleration graph in models (1) to (6) - scenario (1)

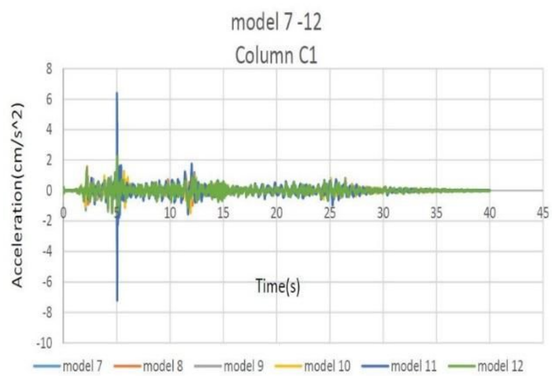


Figure 13. Acceleration graph in models (7) to (12) - scenario (1)

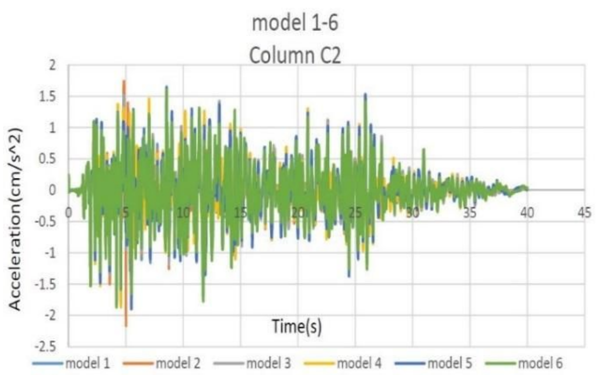


Figure 14. Acceleration graph in models (1) to (6) - scenario (2)

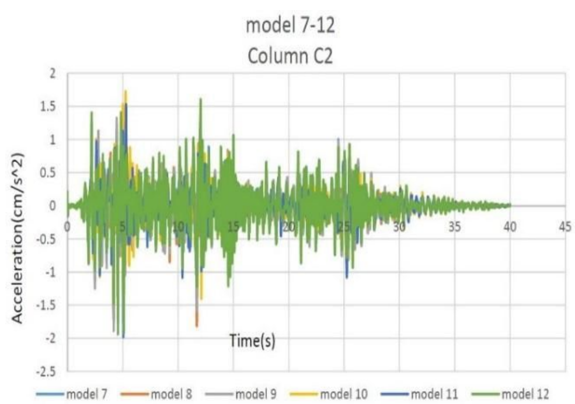
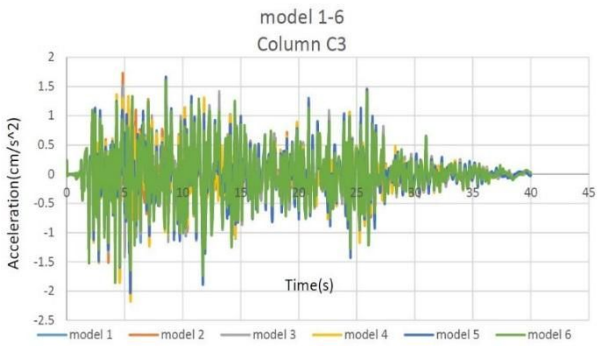


Figure 15. Acceleration graph in models (7) to (12) - scenario (2)



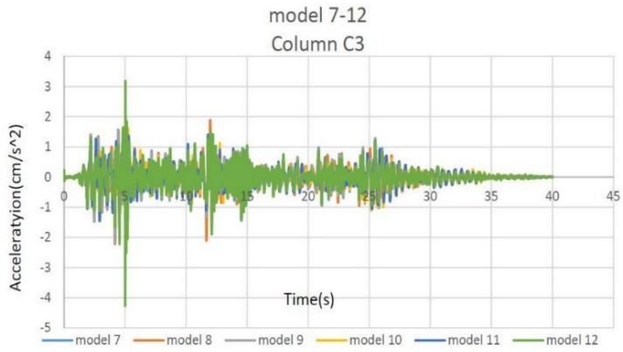
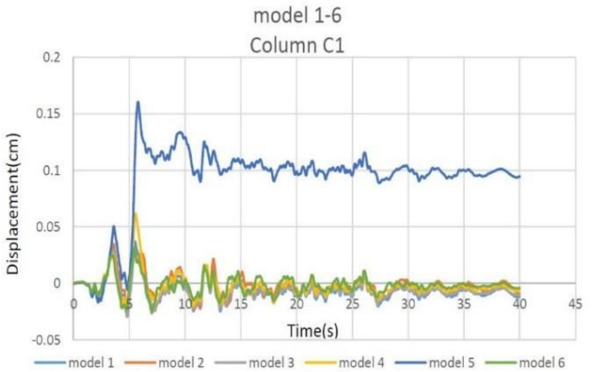
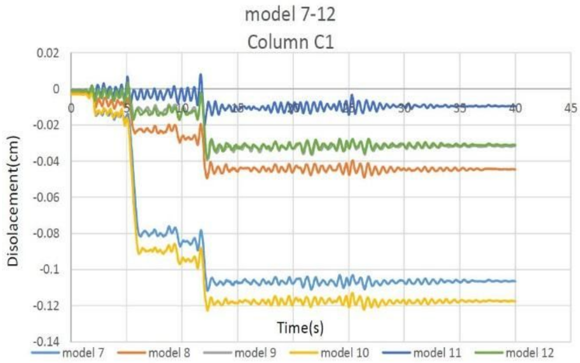


Figure 16. Acceleration graph in models (1) to (6) - scenario (3) Figure 17. Acceleration graph in models (7) to (12) -



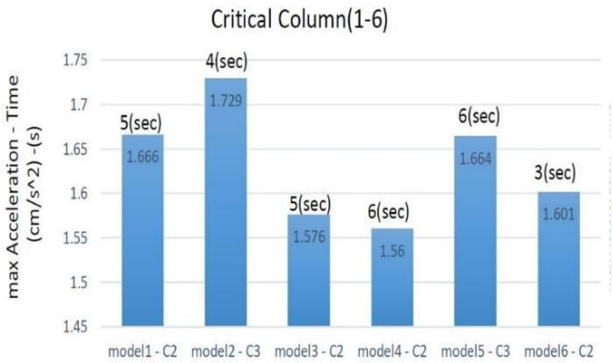


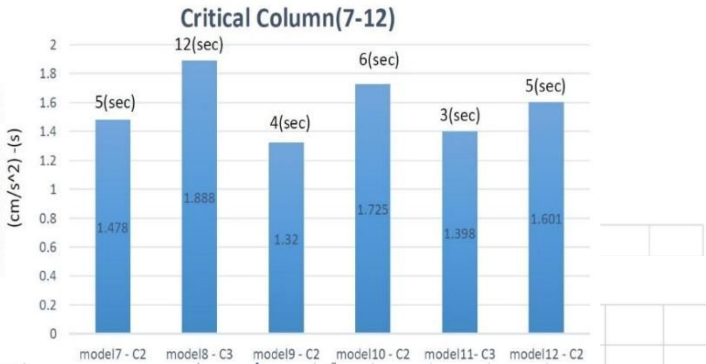
scenario (3)

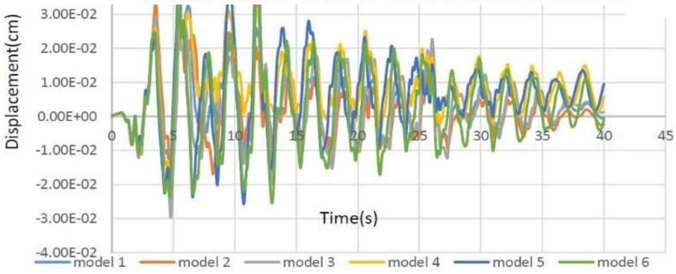
Figure 18. Displacement graph in models (1) to (6) - scenario (1) Figure 19. Displacement graph in models (7) to (12) - scenario (1)

Figure 20. Displacement graph in models (1) to (6) - scenario (2) Figure 21. Displacement graph in models (7) to (12) - scenario (2)

Figure 22. Displacement graph in models (1) to (6) - scenario (3) Figure 23. Displacement graph in models (7) to (12) - scenario (3)







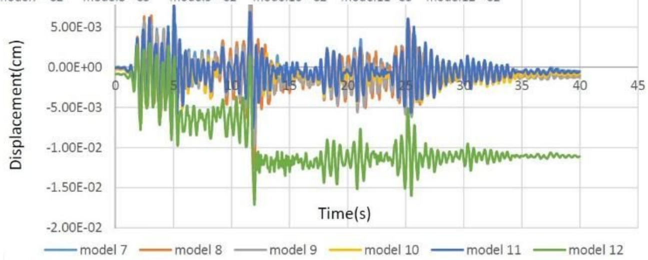
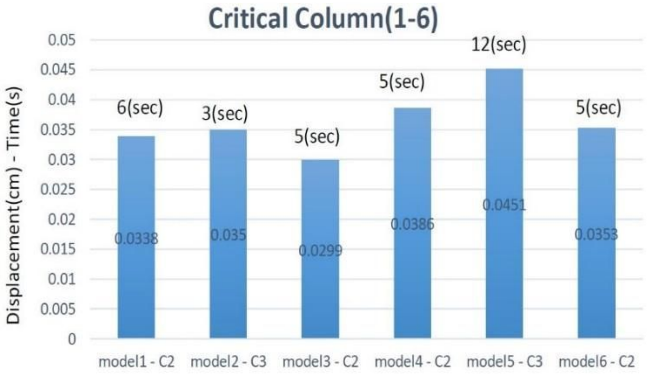


Figure 24. Maximum acceleration graph in models (1) to (6) (disregarding infill effect) - critical column

Figure 25. Maximum acceleration graph in models (7) to (12) (regarding infill effect) - critical column



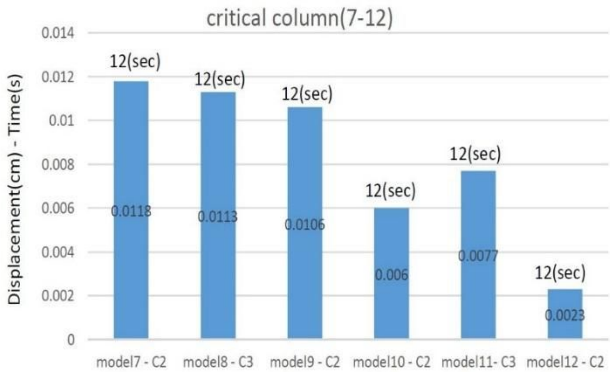


Figure 26. Maximum displacement graph in models (1) to (6) (disregarding infill effect) - critical column

Figure 27. Maximum displacement graph in models (7) to (12) (regarding infill effect) - critical column

12.Analysis results

In this paper, the effect of infills on asymmetric steel moment frames was addressed and various models with asymmetric height and spans were generated and analyzed via OpenSEES. Progressive collapse was then controlled through alternative path method (APM) and structural sustainability and the forces imposed to adjacent members were compared in each state. Following results could be described according to Tables 5, 6, 7 and 8 and also graphs in Figures 24 , 25 , 26 and 27 representing maximum displacement-time and maximum acceleration-time of relevant 12 models, respectively:

1. Given the dynamic analysis, frames have a great potential for damage caused by progressive collapse.
2. Infills increase stiffness, reduce the structure's period, cause rapid damages in columns and retard progressive collapse in moment frames in addition to dead and live loads; so that such delay is an important factor in failure. Therefore, it can be concluded that infills have a positive impact on progressive collapse.
3. The results of OpenSEES show a high period and low frequency in structural frames disregarding the effect of infills; when the effect of infills is considered, the structure's period is decreased by 1.5 s on 1st floor and positive role of infills becomes obvious to upper floors; moreover, the effect of infills enhances the structure's frequency by 8 rad/s on 1st floor in models and the trend is followed toward 6th floor (the top floor).
4. According to maximum displacement-time graphs related to models (1) to (12), as asymmetry and irregularity increase in spans, displacement rises in models; but the effect of infills significantly reduces displacement of the structure and extends duration of the displacement.
5. An investigation into vulnerability of models in critical states indicates that which irregularities have shown the best behavior; according to the graphs in Figure 24 , 25 , 26 and 27, model (3) have shown a better behavior compared to other models regarding maximum displacement and acceleration and its described scenario; it demonstrates that irregularity on side spans can cause a lower impact than interior spans.
6. Hinges are created in all spans, even in spans with no column removed; so that the structure can be rehabilitated against progressive collapse through strengthening a few members. The effect of infills in models (7) to (12) retards the formation of plastic hinges throughout the structure.

7. The result illustrate that infills have a more positive role in retarding progressive collapse in asymmetric moment frames with asymmetrical spans. Due to removal of two stories in larger spans, an investigation on models disregarding the effect of infills indicate that model (5) have shown a better reaction than the other so that the structure's period has decreased and consequently, the structure's frequency has increased.

As practical achievements in designing steel moment frames, these findings have been recommended to reduce structural vulnerability in regard to architectural limitations on location of columns.

13.References

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