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Research Article

Evaluating the effect of infill compressive strength on the progressive collapse process of steel frames

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Abstract: Structural safety has always been a key principle for designer engineers. One of the issues that structural engineers have been considered in recent decades is progressive collapse. Investigating the progressive collapse have become more important in a situation where terrorist attacks on civilian places and facilities are on the rise and buildings are increasingly exposed to the impact of explosions, collisions with vehicles, etc., requiring special attention and consideration. Accordingly, it is necessary to evaluate the factors affecting the progressive collapse in the operation of structures. One of the factors influencing the behavior of structures is masonry infills, which only their weight is considered in designing. The present study aimed to investigate the effect of compressive strength of infills on the progressive collapse process of 2-10 floor steel frames and compare their results with infill-free frames. The process of progressive collapse is performed by alternate path method and losing the middle floor column in OpenSees software. The results indicated that the addition of each MPa to the infill compressive strength will increase by about 33% and also the addition of each floor to the existing floors will increase the strength of the frame about 83%. Then, a relationship is proposed to determine the final strength of the frame based on infill's compressive strength changes and the number of floors. In this regard, the proposed relationship in all cases has a maximum error of 5%.

Keywords: Progressive collapse, steel frame, infill, compressive strength, column loss

1. Introduction

Structural safety has always been a key for design engineers. One of the things that structural engineers have been paying close attention to in recent decades is progressive collapse. Investigating the progressive collapse in building design becomes very important for several reasons. In a situation where terrorist attacks on civilian places and facilities are on the rise, it is necessary to pay special attention to the design of buildings in the face of explosive loads, collisions of land vehicles and aircraft. Ronan Point, the Alfred Murrah federal building, Khobar Tower, the US World Trade Center twin tower, etc. accidents are examples that have caused progressive collapse to structures in whole or in part. Engineers' attention to the issue of progressive collapse first began after the demolition of the Ronan Point building in London in 1968. Then, the institutions made a lot of efforts to prepare and compile building codes to provide solutions to prevent and reduce the phenomenon of progressive collapse. The September 11, 2001, collapse of the World Trade Center twin towers and adjacent buildings shocked researchers to scrutinize the phenomenon of progressive collapse. Many researchers around the world have studied how the World Trade Center towers were created and shaped. In the aftermath of September 11, progressive collapse of high-rise buildings was considered.

The first regulation on progressive collapse was published by GSA in 2003, which was an important step in assisting researchers in this area. Following the GSA Regulation in 2003, the Department of Defense issued Regulation UFC4-023-03 [2]. A new wave of investigations into progressive collapse began after the release of the UFC regulations. After the publication of the NIST Code in 2007 and the amendment of the UFC Code in 2009, significant measures were taken to prevent progressive collapse of structures. According to ASCE7-05 [4], UFC 2009 defines progressive collapse as "the spread of an initial localized collapse from one element of another to an element that ultimately results in the collapse of the entire structure or a large part of it disproportionately." The ASCE7-05 standard also states that the building should be designed in such a way that according to the ASCE7-05 standard, the two main approaches to prevent progressive collapse are direct and indirect designs. Direct design provides explicit requirements for structural resistance to progressive collapse, including Alternate Path (AP) and Specific Local Resistance (SLR) methods. In indirect design, resistance to progressive collapse is achieved through minimum levels of strength, cohesion and ductility. Regulation UFC4-023-03 introduces the tensile force (TF) method in indirect design. The altrenate path (AP) method shows that the structure is able to redistribute the load after removing the column or wall, according to which the deformation and internal forces created in the members do not exceed the allowable values. In the alternate path (AP) method, the structure resists collapse due to the membrane flexural response. The types of analysis methods in the alternate path (AP) method are: linear static analysis, nonlinear static analysis and nonlinear dynamic analysis.

In a laboratory and numerical study, Lee et al., (2016) investigated the progressive collapse of a concrete frame with a building infill without opening. For this purpose, the four-opening frame with two floors with a scale of one-third, which only has a masonry infill on the second floor, has been first evaluated in a laboratory. Then, the numerical modeling has been performed to investigate the important and influential factors in the behavior of the tested frame after validation. The results of study mentioned above showed that the masonry infill has a significant effect on the strength of the concrete frame against progressive collapse [6]. In a laboratory and numerical study, Lee et al. (2016) investigated the effect of opening infill frames on the progressive collapse of reinforced concrete frames. For this purpose, concrete frame without infill and with opening infill have been evaluated and compared in the laboratory. It was found that existing openable frames increases the stiffness, reduces the ductility and increases the maximum strength of reinforced concrete frames against progressive collapse [7].

Emanuele Brunesi (2017) conducted a numerical study to develop fragility curves based on progressive collapse in reinforced concrete frames. In this research, a probabilistic framework for analyzing and preparing fragility curves under progressive collapse on low-rise frames has been investigated [8]. Eren (2019) conducted a numerical study to examine the effect of the presence of infills on the behavior of reinforced concrete frames under progressive collapse. The results indicated that existing infills increases the stiffness and the maximum strength of reinforced concrete frames against progressive collapse [9].

Zhao et al., (2020) conducted a laboratory study to investigate the phenomenon of progressive collapse in precast reinforced concrete structures and in-situ reinforced concrete. It was found that the capacity of the precast concrete sample is 76.9% higher than that of in situ reinforced concrete sample. The final displacement of the middle column in the precast reinforced concrete sample is 106.1% higher than that of the in situ reinforced concrete sample [24]. Gholamreza Nouri et al. (2017) performed a numerical study and investigated the effect of various types of steel connections on the behavior of steel bending frame. The results indicated that the connection with reduced cross section has more resistance than the welded joint reinforced by welding [10]. Reza Abbasnia et al. (2016) conducted a numerical study, investigating the progressive collapse of concrete structures with different ductility. It was found that although the special seismic design is more ductile than the conventional design, it will not necessarily perform better under the column loss process [11]. Gholampour et al. (2018) investigated the progressive collapse under column loss on the seismic performance of dual steel structures. The results indicated that the most critical way to remove a column is to remove a corner column. The removal of the corner column and the braces attached to it rejected the level of life safety performance and the structure did not meet the acceptance criteria [12]. Mehdizadeh et al. (2018) explored the possibility of progressive collapse in steel bending frames (normal, medium and special) due to column loss. The results indicated that the possibility of damage in special steel bending frames is more than that of the medium and ordinary bending frames. In addition, unlike seismic retrofits, which provide ductility as an important way to reduce damage to buildings, increased strength and stiffness of members in the gravity loads-resulted collapse can limit the spread of collapse [13].

The experimental studies on laboratory and numerical specimens show that infill plays a significant role in the stability of the structure after column loss. Many studies have been performed on progressive collapse and the effects of infill in frames under progressive collapse have been investigated. However, the effect of increasing the compressive strength of the infill has not been properly investigated. By examining the compressive strength of infills from 1 to 10 MPa on the progressive collapse process of 2 to 10 steel frames and comparing their results with non-infill frames, the authors have tried to examine the effects of increasing intermediate compressive strength on the behavior of steel frames under progressive collapse. In the study mentioned above, the effect of compressive strength of infills on the progressive collapse process of steel frames in OpenSees software [14] has been investigated. Since modeling infills increases the complexity of modeling and analysis, a coefficient has been presented to apply the effect of the presence of infills with different values of compressive strength.

2. Modeling

The finite element method has been used to accurately investigate the progressive collapse of 2- to 10-story steel buildings. The progressive collapse modeling of the buildings under study was performed in OpenSees software. OpenSees finite element software is a very powerful tool in the field of soil and structural operating systems. In this study, all models are first designed separately using ETABS software [18], an advanced software in building modeling and design. The models has been designed based on the Iranian regulations (Chapter 6 and Article 10 of the national regulations) and the standard 2800, fourth edition [17,16,15]. To design against lateral load, the equivalent static analysis method in ETABS software has been used due to the regularity of the buildings in the facade and plan. Due to some limitations, the progressive collapse phenomenon cannot be simulated using ETABS software. The model of steel buildings with flexural frame system has been modeled using OpenSees software to study the phenomenon of progressive collapse after design. The studied structures according to Figure 1 in all floors in the y direction have 5 openings with a length of 5 m and in the x direction have 4 lengths of 6 m and the height of all floors is 3.2 m.

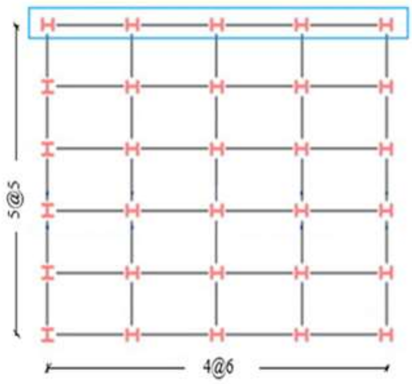

Figure 1. Plan of the buildings under study

1.2. Introducing the models

In the present study, the studied frames under the phenomenon of progressive collapse have 2 to 10 floors with variable compressive strength in infills (1 to 10 MPa). For each story, 11 frame models have been examined, one frame related to non-infill state and the other 10 models related to the infill states with compressive strength ranging from 1 to 10 MPa. The total number of models studied in this study is 99 models. It is worth noting that existing infills typically have a strength between 2.5 and 6 MPa. In this study, the compressive strength of the infills is considered to be from 1 to 10 MPa to properly determine the effect of increasing the intermediate compressive strength on the progressive collapse phenomenon. The columns are of BOX type, the beams are of I type. The dimensions of steel sections are listed in Table 1.

Table 1. Sections used in frames modeling

Floors 1		**Floors 2**	**Floor 3**	**Floors 4**	**Floors 5**	**Floors 6**	**Floors 7**	**Floors 8**	**Floors 9**	**Floors 10**
1	Beam	IPE18	IPE18	IPE20	IPE20	IPE22	IPE22	IPE24	IPE24	**IPE26**
	Column	Box20*20*1	Box20*20*1	Box25*25*1	Box25*25*1.5	Box30*30*1.5	Box30*30*1.5	Box35*35*1.5	Box40*40*1.5	**Box45*45*1.5**
2	Beam	IPE18	IPE18	IPE20	IPE20	IPE22	IPE22	IPE24	IPE24	**IPE26**
	Column	Box20*20*1	Box20*20*1	Box25*25*1	Box25*25*1.5	Box30*30*1.5	Box30*30*1.5	Box35*35*1.5	Box40*40*1.5	**Box45*45*1.5**
3	Beam		IPE18	IPE18	IPE20	IPE20	IPE20	IPE22	IPE24	**IPE24**
	Column		Box20*20*1	Box20*20*1	Box25*25*1.5	Box25*25*1.5	Box25*25*1.5	Box30*30*1.5	Box35*35*1.5	**Box40*40*1.5**
4	Beam			IPE18	IPE18	IPE20	IPE20	IPE22	IPE22	**IPE24**
	Column			Box20*20*1	Box20*20*1	Box25*25*1.5	Box25*25*1.5	Box30*30*1.5	Box35*35*1.5	**Box40*40*1.5**
5	Beam				IPE18	IPE18	IPE20	IPE20	IPE22	**IPE22**
	Column				Box20*20*1	Box20*20*1	Box25*25*1.5	Box25*25*1/5	Box30*30*1.5	**Box35*35*1.5**
6	Beam					IPE18	IPE18	IPE20	IPE22	**IPE22**
	Column					Box20*20*1	Box20*20*1	Box25*25*1/5	Box30*30*1.5	**Box35*35*1.5**
7	Beam						IPE18	IPE18	IPE18	**IPE22**
	Column						Box20*20*1	Box20*20*1	Box20*20*1	**Box30*30*1.5**
8	Beam							IPE18	IPE18	**IPE22**
	Column							Box20*20*1	Box20*20*1	**Box30*30*1.5**
9	Beam								IPE18	**IPE18**
	Column								Box20*20*1	**Box20*20*1**
10	Beam									**IPE18**
	Column									**Box20*20*1**

2.2. Modeling details

In the software model, all the beams and columns are modeled using the forceBeamColumn element. This command is used to construct the nonlinear beam-column element, which extensively considers plasticity along the element to evaluate the actual behavior of the element in the analysis. These types of elements have a flexural connection. The infills are modeled using the trussSection element. This command is used to construct a truss element with a cross-section assignment. Geometric Transformation has been used to estimate the geometric nonlinear behavior. This command is used to construct a convergent coordinate conversion, which transmits stiffness and force by performing a nonlinear geometric transformation from the local system to the general system in a very precise manner.

2.3. Properties of materials

The model features include nonlinear properties of materials, geometric nonlinear behavior, and nonlinear analysis. Steel01 materials are used for steel components in OpenSees software. These materials are uniaxial bi-linear with kinematic and isotropic hardening. The behavior of the materials is elastic up to the yield stress and then they enter the strain hardening phase until the final stress is reached. Yang modulus of steel components is equal to 2.1 x 105 MPa and Poisson's coefficient is equal to 0.3. In this study, ST37 steel was used in all structural components. The yield stress and rupture of steel components are 240 and 370 MPa, respectively. Concrete01 materials are used for infill components in OpenSees software. The compressive parameters of these materials are entered with a negative sign. These materials are defined as uniaxial with difficulty of loading and unloading that decreases linearly. The compressive strength of the infill varies from 1 to 10 MPa. The Young modulus of the infills is considered to be 550 times the compressive strength of the infills. Material properties and design parameters are summarized in Table 2.

Table 2. Design parameters in the software model

Design parameters	Value
Dead load	5 (kN/m^2)
Live load	2 (kN/m^2)
Story height	3.2m
Opening length	6m
Type of soil	II
Steel	240 (N/m^2)
Infill strength	1-10 (N/m^2)
Ceiling	Two-way slab

2.4. Supporting and meshing conditions

In this study, the support of all columns is defined as fixed to support the boundary conditions. In addition, the mesh size of the elements used is determined in such a way that the software can calculate the answers with appropriate accuracy.

2.5. Criteria for accepting collapse

Table 3 presents the criteria for accepting the collapse of members in steel frames under the phenomenon of progressive collapse according to the GSA regulations. By observing the criteria for accepting the collapse of members, the maximum allowable values of the members under the progressive collapse analysis based on the table are provided.

Table 3. The criteria for accepting member collapse in steel frames

Member	Circulation (Rad)	Performance level
Beam	0.21	CP
Column	0.21	LS

2.6. Column loss mechanism

In this study, nonlinear static analysis with alternate path (AP) method proposed by GSA and DoD regulations is used. In all models, the ground floor middle column loss scenario is used. The Alternate Path (AP) method is independent of the collapse factor, which is a concentrated load of unit size applied in the opposite direction to the location of the removed column on the top floor.

2.7. Infill modeling

In this research, the method presented in the Seismic Improvement Instructions of Existing Structures (Journal 360) has been used for modeling masonry infills [19]. In this method, the infill is modeled by pressure handles equivalent to the width a. The width of the replacement strut is calculated according to Equation (1). The modeling parameters are shown in Equations (1 and 2) and also how to place equivalent struts in the software model is shown in Figure (2).

$$a = 0.175[\lambda_1 h_{col}]^{-0.4} r_{inf} \quad (1)$$

$$\lambda_1 = [\frac{E_{me} t_{inf} \sin(2\theta)}{4E_{fe} I_{col} h_{inf}}] \quad (2)$$

In the above equations, a is the width of the strut replacing the infill frame (mm), 1λ a coefficient to calculate the width of the frame, h_{col}, the height of the center to the center of the upper and lower beams of the infill frame (mm), r_{inf} is the diameter length of the infill frame (mm), h_{inf} indicates the height of the infill frame (mm), E_{fe} represents the flexibility coefficient of frame materials (N2 / N), I_{col} is the moment of column inertia (4 mm), t_{inf} is infill thickness and equivalent strut (mm), θ is an angle whose tangent is obtained by dividing the height of the column by the length of the beam. E_{me} is the infill elasticity coefficient (mm^2 / N) which is considered to be 550 times the expected compressive strength [19].

$$E_{me} = 550 f_{me} \quad (3)$$

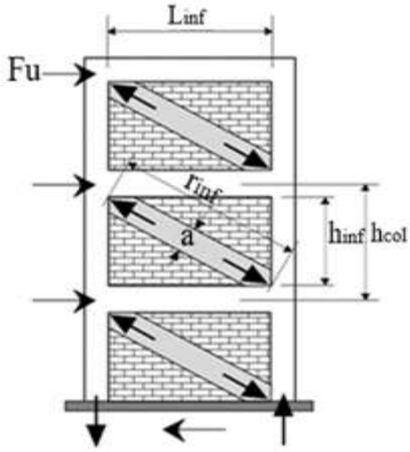

Figure 2. Replacing the infill with an equivalent strut

Equation (1) is presented to obtain the width of the equivalent strut to the infill when lateral loads such as an earthquake are applied. In progressive collapse, since the load is applied in a direction perpendicular to the structure, this equation changes to Equation (4).

$$a = 0.175[\lambda_1 L_b]^{-0.4} r_{inf} \qquad (4)$$

In the equations (4 and 5) L_b is the length of the beam between the center to the center of the columns around the infill (mm), I_b expresses moment of inertia of the beam (4 mm), L_{inf} represents the length of the infill (mm), θ is the angle whose tangent is obtained by dividing the length of the beam by the height of the column.

$$\lambda_1 = [\frac{E_{me} t_{inf} \sin(2\theta)}{4 E_{fe} I_b L_{inf}}] \qquad (5)$$

3. Validation

It is very difficult to evaluate progressive collapse using a real-scale laboratory model. The finite element method is an appropriate option to investigate the phenomenon of progressive collapse. Using the finite element method, the types of models can be investigated under the effect of progressive collapse. In this research, the laboratory model of Lee et al. [6], whose geometry and details of the laboratory model are shown in Figure 3 and Table 3, has been used to validate the software model. For this purpose, Lee et al., simulated the progressive collapse scenario in laboratory samples by applying a quasi-static load on the center column and applied the load based on the Push Down method, as a displacement control, and applied by two jacks. Laboratory specimens are also properly secured by rollers on either side of the frame to prevent off-plate displacement. In making the laboratory sample for the first and second floors, concrete with compressive strength of 41.3 and 31.8 MPa, respectively, and bars with yield strength and final strength of 415 and 588 MPa, respectively, have been used. Besides, the compressive and shear strength of the masonry wall were 12.8 and 1.08 MPa, respectively. Table 4 shows the materials and elements used for modeling and validation in OpenSees software, and Figure 4 shows the force-displacement diagram of the laboratory specimens. According to Figure 4, the laboratory sample modeling in OpenSees software has been conducted with acceptable and sufficient accuracy.

Table 4. Characteristics of the software model for validation

	Story height (mm)	Opening length (mm)	Beam's bar	Column's bar

Floor 1	1400	1700	4Φ 8	12Φ 8
Floor 2	1100	1700	4 Φ8	12Φ 8

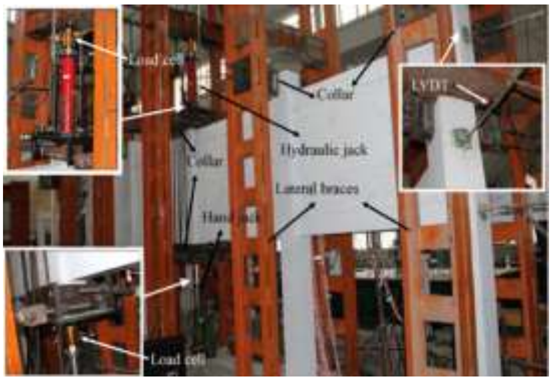

Figure 3. Li's laboratory sample image

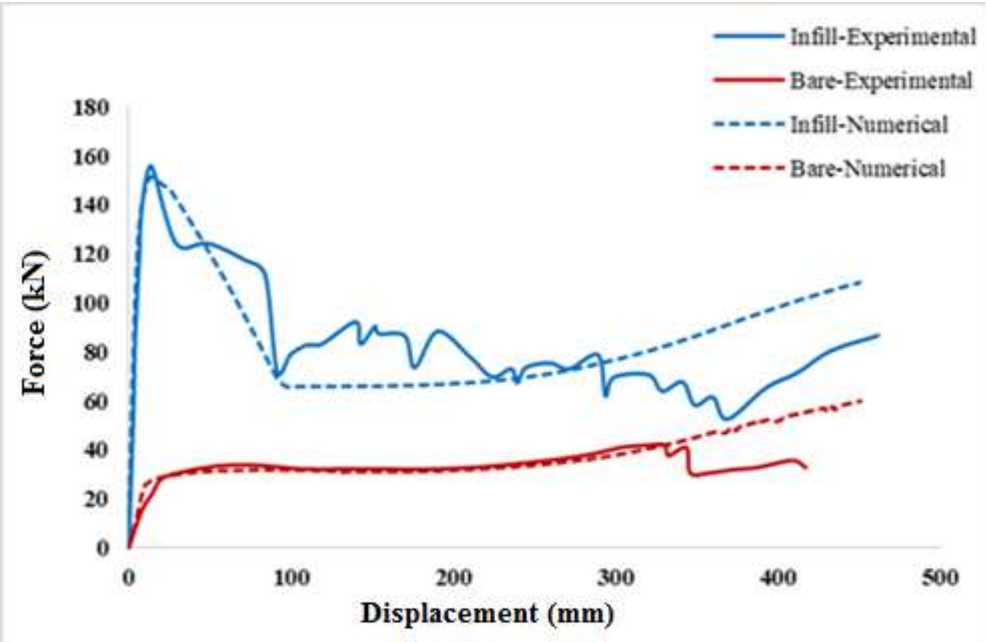

Figure 4. Comparing the capacity curves of models for validation

4. Progressive collapse analysis

For non-linear analysis of frames, the nonlinear static method (Push-down) proposed by DoD and GSA regulations has been used. In this method, first dead and live loads of the design was entered the structure. For this purpose, the displacement of the applied location and the response of the structures to the node above the removed column (ground floor middle column) after vertical displacement are examined. The number of scenarios is presented in Table 5.

Table 5. The scenarios under study

Position of removed column	Studied modes	Models
Middle ground floor	Steel moment frame (2-10 stories), without infill	9
Middle ground floor	Steel moment frame (2-10 stories), with infill	90

4.1. Capacity curve of frames

According to the results obtained from nonlinear static analysis of the studied structures, the capacity curve of each structure has been determined. The capacity curves represent the force-displacement diagram for different column loss scenarios. The vertical axis of the capacity curve represents action and the horizontal axis represents deformation. It should be noted that due to the increase in samples, all the graphs obtained from the study are presented in Figure (5). The maximum strength according to Tables (6 and 7) is presented to better compare the studied structures.

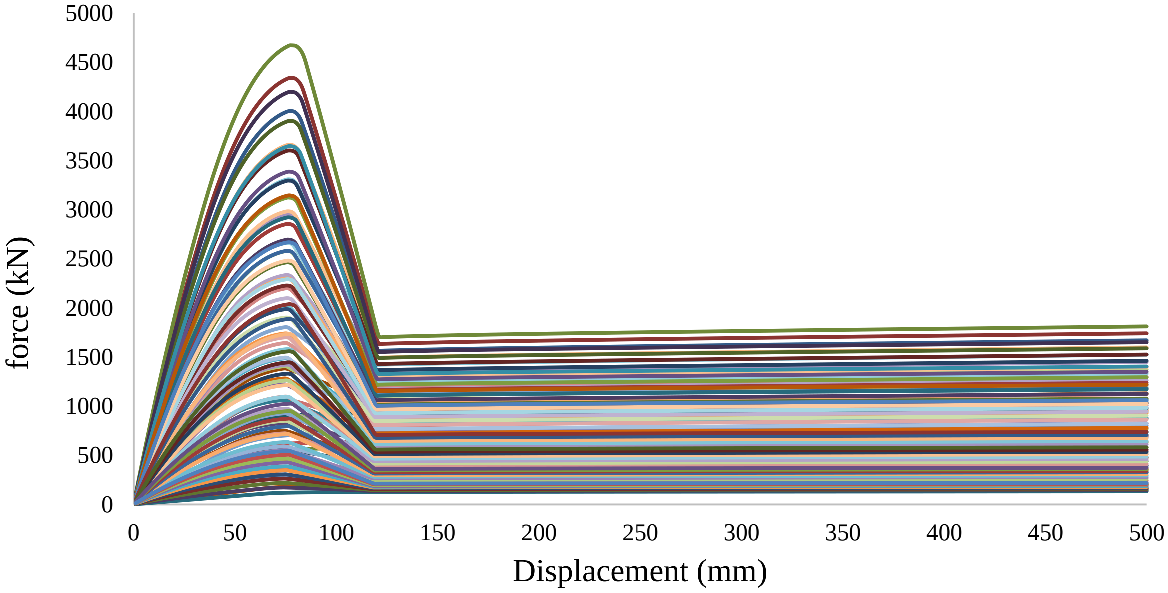

Figure 5. Comparing the capacity curve of all studied frames with the variable values of the infill characteristic strength

Table 6. The maximum strength of frames with variable characteristic strength

Maximum frames' resisting force (kN)												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of	**2**	133	171	218	262	306	347	387	427	466	505	544
	3	202	285	379	467	552	634	716	795	873	950	1025

	4	326	449	590	722	851	974	1095	1214	1331	1445	1558
	5	395	563	750	927	1097	1261	1422	1581	1735	1888	2038
	6	634	827	1061	1281	1494	1701	1901	2098	2292	2481	2667
	7	703	940	1221	1486	1740	1987	2227	2464	2695	2921	3145
	8	800	1079	1406	1715	2012	2299	2580	2855	3132	3387	3647
	9	958	1269	1643	1995	2334	2662	2983	3296	3602	3904	4200
	10	1031	1383	1803	2199	2580	2948	3308	3659	4003	4341	4672

Table 7. Comparing the maximum strength of frames to the frames without infills

Comparing the maximum strength of frames to frames without infills												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of floors	**2**	1	1.29	1.64	1.97	2.3	2.61	2.91	3.21	3.5	3.8	4.09
	3	1	1.41	1.88	2.31	2.73	3.14	3.54	3.94	4.32	4.7	5.07
	4	1	1.38	1.81	2.21	2.61	2.99	3.36	3.72	4.08	4.43	4.78
	5	1	1.43	1.9	2.35	2.78	3.19	3.6	4	4.39	4.78	5.16
	6	1	1.3	1.67	2.2	2.36	2.68	3	3.31	3.62	3.91	4.21
	7	1	1.34	1.74	2.11	2.48	2.83	3.17	3.5	3.83	4.16	4.47
	8	1	1.35	1.76	2.14	2.52	2.87	3.23	3.57	3.92	4.23	4.56
	9	1	1.32	1.72	2.08	2.44	2.78	3.11	3.44	3.76	4.08	4.38
	10	1	1.34	1.75	2.13	2.5	2.86	3.21	3.55	3.88	4.21	4.53

Tables (6 and 7) represent the maximum resistive force created in the frames in terms of (kN) and comparing the maximum resistive force of the frames with the frames without infills, respectively. According to Tables (6 and 7), increasing the floors and the presence of infills with high values of compressive strength causes a significant increase of the resistive force in the frames.

4.2. Diagram of relative displacement of frame floors

Controlling the relative displacement of floors has always been considered as a way to evaluate the performance of structures. To control the deformation created in the structure, the concept of relative lateral displacement of the floor is used. Using the concept of this parameter, the relative lateral displacement of the floor in the structures is calculated and compared with the limit values. These values are intended only for qualitative evaluation of the approximate behavior of structures at the functional level. Table 6 specifies the limitations for controlling the relative lateral displacement of different types of structures. In this study, the level of lateral safety performance has been selected and compared for the structure.

Table 8. Permissible values of relative displacement

Structural/performance system	Continuous use	Safety	Collapse threshold
Concrete moment frame	0.01	0.02-0.01	0.04
Steel moment frame	0.007	0.025-0.01	0.05
Braced steel frame	0.005	0.015-0.005	0.02

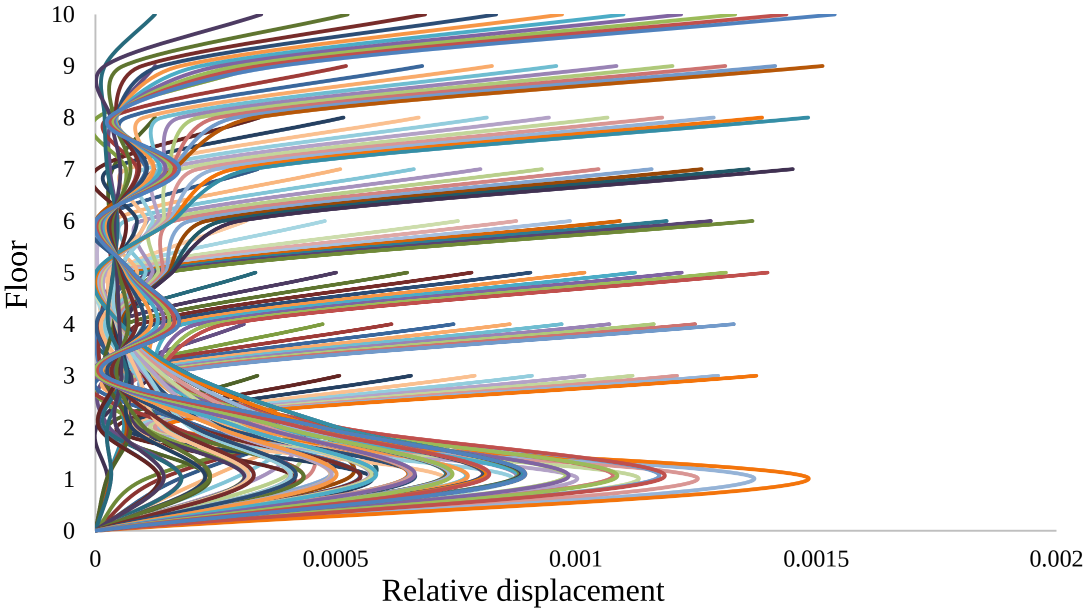

Figure 6. Comparing the relative displacement curves of all studied frames with variable values of infill characteristic strength

Table 9. Comparing the relative displacement of a 10-story frame to a non-infill frame

Comparing relative displacement of 10-story frame to the no-infill frame												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of floors	**1**	1	4.4	7.35	10.13	12.8	15.38	17.9	20.35	22.75	25.9	27.38
	2	1	1.79	4.15	6.34	8.41	10.38	12.27	14.7	15.81	17.49	19.11
	3	1	0.83	0.68	0.54	0.41	0.28	0.16	0.04	0.08	0.19	0.3
	4	1	1.87	2.58	3.22	3.8	4.34	4.83	5.3	5.73	6.14	6.52
	5	1	1.21	1.37	1.5	1.62	1.73	1.82	1.91	1.98	2.05	2.11
	6	1	0.88	0.78	0.67	0.57	0.47	0.38	0.27	0.19	0.11	0.02

	7	1	2.9	2.95	3.68	4.33	4.91	5.43	5.9	6.32	6.71	7.05
	8	1	1.63	1.99	2.2	2.3	2.3	2.24	2.11	1.93	1.71	1.46
	9	1	0.99	2.94	4.94	6.98	9.05	11.15	13.25	15.35	17.45	19.54
	10	1	2.78	4.23	5.53	6.72	7.82	8.85	9.82	10.73	11.59	12.4

Table 10. Comparing the relative displacement of a 9-story frame to a non-infill frame

Comparing relative displacement of 9-story frame to the no-infill frame												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of floors	**1**	1	4.4	7.35	10.13	12.78	15.36	17.87	20.32	22.71	22.06	27.35
	2	1	1.78	4.14	6.32	8.37	10.33	12.21	14.02	15.75	17.43	19.05
	3	1	0.83	0.68	0.54	0.41	0.28	0.15	0.03	0.08	0.2	0.31
	4	1	1.87	2.57	3.2	3.78	4.31	4.81	5.27	5.71	6.11	6.5
	5	1	1.2	1.35	1.49	1.61	1.71	1.81	1.89	1.97	2.04	2.1
	6	1	0.86	0.75	0.63	0.53	0.42	0.33	0.24	0.15	0.07	0.1
	7	1	0.7	2.84	3.44	3.93	4.32	4.64	4.89	5.08	5.22	5.33
	8	1	0.09	0.83	1.8	2.81	3.85	4.92	5.99	7.08	8.18	9.27
	9	1	2.76	4.18	5.45	6.61	7.68	8.69	9.62	10.5	11.34	12.12

Table 11. Comparing the relative displacement of a 8-story frame to a non-infill frame

Comparing relative displacement of 9-story frame to the no-infill frame												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of floors	**1**	1	5.62	9.61	13.39	17.03	20.55	23.99	27.35	30.63	33.85	37
	2	1	0.1	0.03	1.89	2.71	3.48	4.22	4.93	5.61	6.28	6.9
	3	1	2.66	4.02	5.23	6.35	7.39	8.35	9.26	10.11	10.92	11.7
	4	1	1.06	1.11	1.16	1.2	1.24	1.28	1.31	1.34	1.37	11.4
	5	1	0.87	0.75	0.64	1	0.44	0.34	0.25	0.17	0.09	0.02
	6	1	2.08	2.85	3.45	1.94	4.32	4.68	4.88	5. 07	5.21	5.31
	7	1	0.08	0.85	1.83	2.68	3.91	4.96	60.1	7.07	8.13	9.19
	8	1	2.76	4.17	5.43	6.58	7.63	7.61	9.53	10.4	11.21	12

Table 12. Comparing the relative displacement of a 7-story frame to a non-infill frame

Comparing relative displacement of 7-story frame to the no-infill frame											
Resistance of the	**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

building charter (Mp)												
Number of floors	**1**	1	5.63	9.63	13.41	17.05	20.58	24.01	27.37	30.65	33.78	37.03
	2	1	0.15	1.12	2.02	2.86	3.67	4.44	5.19	5.9	6.59	7.26
	3	1	2.95	4.56	6.3	7.38	8.65	9.84	10.96	12.03	13.04	14
	4	1	0.24	1.29	2.26	3.17	4.02	4.82	5.58	6.29	6.97	7.61
	5	1	1.95	2.63	3.17	3.6	3.95	4.23	4.45	4.62	4.75	4.84
	6	1	0.01	1.95	1.93	2.94	3.96	4.99	6.03	7.07	8.11	9.14
	7	1	2.75	4.15	5.40	6.53	7.57	8.53	9.43	10.28	11.07	11.82

Table 13. Comparing the relative displacement of a 6-story frame to a non-infill frame

Comparing relative displacement of 6-story frame to the no-infill frame												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of floors	**1**	1	5.63	9.62	13.38	17	20.51	23.92	27.25	30.51	33.71	36.85
	2	1	0.15	1.11	2	2.85	3.365	4.42	5.15	5.86	6.55	7.21
	3	1	3.21	5.01	6.64	8.14	9.53	10.82	12.04	13.18	14.25	15.27
	4	1	4.5	8.42	11.53	14.02	16	17.56	18.76	19.64	20.26	20.63
	5	1	0.76	0.46	0.11	0.27	0.7	1.11	1.56	2.02	2.52.98	4.84
	6	1	2.93	4.46	5.05	7.05	8.18	9.22	10.2	11.11	11.96	12.75

Table 14. Comparing the relative displacement of a 5-story frame to a non-infill frame

Comparing relative displacement of 5-story frame to the no-infill frame												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of floors	**1**	1	6.88	11.9	16.62	21.12	25.4	29.6	33.7	37.6	41.5	45.3
	2	1	0.28	1.35	2.33	3.24	4.1	4.9	5.68	6.4	7.13	7.8
	3	1	1.93	2.6	3.14	3.6	3.95	4.24	4.5	4.7	4.8	4.95
	4	1	0.05	0.82	1.68	2.5	3.37	4.2	5.04	5.9	6.7	7.5
	5	1	2.74	4.12	5.33	6.4	7.44	8.4	9.23	10	10.8	11.5

Table 15. Comparing the relative displacement of a 4-story frame to a non-infill frame

Comparing relative displacement of 4-story frame to the no-infill frame											
Resistance of the	**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**

building charter (Mp)												
Number of floors	**1**	1	7.2	12.5	17.4	22	26.7	31	35.2	39.3	43.4	47
	2	1	0.5	1.7	2.8	3.8	4.7	5.53	6.3	7.1	7.8	8.5
	3	1	1.93	2.6	3.14	3.6	3.95	4.24	4.5	4.7	4.8	4.95
	4	1	0.05	0.82	1.68	2.5	3.37	4.2	5.04	5.9	6.7	7.5

Table 16. Comparing the relative displacement of a 3-story frame to a non-infill frame

Comparing relative displacement of 3-story frame to the no-infill frame												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of floors	**1**	1	3	16.6	23	29.1	33.75	40	45	50	54.87	59.4
	2	1	10.6	19.5	27	33.8	39.8	45	50	54.8	59	63
	3	1	2.8	4.27	5.5	6.6	7.64	8.6	9.4	10	10.9	11.6

Table 17. Comparing the relative displacement of a 2-story frame to a non-infill frame

Comparing relative displacement of 2-story frame to the no-infill frame												
Resistance of the building charter (Mp)		**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Number of floors	**1**	1	5.5	5.05	4.6	4.18	3.71	3.24	2.75	2.22	1.7	1.3
	2	1	0.92	0.9	0.92	0.93	0.95	0.97	0.98	0.99	0.99	1

According to Figure 6 and the tables presented to compare the relative displacement of the floors, it is observed that the relative displacement of the frame floors against progressive collapse increases as the compressive strength of the infill and the width of the equivalent compression handle increase. In this regard, the higher the compressive strength of the interlayer, the more the equivalent compressive strength increases, leading to the displacement of the connection node under the phenomenon of progressive collapse.

4.3. Proposed equation of resistive force

The results of this study indicate that the scenarios in which the effect of masonry infills are considered can provide a relatively more accurate results of the values of the strength of the frames and relative displacement. In this research, some coefficients are proposed that make possible to achieve acceptable results for frames with infills with different compressive strength without using infills with different compressive strength. In other words, the purpose of proposing these coefficients is to simplify the progressive collapse analysis for frames with masonry infills with different values of compressive strength. The proposed relationship is obtained by comparing the maximum strength of the frames under progressive collapse based on the ordinary least squares (LOS) method. Among the various linear methods for estimating parameters, the ordinary least squares (LOS) method is known as a well-used method. It tries to get the best regression line for the data by minimizing the sum of squares of perturbation sentences.

$$y_i = \widehat{\beta_1} + \widehat{\beta_2}\, x_i + e_i \qquad (6)$$

$$y_i = \widehat{y_i} + e_i \qquad (7)$$

where y_i depends on both e_i and x_i. In addition, $\widehat{y_i}$ is the estimated value (conditional average) of y_i.

$$e_i = y_i - \widehat{y_i} \qquad (8)$$

$$e_i = y_i - \widehat{\beta_1} - \widehat{\beta_2} x_i \qquad (9)$$

ei (remnants) are simply the differences between real and estimated y values. Based on the observations of x and y, we try to determine the sample regression function (SRF) so that it is as close as possible to the real y. The sample regression function should be selected such that the remainder $\sum e_i = \sum(y_i - \widehat{y_i})$ is small as much as possible. The sample regression function (SRF) can be determined by the following equation:

$$min \sum {e_i}^2 = \sum(y_i - \widehat{y_i})^2 = \sum\left(y_i - \widehat{\beta_1} - \widehat{\beta_2}\, {x_i}^2\right) \qquad (10)$$

$$F_n = F_0(1 + 0.33n) \qquad (11)$$

In this regard, F_n is the resistive force of the frame with infills with compressive strength n MPa (Mp), F_0 is the resistive force of the frame without infill (N), n is the compressive strength of the infill (Mp). In order for the proposed coefficient to have appropriate accuracy, frames of 2 to 10 layers were examined in the cases with and without infills with variable compressive strength (1 to 10 MPa). Examining the maximum strength obtained from software analysis and the values obtained from the proposed equation (11) show that the proposed equation in all cases under study has a maximum error of about 5%, indicating the appropriate accuracy of the proposed equation.

5. Conclusion

This study investigates the effect of infill compressive strength on the progressive collapse of steel frames. Most structural designers do not take into account the stiffness of the infills when modeling and simply apply the resulting load on the beams. However, in the absence of modeling of the interfaces and the compressive strength in them, the results obtained from the progressive collapse analysis are not accurate enough. In this study, the studied frames have a number of floors from 2 to 10 floors. The effect of compressive strength of infills on the behavior of steel frames under progressive collapse has been investigated. For each studied floor, 11 frame models were analyzed, one of which is related to the state without infills and the other 10 models are related to the states with infills with compressive strength ranging from 1 to 10 MPa, based on which the number of studied models is 99. OpenSees finite element software has been used to accurately investigate the progressive collapse of frames. The main results of this research are as follows:

• Increasing the compressive strength of the infill increases the amount of resistive force in the frames. Accordingly, increasing each 1 MPa of the infill compressive strength increases the frame strength by an average of 33%.

• According to the capacity curve of the frames, the increase of each floor causes an average increase of 83% of the strength of the frames.

• According to the comparison of the maximum capacity obtained from the analysis of frames and the proposed relationship, it has an average of 5% error in all floors, this percentage of error indicates the appropriate accuracy of the proposed relationship.

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