Evaluation of scale effect on finite element analysis of progressive collapse scenario

Nima Usefi
M.Sc. graduate, Civil Engineering Department, Iran University of science and technology
Email: n_yusefi@alumni.iust.ac.ir

Amin Nooralizadeh
PhD student, University of Qom
Email: aminnooralizadeh@stu.nit.ac.ir

Mohammad Kazem Bahrani
Assistant Professor, Civil Engineering Department, University of Qom,
Email: m.bahrani@iiees.ac.ir

Abstract

Progressive collapse is a complex process and the quantitative understanding and analysis theory about the phenomena are still not mature. Several buildings have collapsed in this fashion in recent years, and the possibility of progressive collapse is a source of continuing concern. Recently, finite element simulations have been used as an alternative for expensive and time-consuming experimental tests. As most of these research were done in scaled mood, in this study the effect of scale on the results of analysis is considered using ABAQUS software. First, a finite element method is proposed and validated using eight half scale specimens. Then the same sub-assemblages are modeled and analyzed in full scale and the results are compared with scaled specimens. Results showed that the full scale specimens had better strength than scaled specimens. In addition, final displacement in catenary action doubled for full scale specimens compared to the scaled types. The graphical results also showed that the number of bar fraction in full scale specimens was more than scaled specimens.

Keywords: Progressive collapse, Scale, Concrete, Finite element, ABAQUS

Introduction:

A building undergoes progressive collapse when a primary structural element fails, resulting in the failure of adjoining structural elements, which in turn causes further structural failure. The general services administration of the United States defines the progressive collapse as a total damage which is disproportionate to the original cause (GSA, 2003). Recently, many research have been done in this field of study in order to find the strength capacity of frame after removal of a column. Some researchers such as Mehrdad et al. (2007); Yi et al. (2008) and Sadek et al. (2011), have examined structural progressive collapse by experiments. Some researchers such as Marjanishvili and Agnew (2006), Alashker et al. (2011) and Sadek et al. (2011) have investigated structural performance using nonlinear static or dynamic procedures. Some researchers used simplified models to study the system behavior of moment frames (Bao et al., 2008) (Bao and Kunnath, 2010). Sasani et al. (2011) used detailed models to model bar fracture of reinforced concrete frame structures. Luccioni et al. (2004) used models to analyze the structural failure of an actual reinforced concrete building caused by a blast load. Talaat and Mosalam (2007) developed a modeling approach to simulate structural collapse of reinforced concrete frame structures under earthquake events. New research have been done in this field by some researchers both in experimental test and numerical modeling (Abbasnia et al, 2016) (Ahmadi et al. 2016) (Mohajeri et al. 2016).
Due to the complexities of experimental tests and large-displacement behavior that reinforced concrete frames exhibit under collapse scenarios, finite element approaches have been extensively used by researcher in these days. In addition, it can be seen that the effect of scale on the analysis of structures under progressive collapse, which is an important factor, is not considered in previous FE modeling. Hence, in this paper, the effect of scale on the results of RC beam-column substructures is the main object, and the mechanism of progressive collapse resistance to applied load is analyzed in different stages of deflections. First the finite element approach is validated using 8 half scale sub-assemblages which were previously tested. Then by the validated FE model, the same sub-assemblages are modeled in full scale and the effects of scale is considered in these specimens. The progressive collapse-resisting capacity curves of 16 specimens is presented and each specimen is compared in two different scale. The objective of this study is to investigate the differences in results of scaled and full scale sub-assemblages under progressive collapse.

Description of Specimens and material properties:

Table 1 shows the details of eight half- scale sub-assemblages which were previously tested in laboratory (Yu and Tang, 2011, 2013). These specimens are also used for validation of proposed FE method. Table 2 shows the geometry and details of modeling for full scale specimens. M1 to M8 are the full scale type of S1 to S8 specimens. To evaluate the effect of scale, in this paper 16 sub- assemblages are compare with each other. The mechanical responses of RC sub-assemblages under progressive collapse depend strongly on the material properties of both concrete and reinforcing steel.
The properties of concrete and steel materials are presented in Table 3. As the scaled and full scale specimens have the same properties, the details are similar in the table.

Table (1): Specifications of half-scale sub-assemblages

Research	Test Specimen	Beam Span (m)	Sections at joint interface and beam end: b×h (cm)	Reinforcement: Top	Reinforcement: Bottom
Yu and Tang (2011,2013)	S1	2.750	15×25	1T13+2T10	2T10
	S2	2.750	15×25	3T10	2T10
	S3	2.750	15×25	3T13	2T10
	S4	2.750	15×25	3T13	2T13
	S5	2.750	15×25	3T13	3T13
	S6	2.750	15×25	3T16	2T13
	S7	2.150	15×25	3T13	2T13
	S8	1.550	15×25	3T13	2T13

Table (2): Specifications of full scale sub-assemblages

Model	Scaled from	Beam Span (m): Left	Beam Span (m): Right	Sections at joint interface and beam end: b×h (cm)	Reinforcement: Top	Reinforcement: Bottom
M1	S1	5.5	5.5	30×50	1T25+2T20	2T20
M2	S1	5.5	5.5		3T20	2T20
M3	S3	5.5	5.5		3T25	2T20
M4	S4	5.5	5.5		3T25	2T25
M5	S5	5.5	5.5		3T25	3T25
M6	S6	5.5	5.5		3T32	2T25
M7	S7	4.3	4.3		3T25	2T25
M8	S8	3.1	3.1		3T25	2T25

Table 3. Details of Material

Steel Property					
Specimen	Bar type	Diameter (mm)	Yield strength (MPA)	Ultimate strength (MPA)	Ultimate strain (%)
S1 to S8	Φ6	6	349	459	-
	Φ10	10	511	622	11
	Φ13	13	494	593	10.92
	Φ16	16	513	612	13.43

Concrete Property		
Specimen	S1 and S2	S3, S4,S5,S6,S7,S8
Strength (MPA)	31.2	38.2

Finite Element Modeling:

For finite element modeling of the specimens, concrete modeling, Steel modeling, steel-concrete interaction and mesh details should be under consideration. Hence a brief introduction to concrete and steel reinforcement is being presented below.

An elastic-plastic material was used for the steel bar with an equal behavior in tension and compression. The steel bar is treated as a uniaxial material throughout the element section. The steel bars used in the reinforced concrete beam were assumed to have the yielding stress. The steel reinforcement was assigned with a Poisson’s ratio of 0.3. Unlike concrete, steel is a homogenous material that is taken to behave the same in tension as in compression. The Von Mises yield criterion is used in order to define the plastic region (ABAQUS documentation, 2013).

The simplified concrete damage plasticity model was used for simulating the concrete in this analysis. Tensile stresses are very small and as a result, compressive strength, *fc*, it is the main criteria of

determining the quality of concrete. This model assumes that the two main failure mechanisms in concrete are the tensile cracking and the compressive crushing. The uniaxial tensile and compressive behavior is characterized by damaged plasticity. In order to simulate the complete tensile behavior of reinforced concrete in *ABAQUS*, a post failure stress-strain relationship for concrete subjected to tension is used which accounts for tension stiffening, strain-softening, and reinforcement interaction with concrete (ABAQUS documentation, 2013).
Full bond contact between the steel reinforcement and concrete was presumed. The embedded element option was used for connecting the reinforcement element to the concrete element, steel reinforcement was used as the embedded element (ABAQUS documentation, 2013).
A 3D element with eight nodes is used for modeling the concrete (*C3D8R*). A truss element, a linear element with two nodes, is used for bar modeling (T3D2). Since axial force has the basic role in steel bars, this linear model is used instead of a multi-nodal element. This would reduce the time and also the amount of computational effort (ABAQUS documentation, 2013).
Mesh size of the solid elements ranged from 35 mm to 80 mm. The maximum aspect ratio of the solid elements was 2.8 for all sub-assemblages. The mesh size of truss elements representing beam-longitudinal bars was 100 mm, while for other truss elements representing other reinforcing bars it ranged from 80 mm to 125 mm. Refined mesh sizes (35 mm) were utilized in critical regions–within the joint and along the beam for a distance of one beam depth from the face of the beam-column joints. Coarser meshes were used in all other regions.

Results and discussion:

First the results of 8 scaled sub-assemblages are presented for validation of modeling. Then in the second stage, the results of the same sub-assemblages in full scale is given and compared with scaled type.

A) Results of half scale specimens (validation)

Figure. 1 compares the results of finite element analysis with experimental data. As the graphs show, finite element procedure could predict the general behavior of sub-assemblages with acceptable accuracy. In order to reach to a precise comparison, important points of the curves are compared in Table 4. The differences between predicted values in comparison to experimental results are reported as errors in Table (4). Mean error for predicting compressive arch action capacity (P_{CAA}) and catenary capacity (P_{CA}) through FE analysis are about 7.65% and 8.6%, respectively. Generally, except two specimens, the prediction of finite element approach is larger than experimental results. As Table 4 and Figure. 1 show, finite element procedure could predict general behavior of sub-assemblages in a collapse scenario.

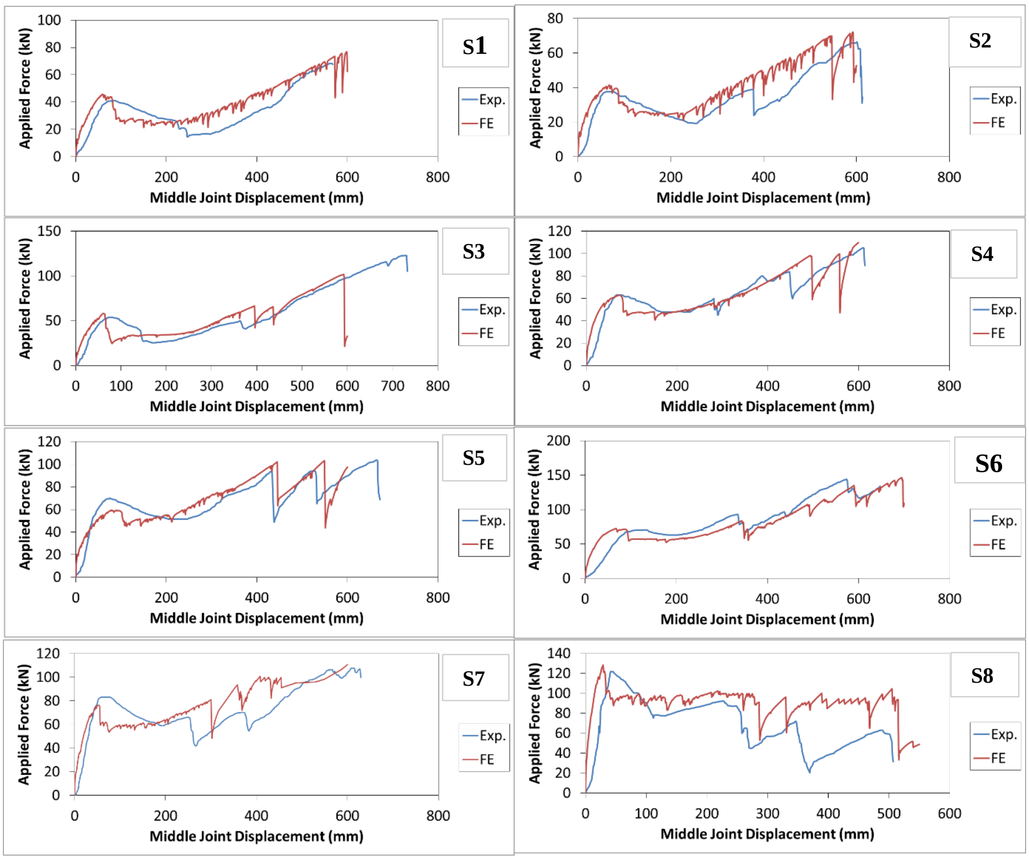

Figure 1: Finite Element (FE) in comparison to Experimental (Exp.) for half scale specimens

Table (4): Obtained results in comparison to Experimental results

Model	Result	Compressive Arch Action		Transient point		Catenary Action	
		Displacement (mm)	*force (kN)*	*displacement (mm)*	*force (kN)*	*displacement(mm)*	*force (kN)*
S1	Experimental	78.0	41.64	246.1	14.87	573.0	68.91
	Finite Element	59.3	45.78	215.4	21.74	599.0	76.83
	FE Error (%)	24.0	9.9	12.5	46.2	4.5	11.5
S2	Experimental	73.0	38.38	252.3	19.33	612.0	67.63
	Finite Element	68.9	41.43	227.7	21.03	592.8	72.09
	FE Error (%)	5.6	7.9	9.8	8.8	3.1	6.6
S3	Experimental	74.4	54.47	189.2	24.31	729.3	124.37
	Finite Element	61.9	58.04	80.9	37.47	592.9	119.57
S4	Experimental	81.0	63.22	167.1	47.78	614.3	103.68
	Finite Element	74.8	62.85	154.5	43.14	601.3	109.87
	FE Error (%)	7.7	0.6	7.5	9.7	2.1	6.0
S5	Experimental	74.5	70.33	205.4	51.60	665.9	105.07
	Finite Element	78.1	55.37	144.7	41.31	549.6	132.94
	FE Error (%)	4.8	21.3	29.6	19.9	17.5	26.5
S6	Experimental	114.5	70.33	181.2	62.96	573.0	143.28
	Finite Element	87.7	71.44	178.9	52.34	693.7	146.22
	FE Error (%)	23.4	1.6	1.3	16.9	21.1	2.1
S7	Experimental	74.4	82.82	267.1	41.90	555.3	105.99
	Finite Element	53.8	76.44	79.9	55.24	604.1	112.02
	FE Error (%)	27.7	7.7	70.1	31.8	8.8	5.7
S8	Experimental	45.9	121.34	111.8	75.24	224.7	91.83
	Finite Element	29.9	128.24	135.1	83.07	464.1	104.52
	FE Error (%)	34.9	5.7	20.8	10.4	106.5	13.8
FE Error (%)	Mean	18.8	7.65	24.5	23.8	18.5	8.6

Since experimental results of S8 sub-assemblage show that shear behavior was dominant, shear failure of beams occurs and hence, catenary action was not developed within the specimen. This happened because of the short spans of beams which limits the possibility of large deformations in beams and middle joint. This limitation causes sudden failure of beams which leads to shear collapse of sub-assemblage. But in real structures due to the larger spans, this happens rarely. So results of S8 are less important as a basis for real structures. Fig. 2 also shows different actions in concrete and steel bars during the progressive collapse analysis of S6.

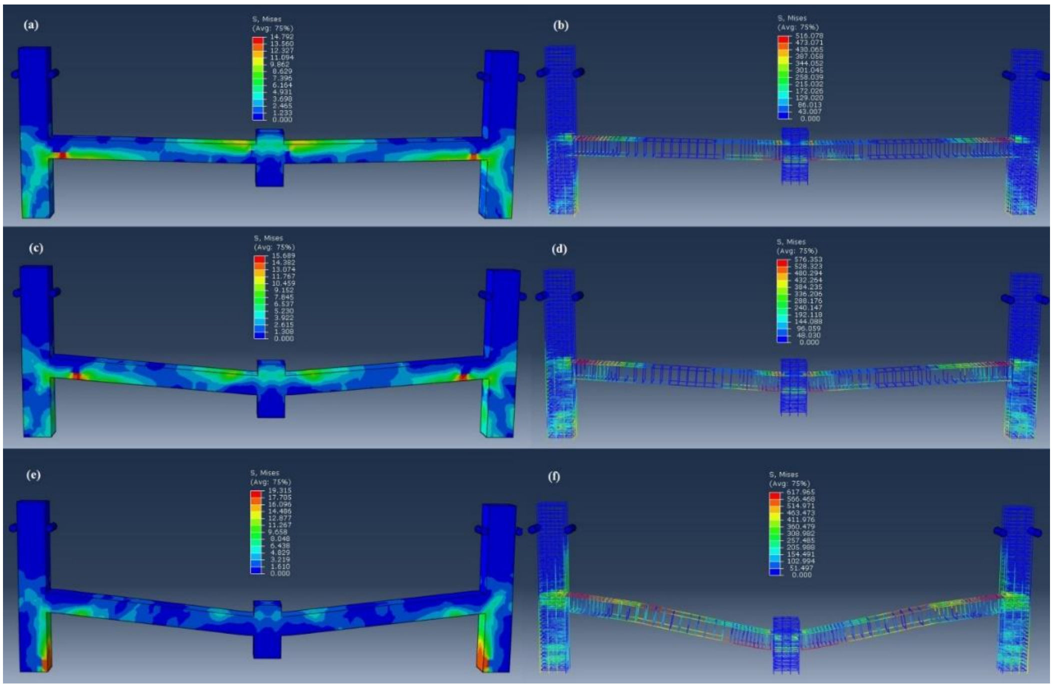

Figure 2: Finite Element Simulation of S6; flexural action in (a) concrete, and (b) steel bars; compressive arch action in (c) concrete, and (d) steel bars; catenary action in (e) concrete, and (f) steel bars.

Results of full scale specimens

Fig. 3 compares the results of scaled and full scale specimens. It is clear that the final displacement for full scale specimens is as twice as the displacement for scaled specimens. Hence, this is one important result from this study, that the effect of scale on the modeling of structures can have important impression on the forces and displacements. In addition, however the scale of the specimens has been doubled, the strength of the sub-assemblages has been trifold. In this regard, force quantity has a straight relationship with the quantity of scale. In specimens S1 and S2, in scaled specimens, the compression arc action capacity is around 50 Kn. However, the figure for full scale specimens is more than 150 Kn. Another important result from comparing the full scale and scaled specimen can be gained which is the number of rebar fraction in each specimen. For full scale specimen the number of rebar fraction is more than scaled specimens. Figure (4) shows the bar fraction in specimen S5 in both Full scale and scaled types. As it is clear, the number of bar fraction in middle connection for the full scale specimen is more than the scaled one. For the specimen S8, because of the short beam span, the failure of the beam was a shear failure. Although for the full scale type of S8 the failure is between shear failure and flexural failure. As the beam span for the full scale specimen is doubled the type of failure is changed.

Structural detailing and geometric specifications of S5 and S6 are the same except for the top reinforcement along the beam. Although S6 has larger reinforcement and naturally it should have a greater compressive arch strength, but same resistances are observed for two specimens in the experiment and only the vertical displacement of the middle joint is different.

Fig. 3 also shows the results of S8 which due to the smaller spans, shear behavior is dominant and catenary action could not develop to a larger capacity in comparison to compressive arch action.

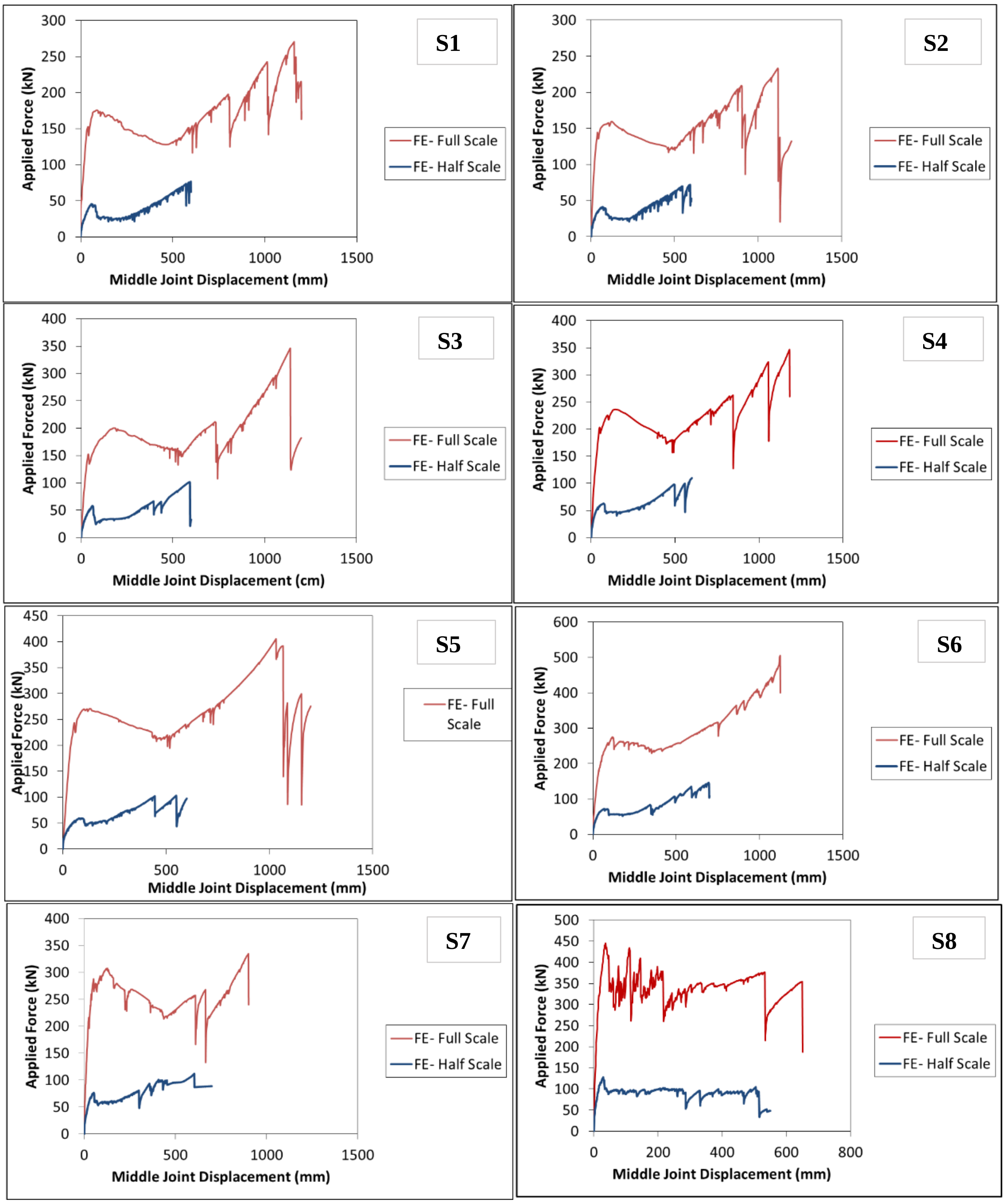

Figure 3. Comparison of FE results for half scale and full scale specimens

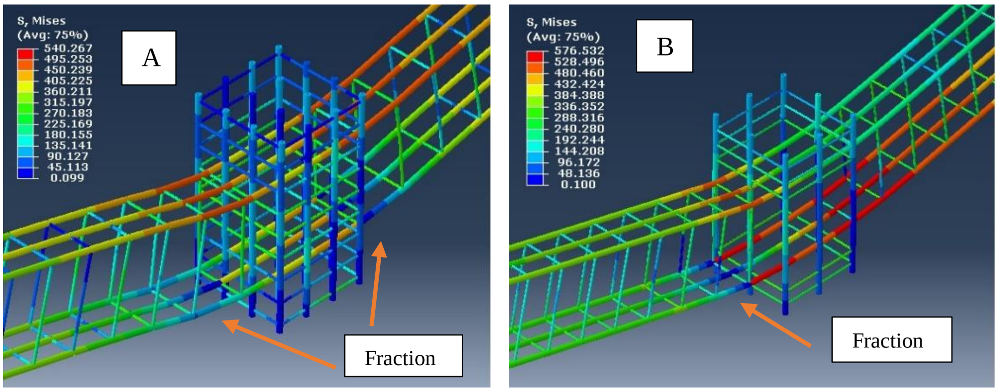

Figure 4. Bar fraction for specimen S5 a) Full Scale b) Half Scale

Although the FE model is simple and also effective, it needs to perform a more comprehensive evaluation using new experimental data. In the FE model, the interaction between concrete and steel is complete and the effect of slippage is ignored in analysis which could influence the resistance of structures. Furthermore, generally progressive collapse is a dynamic event which is investigated statically in the present study. Hence dynamic effects needs to be considered for further studies.

Conclusion

RC sub-assemblage is an effective tool in order to understand progressive collapse behavior of reinforced concrete structures. Hence, in the present study a comprehensive finite element investigation of these sub-assemblages is performed in order to evaluate the effect of scale on the collapse resistance of these structures. Finite element approach is verified based on experimental database in the literature. Explained finite element framework showed good accuracy in prediction the collapse resistances of sub-assemblages. Mean error for predicting compressive arch action capacity (P_{CAA}) and catenary capacity (P_{CA}) are about 7.65% and 8.6%, respectively. Mostly the FE predictions are larger than experimental strengths.

General behavior of the scaled and full scale specimen was similar and in each specimen Arc action and catenary action were occurred. But by increasing the scale of the modeling the strength and the final displacement changed. By doubling the scale, the straight capacity of the sub-assemblages trifled. In addition the final displacement in which the ultimate capacity of the specimen occurred, changed to a better condition. For the scaled specimens the displacement capacity for catenary action was around 600 mm; however the number for the full scale specimens was approximately 1200 mm.

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