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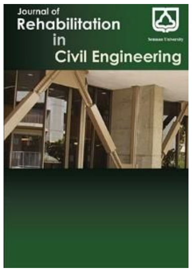

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Investigation of Progressive Collapse in Reinforced Concrete Buildings with Slab-Wall Structural System

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ABSTRACT

Nowadays, Reinforced Concrete (RC) wall-slab systems are being used more extensively due to their effective performance seen in past earthquakes. Progressive collapse is a phenomenon in which all or part of a structure is damaged due to damage or collapse of a small relevant part. The majority of research done in the field of progressive collapse has been on frame-shaped structures. Further, the performance of RC wall-slab structural systems, especially against progressive collapse, has been less studied. In this study, at first, nine concrete buildings of five, ten and fifteen stories with wall-slab structural systems, with the ratio of spans length to the story height (L/H) of 1, 1.5 and 2 and a structural height of 2.75 meters in each story, were designed by the ETABS V16 software. Then, using the SAP2000 software and nonlinear shell-layered elements, nonlinear static analysis was performed by the Alternative Load Path (ALP) method on the models and the results were evaluated. The results demonstrated the relatively high strength of buildings with wall-slab structural systems in withstanding progressive collapse. The rate of vertical displacement of the removal location, the maximum von Mises stress in rebar, the maximum compressive stress and strain in concrete in the interior wall removal scenarios were less extensively compared to the corner wall removal scenarios. In contrast, progressive collapse potential increased significantly with increasing number of stories and the L/H ratio. Also, it was found that, buildings with the wall-slab structural system may exhibit brittle failure behavior influenced by progressive collapse.

1. Introduction

Nowadays, the use of wall-slab structural systems has become more extensively widespread thanks to their effective performance seen in past earthquakes. In such systems, concrete load-bearing walls withstand both gravity and lateral loads.

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Gravity loads are applied to the walls by the slabs, and the walls withstand the forces emanating from these loads in the same way as a narrow and wide column, and transfer the forces to the foundation. Due to the shortage of data on the performance of these buildings, evaluation of their linear and nonlinear behavior is of special significance. In this research having in mind the lack of special standards and established procedures for these buildings, in the initial analyses and designs performed, some parameters of analysis and design of RC shear wall and slab were used.

Additionally, one of the mechanisms that has received increasing attention in recent decades is progressive collapse. It is a phenomenon in which all or part of a structure is damaged due to damage or collapse of a small pertinent part. Progressive collapse is mostly caused by unusual loads and design/construction errors and can lead to local collapse or local instability. Unusual loads that can cause progressive collapse include fires, gas explosions, accidental overloads, vehicle accidents, bomb blasts, etc.

To deal with progressive collapse, it is necessary to have a correct understanding of threats, to understand and recognize some critical factors including probable abnormal loads, load action mechanisms, structural component behaviors, and the structure behavior in response to abnormal loads. Therefore, it would be feasible to evaluate the potential of progressive collapse in structures in different ways, in order to benefit from this information.

The two regulations including the Unified Facilities Criteria (UFC) and General Services Administration (GSA), which are the most distinguished regulations developed in this regard and are widely used to deal with progressive collapse. One of the most important methodologies recommended by regulations to evaluate the progressive collapse in structures is the Alternative Load Path (ALP) method**,** in which a member is or some members of structure are removed, and the ability of other structural components and overall structural strength to withstand this partial failure is examined. There are different analytical methods for using ALP analysis to investigate progressive collapse. To evaluate the potential of buildings in response to progressive collapse, the ALP is the most accurate method available. In this method, the dynamic effects caused by the removal of the column or wall and nonlinear behaviors are accurately considered. Therefore, this analysis is more accurate than the Non-Linear Static (NLS) analysis. However, due to the complexity and time-consuming nature of this type of analysis, NLS analysis can be a good alternative. To consider the dynamic effects of removing a column (or wall), existing codes introduce a dynamic load increase factor. Using the Dynamic Increase Factor (DIF), gravity loads continue to increase in spans that are rapidly damaged as a result of the removal of a member. Therefore, it is necessary to determine the DIF for performing nonlinear static analysis.

Although plenty of experimental and analytical research has been performed on DIF and progressive collapse in structures, there are still many issues that need to be examined and identified more accurately. In addition, the majority of this type of research has been done on frame structures.

In 2014, Yüksel investigated the nonlinear seismic behavior of buildings with wall-slab structural systems. In their study, an experimental study was performed on a rectangular specimen with an equivalent scale under uniformly increasing lateral loads. The results showed that wall-slab structural systems under seismic load may show brittle collapse [5]. Furthermore, in 2015, Janni Praveenkumar et al. analyzed and compared the design details of a slab-wall system and a beam-column system in concrete buildings. Accordingly, a building with the typical floor plan in a structural system of high raised concrete buildings could be easily executed by slab-wall form compared to a column-beam system. In addition, the behavior of the building under gravity and lateral loads was analyzed. Comparisons and analysis results showed high base shear and deformation in the column-beam system more than that of slab-wall systems in concrete buildings [6].

Also, Mohsenian et al., in 2012, with the help of incremental dynamic and Pushover analysis, determined the performance level of tunnel-form structures and calculated the behavior coefficient according to the earthquake demand and acceptable performance levels of the mentioned structures [7]. In 2017, they investigated the multi-level determination for RC tunnel-form buildings. The results showed high capacity and resistance of the system and acceptable seismic performance of the structures under study [8]. In another study in 2017 they estimated the seismic response parameters and capacity of irregular tunnel-form buildings. The results, in general, demonstrated the flexible torsional behavior of irregular tunnel-form structures and their adequate seismic resistance capacity. Moreover, the buildings studied herein, managed to satisfy the immediate occupancy performance requirements under design-basis earthquakes, which implies that the plan regularity requirement for tunnel-form buildings in seismic codes may be too conservative [9]. In 2017 they assessed seismic performance-based of tunnel-form buildings subjected to near and far-fault ground motions. Results illustrated that with increasing the construction height and seismic intensity, the influence of directivity on the structural responses including story shear force and drift became more intensified [10].

In 2017, Masoumi et al. evaluated the effective parameters in the seismic behavior of a concrete tunnel formwork using nonlinear static analysis. They concluded that, the use of tunnel formwork structures was not suitable for short buildings. Also, increasing the number of parallel shear walls had no effect on the strength and shear coefficient of the structure [11]. Additionally, Hashemi and Khosravi (2015) assessed the progressive collapse of RC structures with a bubble deck floor system. The results showed that the middle column removal scenario was the most critical among other scenarios [12].

Shahin et al. in 2016 investigated progressive collapse caused by abrupt removal of the main components of a concrete load-bearing wall system. To this end, a ten-story building with reinforced concrete as a load-bearing wall system was modeled in three dimensions, taking into account non-linear geometric effects and materials. The effect of progressive collapse caused by the removal of the load-bearing elements was evaluated in different positions of stories and structural plans. The analysis results indicated stability, strength and stiffness of the RC load-bearing wall system against progressive collapse. It was observed that the most critical condition for removal of load bearing walls was the instantaneous removal of the surrounding walls located at the corners of the building where the sections of the load bearing elements were changed [13].

In a further study by Mashhadiali et al. (2016) they evaluated the DIF for investigation of progressive collapse potential in tall tube-type buildings using nonlinear static/dynamic analysis. The results of the proposed method were in good agreement with nonlinear dynamic analysis results [14].

In 2017, Grivani et al. examined the effect of the structural system on the possibility of progressive collapse due to the removal of columns in reinforced concrete buildings. Numerical models included 3, 7 and 10-story concrete frames in ordinary moment frames (OMF), intermediate moment frames (IMF) and special moment frames (SMF) along with an intermediate concrete shear wall. The results showed that IMFs were more cost-effective than other structural systems studied, and could better bear the loads resulting from the removal of the column [15].

In another publication by Mohammadi et al., (2017) the effect of infill panels on the progressive collapse of RC structures subjected to extensive initial damage, was examined. In this study, 3D panels were used as infill. The results showed that the possibility of collapse in models with infill panels was more than that of the models without infill panels. This was due to the diminished capacity of the beams and their shear collapse in the presence of infill panels. It was also observed that the behavior of beams was different in the presence of infill panels and collapse, and neglect of their effect could lead to incorrect prediction of structural behavior [16]. Also, Choobineh et al. (2017) investigated impact of infill panels on progressive collapse of steel structures subjected to extensive initial damage. The results showed that the higher the height of the structure in steel structures, the lower the damage to the structure in most removal scenarios. They mentioned that, the L/H ratio affected the ability of the wall to resist progressive collapse. They also found that with increasing opening percentage, the final displacement due to removal of the element and the potential for progressive collapse increased [17]. Also, Shayanfar et al. (2017) examined progressive collapse-resisting mechanisms and robustness of RC frame–shear wall structures [18], and concluded that shear walls showed considerably high resistance to progressive collapse, which can be implemented in collapse-resistant design. Moreover, detailed descriptions of modeling approaches and validations can provide guidelines for simulation of different RC structural elements under large deformations.

In a further study by Khodadadi et al. (2019) calculated the DIF to assess the progressive collapse of steel structures with steel shear walls. The authors' investigations showed that the DIF was lower than the value recommended by progressive collapse regulations such as the UFC and GSA [19].

In 2019, Rouhi et al. analyzed the progressive collapse in RC buildings with an L-shaped plan, and the structures were subjected to non-linear static incremental vertical analysis, and were examined by two methods including the sensitivity index and the Dynamic Increase Factor (DIF). The results showed that corner columns in L-shaped structure had the greatest potential for progressive collapse. In addition, L-shaped structures with higher heights performed better in response to collapse [20]. Design optimization has been carried out with using genetic algorithm by Olawale et al (2020) [21].

2. Analysis method and progressive collapse simulation

Nonlinear Dynamic (NLD) analysis is more accurate than other methodologies utilized to assess building vulnerability to progressive collapse via the alternative path method. However, relevant procedures and design guidelines allow for utilizing Linear Static

(LS) analysis and Nonlinear Static (NLS) analysis in progressive collapse analysis.

In general, in NLS, geometric relationships and materials used are considered nonlinear, and structural elements are allowed to withstand inelastic behavior. Ductility is obtained using the ratio of maximum displacement to yield displacement. Furthermore, evaluation is performed based on maximum strength, maximum ductility and amount of rotation. In addition, in NLS, the same load combination is utilized to calculate force-controlled and deformation-controlled actions. For this purpose, an increased gravity load combination is applied to those bays immediately adjacent to the removed element and at all floors above the removed element.

$$GN = \Omega N\,[1.2\,D + (0.5\,L \text{ or } 0.2\,S)] \quad (1)$$

The following gravity load combination is applied to floor areas away from the removed column or wall:

$$G = 1.2\,D + (0.5\,L \text{ or } 0.2\,S) \quad (2)$$

G: Gravity loads

G_N: Increased gravity loads for NLS

D: Dead load including facade loads

L: Live load including live load reduction according to standard ASCE7

S: Snow load

Ω_N: DIF for NLS analysis. (The following table is applicable to determine the appropriate Ω_N value for framed or load-bearing wall structures).

Table 1. DIF for NLS analysis [1].

Material	Structure Type	ΩN
Steel	Framed	$1.08 + 0.76/(\theta pra/\theta y + 0.83)$
Reinforced Concrete	Framed	$1.04 + 0.45/(\theta pra/\theta y + 0.48)$
	Load-bearing Wall	2
Masonry	Load-bearing Wall	2
Wood	Load-bearing Wall	2
Cold-formed Steel	Load-bearing Wall	2

In the NLS analysis method, the load is applied to the structure in the form of loading history that starts at zero and is increased to the final values. The load must be applied to the structure in at least 10 loading steps. The software used must be capable of incrementally increasing the load and applying it to the structure, and iteratively reaching convergence before proceeding to the next increment. Therefore, loading in this project was applied incrementally to the SAP2000 software according to the mentioned NLS analysis load combination.

In order to control the acceptance criteria in terms of deformation-controlled actions, primary and secondary elements and members should expect a greater deformation capacity than the calculated maximum deformation demand. The expected deformation capacity must be determined according to all deformation-controlled and force-controlled actions based on the document provided in the codes. Additionally, to perform force-controlled actions in all the primary and secondary elements, the following equation must be established:

$$\Phi\, Q_{CL} \geq Q_{UF} \quad (3)$$

Q_{UF}: Force-controlled action, from NLS model.

Q_{CL}: Lower-bound strength of a component or element for force-controlled actions.

Φ: Strength reduction factor

3. Specifications of the models

The structural models in this study included five, ten and fifteen-story buildings with a concrete wall-slab structural system, L/H ratios of 1, 1.5, and 2, a plan (see Fig.1), and a structural height of 2.75 m in each story. The gravity load distribution system of the floors was two-way reinforced concrete slab, and the lateral load-bearing system of the building was a shear wall in two directions. The plan of the mentioned building with an L/H=1, after simplification, was as follows:

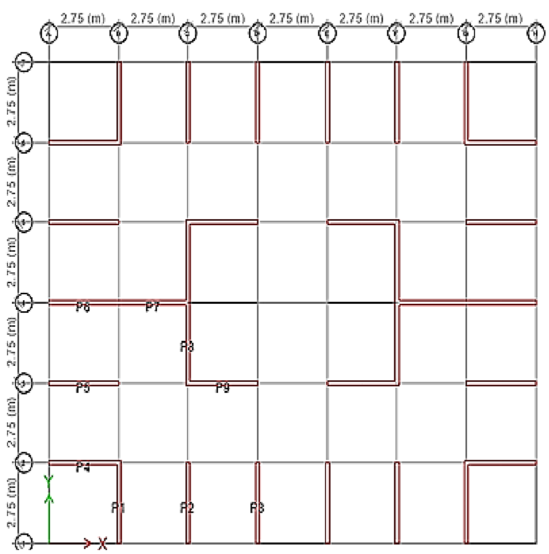


Fig. 1. Selected building plan.

The dead load (DL) of the floors and roof, regardless of the mass of the slab and the concrete wall, and live load (LL) of the floors and roof was 250 and 200 kg, respectively. The design of the building elements was based on the Code 2800, 4th edition, and Procedure ACI 318-14. The load combinations of this regulation and the ninth article of the national building code were applied. The characteristic compressive strength (f_c) of the concrete used in structural models was 250 kg/cm2, and the rebars were of type A3 with yield strength (f_y) of 4000 kg/cm^2. The site of the project was Kermanshah city (an area with high relative risk), and the site soil type was type II.

The axial stress-strain curve provided by Mander was used to define the nonlinear behavior of concrete and the park stress-strain curve was used to model the nonlinear behavior of steel rebars. SAFE V12 and ETABS V9 software were used to model, analyze and design reinforced concrete slabs and walls. The flexural design of the walls was based on the compressed section and double-axial tensile or bending. Given the fixed thickness of the walls, uniform reinforcement was used for designing. After analyzing and designing the desired buildings, the thickness of all walls and slabs was determined as 150 mm. In addition, reinforcement details for reinforced concrete walls and slabs in different floors of five, ten and fifteen-story buildings with a wall-slab structural system in L/H ratios of 1, 1.5 and 2, were obtained.

3.1. Removal scenarios

Removal of critical structural members at different building positions brings about different potentials for initiation of progressive collapse. In this study, to evaluate progressive collapse of buildings with slab-wall structural systems, wall removal scenarios were considered in accordance with the procedures given in the UFC. Since the selected plan was symmetrical, the removal scenarios of the exterior load-bearing wall were considered near the middle of the short side, near the middle of the long side, and at the corners of the building. On the other hand, engineering judgment was used to identify these critical locations. Furthermore, the removal scenarios of the interior load-bearing wall were determined under critical situations, in order to create the worst-case scenario.

3.2. Model denomination

The models were denominated based on the number of stories, the L/H ratio, and removal scenario. Model denomination was in the form of SN-RN-E $(o\ r)$ I$x\ M\ y\ M\ z\ M$ - (AG) or (MH) or (BR). Here, N is a random number. The number subsequent to the letter S represents the number of structural floors, the number subsequent to the letter R denotes the L/H ratio, and finally the letter E or I, indicates the type of removal scenario. Accordingly, E denotes exterior wall removal, I, represents interior wall removal, and z, y, and x represent the wall removal in the directions x, direction y, and height, respectively. M indicates the number of span and removal stories in the structure. AG, MH and BR represent the first story above the ground, story at mid-height, and story directly below the roof, respectively. Therefore, after modeling, nine, five, ten and fifteen-story buildings with a slab-wall structural system with L/H ratios of 1, 1.5 and 2 in each location were defined to remove the element in the plan, and then to perform alternative path analysis for the stories in the first floor (Above the Ground or AG), the last floor (Below the Roof or BR) and the fifth floor (Mid-Height or MH). Details of all wall removal scenarios in the five, ten and fifteen-story building models named above and examined in this study are showed in Table 2.

4. Validation of modeling

In the current research, after assigning the characteristics of the sections derived from the results of the analysis and design of the desired structures, ALP method was used to evaluate progressive collapse in the wall-slab structural systems. Concurrently, the SAP2000 software was used for static and dynamic analysis. For modeling of the reinforced concrete slab and wall, a nonlinear shell-layered element was used in the mentioned software. Also, the shell-layered element is a type of surface element that takes advantage of both membrane and flexural behavior, can withstand both in-plane and off-plane loading, and is considered a good alternative to 2D planar tension elements.

Using shell-layered elements, it would be possible to determine the difference in nonlinear materials in the thickness of the element and define each number of layers, each with a different position, thickness, behavior, and material. In addition, some materials can show a nonlinear behavior. As it can be seen in Fig.2, to determine the resultant stress, the stress in each layer was first calculated, and then numerical integration was performed in the thickness of the element. The Mindlin/Reissner formulation that takes into account shear deformations was considered for analyzing the bending.

According to the layer theory, slab rebars in each direction were modeled by a steel shell with a thickness equal to the area of the rebar per unit length of slab. To create a consistency between the stress-strain behaviors of the non-linear steel layer with axial stress-strain behavior of the rebar, behavior of non-linear materials of the shell should be made inactive in the orthogonal direction of rebar. This is because, in the actual model, rebars have axial behavior, and should not be involved in absorbing the tension of orthogonal directions.

Table 2. Details of all wall removal scenarios.

Scenarios of 5 story models	Scenarios of 10 story models	Scenarios of 15 story models
S5-R1-EX1Y1Z1-AG	S10-R1-EX1Y1Z1-AG	S15-R1-EX1Y1Z1-AG
S5-R1-IX2Z1-AG	S10-R1-IX2Z1-AG	S15-R1-IX2Z1-AG
S5-R1-EY1Z1-AG	S10-R1-EY1Z1-AG	S15-R1-EY1Z1-AG
S5-R1-IX2Y2Z1-AG	S10-R1-IX2Y2Z1-AG	S15-R1-IX2Y2Z1-AG
S5-R1-EX1Y1Z1-MH	S10-R1-EX1Y1Z1-MH	S15-R1-EX1Y1Z1-MH
S5-R1-IX2Z1-MH	S10-R1-IX2Z1-MH	S15-R1-IX2Z1-MH
S5-R1-EY1Z1-MH	S10-R1-EY1Z1-MH	S15-R1-EY1Z1-MH
S5-R1-IX2Y2Z1-MH	S10-R1-IX2Y2Z1-MH	S15-R1-IX2Y2Z1-MH
S5-R1-EX1Y1Z1-BR	S10-R1-EX1Y1Z1-BR	S15-R1-EX1Y1Z1-BR
S5-R1-IX2Z1-BR	S10-R1-IX2Z1-BR	S15-R1-IX2Z1-BR
S5-R1-EY1Z1-BR	S10-R1-EY1Z1-BR	S15-R1-EY1Z1-BR
S5-R1-IX2Y2Z1-BR	S10-R1-IX2Y2Z1-BR	S15-R1-IX2Y2Z1-BR
S5-R1.5-EX1Y1Z1-AG	S10-R1.5-EX1Y1Z1-AG	S15-R1.5-EX1Y1Z1-AG
S5-R1.5-IX2Z1-AG	S10-R1.5-IX2Z1-AG	S15-R1.5-IX2Z1-AG
S5-R1.5-EY1Z1-AG	S10-R1.5-EY1Z1-AG	S15-R1.5-EY1Z1-AG
S5-R1.5-IX2Y2Z1-AG	S10-R1.5-IX2Y2Z1-AG	S15-R1.5-IX2Y2Z1-AG
S5-R1.5-EX1Y1Z1-MH	S10-R1.5-EX1Y1Z1-MH	S15-R1.5-EX1Y1Z1-MH
S5-R1.5-IX2Z1-MH	S10-R1.5-IX2Z1-MH	S15-R1.5-IX2Z1-MH
S5-R1.5-EY1Z1-MH	S10-R1.5-EY1Z1-MH	S15-R1.5-EY1Z1-MH
S5-R1.5-IX2Y2Z1-MH	S10-R1.5-IX2Y2Z1-MH	S15-R1.5-IX2Y2Z1-MH
S5-R1.5-EX1Y1Z1-BR	S10-R1.5-EX1Y1Z1-BR	S15-R1.5-EX1Y1Z1-BR
S5-R1.5-IX2Z1-BR	S10-R1.5-IX2Z1-BR	S15-R1.5-IX2Z1-BR
S5-R1.5-EY1Z1- BR	S10-R1.5-EY1Z1- BR	S15-R1.5-EY1Z1- BR
S5-R1.5-IX2Y2Z1-BR	S10-R1.5-IX2Y2Z1-BR	S15-R1.5-IX2Y2Z1-BR
S5-R2-EX1Y1Z1-AG	S10-R2-EX1Y1Z1-AG	S15-R2-EX1Y1Z1-AG
S5-R2-IX2Z1-AG	S10-R2-IX2Z1-AG	S15-R2-IX2Z1-AG
S5-R2-EY1Z1-AG	S10-R2-EY1Z1-AG	S15-R2-EY1Z1-AG
S5-R2-IX2Y2Z1-AG	S10-R2-IX2Y2Z1-AG	S15-R2-IX2Y2Z1-AG
S5-R2-EX1Y1Z1-MH	S10-R2-EX1Y1Z1-MH	S15-R2-EX1Y1Z1-MH
S5-R2-IX2Z1-MH	S10-R2-IX2Z1-MH	S15-R2-IX2Z1-MH
S5-R2-EY1Z1-MH	S10-R2-EY1Z1-MH	S15-R2-EY1Z1-MH
S5-R2-IX2Y2Z1-MH	S10-R2-IX2Y2Z1-MH	S15-R2-IX2Y2Z1-MH
S5-R2-EX1Y1Z1-BR	S10-R2-EX1Y1Z1-BR	S15-R2-EX1Y1Z1-BR
S5-R2-IX2Z1-BR	S10-R2-IX2Z1-BR	S15-R2-IX2Z1-BR
S5-R2-EY1Z1- BR	S10-R2-EY1Z1- BR	S15-R2-EY1Z1- BR
S5-R2-IX2Y2Z1-BR	S10-R2-IX2Y2Z1-BR	S15-R2-IX2Y2Z1-BR

In addition, in nonlinear behavior of reinforced concrete slabs, effects of concrete cracking based on stress-strain curve of concrete were considered by the SAP2000 software, since slab flexural stiffness reduction factor was not required in the software. Fig.3 shows the Nonlinear modeling of slab and reinforced concrete wall in the SAP2000 Software via the shell-layered element. To test the validity and reliability of the SAP2000 software for modeling and NSL analysis of wall-slab structural systems with shell-layered elements, the results from the laboratory model studies performed by Dazio et al [21] were used, and a shear wall was modeled in the SAP2000 software using shell-layered elements. The wall-to-ground connection was Fixed Support. The 630 KN compressive load was applied to the wall by gravity and a load was applied laterally to the wall, which increased step by step during nonlinear static analysis until the displacement of the wall in the horizontal direction UX reached 60 mm. Lateral profile and wall meshing were obtained via trial and error. In addition, setting mode, load application mode, and the condition of the reinforced concrete wall in the Diazo laboratory model are presented in Fig. 4, 5.

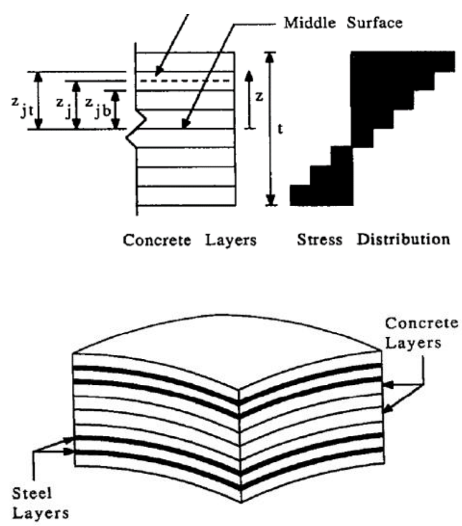
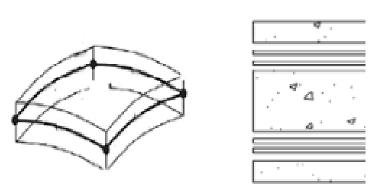


Fig. 2. Stress distribution of the layered shell element [23].



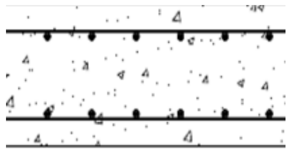


Fig. 3. Nonlinear modeling of slabs and walls.

Fig. 4. Performed settings and load application procedure in Diazo laboratory model [22].

In Fig.6 the displacement of UX and the lateral force of the wall were taken from the SAP2000 software, and compared with the laboratory values.The maximum values of the horizontal force of the laboratory model and numerical analysis were 445.5 and 408.3 KN, respectively. Therefore, it was observed that the difference between the two values obtained was 8.3% and was relatively low.

A comparison of the results from the laboratory operations and modeling in the Sap for the shell-layered element showed that the response obtained from numerical analysis by using shell-layered element could accurately simulate experimental response, and the numerical method used had an acceptable accuracy in estimating stiffness, deformation and capacity of the reinforced planar concrete members, and it could be a reliable method used in nonlinear modeling of various planar models.

For selecting an appropriate software in the current research and for validation and reliability of the modeling method for progressive collapse evaluation, after modeling a five story building with a slab-wall structural system built with a tunnel-form and performing the necessary analysis, the concrete slab and wall of the building were designed via the SAFEV12 and the ETABS V9 software, then the modeling of the building was done with two software, the sap2000v20 and ABAQUS. For this purpose, the concrete damage plasticity model was selected to simulate the behavior of concrete and the Mander relations were considered for the stress-strain curve of concrete. Furthermore, the Elastic-perfectly plastic curve was used for steel reinforcement.

It is worth noting that in the Finite element (FE) modeling in the Abacus Software the Embedded Element (EE) technique was used to define the interaction between the rebars and concrete. Four-node shell elements with reduced integration (S4R) was used to mesh the slab and wall and also truss elements (T3D2) were selected to mesh the rebars. Finally, Non-Linear Static (NLS), and Non-Linear Dynamic (NLD) analysis of the mentioned building were performed using the alternative load path method to investigate the progressive collapse according to the existing codes and subsequently, the results were compared. Static analysis results showed that the maximum deformation and principal stress pattern in the sap2000v20 and ABAQUS software were almost the same, and the maximum displacement in both models was about 0.582 mm, and the maximum principal stress was about 86200 Kg per square meter.

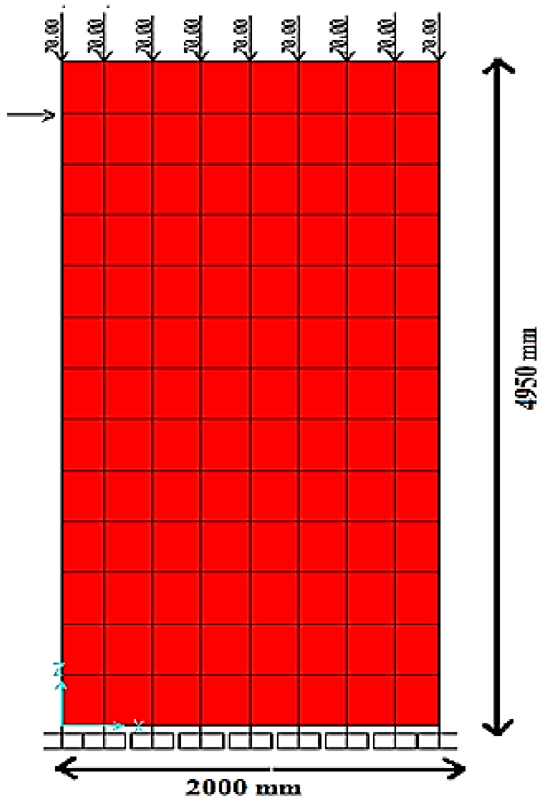


Fig. 5. Peripheral view of the shear walls and meshing procedure of shear wall in the SAP2000 software.

The results obtained from the evaluation of progressive collapse by the nonlinear dynamic method showed that deformation and maximum principal stress in the sap2000v20 and ABAQUS software were almost identical, and the maximum vertical displacement in both models was about 3.65 mm, and the maximum principal stress was almost 310,000 kg per square meter. Besides, other results showed that the features of the Sap2000v20 software and utilization of layered-shell elements and modeling performed in this software were sufficiently accurate in calculating the deformations and stresses, and could be used as a reliable tool in non-linear modeling and LS, ND, and NLS analysis of progressive collapse.

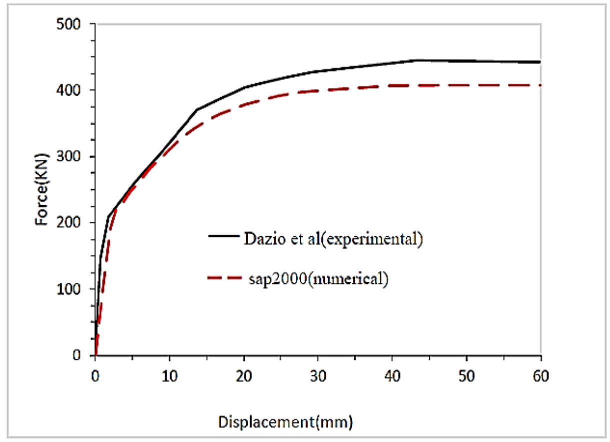


Fig. 6. Comparison of laboratory work results and numerical model of nonlinear shell element.

5. A review of analysis results

After designing the slab and reinforced concrete wall using the SAFE V12 and ETABS V9 software, 3D modeling was performed in the SAP2000 software. After assigning the characteristics of the sections, load combination of Non-Linear Dynamic (NLD) analysis was statically applied to the undamaged structure, and structure analysis was performed. The end forces of the members determined for removal were then identified. Afterwards, the gravity and lateral load combination, as well as the obtained forces of the removed element, were statically applied to the damaged structure in the opposite direction. Finally, the desired NLS analysis was performed on the constructed models, and the results were evaluated. Some of the results are as follows:

Fig.8 and Fig.9 show the deformation contour prior to and following wall removal in the Scenario, S10-R1-EX1Y1Z1-AG.

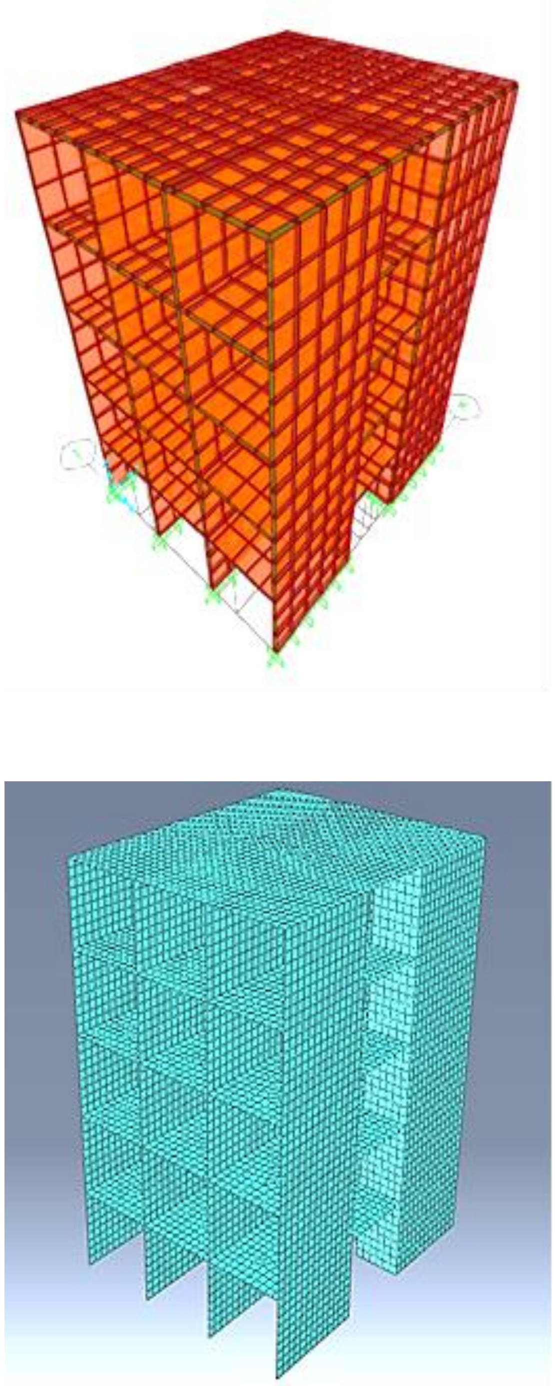


Fig. 7. 3D model designed in sap2000v20 and ABAQUS.

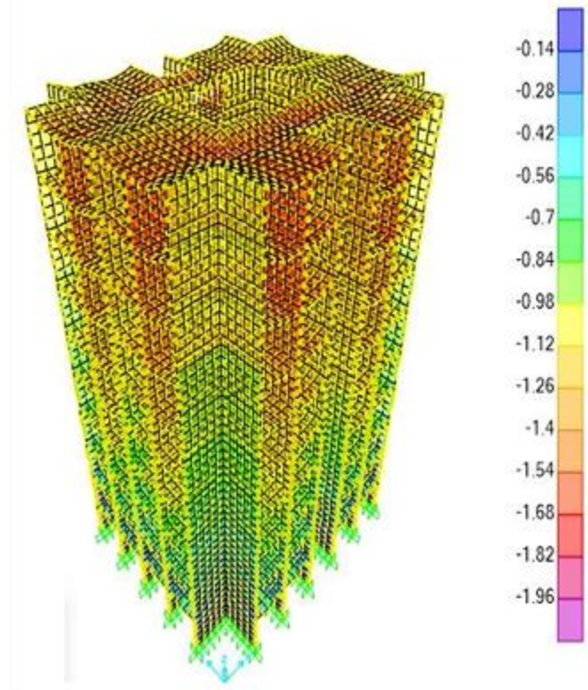


Fig. 8. Deformation contour prior to wall removal in Scenario, S10-R1-EX1Y1Z1-AG.

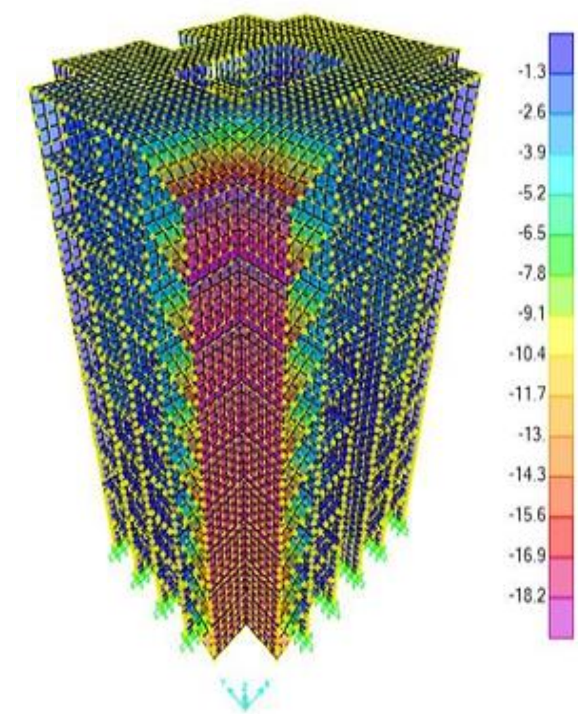


Fig. 9. Deformation contour following wall removal in Scenario, S10-R1-EX1Y1Z1-AG.

5.1. Comparison of displacement of removal location of ten-story structures

Fig.10 shows the vertical displacement of the wall removal location in the modeled ten-story buildings with the L/H ratio equal to 1, in different scenarios in the plan and in the floors where the walls were removed.

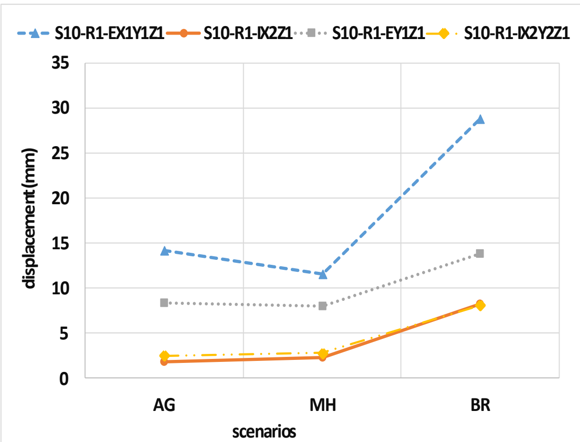


Fig. 10. Comparison of removal location displacement in ten-story models in different scenarios.

As can be seen, the vertical displacement rate in the interior wall removal scenarios was significantly lower than the rate in the corner wall removal scenarios Furthermore, the length of the removed wall was of significance in the displacement rate of the removal location, such that the removal of interior and corner walls with a higher length led to an increase in the vertical displacement rate. The results show that the changes in the vertical displacement rate in the ten-story building models with L/H ratios of 1.5 and 2 in different scenarios in the plan and floors were similar to this trend.

Fig.11 represents a comparison of the vertical displacement of the wall removal location in different scenarios in the AG floor when changing the L/H ratios of 1, 1.5 and 2.

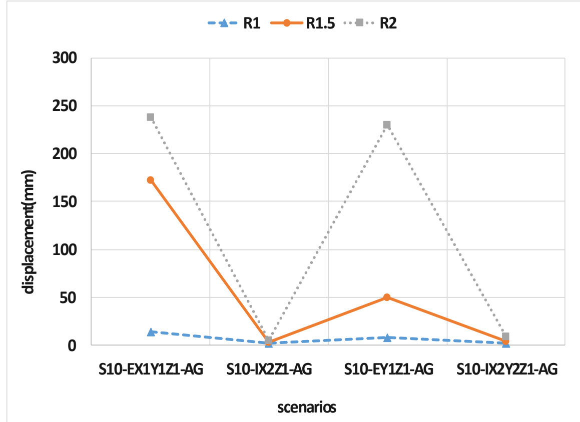


Fig. 11. Comparison of displacement in removal location in ten-story models with changing L/H ratio.

As can be seen, the vertical displacement rate of the removal location in different scenarios in the plan changed significantly by changing the L/H ratio, especially in the external wall removal scenarios. Additionally, by changing the L/H ratio from 1 to 1.5, the vertical displacement rate in the interior wall removal scenarios increased up to two times, and up to twelve times in the exterior wall removal scenarios. The trend of variations in the vertical displacement of removal location in different scenarios in the plan by changing the L/H ratio in the MH floor and BR floor, was similar to the AG floor, the difference was that by increasing the L/H ratio, the rate of increase in the vertical displacement of the removal location in the upper floors was more noticeable in the scenarios of removal of the interior walls and less significant in scenarios of removal of the external walls.

5.2. Status and performance of slabs and walls of ten -story structures

To evaluate the performance of slabs and walls, Table 3 presents von Mises stress in slab rebars and model walls, the maximum stress and compressive strain in concrete under different scenarios in the plan and floors of the ten-story buildings.

Table 3. Von Mises stress condition, maximum stress and compressive strain in ten-story models.

row	scenarios	MAX VM Stress (Mpa)	MAX concrete stress (Mpa)	MAX concrete strain (e-4)	MAX VM strain (e-4)	Start of failure
1	**S10-R1-EX1Y1Z1-AG**	**176.43**	**14.06**	**6.50**	**11.05**	□
2	**S10-R1-IX2Z1-AG**	**55.23**	**6.97**	**2.86**	**3.07**	□
3	**S10-R1-EY1Z1-AG**	**99.38**	**10.71**	**4.85**	**6.27**	□
4	**S10-R1-IX2Y2Z1-AG**	**185.31**	**19.15**	**9.00**	**10.65**	□
5	**S10-R1-EX1Y1Z1-MH**	**82.00**	**10.00**	**4.10**	**6.80**	□
6	**S10-R1-IX2Z1-MH**	**31.00**	**3.90**	**1.58**	**1.80**	□
7	**S10-R1-EY1Z1-MH**	**80.00**	**10.20**	**4.17**	**4.65**	□
8	**S10-R1-IX2Y2Z1-MH**	**108.00**	**12.70**	**5.27**	**6.18**	□
9	**S10-R1-EX1Y1Z1-BR**	**283.41**	**11.50**	**5.32**	**15.00**	□
10	**S10-R1-IX2Z1-BR**	**47.09**	**5.35**	**2.19**	**2.64**	□
11	**S10-R1-EY1Z1-BR**	**75.00**	**9.43**	**4.32**	**5.12**	□
12	**S10-R1-IX2Y2Z1-BR**	**51.34**	**6.92**	**2.84**	**2.89**	□
13	**S10-R1.5-EX1Y1Z1-AG**	**629.27**	**38.55**	**26.22**	**157.82**	□
14	**S10-R1.5-IX2Z1-AG**	**75.00**	**9.00**	**3.88**	**3.89**	□
15	**S10-R1.5-EY1Z1-AG**	**464.39**	**30.26**	**14.54**	**25.84**	□
16	**S10-R1.5-IX2Y2Z1-AG**	**342.76**	**33.30**	**16.09**	**18.25**	□
17	**S10-R1.5-EX1Y1Z1-MH**	**486.00**	**32.00**	**31.00**	**190.00**	□
18	**S10-R1.5-IX2Z1-MH**	**58.00**	**6.80**	**2.85**	**3.25**	□
19	**S10-R1.5-EY1Z1-MH**	**416.00**	**24.10**	**14.00**	**27.00**	□
20	**S10-R1.5-IX2Y2Z1-MH**	**214.00**	**7.86**	**3.41**	**12.00**	□
21	**S10-R1.5-EX1Y1Z1-BR**	**455.77**	**28.56**	**29.21**	**89.00**	□
22	**S10-R1.5-IX2Z1-BR**	**385.24**	**19.02**	**9.25**	**23.15**	□
23	**S10-R1.5-EY1Z1- BR**	**430.10**	**27.11**	**18.20**	**49.88**	□
24	**S10-R1.5-IX2Y2Z1-BR**	**284.00**	**21.93**	**10.96**	**15.06**	□
25	**S10-R2-EX1Y1Z1-AG**	**688.96**	**46.03**	**27.12**	**243.75**	□
26	**S10-R2-IX2Z1-AG**	**124.80**	**17.60**	**8.50**	**8.16**	□
27	**S10-R2-EY1Z1-AG**	**644.73**	**58.67**	**47.88**	**139.65**	□
28	**S10-R2-IX2Y2Z1-AG**	**604.66**	**58.10**	**32.30**	**38.00**	□
29	**S10-R2-EX1Y1Z1-MH**	**578.55**	**42.13**	**25.22**	**223.44**	□
30	**S10-R2-IX2Z1-MH**	**117.12**	**14.28**	**7.09**	**7.54**	□
31	**S10-R2-EY1Z1-MH**	**632.24**	**59.17**	**47.88**	**129.68**	□
32	**S10-R2-IX2Y2Z1-MH**	**397.85**	**17.46**	**11.07**	**21.27**	□
33	**S10-R2-EX1Y1Z1-BR**	**600.00**	**50.00**	**41.25**	**100.00**	□
34	**S10-R2-IX2Z1-BR**	**523.75**	**31.25**	**25.00**	**78.75**	□
35	**S10-R2-EY1Z1- BR**	**550.00**	**52.50**	**52.50**	**87.50**	□
36	**S10-R2-IX2Y2Z1-BR**	**862.50**	**50.00**	**60.00**	**122.50**	□

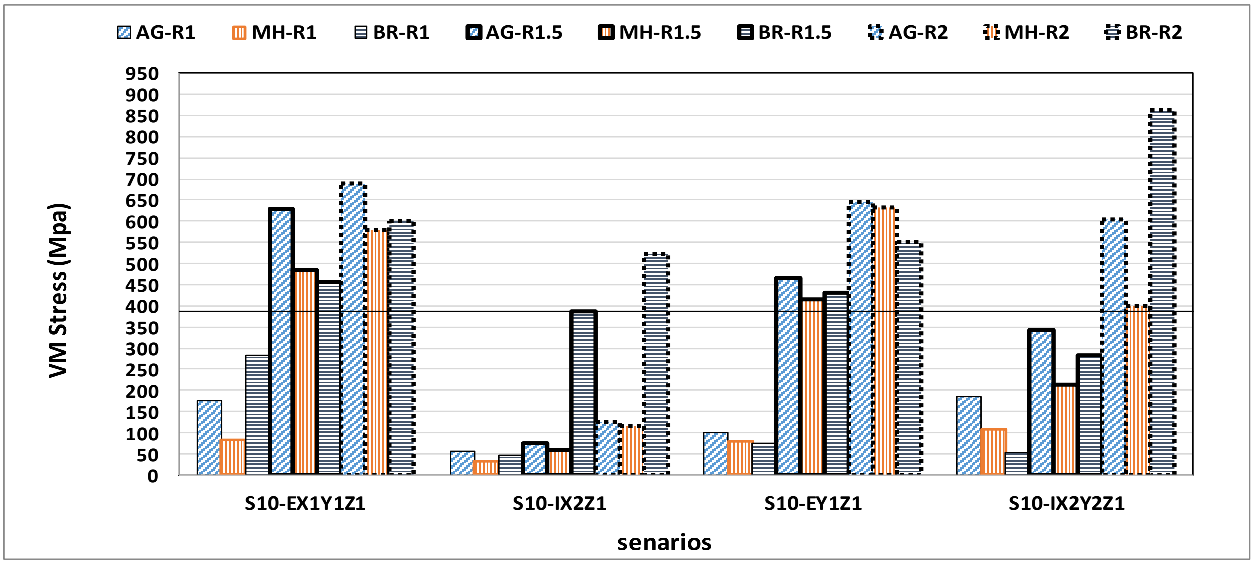


Fig. 12. Von Mises stress condition (MPa) in slab and wall rebars of ten -story models.

On the other hand, Fig.12 reports a comparative analysis of the trend of changes in L/H ratios 1, 1.5 and 2.

As can be seen, in most models and wall removal scenarios in the plan and floors of the ten-story buildings, together with the yield of rebar, the maximum compressive stress in concrete exceeded the characteristic compressive strength with the maximum strain in concrete exceeding 0.002.

In the models with an L/H ratio of 1, no yielding ensued in the removal scenarios of the wall in plans and floors. In contrast, maximum von Mises stress exceeded the yield point in the models with an L/H ratio of 1.5 in all external wall removal scenarios and one of the interior removal scenarios (S10-R1.5-IX2Y2Z1-AG),and in the models with the L/H ratio of 2 in all exterior/interior wall removal scenarios (except S10-R2-IX2Z1-MH, S10-R2-IX2Z1-AG). In the removal scenarios, S10-R1.5-EY1Z1-AG, S10-R1.5-EY1Z1-MH, S10-R1.5-EY1Z1-BR, S10-R1.5-IX2Y2Z1-AG and S10-R2-IX2Y2Z1-MH, despite the fact that the maximum von Mises stress in the slab and wall rebars had exceeded the yield level, and the maximum compressive stress in concrete was higher than or close to the compressive strength, but the maximum strain in concrete was less than 0.002. Additionally, according to the results obtained from NLS analysis, in the scenarios where the maximum strain in concrete was over 0.002, large deformations and maximum strength ensued immediately in the next steps of the analysis, and failure in the structural components was highly likely to occur. Therefore, the maximum strain in the concrete was seemingly a more appropriate criterion for determining progressive collapse. Given the ratio of maximum displacement to yield displacement, maximum strength of structural components, and maximum ductility, it can be concluded that buildings with slab-wall structural system may exhibit brittle failure behavior influenced by progressive collapse.

As can be seen in Fig.12, the von Mises stress was higher in the AG floor removal scenarios than in the MH floor removal scenarios. In addition, when the number of removed walls was the same, von Mises stress and collapse potential in exterior scenarios were greater due to the bridging of members over each other which created a more suitable alternative path in removal scenarios of interior walls and MH floors. The Von Mises stress in different scenarios in the plan increased with increasing the L/H ratio. As a result, under exterior wall removal scenarios and removal scenario of four interior walls (IX2Y2Z1), by increasing the L/H ratio from 1 to 1.5 and 2, the amount of stress exceeded the yield limit and collapse occurred. Also, collapse occurred in exterior wall removal scenarios, removal scenario of four interior/exterior walls (IX2Y2Z1) in the floors, and in exterior/interior wall removal scenarios at the AG floor due to an increase in L/H ratio from 1 to 2.

5.3. the displacement of the wall removal location of 5-story and 15 -story structures

Fig.13 and Fig.14 show the vertical displacement of the wall removal location in the modeled five-story and fifteen-story buildings with the L/H ratio equal to 1, in different scenarios in the plan and in the floors where the walls were removed.

As can be seen, the vertical displacement rate in the interior wall removal scenarios was significantly lower than the rate in the corner wall removal scenarios. Furthermore, in five-story buildings, the vertical displacement rate in the removal scenarios in MH floors was often less than the rate in the removal scenarios in the AG and BR floors. Also, with increasing the height level of the wall removal location in fifteen-story buildings, vertical displacement rate increased. The results showed that the changes in the vertical displacement rate in the five-story and fifteen-story building models with L/H ratios of 1.5 and 2 in different scenarios in the plan and floors were similar to this trend.

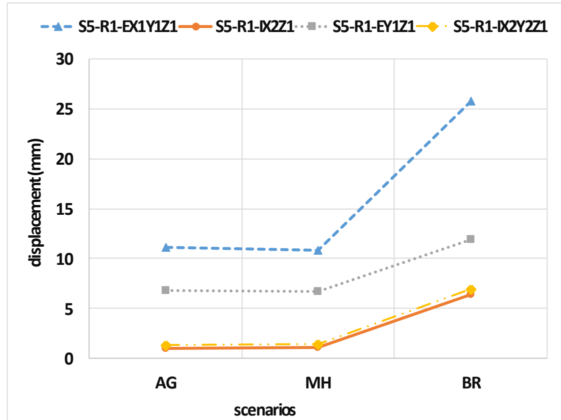


Fig. 13. removal location displacement in five-story models in different scenarios.

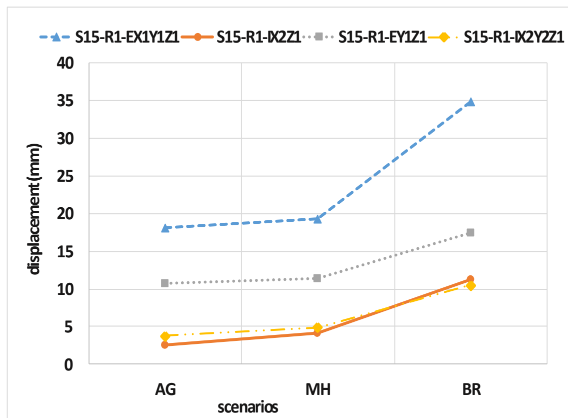


Fig. 14. removal location displacement in fifteen-story models in different scenarios

Fig.15 and Fig.16 represent a comparison of the vertical displacement of the wall removal location in different scenarios in the AG floor with a change in L/H ratios of 1, 1.5 and 2.

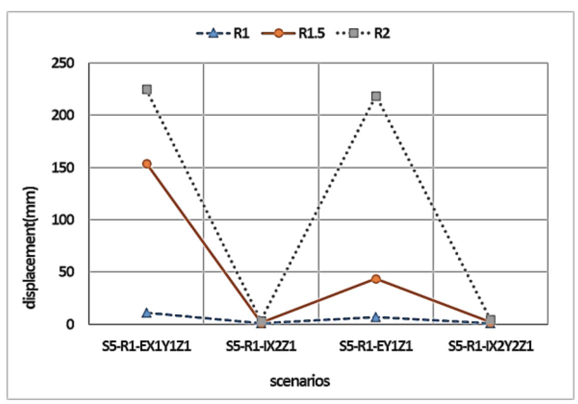


Fig. 15. Comparison of displacement in removal location in five -story models with changing L/H ratio.

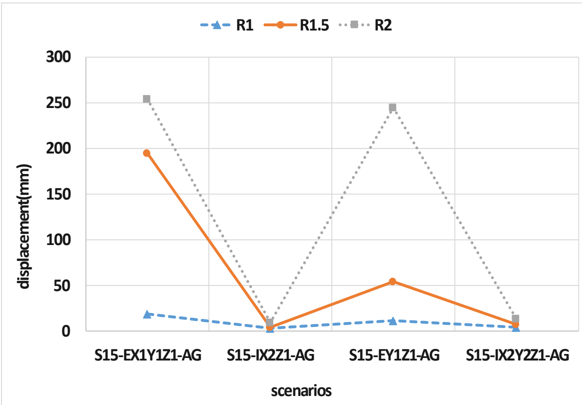


Fig. 16. Comparison of displacement in removal location in fifteen-story models with changing L/H ratio/

As can be seen, the vertical displacement rate of the removal location in different scenarios in the plan, changes significantly by changing the L/H ratio, especially in the external wall removal scenarios. Accordingly, by changing the L/H ratio from 1 to 1.5, the vertical displacement rate in the interior wall removal scenarios increased by up to two times, and up to ten times in the exterior wall removal scenarios. The trend of variations in the vertical displacement of removal location in different scenarios by changing the L/H ratio in the MH and BR floor, was similar to the AG floor, the difference being that, with increasing the L/H ratio, the rate of increase in the vertical displacement of the removal location in the upper floors was higher in the scenarios of removal of interior walls and lower in scenarios of removal of external walls.

5.4. Status and performance of slabs and walls of 5-story and 15 -story structures

Fig.17 and Fig.18 show von Mises stress condition in slab and wall rebars in different scenarios in 5-story and 15 -story Structures by changing the L/H ratios of 1, 1.5 and 2.As can be seen, In the models with an L/H ratio of 1, no yielding occurred in the wall removal scenarios in the plans and yielding floors However, in five-story Structures the maximum von Mises stress exceeded the yield point in the models with the L/H ratio of 1.5 in all external wall removal scenarios, in the models with the L/H ratio of 2 in all external wall removal scenarios at AG floor and MH floor, and also in all interior and exterior wall removal scenarios at BR floor. As can be seen in Fig. 17, the von Mises stress was higher in the AG floor removal scenarios than in the MH floor removal scenarios in addition, when the number of removed walls was the same, von Mises stress and collapse potential in exterior scenarios were greater due to the bridging of members over each other and the creation of a more suitable alternative path in removal scenarios of interior walls and MH floors.

Furthermore, in fifteen-story Structures, S15-R1-IX2Y2Z1-AG, the maximum compressive stress in concrete was higher than the compressive strength. In addition, in the models with an L/H ratio of 1.5 in all external wall removal scenarios, two interior removal scenarios namely; S15-R1.5-IX2Y2Z1-AG, S15-R1.5-IX2Z1-BR, and in the models with the L/H ratio of 2 in all interior/exterior wall removal scenarios except, S15-R2-IX2Z1-AG, S15- R2-IX2Z1-MH, the maximum von Mises stress exceeded the yield point.

The Von Mises stress in different scenarios in the plan increased with increasing the L/H ratio. As a result, under exterior wall removal scenarios, by increasing the L/H ratio from 1 to 1.5 and 2, the amount of stress exceeded the yield point and collapse occurred.

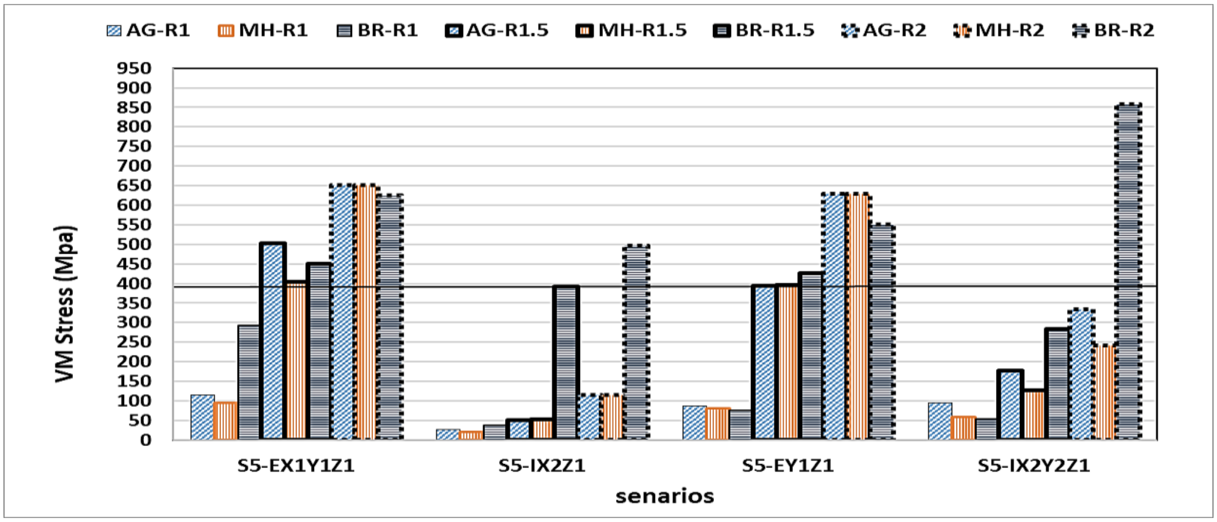


Fig. 17. Von Mises stress condition (MPa) in slab and wall rebars of five-story models.

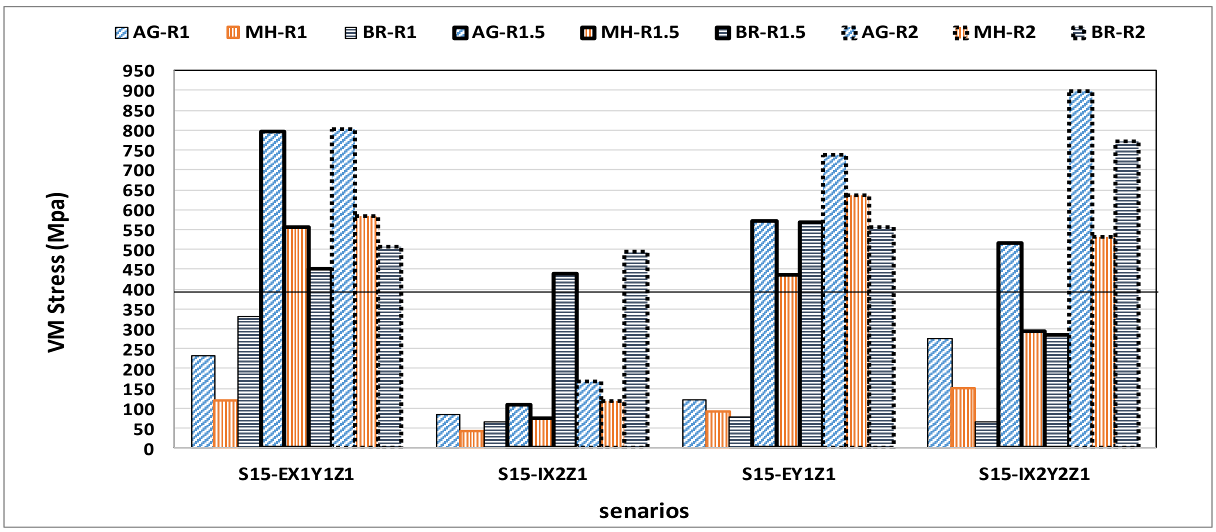


Fig. 18. Von Mises stress condition (MPa) in slab and wall rebars of fifteen-story models.

5.5. Comparison of displacement of the removal location of 5, 10, and 15-story structures

Fig.19 presents the vertical displacement of the wall removal location in five, ten, and fifteen-story structures in different scenarios in the plan in the AG floor with respect to L/H ratios of 1, 1.5, and 2.

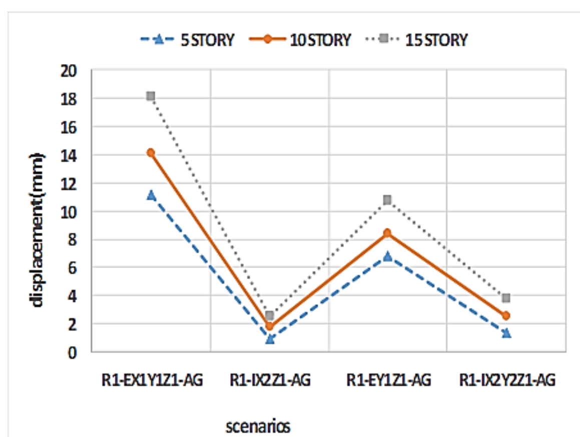


Fig. 19. A comparison of removal location displacement of the models with different floors in the L/H ratio 1.

As can be seen, the vertical displacement rate in the removal location in the plan increased with an increase in the number of floors. Accordingly, in models with the L/H ratio of 1, vertical displacement rate of the removal location in external wall removal scenarios increased by increasing the building floor numbers from 5 to 10 (25%) and from 10 to15 (32%). The increase rate was significantly higher in the interior wall removal scenarios.

In the models with an L/H ratio of 1.5 and 2, vertical displacement rate increased by increasing the number of floors in the interior and exterior wall removal scenarios, with the difference being that the rate of increase in the vertical displacement of the removal location decreased by increasing the L/H ratio of the floor. The trend of changes in the vertical displacement of the removal location in different scenarios in the plan, by changing the number of floors in the MH floor and BR floor, was similar to the AG floor.

5.6. Status and performance of the slabs and walls of 5, 10, and 15-story structures

Fig.20 presents the von Mises stress condition (MPa) in slab rebars and model walls, the maximum stress and compressive strain in concrete under different scenarios in the plan in five, ten and fifteen-story buildings, in the plan in the first story above the ground (AG), with respect to L/H ratios of 1, 1.5, and 2. As can be seen, in all exterior/interior wall removal scenarios in the AG floor, an increase in the number of floors led to an increase in the von Mises stress on the slab and wall rebars, the maximum stress, and compressive strain in the concrete. This trend was also established in different scenarios in the plan by changing the number of floors of the building in the MH and BR floors.

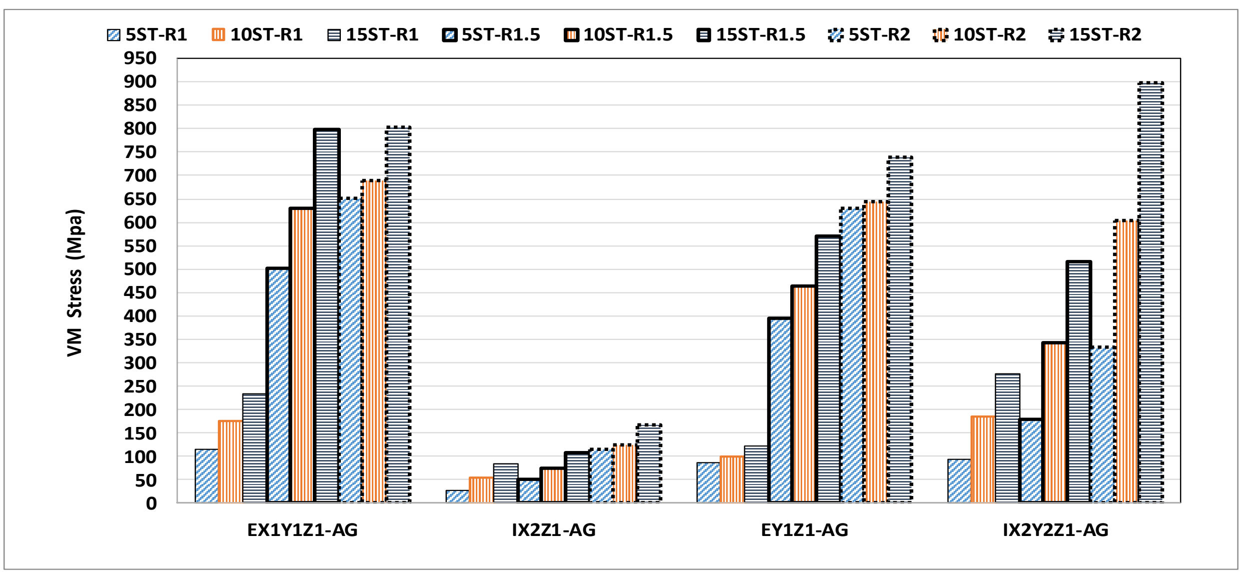


Fig. 20. Von Mises' stress condition (MPa) in slab and wall rebars at AG floor.

6. Conclusion

Due to the importance of progressive collapse and lack of information about the behavior of buildings with the reinforced concrete (RC) wall-slab structural system and considering the potential for progressive collapse of this type of structure and the buildings with related regulations and previous studies were less considered, therefore, in this study the potential for progressive collapse of buildings with the RC wall-slab structural system was investigated. For this purpose, at first, nine concrete buildings of five, ten and fifteen-stories with wall-slab structural systems, the ratio of spans length to the story height (L/H) of 1, 1.5 and 2 and with a structural height of 2.75 meters in each story, were designed by the ETABS V16 software. Then, using the SAP2000 software and nonlinear shell layer element, after modeling validation based on experimental and numerical studies, nonlinear static analysis by the alternative load path method were performed on the desired models and the results were evaluated. The main outcomes can be summarized as follow:

1, The vertical displacement rate in the interior wall removal scenarios was significantly lower than the rate in the corner wall removal scenarios. Furthermore, when the number of removed wall was the same, von Mises stress and collapse potential in the exterior scenarios were greater due to the bridging of members over each other and the creation of a more suitable alternative path in the removal scenarios of interior walls and MH floors. This trend was established in different models by changing the number of floors and the L/H ratio.

2, The rate of vertical displacement of the removal location, the maximum von Mises stress in the slab and wall rebars, the maximum compressive stress and strain in concrete in different scenarios in the plan and floors increased significantly by increasing the L/H ratio. So that in the analysis performed, by changing the L/H ratio from 1 to 1.5, the vertical displacement rate in the interior wall removal scenarios increased up to two times, and up to twelve times in the exterior wall removal scenarios. And also, by increasing the L/H ratio from 1 to 1.5 and 2, in the scenarios of removal of the external and internal walls in 100% and 36% of cases, respectively, a number of the rebars yielded and in 66% and 25% of these cases led to progressive damage to the building.

3, Given the ratio of maximum displacement to yield displacement, maximum strength of structural components, and maximum ductility, it can be concluded that buildings with slab-wall structural systems may exhibit brittle failure behavior influenced by progressive collapse.

4, In the results obtained from NLS analysis, the rate of vertical displacement of the removal location, the maximum von Mises stress in the slab and wall rebars, the maximum compressive stress and strain in concrete in different scenarios in the plan and floors increased by increasing the number of stories. So that in the models with the L/H ratio of 1, the vertical displacement rate of removal location in the external wall removal scenarios increased by increasing the building floor numbers from 5 to 10 (25%) and from 10 to 15 (32%). And also, the vulnerability of the structure to progressive collapse went higher. While the results of previous studies, which had been performed mainly on frame structures, showed that as the height of the structure increased, the reliability of the structure against progressive collapse increased.

5, In most models and wall removal scenarios in the plan and floors, together with the yield of rebars, the maximum compressive stress in concrete exceeded the characteristic compressive strength and maximum strain in concrete exceeded 0.002. Additionally, in the scenarios where the maximum strain in concrete was over 0.002, large deformations and maximum strength ensued immediately in the next steps of the analysis, and failure in structural components was highly likely to occur. So, the maximum strain in concrete was seemingly a more appropriate criterion for determining progressive collapse.

6, The results show the relatively high strength of the buildings with wall-slab structural systems against progressive collapse. As with buildings with an L/H ratio=1 resistance to progressive collapse was seen but by increasing the L/H ratio progressive collapse potential increased.

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