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Comparison of Seismic Performance of Improved Structures Against Progressive Failure under the Influence of Near and Far-Field Earthquakes

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Abstract

In this study, the seismic performance of the improved structures against progressive failure under the influence of near-field and far-field earthquakes was investigated. For this purpose, a 5-story reinforced concrete structure with a medium-flexural frame, which was analyzed, designed, and implemented according to the first edition of the 2800 Code, is analyzed and redesigned according to the fourth edition of the 2800 Code, and after Demand-to-Capacity Ratio (DCR), the necessity of seismic improvement of the building in question is investigated according to the 360 publication. Next, by adding a shear wall to the original structure under study, its performance is investigated in terms of increasing lateral stiffness and improving seismic performance. The variables examined to select the best improved structure include adding a symmetrical or asymmetrical shear wall to the peripheral or internal frame. Removing the corner or middle column from the first or fifth floor, as well as using earthquake records from the far and near fields, are other variables of this study. The results of this study show that the best scenario for using a shear wall is to place it symmetrically in the perimeter frame. The results of the structural response study, including vertical displacement, horizontal displacement of the roof, relative displacement of floors, and dissipated strain energy in different scenarios of the best improved structure, indicate that the worst case is related to the removal of the corner column of the fifth floor, and the best case is associated with the removal of the middle column of the first floor. Also, in comparing the records of the far and near fields, the response of the structures in the near field is greater than in the far field.

Keywords: Progressive failure, Concrete shear wall, Concrete flexural frame, Earthquake in the far and near fields.

1 | Introduction

Currently, the construction or design error of structural members and the vulnerability of the entire structure due to the removal of elements due to unexpected events such as earthquakes and progressive failure of structural members due to seismic excitations have become a significant challenge [1]. When a structure is

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exposed to unexpected loads such as earthquakes, impacts, explosions, fires, etc., that are not considered in the normal design process, the structure may be vulnerable. Progressive failure is referred to as a phenomenon in which local failure and removal of one or more structural elements such as columns due to unexpected loads such as accidents, earthquakes, and terrorist acts, due to lack of appropriate strength, spreads to the entire structure and ultimately leads to the progressive collapse of the entire structure (Similar to domino failure) so that the final failure has no proportion to the initial failure. Research on progressive failure can be classified into two different approaches: 1) the development of structural systems that prevent progressive failure, and 2) an appropriate and efficient analytical method. Structural safety has always been a key concern for engineers designing engineering projects. One of the structural failure mechanisms that has received much attention in recent decades is progressive failure. One or more structural members fail suddenly due to an accident or explosion, and any redistribution of load causes failure of other structural elements, and the building gradually collapses [2].

Errors in the construction or design of structural members due to the removal of one or more load-bearing elements due to unexpected events (Such as earthquakes and explosions) make the entire structure vulnerable. In this mechanism, one or more structural elements are damaged due to reasons such as accidents or explosions. With each redistribution of the load, damage occurs and spreads to other elements, which ultimately leads to the progressive destruction of the structure [1].

By examining various sources and articles on the phenomenon of progressive damage, it is understood that although many studies have been conducted in this regard, due to its importance and special position, the subject of the phenomenon of progressive damage can be further expanded in order to identify all its natures. In fact, progressive damage is a phenomenon in which a minor damage or local failure destroys the entire structure or a large part of it, in such a way that the final damage is not proportional to the initial damage. The nonlinear dynamic analysis method provides more accurate answers than static methods for investigating the progressive damage of structures. Due to the time-consuming nature of nonlinear dynamic analysis, the use of simpler models and approximate methods has become very important [4].

In fact, progressive failure is the spread of an initial local failure from one component to another, which eventually causes the entire structure or a large asymmetric part of it to collapse. To resist progressive failure, the structure must have the ability to bridge to the other side of the defective element. In the alternative path method, which is the most practical and common method of evaluating a structure against this event, three linear static analyses, a nonlinear static analysis, and a nonlinear dynamic analysis are used. Robustness is defined as insensitivity to a local failure. In other words, the robustness of the structure.

The failure is localized when a structural failure occurs. A structurally sound structure can withstand loading without any failure occurring. In fact, the strength index is used to quantify the results of progressive failure. Resistance to collapse can be achieved in various ways, including increasing the strength. Factors affecting strength include ductility, cohesion, and uncertainty. By increasing each of them, the structure will be able to redistribute the applied loads, and the structure will not collapse.

In the case of seismic retrofitting to achieve the highest seismic performance of existing buildings, there are two primary methods before the structure is exposed to an earthquake. One of these methods is to add a new structural element, such as a structural wall or various types of steel braces. The other method involves retrofitting inefficient structural elements using concrete or steel jacketing, or Fiber-Reinforced Polymers (FRP), including Carbon Fiber-Reinforced Polymers (CFRP). In the first method, braces for steel structures and shear walls for seismic retrofitting of concrete buildings are common. However, in recent years, the use of steel braces for seismic retrofitting of concrete buildings has also become common.

The phenomenon of progressive failure can be studied by various analytical methods, ranging from very simple to very complex analyses, which are generally performed using finite element software that has the full capability to consider dynamic and nonlinear properties. It is clear that the phenomenon of progressive failure is a dynamic and nonlinear phenomenon, due to its occurrence in a very short time frame and the imposition of nonlinear deformations on the elements before failure. To prevent progressive failure due to unusual loads, the Canadian National Code has established requirements for the design of major elements, element connections, and methods for creating load transfer paths. The US General Services Administration (GSA) has developed a practical code for design to reduce the potential for progressive failure of federal buildings. The US Department of Defense (DOD) has also developed a code for design methods for existing DOD buildings.

Tavakoli and Kiakjori [1] investigated the progressive failure capacity of steel flexural frames using the alternative load path method. Then, they performed a nonlinear dynamic analysis to estimate the dynamic response of the frames in the explosion and sudden column removal scenario and carefully investigated the response of the structure for various column removal scenarios, with or without external blast loading. Their results showed that the potential for progressive failure is essentially dependent on the location of the column removal and that sudden column removal affects the overall response of the structure under external blast loading. Tavakoli and Kiakjori [1] reviewed the common methods for dynamic column removal. Then, they proposed a new method for dynamic column removal in frame structures. Using this method, the response of a 1-story steel flexural frame structure due to sudden column removal in different scenarios was evaluated, and, as a result, a good agreement between these two methods was obtained by comparing them with the common methods. Their research results also showed that sudden column removal produces larger responses compared to gradual column removal.

In a study by Sojoodi Tousrondani and Naghipour [2], they investigated progressive collapse in off-axis steel braced frames. In this study, three building models of 5, 10, and 15 floors were analyzed and designed in Etabs software in accordance with the seismic criteria of the 2800 Code. Then, according to the designed sections, one of the structural perimeter frames was modeled in the SAP2000 software to perform nonlinear dynamic analysis (Time history).

Then, according to the criteria mentioned in the UFC Code, 18 removal modes were considered for each model. Then, the dynamic response of the structure was examined for each removal mode. This was done for the force control members (Columns and braces) by examining the demand values (The force that is obtained in the structural members after the removal of the member in question) to the capacity, and for the displacement control members by examining the state of plastic joints. The results of the aforementioned study showed that the removal of corner columns creates a more critical state for the structure. Of course, this can be related to the arrangement of the braces because there is no bracing in the vicinity of the corner columns, and the beam-column connection is also hinged. By removing this column, the connection undergoes shear failure and causes progressive failure in all upper floors of the structure. However, by removing the middle columns, because the braces are located in the vicinity of these columns, the structure shows resistance to progressive failure.

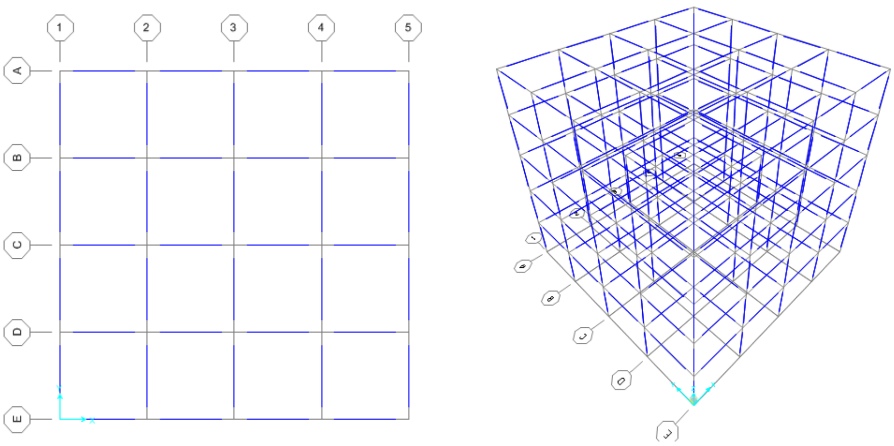
Therefore, it can be said that the appropriate arrangement of braces is very effective in the performance of frames against progressive collapse. Also, by examining the ratio of demand to member capacity, it was observed that the removal of members in the lower floors is more critical than in the upper floors of the structure, because this ratio is much larger in the lower floors than in the upper floors of the structure, so that for the upper floors, progressive collapse can be prevented by a small amount of reinforcement in the sections. Also, by examining the plastic joints formed in the connecting beams, it was observed that the probability of progressive failure increases with an increasing number of floors.

Changing the code or changing the use of existing structures is one of the main reasons for seismic retrofitting of existing structures. During an earthquake, unwanted explosions, or terrorist attacks, the retrofitted structure can also suffer from total failure due to progressive damage caused by the destruction of a column, and can no longer bear the load. In structures that are of high importance, whether in terms of security and law enforcement or health and treatment, these failures are unacceptable under any circumstances. The safety and health of the people using that structure are the most essential concerns for engineers, and in this regard, the difference in earthquakes in the near and far areas can also be an influential factor. For this reason, measures must be taken to prevent these failures in these structures.

2 | Research Method

2.1 | Introduction to the Initial Model

According to the research objective, a five-story structure with a square plan and four openings in two directions was used for analysis and design. The dimensions of the openings are considered to be 4 meters, and the height of the floors is considered to be three meters. The structure has been gravity loaded according to the sixth topic of the national regulations. For this purpose, the dead load of the floors is considered to be 500 and the roof is considered to be 600 kg/m². The live load of the floors and roof is selected to be 200 kg/m². Also, to consider the snow load, 150 kg/m² has been allocated to the roof floor. For the structural skeleton, concrete with a 28-day compressive strength of 21 MPa and reinforcements with a yield strength of 400 MPa for longitudinal reinforcements and 340 MPa for transverse reinforcements have been used. The structure was initially modeled as a finite element in SAP version 19 software, and the mentioned specifications were assigned to it. *Fig. 1* shows the plan and 3D model, and *Table 1* summarizes the relevant specifications [3].



a. b.

Fig. 1. Structural configuration; a. Plan and b. 3D view of the structure.

Table 1. Model gravity load specifications.

Geometric Specifications		
Span length	16	Meter
Number of spans	4	
Span dimensions	4	Meter
Number of floors	5	
Height of floors	3.2	Meter
Parameters	**Amount**	**Unit**
Dead load of floors	500	Kg/m²
Live load of floors	200	Kg/m²
Dead load of the roof	600	Kg/m²
Snow load	150	Kg/m²
Live load of the roof	200	Kg/m²

Table 1. Continued.

Material Specifications		
Parameter	**Quantity**	**Unit**
28-day compressive strength of concrete	2800000	Kg/m^2
Yield strength of rebar	34000000	Kg/m^2
Specific weight of concrete	2400	Kgf/m^3

After the initial modeling of the structure using the first edition of the 2800 standard, it was designed for one-tenth of the weight of the structure. In other words, the earthquake coefficient for this structure was considered equal to one-tenth, and after defining the seismic load, the structure was designed for this load. *Fig. 2* shows the DCR ratio of the structural elements for this loading. The results of the analysis are shown in the form of structural sections in *Table 2* for different floors [4]. In the design of the structure, an attempt has been made to consider the DCR ratio close to one by maintaining the relative displacement criterion, and the structural section that provides a DCR closer to one by maintaining the drift is selected as the final section [5].

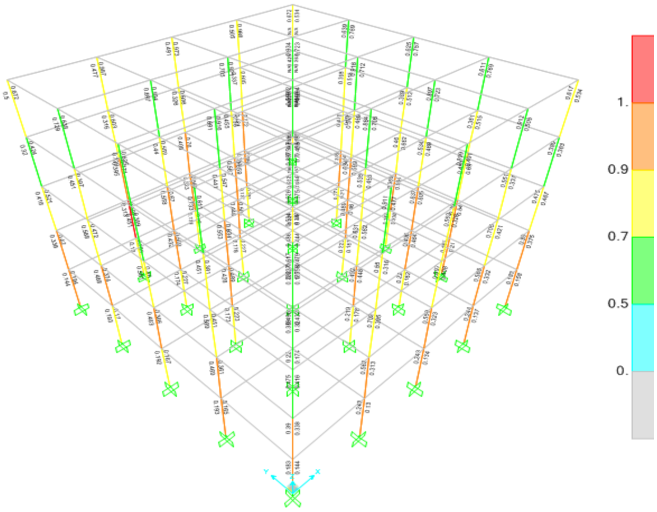


Fig. 2. Initial design results.

Table 2. Structural section specifications.

Armature	Dimensions	Floor	Element
Top: 8 d16 Bot: 5 d16	50x50	1	Beam
Top: 8 d16 Bot: 4d16	40x40	2	
Top: 7 d16 Bot: 4 d16	40x35	3	
Top: 6 d16 Bot: 3d16	40x35	4	
Top: 4 d16 Bot: 3 d16	35x35	5	
24 d16	65x65	1	Column
20 d16	50x50	2	
16 d16	45x45	3	
16 d16	45x45	4	
12 d16	40x40	5	

As previously stated, the structure must be retrofitted to a hospital for a change of use. To retrofit the structure, it must first be determined that the structure requires retrofitting. For this purpose, the structure is first checked for seismic loading resulting from the 2800 standard, fourth edition. It is assumed that the structure is located in a very high seismicity area in type 3 soil with a maximum acceleration of 0.35g. The system is supposed to have moderate ductility, so a behavior factor of 5 is considered for it. Considering the use of the structure, an important factor of 1.4 is considered for it according to the 2800 standard. The seismic loading characteristics are shown in *Table 3*.

Table 3. Seismic loading specifications according to the fourth edition of the 2800 standard.

Parameter	Value
Area	Very high seismicity
Soil type	III
Maximum design acceleration	0.35g
Structural system type	Concrete flexural frame with medium ductility
Behavior coefficient	5
Importance coefficient	1.4

According to the mentioned seismic parameters, the earthquake coefficient and K-factor for the considered structure have been recalculated. According to the combination of design loads of the reinforced concrete structure, they have been applied to the structure. The structure has been redesigned based on the new standard, which has been found to satisfy none of the minimum requirements of the code. For example, the DCR ratio is shown in *Fig. 3*, which shows that the structure is very weak against the applied load.

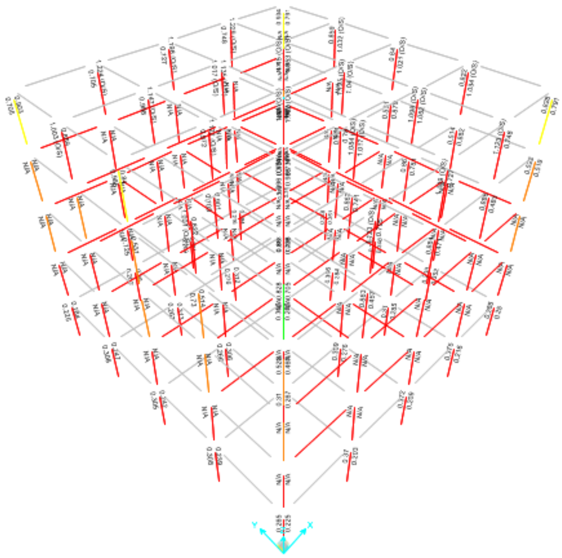


Fig. 3. Results from secondary design of the structure based on Standard 2800, 4th Edition.

The results show that the structure is weak to withstand seismic loads and is in dire need of reinforcement. According to the purpose of this research, the structure should be reinforced using a shear wall. The linear statics method and SAP software were used to reinforce the structure. Shear walls with a thickness of 30 cm were applied to the structure in different arrangements, and the DCR ratio of the structure was re-examined. In the shear wall arrangements in the structure, attention was paid to being symmetrical and being in the inner or outer frame. Four different arrangements were considered. The shear walls were placed symmetrically in the peripheral frame in the first arrangement, asymmetrically in the peripheral frame in the second arrangement, symmetrically in the inner frame in the third arrangement, and finally asymmetrically in the inner frame in the fourth arrangement. *Fig. 4* shows the different shear wall arrangements [5].

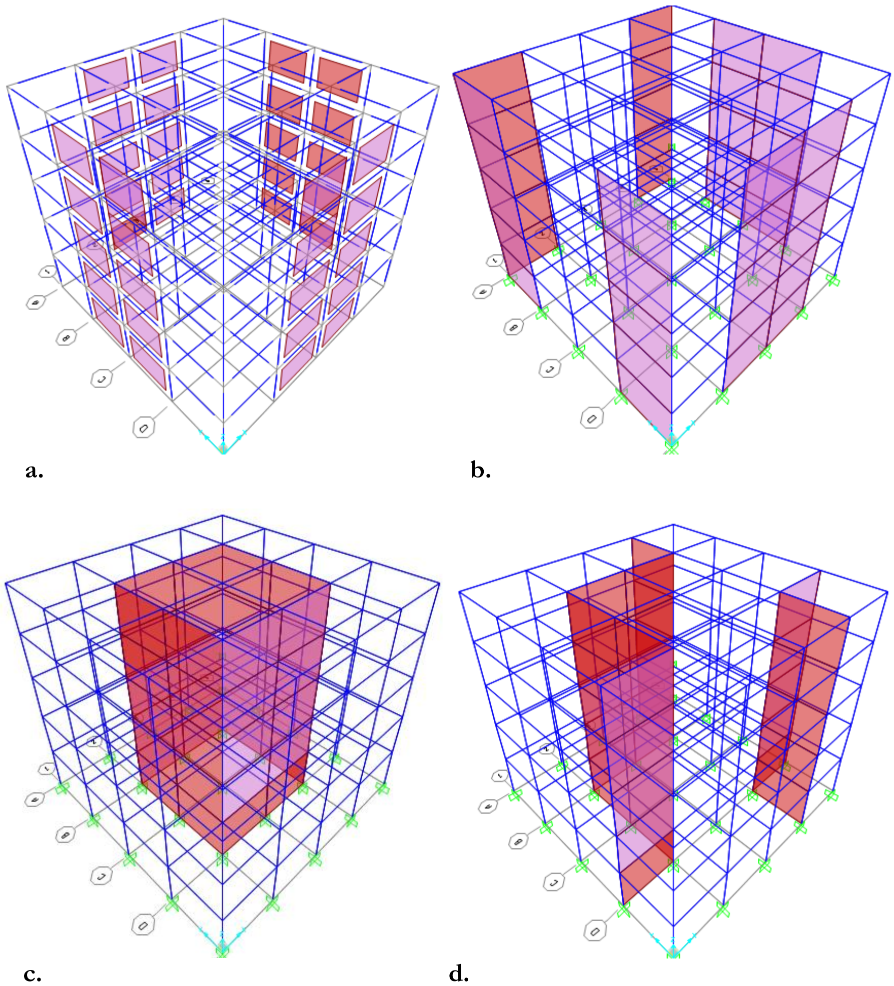


Fig. 4. Different arrangements of shear walls in the reinforced structure; a. Peripherally symmetrical, b. Peripherally asymmetrical, c. Internally symmetrical, d. Internally asymmetrical

Initially, the structures were redesigned with different shear wall arrangements, and their DCR ratios were checked. The results showed that adding shear walls to the structure caused the DCR ratios to decrease drastically. *Fig. 5* shows an example of the DCR values in the column members of the structure.

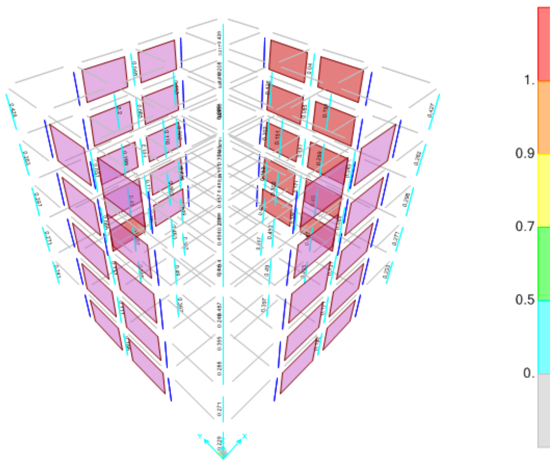


Fig. 5. DCR ratio in the retrofitted structure.

After it was determined that the shear wall was sufficient to reduce the DCR values and to strengthen the structure, to evaluate the best shear wall arrangement on the performance of the strengthened structure, the structures were modeled nonlinearly in the Perform software and subjected to pushover analysis.

3 | Findings

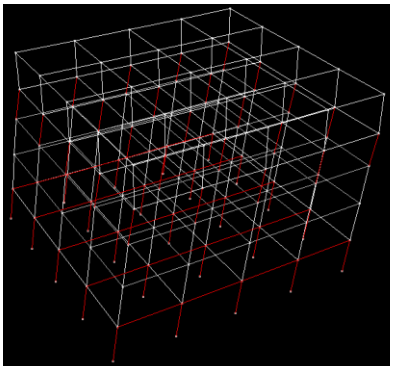
3.1 | Evaluation of the Best Shear Wall Placement Position in the Strengthened Structure

As presented in the third chapter, four placement positions have been selected for the shear walls. The walls have been placed symmetrically and asymmetrically in two spans of the middle frame and the peripheral frame. Initial studies have shown that the use of shear walls in all these positions is suitable for strengthening, but which one can be used better will be examined in this section [5].

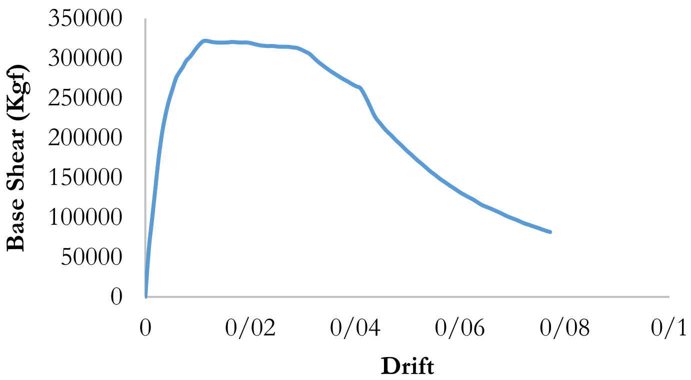
In this section, using nonlinear static analysis and structural pushover curves, the capacities of the structure against lateral load are extracted for each model. By comparing them, the structure that shows the highest capacity and ductility is selected as the desired structure in this study [4].

To perform nonlinear static analysis and evaluate the behavior of structures, the initial target drift in the Perform software is set to a large value of 0.1 so that the capacity of the structure can also be examined under high drifts. The structures are pushed under a triangular load pattern, and the pushover curves are presented and compared as a cut on the structure relative to the overall drift of its structure as pushover curves [11].

The initial structure that was not reinforced was subjected to pushover analysis. *Fig. 6* shows a schematic diagram of the pushover analysis of the structure and its capacity curves.



a.



b.

Fig. 6. Pushover analysis in the initial structure; A. Schematic diagram in the final step of the analysis, B. Capacity curve.

As the capacity curve of the structure shows, the structure reached its maximum shear at a drift of 0.011 and then faced a capacity drop, and this resistance drop increased sharply at a drift of 0.04 onwards. This curve shows that the structure exhibits a maximum capacity force of 32811 kg against lateral load [9].

Figs. 7 and *8* show the schematic figures related to the pushover analysis in the retrofitted models and their capacity curves, respectively.

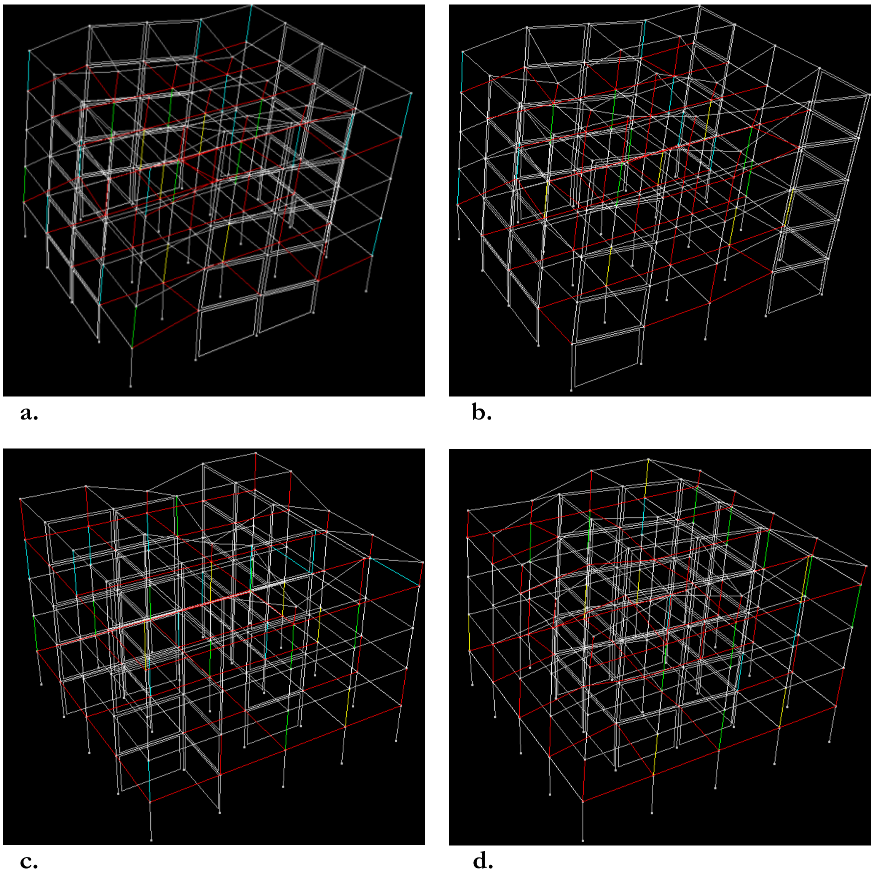


Fig. 7. Schematic diagram of pushover analysis in reinforced structures with shear walls with the following arrangements: a. symmetrical outer frame, b. asymmetrical outer frame, c. symmetrical middle frame, d. asymmetrical middle frame.

Red color is the life safety performance level, yellow color is 80% life safety period, green color is 60%, and blue color is 40% life safety period.

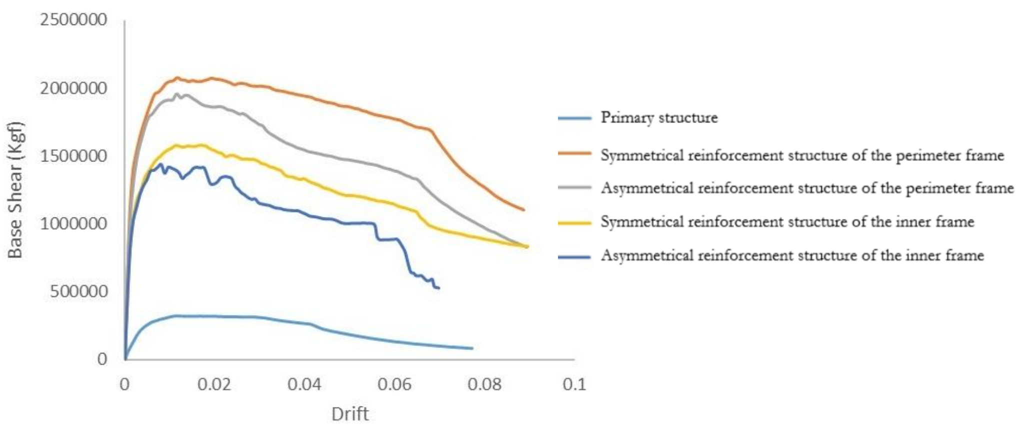


Fig. 8. Capacity curves of the structures considered in this research.

In *Fig. 8*, cases 0 to 4 represent the initial structures, respectively, strengthened by a symmetrical shear wall in the peripheral frame, an asymmetrical shear wall in the peripheral frame, a symmetrical shear wall in the inner frame, and an asymmetrical shear wall in the inner frame. The capacity curves of the strengthened structures show that in a structure strengthened by symmetrically arranged shear walls in the peripheral frames, the capacity curve of the structure is higher than that of other strengthened structures, so it can be said that this structure shows the best performance against lateral load.

Therefore, by choosing a structure strengthened with symmetrically arranged shear walls in the peripheral frame as the target structure of this research, the seismic progressive failure scenarios in this structure will be investigated.

4 | Conclusion

In this study, the seismic performance of improved structures against progressive failure under the influence of near-field and far-field earthquakes was investigated and compared. The phenomenon of progressive failure has received widespread attention as one of the most essential mechanisms of structural collapse in recent decades, because its occurrence is usually caused by the sudden removal of one or more structural members and can lead to the destruction of the entire structure. The results of the studies showed that progressive failure, especially in the face of unexpected seismic excitations, is a serious threat to the safety of structures [10].

The analysis methods used in this study included nonlinear static and dynamic analyses, which have shown higher accuracy in simulating the real behavior of structures against possible failure scenarios. Among the improvement methods studied, the use of steel braces and the reinforcement of inefficient elements with various coatings such as FRP and CFRP were identified as the most effective solutions for increasing the strength and load redistribution capacity of structures.

Previous studies, including those by Tavakoli and Kiakjori, have also emphasized that the location of column removal and its method (sudden or gradual) have a direct and significant impact on the severity and extent of progressive damage. The findings also show that in scenarios of sudden removal of the load-bearing member, the dynamic response of the structure will be much more critical than in scenarios of gradual removal.

Finally, given the importance of the issue and the vital role of structural safety, attention to progressive damage-resistant design and the use of modern seismic retrofit methods should be emphasized as an integral part of the structural design and assessment process, especially in areas at risk of near and far field earthquakes.

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