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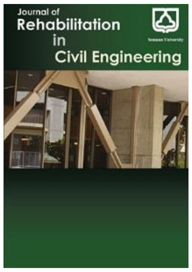

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Evaluation of Intermediate Reinforced Concrete Moment Frame Subjected to Truck Collision

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ABSTRACT

In this study, the progressive collapse of reinforced concrete structures due to vehicle collision to the columns of the ground floor was modeled and examined. For this purpose, a four-story reinforced concrete building with the intermediate moment frame system was designed using ETABS software followed by the simulation of impact loading by SAP2000 software. Performing non-linear time history dynamic analysis, the critical forces required to the column failure were determined via trial and error by considering different live load contribution. Then, the corresponding critical velocities for 4, 8, and 12 ton vehicles were determined. Finally, the progressive collapse of the building was examined by the sudden removal of the column. The results showed that by increasing the percentage of live load contribution, the force and critical velocity for the instability and damage of the column will decrease. Furthermore, comparing the perimeter and corner columns showed that the corner columns are the most critical columns for occurrence of the progressive collapse. In addition, during the assessment of the progressive collapse, it was found that the number of damaged springs in the corner column removal scenario is less than that of the perimeter column removal scenario.

1. Introduction

1.1. Background

Structural safety has always been a key point for engineers in civil engineering projects. One of the mechanisms that can lead to the structure collapse and has attracted much attention in the last decade is the phenomenon of progressive collapse in which one or more members of the structure

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suddenly collapse due to accident, terrorist attacks, etc. Consequently, the building falls apart progressively. Over the last two decades, many important economical, governmental, industrial, and residential buildings have been the target of terrorist attacks. Some of the most important events that drew attentions to the progressive collapse were the accidental gas explosion at 18^{th} floor of Ronan Point in 1968, a terrorist attack on the Murrah Federal Building in 1995, and the attack on the World Trade Center in 2001.

Progressive collapse is a condition in which the occurrence of a local failure in one member of the structure leads to the breakdown of its adjacent members and further collapse of the building[1].

Various factors can cause local damages, and eventually start a progressive collapse. Some of these important factors are explosion in the structure or the intense collision to its peripheral columns. In other words, damage in one or more key elements in the structure leads to progressive collapse of the structure.

Some structures may be subjected to impact forces such as sudden collision, instantaneous brake of cranes, explosion, hits caused by the stumbling of heavy machines, etc. Given the fact that these forces have dynamic nature (variable in time), applying a proper analysis method to design these buildings has particular importance [2].

In most cases, the effect of vehicle transverse impact is combined with the effects of axial gravity load on the column or wall of the higher floors. Central compressive force complicates the impact problem because it reduces the stiffness and strength of the column and in turn, increases the geometric and dynamic instabilities of the column.

1.2. Literature review

Several studies have been conducted on impact and progressive collapse. Sasani et al. [3] experimentally evaluated the behavior of the 10-story building of the Arkansas University dormitory which was supposed to be destroyed in a controlled progressive collapse. In that study, the effect of instantaneous removal of the middle column of the first floor was examined. According to the recorded displacements, the results indicated a good structural resistance due to sudden removal of the middle column. Sasani et al. [4] also examined the behavior of the side continuous 3/8 scale-shaped beam in a concrete frame after sudden removing of the column attached to it. They investigated the potential progressive collapse rate in the structure and the dynamic load distribution after the column removal.

The effects of fast loading and the percentage of longitudinal reinforcement on the failure mode under lateral impact loading have been investigated by Wang et al. [5] and Anil et al. [6]. The results showed that the lateral displacement in the middle of the span and the bottom moment increase as the final load increases. The amplitude of displacements created in the impacted members also increases at higher loading speeds. Furthermore, the interval between the beams has a major role in the failure mode; the greater the distance between the beams, the column has higher tendency to shear failure, otherwise flexural failure occurs. In another research, Yimaz et al. showed that axial load of the column, the impact energy, and the percentage of the shear reinforcement bar have significant effect on the performance of the rectangular reinforced concrete columns

[7]. Tin et al. investigated the dynamic responses of pre-cast concrete columns with non-adhesive pre-tensioned cables under the impact of vehicle collision and found that by increasing the pre-tensioning force, the relative shear slip and lateral displacement are decreased significantly throughout the column [8].

In an experiment conducted on a laboratory scale bus body, it was observed that when the axial pressure was applied in the form of static charge, the plastic deformation was similar to the Euler's column, while in the dynamic impact of the front, the plastic deformation mainly occurs in the place of collision and other parts remain unaffected [9]. Jones and Wierzbicki studied the plastic failure of a free beam with a cross-section at loads exposed to rectangular or triangular, and showed the mechanism of free beam collapse by forming fixed plastic hinge in the center of the beam. According to their results, only 25% of the external energy is absorbed by the plastic deformation[10]. Yang, who examined the behavior of a free beam exposed to the step-central load, observed that the absorption of plastic energy is always less than 33% of the incoming energy [11]. Taromsari et al. investigated the progressive collapse in reinforced concrete moment frames using alternative path load analysis method and the formation of chain performance in order to transfer the overload generated by the removal of key element. It was shown that the removal of corner column is more critical than removal of other columns. In addition, columns in higher floors are more important than those in lower floors and a disorder in the structure may increase the potential of progressive collapse[12].

Torabi and Broujerdian examined the effect of impact in a 4-story steel building and showed that column removal according to the proposed methods is not a good approximation of the possible damage to the structure due to progressive collapse resulting from impact or other sudden loads [13].

Broujerdian et al. also showed that in steel moment frame systems, mass, velocity, and the distance of the collision point from the ground have a significant impact on the collapse of the building. They found that the impacted column is not only damaged during collision; yet, all members of the structure are affected by this dynamic load and most of them enter the plastic area [14].

Most of the studies on the effect of impact are assigned to bridges. Zhou et al. parametrically investigated the effects of impact velocity and mass as well as the strength of concrete and steel on the bridge pier during vehicle collision. The results showed that the force and deformation of the bridge pier depend on the impact energy. The maximum force and maximum displacement caused by impact also grow by increasing the impact energy. It was also observed that the impact force, impact duration, and deformation of the bridge pier are not sensitive to concrete strength, but the deformation is largely influenced by the steel strength [15]. Zhou et al. developed and validated a method for assessing the bridge piers damages caused by vehicle collision. They used numerical models to analyze the failure process and different impact damages to bridge piers [16]. Tin et al. presented a method for designing bridge piers resistant to the burden of a vehicle collision [17]. They also investigated the performance of conventional monolithic columns (CMC) and precast concrete segmental bridge columns (PCSBCs) and showed that the failure in PCSBCs is either due to the high pressure at both ends of the column or a combination of bending and shear failure, whereas in CMC, failure occurs mainly due to the distribution of flexural cracks, shear cracks, and shear punches in several sections [18], [19].

The vertical diaphragm in progressive collapse can also be affected. In most analyses studies, the effect of floor has been neglected. Broujerdian et al. [20] investigated the effect of floor on progressive collapse. Results of this research showed that considering the roof in analysis can be reduce the damage on progressive collapse.

1.3. Aims and scopes

In this research, the progressive collapse behavior of the building due to the impact of various types of vehicles to the ground columns was studied. This study is divided into two parts. Firstly, the behavior of the structure under the impact of vehicle was investigated. The impact force resulting from the collision as well as the critical force and velocity required for the instability of the column were also calculated. Secondly, the progressive collapse in the structure in a column removal approach along with three scenarios of column removal was examined. Finally a comparison is made between the results of these two parts.

2. Materials and methods

2.1. Research method

Initially, the software verification of the numerical model via experimental model was performed. Then, the structural model is designed in accordance with the existing codes. In this research, two types of impact analysis and column removal (progressive collapse) have been considered. The aim of the impact analysis is to determine the collapse forces and critical velocities of collapse in the structure. The column removal analysis is performed in accordance with the proposed codes of procedure without considering the cause of the column removal collapse.

2.2. Verification of numerical model

In order to verify the numerical modeling process, the results of an experimental test conducted by Vecchio and Emara [20] on a one-bay two-story reinforced concrete frame is used as benchmark. Fig.1a shows the geometrical properties of the frame. The compressive strength of concrete was 30 MPa and the yield stress, ultimate stress, and elastic modulus of the reinforcing steel bars were 418 MPa, 596 MPa, and 192500 MPa, respectively.

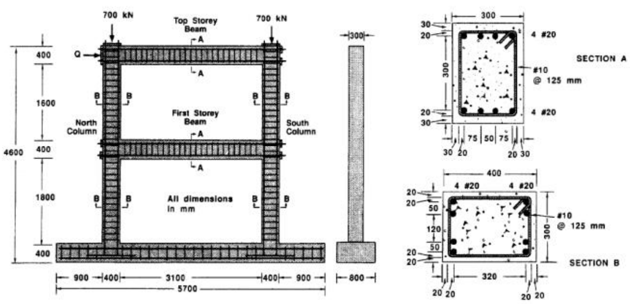
A nonlinear static analysis is done using SAP2000 software. The bases of the columns are modeled as fixed support. Two gravitational loads of 700 kN were applied on top of the columns (roof level). The lateral loading at roof level is applied in a displacement-control manner. The plastic hinges are defined at both ends of all beams and columns. The plastic hinges of beams and columns are considered at 0.15 and 0.05 of the element length from the axis, respectively.

The resulting capacity curve for the numerical model is compared to the experimental one in Fig.1b. As seen in this figure, the accuracy of the model in terms of the initial stiffness and the energy absorption (the area under the load-deformation curve) is good. It must be noted that, the difference between yielding regimes of numerical and experimental models is due to using the concentrated plasticity approach (bilinear plastic hinges) in the numerical modeling. In fact, the smooth curve of the experimental model is due to distributed plasticity of the constituent materials. Another source of approximation is considering the ideal fixed base for the columns in the numerical model.

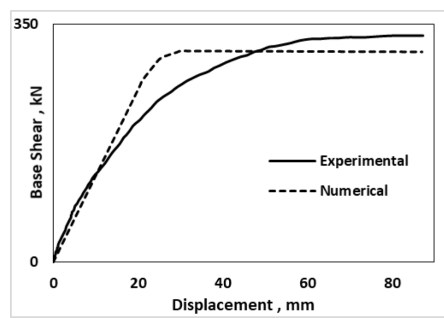
Therefore, SAP2000 software is capable of modeling the nonlinear behavior of reinforced concrete frames with relatively good accuracy under lateral loads.

2.3. Design process of numerical models

The building under study is a residential four-story reinforced concrete building with the same plan in each floor. The span intervals along the X-axis and Y-axis are 5 m and 6 m, respectively, and the height of all the floors is 3.2 m (Fig. 2).

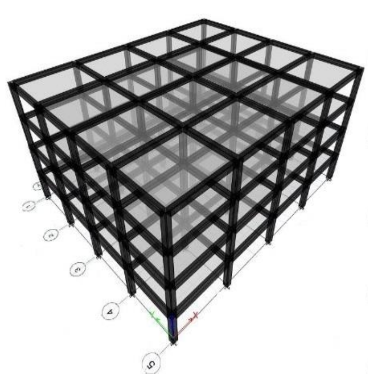


(a)

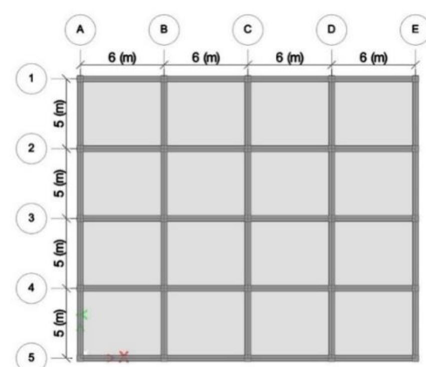


(b)

Fig. 1. (a) Details of experimental reinforced concrete frame, and (b) comparison of numerical and experimental pushover results [21].



(a) 3D view.



(b) Floor plan.

Fig. 2. Studied prototype structure.

The standard design of the structure is made using ETABS 2016 software [22] in accordance with ACI $318-14$ code [23]. The earthquake loads are calculated based on the assumption that the structure is located in Tehran city with the bedrock acceleration of $0.35\,g$ and the shear wave velocity of $375–750\ m/s$ [24], [25].

The roof considered in the modeling is the two-way slab. Due to the fact that cracking occurs in the concrete slabs and the slab thickness is less than the height of the beam, the flexural stiffness of the slabs is usually neglected. Therefore, the slabs are considered as membrane elements in analyses and design.

2.3.1. Material properties

The characteristics of the concrete and reinforcement bars used in this project are shown in Tables 1 and 2, respectively.

Table 1. Properties of the concrete.

Properties	Value
Volume weight	2500 Kg/m^3
Modulus of elasticity	$2e5\ MPa$
Poisson's ratio	0.2
Compressive Strength	250 MPa

Table 2. Propertiesof the reinforcement bar.

Properties	Value
Volume weight	7850 Kg/m^3
Modulus of elasticity	$2e6\ MPa$
Poisson's ratio	0.3
yield stress	400 MPa
ultimate stress	600 MPa

2.3.2. Loading

The values of dead loads and live loads of typical floors and roof along with the load of internal and perimeter partition are presented in Table 3.

Table 3. The gravity loading of floors and roof.

Loading	Floor	Roof
Uniform Dead (Kg/m^2)	550	550
Uniform Live (Kg/m^2)	200	150
Partition (Kg/m^2)	100	–
Dead walls (Kg/m)	550	250

2.3.3. Design results

The results of the beam sections and column are shown in Tables 4 to 6. For the modeling of beams in different parts such as bearings and the middle of spans, reinforcement is used with respect to the required design. In addition, the shear reinforcement of the beam and column sections are designed according to requirements of ACI318-14 code. Due to the fact that the span of beams are different in X and Y directions, the beam sections in these two directions are different.

Table 4. Designed beam sections in X direction.

Floor	Beam size	Reinforcing in the end of element		Reinforcing in the middle of element	
		Top bars	Bot. bars	Top bars	Bot. bars
4	35 × 40	3T16+3T20	3T18	3T16	4T18
3	35 × 40	3T16+5T20	4T18	3T16	4T18
2	40 × 40	3T16+4T25	4T18	3T16	4T18
1	40 × 40	3T16+4T25	4T18	3T16	4T18

Table 5. Designed Beam Sections in Y direction.

Floor	Beam size	Reinforcing in the end of element		Reinforcing in the middle of element	
		Top bars	Bot. bars	Top bars	Bot. bars
4	35×40	3T16+2T18	3T16	3T16	3T16
3	35×40	3T16+4T18	3T18	3T16	3T18
2	40×40	3T16+3T25	5T18	3T16	3T18
1	40×40	3T16+3T25	5T18	3T16	3T18

Table 6. Designed columns section.

Floor	Column size	Reinforcing bars
4	40 × 40	8T20
3	40 × 40	8T20
2	45 × 45	8T22
1	45 × 45	12T25

2.4. Impact and column removal analysis

First, the building was designed using ETABS software [22] in accordance with valid codes. Then, the impact and column removal analyses were performed using SAP2000 software [26]. It must be noted that the analyses were done in non-linear dynamic mode.

2.4.1. Impact loading procedure

The impact load caused by the vehicle collision was simulated in SAP2000 software. At first, a node was assigned in the impact point. Then, the impact force along X direction to the point applied. Finally, to apply the time history, the gravity and impact loads were set for non-linear time history dynamic analysis. The flowchart in Fig. 3 describes the steps for defining and performing of impact loading.

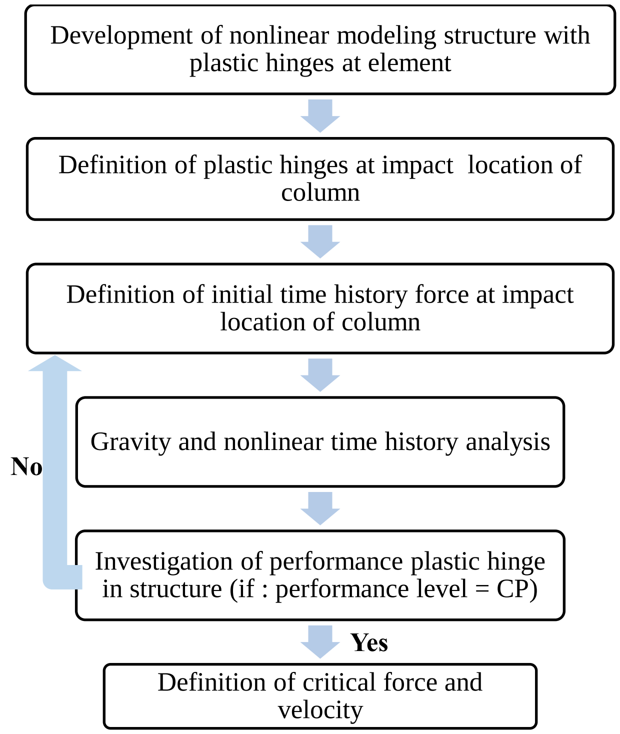


Fig. 3. Impact loading procedure in SAP2000 software.

The first part, the impact force calculated in SAP2000. The second part performed progressive collapse analysis in SAP2000 software.

2.4.1.1. Impact Loading

Unlike the ABAQUS or LS-DYNA software, SAP2000 does not have the capability of modeling the impact wave. As a result, it does not simulate the collision of an object to the target. So, in SAP2000 software, it is not possible to allocate velocity and mass separately to the object in order to simulate the collision of the object to the target [26]. Therefore, force was used to model the collision in SAP2000. In this case, the exact point of the collision is assigned to the element under the impact. Then, the time history force is applied to point.

In the following, to calculate the impact force which is influenced by velocity and mass parameters will be discussed. Then, the impact force is defined and applied in SAP2000 software. The results of Varat and Husher [27] were used to plot the diagram of the force due to the collision of vehicle versus time.

The nonlinear numerical models of vehicles for the reconstruction of the impact are complex. In the numerical simulation process, time history force generated from collision experiments in full-scale are used. However, the response of the accident depends on the vehicle form, stiffness, velocity, and mass, as well as collision mode, dynamic crushing, etc. Several models have been proposed for impact simulation. The sine, square, and triangular shapes are standard forms. These forms are widely used to indicate the effects of impact on the front. As shown in Fig. 4, in the current research, triangular diagram is used to model impact load on the column similar to Reference [27].

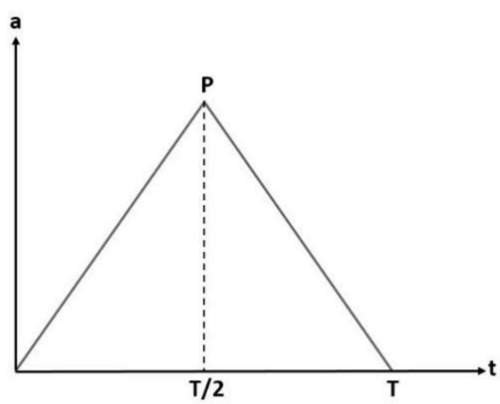


Fig. 4. Triangular acceleration-time loading.

According to Fig. 4, acceleration was determined by the following equations:

$$a = \begin{bmatrix} \frac{2P}{T}.t & 0 \le t \le \frac{T}{2} \\ \frac{-2P}{T}.t + 2P & \frac{T}{2} < t \le T \end{bmatrix} \quad (1)$$

$$P = \frac{2\Delta V}{T} \quad (2)$$

where a is the acceleration at any given time, P is the maximum acceleration, ΔV is the difference between the velocity before and after the collision, t is the time, and T is the duration of the impact, which is 0.1 s in this research [27].

Thus, the maximum acceleration according to equation 3 is:

$$P = 20V_0 \quad (3)$$

Finally, Newton's second law is used to obtain the impact force:

$$F = m.a \quad (4)$$

In other words, the acceleration at any time, a, is multiplied by the mass, m, and finally, the force-time curve. By substituting Equation 3 in Equation 4, the maximum force is:

$$F_{max} = m.P = 20mV_0 \quad (5)$$

Based on static rules, if three hinges are formed in a member (beam or column), that member will be unstable. Therefore, in this research, the critical force has been defined for the column instability that formed 3 hinged reach to CP (red color hinge in software) in column under impact. The critical force was determined by trial and error.

Now, if the critical force obtained by trial and error in SAP2000 software is set to F_{max}, the critical velocity will be obtained from equation (6)

$$\text{if}: F_{critical} = F_{max} \rightarrow V_0 = \frac{F_{critical}}{20m} \quad (6)$$

2.4.2. Column removal analysis

In order to reduce the collapse probability, two methods can be used: reducing the probability of occurrence of abnormal loading, and designing the structure in such a way that the probability of collapse is decreased to a reasonable level. The former does not appear to be efficient since there are many abnormal loads that are beyond the control of the designer. However, the latter seems to be a more logical approach because it is controllable by the designer. Various methods are proposed to reduce the risk of progressive collapse in structures. The most important and common methods are: (1) event control, (2) indirect design, and (3) direct design. The event control method does not increase the structure resistance to progressive collapse. On the other hand, it depends on people which is outside the control of the designer, so it is used less. However, indirect design and direct design method are controllable by the designer. In these two methods, the structures must be designed in such a way that when the local damage occurs, regardless of the cause of the failure, there is a reasonable probability that the damage should not extend to other elements of the structure.

The direct design method during design process for progressive collapse emphasizes explicitly on the total strength of the structure. Two direct design methods are the specific local resistance method and the alternate path method.

In this research, the alternate path method is used to evaluate progressive collapse. Subsequently, the alternate load path, and then the basics of codes and software are explained briefly.

2.4.2.1. Alternate path method

This is the most prevalent method to design and evaluate against progressive collapse. The aim of this method is to prevent progressive collapse by providing alternate load paths. However, local damage to the structure can occur. In this method, first the structure is subjected to gravitational loads. Then, various scenarios of member removal are examined. Finally, the resistance of the structure to progressive collapse is evaluated. It is noteworthy that, the real cause of damaging is not considered in this method. This is an important feature of the method because it can be responsive to any event that may lead to the damage of the bearing member.

2.4.2.2. Dynamic modeling of column removal

The combination of loading used in non-linear dynamic analysis based on the General Services Administration (GSA) code is:

$$G = 1.2D + 0.5L \tag{7}$$

In this research, the building was examined for 0, 50, and 100% live load contribution under non-linear dynamic analysis. The load combinations are shown in equations (8) to (10).

- zero percent live load contribution

$$\mathrm{G = 1.2D} \tag{8}$$

- 50 percent live load contribution

$$\mathrm{G = 1.2D + 0.25L} \tag{9}$$

- 100 percent live load contribution

$$\mathrm{G = 1.2D + 0.5L} \tag{10}$$

where D and L are the dead and live loads, respectively.

2.4.2.3. Steps of column removal

First, the structure was loaded for each live loading contribution according to equations (8) to (10). Then, the internal forces (i.e. shear (V), moment (M), and axial (P) forces) of the column were removed before column removal. Then, the removal column is replaced by these internal forces in another model. In order to simulate the sudden removal of the column, as shown in Fig. 5, the gravity loads and internal forces of the member were simultaneously and gradually applied to the structure for 5 s. In the time 5, the total of gravity loads and internal forces applied to the structure. After 2 s, the member load was suddenly removed to simulate the progressive collapse, while the gravity loads remained on the structure [28].

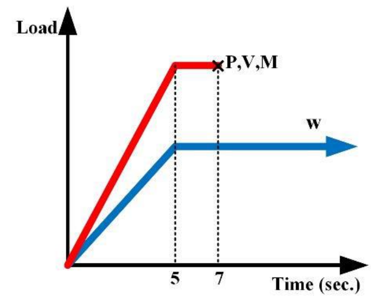
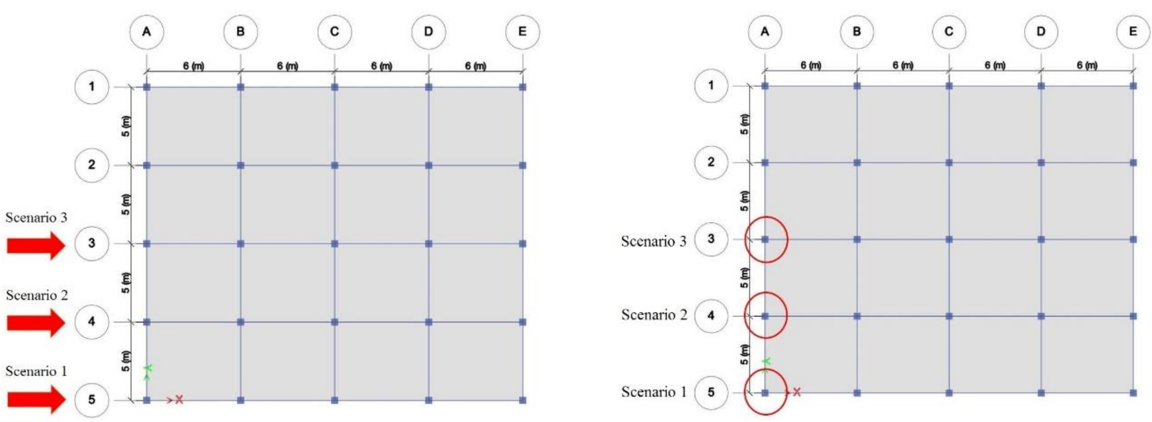


Fig. 5. applying and removing the member load [28].

2.4.3. The studied scenarios

The effects of direct collision as well as the sudden removal of columns are examined in this part. According to Fig. 6, three scenarios of impact to the columns of the ground floor (Fig. 6a) along with three scenarios of sudden removal of column (Fig. 6b) have been studied. The collision height is $1.2\ m$ above the ground. It must be noted that the nonlinear hinges are considered at both ends of the hit column as well as at the collision location. To study the effect of collision, three vehicles ($4, 8,$ and $12\ ton$) with three live load contributions ($0, 50,$ and 100%) are considered.



(a) Impact scenario.

(b) Column removal scenario.

Fig.6. The studied scenarios.

2.4.4. Specifications of nonlinear hinges

In SAP2000 software, the reinforcement should be specified in the concrete members. In columns and beams, the value of reinforcement is determined via linear analysis.

In order to model the non-linear behavior of the members, GSA [29] suggests non-linear hinges according to ASCE 41-17 code for concrete members [24]. In the suggested hinges, the results of the tests are often given in terms of the rotation of the line connecting both ends of the members. These properties are defined by the force-deformation curve shown in Fig. 7. In this figure, the values of a, b, and c have been determined according to ASCE 41-17 code. The slope of strain hardening part (BC) is considered to be 3% of the slope of the elastic section (AB).

Attention to ASCE41-17 code, to assess the quality of the structure elements has been progressed towards the expected performance levels of structure. The performance level represents the maximum expected damage of the structure. If its rate is higher than the determined value, the structure will exceed another performance level. All the structural and non-structural members are effective in defining the performance level. The performance levels are divided into three categories: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP).

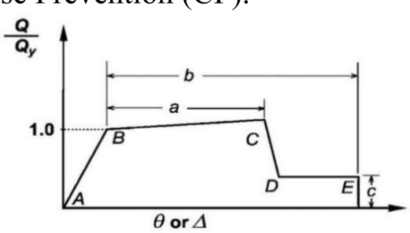


Fig. 7. The force-deformation curve generalized for members and components of reinforced concrete [24].

Considering SAP2000 software capabilities, the values of a, b, c, IO, LS, and CP parameters are defined for modeling the nonlinear behavior of the members. These values are determined with respect to ASCE 41-17 code [24] for structural members.

Based on different codes, the hinge formation site in beams and columns under lateral loads occurs usually at their two ends. As suggested by GSA and seismic codes, hinges should be defined in order to examine the effect of abnormal loads. In this research, hinges for beams and columns are defined in accordance with Fig. 8. In addition to the plastic hinge is defined and assigned at the place of impact.

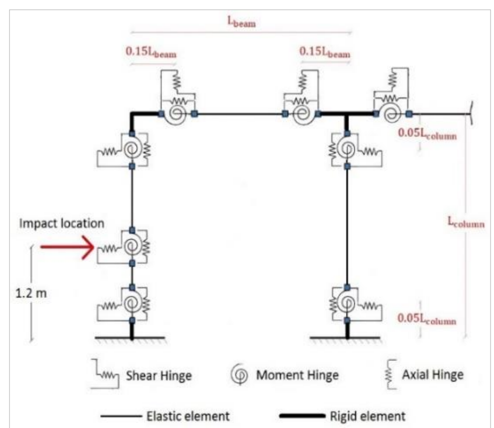


Fig. 8. SAP 2000 model of the frame and plastic hinges structure.

3. Results and discussion

In each analysis, the critical forces are determined for the instability of the entire column using trial and error and, then the critical velocities are calculated for different vehicle masses and other equations mentioned in the previous section. Finally, the progressive collapse by removal of column was evaluated using performance level criteria.

3.1. Impact analysis

In this section, the vehicle hit the columns of the ground floor at the height of 1.2 m from the ground, and the critical force for the column failure was determined by trial and error for each live load contribution. Then, critical forces were calculated for various vehicle masses.

3.1.1. Results of scenario 1

The results obtained for the different live load contributions and vehicle weights are shown in Table 7.

Table 7. The critical forces and velocities in scenario 1.

Live load participation (%)	Critical force (KN)	Critical velocity (km/hr.)		
		4 (ton)	8 (ton)	12 (ton)
0	1540	69.3	34.65	23.1
50	1480	66.6	33.3	22.2
100	1400	63	31.5	21

In Table 7, for example, for the live load contribution of 0%, the critical force for column failure by trial and error and using software was 1540 kN. Therefore, the critical velocities were calculated for weights of 4, 8, and 12 ton based on Equation (6). For example, for a 4-ton vehicle, the minimum speed for column failure was 69.3 km/h.

3.1.2. Results of scenario 2

The critical force for the live load contribution of 0% using the software by trial and error was 1610 kN. Therefore, the critical velocities were calculated for weights of 4, 8, and 12 ton based on Equation (6). For example, for a 4-tonne vehicle, the minimum speed for column failure was 72.45 km/h. The results for different live load contributions and vehicle weights are shown in Table 8.

Table 8. The critical forces and velocities in scenario 2.

Live load participation (%)	Critical force (KN)	Critical velocity (km/hr.)		
		4 (ton)	8 (ton)	12 (ton)
0	1610	72.45	36.23	24.15
50	1555	69.98	34.99	23.33
100	1500	67.5	33.75	22.5

3.1.3. Results of scenario 3

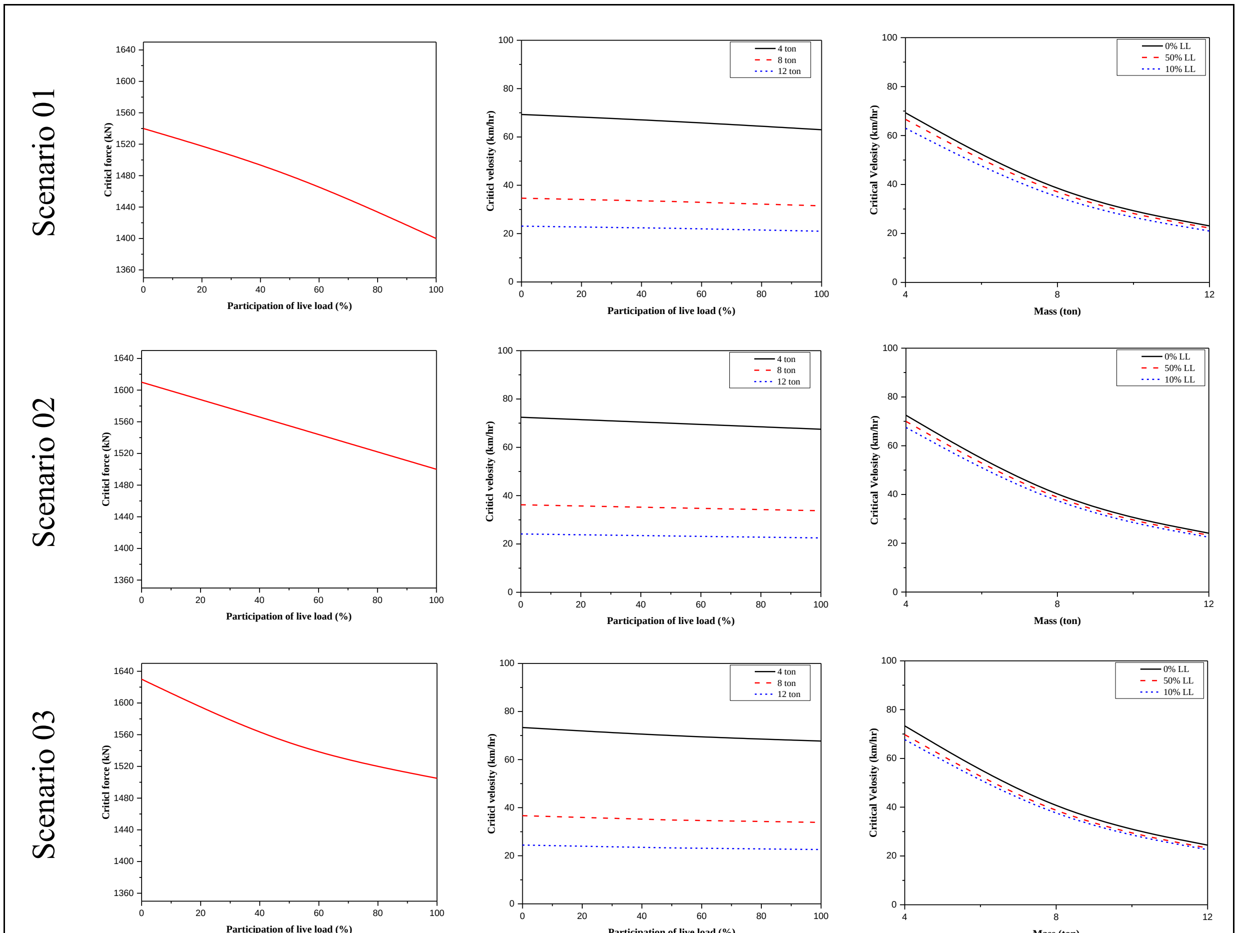
The critical force for the live load contribution 0% using the software by trial and error was 1630 kN. Therefore, the critical velocities were calculated for weights of 4, 8, and 12 ton based on Equation (6). For example, for a 4-tonne vehicle, the minimum speed for column failure was 73.35 km/h. The results for different live load contributions and vehicle weights are shown in Table 9.

Table 9. The critical forces and velocities in scenario 3.

Live load participation (%)	Critical force (KN)	Critical velocity (km/hr.)		
		4 (ton)	8 (ton)	12 (ton)
0	1630	73.35	36.68	24.45
50	1550	69.75	34.88	23.25
100	1505	67.725	33.86	22.58

3.1.4. Discussion of impact effect

Fig. 9 indicates that the more live load contribution, the less critical is the force for column failure (Fig. 9a). On the other hand, the more live load contribution, the lower is the critical velocity for column failure (Fig. 9b). For each live load contribution, the higher the mass of the vehicle, the lower is the critical velocity for column failure (Fig. 9c).



(a) the critical force-live Load contribution. (b) the critical velocity-live load contribution. (c) the critical velocity-vehicle mass.

Fig. 9. Comparative results of impact scenarios.

Fig. 10 shows that in each live load contribution, the critical force for the corner columns (scenario 1) is less than that for perimeter columns (scenario 2 and scenario 3). Furthermore, Fig. 11 shows that for each live load contribution and vehicle mass, the critical velocity for corner columns (scenario 1) is lower than that for the perimeter columns (scenario 2 and scenario 3). The reason for this fact could be that the alternate load paths available for the perimeter column is more than that for the corner column. Therefore, the corner column is more critical in any live load contribution.

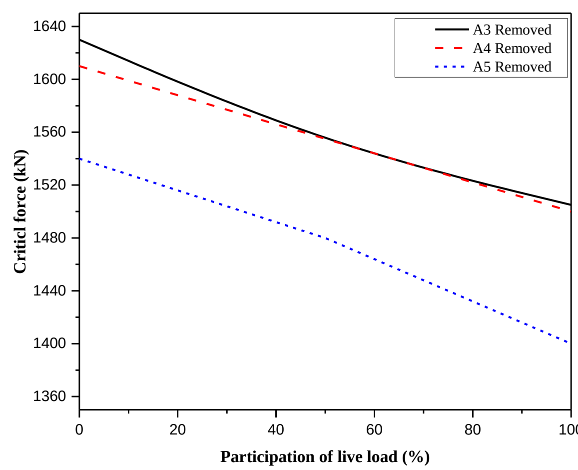


Fig. 10. Comparison of critical force-live load contribution of column.

3.2. Column removal analysis

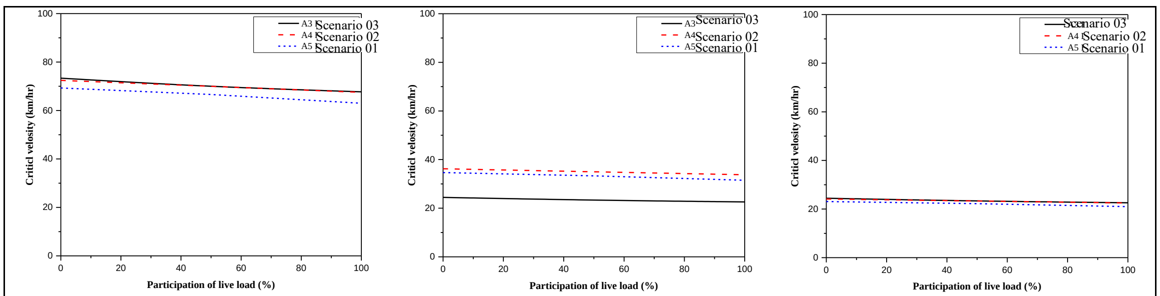
The deformed situation and the way of hinge formation in the removal scenarios of 1, 2 and 3 are shown in Fig. 12. Considering that the figures of plastic hinge formation and failure of members in different live load contributions were similar to each other, the redundant of figure was prevented.

Fig. 12 indicates that for each live load contribution, the building was collapsed after the removal of column in the ground floor. The adjacent and located top of column removal beams, in the direction x and y, has been entered the plastic phase and their performance has exceeded the CP and collapsed. But almost no single element has experienced plastic deformation. In Table 10, the functional columns of the structure are represented by different column removal scenarios for different live load contributions.

Table 10. Performance level of the structure.

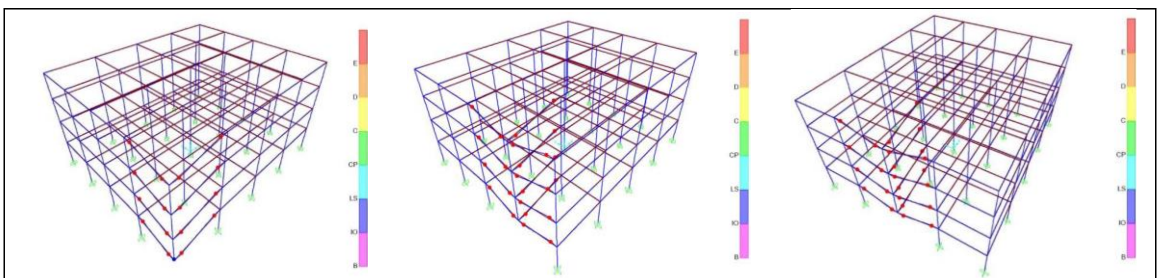
Removed columns	Performance level
Scenario 1	CP
Scenario 2	CP
Scenario 3	CP

It can be observed from Fig. 12 that the number of damaged springs in the column removal mode is lower than that in perimeter column removal mode.



(a) Mass: 4 ton. (b) Mass: 8 ton. (c) Mass: 12 ton.

Fig. 11. Comparison of critical velocity-live load contribution of column.



(a) (b) (c)

Fig. 12. Deformation of the structure and hinges formed in the structure after column removal in the ground floor; (a) scenario 1, (b) scenario 2, and (c) scenario 3.

3.3. Comparison of the results

In the previous sections, the structure under consideration was examined by two different approaches: 1) Collision modeling and 2) Column removal analysis. In the collision modeling approach, structural collapse depends on the mass and velocity of the colliding body. However, in the column removal approach, the considered structure collapsed. Therefore, despite the column removal approach which overestimates the progressive collapse, the collision modeling approach provides a more realistic estimation of the structural response against progressive collapse.

4. Conclusions

This paper consists of two main parts: progressive collapse and impact analysis. In the first part, the results of the analysis in determining the force and critical velocities indicated that the more live load contribution, the less critical is the force for column failure. In the case of critical force and constant mass, the higher the live load contribution, the lower is the critical velocity for the column failure. For the same live load contribution, the higher the mass of the vehicle, the lower is the critical velocity for the column instability, and by comparing the results of the impacted columns, it was observed that the less force and velocity are needed for the failure of the corner columns comparing to the perimeter columns. Therefore, the corner columns are the most critical columns for the impact analysis.

In the second part, in the evaluation of the progressive collapse, it was concluded that for any live load contribution, the building was ruptured after the removal of any single column of the ground floor. Furthermore, the analyses showed that the connected beams to the removed columns have undergone plastic deformations and their performance has exceeded the CP limit state. However, almost no column has undergone plastic deformation. In addition, the number of damaged elements in the corner column removal scenario is lower than that in the perimeter column removal scenario.

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