ISSN 2303-4521
Periodicals of Engineering and Natural Sciences *Original Research*
Vol. 11, No. 4, August 2023, pp.175-188

Pushover analysis of reinforced concrete structures applied to blast load using different plastic hinge models

Asher S. Dawood [1], Ali N. Attiyah [2]
[1] Structural Engineering Faculty of Engineering, University of Kufa
[2] Civil Engineering, Faculty of Engineering, University of Kufa

ABSTRACT

The current work developed a modified pushover method using the Dynamic Load Factor DLF concept to give reasonable results compared to the more complex and time-consuming method (i.e., the non-linear time history method). A charge of 100 kg TNT is assumed to explode at different stand-off distances to cover the three blast design ranges of the (UFC 340-02) Code. The values of (DLF) were checked by applying them to the value of the blast load at the stand-off distances range between (10-70m). The results of the modified pushover method approached that of nonlinear time history with differences not exceeding (11.8%) and (4%) for maximum displacement and shear force, respectively. The DLF was suggested to be constant and equal (2.5) for the (high-pressure) design range and (1) for the (very-low pressure) design range. A formula was proposed for the (low-pressure) design range to simulate the descending values from (2.5) to (1). The prior plastic hinge models proposed by other researchers (Hawraa 2019 and Samer 2020) were used to explore the more realistic structural response to blast loads compared to the standard model of ASCE41-17. Both models of the plastic hinge demonstrated a Collapse Prevention (CP) performance at the (high-pressure) design range. However, the ASCE model indicated that more columns failed in this range. Considering the ASCE 41-17 and proposed approaches, the building performance at the (low-pressure) design range corresponds to the CP and Immediate Occupancy (IO) categories. Some plastic hinges were found when using the proposed plastic hinge model, but the number was nearly identical to that obtained using the ASCE method. The structure did not go beyond the elastic behavior if the proposed plastic hinge model is used in the (very low-pressure) design range. In the same design range, the structure performance lies within the (IO) category concerning the ASCE model. Generally, the suggested plastic hinge approach has been deemed sturdy due to being developed using the blast load and considered more dependable than those of ASCE41-17, which is acceptable for seismic events.

Keywords: Blast loads, Pushover analysis method, Time-history analysis method, Scale distance, Stand-off distance, Developed Pushover method, and Proposed plastic hinge approach

Corresponding Author:

Asher S. Dawood
Structural Engineering Faculty of Engineering
University of Kufa
Al-Najaf, Iraq
azhr_kufy@yahoo.com

1. 1. Introduction

The latest terrorist incidents demonstrated that public facilities could be unsafe when subjected to blasts. Despite the reality that the most common cause of injuries occurs due to explosion pressure and heat, other threats also pose a danger. Other potential sources of injuries that arise from an explosion are debris that falls, glass that shatters, and, finally, partial or complete building collapse. Considering this, improving the explosion resistance of buildings is crucial to help prevent human deaths. It could be accomplished by implementing appropriate measures to mitigate the impact of blast loads on buildings to minimize the brutal repercussions of the blast.

There is a pity that no established standards can provide guidelines for improving the resistance to explosions in buildings; however, this can be accomplished by empirical and numerical analysis methods [1]. Blast waves are considered to have a large velocity and concentrated energy in a very short period of time, lasting only milliseconds. In the event of blast loads, the design approach should overtake the elastic stage to be more cost-effective and reasonable regarding the element size. Consequently, the plastic hinge is considered an optimal approach for a practical design utilizing non-linear methods. The Pushover method is deemed a non-linear static analysis method, but still not well known to analyze structures applied to blast loads [2]. In many studies, researchers have studied explosive loads, important explosive parameters, and calculation methods; also, the relationships and different models of explosive loading were studied [3]. developed “an Equivalent Static Force” (ESF); also, the design technique with ESF for “single-degree-of-freedom (SDOF) systems” was expanded to include the design of a structural frame made of reinforced concrete undergoing blasting conditions from a distant location. Two reinforced concrete frame constructions with six stories each were used to illustrate how the approach is used [4]. offered in-depth explanations of the blast phenomena, various blast load prediction techniques, as well as the responses of the structures. A 52-storey structure was examined for explosion at the ground level using LS-DYNA, taking into account material and geometric nonlinearities, and discovered the failed members of columns, beams, and slabs [5]. assessed how well the G+3 structure would hold up in the event of the inevitable explosions that are still to come, and a nonlinear pushover study was used. The structure was modeled using SAP2000. Specifically, beams and columns were treated as nonlinear frame components. Levels of performance, such as collapse safety, quick occupancy, and operation, were also provided in the report. The damage was rated as either severe, moderate, light, or very light, depending on the building's performance level. It was determined that the G+3 building needed a displacement of 0.0023m and a base shear of 2185.08 kN to meet its design criteria [6]. focused on analyzing the dynamic responses of a structure modeled using SAP2000. A six-storey building was subjected to various TNT stand-off distances totaling 500kg. The blast loads were taken into account utilizing the methods outlined in “section 5 of TM5-1300 (UFC 3-340-02),” and nonlinear modal analysis was utilized to analyze the dynamic load of the blast. The results illustrated that the maximum storey drift did not meet the requirements of the IS code [7]. explored the non-linear two-dimensional dynamic response of a (G+10) building applied to blast loads. A reinforced-concrete building was designed for regular loads such as dead, live, and wind loads. A total of five kinds of explosive charges were put into use: 700, 1500, 2500, 3500, and 4500 kg of TNT at distances of 15, 30, 45, 60, and 75 m, and these loads were analyzed by TM-5 1300. A 2D frame analysis of explosions on the rise structural facade indicated that reflected pressure distribution diminishes with structure height [8]. investigated the overall response of a ten-storey building to ten different earthquake scenarios to track the severity of deformations from both events. Blast loads have been applied at the nodes according to the standoff distance, angle of incidence, charge weight, and tributary region. Sidesway drifts from the explosion load were found to be substantially greater than drifts caused by an earthquake, according to the results of the blast analysis, which involved modeling the structure in two and three dimensions. Most researchers considered in their investigations the nonlinear time-history method. This method is more accurate and realistic and gives significantly good results. But this method has seemed rather complex since it needs so many inputs, and the time of analysis is relatively long. A crucial point must be clarified an absence and lack was observed in studies that tried to find the actual relationship between the pushover and time history method of analysis. Some previous investigations simulated material nonlinearity using default plastic hinge analysis programs. By default, the plastic hinge model employed by the software is based on the ASCE 41 model for the seismic load. However, the model may not accurately capture the proper response when subject to blast loads.

2. Pushover and time-history methods

2.1. Archetype frame definition

Five-story reinforced concrete frames were designed using ACI 318-19 Code, and ETABS 2019 v13 was employed to analyze blast load effects on the structure. The concrete frames have similar properties, loadings, dimensions, and compressive strength. Ground-floor height is four meters, other stories are three meters, and bays are five meters wide, as clarified in Fig.1. The frame is modeled as a reinforced concrete structure. All materials properties relating to steel and concrete are outlined in Tab.1. Some assumptions must be demonstrated in order to achieve the analysis approach, and these assumptions are as follows: (1) Moment connections are assumed to be available for all connections by default.; (2) The connections between the columns and the foundation are considered fixed.; (3) All surfaces are rigid diaphragms. Using the ETABS program, the archetype structure is analyzed, and members' cross-sections and their reinforcement ratios are checked. Tab.2 illustrates the dimensions of beams and columns and the appropriate distribution of reinforcements. The archetype structure is assumed as an office building with a dead load of 3.5 kN/m2 and a live load of 2.4 kN/m2, both dead and live loads are distributed uniformly on the floors; also, the partition wall load is uniformly distributed on the beams and equals to 13 kN/m. Combinations according to standard specifications must be considered as part of the design process. The ACI 318-19 combinations of design loads are adopted, where the lateral loads, such as wind and earthquake loads, are ignored for simplicity.

Table 1. Materials properties

Material Property	**Values**
Grade of concrete	C25
Density of concrete	2500 kg/m^3
Modulus of elasticity for concrete	23500 MPa
Poisson's ratio of concrete	0.2
Steel yielded strength	420 MPa
Steel ultimate strength	500 MPa
The elasticity modulus of steel	200 GPa
The ratio of Poisson in steel	0.3

Table 2. Detailing of columns and beams and their reinforcement

Components	**Steel**		**Concrete**
Columns	Longitudinal	8Ø20mm	350 x 350 mm
	Transverse	Ø10@240mm	
Beams	Longitudinal	4Ø16mm	300 x 500 mm
	Transverse	3Ø16mm	

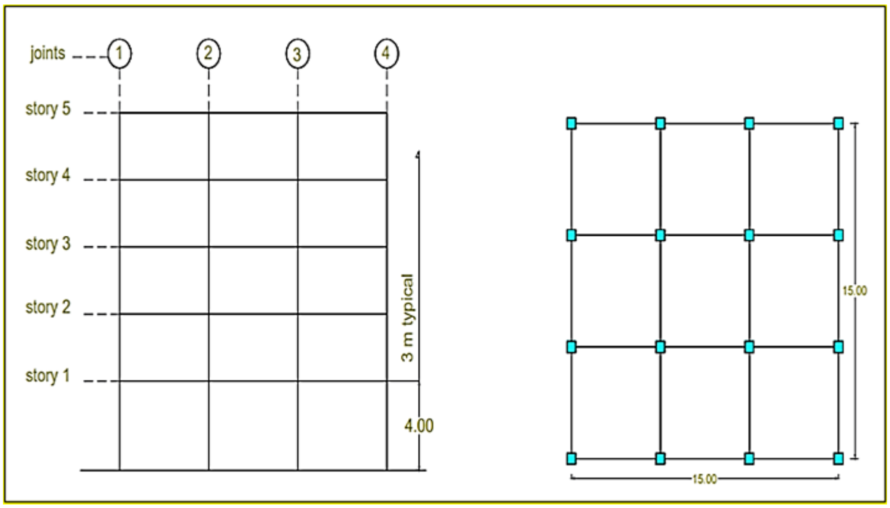

Figure 1. Top and side dimensions of the frames

2.2. Blast load patterns

When calculating the applied blast load on the structure, the areas surrounded by beams and columns are assumed to be applied to the blast pressure, as shown in Fig.2. The resulting pressure from the explosion should be transformed into a horizontal load concentrated at each connection between the column and beam. The surface burst adopted is an explosion of charge weight of (100 kg) TNT. The blast is assumed to happen due to the same charge weight at different standoff distances: 10, 15, 20, 25, 30, 50, and 70m, as clarified in Fig.3. Charge explosion is considered to originate in the x-direction from the building center and rises from the ground by 1m. The stand-off distance (R) from the blast origin point to the target point will be considered depending on the inclination angle. The value of the scaled distance (Z) will be calculated on the basis of Eq.1 for each joint of the story. Thus, the values of (Pso) and (Pr) can be calculated using the chart in Fig.4, where (Z) is the guide value. The affected area can be calculated by dividing the front area of the structure into rectangular regions. Consequently, the joint load can be calculated by distributing the blast pressure value to the attributed areas.

$Z = R/\sqrt[3]{W}$ -------------------- (Equaion.1) where: W = the weight of the explosion [kg], R = the distance from the explosion origin to the target point (m).

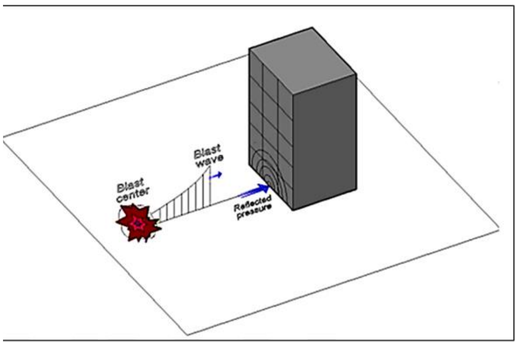

Figure 2. Surface division for the blast pressure calculation

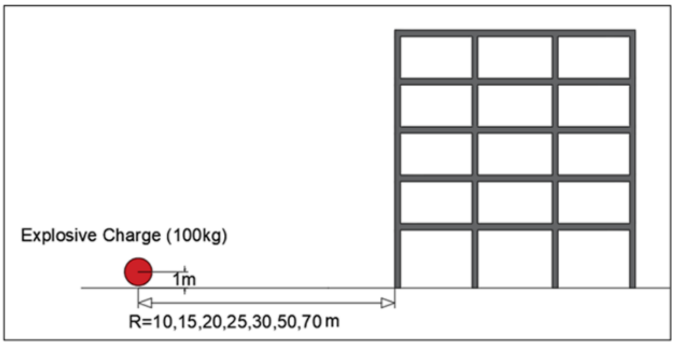

Figure 3. Blast at different standoff distances

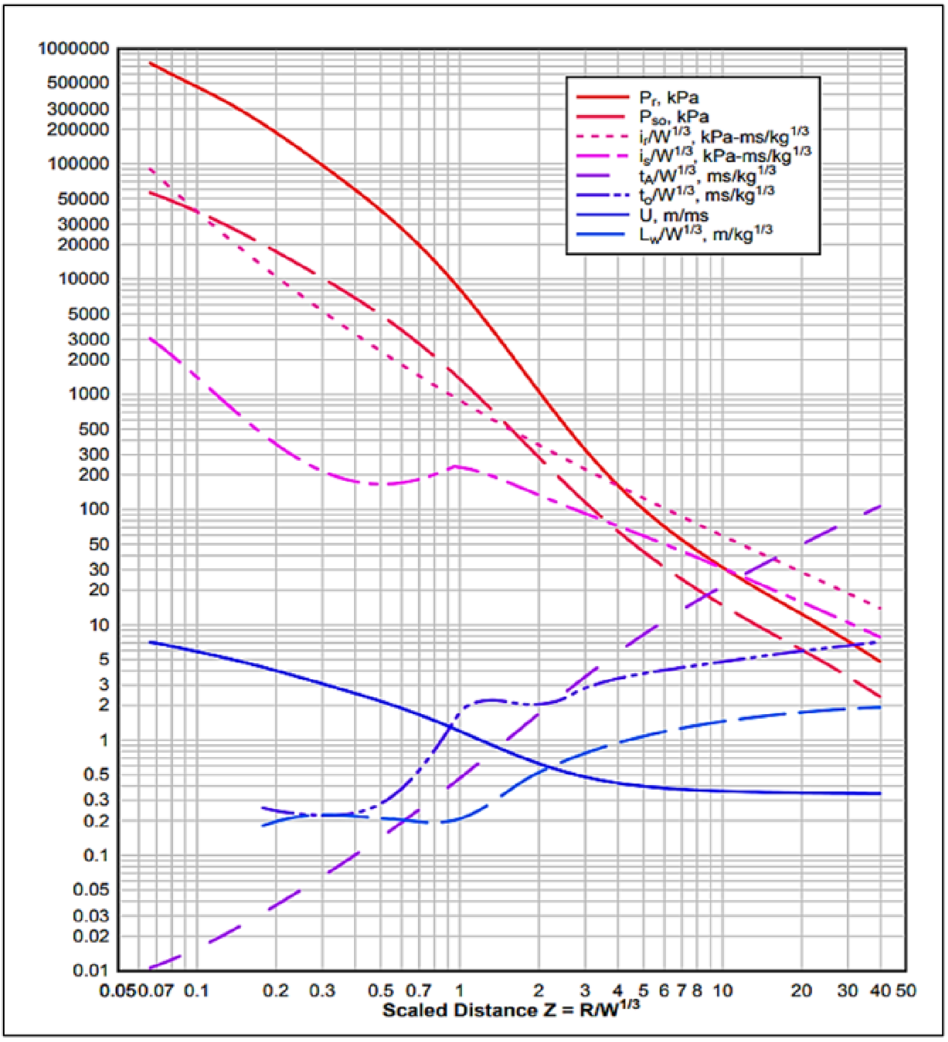

Figure 4. Parameters of positive stage shock waves for a hemispherical burst at the surface based on sea level [9]

2.3. Dynamic increase factor

The dynamic increase factor has been considered when evaluating the structural behavior under the blast load effect. The influence of this factor will be considered for steel reinforcement and concrete. Depending on the controlling scenario, this effect varied according to the bending at beams and compression at columns. Tab.3 demonstrates the values of the dynamic increase factor for rebars and concrete.

Controlling scenario	Locations	Material property (MPa)	Factor	Value (MPa)
Compression	Columns	f_c=25	1.16	f_{dc}=29
		fy = 420	1.23	f_{dy} = 516.6
Bending	Beams	f_c=25	1.25	f_{dc}=31.25
		f_y = 420	1.13	f_{dy} = 474.6

3. Plastic hinges model

3.1. Archetype structure

Two models of plastic hinges from previous researchers [10] and [11] will be used in the analysis. It was approved by both researchers that the seismic plastic hinge model of ASCE is not suitable for blast loads. Hence, the plastic hinge model suggested by [10] will be adopted for columns, whereas the model proposed by [11] will be adopted for beams. The archetype structure used by the mentioned researchers will be used here to study the effect of the plastic hinge model on the overall behavior of the structure. The ground floor is four meters high, all other floors are three meters high, and each bay measures three meters wide, as illustrated in Fig.5. The dimensions of columns and beams were based on the values mentioned in the previous work of, respectively. However, the amount of steel reinforcement is used based on the analysis approach by the ETABS program, as mentioned in Tab.4.

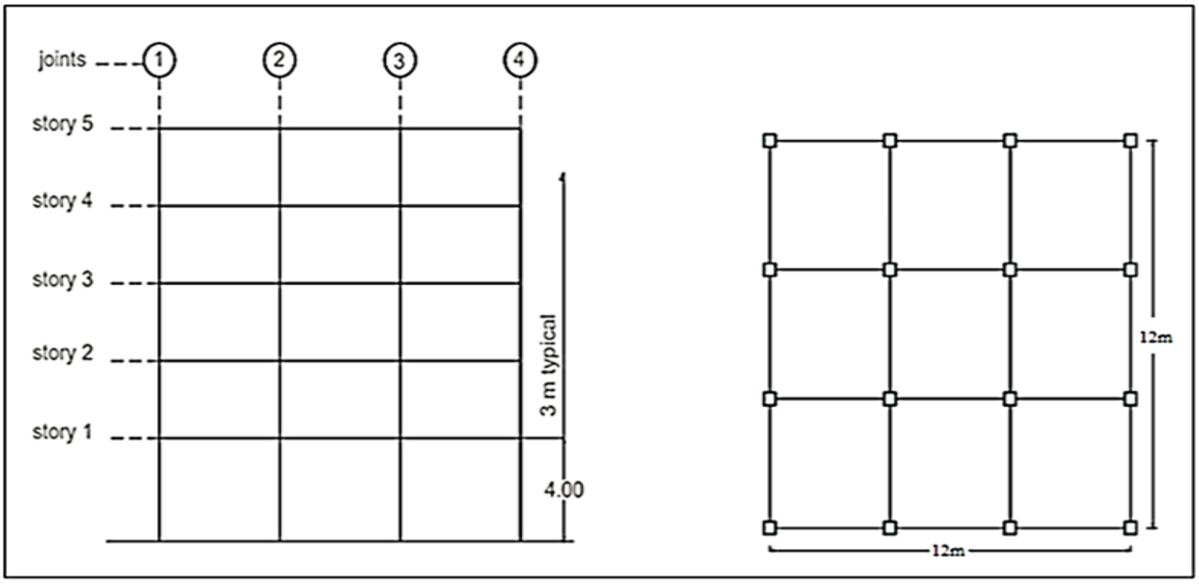

Figure 5. Top and side dimensions of the frames

Table 4. Detailing of columns and beams and their reinforcement

Components	Steel		Concrete
Columns	Longitudinal	8Ø16mm	300 x 300 mm
	Transverse	Ø10@270mm	
Beams	Longitudinal	4Ø12mm	200 x 400 mm
	Transverse	3Ø12mm	

3.2. Calculation of the blast load

The inclination angle will be determined based on the stand-off distance (R) between the original blast point and the intended point. A calculation of the (Z) value based on Eq.1 can be carried out for each story joint.

3.3. ASCE 41-17 plastic hinge model

According to (ASCE 41-17) and its characteristics, the plastic hinge model is allocated ten percent away from both edges for any element in the structure, as clarified in Fig.6. The (ASCE 41-17) auto-hinge type is adopted, and the degrees of freedom for the hinges allotted to the columns are (P-M2-M3), while the hinges assigned to the beams were (M3), as shown in Figs.7 and 8, respectively.

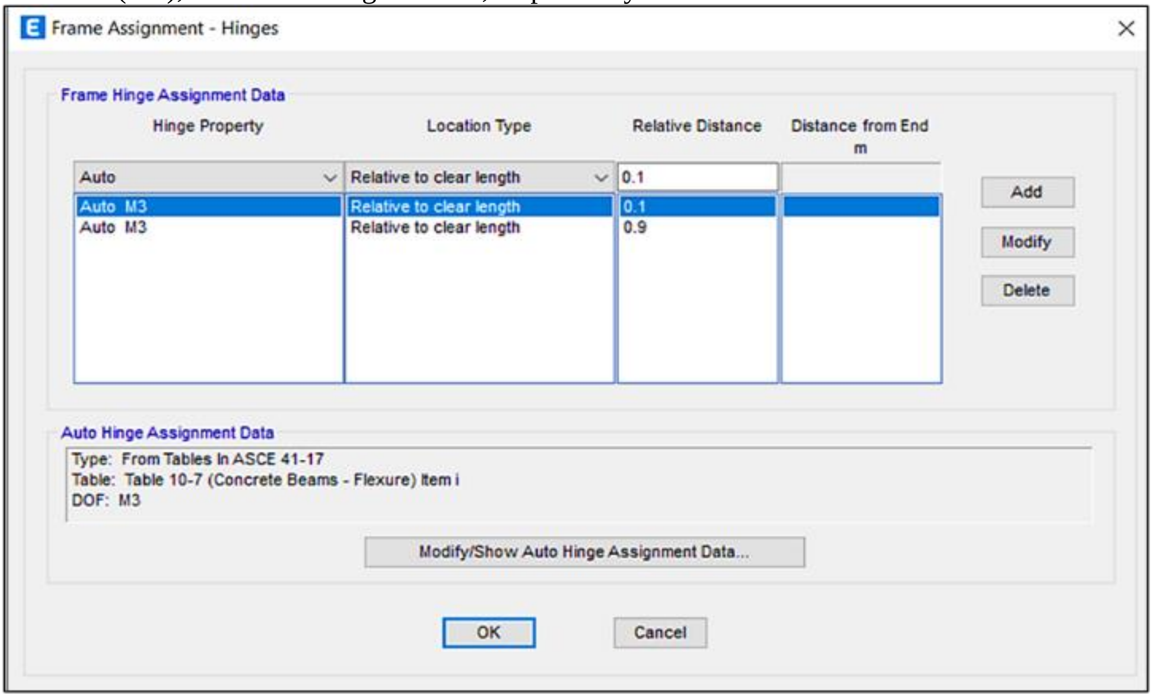

Figure 6. Frame hinges assignment data

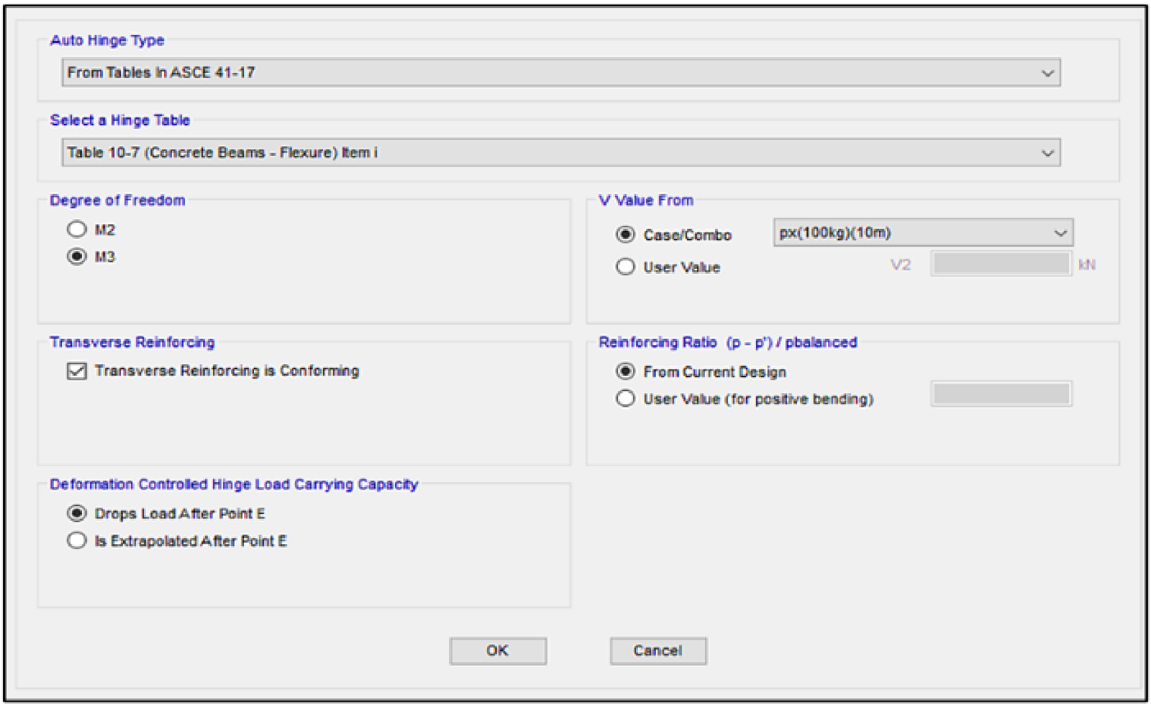

Figure 7. Assignment hinges for the column

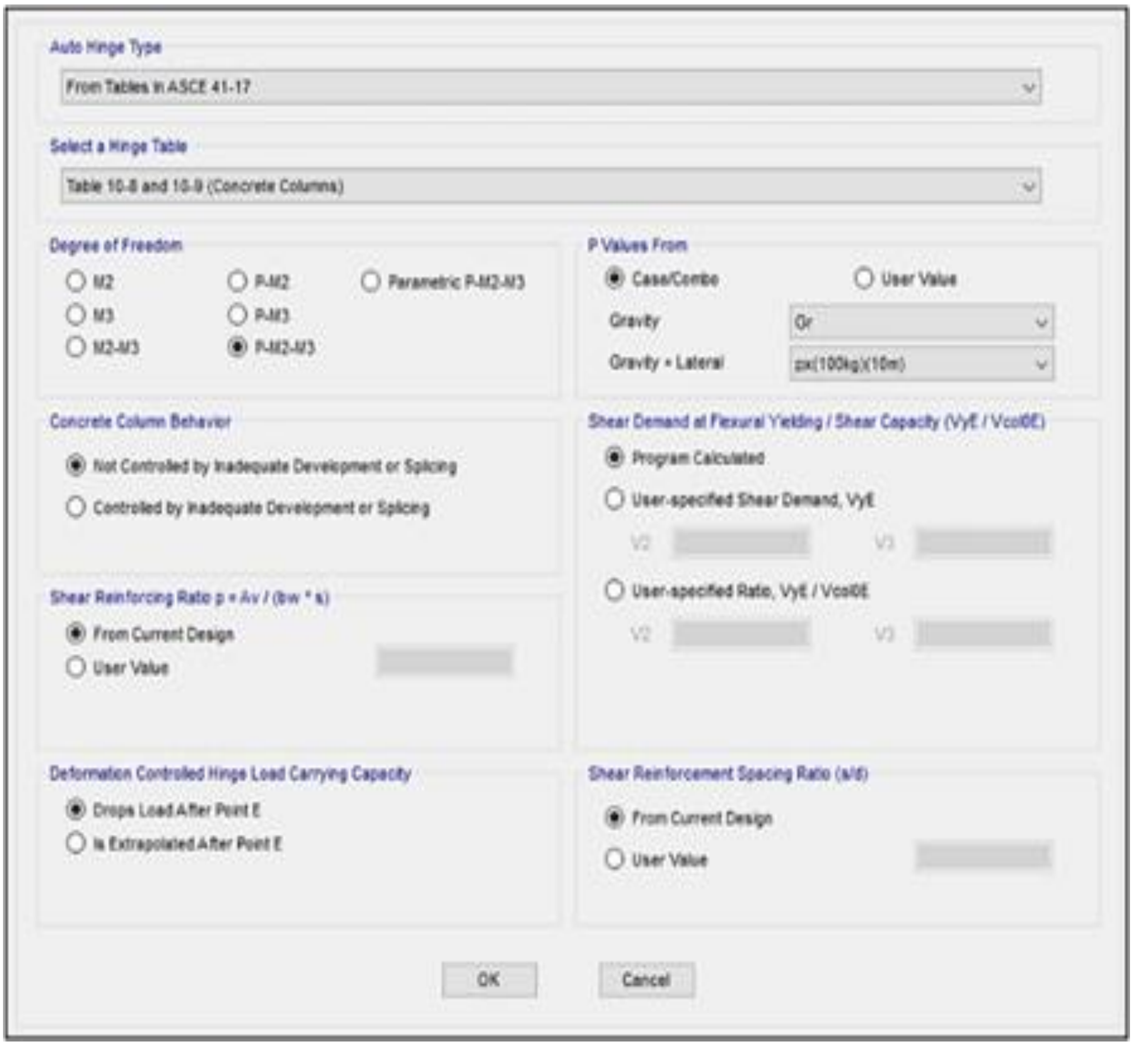

Figure 8. Assignment hinges for beam

3.4. Hawraa and Samer plastic hinge models (proposed model)

The concrete compressive strength will be considered to be 30 MPa, and the moment-rotation curve will be found following the analytical approach proposed by (Hawraa, 2019). In light of the high strain rate caused by blast load, this value must be augmented by the dynamic increase factors, as shown in Tab.5. Fig.9 demonstrates the moment-rotation curves for three types of compressive strength of concrete. However, the f′c = 30Mpa is selected as an opportunity to experience the analytical method used in the current study. As shown in Fig.10, an approximate moment-rotation curve is utilized to represent the plastic hinge models. Parameters a and b refer to the strain-buckling of compression reinforcement and the ultimate strain of tension reinforcement in that order; therefore, the impact of concrete compression strength f`c on the plastic hinge model was not observed. Based on the plastic hinge approach adopted for beams by (Samer, 2020), Tab.6 shows the properties of this model where ρs refer to the ratio of the volume of transverse reinforcement to the volume of concrete core measured to the outside of hoops. Fig.11 clarifies that parameters a and b came into two cases, and it has been considered the case that follows the red line because it represents the model under blast load.

Table 5. The dynamic increase factor of concrete compression strength [10]

f'_c **(MPa)**	**Dif**	f'_{dc} **(MPa)**
30	1.25	37.5

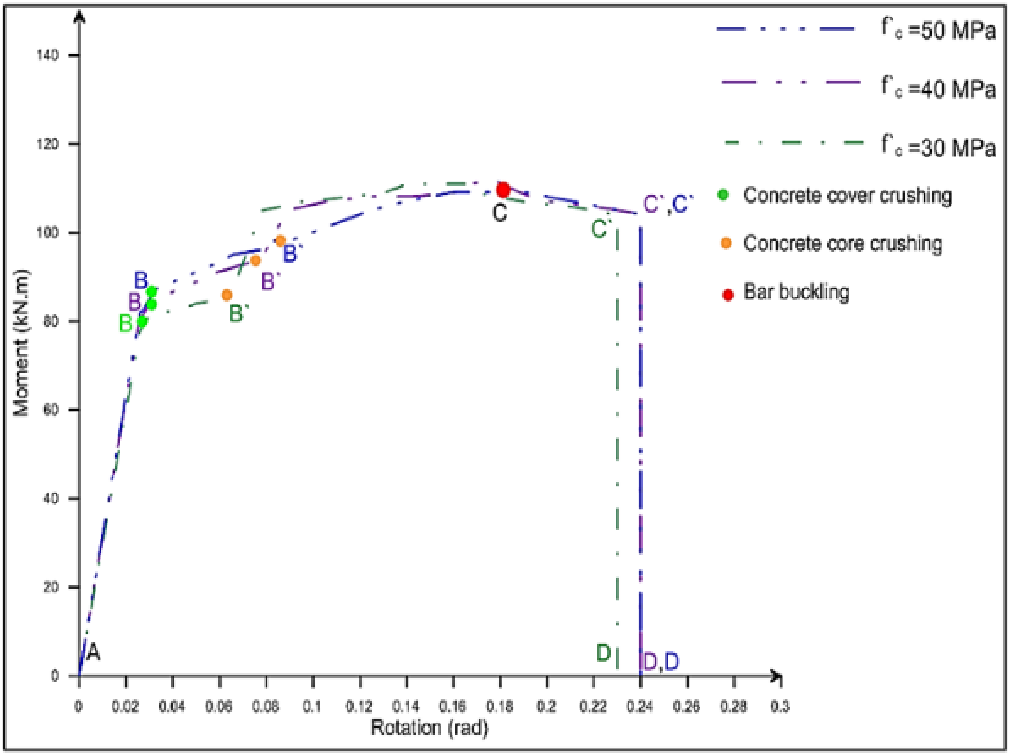

Figure 9. Moment-rotation curves for the three types of concrete compressive strength (f'_c) [10]

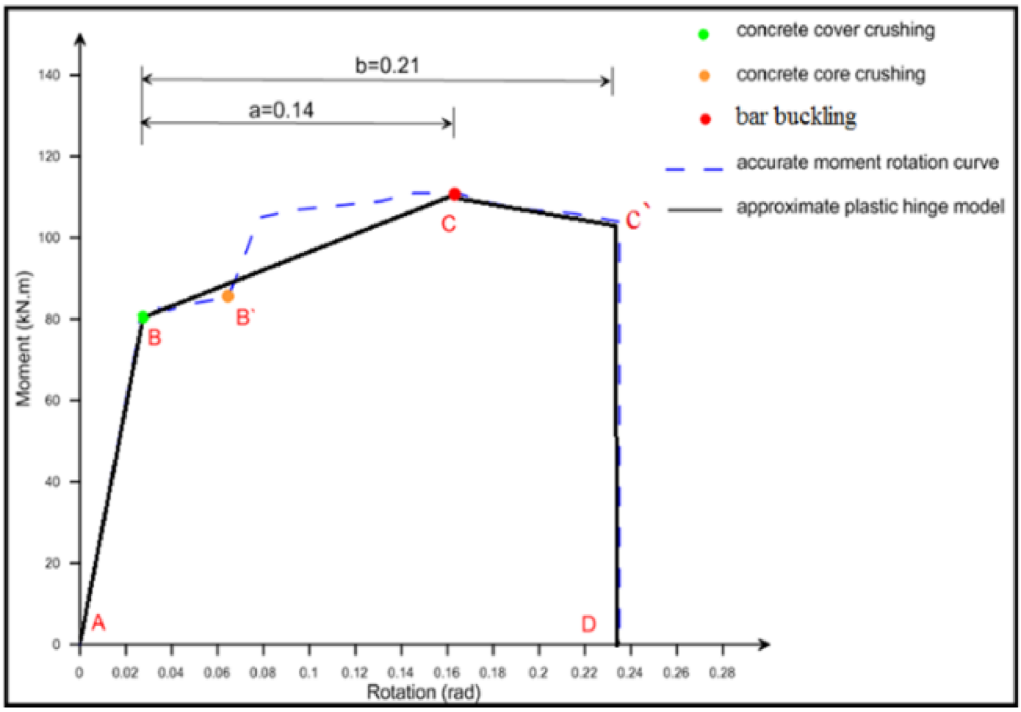

Figure 10. Plastic hinge model for compression concrete strength (f'_c) = 30MPa [10]

Table 6. The properties of the plastic hinge (Samer, 2018) [11]

f'_c (MPa)	Dif	f'_{dc} (MPa)	$f's$	K
32	1.25	40	0.02	1.125

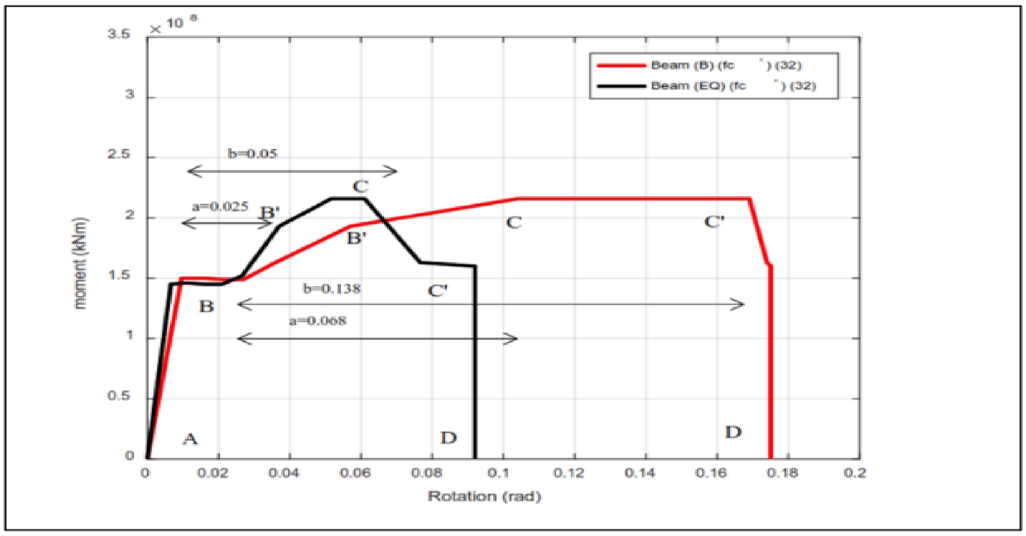

Figure 11. Plastic hinge model based on (Samer, 2018) for beams [11]

4. Results and discussions

4.1. Results for development of pushover method for blast loads

From the analysis results of both methods, it was noticed that at small-scaled distances, there was a significant difference in the number of plastic hinges. The difference started to decrease for larger-scaled distances. In other words, the best results of the Pushover method can be found for a stand-off distance of 70 m, as the two non-linear analysis methods become identical. According to the American Standard (UFC 340-02), regarding the design ranges of explosions, if t_m/t_o is more than 3, the effect of the load is considered an (Impulse) since time only plays a vital role at a high-pressure range. When t_m/t_o is less than (3) and more than (0.1), then the effect of the load is considered a (Pressure-Time), in which the pressure along with time is of utmost relevance in an influential load that came from the blast at a low-pressure range. Meanwhile, since the ratio of t_m/t_o is less than 0.1, the pressure coming from the blast has the only effect at the very-low pressure range. In this regard, in light of the (UFC 340-02) Code ranges and following the summarized results in Tab.6, thus can develop a reliable relationship between two primary non-linear analyses, including the static method (Pushover) and dynamic method (Time-history) for blast load. Figs. 12 and 13 clarify the relationship between the Pushover and Time-history method at a range of distances of blast load. The reliability of developing a relationship between two primary non-linear analyses can be observed through the design ranges of the (UFC 340-02) Code. Therefore, for the distance ranges from (0 to 20) m, the ratio of t_m/t_o ranges between (7.96-15), and the time effect is considered to be (Impulse) under these conditions. Concerning the range of distances between (25-70) m, the ratio of t_m/t_o ranges between (0.83-2.77), and the time effect is considered to be (Pressure-Time) in consideration of pressure for these distances. Finally, the ratio of t_m/t_o becomes less than 0.1 at a distance of 70 m, which means that the effect is only for the pressure but only for distances greater than 70 m. The last distance showed identical results between the method since the Pushover method considers the pressure alone in the analysis. This distance range lies in the effect of pressure according to the (UFC 340-02) Code.

Table 6. The analysis results by Pushover and Time-history of blast load at a range of scale distances

Scale distance $(\frac{m}{kg})$	**Analysis method**	**Max disp. (mm)**	**Max S.F (kN)**	**Disp. %***	**S.F %****	**Type of concrete structure**	**t_o (ms)**	**t_m (ms)**	$\frac{t_m}{t_o}$	**UFC340-02 classify**
2.155	Pushover	175	2758	40	69	CP	24	360	15	**Impulse** at **High-pressure range**
	Time-history	440	4000							
3.232	Pushover	172	2764	40	73	CP	39	390	10	
	Time-history	428	3800							
4.31	Pushover	225	3402	55	95	CP	49	390	7.96	
	Time-history	412	3600							

Scale distance ($\frac{m}{kg}$)	Analysis method	Max disp. (mm)	Max S.F (kN)	Disp. %*	S.F %**	Type of concrete structure	t_o (ms)	t_m (ms)	$\frac{t_m}{t_o}$	UFC340-02 classify
5.388	Pushover	190	3112	7	97	CP	65	180	2.77	**Pressure-Time** at **low-pressure range**
	Time-history	270	3210							
6.465	Pushover	188	3106	81	101	CP	83	180	2.17	
	Time-history	231.5	3080							
10.775	Pushover	86	2129	81	101	IO	156	130	0.83	
	Time-history	106	2110							
15.086	Pushover	88	2116	99	100	IO	205	20	0.09	**Pressure** at **very-low pressure range**
	Time-history	89	2116							

* % = Maximum displacement in (Pushover/ Time-history) x100%.

** % = Maximum shear force in (Pushover/ Time-history) x 100%.

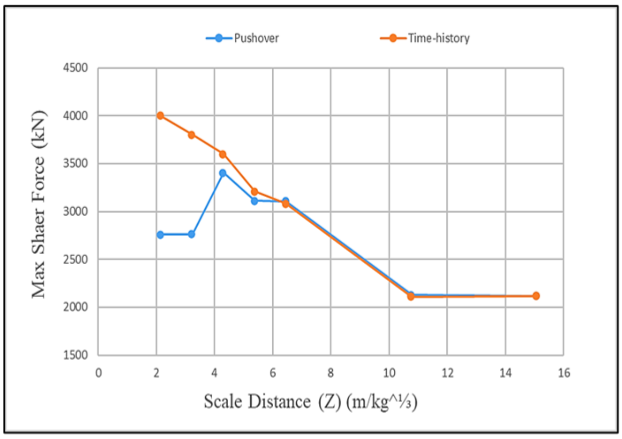

Figure 12. The relation between the Pushover and Time-history method at respecting to maximum shear force

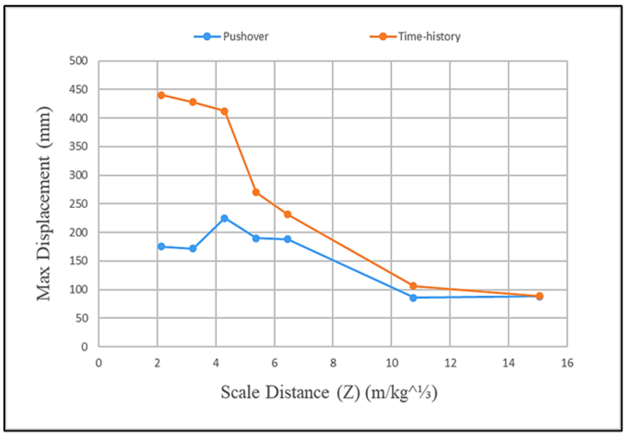

Figure 13. The relation between the Pushover and Time-history method for maximum displacement

4.2 Dynamic load factor (DLF)

The development of the Pushover analysis method enables obtaining accurate analysis results similar to that of the Time-history method using a simplistic approach. It is convenient to consider the concept of the dynamic load factor to obtain the response of a linear elastic system. This factor is defined as the ratio of the maximum dynamic deflection to the deflection which would have resulted from the static application of the peak load P, which is used in specifying the load-time variation. Thus the dynamic load factor (DLF) is given by Eq.2: DLF = X_m/X_s -------------------- (Equaion.2). Where Xs = Static deflection or, in other words, the displacement produced in the system when the peak load is applied statically, and Xm = Maximum dynamic deflection. The Pushover method can be adopted to get accurate results with less time and effort required by the Time-history method in the analysis process by taking advantage of the dynamic load factor concept, as shown in Fig.14 and Tab.7. According to the results of the dynamic load factor for the specified study range, it can be noted that the findings indicate a constant value of the dynamic load factor of 2.5 when the stand-off distance is less than 15m. In the same way, regarding distances greater than 70 m, the dynamic load factor will be constant at a value of 1. Based on the values of Tab.7 for the scaled distance range between 2.16 m and 15.09 m/kg, an equation has been developed to calculate the dynamic load factor as in Eq.3. The developed Pushover method is used to verify the results by considering the DLF at each stand-off distance within the prescribed range. Based on that, the results were nearly identical to that of the Time-history method. Tab.8 lists the maximum shear force before and after applying the DLF.

$DLF = \frac{4.7682}{Z^{0.655}}$ -------------------- (Equaion.3). Where Z: the scaling distance between (3 and 11 $\frac{m}{kg}$).

Table 7. The value of DLF for each distance

Scale distance (Z)	$\frac{X_m}{X_s}$
2.155	2.5
3.232	2.49
4.31	1.83
5.388	1.42
6.465	1.23
10.775	1.14
15.086	1.01

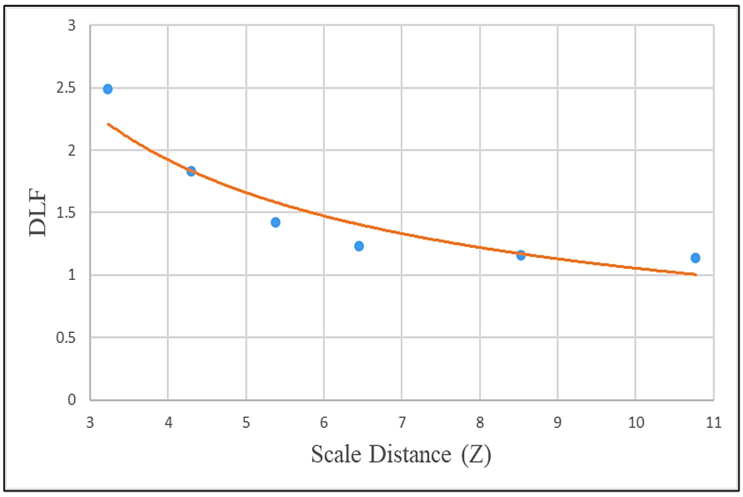

Figure 14. The DLF for each distance

Table 8. The maximum shear force and displacement before and after applying the DLF

tand-off distance	Max. shear force (kN)			Max. displacement		(mm)
	Time history	Pushover with DLF	Difference %	Time history	Pushover with DLF	Difference %
10	4000	4160	4	440	388	11.8
15	3800	3739	1.6	428	434	1.4
20	3600	3557	1.2	412	430	4.4
25	3210	3225	0.5	270	268	0.7
30	3080	2995	2.7	231.5	232	0.2
50	2110	2104	0.3	106	96.9	8.6

4.2. Results of plastic hinge models

In light of the results obtained from the analysis of concrete frame by the Pushover method, it is revealed some crucial point getting from applying two models of plastic hinges. The first model of the plastic hinge is based on the ASCE41-17 Standard, which is specialized for earthquakes. Another model is based on the incorporation of two models of plastic hinges from prior researchers (Hawraa 2019 and Samer 2020) that have been developed for blast loads. As part of this investigation, the result showed some essential differences in the structural response to blast loading when the two models are used at three stand-off distances representing the blast design ranges. The outcomes at stand-off distance (R=10m) showed that when the proposed plastic hinge is applied, the base shear will slightly increase by (2.8%) compared to the ASCE model of the plastic hinge, and it can be ignored for practical purposes. Meanwhile, the displacement achieved a decrease of (12.6%) compared to the ASCE plastic hinge model. The findings illustrate many plastic hinges in the case of the ASCE approach, and the performance of the frame can be classified as (CP) under the acceptance criteria, as clarified in Fig.15. In general, the analysis findings demonstrate that applying the proposed plastic hinge model will yield fewer plastic hinges than the ASCE model. However, the performance of the building is still within a (CP) classification, as shown in Fig.16. According to the ASCE, the plastic hinge approach showed the presence of many failed columns, as they cannot be rehabilitated. At the same time, there were only two failed columns in the proposed approach only, as they can be repaired or replaced to avoid failure in practice. For this, it is preferable to use the proposed approach in this blast design range (i.e., the high-pressure design range described in the UFC340-02 code). The outcomes at stand-off distance (R=30m) show that when the proposed plastic hinge is applied, the base shear will increase by (12.1%) compared to the ASCE model of the plastic hinge. Furthermore, the displacement decreased by (12.5%) compared to the ASCE plastic hinge model. Many plastic hinges are identified in the ASCE approach, which led to the classification of the performance of the frame as (CP) concerning the acceptance criteria of the ASCE Standards; contrary, it lies with the (IO) based on the proposed plastic hinge model, which can be seen in Figs.17 and 18. With regard to this blast design range according to the UFC 340-02 Code, which includes the (low-pressure range), it is preferable to use the proposed approach of plastic hinge because it does not have any failed column compared to the ASCE approach, which discloses that many columns have failed. Overall, the results in the present study, after utilizing a proposed plastic hinge approach, revealed some plastic hinges but nearly similar numbers to the ASCE strategy. At stand-off distance (R=70m), the findings show that when the proposed plastic hinge is applied, the base shear value is increased by (10.4%) compared to the ASCE model of the plastic hinge. Additionally, the displacement slightly decreased by (6.3%) compared to the ASCE plastic hinge model. In the ASCE approach, some plastic hinges are formed, causing the frame to be categorized as (IO) under the acceptance criteria specified by the ASCE; contrary to this, no plastic hinges are recognized in the proposed model, as depicted in Figs.19 and 20. Generally, the findings of the current investigation, following the use of the suggested model of plastic hinges, indicate that no plastic hinges are identified. However, several of them exist when applying the ASCE model. According to the proposed approach, the results showed that the structure did not enter the plastic stage and that the structure was structurally safe.

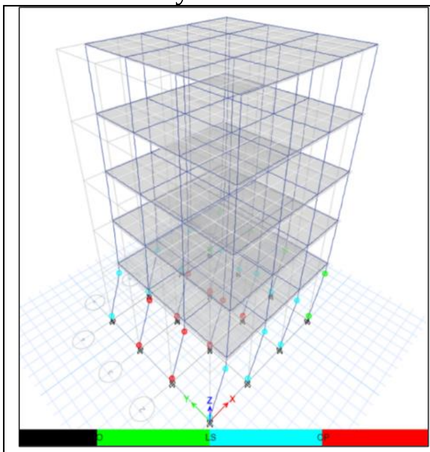
Figure 15. The State of the frame according to the ASCE approach at (R =10m)

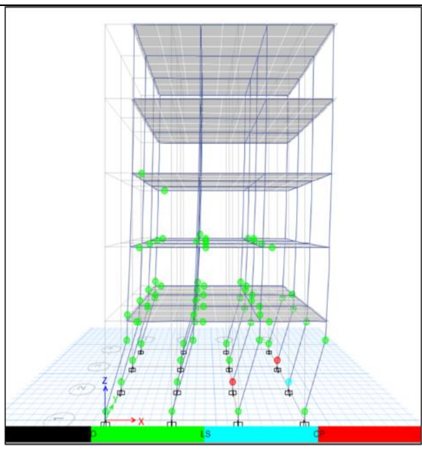
Figure 16. The State of the frame according to proposed model at (R =10m)

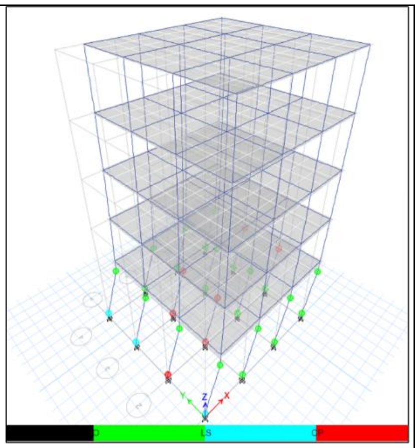
Figure 17. The State of frame according ASCE approach at (R =30m)

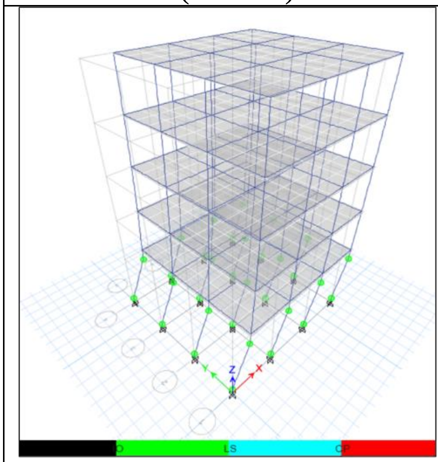
Figure 18. The State of the frame according to the proposed model at (R =30m)

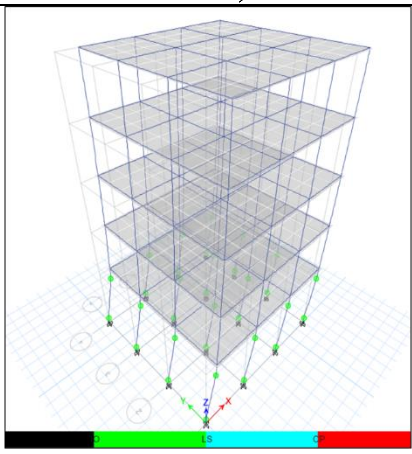
Figure 19. The State of frame according ASCE approach at (R =70m)

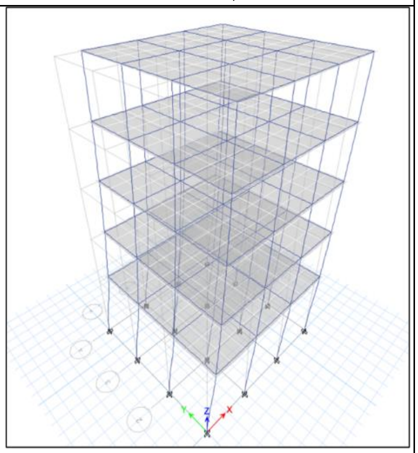
Figure 20. The State of frame according to proposed approach at (R =70m)

5. Conclusions

Considering the present work, the following conclusions are worth being highlighted, which are summarized as follows:

1. For all blast design ranges, where the scaled distances range between (2.16-15.09), the pushover method of analysis underestimates the structural response compared to the time-history analysis. Larger values of base shear and displacement are seen in the nonlinear dynamic analysis.
2. According to the plastic hinge model based on (ASCE41-17) Standards, which was adopted in the analysis of each method, the state of the performance was lying within the (CP) classification of acceptance criteria for scaled distances between (2.16-6.47) and (IO) for the scaled distances between (10.78-15.09).
3. The Dynamic load Factor (DLF) concept is used to develop a modified pushover method, which can be used in all blast design ranges. The suggested DLF is constant for a scaled distance more than (15.09) and equals to (1), and constant also for a scaled distance less than (2.16) and equals to (2.5). For scaled distances between the aforementioned values, the DLF is reduced according to the suggested formula.
4. The suggested (DLF) values give reasonable pushover analysis results compared to the time history analysis. The findings recorded a max difference in base shear force and displacement (4% and 11.8%), respectively.
5. At a distance of (R=10m), which represents the impulse design range, both models of the plastic hinge: the ASCE41-17 and the proposed model, demonstrated a (CP) state according to the acceptance criteria. However, the ASCE model indicated more failed columns. Compared to the ASCE model of the plastic hinge, the proposed plastic hinge increases the base shear by (2.8%), which practically can be ignored. But, the displacement decreased by 12.6%.
6. At the distance of (R=30m), which represents the pressure-time design range, the condition of the building was situated in the (CP and IO) categories as per the models of the ASCE 41-17 and proposed approach, respectively. Using the proposed plastic hinge approach in this range revealed some plastic hinges but nearly similar numbers to the ASCE method. The results demonstrated that when the proposed plastic hinge is applied, the base shear will increase by (12.1%) and the displacement will decrease by (12.5%) compared to the ASCE model.
7. Considering the far distance (R=70m), which represents the pressure design range, the building does not appear with any plastic hinges based on the proposed model, while it lies within the (IO) category concerning the proposed plastic hinge model. According to the proposed model, the results showed that the structure did not enter the plastic stage and that the structure was structurally safe. When the proposed plastic hinge is applied, the base shear value is increased compared to the ASCE model by (10.4%). Furthermore, the displacement was reduced by 6.3% compared to the ASCE plastic hinge model.

Declaration of competing interest

The authors declare that they have no known financial or non-financial competing interests in any material discussed in this paper.

Funding information

No funding was received from any financial organization to conduct this research.

References

[1] M. S. B. A. Shah, *Formulation of the Theory of Critical Distance for Fatigue Characteristic in Concrete Incorporating various water-Cement ratios", doctoral dissertation*. Malaysia, 2020.

[2] H. M. H. Hassan and A. N. Attiyah, *Nonlinear Resilience Analysis of Reinforced Concrete Members Subjected to Blast*. 2018.

[3] B. Li, H.-C. Rong, and T.-C. Pan, "Drift-controlled design of reinforced concrete frame structures under distant blast conditions—Part II: Implementation and evaluation," *Int. J. Impact Eng.*, vol. 34, no. 4, pp. 755–770, 2007.

[4] T. Ngo, P. Mendis, A. Gupta, and J. Ramsay, "Blast loading and blast effects on structures-an overview," *Electronic journal of structural engineering*, no. 1, pp. 76–91, 2007.

[5] P. Poluraju and N. Rao, "Pushover analysis of reinforced concrete frame structure using SAP 2000"," *International Journal of Earth Sciences and Engineering*, vol. 4, no. 6, pp. 684–690, 2000.

[6] A. Priyanka and S. V. Rajeeva, "Dynamic response of a multi-story building under blast load"," *The International Reviewer*, vol. 2, 2015.

[7] E. Hanumaiah and K. P. Devi, *Determination of Blast Load Parameters for A Multi Storey Structure*. 2016.

[8] D. (danesh) Nourzadeh, J. Humar, and A. Braimah, "Comparison of response of building structures to blast loading and seismic excitations," *Procedia Eng.*, vol. 210, pp. 320–325, 2017.

[9] U. Defense, Ed., *Structures to Resist the Effects of Explosions"*. 2008.

[10] A. Attiyah and H. Hussain, "Analytical approach to predict nonlinear parameters for dynamic analysis of structures applied to blast loads"," *Kufa Journal of Engineering*, vol. 10, no. 3, pp. 1–18, 2019.

[11] A. Attiyah and S. Dahash, "Effect of Reinforced Concrete Rotational Capacity on Plastic Hinge Performance Applied to Extreme Loads," *Kufa Journal of Engineering*, vol. 4, no. 7, pp. 16–24, 2020.