Large Deflection Behavior Effect in Reinforced Concrete Columns Exposed to Extreme Dynamic Loads

[1,4,a]M. Abedini, [1,b]A. A. Mutalib, [2]J. Mehrmashhadi, [1]S. N. Raman, [4]R. Alipour, [4]T. Momeni, [1]M. H. Mussa

[1]Department of Civil and Structural Engineering, Universiti Kebangsaan Malaysia, 43600 UKM Bangi, Selangor, Malaysia

[2]Department of Mechanical and Materials Engineering, University of Nebraska-Lincoln, Lincoln, NE 68588, United States

[4]Department of Mechanical Engineering, Mahshahr Branch, Islamic Azad University, Mahshahr, Iran

*Corresponding Author: masoud.a877@yahoo.com[a], azrulaam@ukm.edu.my[b]

Abstract

Reinforced concretes (RC) have been widely used in constructions. In construction, one of the critical elements carrying a high percentage of the weight is columns which were not used to design to absorb large dynamic load like surface bursts. This study focuses on investigating blast load parameters to design more resistant RC columns to blast loads. The numerical model is based on finite element analysis (FEA) using LS-DYNA. Numerical results are validated against blast field tests available online. Couples of simulations are performed with changing blast parameters to study effects of various scaled distances on the nonlinear behavior of RC columns. According to simulation results, the scaled distance has a substantial impact on the blast response of RC columns. With lower scaled distance, higher peak pressure and larger pressure impulse are applied on the RC column. Eventually, keeping the scaled distance unchanged, increasing the charge weight or shorter standoff distance cause more damage to the RC column. Intensive studies are carried out to investigate the effects of scaled distance and charge weight on the damage degree and residual axial load carrying capacity of RC columns with various column width, longitudinal reinforcement ratio and concrete strength. Results of this research will be used to assessment the effect of an explosion on the dynamic behavior of RC columns.

Keywords: RC column, Scaled Distance, Blast load, LS-DYNA

1. Introduction

Traditionally, reinforced concrete (RC) columns were designed to withstand only gravity loads. With time and improved analytical tools, seismic activity was included in the design as well. Recently, the susceptibility of columns to transverse loadings caused by extreme shocks, such as impacts and explosions, has garnered increasing attention [1]. An RC column may be subjected to different loading conditions such as static, dynamic, or short-duration dynamic loads. Generally, static loads are considered time-independent since they do not produce inertia effect and may last very long such as gravity compared to dynamic loads [2]. Dynamic loads may be referred to earthquake loads or wind gusts as time-dependent loads. However, short-term dynamic loads like load caused by explosive are of order 10^{-2}s which are approximately one thousand times shorter than earthquake periods [3]. Figure 1 provides an example of different dynamic hazards with their respective amplitude-frequency relationships.

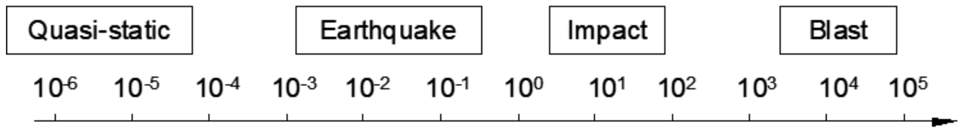


Figure 1. An estimation to strain rates caused by different types of loading

Some researchers have already studied the behavior of RC columns under surface burst [4-8]. Blast parameters which change the RC performance are the shape of structures and geometries, standoff distance, the part of the structure facing toward the blast load, and the opening of the structures [9-11]. Ngo claimed two most important parameters describing the severity of the damage are standoff distance and the charge weight [12]. Almusallam [13] studied the blast performance of an eight-story building framed with RC structure. He showed those columns experiencing reflected pressure as they were placed toward the blast waves, received the most damage. Steel bars in those columns were damaged, and the concrete fragmented. Consequently, with no load-bearing capacity, the gravity loads initiated some partial collapse. Remennikov compared some analytical approach with numerical techniques to predict blast loads [14]. He determined the limitation and simulated a simple explosion test. Calculating the blast pressure using UFC standard allowed Remennikov to apply directly to the structure [15]. He modeled the structure but not air nor the charge. Simulation with no air elements was very computationally efficient and required less time

This work focuses on investigating the effect of blast variables on RC columns. In this research finite element analysis and validation of experimental field test are investigated for RC columns when subjected to blast detonation [16]. Parametric studies are accomplished to examine the consequence of scaled distance on RC columns against explosive loadings.

2. Preparing the Finite Element Model

The Numerical model of the RC column with the height 4.4 m, the cross section of 500 × 700 mm^2 including eight longitudinal reinforcements of ϕ25 mm and transverse reinforcement of ϕ12mm is modeled in LS-DYNA. Bars are meshed with Hughes-Liu beam elements with 2×2 Gauss integration (see Figure 2), and the concrete is meshed with constant stress solid elements of size 50mm [17]. RC column is constrained on both ends except the vertical degrees of freedom (DOF) of nodes on top of the column which are free. These nodes are subjected to an axial load. Material properties are listed in Table 1. Detail description of the RC column are represented in Figure 3.

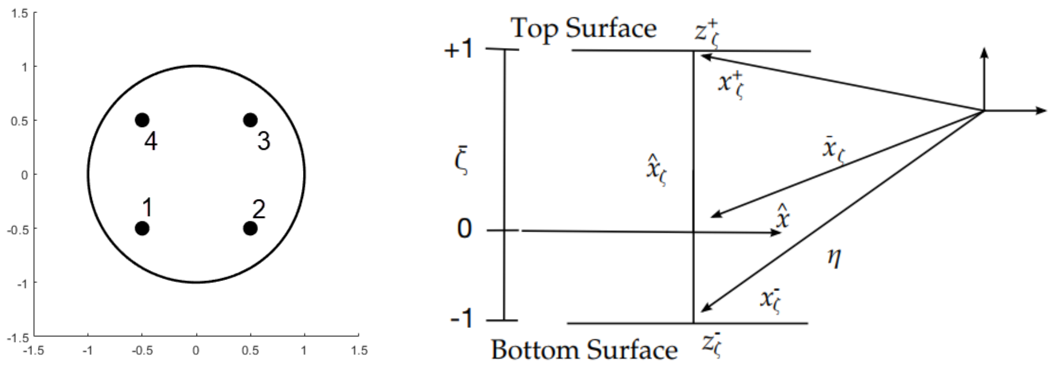


Figure 2. Integration possibilities for circular cross sections(left), Hughes-Liu beam element (right)[18]

Table 1. Concrete and steel reinforcement properties

Material	Parameters	Value
Concrete	Uniaxial compressive strength	42 MPa
	Mass density	2400 kg/m^3
	Poisson's ratio	0.2
	Tensile stress at failure	6.0 MPa
Steel Reinforcement	Young's Modulus	200 GPa
	longitudinal Steel strength	460 MPa
	transverse Steel strength	250 MPa
	Mass density	7800 kg/m^3
	Poisson's ratio	0.3
	Plastic strain at failure	0.18

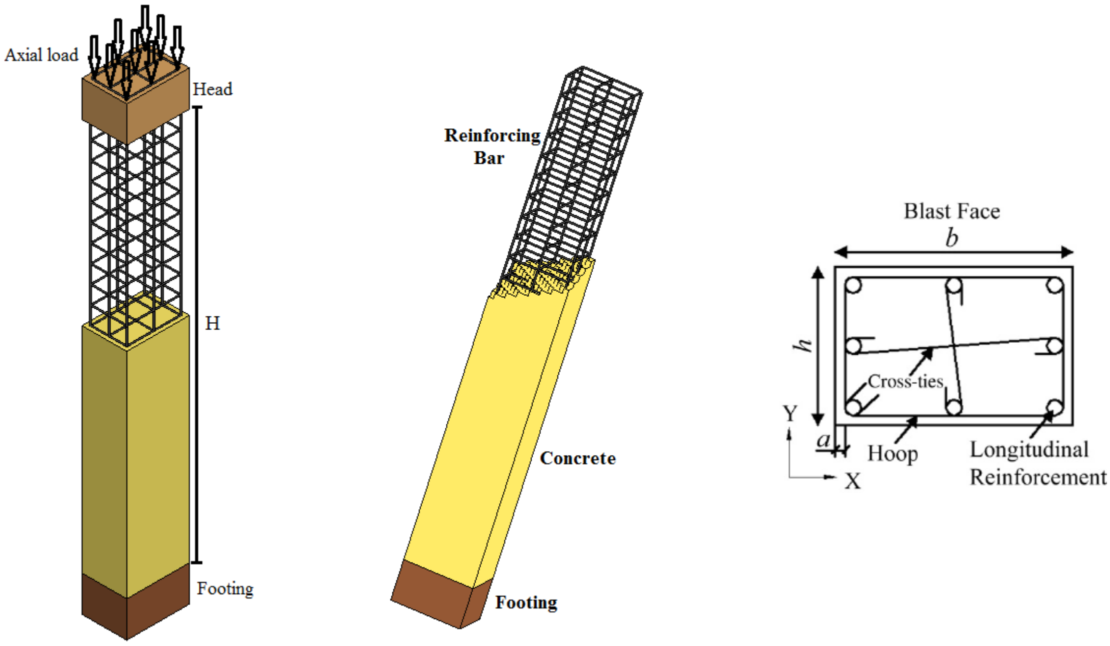


Figure 3. Detail description of the RC column

2.1. Material Models

LS-DYNA provided a comprehensive material database covering different concrete behavior. Concrete may act ductile under hydrostatic pressure or may act brittle under tensile loads like explosive loads [19, 20]. The concrete is modeled with *MAT-CONCRETE-DAMAGE-REL3 which requires only the unconfined compressive strength in the calibration process [21, 22]. The Karagozian & Case Concrete Model is a three-invariant model which uses a three-parameter function to represent the variation of compressive shear strength with mean stress of the form shown in Equation (1). This material model also includes damage and strain-rate effects.

$$SD = a_0 + \frac{P}{a_1 + a_2 P} \tag{1}$$

Where SD is the stress difference and P is the mean stress in a triaxial compression failure test, and the parameters (a_0, a_1, a_2) are determined by a regression fit of Equation (1) to the available laboratory data. Table 2 shows concrete material properties where ρ is mass density, f_c is concrete strength and υ is Poisson's ratio. The material of rebars is considered as material type 24 shown in Table 3 where E is Young's Modulus, f_y is longitudinal steel strength, and f_{yt} is transverse steel strength [17, 23] .

Table 2. The concrete material properties

ρ	f_c	υ
2400 kg/m^3	42MPa	0.2

Table 3. Material properties of rebars

ρ	E	υ	f_y (longitudinal rebars)	f_{yt} (transverse rebars)
7800kg/m^3	200GPa	0.3	450MPa	400MPa

LS-DYNA provided the keyword of *MAT_ADD_EROSION to delete those elements meeting erosion criterion [19]. This keyword adds erosion criterion to materials which do not have any failure criteria. Although this keyword helps to understand the failure mechanism, it also affects the mass and the inertia properties of the model by removing elements. Therefore, using this keyword is only suggested to investigate the damage mechanism graphically. A number of criterions are available in LS-Dyna for this keyword. We used the maximum effective strain at failure shown in Equation 2 is used for this studied.

$$\epsilon_{\text{eff}} = \sum_{ij} \sqrt{\frac{2}{3} \epsilon_{ij}{}^{dev} \epsilon_{ij}{}^{dev}} \tag{2}$$

Where $\epsilon_{ij}{}^{dev}$ is the deviatoric strain states.

Among those various methods available in the literature, peridynamics as a nonlocal form of continuum mechanics has increasingly used to study fracture and crack propagation in many fields and has been validated against a variety of experimental tests [24-29]. In peridynamics, the damage is a part of the solution not a part of the problem. However, peridynamic (PD) modeling of RC column requires a dense grid, and the stable timestep in the explicit integration would be relatively low for the current study [30]. Therefore, PD modeling of the RC column might not be computationally efficient.

2.2. Strain Rate

Higher strain rate can sometimes increase the strength of the material. This behavior is identified as the dynamic increase factor (DIF). Tensile DIF (TDIF) is a function of tensile strengths at high strain rate and tensile strength at static loadings. Similarly, Compressive DIF (CDIF) represents compressive strengths at high strain rate versus compressive strength at static loadings. To investigating the effect of high strain rate loads such explosive on the behavior of the RC column, DIF is calculated for each type of loading and is applied directly to the material model.

2.2.1. Modified Strain Rate for Concrete in Compression

Many researchers have studied the influence of high strain rate on the behavior of concrete materials. Watstein [31], Jones and Richard [32], and Granville [33] showed that the increase of loading rate also increased the compressive strength of the concrete. For a strain rate of 10 s^{-1} Watstein [31] recommended an increase of 80% in compressive strength. DIF for the compressive strength of the concrete was formulated using the CEB-FIB Model Code [34] as follows:

$$CDIF = \frac{f_c}{f_{cs}} = [\frac{\dot{\varepsilon}}{\dot{\varepsilon}_{cs}}]^{1.026\alpha} \qquad \text{for} \qquad \dot{\varepsilon} \leq 30\ s^{-1} \tag{3}$$

$$CDIF = \frac{f_c}{f_{cs}} = \gamma(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_{cs}})^{\frac{1}{3}} \qquad \text{for} \qquad \dot{\varepsilon} > 30\ s^{-1} \tag{4}$$

$$log\gamma = 6.156\alpha - 0.49 \tag{5}$$

$$\alpha = \frac{1}{5 + \frac{3f_{cu}}{4}} \tag{6}$$

where;

f_{cd} = Compressive strength (dynamic) at $\dot{\varepsilon}$

f_{cs} = Compressive strength (static) at $\dot{\varepsilon}_{cs}$

$CDIF$ = Compressive DIF

f_{cu} =Static cube strength

$\dot{\varepsilon}_{cs}$= $3 \times 10^7 \frac{1}{s}$ (static strain rate)

$f'_{co} = 10\ MPa$

$\dot{\varepsilon}$ = Strain rate (3×10^7 - $300 \frac{1}{s}$)

2.2.2. Modified Strain Rate for Concrete in Tension

Concrete is also sensitive to tensile strain rate due to the heterogeneity of the material [35]. Tensile strength can be increased a substantial amount for loading rates beyond 10 *MPa/s*. Tensile DIF for a given strain rate may be estimated from the following equations.

$$TDIF = \frac{f_t}{f_{ts}} = [\frac{\dot{\varepsilon}}{\dot{\varepsilon}_{ts}}]^{\delta} \qquad \text{if} \qquad \dot{\varepsilon} \leq 1s^{-1} \tag{7}$$

$$TDIF = \frac{f_t}{f_{ts}} = \beta[\frac{\dot{\varepsilon}}{\dot{\varepsilon}_{ts}}]^{\frac{1}{3}} \qquad \text{if} \qquad \dot{\varepsilon} > 1s^{-1} \tag{8}$$

$$\beta = 7.11\delta - 2.33 \tag{9}$$

$$\delta = \frac{1}{10 + \frac{6f_c'}{f_{co}'}} \tag{10}$$

where;

$\dot{\varepsilon}$ = Strain rate ($3 \times 10 - 6 - 300 \frac{1}{s}$)

$\dot{\varepsilon}_{ts}$= $3 \times 10 - 6 \frac{1}{s}$ (static strain rate)

$f_{co}' = 10$ MPa

f_{ts} = Tensile strength (Static) at $\dot{\varepsilon}_{ts}$

f_t = Tensile strength (Dynamic) at $\dot{\varepsilon}$

f_c' = Static uniaxial strength of concrete (in *MPa*)

2.2.3. Modified Strain Rate for Steel

The sensitivity of stress and strain curves of steels to loading rates is called the strain rate sensitivity [36, 37]. Strain rate sensitivity has a important consequence on the inertia effect of the material and affects the load-displacement curve tested under different uniaxial compression strain rates [38, 39]. Malvar introduced DIF as the new equation for steel ASTM rebars which represented the effect of strain rate on the strength improvement [40]. Malvar leveraged test results available in the literature to derive his equation as follows:

$$DIF = \frac{(\dot{\varepsilon})}{10^{-4}}^{\alpha} \tag{11}$$

$$\alpha = 0.019 - 0.009\frac{f_y}{414} \qquad \text{for} \qquad \text{ultimate stress} \tag{12}$$

$$\alpha = 0.074 - 0.040\frac{f_y}{414} \qquad \text{for} \qquad \text{yield stress} \tag{13}$$

where;

f_y = steel yield strength

2.3. Contact Algorithm

In this study, the keyword of *CONTACT_1D is implemented to consider the bond-slip interactive effect between the concrete and longitudinal rebars [41]. The bond between the rebar and concrete is assumed to be elastic perfectly plastic. The maximum allowable slip strain is given as:

$$\mu_{max} = \mathrm{SMAX} \times e^{-\mathrm{EXP}\times D} \tag{14}$$

where D is the damage parameter $D_{n+1} = D_n + \Delta u$. The shear force, acting on area A_S, at time n + 1 is given as:

$$f_{n+1} = \min[f_n - \mathrm{GB} \times A_s \times \Delta u, \mathrm{GB} \times A_s \times u_{\max}] \tag{15}$$

where GB is bond shear modulus and SMAX is the maximum shear strain. This contact algorithm makes steel nodes dependent on concrete nodes and allows stress transfer between different materials. The stress transfer can affect the dynamic behavior of the RC column [42, 43]. Methods considering perfect bond assumption have been previously used by researchers such as Fanning [44], and Tavárez [45]. In this method, steel nodes are merged into concrete nodes. Consequently, the failure criterion for the steel material would entirely depend on the failure of the concrete.

3. Simulation of Explosive Load in LS-DYNA

Several ways can be used to simulate explosive loads in LS-DYNA considering explicit integration [30]. The simplest method is computing the time history of the blast pressure at the point of interest from other source and then apply the pressure directly on the structure [46]. The idealized pressure profile can be of the triangular ramped form (see Figure 4) applied uniformly on the front face [47]. The keyword of *LOAD_SEGMENT_SET is used to define the pressure profile and column front surface [48]. Although the reflected pressure and pressure superposition near the front face are neglected, this approach can qualitatively capture the failure mechanisms of RC columns subjected to surface burst and to reveal the effectiveness of the multi-hazard detailing on the blast resistance of ordinary highway bridges. Compared to other blast load techniques, the pressure time history method offers computational time savings.

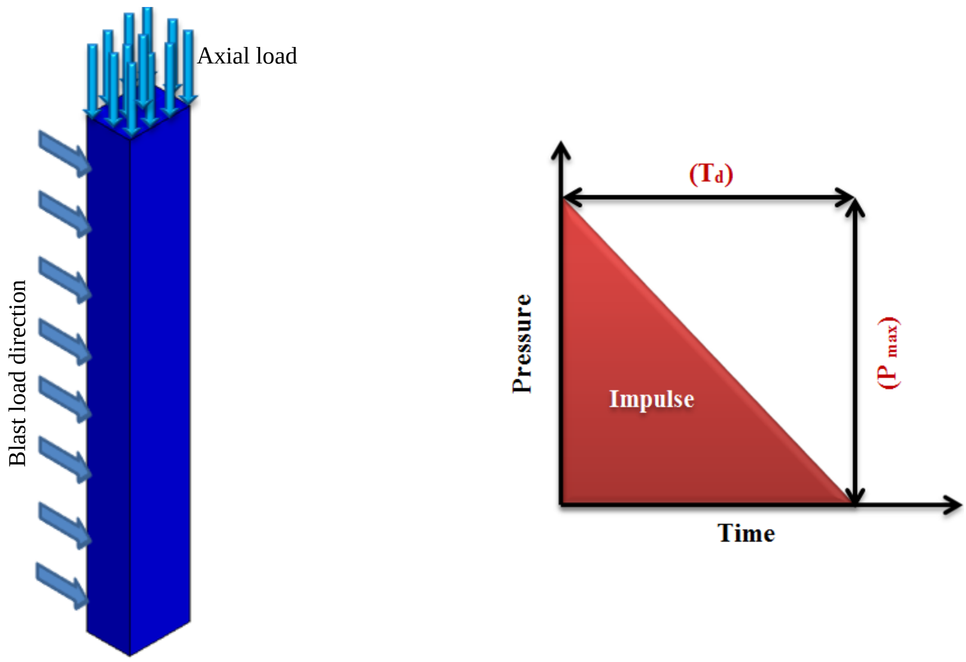


Figure 4. Simplified blast pressure-time method

4. Verification of Numerical Models

The proposed numerical model is validated against Baylot's and Benvis' experiment No.02 which investigated the behavior of the exterior middle column (see Figure 4)[16]. The dimensions of the column were: the cross-section of 85×85 mm^2, span length of 0.935m, eight longitudinal rebars of ϕ7mm, and stirrups of ϕ3.5mm which closed longitudinal reinforcements. Material properties of the column were: unconfined concrete strength of 42MPa, ρ = 2068 kg/m^3, and E = 28.7GPa. Material properties of rebars were: yield stress of 450 and 400 MPa for longitudinal and transverse reinforcement, respectively. Charge weight of 7.087 kg C4 was placed at the standoff distance of 1.07m and 228.6mm above the ground (see Figure 5). Baylot and Bevins provided finite element analyses in addition to their experiments. The sequence of effective plastic strain variations available in the *CONCRETE_DAMAGE_REL3 material model as damage parameter is illustrated in Figure 6. Colors show the level of concrete damage. The blue color denotes no damage, the red color represents the residual capacity of the concrete, and other colors represent the damage levels of the concrete.

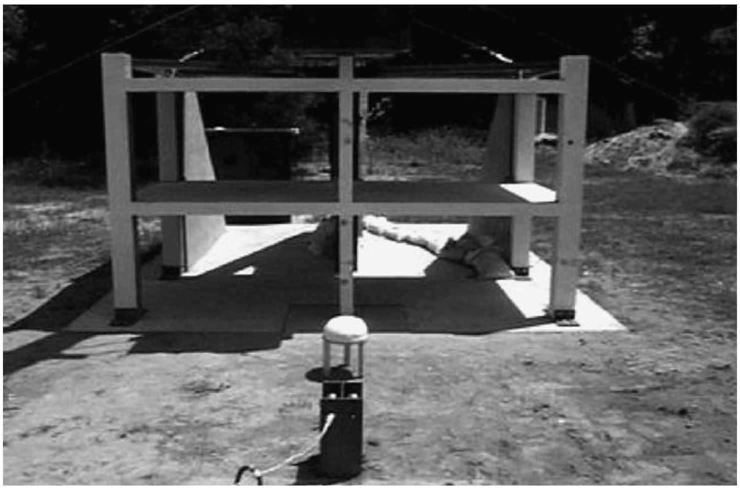
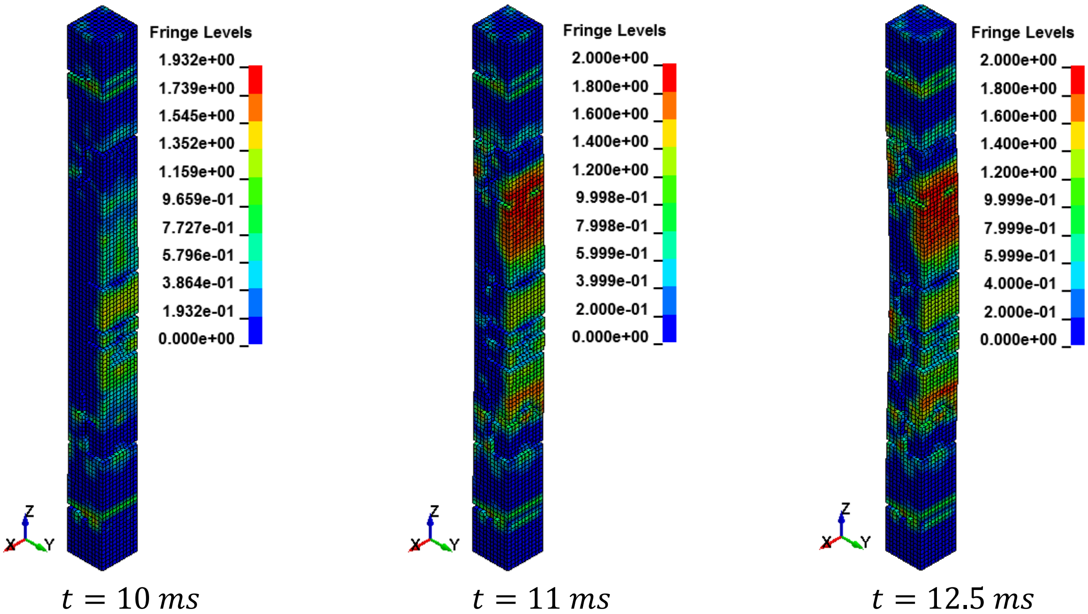
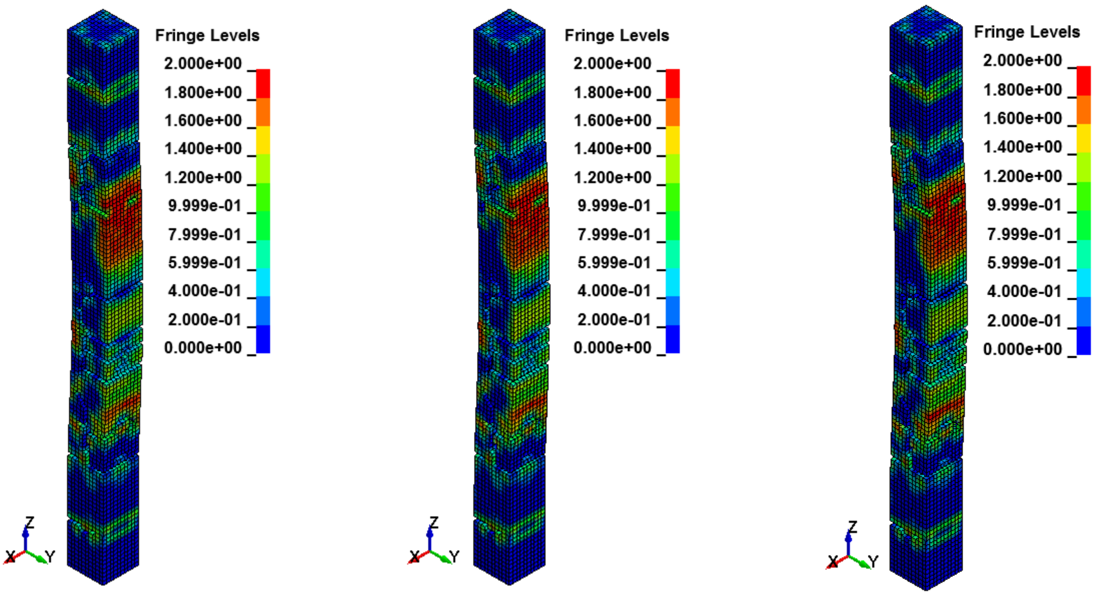


Figure 5. The ¼ scale structural model for experiment Number 02 by Baylot and Benvis [16]





$t = 14.5\ ms$ $t = 15.5\ ms$ $t = 16.5\ ms$

Figure 6. plots of effective strain diagrams at different times

The variation of the lateral displacement at mid-height of the column is compared with the experiment (see Figure 7). The horizontal displacement at the mid-height was 12.5 and 12 mm in experiment and the present study, respectively. The difference in the lateral deflection was only about 4.16 percent. However, residual deflections are almost the same in both present analysis and experimental results (6.3mm). In conclusion, the presented finite element model is validated using experimental data obtained by Baylot and Bevins [16].

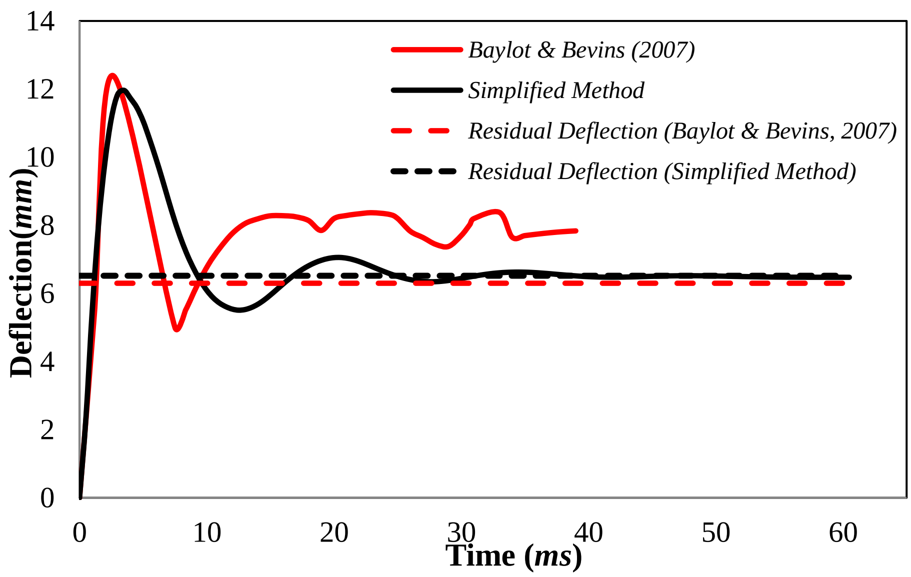


Figure 7. Deflection time histories of mid-height [16]

5. Numerical Analysis

Structural response exposed to explosive loads can be classified based on the strength of the explosive pressure called high/low pressure. Scaled distance (Z) defines the intensity of the blast pressure [49] and is defined as the ratio of standoff distance and the cube root of the charge weight. The target designed to stand against the high-pressure waves is typically placed near the charge and absorbs reflected pressure whereas, targets designed for low-pressure range are often experience the side on pressure and are mostly positioned parallel to wave propagation [49]. Blast parameters for any blast event are found as functions of the distance from the blast center (R) and the equivalent charge weight (W). Scaled distance is of the following form:

$$\boldsymbol{Z} = \frac{\boldsymbol{R}}{\boldsymbol{W}^{1/3}} \tag{16}$$

Three regimes are defined by Smith [50] using *Z* shown in Table 5.

Table 5. Categories of response regime [50]

Scaled distance	*Z* (m/kg$^{1/3}$)	*Z* (ft./lb$^{1/3}$)
Close-in	Z < 1.190	Z < 3
Near-Field	1.190 < Z < 3.967	3 < Z < 10
Far-Field	Z > 3.967	Z >10

Scaled distances corresponding to the charge mass of 100kg and three standoff distances are calculated using Equation (12) and presented in Tables 6.

Table 6. Scaled distances at 100kg charge weight subjected to close-in, near-field, and far-field detonation

	Standoff distance (m)	Charge weight (kg)	Scaled distance (m/kg$^{1/3}$)
Close-in	2.79	100.0	0.6
	3.71		0.8
	4.64		1.0
Near-Field	6.96	100.0	1.5
	9.28		2.0
	11.6		2.5
Far-field	18.56	100.0	4.0
	20.88		4.5
	23.2		5.0

6. Results and Discussion

6.1. The RC Column behavior under various scaled distances

Responses of the RC column are simulated against a couple of blast loads. The performance of the RC column is assessed with plotting the maximum and minimum principal stresses plotted in Figure 8 and 9. The scaled distance for the selected column is 0.6 m/kg$^{\wedge 1/3}$. The effective plastic strain at the mid-height cross section is shown in Figure 8.

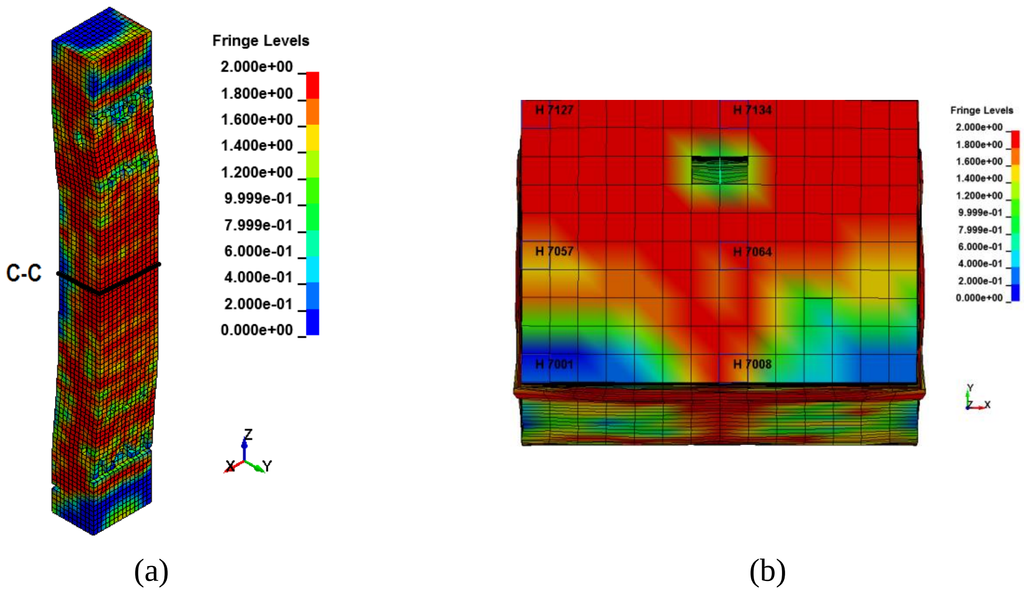
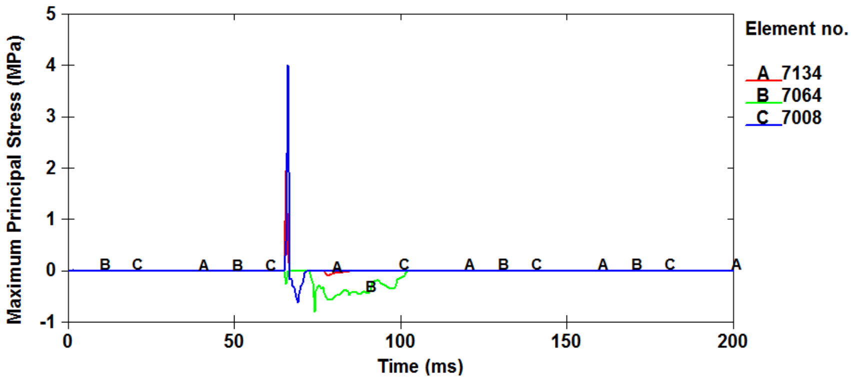


Figure 8. (a) The cross section at mid-height of the column and (b) Cross section C-C with selected elements in concrete

Maximum and minimum principal stresses for nodes identified in Figure 8b are depicted in Figures 9 and 10. In Figure 8, stress values at elements 7134 and 7064 are very close, and hence the graphs coincide. Since both principal stresses reach zero, the column has lost its load carrying capacity after the blast load hit it. Bond and adhesion failure occurred between rebars and concrete in this column.



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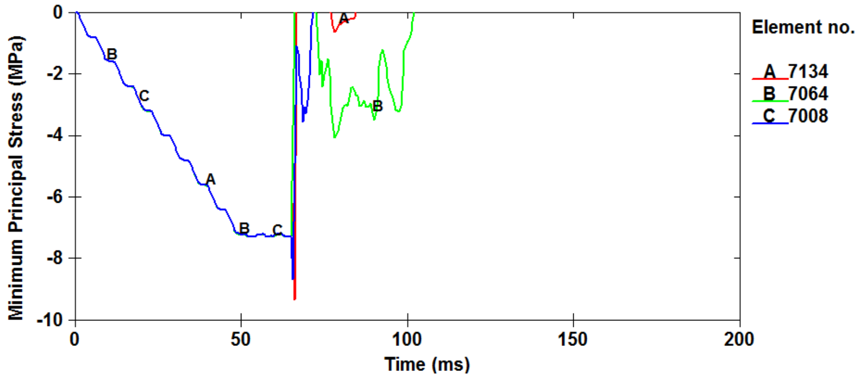


(b)

Figure 9. (a) Maximum and (b) minimum principle stress plots for elements shown in Figure 7b

(a)

(b)

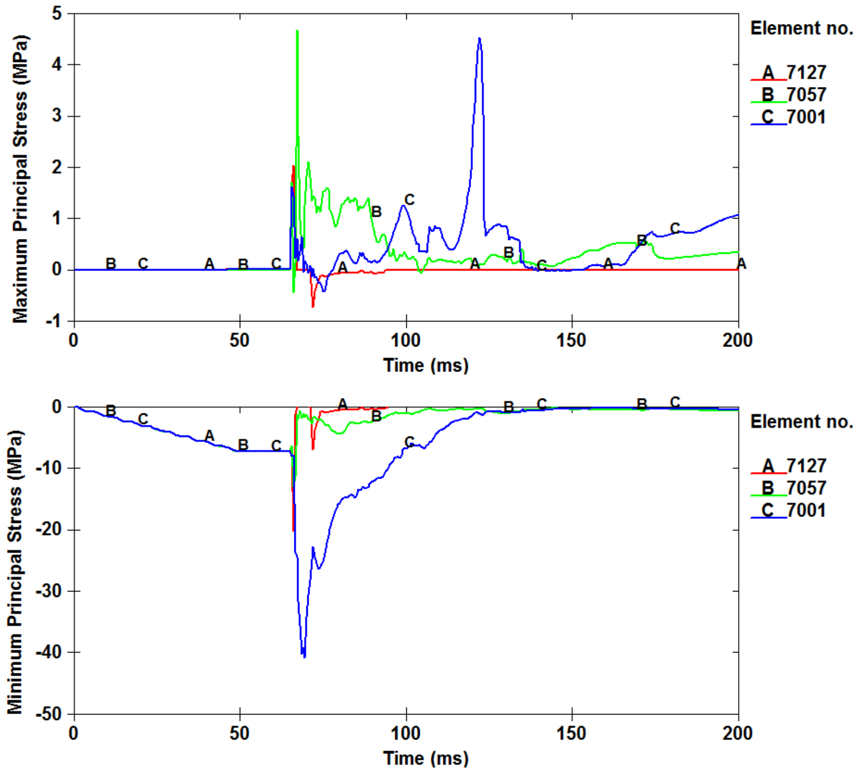
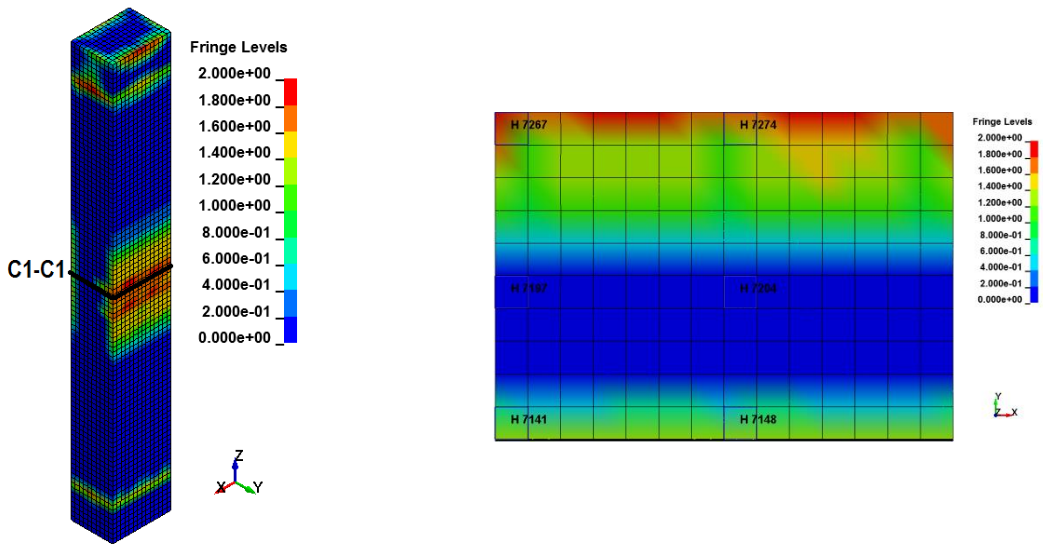


Figure 10. (a) Maximum and (b) minimum principle stress plots for elements shown in Figure 7b

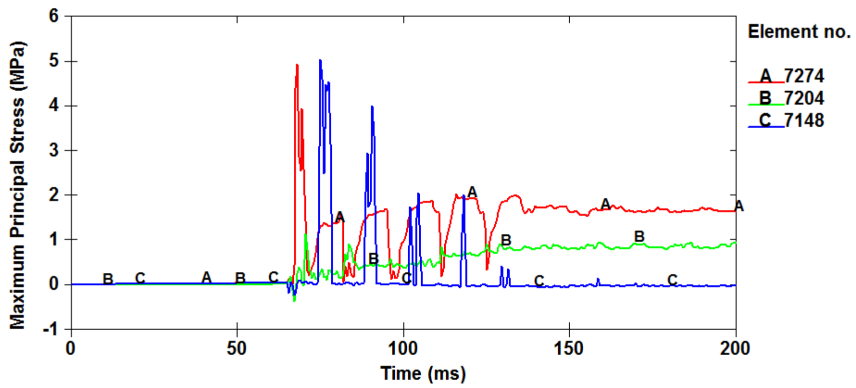
Weak points of the column and the column cross section at the scaled distance of 2 $\frac{m}{kg^{1/3}}$ are shown in Figure 11. The effective plastic strain is plotted in Figure 11 at time of 15ms before the progressive damage initiated.



(a) (b)

Figure 11. (a) The cross-section and weakest points of the column and (b) Cross section C_1-C_1 with selected elements in concrete

The profile of max and min principal stresses for element shown in Figure 11b is depicted in Figures 12 and 13. The element's number of 7148 and 7141 no longer hold any stress after hitting by the blast load. Concrete elements of 7274, 7204, 7267, and 7197 could hold gravity load since their stresses are not zero. The stress plots confirm that the concrete does not yield. When the bond between rebars and the concrete breaks, explosive loads can loosen the concrete confinement. However, the concrete damaged area experienced that the steel reinforcement became ineffective and concrete returns to the unconfined state. Consequently, the load carrying capacity of the column plunged dramatically. The minimum cross-sectional area of the undamaged concrete at weakest points of the column can be used to calculate the residual capacity for the undamaged concrete.



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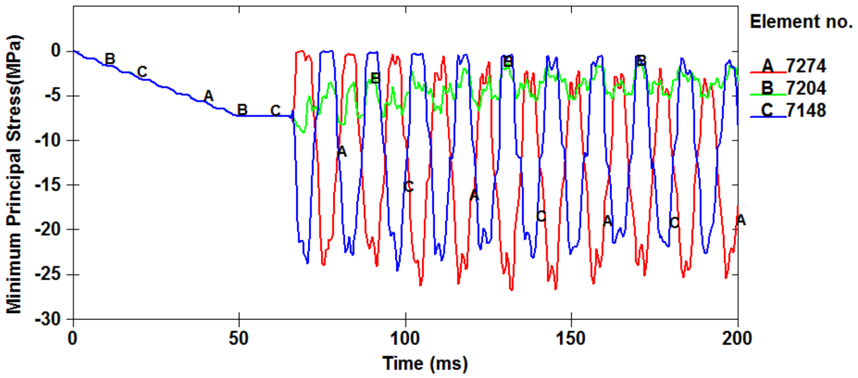

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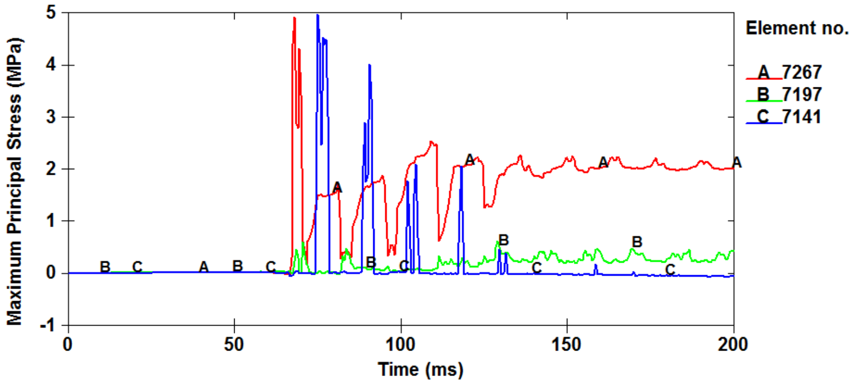
Figure 12. (a) Maximum and (b) minimum principle stress plots for elements shown in Figure 10



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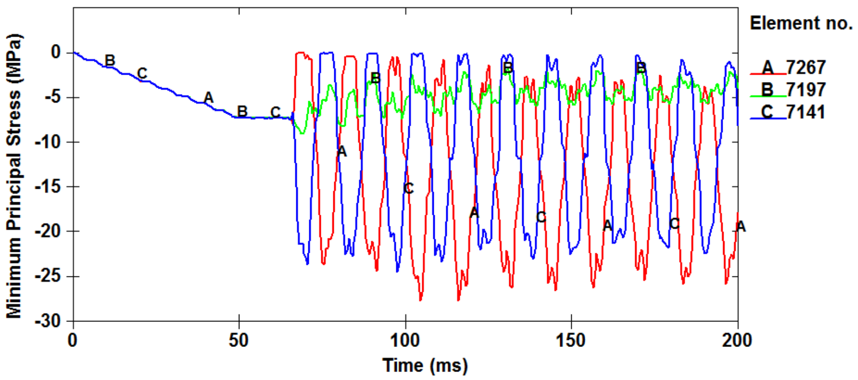


(b)

Figure 13. (a) Maximum and (b) minimum principle stress plots for elements shown in Figure 10

Stress contour plots for different scaled distances under near-field, close-in, and far-field explosions are illustrated in Figure 14-16. In the case of close-in detonations, the column lost their load carrying capacity and failed utterly. In case of near-field and far-field detonations, the column remains undamaged and can sustain more blast loads as represents in Figure 15 and 16. Results demonstrate that increasing the scaled standoff distance significantly reduces the amount of damage to the structural system.

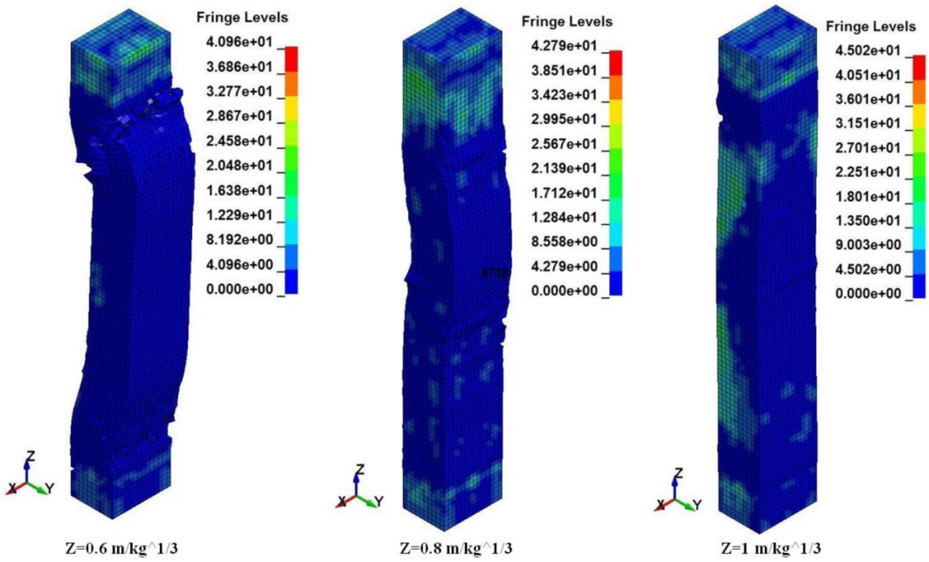


Figure 14. Effective stress plots under close-in detonation

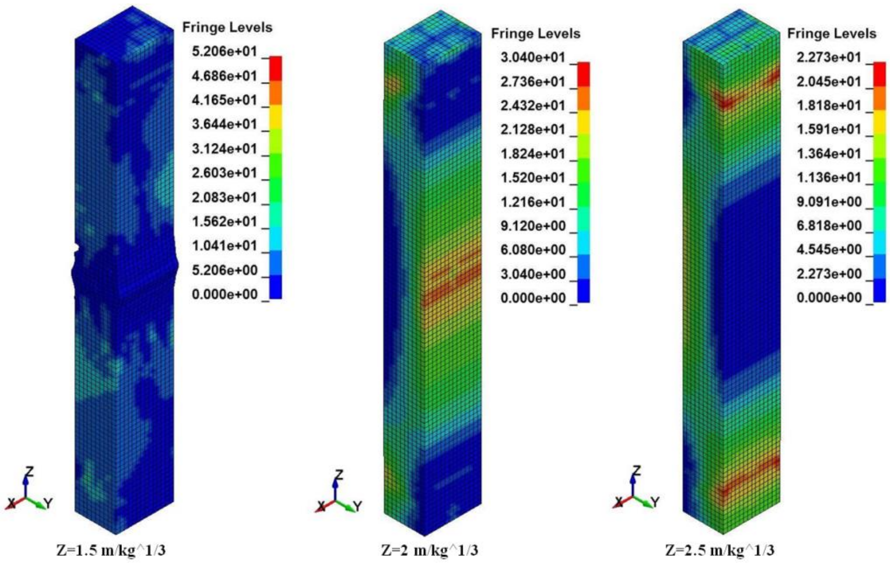


Figure 15. Effective stress plots under near-field detonation

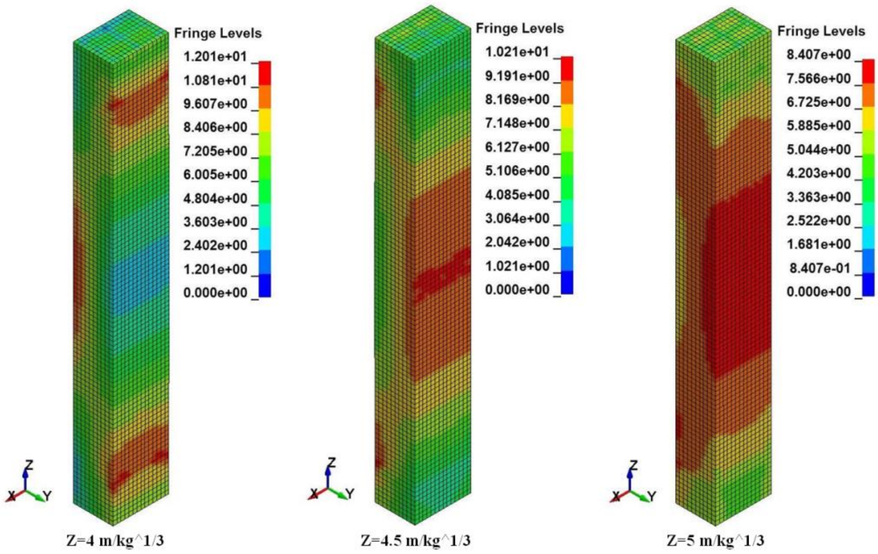


Figure 16. Effective stress plots under far-field detonation

The peak pressure is incredibly intense in close-in detonation. In this case, the period of the blast wave is relatively shorter than the natural period of the column, and the column respond mainly to the impulse of the blast load as shown in Figure 17. As a result, the impulse can be a better parameter than the peak pressure to design the target. According to Figure 17, when the scaled distance is increased, the pressure and impulse in the RC column are decreased. In case of far-field detonation, the peak pressure smaller than the one in the high-pressure range, impacts the RC column. Duration of the blast waves in far-field cases is remarkably more extended than the period of natural of the column shown in Figure 19. Therefore, the explosive load can be considered as a quasi-static load. In a quasi-static load, the response of the structure is a function of applied load and may reach to the maximum deflection before the blast pressure drops. Hence, the maximum deflection depends on the peak pressure and structural stiffness. In case of near-field, the response regime is called the dynamic regime and lies between the quasi-static and the impulsive regimes (see Figure 18). For this regime, the period of the blast waves is almost the same as the natural period of vibration of the column. Simulation of these types of dynamic responses is complicated. However, it is possible to approximate the response based on the impulsive and quasi-static cases.

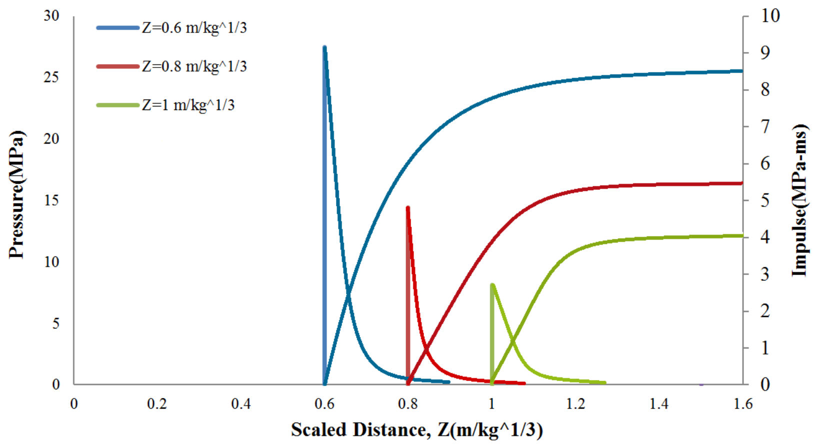


Figure 17. Pressure and impulse graphs at different Z under close-in detonation

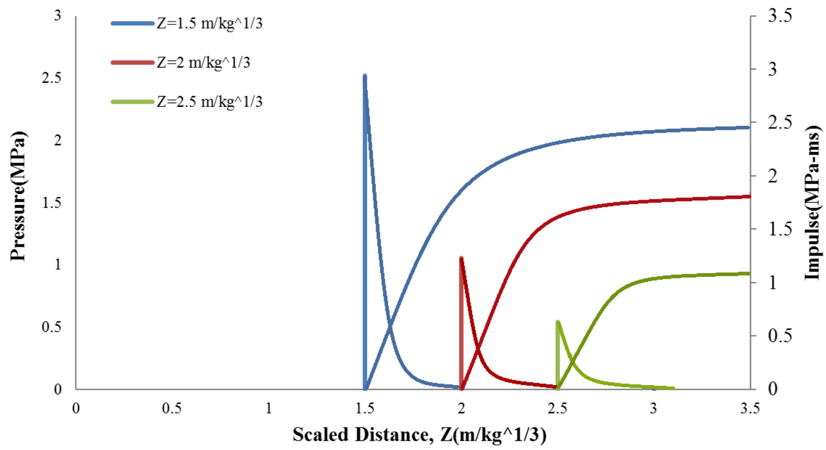


Figure 18. Pressure and impulse graphs at different Z under near-field detonation

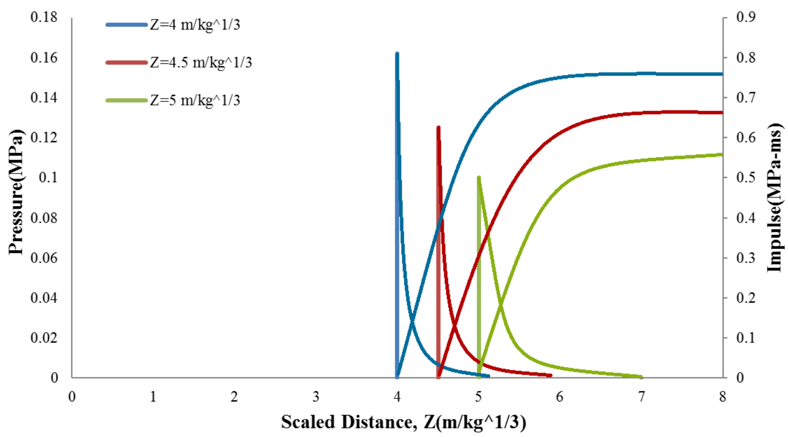
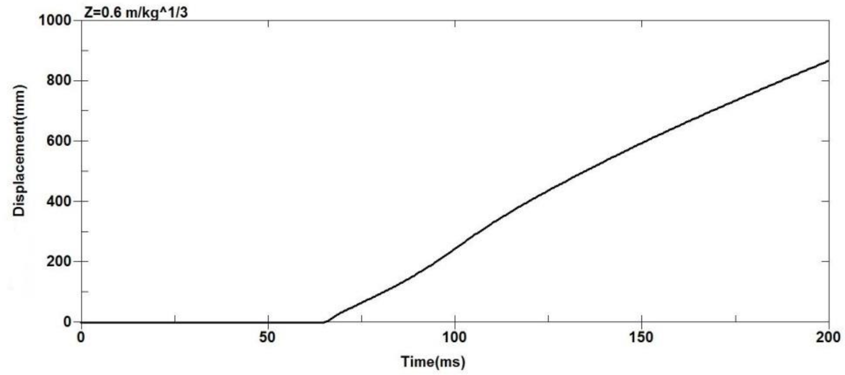
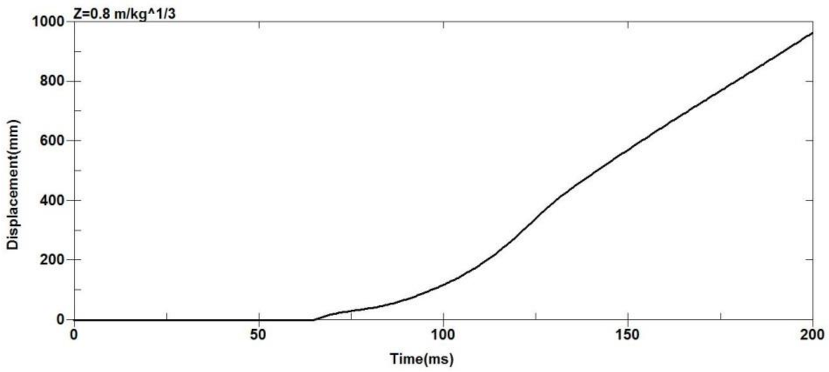


Figure 19. Pressure and impulse graphs at different Z under far-field detonation

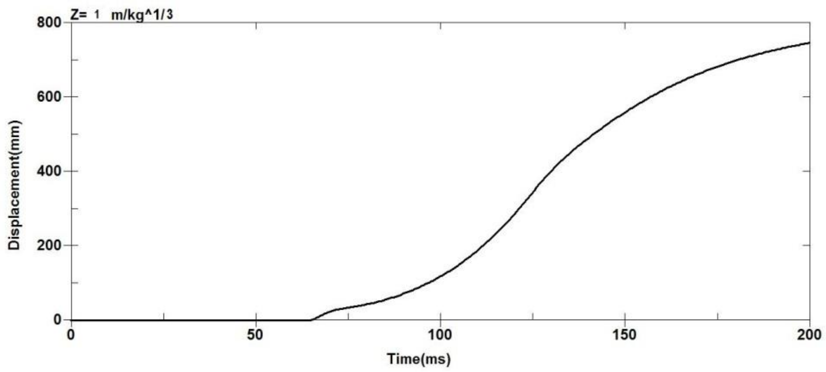
Deflections of the RC column subjected to different scaled distances over 200 ms simulation are presented in Figures 20-22. In the case of close-in detonation, the column fails due to the highly impulsive load. For such cases, significant structural deformations occur after the blast wave passed the structure. When the column is subjected to near-field detonations, the intensity of the blast loads reduced and the column sustain less blast damage as shown in Figure 21. At a higher scaled distance, the lateral displacements decreased significantly in the near-field detonations range. When the column was under far-field detonation, the peak deflection recorded decreased in comparison to near-field, and close-in detonations. Contour plots indicate less blast damage as represented in Figure 22.



(a)

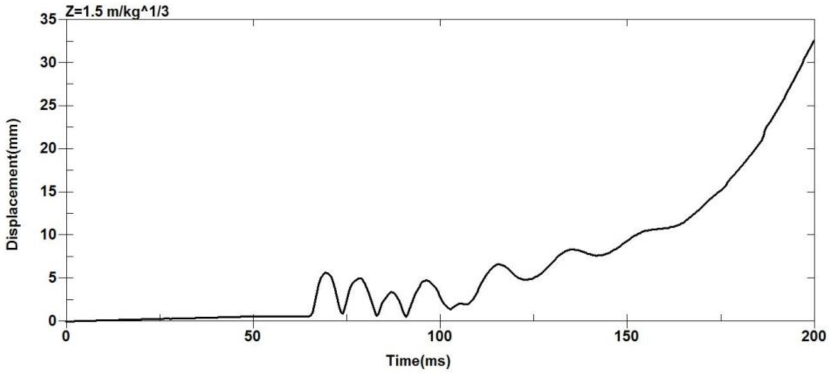


(b)

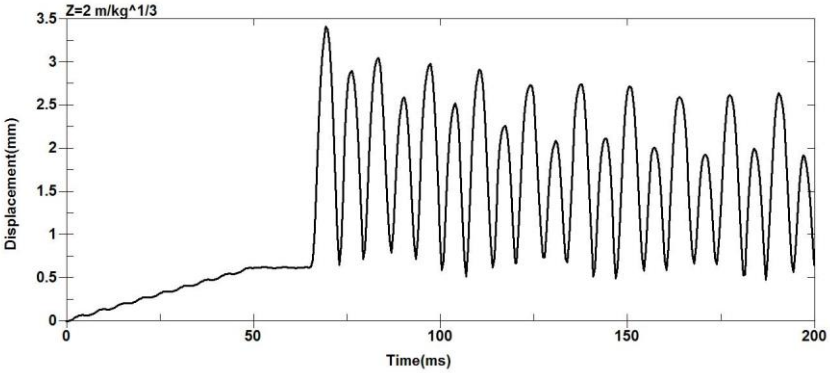


(c)

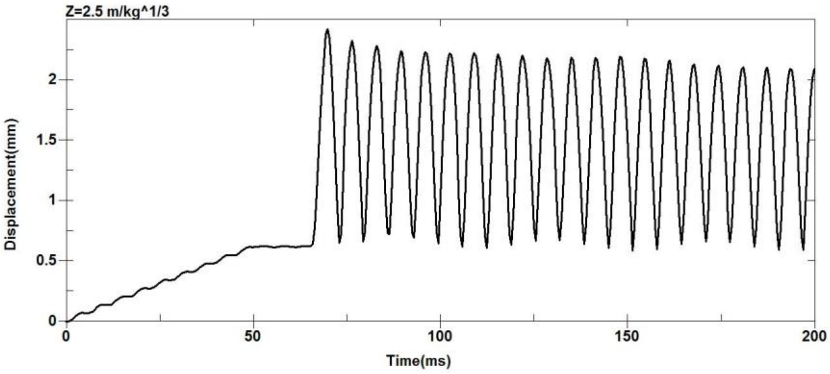
Figure 20. Displacement plots for the RC column at a) Z = 0.6 b) Z = 0.8 and c) Z = 1.0$\frac{m}{kg^{1/3}}$,



(a)

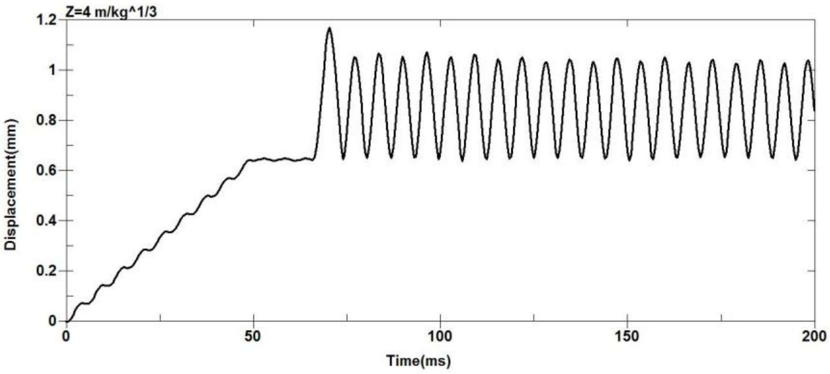


(b)

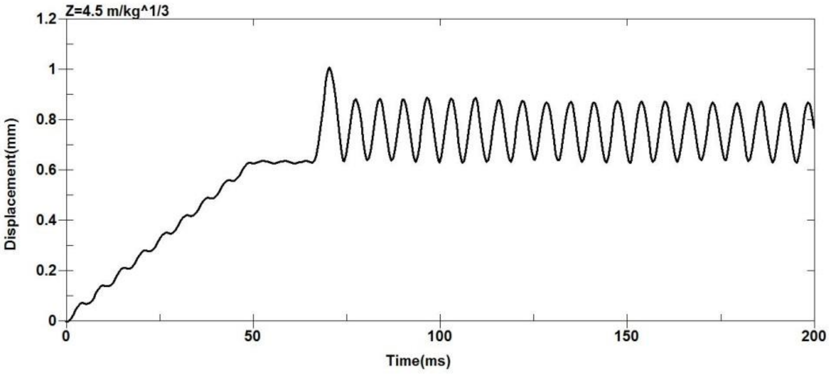


(c)

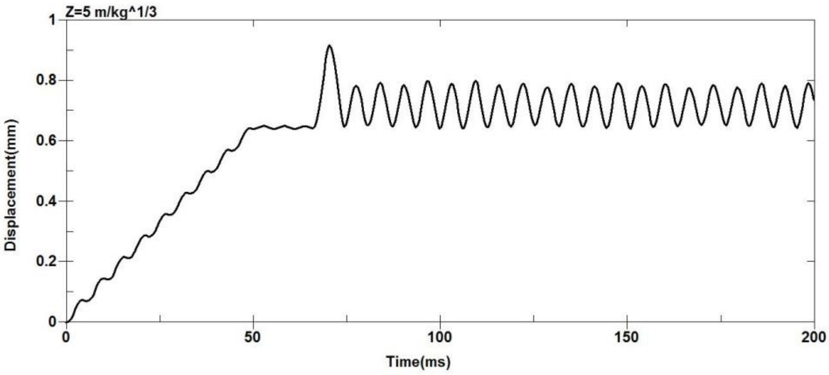
Figure 21. Displacement plots for the RC column at a) Z = 1.5, b) Z = 2 and c) Z = 2.5$\frac{m}{kg^{1/3}}$,



(a)



(b)



(c)

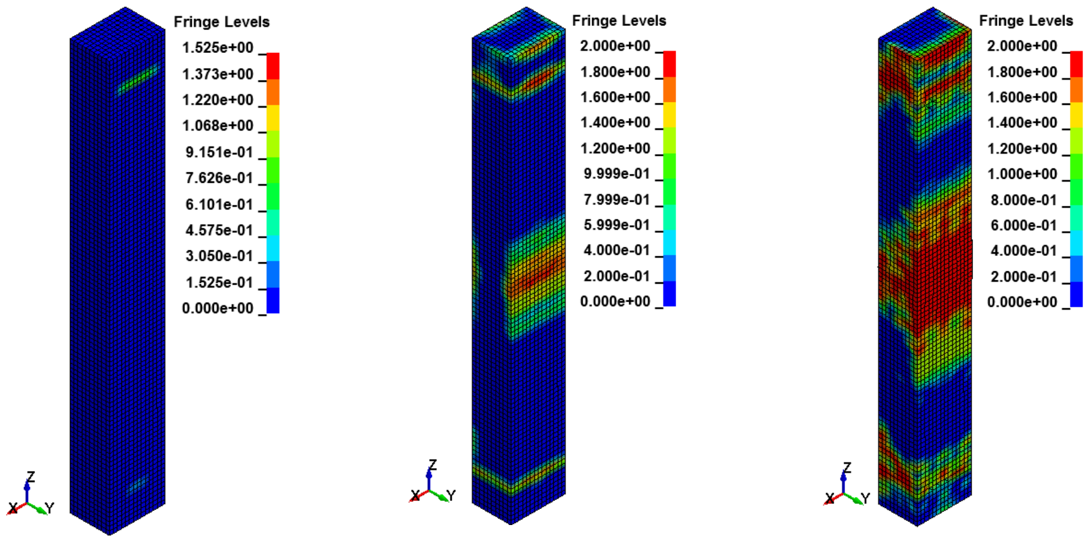
Figure 22. Displacement plots for the RC column at a) Z = 4, b) Z = 4.5 and c) Z = 5$\frac{m}{kg^{1/3}}$,

6.2. The response of the RC Column under Same Scaled Distance

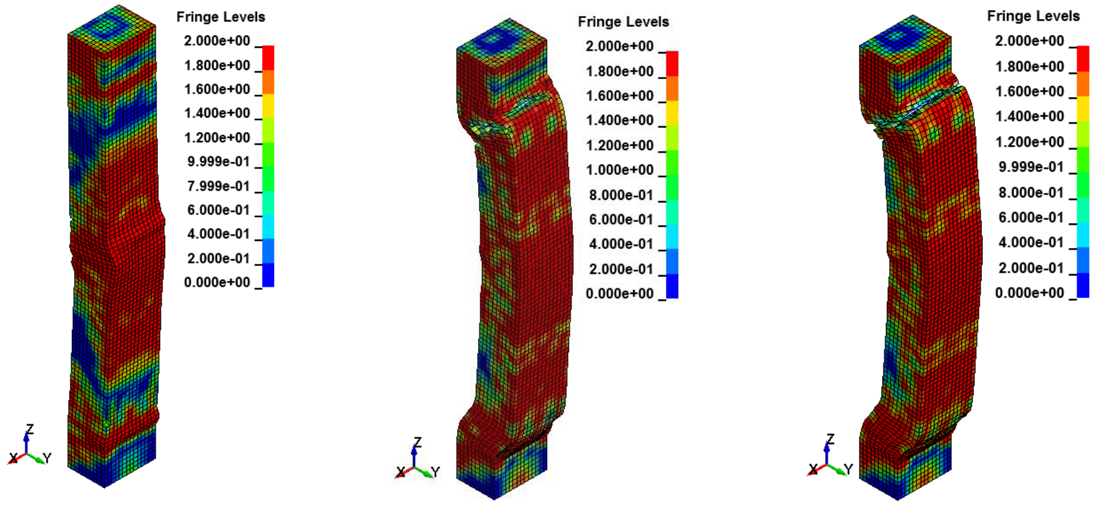
The numerical analysis is extended to investigate the behavior of the RC column at Z =0.95 $\frac{m}{kg^{1/3}}$ with different standoff distances and charge weights. In this section, the charge masses of 0.5, 5, 50, 597.2, 9330, and 15000 *kg* were used at the matching standoff distances of 0.753, 1.62, 3.5, 8, 20, and 23.58 *m*. Table 9 represents the range of charge weights and standoff distances at 0.95 $\frac{m}{kg^{1/3}}$. Figure 23 represents the dynamic behaviour of the RC column under same scaled distance at 100 ms time. As the scaled distance is constant in Figure 23, the level of damage increased with more charge weight and larger standoff distance. In this situation, the blast duration and blast impulse vary with different charge masses at the specific scaled distance. Heavier charges make longer blast loads. Therefore, at the same scaled distance, heavier charge produces higher impulse.

Table 9. The range for charge weights and standoff distances at 0.95 m/kg$^{1/3}$ scaled distances

Scaled distance ***(m/kg^1/3)***	**Charge weight** ***(kg)***	**Standoff distance** ***(m)***
0.95	0.5	0.753
	5	1.62
	50	3.5
	597.2	8
	9330	20
	15000	23.58



$R = 0.753\ m\ \&\ W = 0.5\ kg$ $R = 1.62\ m\ \&\ W = 5\ kg$ $R = 3.5\ m\ \&\ W = 50\ kg$



$R = 8\ m\ \&\ W = 597.2\ kg$ $R = 20\ m\ \&\ W = 9330\ kg$ $R = 23.58m\ \&\ W = 15000\ kg$

Figure 23. Response of the RC column under same scaled distance

6.3 Influence of Scaled Distance on the Damage Degree of RC Columns with Different Longitudinal Reinforcement Ratio

Numerical simulations were conducted to study the effect of scaled distance on the damage degree of RC columns with different longitudinal reinforcement ratio when subjected to explosive loads. The change in the longitudinal reinforcement ratio is accomplished by the change in the diameter of the longitudinal steel bar. The longitudinal reinforcement ratios in this study ranged from 0.011 to 0.028. Comparisons of the damage levels in the RC columns with different scaled distance and longitudinal reinforcement ratios are shown in Figure 24. Besides the column depth, the reinforcement of the column could also have significant influence on the damage degree of RC columns.

This outcome indicates that with the increase of the longitudinal reinforcement ratio, damage degree decreases as the scaled distance increases. The increase in longitudinal reinforcement significantly enhances the bending strength of the column. The damage level of the RC columns increases by 26% when the longitudinal reinforcement ratio decreases from 0.028 to 0.011. The fitted polynomial graph and contour plot are then expressed in the form of surface plots to illustrate the damage degrees of RC columns with different longitudinal reinforcement ratios under explosion loads is shown in Figure 25, and the corresponding equation is given below.

$$D = -1.6916 + (\frac{1}{\rho})^{0.082}(Z^{-0.578}) \quad (17)$$

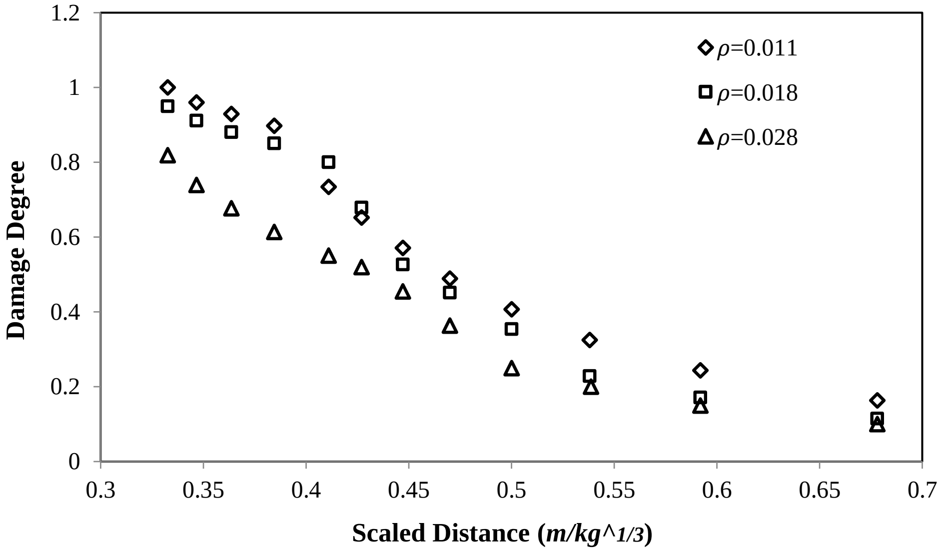
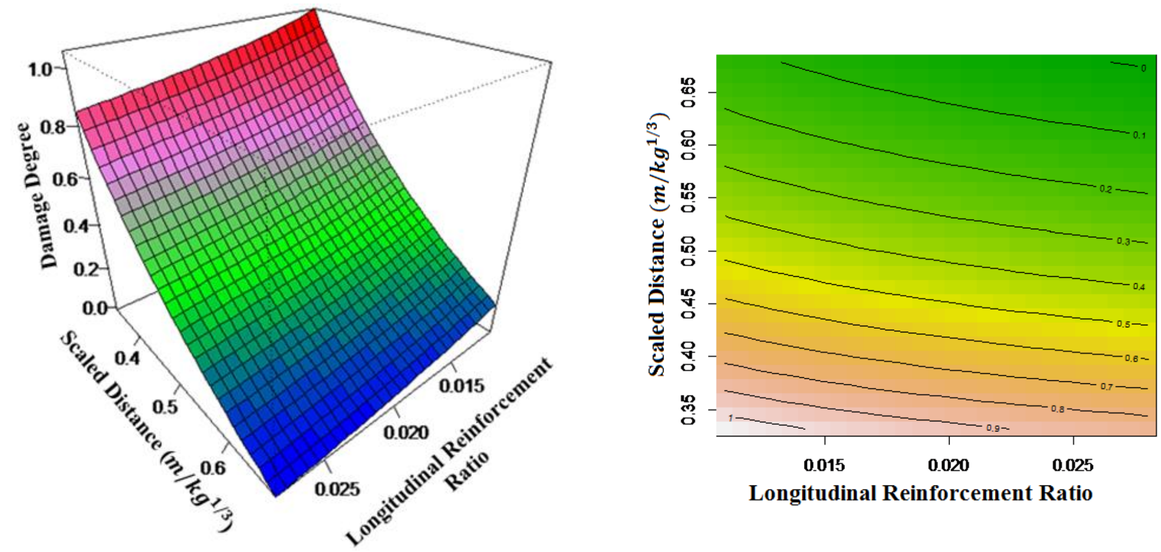


Figure 24 Damage degree in RC columns with different ρ and Z



(a) (b)

Figure 25 (a) The best fitted curve, and (b) contour plot to predict the level of damage with different ρ

Influence of Charge Weight on the Residual Capacity of RC Columns with Different Concrete Strength

In this section the effect of charge weight on residual axial load carrying capacity of the RC columns with different concrete strength was evaluated. The analysis to generate residual capacity of the RC columns consists of three stages: pre blast loading, blast loading, post blast loading stages. The axial load applied to the column in stage one and after that blast load is applied to the column after the time for stress equilibrium is attained along the length

of the column in the stage two and in the third stage Post-blast analysis is carried out to evaluate the residual capacity of the column. This simulates a displacement controlled load testing. The concrete strength can have a significant affect in increasing the residual axial load carrying capacity of the RC columns under explosive loads. The concrete strength was varied between 32 and 52 MPa. Figure 26 shows the effect of concrete strength on the residual axial load carrying capacity of the RC columns. It can be seen that the concrete strength efficiency of residual axial load carrying capacity of RC columns increases with augmenting concrete strength. Generally, residual axial load carrying capacity of RC columns improves with increasing concrete strength.
The best fitted boundary surface and counter plot for the residual axial load carrying capacity of RC column with different concrete strength is shown in Figure 27, and the corresponding equation is given below.

$$\boldsymbol{P_{residual}} = \mathbf{3046.85} + (\boldsymbol{f_c^{2.18}})(\boldsymbol{W^{-1.02}}) \tag{18}$$

Where $\boldsymbol{P_{residual}}$ is the residual axial load carrying capacity of RC column, f_c is the concrete strength and W is the charge weight.

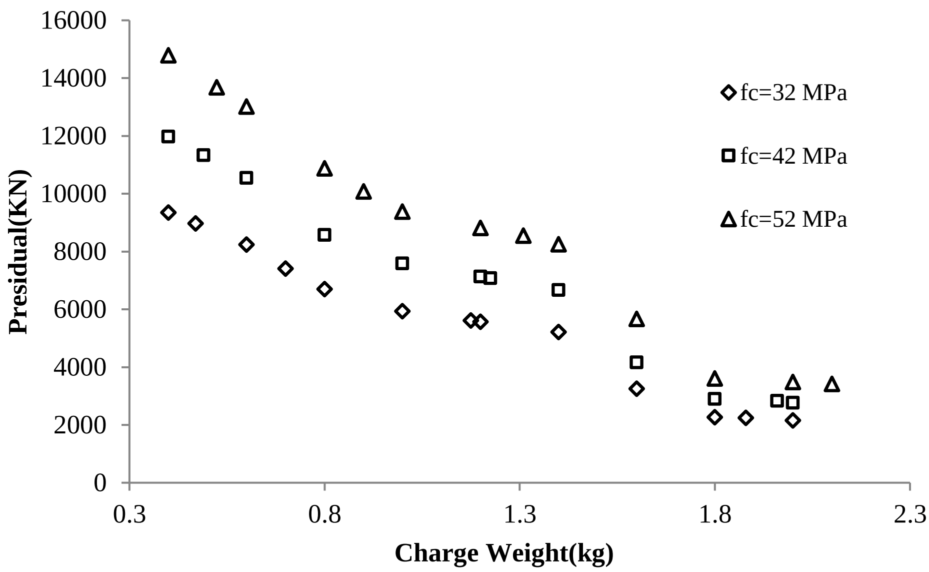
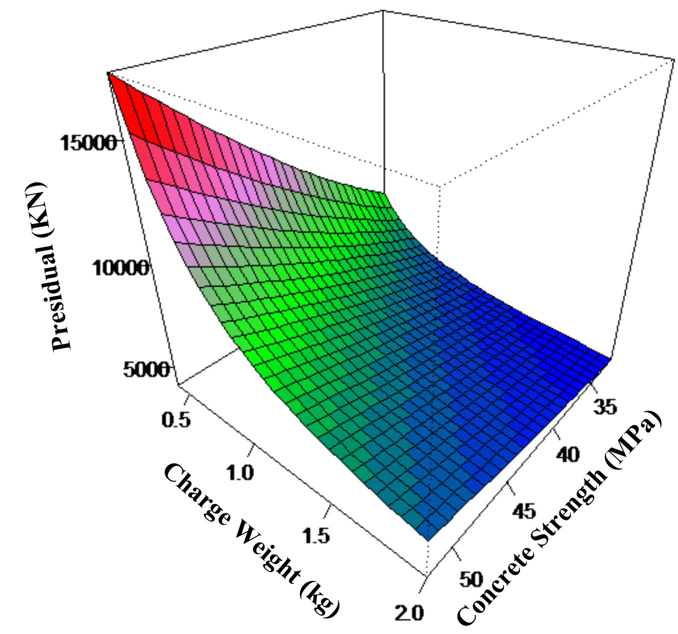
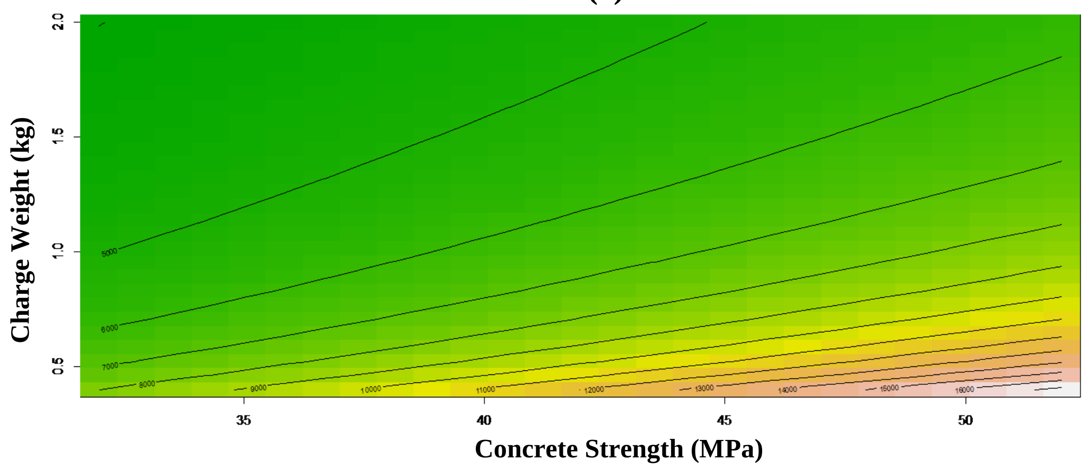


*Figure **26** Effects of concrete strength on the residual axial load carrying capacity of RC column with different charge weight*



(a)



(b)

*Figure **27** (a)The best fitted curve and (b) counter fringe for the residual axial load carrying capacity of RC column with different concrete strength*

Influence of Charge Weight on the Residual Capacity of RC Columns with Different width

The columns width range was taken between 500 mm and 900 mm to investigate the charge weight effect on the residual axial load carrying capacity of the RC columns under blast loads. Figure 28 shows the effect of column width on the residual capacity of the RC columns with various TNT charge weight. It can be seen that residual axial load carrying capacity of RC columns increase with the rise in column width. The results show that the residual axial load carrying capacity of RC column with low column width is significantly less than that of a column with high column width.

The best fitted boundary surface and counter plot for the residual axial load carrying capacity of RC column with different width is shown in Figure 29, and the corresponding equation is given below.

$$P_{residual} = 1307.17 + \left(w_i^{1.417}\right)\left(W^{-0.792}\right) \tag{19}$$

Where $P_{residual}$ is the residual axial load carrying capacity of RC column, w_i is the column width and W is the charge weight.

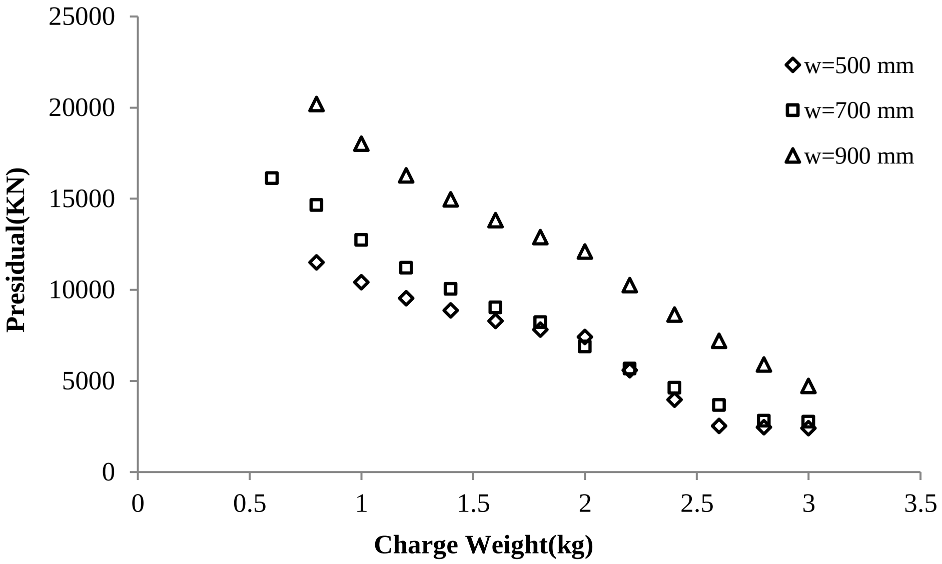
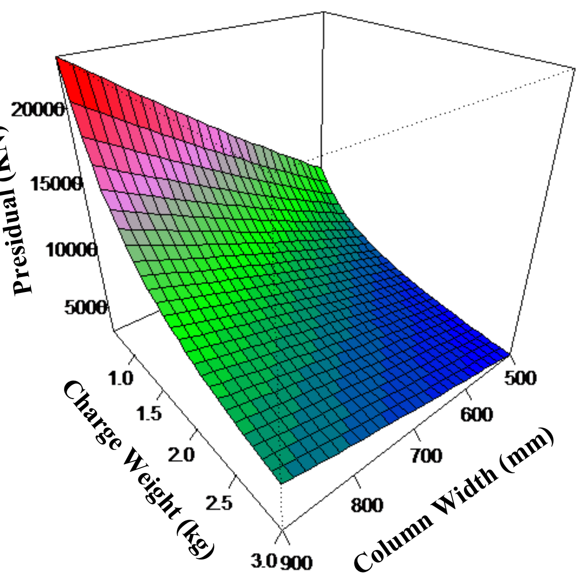
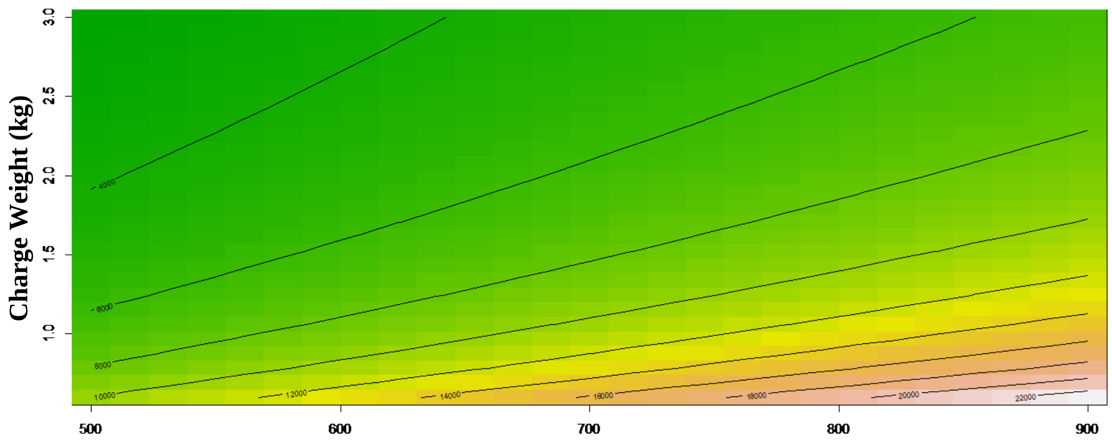


Figure 28 Effects of column width on the residual axial load carrying capacity of RC column with different scaled distances



(a)



(b)

Figure 29 (a)The best fitted curve and (b) counter fringe for the residual axial load carrying capacity of RC column with different columns width

7. Conclusion

In this research finite element analyses were performed to investigate the behavior of RC columns against blast detonations. The numerical simulations were validated against the blast field tests. The scaled distance was found a critical parameter to analyze the response of the RC column under explosive loads. The column experienced the maximum pressure and maximum impulse when the scaled distance was low. As a consequence, the column failed under intense impulsive regime loading. Also, results showed that higher scaled distant could decrease the damage level of RC column even further. Based on intensive numerical simulation data, analytical expressions are derived to predict damage degree and residual axial load carrying capacity of RC column in terms of the Scaled distance, charge weight, column width concrete strength and longitudinal reinforcement ratio. This research work and the conclusions drawn may be utilized for evaluation of the effect of an explosion on the RC column.

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Conflict of Interests The authors declare that there is no conflict of interests regarding the publication of this paper.

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