Structural Engineering and Mechanics, Vol. 36, No. 3 (2010) 301-319
DOI: http://dx.doi.org/10.12989/sem.2010.36.3.301

Progressive collapse analysis of a RC building subjected to blast loads

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(Received November 23, 2009, Accepted June 16, 2010)

Abstract. The paper seeks to explore some aspects of the current state of knowledge on progressive collapse in the technical literature covering blast loads and structural analysis procedure applicable to reinforced concrete (RC) buildings. The paper describes the progressive collapse analysis of a commercial RC building located in the city of Riyadh and subjected to different blast scenarios. A 3-D finite element model of the structure was created using LS-DYNA, which uses explicit time integration algorithms for solution. Blast loads were treated as dynamic pressure-time history curves applied to the exterior elements. The inherent shortcomings of notional member removal have been taken care of in the present paper by simulating the damage of structural elements through the use of solid elements with the provision of element erosion. Effects of erosion and cratering are studied for different scenarios of the blast.

Keywords: progressive collapse; blast pressure; finite element analysis; RC building.

1. Introduction

In the recent past, structures all over the world have become susceptible to the threat of terrorist attacks, accidental explosions and other unthought-of explosion related events (Luccioni *et al*. 2004, Mlakar *et al*. 1999, Mohamed 2006). Buildings and critical infrastructure vulnerable to explosions include government buildings, embassies, financial institutions, densely populated commercial structures, and other buildings of national heritage or landmarks. Consequently, a number of concerns have been raised on the vulnerability and behavior of these structures under extreme loadings. Blast loading or other unforeseen events can cause progressive collapse due to the damage of some key elements which can either make the structure unstable or trigger the failure of the main portions of the structural system. Progressive collapse is defined as the spread of an initial local failure from element to element, eventually resulting in the collapse of the entire structure or disproportionally large part of it. A blast generally results in a high-amplitude impulse loading, which lasts for a very short time and produces high pressure loading. The loads experienced by structural elements depend on size, geometry and the proximity of the explosion. Large amount of explosives at short distances from the structure can cause huge pressure forces, which cannot be

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accommodated in the design of any structure. Thus, other measures such as increasing the standoff distance and perimeter control can reduce the possibility of extreme damage to the structure. Recent developments in the efficient use of high strength building materials, innovative framing systems, larger floor spans and refinements in the analysis techniques have resulted in building structures with considerably smaller margin of safety that are more vulnerable to progressive collapse. The prediction of possible progressive collapse under specific conditions can provide very important information which could be used to prevent such failure of the structure.

Progressive collapse of buildings first attracted the attention of engineers from the structural failure of a 22-story precast concrete bearing wall apartment building at Ronan Point, London, UK, on 16 May 1968. A gas explosion in a corner kitchen on the 18th floor blew out the external wall panel and failure of the corner bay of the building propagated upward to the roof and downward almost to the ground level (Ellingwood *et al*. 2007). However, serious studies on progressive collapse started after the malevolent bombing of the Alfred P. Murrah Building in Oklahoma City on 19 April 1995 (Mlakar *et al*. 1999) and the terrorist attack on the World Trade Center in New York on 11 Sept. 2001. The literature on progressive collapse mitigation has thus expanded significantly and many building design codes incorporated provisions for the avoidance of disproportionate collapse. But currently, there are no provisions or recommendations in the Saudi Arabian standards that address the problem of progressive collapse. The resistance of buildings to progressive collapse has been an important task for the development of structural design codes. The approaches adopted by different codes and design strategies have been reviewed and discussed by many investigators (Dusenberry 2002, Ellingwood 2006, Kaewkulchai and Williamson 2004, Mohamed 2006, Nair 2006, Starossek 2006, Starossek and Wolff 2005). Important issues examined by investigators include abnormal events leading to progressive collapse, assessment of loads, analysis methods, and design philosophy. In recent years, the development of analysis methods for evaluating the progressive collapse potential of an existing or new building has been an imperative subject. Advantages and disadvantages of different approaches for progressive collapse analysis have been discussed by Marjanishvili and Agnew (2006) and Marjanishvili (2004). Detailed descriptions of a step-by-step, linear static procedure for progressive collapse analysis have been issued by the US General Service Administration (2003) and Department of Defense (2005).

Several investigators have investigated the progressive collapse potential of reinforced concrete (RC) framed and steel framed buildings (Bae *et al*. 2008, Fu 2009, Khandelwal 2009, Kim and Kim 2009, Lee *et al*. 2009, Paik and Kim 2008, Pekau and Cui 2006). Izzuddin *et al*. (2008) and Vlassis *et al*. (2008) proposed a simplified framework for progressive collapse assessment of multi-storey buildings, considering sudden column loss as a design scenario. It analyzed the non-linear static response with dynamic effects evaluated in a simple method. Tsai and Lin (2008) conducted the progressive collapse analysis of an earthquake-resistant RC building following the linear static analysis procedure recommended by the US General Service Administration (2003). Asprone *et al*. (2010) proposed a probabilistic model for multi-hazard risk associated with the limit state of collapse for a RC structure subjected to blast threats in the presence of seismic risk. The blast and seismic fragilities of a generic four-storey RC building located in seismic zone were calculated and implemented in the framework of a multi-hazard procedure, leading to the evaluation of the annual risk of collapse. In a study carried out by Luccioni *et al*. (2004), the analysis of the structural failure of a RC building caused by a blast load showed good agreement between actual damage and that obtained numerically. This demonstrates that the simplifying assumptions made for the structure and materials are allowable for this type of analysis. Lan *et al*. (2005) carried out an investigation to develop a high fidelity physics based finite element model for RC columns for computing responses in case of close-in suitcase bombs. The model was also used to compute damage curves for different tie spacing for a specific type of column. It was shown that the tie spacing plays a significant role in the post blast loading capacity of RC columns.

The present study seeks to explore some aspects of the current state of knowledge on progressive collapse in the technical literature covering blast loads and structural analysis procedure applicable to RC buildings. The performance of a RC building against blast loads is assessed for its vulnerability to progressive collapse. A significant shortcoming of the notional member removal provisions (Izzuddin *et al.* 2008) is the assumption of a static structural response, when the failure of vertical members under blast load is a highly dynamic phenomenon. This shortcoming has been taken care of in the present paper by simulating the damage of structural elements through the use of solid elements with the provision of element erosion.

2. Blast loading

When an explosive charge is detonated in air, the rapidly expanding gaseous reaction products compress the surrounding air and move it outwards with a high velocity thus forming a blast wave. The abrupt pressure increase at the shock front is followed by a quasi-exponential decay back to ambient pressure and a longer negative phase in which the pressure is less than ambient pressure (Fig. 1). The overpressure-time history of a blast wave is usually described by Friedlander's equation (Kinney and Graham 1985)

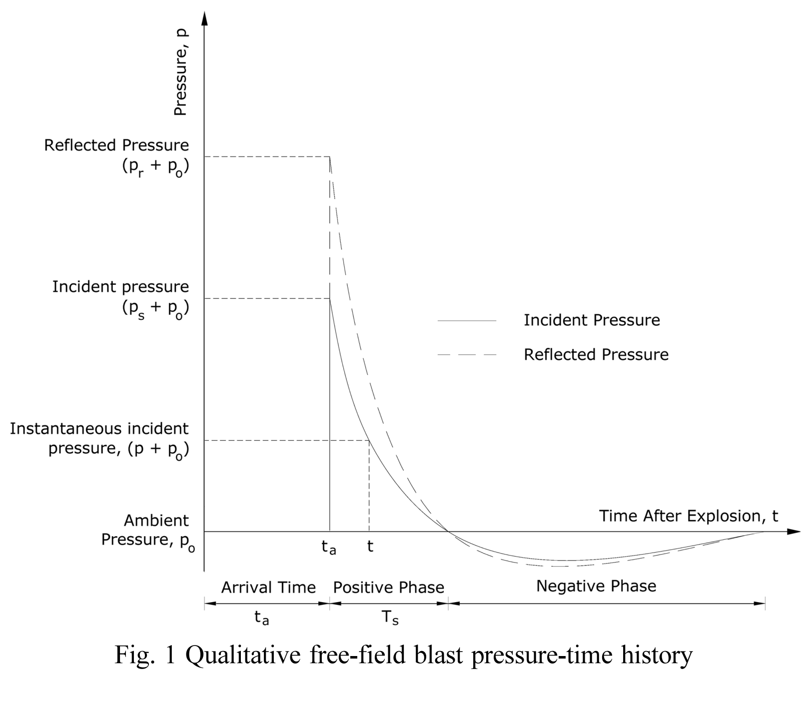


Fig. 1 Qualitative free-field blast pressure-time history

$$p = p_s(1-x)e^{-bx} \tag{1}$$

where, $x = \dfrac{(t-t_a)}{T_s}$

where t is the time, p is the static overpressure at time t, p_s is the peak static overpressure, T_s is the duration of the positive phase, t_a is the arrival time and b is a positive constant called the waveform parameter that depends on the peak static overpressure. The corresponding absolute pressure may be obtained by adding atmospheric pressure (i.e., 101.3 kPa) to the overpressure values.

There is a limit of one atmosphere negative pressure (i.e., complete vacuum or zero absolute pressure) which may occur during the negative phase of a blast wave, and if the positive overpressure exceeds this value greatly, it is unlikely that the negative phase impulse will pose a significant hazard in comparison with the positive impulse. For this reason, negative phase impulse is often ignored by investigators especially for rigid structures like RC frames (TM5-1300 1990). Some important blast load parameters required for analysis are explained in subsequent subsections.

The blast energy from an explosion imparted to a building may be obtained from impulse which may be obtained by integration of pressure-time curve

$$I_s = \int p\,dt \tag{2}$$

The variable p in the above expression is given by Eq. (1). Both the negative and positive phases of the pressure-time waveform contribute to impulse but the negative phase being insignificant, as discussed above, is usually ignored.

2.1 Blast scaling law

The most widely used approach for blast wave scaling is the cube root scaling law of Hopkinson (1915) and Cranz (1926) developed independently, which establishes that similar explosive waves are produced at identical scaled distances when two different charges of the same explosive and with the same geometry are detonated in the same atmosphere. Thus, any distance R from an explosive charge W can be transformed into a characteristic scaled distance z

$$z = \frac{R}{W^{1/3}} \tag{3}$$

where W is the equivalent TNT charge weight expressed in kg and R is the distance in m. The use of z allows a compact and efficient representation of blast wave data for a wide range of situations.

2.2 Incident free-field peak static overpressure

A large number of models for calculating the blast wave parameters, especially the peak incident overpressure, are available in literature. These models are either based on theoretical analysis, numerical calculations or on a large number of experimental results. Its variation for a free air burst as a function of the scaled distance are presented in the form of graphs by Baker *et al*. (1983), Mays and Smith (1995) and TM5-1300 (1990) manual. Whereas, Brode (1955), Henrych (1979) and Kinney and Graham (1985) have formulated the problem of explosion theoretically and obtained the variation of overpressure in near and far fields using the theory of gas dynamics. Some of the useful models predicting peak overpressure p_s in kPa as a function of y (= $1/z$ with unit of z as m/kg$^{1/3}$), are:

Brode (1955)

$$p_s = 670y^3 + 100 \quad \text{for} \quad p_s \geq 1000 \text{ kPa} \tag{4}$$

$$p_s = 97.5y + 145.5y^2 + 585y^3 - 1.9 \quad \text{for} \quad 10 \leq p_s \leq 1000 \text{ kPa} \tag{5}$$

Henrych (1979)

$$p_s = 1407.2y + 554.0y^2 - 35.7y^3 + 0.625y^4 \quad \text{for} \quad 0.05 \leq z \leq 0.3 \text{ m/kg}^{1/3} \tag{6}$$

$$p_s = 619.4y - 32.6y^2 + 213.2y^3 \quad \text{for} \quad 0.3 \leq z \leq 1.0 \text{ m/kg}^{1/3} \tag{7}$$

$$p_s = 66.2y + 405.0y^2 + 328.8y^3 \quad \text{for} \quad 1.0 \leq z \leq 10.0 \text{ m/kg}^{1/3} \tag{8}$$

Mills (1987)

$$p_s = 108y - 114y^2 + 1772y^3 \tag{9}$$

Kinney and Graham (1985)

$$p_s = \frac{82000\left[1 + \left(\frac{z}{4.5}\right)^2\right]}{\sqrt{1 + \left(\frac{z}{0.048}\right)^2}\sqrt{1 + \left(\frac{z}{0.32}\right)^2}\sqrt{1 + \left(\frac{z}{1.35}\right)^2}} \tag{10}$$

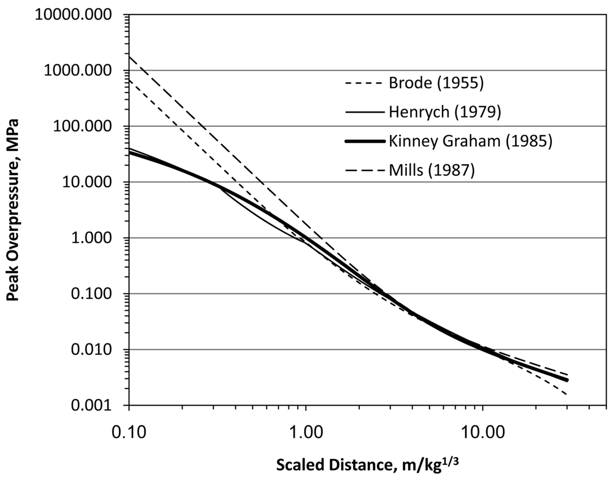


Fig. 2 Comparison of peak static overpressure prediction models

The variation of peak blast overpressure with scaled distance for the above formulae has been plotted in Fig. 2. The smallest scaled distance of 0.054 m/kg$^{1/3}$ represents the radius of the spherical TNT explosive, which is usually taken as a limit to the blast pressure prediction (TM5-1300-1990). However the minimum value of scaled distance taken in the figure is 0.1 m/kg$^{1/3}$. It is seen from the figure that the prediction of the simplest model of Mills is much higher than others. The prediction by Brode's model for smaller values of z is very high compared to Henrych and Kinney-Graham models. The diverging trend in the near field is probably due to the complexity of blast phenomena close to the charge (Smith and Hetherington 1994). The model implemented in ConWep (1990) is the one proposed by Kingery-Bulmash (Smith and Hetherington 1994).

2.3 Duration of air blast and waveform parameter

The description of loading curve represented by Fig. 1 requires the positive phase duration of blast waves, T_s. The blast pressure given by Eq. (1) being asymptotic to the time axis, the duration of negative phase duration obtained mathematically from the model is infinite. The efforts have been made to capture the negative phase of blast but remained inconclusive (Rose and Smith 2002). However, for practical purposes, the duration of the negative phase may be taken as four times the duration of the positive phase.

The positive phase duration has been presented in the form of charts (TM5-1300 1990), whereas Kinney and Graham (1985) have proposed a model

$$T_s = \frac{980 W^{1/3}\left[1+\left(\frac{z}{0.54}\right)^{10}\right]}{\sqrt{1+\left(\frac{z}{0.02}\right)^{3}}\sqrt{1+\left(\frac{z}{0.74}\right)^{6}}\sqrt{1+\left(\frac{z}{6.9}\right)^{2}}} \tag{11}$$

The units of variables T_s, W and z in the above equation are ms, kg and m/kg$^{1/3}$ respectively. The values of blast waveform parameter b, required in Eq. (1), are given in Table 1 (Smith and Hetherington 1994).

2.4 Reflected pressure

When the blast waves encounter an infinite large wall on which they impinge at zero angle of

Table 1 Blast waveform parameters (Smith and Hetherington 1994)

z	b
0.4	8.50
0.8	10.00
1.5	3.50
2.0	1.90
5.0	0.65
10.0	0.20
20.0	0.12

incidence, they are normally reflected. All flow behind the wave is stopped and pressures are considerably greater than incident.

The peak reflected pressure p_r can be obtained from Rankine-Hugoniot relationships for an ideal gas and gives (Mays and Smith 1995)

$$p_r = 2p_s\left(\frac{7p_o+4p_s}{7p_o+p_s}\right) \tag{12}$$

The magnification factor for the peak reflected pressure thus varies from 2 to 8. In the absence of more accurate prediction methods, the reflected impulse can be estimated by assuming similarity between the time histories of incident overpressure and normally reflected overpressure, thus giving (Baker *et al*. 1983)

$$I_r = I_s\left(\frac{p_r}{p_s}\right) \tag{13}$$

where I_s is the incident blast impulse given by Eq. (2) and I_r is the reflected impulse.

3. Relevant codes and standards

There are several codes and standards (NBCC 1996, ISC 2001, GSA 2000, ASCE-7 2005, DoD 2005, IBC 2000) that provide insight into regulatory approaches employed to provide general structural integrity to buildings, with the intent to reduce the potential for progressive collapse. The provisions of some of the important codes and standards are discussed in the subsequent subsections.

3.1 National building code of canada (NBCC 1996)

This code has addressed progressive collapse in some form for decades. It requires that: "Structural systems for buildings shall be designed to minimize the probability that an initial local failure of a structural element, caused by an abnormal event or severe overload, will spread to other structural members and precipitate the collapse of a disproportionately large portion of the structure".

Measures suggested to prevent widespread collapse include: control of accidental events, designing key elements to resist accidental events, designing adequate ties, providing alternate paths of support, and compartmentalizing the structure to limit the spread of a collapse. Within these general descriptions of preventative measures are most of the specific means listed in earlier editions of the Commentary.

3.2 General services administration (GSA 2003)

The GSA publishes guidelines that include recommendations to address the potential for progressive collapse in federal buildings. A portion of the prescribed procedure involves identifying whether the structure under consideration must be evaluated for the potential of progressive collapse and, hence, requires a detailed analysis. This reference suggests that sophisticated analyses (e.g., nonlinear dynamic finite element analyses, linear dynamic finite element analyses, etc.) may be used to determine the potential for progressive collapse, but warns that such analyses are complex, costly, and sensitive to small changes in assumptions. However, to facilitate decisions about survivability, this reference reproduces a table of maximum allowable ductility and/or rotation limits for many structural components of various construction types.

The analysis and design for resistance to progressive collapse are described for new as well as existing construction. Structural features that are encouraged in new construction include the use of redundant lateral and vertical force resisting systems, ductile structural elements and detailing, designing to resist load reversals, and prevention of shear failure. The Code addresses procedures for minimizing the potential for progressive collapse, analysis considerations and loading criteria for typical structural configurations such as framed or flat plate structures and shear/bearing wall structures, and atypical structures. Guidance regarding material properties, modeling, and redesign of structural elements are also included. For typical systems, this reference recommends linear elastic, static analyses for the instantaneous removal of some critical first-floor structural elements. The live load for the analysis is taken as 25% of design live load.

The potential for progressive collapse is determined by the calculation of a Demand-Capacity Ratio for each primary and secondary structural element. For high potential against progressive collapse, Demand-Capacity Ratio should be greater than 2 for typical buildings.

3.3 American society of civil engineers (ASCE-7 2005)

The Minimum Design Loads for Buildings and Other Structures, ASCE-7 is a standard maintained by the American Society of Civil Engineers. It suggests that: “Buildings and structural systems shall provide such structural integrity that the hazards associated with progressive collapse, such as that due to local failure caused by severe overloads or abnormal loads not specifically covered herein, are reduce to a level consistent with good engineering practice”.

For enhancement of general structural integrity, the Code requires (i) Good plan layout, (ii) Integrated system of ties among the principal structural elements, (iii) Returns on walls, (iv) Design of floor slabs for alternate load path, (v) Load-bearing interior partitions, (vi) Catenary action in floor slab, (vii) Beam action of walls, (viii) Redundant structural systems for secondary load path, (ix) Ductile detailing, (x) Additional reinforcement to resist blast loads, (xi) Use of compartmentalized construction in combination with special moment resisting frames.

3.4 ISC security criteria (ISC 2001)

The Interagency Security Committee (ISC) handled the problem of progressive collapse analysis indirectly and the focus is on resistance to blast events. This document refers the reader to the American Society of Civil Engineers Standard ASCE-7 for details on prevention of progressive collapse.

In general, avoidance of progressive collapse is addressed by identifying scenarios such as: “if local damage occurs, the structure would not collapse or be damaged to an extent disproportionate to the original cause of the damage”. This could be achieved by “designing for the loss of a column for one floor above grade at the building perimeter without progressive collapse.” This requirement has been included to “ensure adequate redundant load paths in the structure should damage occur for whatever reason”.

This reference suggests that designs to prevent progressive collapse be based on dead load plus a realistic estimate of live load for the structure that could be as low as 25% of the code-prescribed live load. Further, the reference suggests that design should use ultimate strengths with dynamic enhancements based on strain rates, and acknowledges that responses generally are post-elastic.

The Code also focuses on considerations to mitigate the effects of blast on structures that contribute to the reduction of the risk of progressive collapse. These include design of components with reinforcement patterns that support loads in directions other than the primary loading directions, ductile detailing of connections, special shear reinforcement to allow large post-elastic behavior, redundancy and alternative load paths etc.

4. Building description

In order to predict the response of a RC structure subjected to blast loading, a typical RC commercial building in the city of Riyadh was selected for the study. The building is ten storey high including two storeys of basement. The structural system is comprised of flat slab system of average thickness as 250 mm and octagonal columns with 250 mm side. There is a glass façade in the first and second storeys above ground, whereas all other stories have masonry façade with glass windows. One important feature of the building is the presence of a low height bay surrounding the main core of the building which may act as a sacrificial corridor for outside blast (Fig. 3).

5. Finite element analysis

The 3-D finite element model was developed using a general-purpose pre-processor FEMB. LS-DYNA (2007), a general-purpose finite element program, was employed for blast analysis. The blast analysis requires the modeling of all the structural as well as the non-structural elements as they play an important role in the propagation of the blast pressure waves.

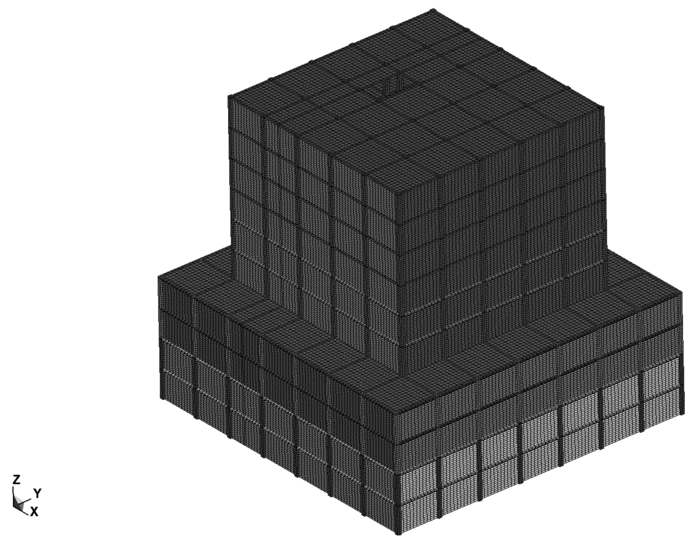


Fig. 3 Finite element geometry of the whole building

5.1 Model geometry

In order to mimic the real behavior of RC structures, it is imperative that the structural elements be modeled using solid elements. For this reason, a 3-D model of the building was created using 8-node reduced integration solid elements. However, for keeping the nodes within the limits of the software (i.e., 250,000), all elements could not be modeled using solid elements and thus shell elements were also used. The RC slabs of the top four stories were modeled using 4-node shell elements and all other slabs were modeled using 8-node hexahedron solid elements. However, the columns for the entire building were modeled using solid elements. For modeling purposes, the octagonal columns were replaced by an equivalent square cross-section. Each of the columns shared the same meshing pattern for its full height. Glass and masonry facades as well as the retaining walls in the basement were also modeled as 4-node shell elements. The Belytschko-Tsay (1981) element formulation was used for all shell elements. The finite element geometry of the building is presented in Fig. 3.

5.2 Material modeling

Winfrith concrete model (Broadhouse 1995) was used for all solid elements. The Winfrith concrete model is a smeared crack, smeared rebar model, with many important features of concrete such as fracture energy, strain rate effects etc. This model has been extensively used in numerous finite element models to predict the structural response to blast and impact loads and has been validated against experimental results. The kinematic plasticity model was assigned to shell elements of RC slabs, retaining walls and exterior façade. The uniaxial cylinder compressive strength of concrete was taken as 30 MPa for all of the structural elements.

5.3 Erosion

The erosion option provides a way of including failure to the material models. This is not a material or physics based property; however, it lends a great means to imitate concrete spalling phenomena and produce graphical plots which are more realistic representations of the actual events. By activating this feature, the eroded solid element is physically separated from the rest of the mesh. Material failure was simulated by element erosion at a specific plastic strain; thus, whenever an element reaches this critical value, it is removed from the computation (Blankenhorn 2007, Mattern *et al*. 2006, Xu and Lu 2006). This erosion model represents a numerical remedy to distortion, which can cause excessive and unrealistic deformation of the mesh. The application of erosion to the simulated model requires calibration with experimental results; however, in the absence of experimental validation, the consequence of possible discrepancy in the erosion specified is limited. This is because the damage level of the concrete material is basically governed by the material model itself.

5.4 Cratering effect

A crater is defined as a hole in the ground formed by an explosion. The primary variables which govern crater prediction are: amount and type of explosive, depth of burst and type of material in which the cratering occurs. Depending on the explosive type and depth of burst, the crater diameter and depth was predicted using the software ConWep (1990), which employs empirical models for the predictions. It was observed from this study that significant portion of the retaining walls gets exposed due to cratering. For this reason, in order to include the effect of cratering into the model, the retaining walls for the first basement level were taken as contact segments for the blast interface.

5.5 Solution strategy

LS-DYNA uses explicit time integration algorithms for solving the problems, which are less sensitive to machine precision than other finite element solution methods. The benefits of this are greatly improved utilization of memory and disk. An explicit FE analysis solves the incremental procedure and updates the stiffness matrix at the end of each increment of load (or displacement) based on changes in geometry and material. The termination time of 4 second was set in order to realize the complete blast-related response of the structure.

5.6 Application of load

The load application process in LS-DYNA is time-history dependent. The gravity load consisting of self-weight and superimposed dead and live loads was applied as a ramp function with a ramp time of 0.5 sec as displayed in Fig. 4. Based on trial analysis, the ramp duration of 0.5 sec was found sufficient to eliminate the oscillatory dynamic response of the building due to gravity loading. An explosion of 1000 kg equivalent weight of TNT was set to trigger at 0.5 sec. This feature of applying blast loading is available in LS-DYNA through the LOAD_BLAST card where the location and the charge weight of the blast may be specified. The package calculates the blast pressure distribution on the contact segments using ConWep (1990). The quantity of explosive was selected assuming a large vehicle as a carrier. The vertical height of the blast location was taken as 1m above ground because the explosive is assumed to be carried in a vehicle. Thus, the shock transmitted to the building through ground gets diminished due to which it has been ignored in the analysis.

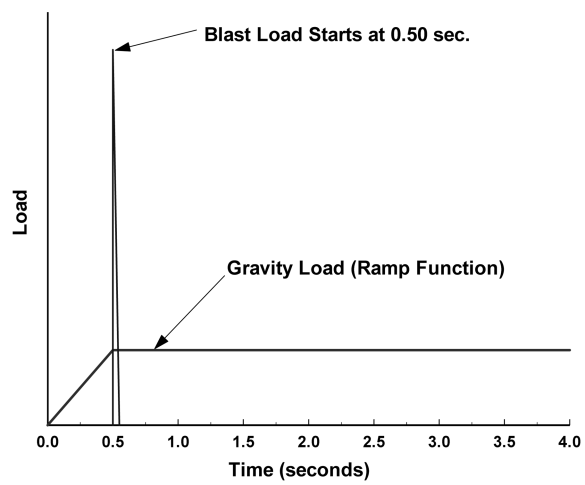


Fig. 4 Loading application for gravity and blast loads

6. Analysis for different blast scenarios

Blast load is generally impulse type high amplitude loading which lasts for a very short period of time, usually measured in milliseconds. But it produces an extremely high pressure loading over the structure. The loads applied by the blast are usually local, and only those elements closest to the blast may be directly impacted. Structural elements which are far from the blast site may experience little or no damage due to the sharp attenuation of blast energy with distance. The impact forces experienced by the structural elements depend upon the size and proximity of the explosion. The damage caused by the pressure may cause failure of exterior walls, slab system and columns resulting in the spread of the localized failure from element to element thereby resulting in a disproportionate total damage. In order to ascertain the critical scenarios of blast loadings for the progressive collapse of the structure, a preliminary study was carried out by modeling the structure

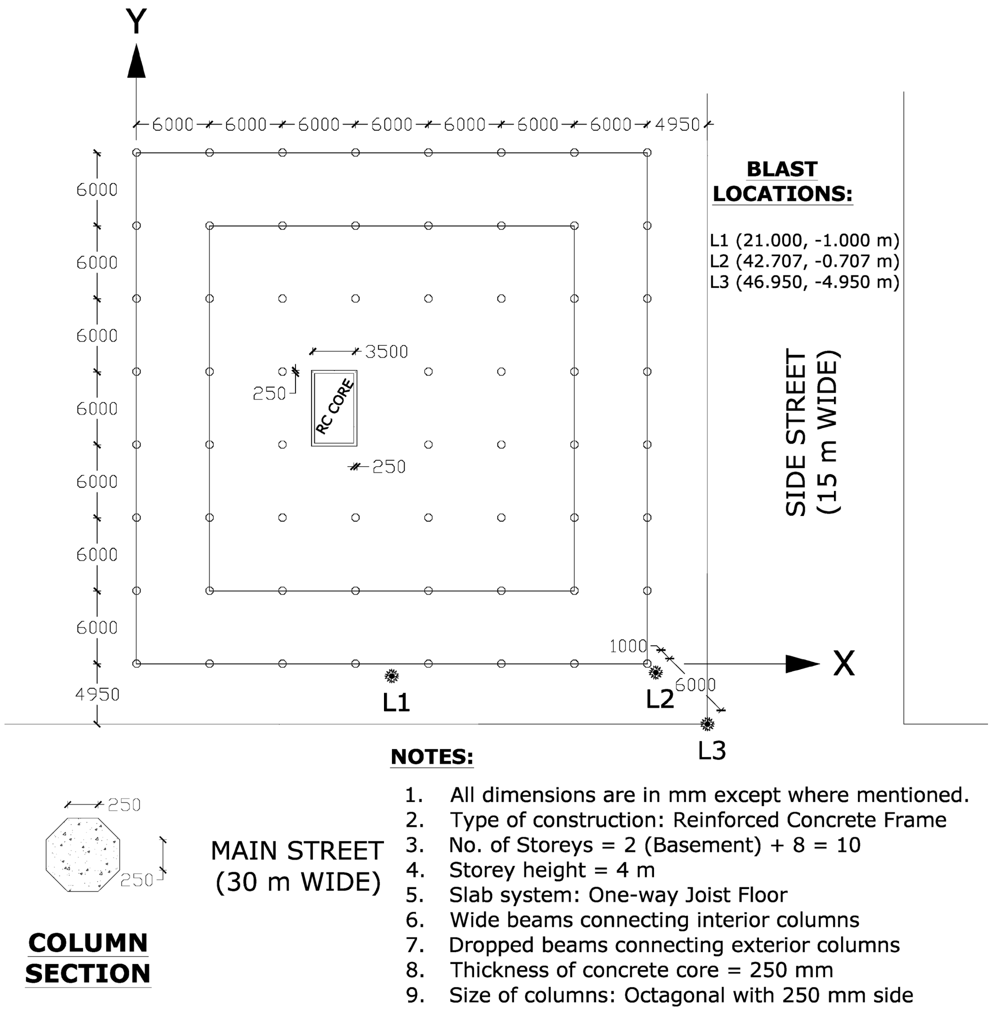


Fig. 5 Location of blast charge for different scenarios

using beam and shell elements and analyzing the effects of different blast scenarios on that structure. The selection of these scenarios was based on the layout of the building with respect to the streets, the standoff-distance provided, and the available access to the building. From the preliminary study, the critical scenarios for the progressive collapse of the structure are:

Scenario 1: Charge placed at the middle of the building side facing the main street at a distance of 1 m (shown as location L1 in Fig. 5) with cratering effect included.

Scenario 2: Charge placed diagonally at a distance of 1 m on the left hand corner of the building side facing the main street (shown as location L2 in Fig. 5) with cratering effect included.

Scenario 3: Charge placed diagonally at a distance of 7 m on the left hand corner of the building side facing the main street (shown as location L3 in Fig. 5) with cratering effect included.

Scenario 4: Charge placed diagonally at a distance of 1 m on the left hand corner of the building side facing the main street (shown as location L2 in Fig. 5) with no cratering.

The fourth scenario is same as scenario 2 in terms of charge placement but with a difference that the cratering was not included in this scenario in order to study the effect of cratering. All these scenarios were applied to the building and the effects of the blast were analyzed. Fig. 5 shows the locations of the blast for all of the scenarios. The four scenarios considered in the study are with the objective of studying the impact of location of blast, standoff distance and the cratering effect.

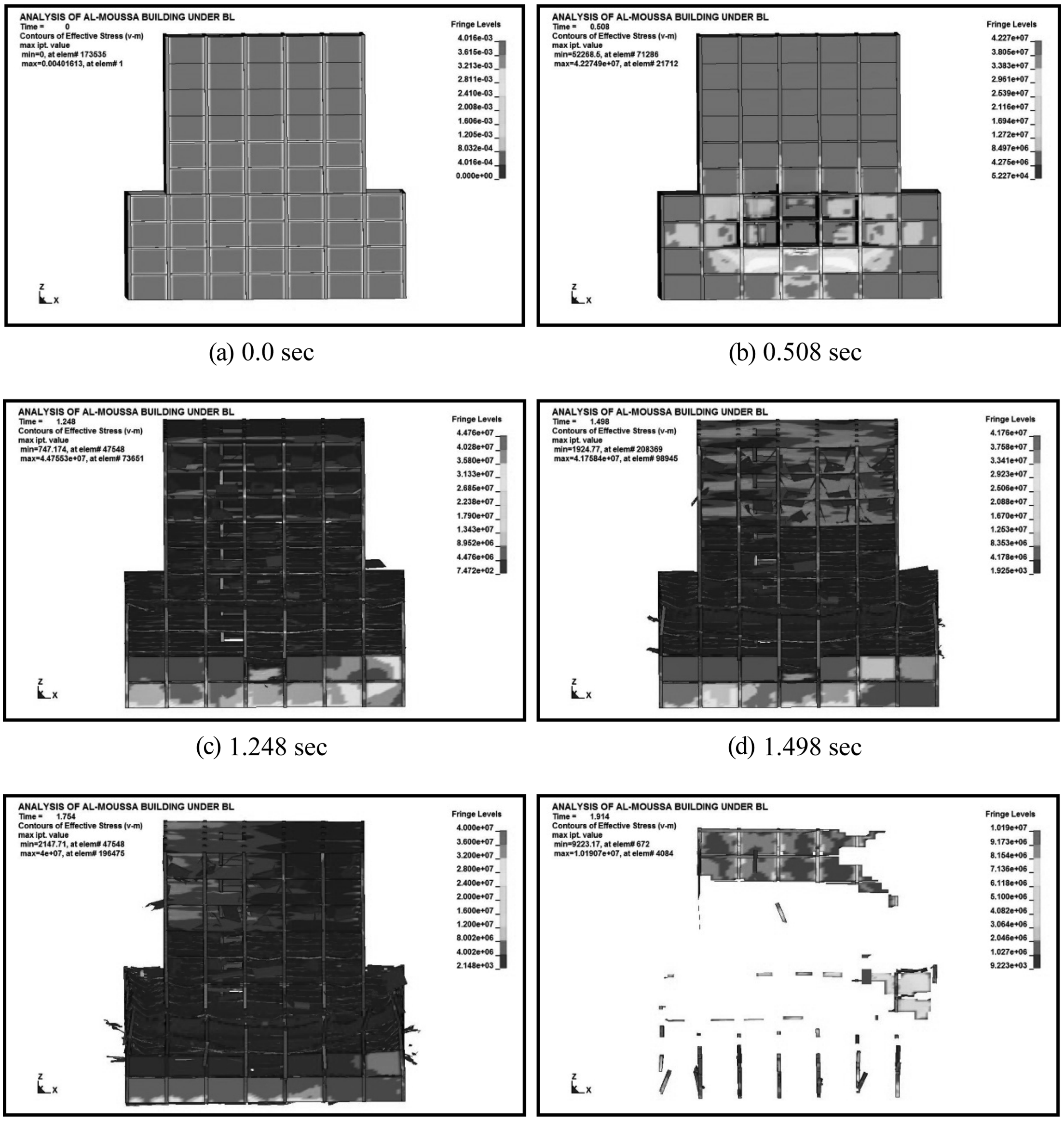
6.1 Analysis of finite element results

The results of analysis for blast scenario 1, depicted in Fig. 6, indicate the total progressive collapse of the structure at 1.914 sec. It shows that the blast scenario has the potential for progressive collapse of the building. The collapse was initiated by failure of columns close to the blast, resulting in the failure of the slab system of the ground and the first floor. As the blast shock propagated through the rest of the floors, it resulted in taking out the columns and the slab system, thereby, with no support, welted under its own weight and collapsed on the slab below.

The scenario 2 also resulted in the progressive collapse of the structure as depicted in Fig. 7, which shows the state of the building at 1.66 sec. It is seen from the figure that both the glass and masonry facades of the building were completely destroyed and the failure was explosive which could cause great loss of life and property within and in the vicinity of the structure. The retaining walls in the basement also experienced total failure.

The scenario 3 was selected keeping in mind that this building might be provided with some kind of perimeter control like concrete barriers to discourage any vehicles trying to breach the perimeter. Fig. 8 depicts the structure in a state of progressive collapse at 1.972 sec. All the non-structural components including the glass and masonry facades were completely destroyed and once again the failure was extremely explosive as seen in the figure. The columns over the entire geometry of the building experienced total collapse which proves that providing barriers at the available stand-off distance might not be an effective solution to guard against the progressive collapse of this structure. Other means of protection might be necessary.

The scenario 4 did not result in the total collapse of the building (Fig. 9). However, extensive damage was caused to the entire structure and it can be safely assumed that the structure was



(a) 0.0 sec (b) 0.508 sec

(c) 1.248 sec (d) 1.498 sec

(e) 1.754 sec (f) 1.914 sec

Fig. 6 Evolution of the structural damage produced by the explosion for Scenario1

rendered useless, and rehabilitation of this structure might not be feasible. As seen in Fig. 9, columns on the ground and the first floor on the opposite side of the blast wave were sheared out completely due to the blast shock wave. The pressure also resulted in some of the columns of the basement to be sheared. Most of the columns from the second floor onward remained intact. RC slabs in the basement, ground and first floor levels closer to the explosion focus, were completely destroyed and had come to rest on the floors below them. As the blast wave propagated, it did not cause much damage to the concrete slabs of the other top floors from the second floor onward.

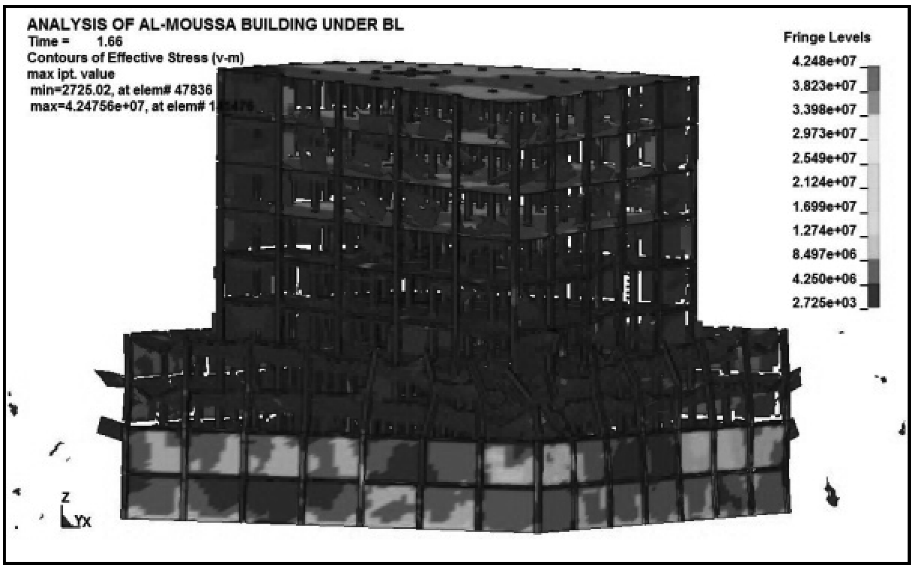


Fig. 7 Damage caused to the entire structure at 1.66 seconds for Scenario 2

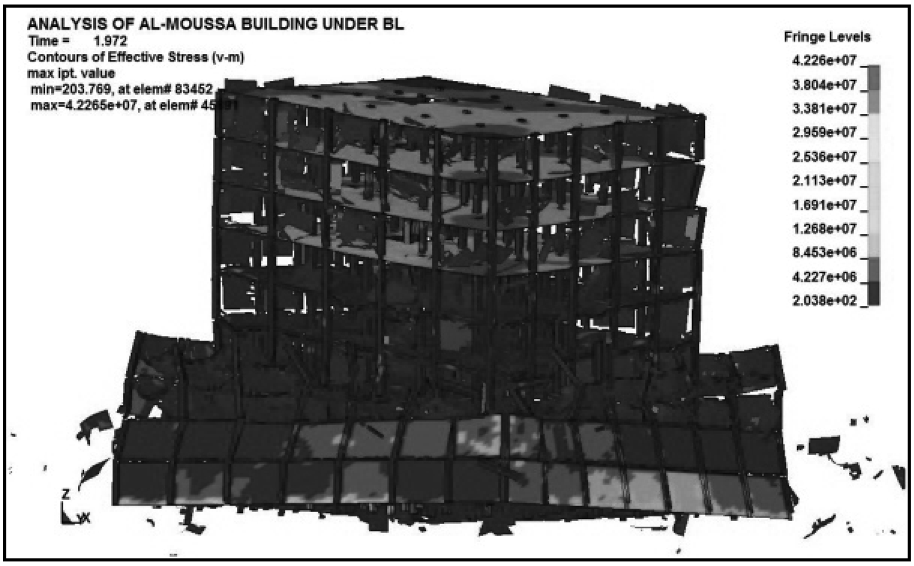


Fig. 8 Damage caused to the entire structure at 1.972 seconds Scenario 3

Masonry facade on the two front faces was destroyed and the failure type was explosive in nature capable of causing great loss of property and life in the vicinity of the structure. The glass façade on all the sides got completely disintegrated as seen in Fig. 10. Much less damage was caused to the retaining walls of the basements due to the absence of cratering in this scenario.

From the results obtained herein, it is evident that the cratering effect has quite an impact on the performance of a structure subjected to blast loads especially in the case of close-in detonations. It

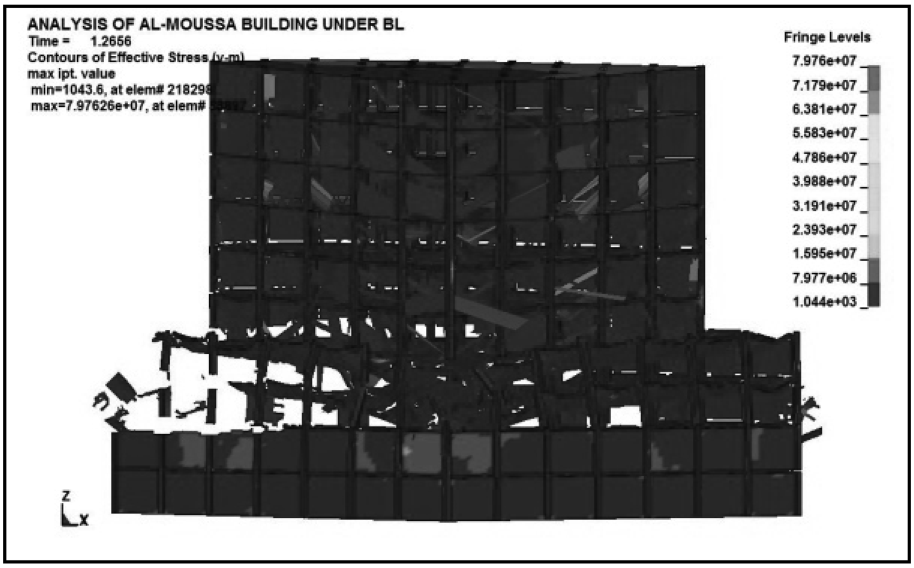


Fig. 9 Damage caused to the entire structure for Scenario 4

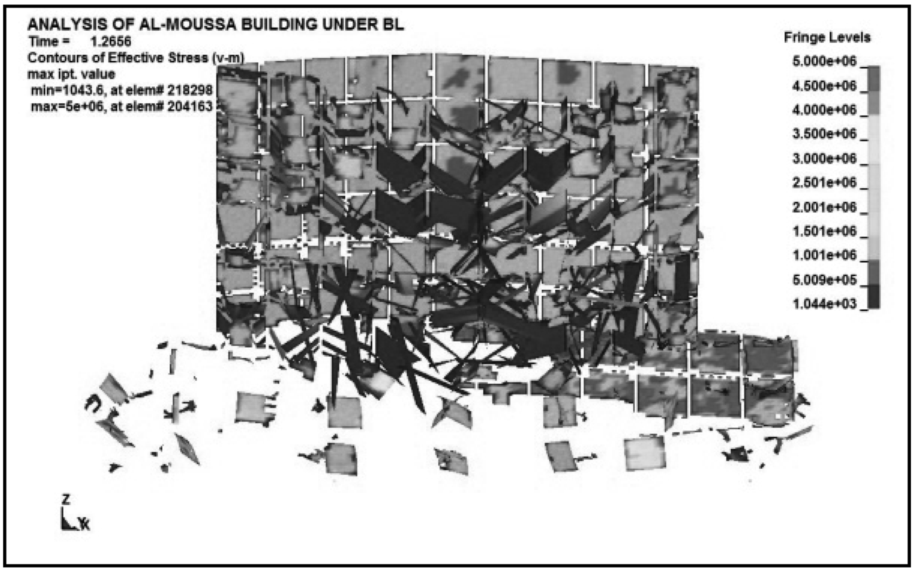


Fig. 10 Scenario 4: Damage caused to the glass and masonry facade of the building

may be inferred from the study that one of the significant measures for avoiding progressive collapse may be the strengthening of the pavement around the building thus avoiding the cratering effect.

7. Conclusions

The blast load parameters required for the prediction of blast loads on structure are reviewed and compared. The blast load prediction formulae are observed to show diverging trend in the near field which is probably due to the complexity of blast phenomena close to the charge.

The 3-D finite element analysis of a RC building shows that the building is susceptible to progressive collapse when subjected to blast impact loads from an equivalent TNT charge weight of 1000 kg. The scenarios of varying location of blast and the available standoff distance do not have significant effect on the progressive collapse response of the building. The increase in the standoff distance from 1 m which is available at present to 7 m was based on the assumption of using exterior barriers are provided all around the walkways.

It is shown that the shortcomings of notional member removal requirements of many codes may be addressed by improved blast analysis through the use of solid elements with the provision of element erosion. Thus, the regulatory requirements of approximate static structural response for the failure of vertical members under blast load get replaced in the present analysis by the improved dynamic phenomenon of the collapse of members.

The effect of cratering has quite an impact on the behavior of a structure subjected to a close-in detonation as evident from the two scenarios; with and without cratering. One of the significant measures for avoiding progressive collapse may be the strengthening of the pavement around the building thus avoiding the cratering effect.

Acknowledgements

The financial grant received from the Knowledge Exchange and Technology Transfer Program of King Saud University, Riyadh, KSA, to the "Specialty Unit for Safety and Preservation of Structures" to which the authors are attached is gratefully acknowledged.

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