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Progressive collapse analysis of a typical RC high-rise tower

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ABSTRACT

The world has recently witnessed tremendous increase in terrorist activities. This led to the requirement of blast resistant design of structures. The progressive collapse of structures, being the most severe consequence of blast generated waves, has been the subject of several studies. Although structural engineers are developing methodologies for the mitigation of progressive collapse, there is a lack of adequate tools that can be employed for simulating and predicting the progressive collapse response of structures with acceptable confidence. An attempt has been made in this paper to develop a practical and acceptable procedure for the progressive collapse analysis of reinforced concrete (RC) framed structures. The adequacy of the procedure has been demonstrated by studying the progressive collapse behavior of a typical RC framed high-rise building in Riyadh when exposed to blast generated waves.

1. Introduction

Due to the increasing threat of terrorism, the proliferation of weapons and accidental explosions, the infrastructure around the world has become more vulnerable to blast loads (Almusallam et al., 2010a; Al-Salloum et al., 2015; Alsayed et al., 2016; Elsanadedy et al., 2014; Luccioni et al., 2004; Mlakar et al., 1999; Mohamed, 2006). The use of reinforced concrete (RC) being more common in the overall infrastructure around the globe, the progressive collapse mitigation of RC structures has attracted the attention of structural designers and researchers (Almusallam et al., 2010b, 2010c, 2017; Al-Salloum et al., 2016; Elsanadedy et al., 2011, 2017). Recent developments in high-rise RC structures such as the efficient use of high strength concrete, advanced framing systems, and increased level of precision in the methods of analysis lead to building designs with considerably smaller margin of safety, which results in greater vulnerability to progressive collapse. The capability to predict the progressive collapse potential under the action of blast loads can provide useful information which can be used for preventing progressive collapse type of failure.

The structural failure of a 22-story precast concrete apartment building at Ronan Point, London, UK, on 16 May 1968, attracted the attention of structural engineers towards the progressive collapse of buildings. An external wall panel failure occurred due to gas explosion in the 18th floor. The failed external panel of the building was a load bearing wall, which leads to a propagation of failures on the corner bay of the building upward to the roof level and downward to several level above the ground (Ellingwood et al., 2007). However, thorough investigation on progressive collapse of structures started only after the bombing of Alfred P. Murrah Building in Oklahoma City on 19 April 1995 (Mlakar et al., 1999), and the well-known terrorist attack on the World Trade Center in New York on 11 Sept. 2001. Since then, the literature on blast protection especially the progressive collapse mitigation has expanded considerably, and many building design codes incorporated provisions for the avoidance of disproportionate collapse.

The resistance of RC framed buildings to progressive collapse has been adopted in the revised versions of structural design codes. The approaches adopted by different codes and design strategies have been reviewed and discussed by several investigators (Mohamed, 2006; Dusenberry, 2002; Ellingwood, 2006; Kaewkulchai and Williamson, 2004; Nair, 2006; Starossek, 2006; Starossek and Wolff, 2005). Several important issues such as abnormal events causing progressive collapse, load combinations, analysis and design procedures were examined by the investigators. In addition, advantages and disadvantages of different

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approaches for progressive collapse analysis have been discussed in literature (Marjanishvili and Agnew, 2006; Marjanishvili, 2004). General Service Administration (2003) and DOD (2005) have formulated a linear static method for progressive collapse analysis of structures.

This paper presents an advanced numerical analysis procedure to predict the progressive collapse potential of RC buildings exposed to blast generated waves. The applicability of the procedure is demonstrated by analyzing a typical high-rise tower when exposed to blast loads arising from an explosion in the vicinity of the tower. A 3-D finite element (FE) model was developed for the high-rise tower. Both structural as well as non-structural members were modeled. The structural elements included columns, beams, slabs and core walls. LSDYNA explicit finite element analysis code was used in this exercise.

2. Codes and standards for blast-resistant design

The structural designers have access to several codes and standards such as NBCC (2015), GSA (2013), ASCE/SEI-7-10 (2010), DOD (2005), and IBC (2015) for designing structures to reduce the potential for progressive collapse. The salient provisions of some of the important codes and standards are discussed in the following.

2.1. National building code of Canada (NBCC, 2015)

For minimize the probability of progressive collapse, this code requires that structural systems shall be designed such that an initial local structural failure, due to an abnormal event, will not spread to adjoining structural elements. The measures suggested to prevent progressive collapse include: (i) control of abnormal events, (ii) designing critical members for resisting loads from abnormal events, (iii) providing adequate member ties, (iii) providing alternate paths of load transfer to supporting members, and (iv) compartmentalizing the structure for minimizing the spread of the local collapse to adjoining members.

2.2. Minimum design loads for buildings and other structures, ASCE/SEI-7-10 (2010)

This standard requires that the hazards associated with progressive collapse of structures should be reduced by adopting: (i) Good plan layout, (ii) Integrated system of ties for the principal structural members, (iii) Design of floor slabs for alternate load path (ALP), (iv) Structural interior partitions capable of carrying loads, (v) Catenary action in floor slab, (vi) Beam action of walls, (vii) Alternate load path through redundant structural systems, (viii) Ductile reinforcement detailing, (ix) Additional steel rebars to carry blast loads, and (x) Compartmentalization of structure together with special moment resisting RC frames. The weak side of the code is that the degree of redundancy is not specified, and the requirements are entirely threat-independent.

2.3. GSA alternate path analysis & design guidelines for progressive collapse resistance (2013)

This code requires that detailed analyses such as linear and nonlinear dynamic FE Analysis (FEA) be employed to assess the progressive collapse potential. However, the code provides maximum allowable limits for rotation and ductility for many structural components to facilitate decisions about the survivability of structures. The analysis and design procedure for providing resistance to progressive collapse is also given for existing and new structures.

For new construction, the code encourages the use of redundant systems for resisting vertical as well as lateral loads, ductile detailing of structural members, and avoiding shear failures.

For typical structural frames, the code suggests linear elastic static analyses for the first-floor column removal scenario. The live load for the analysis is taken as 25% of design live load. The progressive collapse potential is assessed with the help of Demand-Capacity Ratio of structural members. For high potential against progressive collapse, Demand-Capacity Ratio should be greater than 2 for typical buildings.

3. High-rise structure case study

A typical twenty-eight storey (4 Basements + Ground + 23 levels) building located on one of the busiest highways of Riyadh has been chosen as a blast resistance investigation case study. The building chosen for the case study is an approximation of an existing structure. The building is a RC framed high-rise tower. Fig. 1 shows the layout of beams and columns at different floor levels. The floors consist of 250 mm thick flat slab system. There is a lift and stair-well core of 350 mm thick RC for resisting the lateral load on the structure. The perimeter of the structure consists of glazing façade. With the exception of a few columns with irregular cross-section, the external and internal columns' cross-sections are rectangular of varying sizes from 0.9×0.9 m to 2.1×1.6 m. The column sizes are summarized in Fig. 1. There are four basement levels for car-park in addition to the building perimeter open area car-park. The floor-to-floor height of the lobby area is approximately 7 m, whereas the storey height for all other levels is 4 m.

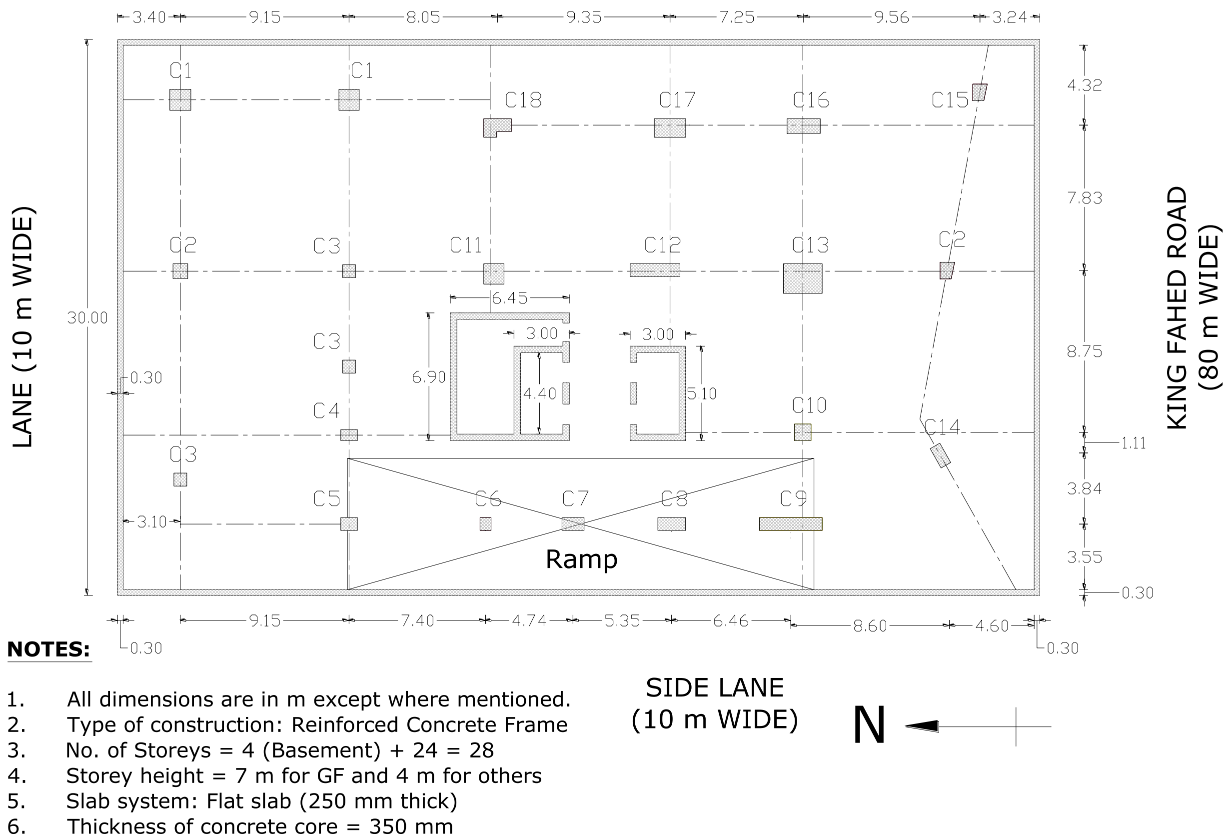
The building design chosen for the case study is similar to an existing structure. The foundation of the tower being a thick RC raft, the column bases were fixed. The uniaxial cylinder compressive strength of concrete is assumed to be 40 MPa and the yield strength of steel rebars is taken as 400 MPa. The member sizes were taken as per design, whereas the percentage of rebars in different structural members has been assumed to be typical for each storey.

4. Threat assessment

The probable blast scenario was identified by qualitatively estimating the vulnerability of the critical structural elements of the tower. The following factors were taken into account in the qualitative assessment:

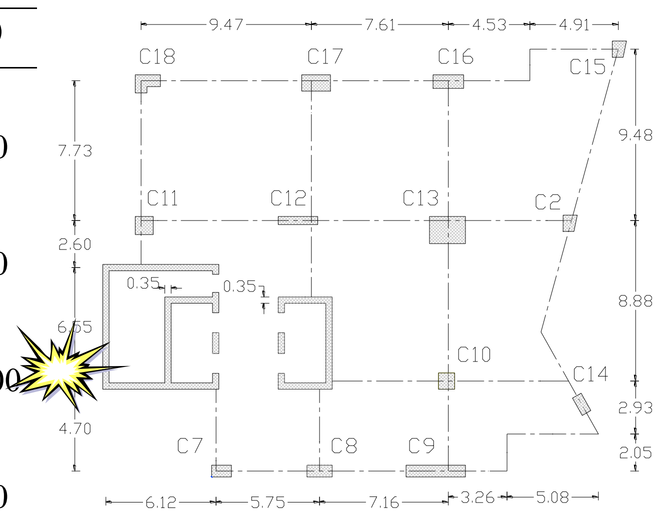
- **Element visibility:** Factors such as external critical elements and critical element prominence in the architectural design of the building often influence the likely blast threat scenario. The potential attackers would establish the target based on visual observation provided that architectural/structural drawings are not accessible.
- **Element criticality and significance:** Vulnerable structural system features such as long columns, large spans and/or transfer system were taken into consideration in the threat identification process in order to establish the worse feasible blast scenario.
- **Element accessibility or exposure:** Critical element accessibility features such as location relative to major roads, location relative to car-parks or loading docks and absence of architectural element shielding were taken into consideration in the threat identification process.

The terrorist bombing involving the use of non-nuclear explosion outside the building is considered. The magnitude of blast



(a) Floor plan for 1st to 4th basement

Column	Size (mm)	Column	Size (mm)
C1	1140x1140	C10	900x900
C2	900x900	C11	1100x1100
C3	700x700	C12	2700x700
C4	900x600	C13	2100x1600
C5	900x700	C14	1200x600
C6	700x600	C15	900x800/600
C7	1200x700	C16	1800x800
C8	1500x700	C17	1700x1000
C9	3400x700	C18	1500x1000
		L-shape	& 700 thick



(b) Floor plan for GF to 24th storey

Fig. 1. Layout of the building with location of threat.

pressure from an explosion depends on the type of explosive, charge weight and standoff distance. Considering the charge weight of an explosive device in terms of equivalent weight of TNT reduces the blast load parameters from three to two namely the charge weight and standoff distance. Based on the identification criteria of element visibility, accessibility, and criticality, a potential worst-case scenario was assumed. The major threat to the high-rise tower from terrorist activity is explosion of a bomb close of the tower with the bomb delivered using a vehicle. The threat scenario is shown in Fig. 1.

5. Numerical modeling

A general purpose transient dynamic FE analysis software, LSDYNA, was used for analyzing the high-rise tower. A two stage approach, involving the local model analysis and the global model analysis, was employed. The local model analysis was used to check the vulnerability of individual ground-floor columns exposed to blast generate waves. Whereas the global model analysis was performed for whole structure of the tower after removing the columns that failed in local model analysis.

5.1. Local model

The critical structural elements of the tower were identified as the ground-floor perimeter columns located close to the Vehicle-Borne Improvised Explosive Devices (VBIED). Three critical components were identified in the threat scenario. They are the core wall, columns C7 and C11. However, only column C7 is analyzed in detail. This is due to the fact that column C7 is the farthest critical element from the threat location. Hence, a charge which leads to column C7 failure would induce failure on C11 and the core wall. Hence the FE model of the column was prepared for assessing it performance against blast loads. The column was modeled using hexahedronal constant stress solid elements with one-point integration rule, whereas the longitudinal column rebars and ties were modeled using two-node beam elements with Hughes-Liu (Hughes and Liu, 1981) cross-section integration element formulation. In this beam element, the number of integration points over the cross section can be controlled and thus desired accuracy at the cross section level can be achieved. The column rebars consisted of ϕ 12 ligatures at 300 mm spacing and ϕ20 longitudinal rebars.

Concrete Damage Release 3 material developed by Karagozian & Case (Malvar et al., 2000) was used for the concrete component, and the Plastic Kinematic material for the rebars. The Concrete Damage material model is capable of taking into account the shear failure surface, strain-rate effects and the volumetric damage of the concrete component. The Plastic Kinematic material model is capable of modeling the plastic deformation of steel component, including the strain hardening stage.

5.2. Global model

The global model of the tower was developed with the help of its structural drawings. The structural elements of the tower were categorized as beam and shell elements. The beam elements were employed to model RC beams and RC columns, while, the shell elements were used to model the floors and lift core. The 2-node axial beam element with the capabilities of simulating compression, tension, bending and torsion, was used for modeling beams. This element allows offsetting of nodes from the axis of the beam and permits an unsymmetrical geometry. The Hughes-Liu (Hughes and Liu, 1981) element formulation with cross-section integration was used. The cross-sections of the columns are generally rectangular, whereby the non-rectangular column cross-section is approximated to a rectangular cross-section column component with equal cross-sectional area.

The RC floor slabs, lift core walls, and façade components were modeled using shell elements. Both four node quadrilateral and three node triangular elements, with bending and membrane capabilities, were used for appropriate representation slab and wall geometries. These elements can carry in-plane as well as normal loads. Belytschko-Tsay theory (Belytschko and Tsay, 1981, 1983) was employed for the element formulation. Since the element formulation is based on flat geometry of element, warpage of element is not included in the model. The finite element model of the structure contains: 39,394 nodes, 1603 beam elements, 38,774 quadrilateral shell elements, and 1050 triangular shell elements.

The RC of columns, beams and shell elements was simulated using Concrete Eurocode (EC2) material model (Eurocode 1: BS EN 1991-1-7, 2006) which is capable of modeling concrete and rebars and simulating large deformation and stress stiffening. The model is appropriate for the beam as well as the shell elements. The material model parameters for different structural components are given in Table 1.

The failure strain threshold was used as a failure criterion for the glass façade system. The glass façade system failure often depends on the type of glass, composition of the panel and flexibility of the support system. Glass component without further lamination or film application typically fails in brittle manner; in this case, plastic strain would not occur in the component. However, most glazing components in a façade system are laminated with shading film. Hence, assuming that the façade system is not specifically designed to resist the effect of blast pressures, the failure criteria of the glass façade system in the model is established at 0.6% plastic strain to cater for the limited ductility due to the contribution of the laminates.

5.3. Blast scenario

The structural layout of the tower has rectangular beam-column grid. The entrance and exit of the tower are located on the south face which lies on a major road. The east side lane is limited to the pedestrian sidewalk, whereas the north and west side streets are meant for car parking. The car parking streets with a parked VBIED on the north and the west side are a source of major threat to the building from terrorist bombing. An examination of the structural layout of the tower and the location of streets shows that a VBIED may be parked close to the tower at a minimum standoff distance of 2.5 m, as shown in Fig. 1.

The threat location, shown in Fig. 1, was established through a thorough risk assessment (Almusallam et al., 2016), which identifies the critical component and most vulnerable location. A charge weight of 500 kg of TNT (Rounded value for 1000 lbs), which can be easily carried in a Sedan car (FEMA 452, 2005), is considered for evaluating the progressive collapse potential of the tower. As the explosive is supposed to be carried and detonated in a car, the explosion is assumed to occur above ground. Therefore, the height of explosion above ground is taken as 1 m. Consequently, the vibrations transmitted to the tower through ground get considerable weakened which are thus ignored in the analysis.

5.4. Blast load application

The detonation of a blast charge in air produces rapidly expanding gases that compress the surrounding air and pushes it outwards with a tremendous force thus forming a high velocity blast wave. After the rapid increase of pressure at the shock front, there is an exponential decay with pressure reaching back to the ambient pressure. The negative pressure phase is quite longer (Fig. 2). The overpressure-time history of a typical blast wave is usually described by Friedlander's equation (Kinney and Graham, 1985):

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

where $x = \frac{(t-t_a)}{T_s}$, t is the time, p is the static overpressure at time t, p_s is the peak static overpressure, t_a is the arrival time of blast wave, T_s is the duration of the positive pressure phase, and b is the waveform parameter, which is positive constant. The corresponding absolute

Table 1
Constitutive material parameters.

Structural Element	Constitutive model	Compressive strength (MPa)	Tensile strength (MPa)	Young's Modulus of steel (GPa)	Yield strength of steel (MPa)
Concrete columns/slab/core	Concrete EC2	40	2.53	200	400
	Constitutive model	Young's modulus (GPa)	Poisson's ratio	Yield stress (MPa)	Failure strain
Glass facade	Plastic Kinematic	50	0.2	5.0	0.6%

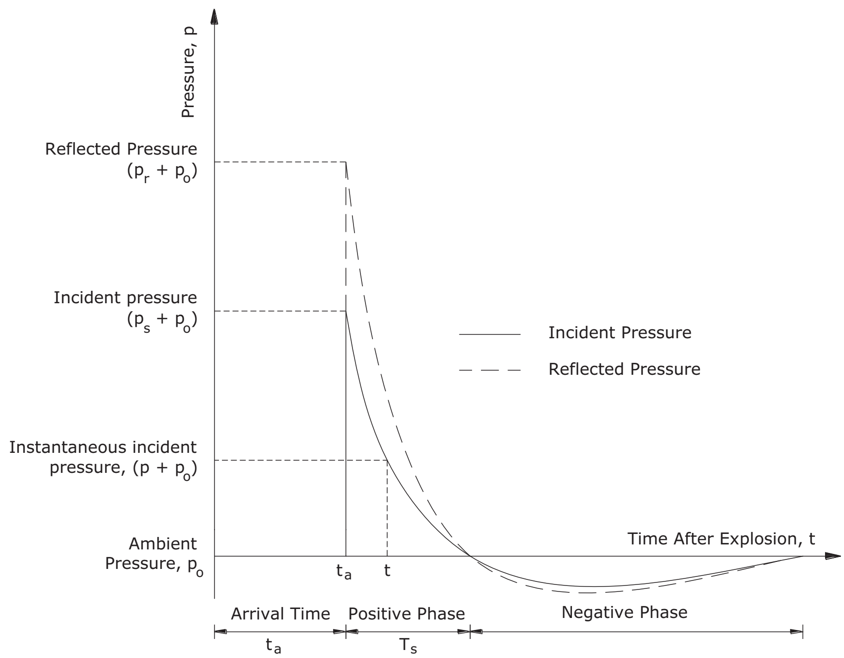


Fig. 2. Qualitative free-field blast pressure-time history.

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). If the positive overpressure is large, the negative pressure phase will not be critical. The negative pressure phase impulse is thus usually ignored, especially for rigid structures like RC frames (TM 5-1300, 1990).

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). The negative phase being insignificant, as discussed above, is usually ignored in the above integration. When a rigid wall obstructs the blast waves, the waves get reflected. The normal strike of waves on the wall causes the stoppage of air flow behind the wave and the pressure on the wall increases which becomes much greater than the incident pressure. The peak reflected pressure p_r, caused by the normal strike of blast waves on an infinite rigid wall, can be obtained from (Mays and Smith, 1995):

$$p_r = 2p_s\left(\frac{7p_o + 4p_s}{7p_o + p_s}\right) \tag{3}$$

The ratio of peak reflected to the incident pressure, obtained from the above equation, varies from 2 to 8. Assuming the time history of reflected blast wave to be similar to the incident wave, the impulse of the reflected blast wave can be obtained using (Baker et al., 1983):

$$I_r = I_s\left(\frac{p_r}{p_s}\right) \tag{4}$$

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

In the FEA, the gravity and blast loads were applied on the critical elements in two stages. The gravity load was applied as a ramp loading function with linearly rising from 0 to 3 s and then it was maintained constant after reaching the peak. The blast pressure, as discussed above, was applied to the façade components of the tower using the CONWEP function available in LS-DYNA.

Fig. 3 shows the variation of incident pressure, reflected pressure and positive phase duration with stand-off distance obtained from ConWep (1990) for the charge weight of 500 kg considered in the study. The equations developed by Kingery and Bulmash (1984) are used in ConWep calculations of air blast parameters. The minimum range for the use of ConWep for 500 kg charge weight is found as 1.42 m (ConWep, 1990).

6. Results and discussion

In the local analysis, the failure criterion of the column is defined as the loss of gravitational load resistance of the column. Column C7 was subjected to varying charge weights at the specified charge location, which has been predetermined in the threat scenario. The analysis indicates that a charge weight of 500 kg TNT equivalent is adequate to induce the failure criteria of components. Fig. 4(a) shows the rebar mesh and Fig. 4(b) shows concrete mesh of the column. The column ends were taken as fixed. The typ-

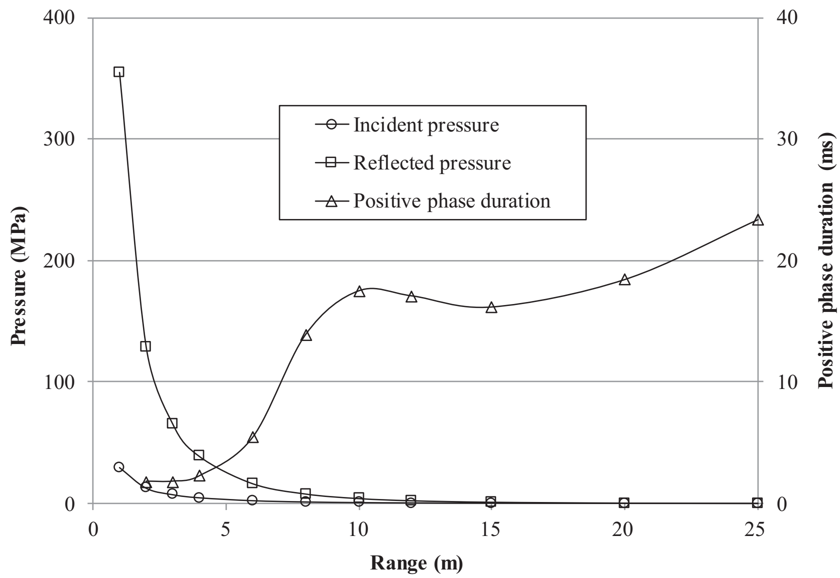
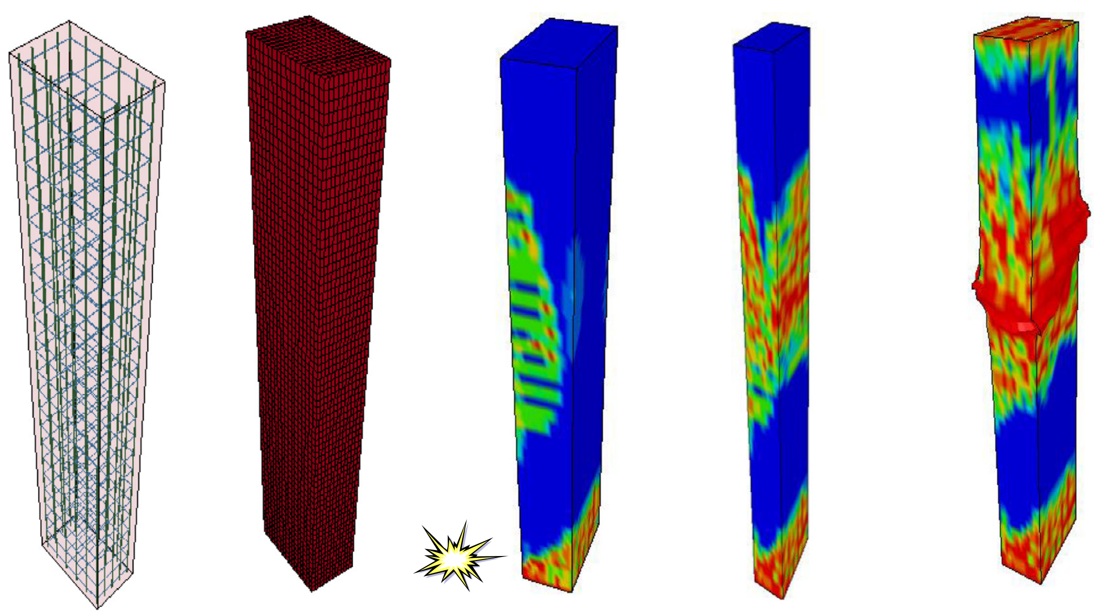


Fig. 3. Pressure and positive phase duration variation for 500 kg charge weight obtained using ConWep (1990).



(a) Rebar cage (b) Concrete mesh (c) T=27 ms (d) T=70 ms (e) T=160 ms

Fig. 4. Typical damaged column – C7.

ical column damage observed in the analysis is shown in Fig. 4(c), (d), and (e). The results of local model analysis indicate that the column C7, column C11 and core wall will be severely damaged. The damage mechanism of the columns is governed by the rupture of hoop ligatures, which leads to the loss of confinement and eventually the loss of load bearing capacity of the component. The effect of fragments and projectiles on different parts of the structure has not been considered.

Fig. 5 shows the progress of damage of the façade of the tower and the structural system at different times after the arrival of the blast waves. The consequent structural damage of the tower is shown in Fig. 5(c), which indicates progressive collapse of the structure. The loss of the core walls and outer columns C7 and C11 located close to the detonation point use the transfer of the gravity load to the adjoining vertical members including the core walls and columns through the flexural and membrane action of the RC floor slabs. The increase in flexural stress leads to high rotational demand in connection region especially in the slab-column or slab-core wall connection regions in bays adjacent to the failed gravity load bearing component. These events lead to a progressive collapse mechanism of the structure.

The paper demonstrates that the two-stage analysis is a convenient method for progressive collapse investigation of RC structures. The steps involved in this approach, can be summarized as:

i) In the first stage, identify the critical load bearing members and perform local model analysis to evaluate their performance against the blast pressure for known charge weight and standoff distance. The local model analysis of this stage should employ solid elements for concrete and beam elements for the rebars.

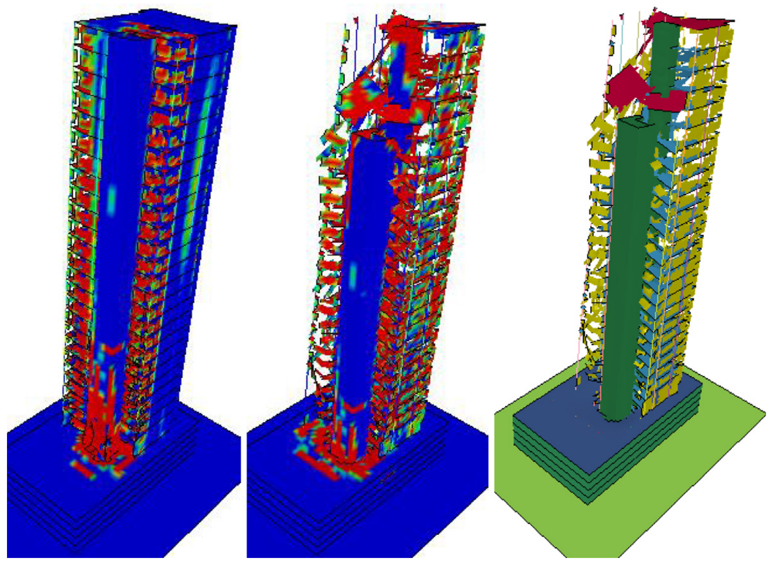


Fig. 5. Damage state of building.

ii) Prepare the global model of the structure after removing the load bearing members found vulnerable in the first stage. For keeping the analysis affordable, use beam elements for columns and beams and shell elements for the floor and RC walls. Analyze the structure against the blast pressure to assess the overall response of the structure.

7. Conclusions

To avoid catastrophic structural failure of high-risk structures against blast loads, it is very important to establish the vulnerability of structures to progressive collapse events. Currently, there are no provisions or recommendations in the current standards of several countries with regard to the progressive collapse of buildings.

The efficient assessment method presented in this paper may be used to establish the critical location in structural components, which would enable engineers to identify detailing requirements for the critical component to mitigate the progressive collapse events. For example, in this case study, critical information such as the potential area of attack, the performance of load-bearing components and areas subjected to high stresses, were established through local column analysis and global analysis of the tower. Based on this set of information, engineers and security consultant would be able to develop an effective mitigation plan, which may be in the form of access prevention, critical column strengthening and proper structural detailing to improve connection ductility.

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