Blast Analysis of a Typical Reinforced Concrete Building of Riyadh

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Abstract: In the recent past, structures all over the world have become susceptible to the very real threat of terrorist attacks, accidental explosions and other unthought-of explosion related events. Consequently, a number of concerns have been raised on the vulnerability and behavior of buildings and critical infrastructure under extreme loadings. The paper presents a blast analysis of a typical multi-storey reinforced concrete framed building located in Riyadh for its vulnerability assessment by considering different threat scenarios. The building taken up for the study is ten storey high including two storeys of basement. 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. LS-DYNA software, which uses explicit time integration algorithms for solution, has been employed for finite element analysis. The results of the study are proposed to be used to control or prevent progressive collapse of the building. The paper also presents current state of knowledge on progressive collapse in the technical literature covering blast loads and structural analysis procedure applicable to reinforced concrete buildings.

Keywords: blast; progressive collapse; RC building

1 Introduction

Historical records indicate that the majority of terrorist incidents have occurred in an urban environment in the presence of nearby buildings forming the street geometries. It has been observed that the confinement, provided by tall buildings, could drastically increase the blast loads by an order of magnitude or more above that produced in the free field by the same explosion source. To date, a satisfactory engineering model capable of accounting for these effects has not been developed.

The dynamic analysis procedures proposed by Kaewkuchai and Williamson[1] seem to work well for a two-bay structure. The application of the procedures for structures that have more than two bays will generate inaccurate structural responses. Instead of applying dynamic loads to the entire building, as proposed by Kaewkuchai and Williamson[1], this study uses a dynamic analysis procedure, based on the column removal scenario, to represent the dynamic responses of structures associated with progressive collapse. Furthermore, the analysis procedures proposed by Buscemi and Marjanishvili[2] are originally for single-degree-of-freedom (SDOF) systems, whereas the energy-based methods proposed by Dusenberry and Hamberger[3] are only useful for a simple structure. The application of the approaches requires further development. It is obvious that a dynamic analysis procedure is required to capture the actual response of a structure. In addition, the alternate load path approach for progressive collapse analysis is based on the dynamic response of the structure due to the instant and clear removal of load bearing elements, such as a column. This approach is easily applied because of its simplicity and directness[4] and its independence from specific causes[5]. However, it is still necessary to understand the characteristic of the structure’s response due to particular causes. More accurate analysis methods are required in order to predict the extent of damage to the structures.

Understanding the conditions and mechanisms of load redistribution (normally not considered in the design process, such as membrane action of beams and floor systems), that could result in reaching a state of stable equilibrium after local damage of load-bearing structural elements has been identified as one of the more important tasks for improving the current knowledge in progressive collapse mitigation[6]. Catenary (tensile membrane) action and Vierendeel action of beams and floor systems have been identified as the primary mechanisms of load redistribution and progressive collapse resistance in reinforced concrete (RC) frame buildings subjected to the removal of a ground floor exterior column, as prescribed by GSA and DOD guidelines[7-10]. The complex nature of the structural response of RC buildings subjected to such scenarios is difficult to implement in experimental program that can provide enough comprehensive data to develop a complete understanding of the mechanics of progressive collapse resistance based solely on experimental data. Detailed finite element models and dynamic analyses considering material and geometric nonlinearities, complemented with the understanding of element level behavior developed through analytical and experimental studies, can be used to study the mechanisms of progressive collapse resistance of RC buildings at the structure level.

An important goal in studying progressive collapse resistance is the identification of response parameters that can be used to assess the structural integrity and potential for progressive collapse of a structure, in the event of failure of one or more load-bearing structural elements due to natural or man-made hazards. Such identification would be critical in assessing the safety of a building that has suffered an initial local damage. Such an application, complemented with the ability to acquire this critical information fast and reliably after such an event (through a proper instrumentation and monitoring of the critical response parameters of a building) can improve significantly our ability to respond and take proper measures in the event of a disaster.

2 Building for Blast Analysis

A typical building located in the city of Riyadh is taken for blast analysis. This building is located in a crowded area with cars parked all over the side and front of the building. The building also attracts a significant numbers of visitors with its stores and residential apartments. A close inspection of the building premises indicated that the stand-off distance is virtually zero on either side of the building as there are no barriers to stop any vehicle being parked very close to the building. A floor plan for the basement, ground and the first floors including the column layout and dimensions for the building is shown in Fig. 1 and the layout for the other floors in also shown in this figure with dashed perimeter line. The building comprised of two stories in the basement. The structural system comprised of a flat plate slab with 250 mm thickness and octagonal columns with 250 mm side. The type of façade provided for the building is glass façade for the ground and the first floor and masonry and glass for all the other floors.

2.1 Threat identification

The likely blast threat scenario can be identified by qualitatively assessing the vulnerability of the critical element of the structure. The following factors are 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 are 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 carparks or loading docks and absence of architectural element shielding and are taken into consideration in the threat identification process.

Based on the identification criteria, three potential different scenarios were assumed – one external (Scenario 1: TS1) and two internal blasts (Scenarios 2 and 3: TS2 and TS3) as shown in Fig. 1. These scenarios were selected based on the layout of the building with respect to the streets, the standoff-distance provided, and the available access to the building.

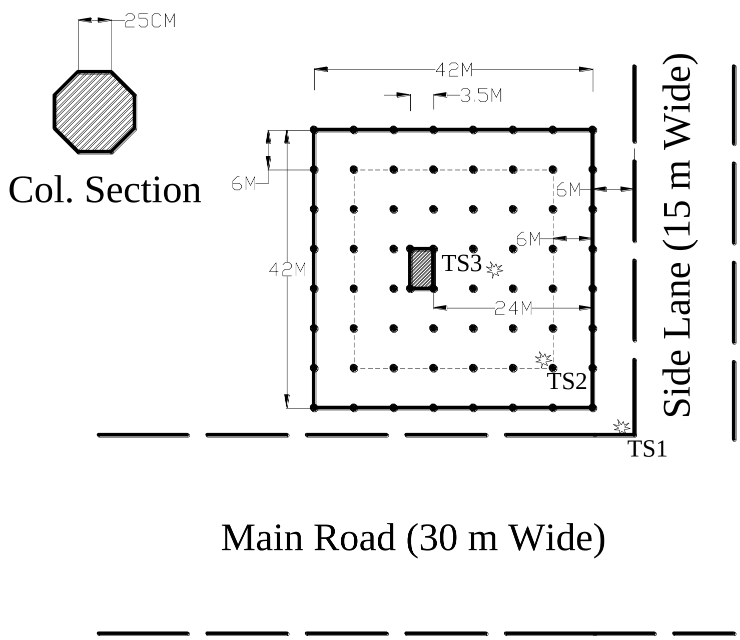
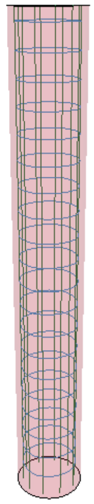


Fig. 1 Building plan (Ground Floor) with threat scenarios

2.2 Finite element model description

The finite element modelling was carried out in two stages the local model stage to assess the individual columns performance against blast pressures and the global modelling stage to assess the overall response of the structure due to the failure of the critical columns. The critical structural components are the columns in the vicinity of the Vehicle Born Improvised Explosive Device (VBIED). Hence a typical column model was built in order to establish the vulnerability of the vertical component.

The local model of the columns was developed using a combination of solid and beam elements. The concrete component of the column was modeled using hexahedronal constant stress solid elements with one point integration rule (LS-DYNA default). The one point integration solid element will help in terms of maintaining the numerical stability of the model during the analysis. The shear and longitudinal reinforcements of the columns were modeled as a discreet component using two node beam elements with Hughes-Liu cross-section integration element formulation. The column model was developed as an equivalent circular column as opposed to the octagonal shape of the actual column. Fig. 2 shows the model details. 12mm diameter bars were used for hoop ligatures at a spacing of 300mm and 20mm diameter bars were used for the longitudinal reinforcement.


(a) Typical column reinforcement details

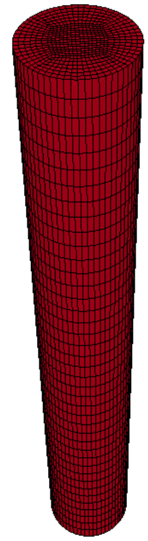

(b) Typical column model

Fig. 2 Local column model

For the global model, the structure geometry was built based on the available detailed drawings. In the global modelling phase, whenever possible, the elements of the structure were simplified into beam elements and shell elements. The beam elements are mainly used to represent the beam and column components of the structure, while the shell elements are used to represent the core wall and floor systems. The building model can be divided into several sub models:

- Core model – shell elements
- Column model – beam elements
- Floor slab model – shell elements
- Façade and retaining wall models – shell elements

The model was built by identifying key-points and typical nodes in the structure. Once the coordinates of the key-points and nodes were defined, the beam elements and shell elements were allocated to the appropriate coordinates. The appropriate section properties and material properties were then applied to the corresponding beam and shell elements. The completed global model is shown in Fig. 3.

Reinforced concrete columns were modeled as 2-node axial beam elements with tension, compression, torsion, and bending capabilities. The element has six degrees of freedom at each node: translations in the nodal x, y and z directions and rotations about the nodal x, y and z-axes. This element allows a different unsymmetrical geometry at each end and permits the end nodes to be offset from the centroidal axis of the beam. A plane through three nodes defines the orientation of the principal plane of the beam. The element formulation theory used in the modeling of columns was Hughes-Lui with cross-section integration. The actual cross-section of the column was octagonal; however, for ease in modeling, the cross-section of the column was considered to be circular with an effective external diameter of 600 mm.

The concrete slabs were modeled using a four node shell element. This element has both bending and membrane capabilities. Both in-plane and normal loads are permitted. The element has six degrees of freedom at each node: translations in the nodal x, y and z directions and rotations about the nodal x, y and z-axes. The element formulation theory used in the modeling of slab was Belytschko-Tsay theory. The shell is assumed to be perfectly flat and the local co-ordinate system originates at the first node of connectivity. The thickness of the typical concrete slab is 250 mm.

All of the glass and masonry façades and the retaining walls were modeled as 4-node shell elements using the Belytschko-Tsay element formulation theory. Retaining walls were provided in the two floors of the basement to retain the earth fill. Two different types of facades were used: glass façade for the ground and the first floors, whereas masonry and glass facade for all the other floors.

An important aspect of finite element modelling is the establishment of material constitutive models, which represent the real behaviour of the structure in question. The primary elements under consideration are the core structure, columns, floor slab and façade system. These critical elements performance analyses were performed with LS-DYNA finite element code (version 971). The material models in the LS-DYNA constitutive model library are more than capable of accurately simulating the actual material behaviour in the model.

In the local columns model, the material models used are Concrete Damage Release 3 material developed by Karagozian & Case for the concrete component, and the Plastic Kinematic material for the reinforcement component. The Concrete Damage material model is capable to take 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 modelling the plastic deformation of steel component, including the strain hardening stage.

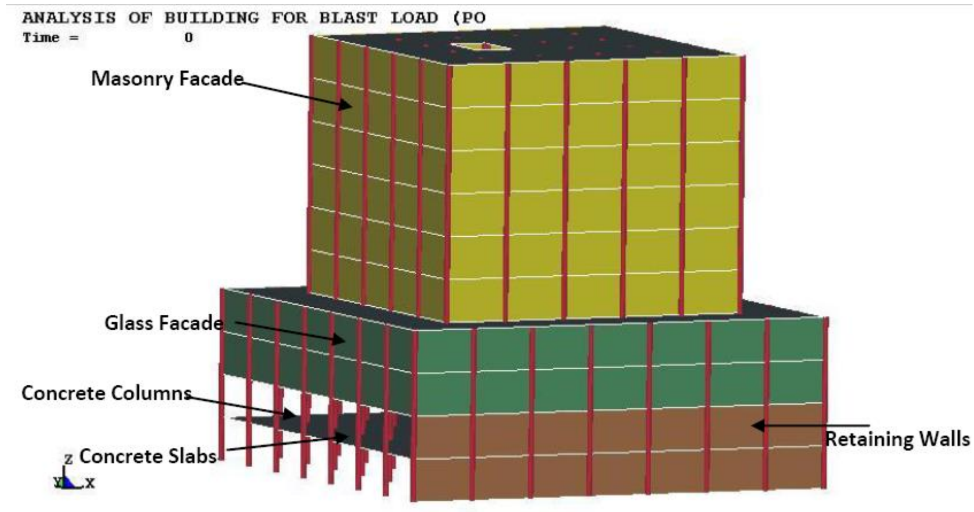


Fig. 3 Finite element model of the building

In the global building model, the material used is mainly reinforced concrete. The primary constitutive model applied was the Concrete Eurocode (EC2) material model, which is suitable for beam and shell elements. The Concrete EC2 material model is capable of representing plain concrete, reinforcement bars, and concrete with smeared reinforcement, which is predominantly used in the global model. The model includes tensile cracking behaviour, compressive crushing behaviour, and reinforcement yield, hardening and failure behaviour.

The façade components material properties are adopted based on the actual façade system behaviour. In order to visualize the damage on the façade of the building, the façade elements with excessive plastic strain were eroded from the model. The failure of the glass façade system in the model is established at 0.006 plastic strain. 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 component in a façade system is laminated with shading film. Hence, the 0.006 plastic strain is adopted as a failure criteria to cater for the limited ductility due to the contribution of the laminates. Similarly, the masonry wall component failure generally occurs in a very brittle manner, so that there would be no allowance for plastic strain in the component. However, when loaded laterally the inertia of the unreinforced masonry wall may provide a corrective momentum on the wall. The 0.003 plastic strain is adopted as a failure criteria to cater for the corrective motion. This modeling approach is adequate as a visual aid to establish the damage on the façade system in a blast event since façade system performance would have a very small contribution towards the overall structural system performance against blast pressures. Further detailed local analysis of the façade system would need to be carried out if an accurate façade failure needs to be established.

2.3 Blast load application

In the analysis, the loads on the critical element have to be applied in two stages to account for both gravity load and blast loads. In the detailed analysis of the structure, the gravity load was applied as a ramp loading function, and maintained constant once it had reached the peak gravity load level at 2.5 second. The blast pressure was applied to the façade component of the structure using the in-built CONWEP function in LS-DYNA. Fig. 4 illustrates the load stages in the model.

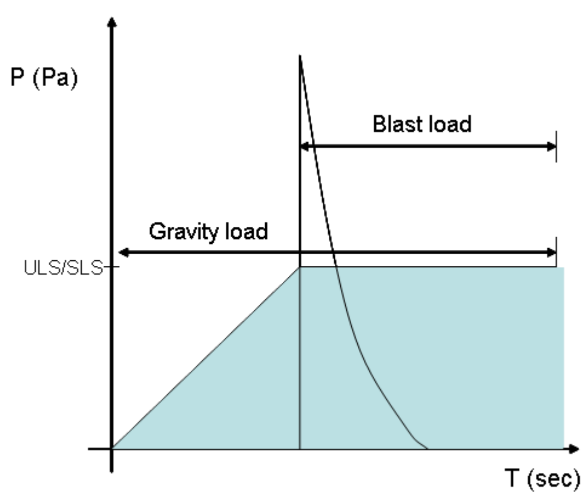


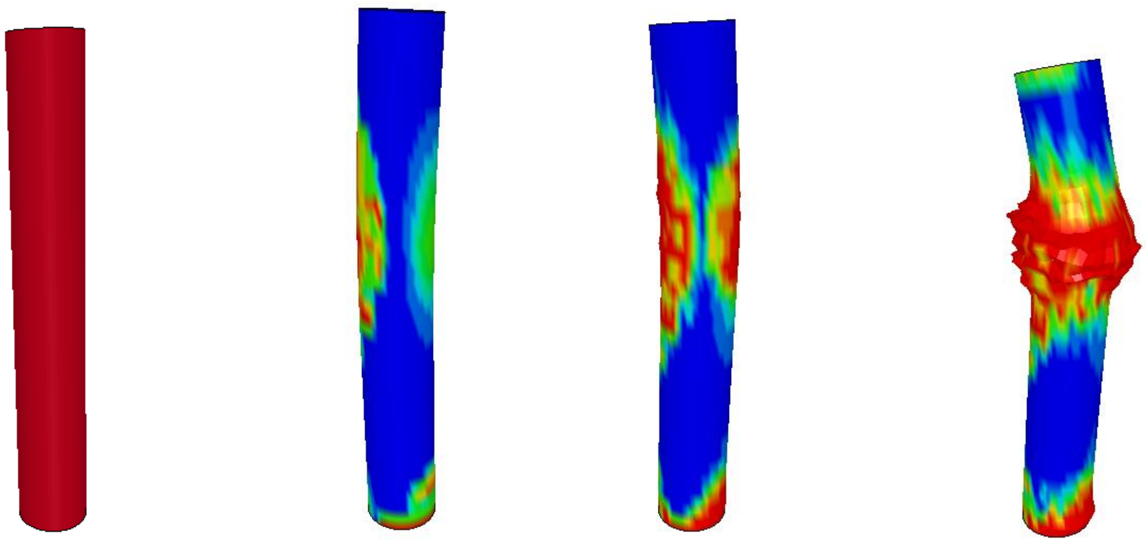
Fig. 4 Loading stages

3 Results of Analysis

Blast load is generally impulse type high amplitude loading which lasts for a very short period of time, measured in milliseconds. However, it produces an extremely high pressure loading over the structure. The damage caused by the pressure may cause failure of exterior walls, slab system and columns. Progressive collapse refers to the spread of this localized failure from element to element thereby resulting in a disproportionate total damage. The critical charge required to induce failure on the critical component is established through detailed analysis of the local component.

3.1 Local Model Analysis

In the local analysis the failure criteria of the typical column is defined as the loss of gravitational load. The typical column model was subjected to varying charge weight/stand-off combination. The columns in the building are located at regular grid at interval of 6m as shown in Fig. 1. The analysis indicates that a charge weight of 1000kg TNT equivalent (large van) is adequate to induce the failure criteria of the typical column at a distance of 13m. This indicates that columns within two spans of the floor slab would fail when subjected to the blast threat. Fig. 5 shows the typical column damage observed in the analysis.



T = 0msec T = 15msec T = 30msec T = 70msec

Fig. 5 Typical damaged column

3.2 Threat Scenario – 1 (TS1)

The first threat scenario was selected in which, the 1000 kg charge was placed on the vicinity of the columns located at the corner of two streets as shown in Fig. 1. The blast location is situated at 2.5m stand-off distance, which is the distance between the centre of the explosive and the building. The local model analysis results indicate that six columns in the vicinity will be severely damaged and eventually lost their load bearing capacity.

Figs. 6 and 7 show the damages on the façade of the building at progressive time after the arrival of the blast pressures. The structural damage in the event is shown in Fig. 8 which indicates partial collapse of the structure. Due to the loss of six columns in the vicinity of the blast event, the gravity load has to be transferred to adjacent vertical components such as the next columns and the core structure via flexural action of the floor slab. The partial collapse occurs because the aforementioned flexural stresses exceed the flexural capacity of the floor slab. Hence, the floor slab is extensively damaged and subsequently lost its load transfer capacity. Stress concentration on the slab-column connection region was observed in the analysis due to the flat slab floor system as shown in Fig. 9. The damages on the slab are shown in Fig. 10.

3.3 Threat Scenario – 2 (TS2)

The second threat scenario was selected in which, the 1000 kg charge was delivered to the underground car park area and detonated on the vicinity of the columns located at the corner of building perimeter. It was indicated in the local column analysis that columns within two spans distance from the charge would fail under the blast event.

Figs. 11 and 12 show the damages on the building at progressive time after the arrival of the blast pressures, which indicate partial collapse of the structure. Similar to the threat scenario 1, the partial collapse occurs because the floor slab could not transfer the load from the failed columns to the adjacent vertical components. Stress concentration on the slab-column connection region was observed in the analysis due to the flat slab floor system. The structural damage in the event is shown in Fig. 13 which indicates partial collapse of the structure.

Fig. 6 Damage state of building at 2.55s for threat scenario – 1

Fig. 7 Damage state of building at 2.80s for threat scenario – 1

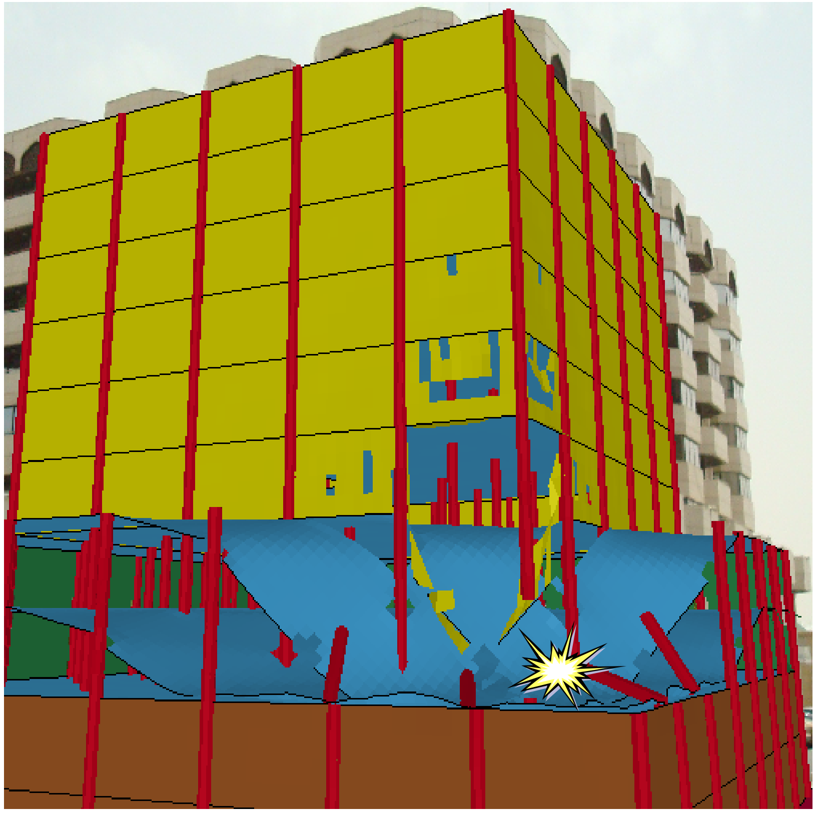


Fig. 8 Partial collapse of building for threat scenario – 1

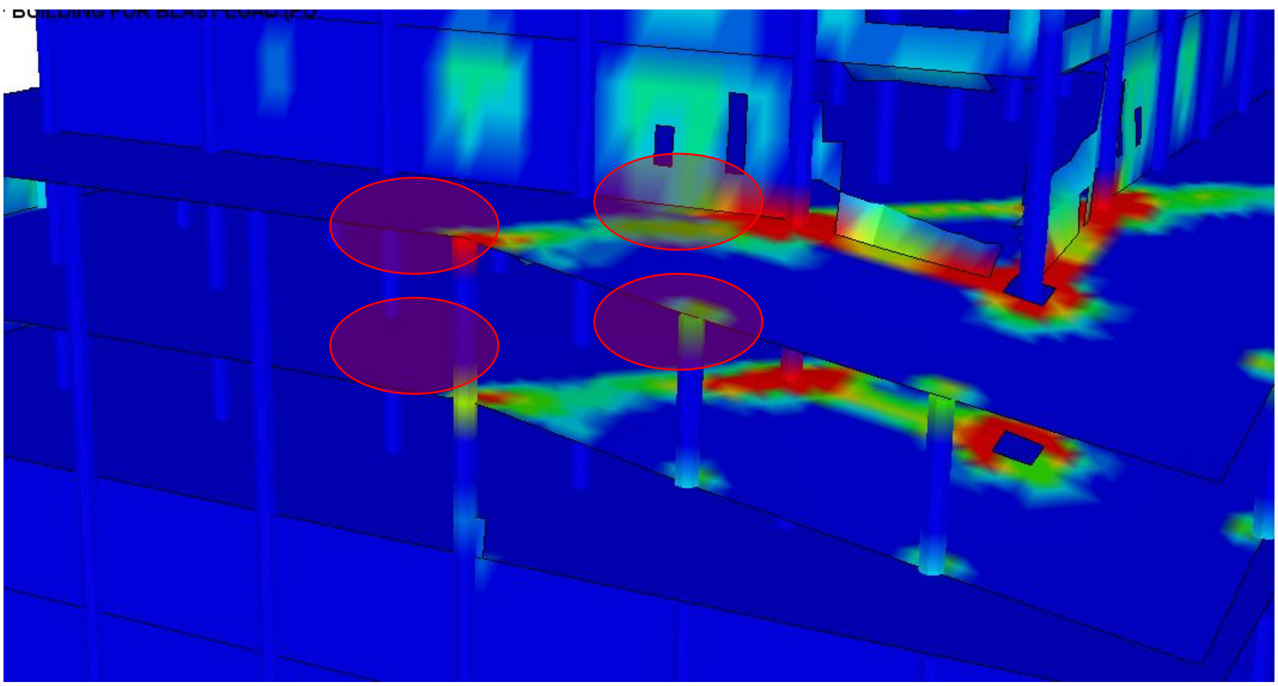


Fig. 9 Damages on the slab-column connection area for threat scenario – 1

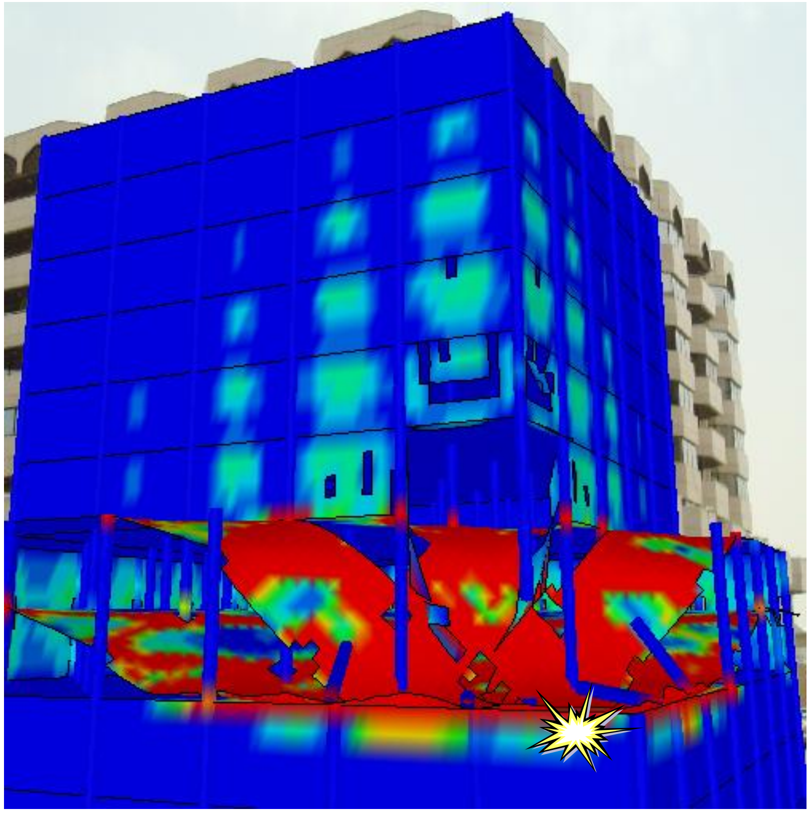


Fig. 10 Damages on the floor system of building for threat scenario – 1

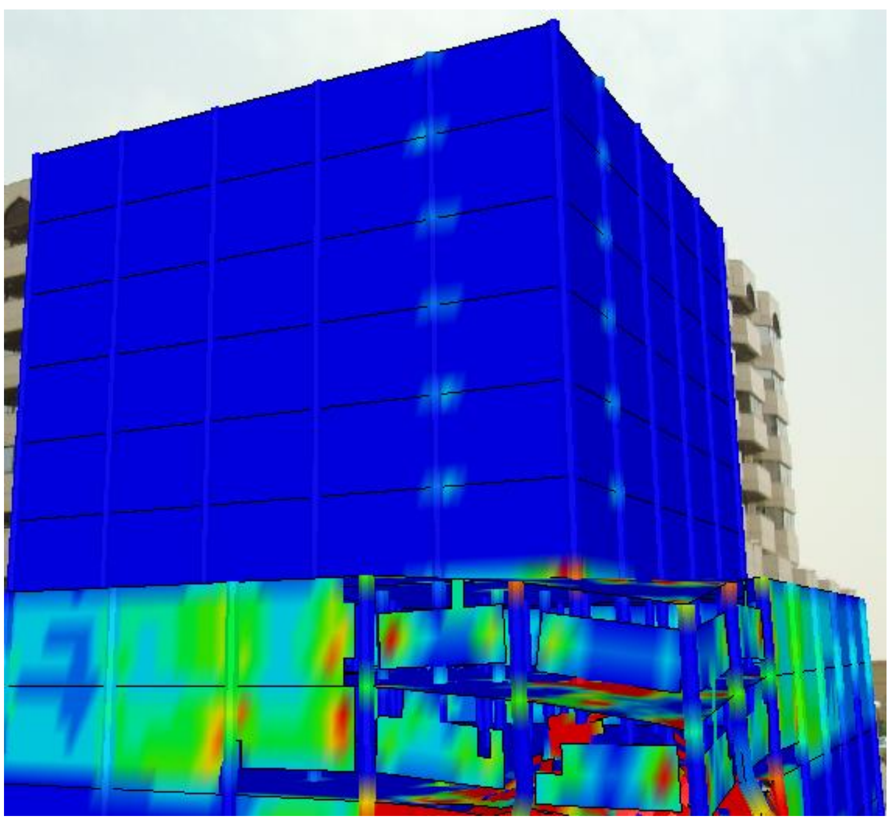


Fig. 11 Damage state of building at 2.83s for threat scenario – 2

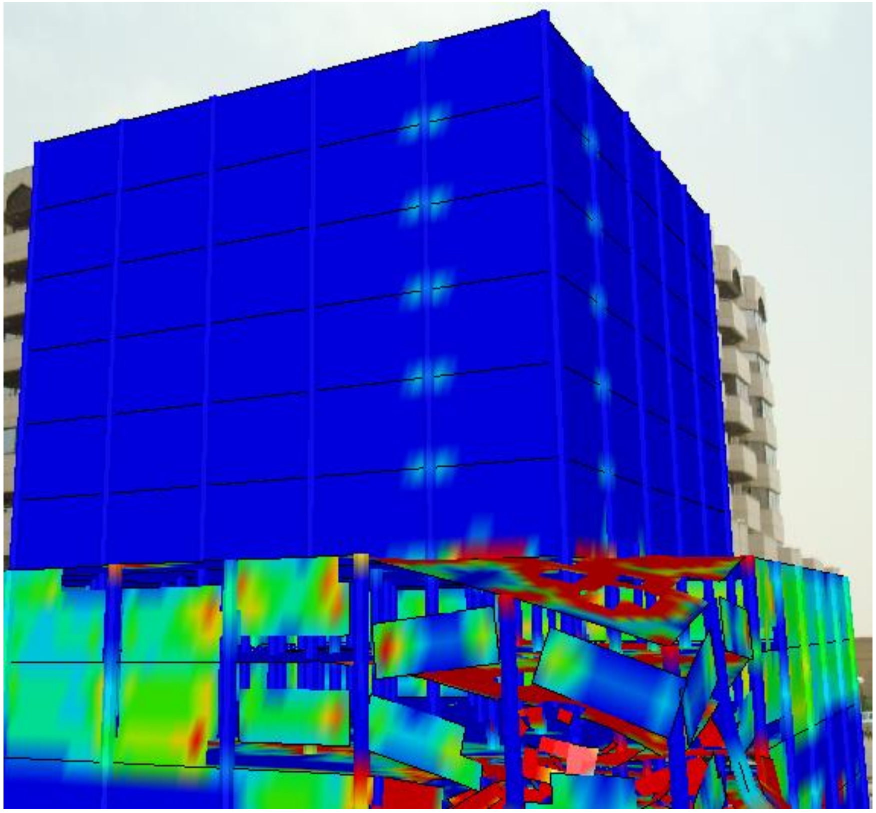


Fig. 12 Damage state of building at 3.10s for threat scenario – 2

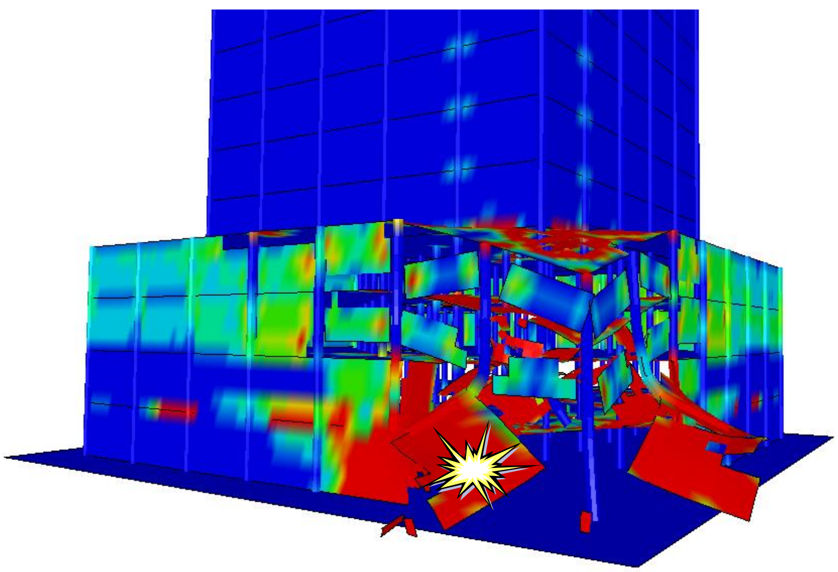


Fig. 13 Partial collapse of building for threat scenario – 2

3.4 Threat Scenario – 3 (TS3)

The third threat scenario was selected in which, the 1000 kg charge was delivered to the underground car park area and detonated on the vicinity of the columns located near the core of the building. The local column analysis indicated that columns within two spans distance from the charge would fail under the blast event.

Figs. 14 to 15 show the damages on the building at progressive time after the arrival of the blast pressures, which indicate partial collapse of the structure. It must be noted that in comparison to threat scenario 1 and threat scenario 2, the damaged area in threat scenario is greater than the remaining scenario due to failure of more columns in threat scenario 3. The structural damage in the event is shown in Fig. 16 which indicates partial collapse of the structure.

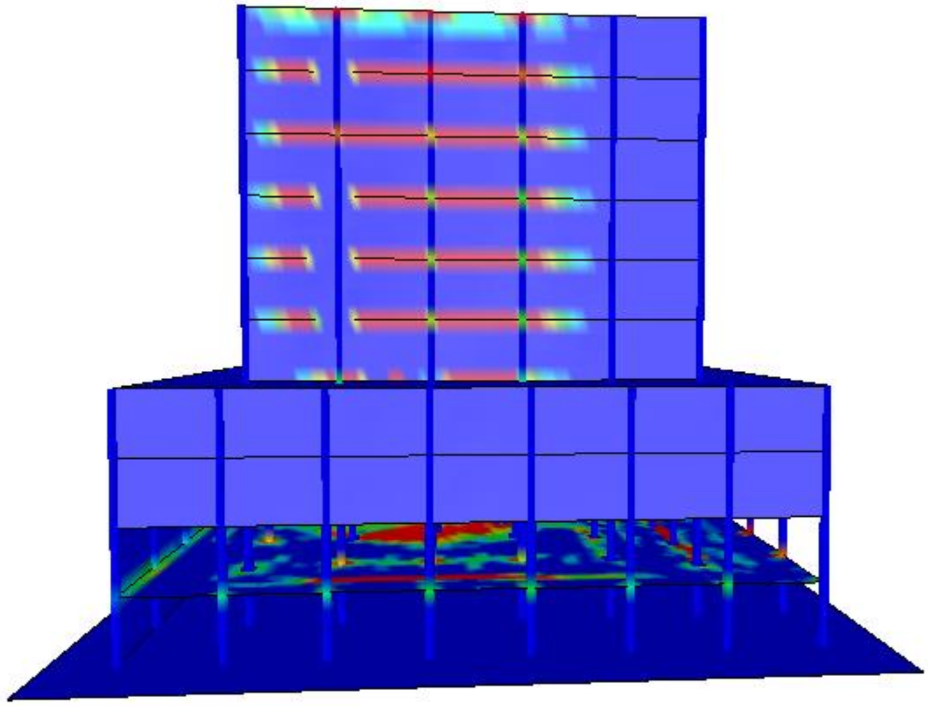


Fig. 14 Damage state of building at 2.85s for threat scenario – 3

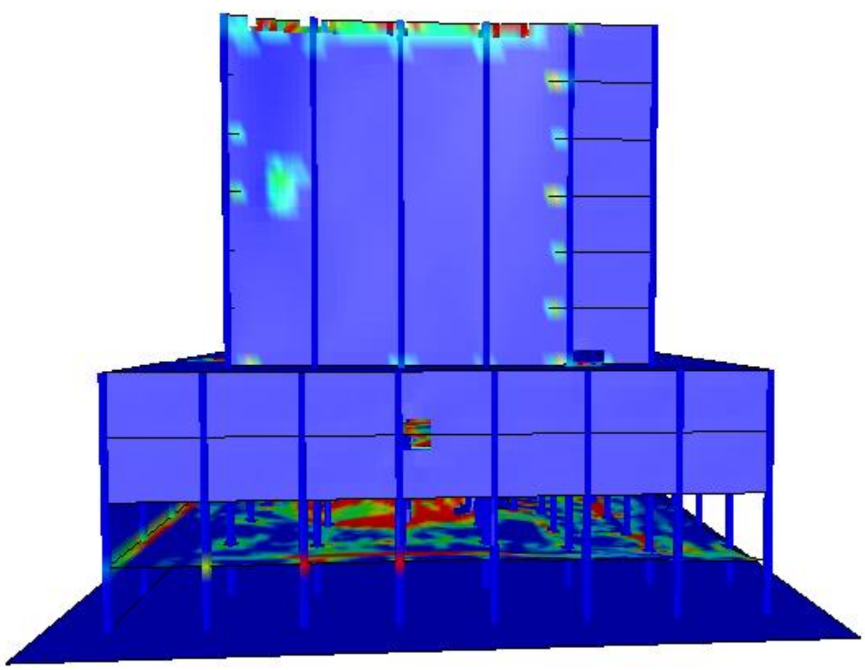


Fig. 15 Damage state of building at 3.1s for threat scenario – 3

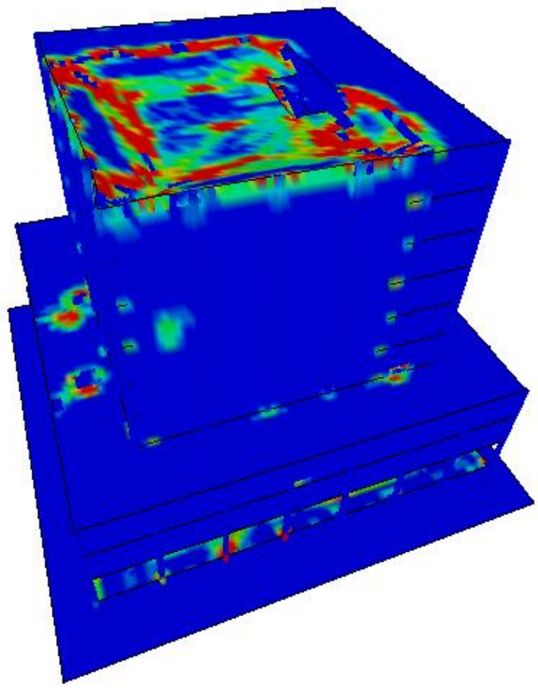


Fig. 16 Partial collapse of building for threat scenario – 3

4 Conclusions

The paper presents the procedure of progressive collapse analysis of RC buildings with practical application. The 3-D finite element analysis of a typical RC building considered for blast analysis 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 internal blast scenario is found to have the most significant effect on the progressive collapse response of the building.

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