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Progressive Collapse Analysis of a Typical RC Building of Riyadh

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ABSTRACT

Recent historic events have shown that buildings that are designed in compliance with conventional building codes are not necessarily able to resist blast effects. Progressive or disproportionate collapse has been observed in past events due to deficient blast performance of the structure, despite compliance with conventional design codes. Safety of structures against blast effects is usually ensured, to a limited extent, through perimeter control; which minimizes damage by preventing the direct impact of the blast effects on the building. With the emergence of blast resistant structural design, methodologies to inhibit progressive collapse through the structural components performance can be developed, though there are no available adequate tools to simulate or predict progressive collapse behaviour of concrete buildings with acceptable precision and reliability. This paper presents part of an effort to find an affordable solution to the problem. Review of the progressive collapse analysis procedures is presented. Preliminary analysis has been carried out to establish the vulnerability of a typical multi-storey reinforced concrete framed building in Riyadh when subjected to accidental or terrorist attack blast scenarios.

1. INTRODUCTION

The study of progressive collapse, although it has intermittently been a subject of interest in the academic, industry and structural engineering communities for several decades, has gained a heightened interest from not only engineers and academics but also from the general public and government institutions. Events such as the partial collapse of the Ronan Point apartment building in 1968 due to a gas explosion, the attack on the Murrah Federal building in 1995, and the terrorists attacks on the World Trade Center and the Pentagon in 2001, have caused waves of interest in the structural engineering community for better understanding the phenomena of progressive collapse resistance and failure of structures through experimental and analytical research. This has resulted in the development of general procedures and guidelines for the design and analysis of structures for progressive collapse prevention, and their implementation on design codes and standards. In parallel, US government agencies such as the General Service Administration (GSA) and the Department of Defense (DOD), have developed guidelines for assessing the potential for progressive collapse of buildings.

The commonly used finite element (FE) programs cannot be used to simulate dynamic collapse behaviour which is characterized by acute nonlinearities associated with the sudden failures of members. It has been suggested in the NIST/GSA workshop [1] and many other engineering forums that urgent research is needed to develop multi-hazard retrofit strategies for existing buildings subject to bomb blasts and other extreme events. Published design guidelines and codes are now available to design engineers for preventing progressive collapse or minimizing the damages caused by progressive collapse of a structure. These include the ACI 318 [2], GSA 2003 [3], DOD 2005 [4], BS 8110 [5], Guidelines for progressive collapse control design [6] and the Eurocode [7]. However, to date, adequate tools to simulate or predict progressive collapse behaviour of concrete buildings with acceptable precision and reliability are not available.

The prediction of possible progressive collapse under specific conditions can provide very important information that could be used to control or prevent progressive collapse. However to date, no adequate tools exist to simulate or predict progressive collapse behaviour of concrete buildings with acceptable precision and reliability. The present paper presents an efficient assessment method and an advanced numerical procedure to assess the likelihood of progressive collapse of reinforced concrete buildings subjected to blast effects. A 3-D FE model has been created for the building by modeling structural elements (such as columns, beams, slabs and core) as well as the non-structural components (such as glass façade, masonry walls) using LSDYNA. The outcome of this study on progressive collapse behavior of concrete buildings may be directly utilized for the design, vulnerability assessment and strengthening of different types of structures ranging from civilian buildings to military facilities.

2. Progressive Collapse Design Procedures

Following the approaches proposed by Ellingwood and Leyendecker [8], ASCE/SEI-7 [9] defines two general methods for structural design of buildings to mitigate damage due to progressive collapse: indirect and direct design methods. Indirect design approach incorporates implicit consideration of resistance to progressive collapse through the provision of minimum levels of strength, continuity, and ductility. Direct design approach incorporates explicit consideration of resistance to progressive collapse through two methods. One is the Alternative Path Method in which local failure is allowed to occur, but seeks to provide alternative load paths so that the damage is absorbed and major collapse is averted. The other method is the Specific Local Resistance Method that seeks to provide strength to resist failure.

Whereas direct design is utilized in the design provisions specifically developed for progressive collapse analysis of structures [3, 4], general building codes and standards [2, 9] use indirect design by increasing overall integrity of structures. ACI 318 [2] requirements for structural integrity are to improve the redundancy and ductility in structures, which are primarily based on providing some continuous reinforcement in beams and floor systems to bridge a damaged support. Breen [10] has shown that improved structural integrity is obtained by provision of integral ties throughout the structure (indirect design) and that the amount of ties can be determined from considerations on debris loading and the amount of damage to be tolerated without determination of the magnitude of the explosive or other abnormal load. Although the indirect design method can reduce the risk of progressive collapse [11,12,13,14], estimation of post-failure performance of structures designed based on such a method is not readily possible.

The GSA 2003 [3] guidelines are primarily based on the Alternative Path Method (APM) and mandates instantaneous removal of one load-bearing element with different scenarios as the initiation of damage. Also the maximum allowable extents of collapse are described. In a linear static analysis the load combination 2*(DL+0.25LL) is considered, where DL and LL are dead and live loads respectively. For an elastic or nonlinear dynamic analysis the factor 2 in the load combination is removed. The acceptance criteria for a linear analysis are similar to the criteria in FEMA 356 [15], which are based on internal force Demand-Capacity Ratios (DCR). GSA 2003 [3] also recommends application of a nonlinear analysis, particularly for buildings having more than ten stories above the grade. For such analysis, acceptance criteria based on rotation or rotation ductility as given in DOD 2005 [4] is provided. DOD 2005 [4] provides two design methods: one employs the Tie Force Method (indirect design), and the other employs the Alternative Path Method (direct design). Distinguishing between ductile and brittle modes of failure, acceptance criteria consist of strength requirements and deformation limits. If an element fails to satisfy deformation limits or its behavior is brittle and fails to satisfy strength requirements, the element is removed and its internal forces are (dynamically) redistributed. Detailed guidelines for analysis procedures are presented in DOD 2005 [4]. The GSA guidelines allow an iterative linear static analysis, as well as nonlinear static and dynamic analyses, whereas DOD 2005 [4] does not allow linear analyses.

The GSA 2003 [3] and DOD 2005 [4] analysis guidelines provide different scenarios for the initiation of a local failure to examine the progressive collapse potential of a building. One of these scenarios is the instantaneous removal of a ground floor column located near the middle of exterior frames. The minimum requirement is that a building can withstand such a local failure without developing progressive collapse. It should be noted that such scenario of an “analytical column removal” is not intended to represent an actual threat scenario, but is intended to be a method for providing redundancy and continuity at the structure level, as well as deformation and load carrying capacity in structural members, for a structure to develop alternative paths of load redistribution and mitigate the occurrence of progressive collapse [16].

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. Complemented with the ability to acquire critical information on initial structural damage quickly and reliably after an event (through a proper instrumentation and monitoring of the critical response parameters of a building), the analytical capability would significantly improve engineers ability to respond and take proper measures in the event of a disaster.

3. BLAST ANALYSIS OF A TYPICAL RC BUILDING

A typical eight storey (Ground + 7 levels) building of Riyadh has been taken up for blast resistance investigation. The building is a reinforced concrete (RC) framed structure with the layout of beams and columns as shown in Fig. 1. The structure has a RC core for lift shafts. The floors consist of one-way joist RC floor system. The peripheral facade consists of in-filled brick masonry with large glazed windows.

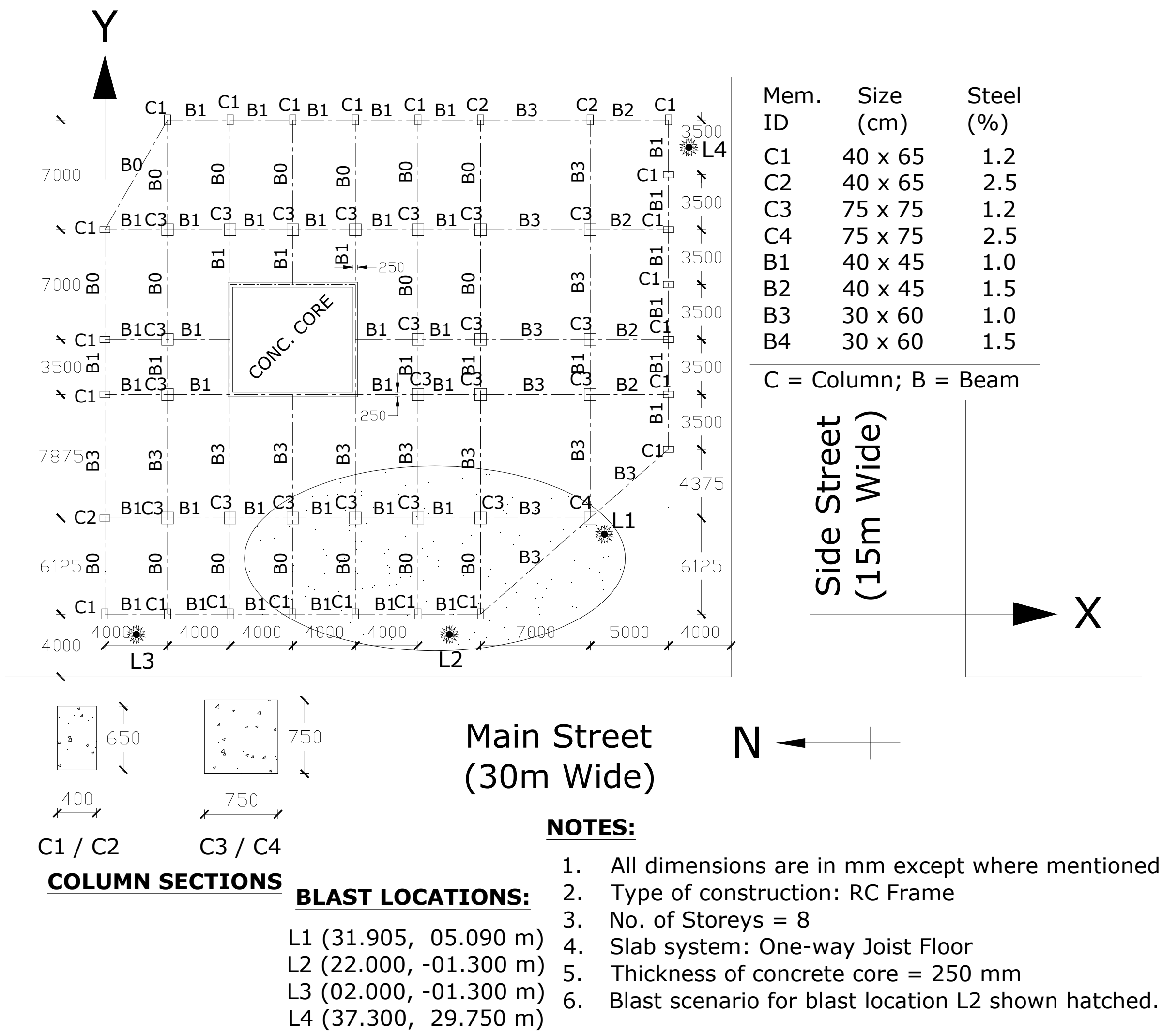


Fig. 1. Layout of building with threat scenario for location L2

Although the building taken up for the investigation is similar to an existing structure, only general geometries and dimensions were used, whereas the reinforcement detail was obtained by first designing the structure based on the geometries. STAAD-Pro software with ACI 318 [2] was used in the design process. The beams and columns were represented by beam element and the concrete core was represented by shell element in the design process. The foundation is assumed to be a thick RC raft. Thus, the columns are fixed at the base. The uniaxial cylinder compressive strength of concrete used in the design is 40 MPa and the yield strength of steel is assumed to be 500 MPa. The section dimensions of various elements taken in the design are the same as those obtained for the existing structure. The section dimensions and the corresponding percentage of reinforcement obtained for different groups of elements are shown in Fig. 1. The percentage of reinforcement in various members has been assumed to be typical for each storey. The percentage of steel adopted for beams is the average value. Based on the structural design, the percentage of horizontal and vertical steel in the concrete core is 1% each in both directions. The thickness of concrete core has been taken as 250 mm and the thickness of the typical concrete slab is 150mm.

3.1. Model Description

The FE analysis was carried out using an explicit finite element code, LS-DYNA, a general purpose transient dynamic finite element program capable of simulating complex real world problems. LS-DYNA uses explicit time integration algorithms and updates the stiffness matrix based on geometry changes and material changes at the end of each load increment.

Geometric Modeling

The FE modeling was carried out in two stages – the local model stage to assess the performance of individual columns against blast pressures and the global modeling stage to assess the overall response of the structure due to the failure of the critical columns.

Fig. 1 shows that there are four type of columns (C1, C2, C3 and C4) used in the building. Whereas, the critical structural components are the perimeter columns in the vicinity of the Vehicle-Borne Improvised Explosive Devices (VBIED). Hence a typical column model was built in order to establish the vulnerability of the vertical component. The columns were modeled using hexahedronal solid elements, while the shear and longitudinal reinforcements of the columns were modeled as a discrete component using beam elements.

The structural geometry was built based on the available information established in the design process. In the global modeling phase, the elements of the structure were simplified into beam elements and shell elements. Reinforced concrete columns and beams 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 – three translations and three rotations about the local Cartesian coordinate 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 model was Hughes-Liu with cross-section integration. The columns are generally rectangular.

The concrete slabs were modeled using a four node quadrilateral and three node triangular shell elements. 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 – three translations and three rotations about the local Cartesian coordinate axes. Stress stiffening and large deflection capabilities are included in the material model. 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. Since the Belyttschko-Tsay element is based on perfectly flat geometry, warpage is not included in the model. All of the façade components were modeled as 4-node shell elements using the Belytschko-Tsay element formulation theory. The use of beam and shell elements for the modeling of the structure leads to an affordable model with reasonable accuracy.

The mesh discretization of shell is such that the aspect ratio of quadrilateral shell elements varies from 1.00 to 1.53, whereas, the minimum included angle for the triangular shell elements is more than 30 degrees. The maximum length of the side of a shell element is taken as 1.66 m. The finite element model of the structure contains a total of 12336 nodes leading to 73734 unrestrained degrees of freedom. The model has 16282 beam elements, whereas the number of shell elements representing RC core, RC slab and facade are 1024, 8496 and 1920 respectively. The column bases have been fixed at the level of raft slab. The completed global model is shown in Fig. 2.

Material Model

The material used in the building 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 constitutive model adopted for RC elements is "Concrete EC2", plastic kinematic model has been used for façade elements. The compressive and tensile strength of concrete taken for the study are 40 and 2.53 MPa respectively and the yield strength of steel was taken as 500 MPa. The yield stress and failure strain for façade is 5 MPa and 0.2% respectively.

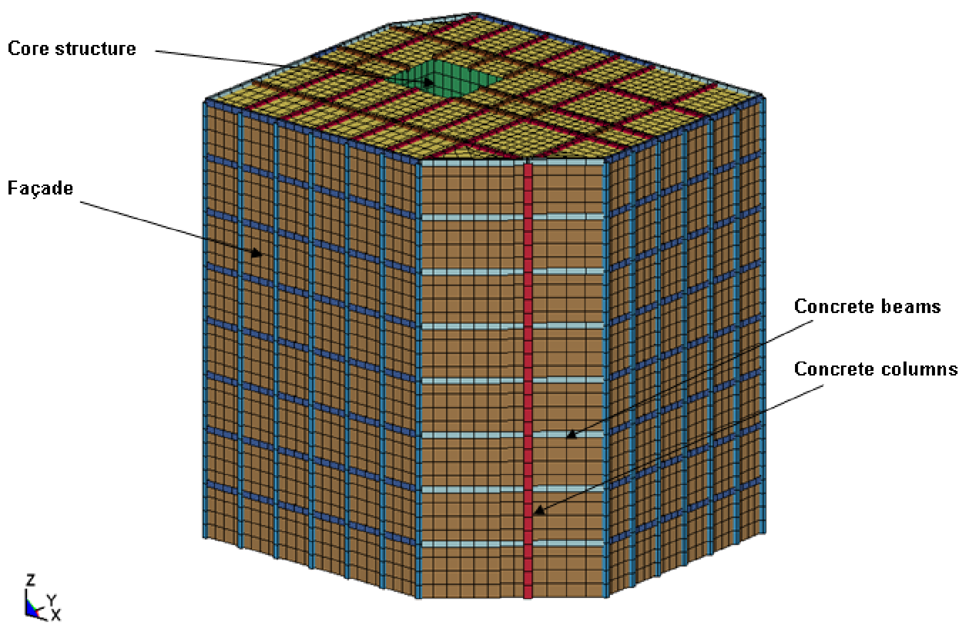


Fig. 2 Finite element model of the building

3.2. Blast Loading

The threat for a conventional bomb is defined by three equally important parameters, namely the type of explosive, charge weight and the standoff distance. There are many explosive devices such as Ammonium-Nitrate Fuel Oil (ANFO) mixture, TNT, C4, Semtex and so forth that may be used by terrorists, but to standardize the parameters, the charge weight of an explosive device is expressed in terms of equivalent TNT weight. Thus there are only two parameters to be considered in the blast analysis i.e. the charge weight and the standoff distance.

Fig.1 shows the layout of the building which is rectangular in plan with chamfered South West (SW) corner. The main entrance and exit of the building is located in the SW corner. The building is located on the intersection of two major roads with the West and South faces of the building facing the roads. There is street-side parking on the South and West sides of the building, whereas the North and East side accesses are limited to pedestrian sidewalk. The major threat to the building from terrorist bombing is through explosion in a parked vehicle. The layout of the building and its surroundings suggest that a vehicle may be parked close to the building on South or West faces of the building which are facing the roads. Thus the minimum standoff distance of the location of explosion for the building has been taken as 2.5 m. Four possible critical locations of explosion, as shown in Fig. 1, have been considered in the study.

The weight of explosive as TNT equivalent is taken as 1000 kg which is the weight that can be carried in a van packed to its full capacity with explosives. The height of blast above the ground has been taken as 1 m because the explosive is assumed to be detonated in a vehicle. Thus the shock transmitted to the building through ground gets diminished and subsequently neglected in this analysis.

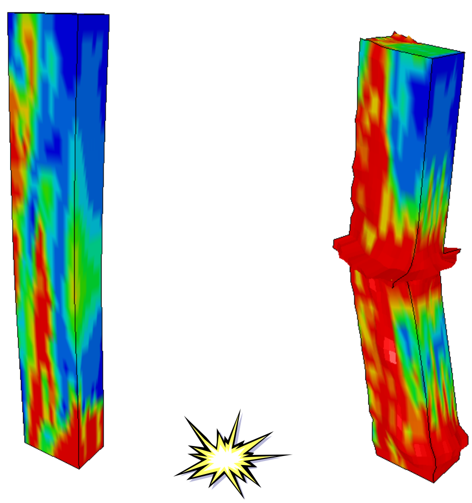
In the analysis, the loads on the critical element have to be applied in two stages to account for both gravity load and blast loads. The gravity load was applied as a ramp loading function, and maintained constant once it had reached the peak gravity load level. The blast pressure was applied to the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

3.3. Results of FE Analysis

Four potential different scenarios were suggested for the analysis as shown in Fig. 1. The selection of these scenarios depended upon the layout of the building with respect to the streets, the standoff-distance provided, and the available access to the building. The detailed results of the second threat scenario are presented in this paper.

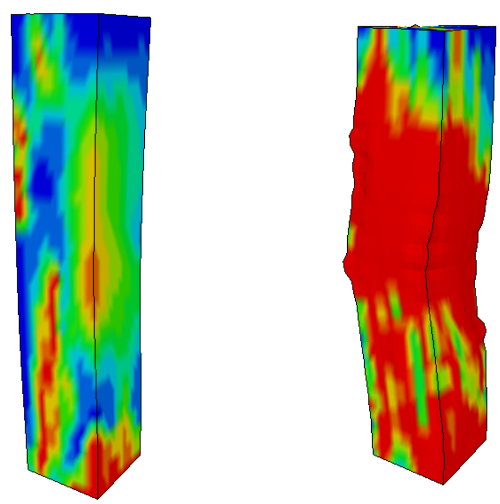
The second threat scenario was selected in which, the 1000 kg charge was placed on the vicinity of the columns located at location L2 as shown in Fig. 1. The threat is located at 2.5m stand-off distance, which is the distance between the centre of the explosive and the building.

The results of local model analysis indicate that the nine columns shown enclosed in the elliptical shaded area will be severely damaged due to fragmentation of concrete and rupture of longitudinal as well as the transverse steel bars and eventually lost their load bearing capacity. The impact of flying debris on different parts of the structure has not been considered. Figures 3 and 4 show the typical damage of columns observed in the analysis.



(a) Time=5ms (b) Time=30ms

Fig. 3. Typical damaged column – C1



(a) Time=5ms (b) Time=10ms

Fig. 4. Typical damaged column – C4

Figures 5 and 6 show the damage on the façade of the building and the structural system at progressive time after the arrival of the blast pressures. The structural damage in the event indicates partial collapse of the structure (Fig. 7). Due to the loss of the columns in the vicinity of the blast event, the gravity load has to be transferred to adjoining vertical components such as the next columns and the core structure via flexural action of the beams and floor slabs. The partial collapse occurs because the flexural stresses exceed the flexural capacity of the beams and floor slabs. These structural components are extensively damaged and, subsequently, lost their load transfer capacity. The slab damage is shown in Fig. 8. One important feature observed in the progressive collapse analysis of the structure is that the damage is localized to the area directly above the failed columns even after the removal of nine columns due to such a severe blast. This is primarily due to the failure of the flexural components which prevents further load transfer to the adjacent spans.

4. CONCLUSIONS

Reinforced concrete structures are vulnerable to progressive collapse if one or more columns are lost due to blast event. It is very important to establish the likelihood of progressive collapse of structures to avoid catastrophic events. Currently, Saudi Arabian standards do not have provisions or recommendations with regard to progressive collapse of buildings. The efficient assessment method and an advanced numerical procedure presented in the paper may be used to assess the likelihood of progressive collapse of concrete buildings. The outcomes of this study can be directly utilized for the design, vulnerability assessment, and strengthening of different types of structures ranging from civilian buildings to military facilities.

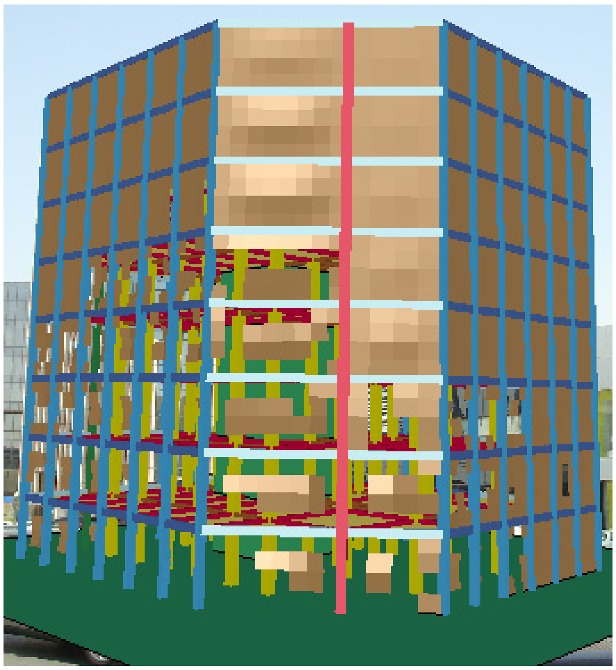


Fig. 5 Damage state of building at 2.75s

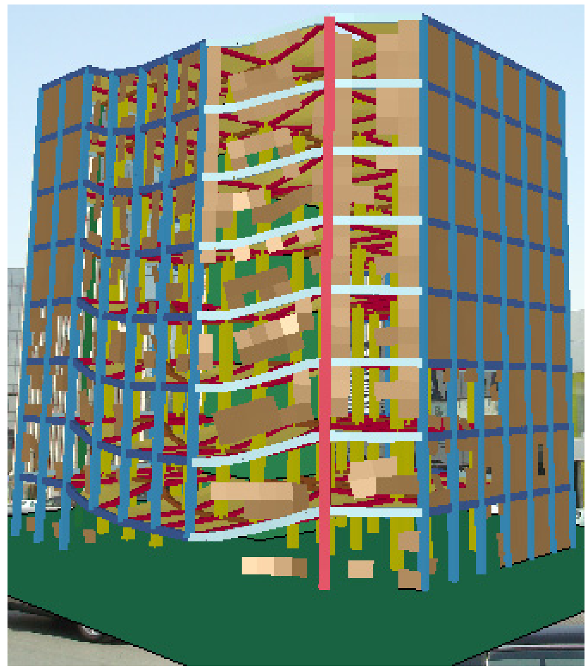


Fig. 6 Damage state of building at 5s

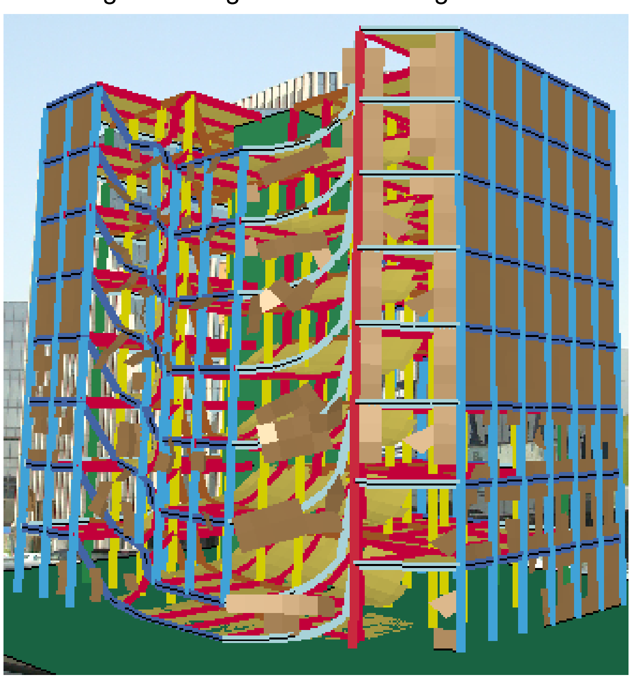


Fig. 7 Partial collapse of building

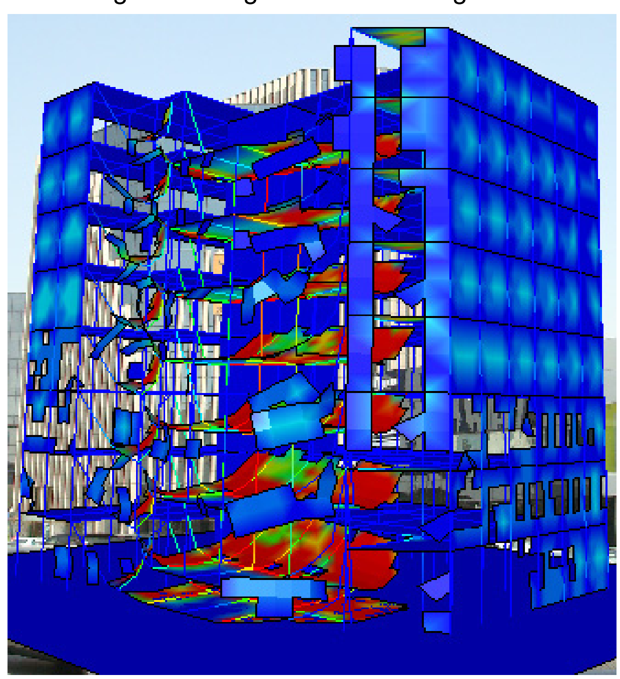


Fig. 8 Damages on the floor system of building

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