Introduction

Progressive collapse, as described in American Society of Civil Engineers (ASCE) standard 7-05, is the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it. To occur, progressive collapse needs abnormal loading to initiate damage as well as a structure that lacks adequate continuity, ductility, and redundancy. With repeated catastrophic collapses of many buildings around the world, many research activities have led to more detailed guidelines on preventing progressive collapses [1–3]. A literature survey on the currently used methodologies to avoid progressive collapse is found in preventing disproportionate collapse and progressive collapse of structures [4–9]. Besides, several research papers have been published presenting both experimental and numerical analysis of progressive collapse in reinforced concrete frames. For example, Brunesi et al. [10] developed fragility functions for low-rise reinforced concrete (RC) framed building structures to be implemented in progressive collapse risk assessment. Two building classes representative of European buildings designed for gravity loads and earthquake resistance in accordance with Euro codes were investigated in the study. Fam and Tan [11] carried out a series of dynamic tests on two-dimensional reinforced concrete beam-column frames simulating the sudden removal of a supporting column via a quick-release device. Weng et al. [12] traced the collapse sequence of the progressive collapse process using a member removal algorithm based on combined flexural/shear/axial failure criteria. Brodsky and Yankelevsky [13] evaluated the possible contributions of infill masonry walls to prevent progressive collapse. Weng et al. [14] tested three scaled moment-resisting RC frames to validate the proposed flexural and axial damage criteria. In addition, three shear-dominant damaged tests were also modeled by authors to assess the proposed shear damage criteria. Lu et al. [15] conducted laboratory progressive collapse tests of five 1/3-scaled RC frame substructure specimens subject to an edge-column-removal scenario.

In the research presented above, most studied frames have either been cast without a column or the column has been suddenly removed during the test without a dynamic load. However, the main reason of column collapse will certainly have a different effect on the overall structural behavior. Between the various abnormal events, explosions have recently become a major cause of structural collapses due to terrorist threats around the world. Explosion is an instantaneous release of energy resulting in a fast increase in pressure. From the explosion, a wave front is formed and spreads spherically. The temperature and pressure resulting from explosion decrease with increasing distance from the center of detonation. After a few milliseconds the whole process is over, and the pressure returns to atmospheric pressure, $P_a$. The typical pressure profile of the explosion wave in time for the explosion in the air is given in Figure 1. At the time of arrival, $t_a$, there is an immediate increase in pressure followed by the positive and negative phases. A simplification of the shock wave is often made to make it easier to describe the load that the wave will give rise to. Usually, the negative phase is neglected and the positive pressure is considered as a critical case.

The specific wave impulse, $i_s$, that is equal to the area under the pressure-time curve from the moment of arrival, $t_a$, to the end of the positive phase and is given by expression:

$$i_s = \int_{t_a}^{t_p} P_s(t)dt$$

Based on earlier numerical and experimental studies, many empirical expressions were proposed to calculate Peak static wave front overpressure, $P_s$; for example, Brode [16] suggested the following equations:

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Received March 29, 2017; Accepted May 21, 2018; Published May 24, 2018

Citation: Hekal GM (2018) Collapse Analysis of a Reinforced Concrete Frame Due to Middle Column Loss by Explosion. J Civil Environ Eng 8: 311. doi: 10.4172/2165-784X.1000311

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Objective of the research

In the current research, the finite element package, Abaqus, is used to conduct dynamic nonlinear analyses of the previous frame to re-evaluate its behavior if the middle column collapsed due to an explosion incident wave. The main aim here is to compare both load-deflection curves and failure patterns of the frame if it was cast from the beginning without a column and if the column exists and then collapsed due to an explosion. In addition, the influence of the location where explosion is applied on the column is studied.

To achieve this goal, a numerical model representing the tested frame was presented and verified against the tested one. Then, the formerly removed column was added to the verified numerical model and loaded by 150 kN distributed as a pressure load on its upper surface. The column was then exposed to a large amount of TNT leading to its total collapse. After column failure, the 150-kN load, carried up to that time by the column, began to transfer to the frame resulting in a progressive collapse.

Four different positions of incident wave action on column were considered in the analysis: the upper part, the middle part, the bottom part as well as the entire front surface of the middle column. In the former three cases, the explosion load was assumed to be acting on an area of 12 × 25 cm of the column front surface. Figure 5 sums the different locations considered in the analysis.

Finite Element Model

Two models were built in Abaqus; a model without the middle...
Applying Explosion on Abaqus/Explicit

The loading effects due to an explosion in air were defined in

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2570</td>
</tr>
<tr>
<td>Young’s Modulus: E (MPa)</td>
<td>25,200</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
</tr>
<tr>
<td>Dilatation angle</td>
<td>20</td>
</tr>
<tr>
<td>Yield stress in compression (MPa)</td>
<td>8.4</td>
</tr>
<tr>
<td>Compressive ultimate stress (MPa)</td>
<td>70.5</td>
</tr>
<tr>
<td>Inelastic strain</td>
<td>0.0018</td>
</tr>
<tr>
<td>Failure stress in compression (MPa)</td>
<td>7.8</td>
</tr>
<tr>
<td>Strain at failure</td>
<td>0.0056</td>
</tr>
<tr>
<td>Ultimate tensile stress (MPa)</td>
<td>8.4</td>
</tr>
<tr>
<td>Tension stiffening</td>
<td>$2 \times 10^{-6}$</td>
</tr>
</tbody>
</table>

Table 1: Data defines concrete material.

Figure 3: Reinforcement detailing of the studied frame (Dimensions in mm).

Figure 4: The frame of study as per Tayel et al. [35].

Figure 5: Different locations of application of explosion load on the middle column.

Figure 6: The compressive and tension behavior at static and dynamic load.
Abaqus/Explicit by the CONWEP model. This model uses a scaled distance, $Z$, based on the distance of the loading surface from the source of the explosion and the amount of explosive detonated. For a given scaled distance, the model provides the following empirical data: the maximum overpressure (above atmospheric), the arrival time, the positive phase duration, and the exponential decay coefficient for both the incident pressure and the reflected pressure. Using these parameters, the entire time history of both the incident pressure and reflected pressure can be constructed. The total pressure $P(t)$, on a surface due to the blast wave is a function of the incident pressure, $P_{\text{incident}}(t)$, the reflected pressure, $P_{\text{reflected}}(t)$, and the angle of incidence, $\theta$, which is defined as the angle between the normal of the loading surface and the vector that points from the surface to the explosion source [37].

The total pressure is defined as:

$$ P(t) = P_{\text{incident}}(t) + \cos \theta - 2\cos^2 \theta + P_{\text{reflected}}(t)\cos^2 \theta \quad \text{for } \cos \theta \geq 0 $$

$$ P(t) = P_{\text{incident}}(t) \quad \text{for } \cos \theta < 0 \tag{6} $$

In Abaqus model, the air blast was defined in interaction module. Based on several trials, the amount of explosive material required to cause failure of column was found to be 100 kg of TNT. The source point was assumed at 1 meter far away from the center of the target surface of the column.

In the model, a vertical pressure was applied on the middle column using an amplitude curve where load gradually increased from zero to 150 kN in 0.007 s. This was found to be the minimum period needed to make the middle column fully loaded by load before applying incident wave, i.e., the time required for the load to reach supports. The load was then kept constant with time till the end of analysis. After 0.007 s, the bomb of 100 kg TNT was exploded. The applied pressure and source of detonation are shown in Figure 7, while Figure 8 shows the incident wave due to detonation of 100 kg TNT applied at 1 meter away from the middle column. The incident wave reached column surface after 0.00025 s of the detonation with maximum pressure of $1.06 \times 10^5$ Pa. this pressure was rapidly decreased in an exponential manner as shown in figure.

**Analysis, Results and Discussion**

**Comparison between model (1) and experimental results**

Figure 9 compares the failure modes in the tested frame and Model (1). It is clear that the shape of collapse is identical in both experimental model and numerical Model (1) where the collapse occurred beside connection 1 and at connection 2 in one side. The load-deflection relationships is also plotted in Figure 10 which shows fair agreement where the maximum capacity in the tested frame reached 60.5 kN at deflection 24 mm while in Model (1) the capacity is 60.25 kN at 20 mm deflection.

**Comparison between Model (1) and Model (2)**

Load deflection relationships and failure patterns for the all studied cases are summed in Figures 11 and 12 respectively. Besides, the participating ratios of column (B-2) in the total capacity in all cases are presented in Table 4. In general, it can be observed that when the explosion was applied to the column, the load-deflection relationships as well as the frame collapse patterns differed significantly from Model (1), where the four cases of Model (2) showed a symmetrical pattern of failure and irregular shapes of load-deflection curves in contrast to Model (1).

Moreover, the total behavior of frame was strongly dependent on the location of application of explosion load. When explosion was applied at the upper part of the column, the capacity decreased to 51 kN at 2 mm indicating a sudden failure. The same case appeared when the explosion was applied to the entire column face as the capacity was 46 kN.
Table 3: Data defines stirrups steel material.

<table>
<thead>
<tr>
<th>Reference density (kg/m³)</th>
<th>Young’s Modulus, E (MPa)</th>
<th>Tensile Strength, f (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2,000,000</td>
<td>316</td>
<td>125</td>
</tr>
<tr>
<td>250</td>
<td>316</td>
<td>125</td>
</tr>
</tbody>
</table>

Table 4: The percentage of reaction at column (B-2) of total capacity of frame when explosion was applied at different position of column.

<table>
<thead>
<tr>
<th>Model (1)</th>
<th>Upper explosion</th>
<th>Middle explosion</th>
<th>Lower explosion</th>
<th>Explosion on total column</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.7%</td>
<td>59%</td>
<td>20%</td>
<td>23%</td>
<td>54.50%</td>
</tr>
</tbody>
</table>
kN and the deflection was 6mm. Application of blast at middle part of column led to a capacity of 69 kN at 35 mm deflection while exploding the lower part of the column raised the capacity to 81 kN at 33 mm deflection.

The previous values indicate that the worst case appeared when explosion was applied at the upper part of column while the performance of the frame got better as the location of explosion moved away from the connection as the frame showed a remarkable increase in capacity.

The ratio of column B2 participation in the overall capacity reached 59% and 54.5% when explosion was applied at upper part and on the entire front face of the column respectively. This ratio decreased to 20% and 23% at middle and lower explosion. In contrast, column participation was only 6.7% in case of Model (1).

Conclusions

The main findings of the current work can be summarized as follows:

1. Finite element model results showed fair agreement with the tested frame regarding to mode of failure and load-deflection relationships.

2. Concrete at high strain rate has higher properties in both tension and compression which significantly affects the structural behavior in the numerical model.

3. The total behavior and failure pattern of the frame is significantly influenced by the nature and position of applications of the explosion load. So, the common methods followed to study progressive collapse by removing some elements don’t represent the real behavior as they ignore the main reason of collapse.

4. Beside the damage that occurred in frame FR (1), there was damage that observed in the transverse beam and hence column B-2 participated in total capacity of the structure. This indicated that considering 2D structures only is not enough to express the real behavior of a structure.

References


