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Mitigation of progressive collapse of buildings resulting from accidental explosions

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Abstract— Accidental explosions are frequently encountered nowadays due to several reasons as gas vehicles, military works and terrorist attacks. This paper introduces a study of the effects of accidental explosions in front of façade columns of RC buildings and their mitigation. Nonlinear finite element analysis is carried out on space frame due to the rapid release of energy in the form of light, heat, sound, and a shock wave resulting from explosion. The spherical free-air bursts of TNT detonation and expansion of the spherical charges are developed using a one dimensional Euler wedge through the ANSYS AUTODYN software. Different cases of charge weights (500 and 1000 kg TNT) and stand-off distances (3, 5 and 10 m) are used in the study. Overpressure, displacements, stresses, and damage index are the common response parameters investigated. Interior shear walls are applied as common mitigation techniques and their effects on the building damage are investigated.

Index Terms— Explosive load- Reinforced concrete frames - Numerical analysis – AUTODYN - Mitigation.

1 INTRODUCTION

Progressive collapse as major field of study has been attracting researchers and code writers during recent decades. Progressive collapse is defined as the failure of one or a group of key structure load-carrying elements that gives rise to a more widespread failure of the surrounding structural elements and partial or complete structure collapse or the spread of an initial local failure from member to member resulting in the collapse of an entire structure or a disproportionately large part of it [2]. Loss of lives and economic losses encountered in previous events and accidents verified the importance of studying progressive collapse. The famous Ronan Point apartment building partial collapse, the disproportionate collapse of the Alfred P.Murrah Federal Building [3], the damage of Al-Khobar building [4], and the catastrophic collapse of the twin World Trade Center towers [5], all except the first caused by terrorist attacks. Regulations and standards were developed to consider the probability of accidental or terrorist explosions into account in designing new buildings or evaluating existing buildings to avoid progressive collapse [6].
Research related to progressive collapse includes too many branches as exprimental investigations, loss of support analysis and the direct effect of explosions in addition to mitigation techniques. Luccioni, et al. [7] tested concrete slabs under explosive loads and concerned with the behavior using a nonlinear dynamic analysis. A novel simplified framework for the progressive collapse assessment of multi-story buildings, considering the sudden column loss as a design scenario was also investigated [8]. A case study was proposed to learn the progressive collapse process of a typical steel-framed composite building. It was concluded that steel-framed composite buildings with typical structural configurations could be prone to progressive collapse initiated by local failure of a vertical supporting member.

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A large number of applications using the finite elements were also developed for the detailed 3D model structure, and the main modelling parameters affecting the numerical results were estimated [8]. ANSYS AUTODYN is used to calculate TNT equivalencies for three different explosives at varying distances from the explosive charge, using both peak pressure and impulse methods. The TNT equivalency curves were different depending on whether the equivalency was based on peak pressure or impulse. When TNT equivalencies are used, an understanding of how they are determined and when they are valid is essential for engineers designing against blast loads [9].

Experimental results about the behavior of steel fiber reinforced concrete panels subjected to blast loads were also carried out [10]. Nonlinear static and nonlinear dynamic analyses to estimate the progressive collapse resistance of a building subjected to column failure are carried out by Tsai and Lin [11]. Mohamed [12] proposed a new mitigation scheme to resist the progressive collapse of reinforced concrete buildings that resulted from potential column failure by installing steel cables parallel to the columns either externally connected the ends of the beams for retrofitting existing buildings or embedded in the columns for upgrading new buildings. Also, the proposed scheme included placing a hat braced steel frame on the top of the buildings by which the vertical cables are hanged and supported. When a column failure occurs, the vertical cables will transfer the floor loads upward to the hat braced frame which in turn redistributes these transferred loads to the adjacent columns. Kwasniewski [13] carried out the progressive collapse analysis of an existing eight-story steel framed structure built for fire tests using nonlinear dynamic finite element simulations according to the GSA guidelines. Adding fibers to reinforced concrete enhanced the durability and ductility of concrete. For fiber-reinforced concrete, fiber types, and properties of fiber length, and concrete strength were discussed. Adding steel fibers to concrete had been shown to enhance the concrete's post-crack behavior from a blast [14].

The objective of this paper is to study RC buildings subjected to accidental explosions facing external columns. Different charge weights of 500 and 1000 kg equivalent weight of TNT and different stand-off distances of 3, 5 and 10 m are considered. These different cases are investigated through the analysis of the building response including displacements, stresses and damage ratios. Then the effects of common mitigation techniques are then applied to the building to investigate their effects on previously mentioned parameters. Conclusions about the effects of explosive charges, stand-off distance, and mitigation techniques used are all investigated.

2 NUMERICAL SIMULATIONS FOR AIR AND TNT

In the analysis of RC building under explosion in air, the problem may be divided into two phases, the detonation of charge forming air shock wave, and the propagation of air shock wave including interaction with rigid RC building. Before explicitly addressing air-explosive effects on a building system, it is important to understand the nature of explosions themselves [15]. An explosion is a rapid release of energy in the form of light, heat, sound, and a shock wave. The shock wave consists of highly compressed air traveling radially outward from the source at supersonic velocities [15, 16]. Overpressures reduce rapidly with distance. These overpressures decay exponentially with time and their duration is typically measured in milliseconds [17].

ANSYS AUTODYN software is used to specify spherical free-air bursts of TNT. The detonation and expansion of the spherical charges are developed using a one dimensional Euler wedge, with a predefined geometry type "wedge", as shown in Fig. 1. The Euler grid was filled with the material model for the blast over a quarter-circular area to represent a hemispherical charge positioned at the origin. A flow out condition was applied to the end of the wedge to allow the pressure wave to flow out of the grid. The explosives and air are modelled using the numerical models available in the AUTODYN library.

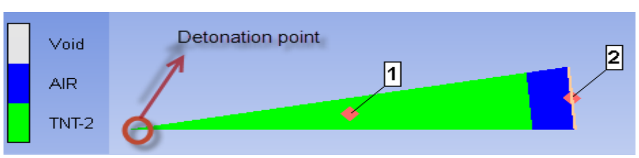

Figure 1: One-dimensional numerical model.

The Jones-Wilkins-Lee (JWL) equation of state is used for explosive. The wedge is filled with an explosive material model to the estimated charge radius (dependent on the mass and density of the explosive being simulated), and the remaining elements are filled with the material model for air. The air is identified a 3D Euler FCT sub grid with the simplest forms of equation of state. It is an ideal polytrophic gas, which may be used in many applications developing the motion of gases which may be derived from the laws of Mader [18]. Additionally, the first two terms of the JWL EOS (eq. 1) become negligible and the EOS collapses to that of an ideal gas EOS, at large volumetric ratios. Nonlinear finite element analysis is carried out on RC building due to the rapid release of energy in the form of light, heat, sound, and a shock wave resulting from explosion. The spherical free-air bursts of TNT detonation and expansion of the spherical charges were developed using a one dimensional Euler wedge.

$$p = A[1-\frac{\omega}{R_1 v}]e^{-vR_1} + B[1-\frac{\omega}{R_1 v}]e^{-vR_2} + \frac{\omega}{v}E \qquad (1)$$

Where A, B, R1, R2, and ωare constants obtained by calibration of test data, p is the pressure, V is the relative volume and E is the specific internal energy. The C-J pressure is a function of the kinetic energy and then velocity of the expanding copper cylinder. The velocity of the expanding copper is specified until the wall of the copper cylinder fractures. A table of parameters for the JWL EOS for many explosives is available in the LLNL Explosives Handbook [19].
The JWL EOS is converted to an ideal gas EOS with Gama = 1.35 and reference density = 1 e^{-4} [1] Modeling the air and TNT are carried out using parameters shown in Tab. 1.

TABLE 1: PROPERTIES OF AIR AND EXPLOSION

Material	Air	TNT	TNT(Ideal)
Equation of state	Ideal Gas	JWL	Ideal Gas
	$\gamma = 1.4$	Standard	$\gamma = 1.35$
	$\rho = 1.225 \times$ 10-3 g/cm3	Library data	$\rho = 1.0 \times$ 10-4 g/cm3
	Ref. Energy = 0.0 µJ		Ref. Energy = 0.0 µJ
	Press. Shift = 0.0 kpa		Press. Shift = 0.0 kPa
Initial conditions	$\rho = 1.225 \times$ 10-3 g/cm3	Default	From detonation
	Ref. Energy = 2.068e5 µJ/mg		Model/remap data

For this study, the propagation of air shock pressure waves subjected to the RC building above ground due to the explosions with different charge weights of 500 and 1000 kg equivalent weight of TNT at standoff distances of 3, 5 and 10 meters are considered in the study in front face of building for intermediate column. The detonation charge is located at a height h = 1.5 m above ground surface and its interaction with the model is produced with AUTODYN 3D. The Reinforced concreteis modelled with RHT standard material model in the material library of AUTODYN that describes the behavior of concrete. Figure (2) shows the geometry and material properties of RC framed building. The two story frame building considered has two bays with a span of 5 m each in the z-

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direction, and one bay with a span of 3 m in the x-direction. The story height is 3 m for all levels. The dimensions of all the columns are 300 mm × 500 mm, and the beams are 200 mm ×400 mm. All the columns and beams are modeled with 35 MPa as solid elements with a single integration point while the 200 mm thick slab was modeled with a dimension of 5 m×3 m as shell element. At ground floor, gauge 1 taken at the fixed base for façade intermediate column fronting to the explosive load and gauge 2 taken at this column - beam connection.

The constitutive equation for this model is as in Eqn. 2 which is able to represent the pressure and deformation of this material as reported by Hiermaier, Riedel and Thoma [21] as follow;

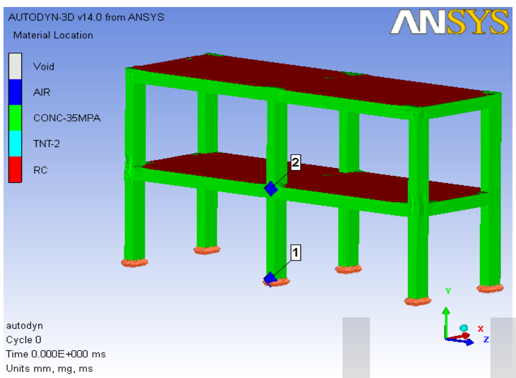


Figure (2): Material status of original building.

Where;

$$Y_{fail} = f_c(A(\frac{p}{f_c} - \frac{p_{HTL}}{f_c}.F_{Rate})^N)R_3(\theta)F_{Rate}(\varepsilon) \quad (2)$$

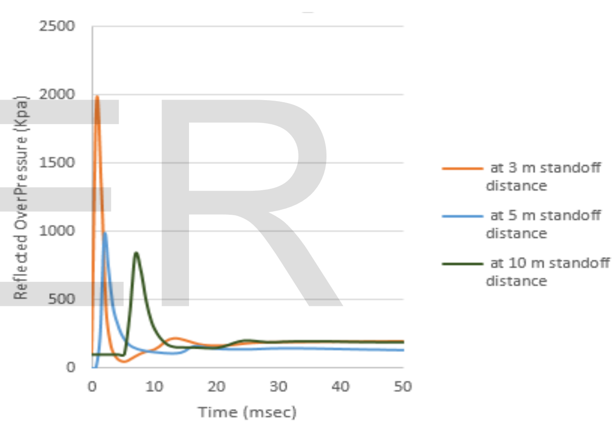
Y_{fail} = Failure surface
f_c = Compressive Strength
P_{HTL} = Tensile Strength
A and N = Constant value
P = Hydrostatic Pressure
F_{Rate} = Strain Rate Factor
R_3 (Θ) = Internal Resistance Force for the concrete

3 Analysis Results

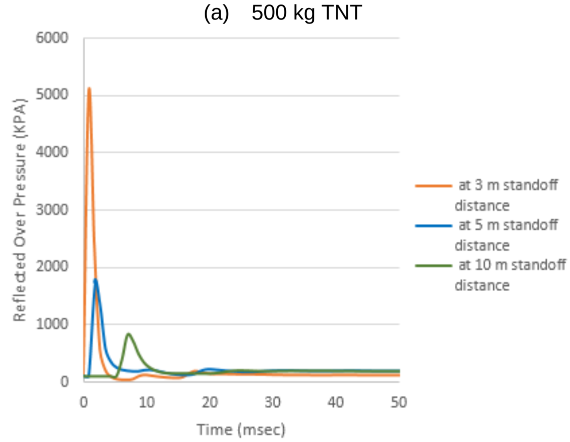
The results obtained for an explosive loads of 500 and 1000 kg of TNT located 1.5 m above the ground level with 3, 5 and 10 m standoff distances from the front façade intermediate column are presented in this section. Reflected overpressure, lateral displacements and damage index are the common response parameters investigated in this section.

3.1 Reflected overpressure Time History

The reflected overpressure histories at the column – beam connection of the front façade intermediate column are shown in figure (3) for 500 and 1000 kg TNT. It is clear from this figure that the reflected overpressure is very high on the surface of the intermediate column as the shock front propagates across and around the column. The shape of the pressure time history, as commonly encountered, has sudden increase in pressure after short time followed by gradual decrease till reaching small value then vanishes. It is also observed that the peak value of overpressure is lower and its arrival time is later for far stand-off distance. As shown in figure (3-a), For the 500 kg TNT charge weight at 3 m stand-off distance, the peak pressures at the shock front (approximately 2 MPa) at time 0.776 millisecond. The overpressure then decays to reach its steady state value of 0.187 MPa at time 29.7 millisecond. When the stand-off distance increased to 5 m and 10 m, the peak pressures at the shock front reaches approximately 1 and 0.8 MPa at times 2 and 7 millisecond respectively, and the steady state reaches 0.144 and 0.193 MPa after 19.1 and 28.1 millisecond respectively. It is shown from figure (3-b) that for 1000 kg TNT charge weight at 3, 5 and 10 m standoff distance, the peak pressures at the shock front is approximately 5, 1.8 and 1 MPa at time 0.87, 1.7 and 8.7 millisecond respectively. It then decays with time reaching 0.081, 0.184 and 0.148 MPa at times 20.2, 29 and 19.3 millisecond for standoff distance 3, 5, and 10 m respectively.



(a) 500 kg TNT



(b) 1000 kg TNT

Figure (3): Reflected overpressure for spherical charge at front façade intermediate column-beam joint (gauge # 2)

Figure (4) shows the relation between the maximum reflected overpressure at front façade intermediate column – beam joint and the standoff distance due to 500 and 1000 kg explosive loads. It is observed that increasing the standoff distance decreases the maximum reflected overpressure with sharpe rate for near standoff distance and with small rate for far one. For the 500 kg TNT, overpressure is decreased from 2.0 to 1.0 Mpa for increasing the standoff distance from 3 to 5 m while decreased only from 1.0 to 0.8 for increasing the standoff distance from 5 to 10 m. The same observation was noticed for the 1000 kg TNT case where overpressure is decreased from 5.0 to 1.8 Mpa for increasing the standoff distance from 3 to 5 m and decreased only from 1.8 to 1.0 for increasing the standoff distance from 5 to 10 m. This also indicates that for higher explosive charges, the decrease of overpressure is more sensitive to the first small distance changes. The decrease of overpressure with distance can be attributed to the fact that the energy from a blast decreases rapidly over distance.

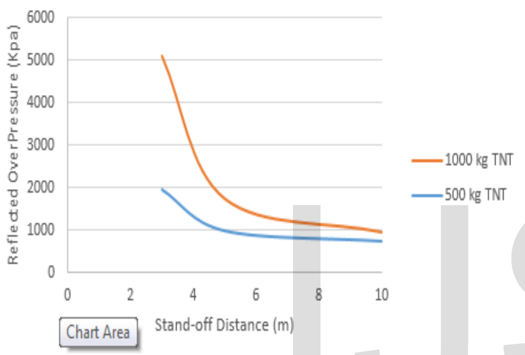


Figure (4): Relation between the maximum reflected overpressure at front façade intermediate column – beam joint (gauge#2) and standoff distance.

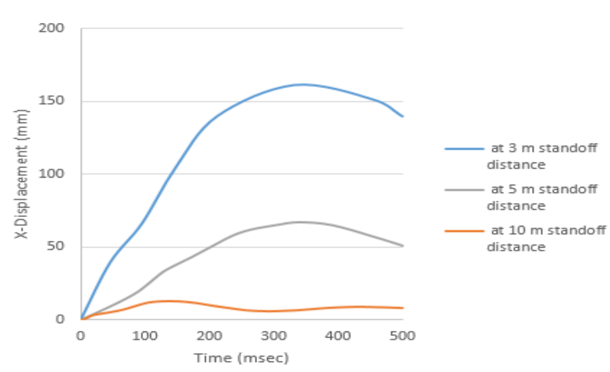
3.2 The Original RC Building

At first, the response of the original building to blast loads is discussed. The time history of displacement at the intermediate column-beam connection (gauge 2) is shown in figure (5) for the 500 and 1000 TNT charges. As shown in the figure, the lateral drift (displacement in x-direction) is plotted against time for 500 millisecond interval. For all cases, the displacement tends to increase with time after the release of explosion energy to reach its peak value and then after that decrease again gradually. Its peak values are observed to be 161.2, 67 and 13.3 mm due to 500 kg TNT for 3, 5 and 10 m standoff distance and 229, 111 and 29.5 mm due to 1000kg TNT for 3, 5 and 10 m standoff distance respectively. These values indicate logical behavior in terms of increased displacements for heavy TNT charge and for small stand-off distance. Peak values of deformation are obtained at times 332, 339 and 139 milliseconds for 3, 5 and 10 m standoff distance in case of the 500 TNT charge and 332, 300 and 116 milliseconds for 3, 5 and 10 m standoff distance in case of the 1000 TNT charge. This can be attributed to the early dispersion of the reflective waves for far stand-off distance and the weakness of the released energy.

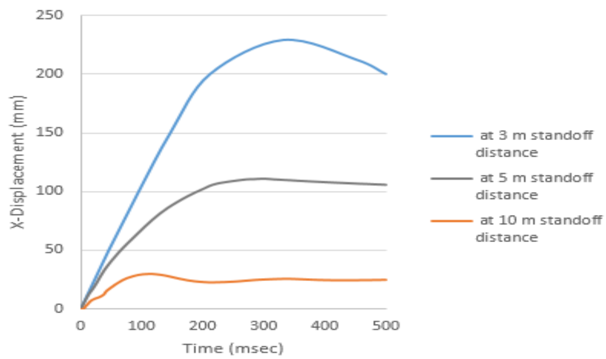
Figure (6) shows damage index contours of the building subjected to 500 kg and 1000 kg charge weights at 3 and 10 m stand-off distances. It can be clearly observed that the case of 3 m stand-off distance caused the building to damage totally (damage index =1) at several regions. The fixed base is observed as the most affected part of the frame such that the damage index reached 1 for all columns across their cross sections because the explosion energy tends to overturn the building about its base. Less effect is observed at the column-beam connection in the first and second levels. The majority of column-beam connections are damaged with damage index less than unity. In case of 500 kg TNT, more increase of stand-off distance to 10 m decreases the extent of damage such that fixed bases and all column-beam connections are less damaged. While, in case of 1000 kg TNT, more increase of stand-off distance to 10 m decreases the extent of damage such that only fixed base reached unity damage index and all column-beam connections are less damaged.

The time histories of damage at fixed base and at the column-beam connection for the 500 kg explosive load at 3, 5 and 10 m stand-off distances are shown in figure (7). As shown in figure (7-a), at fixed base, the peak damage index equals 1.0, 1.0 and 0.236 at time 70, 127 and 137 milliseconds for the 3, 5, and 10 m stand-off distances, respectively. These values illustrate that the column base, damaged completely in case of 3 and 5 m stand-off distances after very short time while in case of 10 m stand-off distance, the frame can survive without complete damage. As shown in figure (7-b), at column – beam connection, the peak damage index equals 0.23, 0.17 and 0.085 at 287, 254 and 154 milliseconds following steady case of the peak damage. This indicates that the column at its beam intersection has little damage for all stand-off distances and that the critical section for damage is at the column base.

The damage index time histories for the 1000 kg explosive load at 3, 5 and 10 m stand-off distances are shown in figure (8) at fixed base and at the column-beam connection. As shown in figure (8-a), at the fixed base, the peak damage index reaches the value of full damage (1.0) for all stand-off distances. This full damage occurs at time 25, 54 and 137 milliseconds for the 3, 5, and 10 m stand-off distances, respectively. These values illustrate that the column base is damaged completely in all cases of stand-off distances after very short time. The time consumed to develop the full damage is more for more stand-off distance. As shown in figure (8-b), at column – beam connection, the peak damage index equals 0.57, 0.26 and 0.126 at 111, 381 and 243 milliseconds following steady case of the peak damage. This indicates that the column beam connection is damaged relatively in case of 3 m stand-off distance and has little damage for 5 and 10 m stand-off distances and that the critical section for damage is at the column base.



(a) 500 TNT charge



(b) 1000 TNT charge

Figure (5): Displacement time history of front façade intermediate column - beam joint (gauge#2)

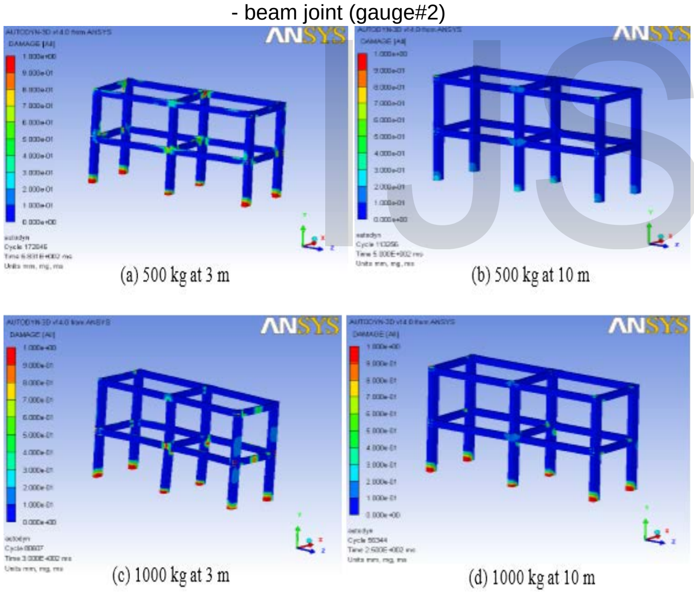
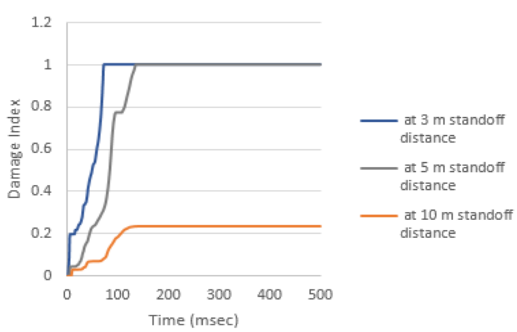
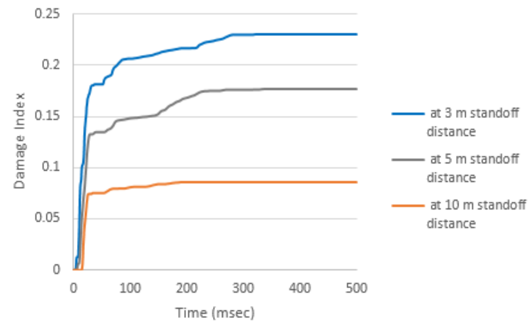


Figure (6): Damage contours for RC frame building for different blast loading.

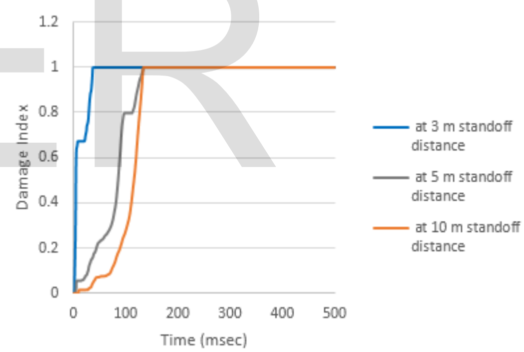


(a) At fixed base

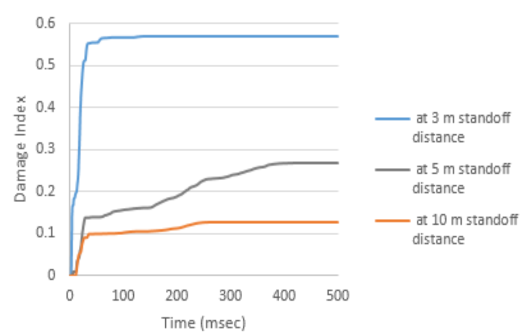


(b) At column – beam joint

Figure (7): Damage index response for front façade intermediate column for RC frame building due to 500 kg TNT at 3, 5, 10 m standoff distance cases.



(a) At fixed base



(b) At column – beam joint

Figure (8): Damage index response for front façade intermediate column for RC frame building due to 1000 kg TNT at 3, 5 and 10 m standoff distance cases.

3.3 Mitigation of Progressive collapse using shear walls

To mitigate the progressive collapse resulting from explosions, shear walls are added on the ground floor. The thickness of the wall is 200 mm and the height is the full story height (3000 mm). Shear walls are characterized with an elasto-perfectly plastic model as mitigation for RC building against blast loads. RC walls are widely used in construction as well as protective structure, because of its good energy absorbing characteristic under high pressures, high quality, speedy construction, cost and stiffness efficiency. It is important to follow the proper design standards or guidelines, and also to identify the possible threats and their risk of occurrence to enable the characteristic of the explosive loads, in designing of protective structures. RC walls are used to protect a building or an area from blast damage when exposed to explosions. CONCRETE-140 MPA [1] is used for shear walls modelling as shown in figure (9). The same charge weights and stand-off distance used for the original building are used in this case.

The time history of lateral displacement at the intermediate column-beam connection (gauge 2) is plotted in figure 10 for the 500 and 1000 TNT charges. The figure illustrates that the lateral drift of the building increases too much during the first few milliseconds, oscillates little bit, and then decay to very negligible value afterwards for both cases. The oscillation behavior shown reflects the effect of adding the shear wall on the natural period of the building. The shear walls increase the stiffness of the building and then shorten the period of the building leading to such vibrating behavior in the studied time (500 ms). As shown in figure (10-a), the maximum lateral displacements registered at gauge 2 in front façade intermediate column – beam connection are 0.7, 0.5 and 0.26 mm due to 500 kg TNT at 3, 5 and 10 m standoff distances, respectively. These results demonstrate that the existence of shear walls has great effect on reducing the lateral drift resulting from blast loads. Referring to figure (10-b), for the 1000 kg TNT charge, the maximum lateral displacements registered at gauge 2 in front façade intermediate column – beam connection are 1.4, 0.9 and 0.4 mm due to 1000 kg TNT at 3, 5 and 10 m standoff distances, respectively. This indicates also the effectiveness of shear wall in lateral drift resistance such that it reduced the drift peak value from 229 to 1.4 mm in case of the 3 m stand-off distance. Therefore, the shear walls can be considered as an effective solution for resisting and mitigating the collapse of RC building due to blast effects.

Figure (11) shows damage index contours of RC building strengthened by shear walls due to 500 and 1000 Kg charge weights at 3 and 10 m stand-off distances. Adding shear walls changes the damage behavior of building frame such that the critical damage positions are different. Less effect is observed on selected elements at the base and at the column-beam connection at the first and second levels which are enhanced and produced damage index too much less than unity. All fixed bases and column-beam connections are less affected such that no sections reached unity damage index in case of 500 kg TNT. On the other hand, new damage regions are observed in case of small stand-off distances of 3 meters. These regions are the walls added at the side panel which show relative damage due to the increase of the reflected wave effect. More increase of stand-off distance to 10 m too decreases the extent of damage for all fixed bases and all column-beam connections are too less damaged in case of 1000 kg TNT. It is also observed that for the retrofitted RC building, the damage at the first floor beam – column connection is greater than that on the fixed base which can be attributed to the high stiffness of shear walls at the ground floor. The second floor then behaves as if it is fixed at the first floor.

The time histories of damage index at fixed base and at the column-beam connection for the 500 kg explosive load at 3, 5 and 10 m stand-off distances are shown in figure (12). As shown in figure (12-a), at fixed base, the peak damage index equals to 0.0116 and 0.0065 reached at time 13.2 and 22 milliseconds for the 3, and 5 m stand-off distances, respectively. The case of 10 m stand-off distance shows no damage (zero damage index). These values illustrate that at the column base, minor or no damage is developed in case of shear wall strengthening. As shown in figure (12-b), for column–beam connection, the peak damage index equals 0.023, and 0.0103 at times 19.4 and 64.1 for the 3, and 5 m stand-off distances, followed by steady case of the peak damage. Although these values also indicate minor damage, they are more than corresponding damage in the fixed base as discussed for damage contours.

Shown in Figure 13, the time histories of damage at fixed base and at the column-beam connection for the 1000 kg explosive load at 3, 5 and 10 m stand-off distance. As shown in figure (13-a), at fixed base, the peak damage index equals to 0.023 and 0.0103 reached at time 14.1 and 14.9 milliseconds for the 3, and 5 m stand-off distances, respectively. The case of 10 m stand-off distance shows no failure (zero damage index). These values illustrate that at the column base, minor or no damage is developed in case of shear wall retrofitting. As shown in Figure (13-b), for column–beam connection, the peak damage index equals 0.0674, and 0.0326 at times 18.5 and 163, followed by steady case of the maximum damage. Therefore, the RC wall increases the building stiffness and absorbs the energy from the detonation leading to negligible damage.

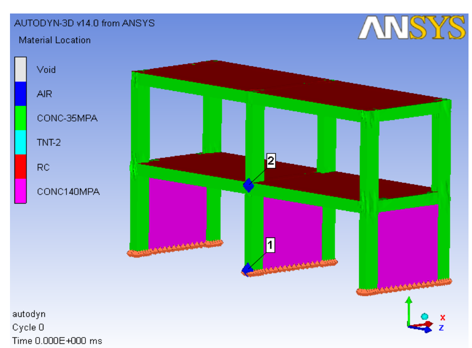
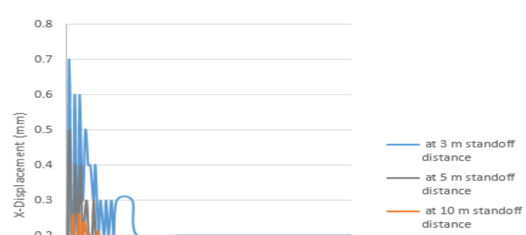
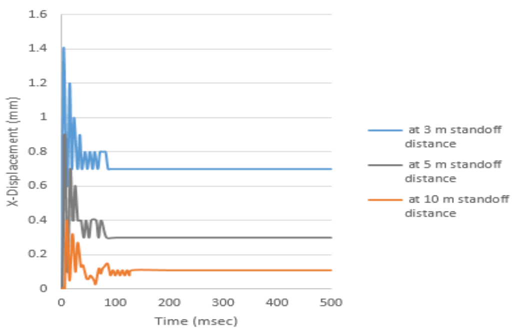


Figure 9: RC building with shear walls.



(a) 500 TNT charge



(b) 1000 TNT charge

Figure (10): Displacement time history for front façade intermediate column - beam joint (gauge#2) at 3, 5 and 10 m standoff distance cases.

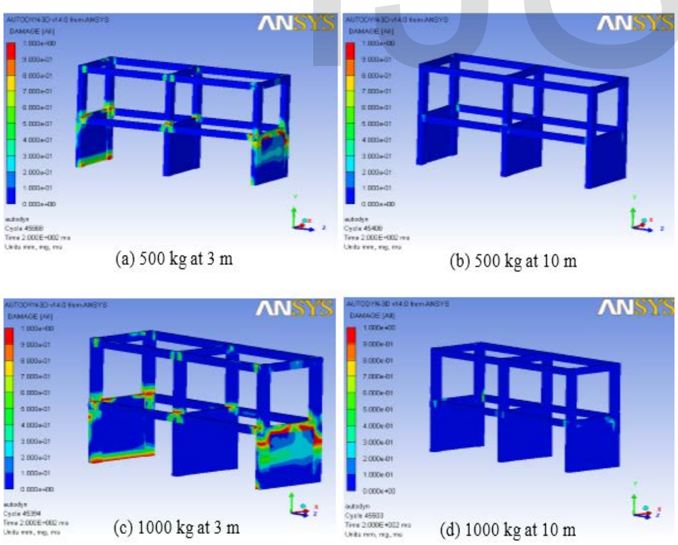
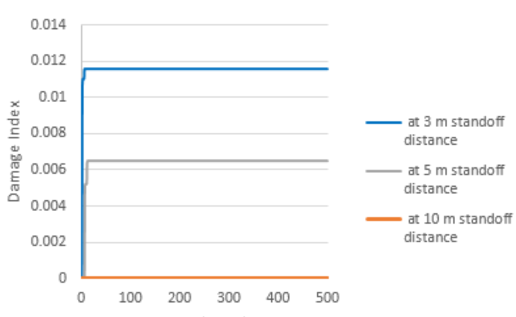
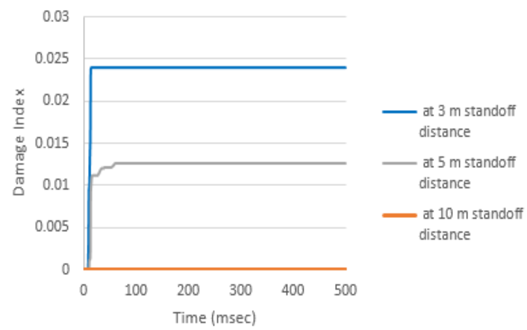


Figure (11): Damage index contour for retrofitted RC frame with shear walls

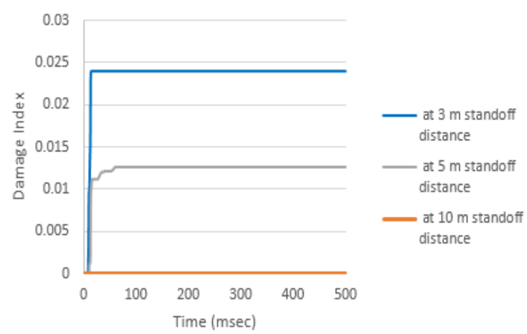


(a) At fixed base

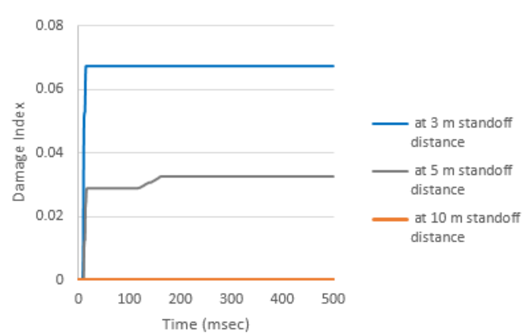


(b) At column – beam joint

Figure (12): Damage index time history at front façade intermediate column for RC frame building with shear walls due to 500 kg TNT at 3, 5 and 10 m standoff distance cases.



(a) At fixed base



(b) At column – beam joint

Figure (13): Damage index time history at front façade intermediate column for RC frame building with shear walls due to 1000 kg TNT at 3, 5 and 10 m standoff distance analyses.

4 Conclusion

The analysis of the structural failure of RC frame building caused by blast load is presented in this paper. All the process from the detonation of the blast load to the complete demolition, including the propagation of the blast wave and its interaction with the structure is reproduced. The paper presents the study of different blast loads effect in the RC frame building. This building is subjected to blast loads with 500 and 1000 kg equivalent TNT at height 1.5 m from the ground at the front facade of the middle column at different distances (3, 5 and 10 m) from the building. The effect of shear walls at ground floor as a mitigation technique is also studied by using AUTODYN software. Parameters studied are the blast load charge and the stand-off distance and results are summarized in the following main conclusions:

- The reflected over pressure on the front façade middle column - beam joint in the building increases with increasing the blast charge or decreasing the standoff distance of the blast load from the building. The arrival time of reflected over pressure increases with increasing standoff-distance. The rate of decrease of overpressure is observed to be sharpe for smaller standoff distance than that for far distances.
- For heavy explosive load (1000 TNT), full damage is observed at the fixed base of the building for all stand-off distances (3, 5, 10m), and is observed for charge (500 TNT) with small stand-off distances (3, 5 m). For small charge (500 TNT) and far stand-off distance (10 m), minor damage is observed.
- Using shear walls as a mitigation technique enhances too much the behavior of building to explosion. It reduces the displacement and stresses to desired design values.
- The existence of shear walls highly increases the building stiffness which shortening natural period that leads to oscillatory response.
- The maximum damage in case of adding shear wall at the ground floor is significantly decreased to very low levels. The maximum damage is moved to the first floor level in state of the building base.

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