Nonlinear Analytical Design Procedure of Concrete Framed Structures Subjected to Distance Blast Load

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Abstract— Designing a structure that could face a threat from an external bombing to the structure needs finding the most effective way to meet the current standards requirements. When considering protection for a building, owners and architects must work together with structural engineers and blast consultants to find out the blast forces to bear that risk in mind and assess the desired protection levels. This paper is presenting the dynamic response of reinforced concrete moment resisting framed building subjected to blast loading. The building consists of ten stories with 5x5 bays having bay span of 6m and exposed to 30kg and 60kg with three different standoff distances of 3m, 5m and 7m respectively. A non-linear three-dimensional finite element model is used for analyzing the dynamic response of the structure. CSI SAP2000 finite element program is used for modeling and investigating the dynamic response of a concrete framed structure subjected to blast loading. The applied blast load as outlined in TM5-1300 Manual. The results obtained are in terms of time history function, displacements and influence forces considering the resistance of structure. Progressive collapse is also checked by providing plastic hinges in the structure as per ASCE/SEI 41-13 provisions as well as UFC 4-023-03 guidelines. The alternate approach followed shows that the structure is safe by removing middle periphery column. The results obtained shows that the plastic hinges formed in the middle column are higher compared with the internal columns. Further improvements adopted can be take place by applying special design for reinforcements as a apart from general design and it is recommended that guidelines on abnormal load cases and provisions on progressive collapse prevention should be included in the current Egyptian Code of Practice for special and important buildings.

Key words: Blast Load; Concrete Framed Structure, Dynamic Response, Standoff Distance, Progressive Collapse

I. Introduction

An explosion is a large-scale and sudden release of energy that generally generates high temperature and a large amount of gas (Baker et al. 1983) [4]. In general, explosions can be classified into four basic types; a vapor cloud explosion, a vessel explosion, a dust explosion and a condensed phase explosion, ASCE 2013 [1]. The explosions can be formed by different kinds of explosive materials and vary from homemade to military or commercially available types. TNT (Trinitrotoluene) had been used as an explosive charge datum and is regarded as the standard “Explosion Bench Mark” or the reference in the explosion analysis. Quantity of energy released by the explosion (or charge weight) and distance to a target from the origin of the explosion. Therefore, the distance is important and is referred to as stand-off distance which is graphically illustrated in Fig. 1 (Ngo et al. 2007) [10].

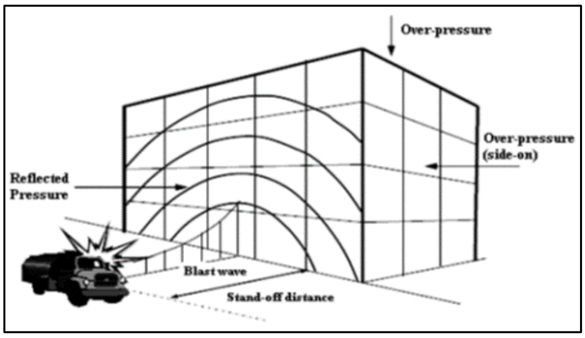

Fig. 1: Blast Loads on a Building (Ngo et al. 2007) [10]

Structures which are subjected to blast load extended their importance due to accidental effect. Blast load always produces vibration waves then can cause vibration damages to the structures; furthermore, loss of life can result due to debris or structure collapse. Army TM5-1300 Manual [12] is developed by empirical method used by military engineers to predict the blast loads and analyzing of the structural elements under explosion with given charge weights and standoff distances. The impulsive load produced by an explosion is highly nonlinear and cause pressure in an extremely short duration, analysis of the reinforced concrete frame structure is difficult way to increase the structures capacity to resist dynamic loads like earthquake, extreme wind, sea waves...etc. was to increase its sections’ size and overall indeterminacy.

This paper illustrates and investigates a reinforced concrete framed building subjected to blast loads using nonlinear dynamic analysis procedure. First, the building is designed for dead, live, and earthquake loads according to the specifications of the Egyptian Code of Practice (ECP) [8] requirements. Then the structure was subjected to blast loads resulting from detonation of 30kg and 60kg weight at different standoff distances to investigate the minimum standoff distance to resist blast induced progressive collapse. Finally, the results which are obtained compared with the UFC 4-023-03 [7] guidelines.

II. Blast loading on structures

In reality, a building is a finite size and has a roof, sides, and a rear face, i.e. it is a solid object. As the wave propagates through the air, the wave front surrounds the structure and all its surfaces so that the whole structure is exposed to the blast pressure. The magnitude and distribution of the structural loading depends on the following factors:

1) The characteristics of explosives that depend on the type of explosive material, released energy (size of detonation) and weight of explosive,
2) The detonation location relative to the structure, and
3) Intensity and magnification of pressure interrelated with the ground or the structure itself.

Time record of the explosion pressure wave is usually described as an exponential function in the form of Friendlander's equation (1946), in which the b is the parameter of the waveform:

$$P_s(t) = P_{so}(1 - \frac{t}{t_o})e^{-b\frac{t}{t_o}} \qquad (1.1)$$

where:

$P_s(t)$ The pressure at time t

P_{so} Peak static wave front overpressure, bar

t_o The positive phase duration, sec

t The time elapsed, measured from the instant of blast arrival and

b A decay coefficient of the waveform

The decay coefficient b can be calculated through a non-linear fitting of an experimental pressure time curve over its positive phase. For the various purposes approximations are satisfactory. This change in pressure over time is shown in Fig. 2 and Fig.3.

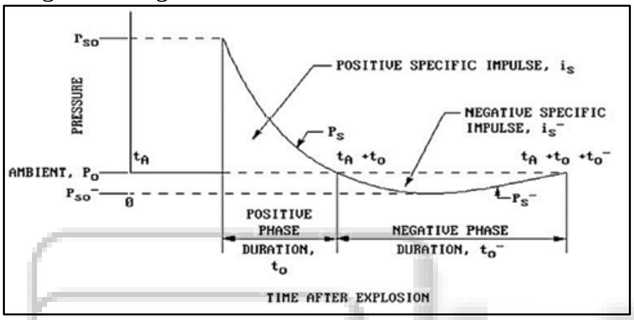

Fig. 2: Free-Field Pressure-Time Variation (UFC 3-340-02 2014) [7]

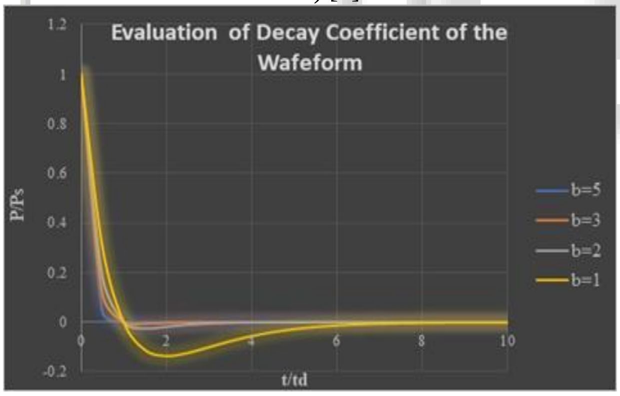

Fig. 3: Evaluation of Decay Coefficient of the Wafeform

A full discussion and extensive charts for predicting blast pressures and blast durations are given by TM5-1300 Manual [12]. For design purposes, reflected overpressure can be idealized by an equivalent triangular pulse of maximum peak pressure Pr and time duration td, which yields the reflected impulse ir, where; i_r= 0.5P_r t_d.

III. ACCEPTANCE CRITERIA

ASCE/SEI 59-11 [3] specified the level of progressive collapse design required for a structure correlated with the Level of Protection (LOP) that the Project Planning Team (Owners) develops and gives to the designer as given in Table 1. For all Levels of Protection, all multistory vertical load-carrying elements must be capable to supporting the vertical load after the loss of vertical support at any floor level.

Level of Protection	Performance Goals
Very Low	Collapse prevention; surviving occupants will likely be able to evacuate but the building is unlikely to be safe enough for them to return; contents may not remain intact.
Low	Life safety; surviving occupants will likely be able to evacuate and then return only temporarily; contents will likely remain intact for retrieval.
Medium	Property preservation; surviving occupants may have to evacuate temporarily but will likely be able to return after cleanup and repairs to resume operations; contents will likely remain at least partially functional but may be impaired for a time.
High	Continuous occupancy; all occupants will likely be able to stay and maintain operations without interruption; contents will likely remain fully functional.

Table 1: Level of Protection and Performance Goals (ASCE/SEI 59-11) [3]

Four main analysis procedures based on column removal included in General Services Admiration GSA (2016) [9]: Linear Static, Nonlinear Static, Nonlinear Dynamic and Nonlinear Dynamic. The procedures are based on the category of the buildings, with consideration of their degree of structural regularity.

The nonlinear dynamic procedure has been used to show the mitigation strategies for progressive collapse of a reinforced concrete framed building subjected to an air surface blast explosion. The global and local structural behavior of the building investigated to meet progressive collapse prevention. The nonlinear procedures and the acceptance criteria are based on less enormous rotation and ductility demands for the members and connections considered. ASCE/SEI 41-13 [2] gives the detailed rotation and ductility requirements based on the different structural materials (such as steel or concrete) and several types of structural members (beams or slabs).

IV. PROGRESSIVE COLLAPSE

Explosive loading incidents have become serious problems that necessity to discussed, moreover the immediate and localized blast effects, can lead to progressive collapse that could affect people and property in an entire building. Progressive collapse occurs when a structure has its loading pattern or boundary conditions changed such that structural elements loaded beyond their capacity and fail. The residual structure forced to seek alternative load paths to redistribute the load applied. As a result, other elements may fail, causing further load redistribution. This process will continue until the structure can find equilibrium either by shedding load as a by-product of the failures of other elements or by finding stable alternative load paths (T. Krauthammer 2008) [11].

Remarkable example of such a failure is the Ronan Point collapse. Since an explosion caused the progressive collapse at Ronan Point, many studies were dedicated to including the relationships of abnormal loadings and progressive collapse in building standards. Progressive collapse could be a result from a blast or impact at closed proximity. It is estimated that at least 15 to 20% of the total number of building failures are due to progressive collapse (T. Krauthammer 2008) [11],

The Ronan Point incident (1968) led to the UK Building Regulations (HM Government, 2013) [5], which aim to ensure a least (minimum) level of structural integrity; the British Standards employed three design approaches for resisting progressive collapse; Tie Forces, Alternate Path, and Specific Local Resistance.

In 2005, the Department of Defense in United States published the Unified Facilities Criteria. This Unified Facilities Criteria (UFC 2005) provides the design requirements necessary to reduce the potential of progressive collapse for new and existing Department of Defense (DoD) facilities that experience localized structural damage through normally unexpected events.

Recently DoD published UFC 4-023-03, 2014 [7] which provides the analysis and design criteria for the buildings to resist progressive collapse following the failure of key components. UFC 4-023-03 [7] recommends a structural analysis with a removed critical column in the building to identify the structural response for the normal loading conditions of dead, live and wind. However, this paper is to investigate the minimum standoff distance to resist blast induced progressive collapse using the alternate path method

V. ANALYTICAL MODEL

The assumed structure consists of 10-stories each story has 3m in height, 5-bays in X- direction and 5-bays in Y-direction and each bay has 6m in length, Fig. 4 and Fig.5. The structure has the following properties:

1) All connections are moment resistance.
2) Column to foundation connections are fixed.
3) Material properties: concrete strength (fcu) = 30 MPa, rebar yield strength (fy) = 360 MPa.

The static vertical loads, and earthquake load (only as a lateral load) applied as per Egyptian code of practice as follows:

1) Supper imposed dead load taken as 5.0 kPa without self-weight.
2) Live load taken as 3.0 kPa.
3) Building is located at Zone 3, with importance factor 1.0 and soil type is c.

The building has been designed and detailed to satisfying enough ductility requirements as per Egyptian code of practice. The designed member sizes and their reinforcement are shown below in Table 2.

The nonlinear dynamic analysis carried out by a systematic (step-by-step) integration of the equilibrium equations in the time domain. Hilber-Hughes-Taylor alpha's method of numerical integration is used with gamma = 0.5, beta = 0.25 and alpha = 0, with these parameters, the method is equivalent to the average acceleration method (also called the trapezoidal rule) and is unconditionally stable with no energy dissipation. Geometric nonlinearity is considered by considering of P-delta with large deformation setting. Mass and stiffness proportional damping (Rayleigh damping) was used to damp both high and low frequency modes outside of the range significant to dynamic response.

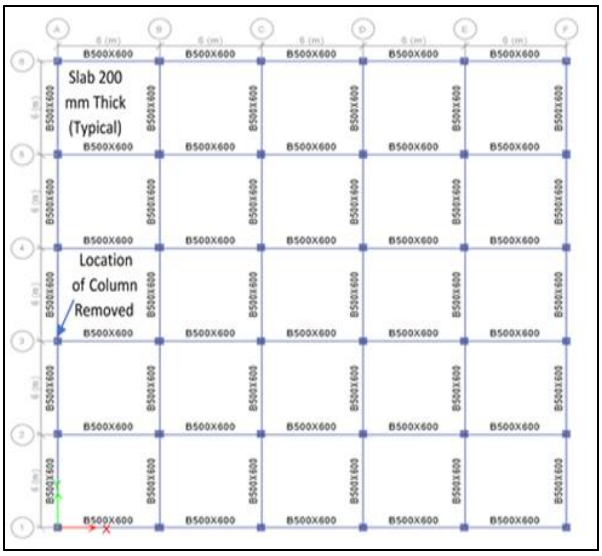

Fig. 4: Reinforced Concert Building Plan and Location of Column Removed

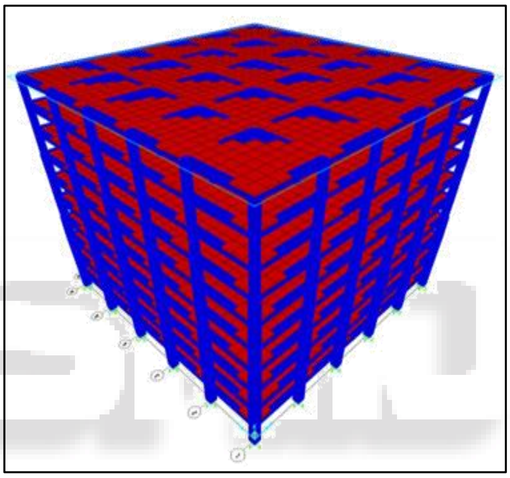
Fig. 5: 3D Perspective View of the Building

Member	Dimensions (mm)	Top RFT (mm^2)	Bottom RFT (mm^2)
Beam	500 x 600	1570	1570
Column (Ground - 4th Floor)	750 x 750	6280 (mm^2)	
Column (5th - 7th Floor)	650 x 650	5024 (mm^2)	
Column (8th - 10th Floor)	500 x 500	3768 (mm^2)	

Table 2: The Designed Member Sizes

The range of important natural frequencies is identified during the modal analysis and was used to identify the two frequencies needed to calculate Rayleigh damping coefficients. These coefficients were then used throughout the time history analysis. The maximum time step used was 0.001sec for all cases. The time step is automatically reduced if the relative unbalance is greater than one as per CSI SAP2000 [6] manual. Moreover, the following assumptions are taken while the dynamic analysis procedure:

1) ASCE/SEI 41-13 [2] hinge property is assigned for each design section. Moment M3 and moment M2 are considered to cause a plastic hinge in flexural members and the axial-moment interaction (P-M2-M3) is considered to cause a plastic hinge in a column.

2) All beam-to-column connections are moment-resistant and columns are stronger than the beams, so plastic hinges will form in the body of the beam and not in the column or in the joint (Strong column – weak beam principle).
3) All beams and columns are adequate for shear reinforcement; therefore, the beams are not shear controlled.

Load time history of explosive device for building and structural members have been calculated by dividing members in to sub sections and calculate a pressure time history for each small element. The blast pressure applied to the members are computed based on the radial distance from the point of explosion to the middle of each members. The blast loads are distributed uniformly along the elements length, each distributed load is a function of time.

An explosion (surface) yield of 30kg and 60kg of TNT corresponding to a compacted truck is considered. The explosion is assumed to occur at 3m, 5m and 7m standoff distance from the center of a building. Part of the blast loads parameters which applied on the structure for each case are shown in Table 3.

After performing a sequential nonlinear static, nonlinear direct integration time history and free vibration analysis for each case the final deformed shapes are shown in Figure 6, Figure 7, and Figure 8. From the deformed shapes, it is observed that the building can resist blast induced progressive collapse from detonation of 30kg and 60kg of TNT charge weight at 3m, 5m and 7m standoff distances.

Range Calculation (G+9)				Blast Charge (TNT)		60 kg				
Standoff distance.		10								
Floor Hight		3								
Element Number	Range (m)	Distance from upper line	Velocity (m/msec)	Time of Arrival (msec)	Pressure (kPa)	Impulse (kPa-msec)	Load Duration (msec)		Function Time History	
30-07-03-L01	18.3	0	0.41	28.62	117.9	535.65	9.09		0.0234	
30-07-03-L02	16.8	2	0.41	24.88	142.38	591.09	8.3		0.0235	
30-07-03-L03	16.2	3	0.42	23.42	154.79	616.53	7.97		0.0314	
30-07-03-L04	15.3	4	0.43	21.26	177.2	659.07	7.44		03-L03	
30-07-03-L05	13.8	6	0.45	17.77	229.04	744.36	6.5		0.0145	
30-07-03-L06	12.6	8	0.47	15.11	291.03	829.92	5.7		0.0146	
30-07-03-L07	12.3	9	0.47	14.46	310.75	854.4	5.5		0.02	
30-07-03-L08	11.7	10	0.49	13.19	356.94	907.76	5.09		03-L07	
30-07-03-L09	11.1	12	0.5	11.97	414.65	968.16	4.67		0.0114	
30-07-03-L10	10.5	14	0.51	11.37	448.92	1001.33	4.46		0.0115	
30-07-03-L11	10.5	15	0.51	11.37	448.92	1001.33	4.46		0.0158	
									03-L11	
Element Number	Range (m)	Distance from upper	Velocity (m/msec)	Time of Arrival	Pressure (kPa)	Impulse (kPa-msec)	Load Duration	Function Time History		
30-07-03-LC1	18.3	0	0.41	28.62	117.9	535.65	9.09	0.0286	0.0178	0.0114
30-07-03-LC2	13.8	6	0.45	17.77	229.04	744.36	6.5	0.0287	0.0179	0.0115
30-07-03-LC3	10.8	12	0.51	11.37	448.92	1001.33	4.46	0.0377	0.0243	0.0158
								03-LC1	03-LC2	03-LC3

Table 3: Part of Applied Blast Loads Parameters

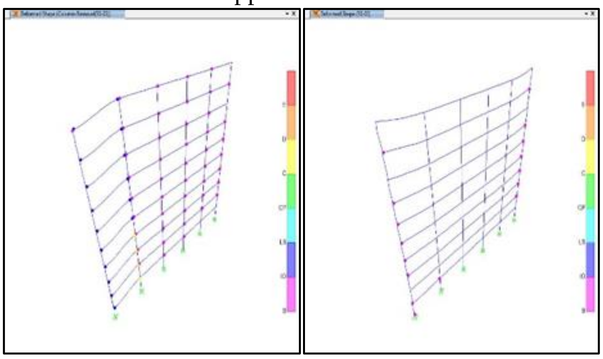

Fig. 6: Displacement Time History of Selected Column Removed on Axis 3

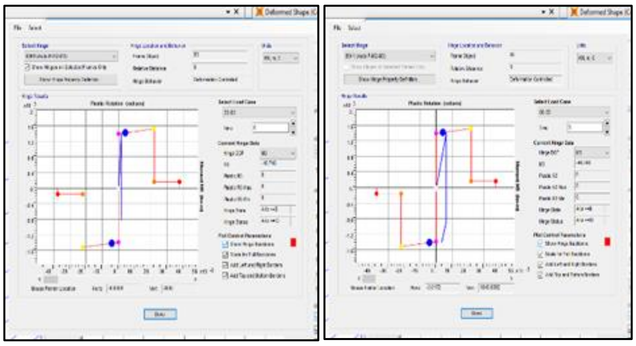

Fig. 7: Hinges Properties of Selected Column Removed on Axis 3

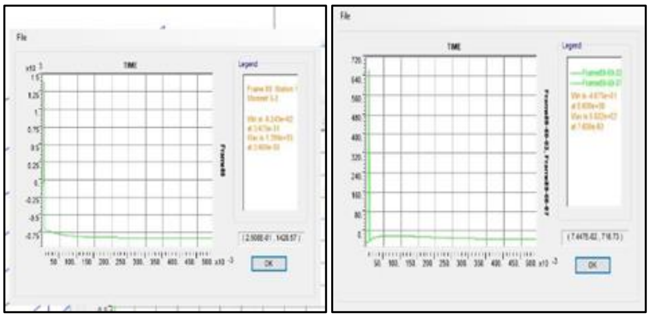

Fig. 8: Moment Time History of Selected Column Removed on Axis 3

VI. DISCUSSION AND ANALYSIS RESULTS

In this paper, the dynamic response and damage of reinforced concrete framed structure under external blast loading using recommended procedures and code provisions is investigated. Blast analysis was performed based on sequences of nonlinear dynamic analysis using CSI SAP2000 [6] different standoff distances. Based on the nonlinear dynamic analysis carried out the following observations and conclusions can be drawn:

1) Buildings designed complying with ECP guidelines with enough ductility requirements can satisfy the requirements of UFC 4-023-03 [7].
2) Buildings designed according to the requirements of UFC 4-023-03 [7] can resist a blast load of more than 30kg and 60kg at a standoff distance above or equale 15.0m. But the design is not adequate when the same explosion may happen at standoff distance less than 10.0m. Therefore, it is necessary to have limited use for UFC 4-023-03 [7], when the blast is the cause of the probable progressive collapse.

VII. CONCLUSIONS

It is not practical to design buildings to withstand explosive loads, it is possible to improve the performance of structures in resisting blast induced progressive collapse.

Defining the critical standoff distance and hardening key elements (columns), Designers can enhance the life safety of the persons within the building and ease rescue efforts during such an event.

It is recommended that guidelines on abnormal load cases and provisions on progressive collapse prevention should be included in the current Egyptian Code of Practice.

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