QUALITY OF COMPUTERIZED BLAST LOAD SIMULATION FOR NON-LINEAR DYNAMIC RESPONSE ANALYSIS OF FRAMED STRUCTURES

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ABSTRACT

A numerical study has been conducted to determine the quality of blast load simulation for non-linear dynamic response analysis of framed structures subjected to such loads at various stand-off distances with due consideration to the provisions and requirements of Unified Facility Criteria UFC 2005.

Simulation has been carried out using a general-purpose, commercial software system and a special-purpose, blast-specific software product to assess and compare the quality of response prediction of such computational models.

Nonlinear dynamic analysis has been performed using a three-dimensional model of a structure and its corresponding elasto-plastic analysis on its single-degree-of-freedom representation. The results obtained for different positions of explosive charges under the two analysis models have been presented.

A comparative analysis of the results indicates that the quality of blast load simulation and associated structural response depend both on the analysis model of choice and the stand-off distances. It was concluded that the quality of response prediction by commonly available general purpose software systems is of inferior quality and that special-purpose software systems need to be implemented when dealing with generalized impulse loads as the standoff distance for the detonation is getting closer to the structure.

Key Words: blast loads, Unified Facility Criteria, elasto-plastic analysis, nonlinear analysis, single-degree-of freedom, stand-off distance

INTRODUCTION

Protecting buildings and other structures from damage as a result of blast actions is becoming one of the most critical challenges for structural engineers in recent years. Important and high-value targets, such as the UN building in Abuja (Nigeria), the UN building in Kabul (Afghanistan), the US embassies in Nairobi (Kenya) and Dar es Salam (Tanzania) and subjected to explosive attacks are all indicators of potential vulnerability of the structure if proper mitigation action is not taken by way of designing and detailing reinforced concrete and other structures. Events of the past few years have greatly heightened the awareness of threats from explosive damages [1, 2]. Extensive research into blast effects analysis and techniques to protect buildings has been initiated in many countries to develop methods of protecting buildings and infrastructures.

Although it is recognized that no civilian buildings can be designed to withstand all conceivable types of damage resulting from blast actions, it is, nevertheless, possible to improve the performance of structural systems by better understanding the factors that contribute to the resistance capacity, the blast loading simulation to be used and identification of the appropriate analysis tools in modeling the structure.

With respect to blast loading simulation, the quality of computerized response prediction depends on the analysis procedure implemented in such systems. On the other hand, the effects of blast loads on a structure are influenced by a number of factors including charge weight, relative location of the blast to the structure of interest (or *stand-off distance*), configuration and spatial orientation of the structure in relation to the blast point (referred to as *direction of the blast*), and ductility of the structural system. Structural response to such loads varies according to the way these factors combine with each other. The potential threat of an explosion is random in nature; therefore, the analysis becomes complex and it is necessary to identify the influence of each factor in relation to the most credible event when assessing the vulnerability of structures.

This paper addresses the blast-load simulation capabilities and subsequent response prediction qualities of a commonly available, general-purpose software system SAP2000 [3] on one hand and a special-purpose, blast-specific software A.T.-Blast [4] on the other in relation to stand-off distances.

BLAST PHENOMENA

In describing blast phenomena, loading on structural systems and the corresponding response variables as well as analysis methods that have been developed to study those responses will be presented.

Blast loading is the result of an explosion where this refers to a rapid and sudden release of stored energy.

Some portion of the energy is released as a thermal radiation while the major component of the response is coupled into the air as air blast and into the soil as ground shock, both as radially expanding shock waves [5].

This violent release of energy from a detonation in a gaseous medium gives rise to sudden pressure increase in that medium. The consequential pressure disturbance, termed the *blast wave*, is characterized by an almost instantaneous pressure surge from the ambient pressure (P_o) to a peak incident pressure (P_{io}) as shown in Fig. 1 [6].

Blast loading on structures can be from unconfined or partially confined explosion charges. Surface burst load is one of unconfined explosion type where blast pressure is located close to or on the ground so that the shock wave becomes amplified at the point of detonation due to ground reflection; this type of blast load is the one considered in this paper. Other types have been discussed elsewhere [7, 8].

Following an unconfined blast, the ensuing shock wave travels radially from the burst point and it is associated with a dynamic pressure (q_o); the latter is a pressure formed by the winds produced by the shock fronts and it is a function of air density and wind velocity.

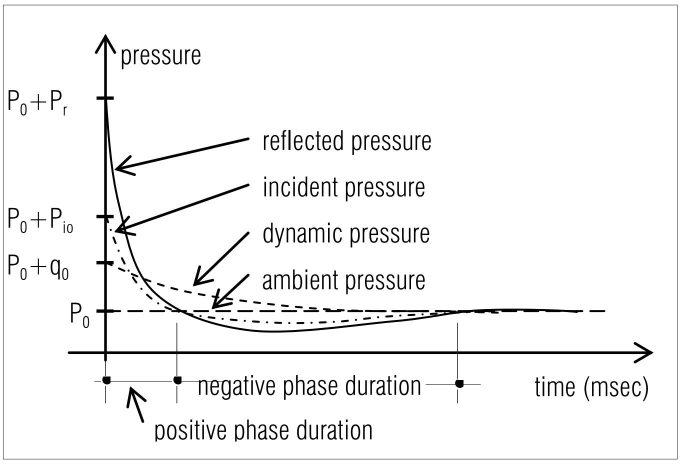


Fig. 1 Complete Over-Pressure – Time Profile [6]

If the shock wave impinges on a rigid surface such as, for example, a building, oriented at an angle to the direction propagation of the wave, a reflected pressure is instantly developed on the surface. This pressure is a function of the pressure in the incident wave and the angle formed between the rigid surface and the plane of the shock front [9, 10]. This aspect is important in this paper in view of the effect of the incident blast wave on the rigid boundary surface and acting perpendicular to the latter.

PREDICTION AND EVALUATION OF BLAST PRESSURE

It is important to establish a representative load model for a blast load action on a structure. To this goal, a dynamic blast load exhibiting a sudden rise and, then, linearly decaying to zero – a triangular load – is assumed. The negative phase as shown in Fig. 1 is neglected because it usually has little effect on the maximum response [9]. A full discussion and extensive charts for predicting blast pressures and blast durations are given by TM 5-1300 manual [11]. Furthermore, detailed account of prediction of such loads is also provided in [8], [12].

Two software products – a commonly-available, widely used commercial software product SAP2000 and a publicly available, special-purpose software product named A.T.-Blast (Anti-Terrorism Blast) – have been used to assess their prediction qualities of structural response under blast loading scenarios with various stand-off distances. A.T.-Blast has been developed for blast load prediction according to TM 5-1300.

In this paper, both software products have been used for the purpose of estimating the blast pressure and impulse from a high explosive detonation as a function of standoff distance and, subsequently, evaluate their blast-load modeling and response prediction qualities as a function of stand-off distances.

The other important feature of blast - structure interaction is the phenomenon related to the mechanical properties of materials from which the structure in made. Blast loads typically produce very high strain rates in the range of 10^2 to 10^4 s^{-1} [6]. This high straining rate generally alters the dynamic mechanical properties of target structures and, accordingly, the expected damage mechanisms for various structural elements.

In framed structures, generally constructed from reinforced concrete and steel structures, and subjected to blast loads, the strength of concrete and steel reinforcing bars can momentarily increase significantly due to strain rate effects [9]; this is also important in understanding the structural response of such systems to blast loads.

A peculiar feature of the blast – structure interaction is the modalities of failure if the latter comes and it may take in the form of progressive collapse of the structural components which may eventually result in the total destruction of the entire structure. It is important, therefore, to clearly understand the mechanics of progressive collapse to effectively design structures under blast loads.

Progressive collapse 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. It is estimated that at least 15 to 20% of the total number of building failures are due to progressive collapse [9].

Several approaches have been proposed for including progressive collapse resistance in building design. In 2005, the Department of Defense in United States published the Unified Facilities Criteria (UFC 2005) [13]. This provides recommendations the design requirements necessary to reduce the potential of progressive collapse for new and existing facilities that experience localized structural damage through normally unforeseeable events.

There are three allowable analysis procedures for assessing progressive collapse [14]; these are linear static, nonlinear static, and nonlinear dynamic. Several analysis methods are used for the prediction of structural response to blast loads [14]. These include simple hand calculations and graphical solutions to more complex computer dynamic based applications.

A commonly employed analysis method for assessing structural response to a blast loading is the single-degree-of-freedom SDOF method. This method has been effectively used to alleviate the complexities involved in analyzing the dynamic response of blast-loaded structures taking into account the effect of high strain rates, the non-linear inelastic material behavior, the uncertainties of blast load calculations and the time-dependent deformations. In this approach, a structure is idealized as a single degree of freedom (SDOF) system and the link between the positive duration of the blast load and the natural period of vibration of the structure is established. This leads to blast load idealization and it simplifies the classification of the blast loading regimes. Both elastic and elasto-plastic SDOF analysis models have been implemented in blast effect analysis; details have been given elsewhere [14, 15]. Elasto-plastic analysis will be implemented in the study covered by this paper; accordingly a brief description of the method is outlined subsequently.

Structural elements are expected to undergo large inelastic deformation under blast loads or high velocity impacts. Exact analysis of dynamic response is then only possible by step-by-step numerical solutions requiring nonlinear dynamic finite element procedures. However, the degree of uncertainty in both the determination of the loading and the interpretation of acceptability of the resulting deformation is such that the solution from a postulated equivalent ideal elasto-plastic SDOF system as shown in Fig. 2 is commonly used [6]. Interpretation of results is based on the required ductility factor $\mu = y_m/y_e$.

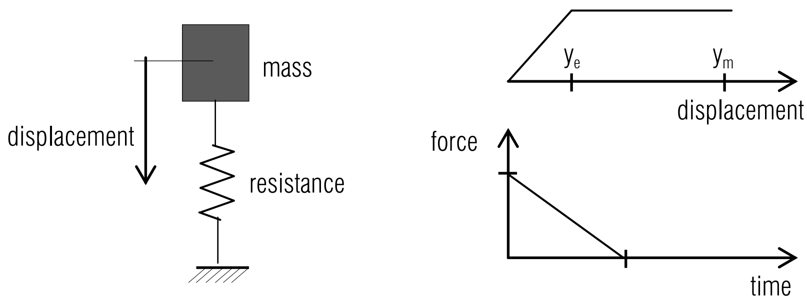


Fig. 2 Simplified elasto-plastic SDOF model for blast load analysis [4]

While a number of methods have been developed to carry out the computational details of elasto-plastic SDOF analysis [16], the Newmark numerical integration method, also known as the time-history method, will be implemented in this paper.

For a dynamic equilibrium equation N. M. Newmark developed a family of time- stepping solution based on the following equations [16]:

$$[M]\ddot{y}+[C]\dot{y}+[K]y = Ft$$
$$\dot{y}_{i+1} = \dot{y}_i + [(1-\gamma)\Delta t]\ddot{y}_i + (\gamma\Delta t)\ddot{y}_{i+1}$$
$$y_{i+1} = y_i + (\Delta t)\dot{y}_i + [(0.5-\beta)(\Delta t)^2]\ddot{y}_i + [\beta(\Delta t)^2]\ddot{y}_{i+1} \quad (1)$$

It is most commonly used with either constant-average or linear acceleration approximations within the time step. An incremental solution is obtained by solving the dynamic equilibrium equation for the displacement at each time step. Results of pervious time steps and the current time step are used with recurrence formulas to predict the acceleration and velocity at the current time step. To insure an accurate and numerically stable solution, a small time increment must be selected.

Dynamic equilibrium equation is solved by applying numerical time integration method according to Newmark [16]. Newmark's computational procedures can be easily programmed for a general resistance-deflection function using VBA programming language and this coding, based on the step-by-step Newmark's linear acceleration method, has been used to carry out the elasto-plastic blast analysis presented in this paper.

A better and more robust analysis method for structures subjected to blast loading is the finite elements method. The method is recommended since overall structural behavior is to be evaluated with regard to structural stability, gross displacements and P-Delta effects, among others. The method is specifically suited when one or more of the following conditions exist [11]:

a. The ratio of a member's natural frequency to the natural frequency of the support system is in range of 0.5 to 2.0, such that an uncoupled analysis approach may yield significant inaccurate result.

b. Overall structural behavior is to be evaluated with regard to structural stability, gross displacements and P-Delta effects.

c. The structure has unusual features such as unsymmetrical or non-uniform mass or stiffness distribution characteristics.

Many commercial finite element based programs are available for nonlinear dynamic analysis although the qualities of their computational results can be greatly influenced by standoff distances as will be shown later in this paper. Computational methods used by those packages for blast analysis can be categorized as coupled or uncoupled analysis [16]. Coupled analysis tends to be less accurate due to software limitations. In this paper, the uncoupled analysis feature of SAP2000 will be implemented to perform nonlinear dynamic analysis for better approximation.

The qualities of blast load simulation using SAP2000 and A.T.-Blast will be presented subsequently through a cases study.

CASE STUDY

Assumptions

An investigative study was carried out on a four story reinforced concrete frame building. After initially proportioning the structural elements to meet design code requirements and those of UFC 2005 provisions, an explosion yield of 113.5 kg (250 lb) TNT corresponding to a compacted truck has been considered [6]. This explosion has been assumed to occur at different standoff distances from the center of a building.

The assumed structure consists of 4-stories, each story 3 m in height, and 4-bays in X-direction and 2-bays in Y-direction, each bay being 6.0 m in length (Fig. 3). Dead load of 3.23 kPa without self-weight, live load of 2.00 kPa, and lateral load, have been considered.

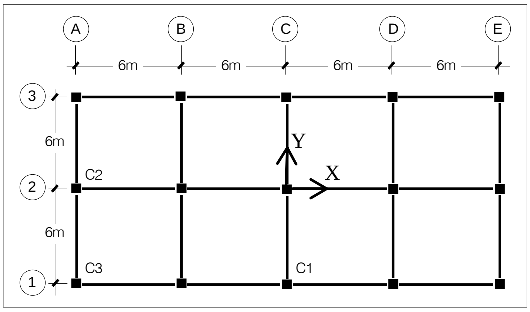


Fig. 3: RC Building Plan and Location of Removed Columns.

Once the loads were determined, linear elastic structural analysis has been performed and members designed using the most severe design requirements for any member in a group.

Following and initial investigation based on assumed preliminary proportions of the structural elements, the member sizes shown in Table 1 and their reinforcement have been adapted to investigate the blast-load simulation quality for blast load analysis.

Table 1: Properties of Structural Model

Member Group	Dimensions	Top Reinforcement	Bottom Reinforcement
Beams	0.3m × 0.5m	1500 mm^2	1500 mm^2
Columns	0.4m × 0.4m	2800 mm^2	

Analysis of the Building Model for UFC 2005 Requirements [13]

A full three-dimensional building model subjected to simulated blast loads and the corresponding elasto-plastic SDOF models have been established and processed to study both the quality of blast simulation and the influence of stand-off distances of the simulated action.

According to the UFC 2005 requirements for building safety against progressive collapse [13], first, the column(s) as shown in Fig. 3 have been removed sequentially from the structure to simulate element collapse; then a 25% increasing factor for the material strength has been applied.

Finally, per the requirements of the UFC, a (1.2×DL + 0.5×LL) load combination has been defined for analysis. For the nonlinear alternate path method, plastic hinges are allowed to form along the members. These hinges are based on maximum moment values calculated using the section design property employed to model the reinforced concrete structural elements.

Only moment M3 [3] is 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. Extensive discussion on the modeling assumptions have been provided in [14] and only a brief summary will be given subsequently.

In preparation for analyzing the structure using SAP2000, assumptions have been made to simulate the anticipated scenario of failure of certain structural elements as a result of the action of the blast load. Accordingly, to simulate the column removal the "non-linear staged construction" feature in the software has been implemented.

The model has been analyzed in two stages using a maximum of one-hundred steps per stage. In the first stage, the total load has been was applied to all elements; in the second stage, the column has been removed and the analysis has been carried out until the computational process converged.

After the building has come to a stable position following the blast, the maximum plastic hinge rotations have been observed. If the maximum plastic rotations were found to exceed the established limit, the members must be redesigned and the analysis repeated until the plastic rotations from the analysis turn out to be within established acceptable limits.

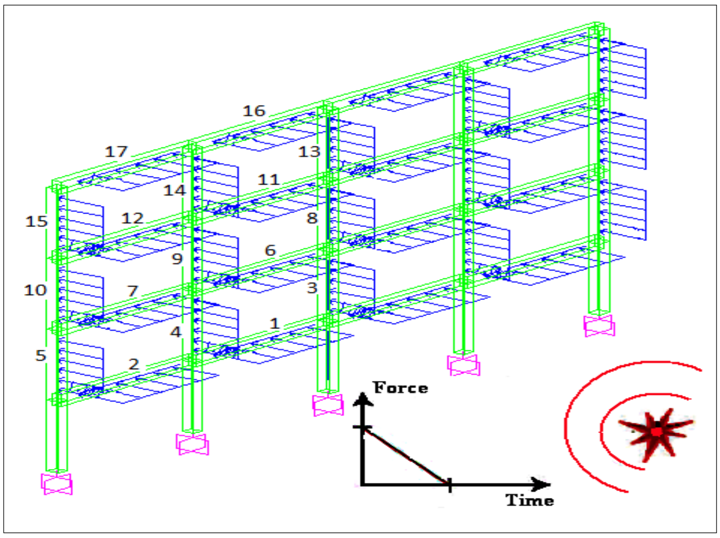


Fig. 4: Distribution of Blast Load on Structural Elements.

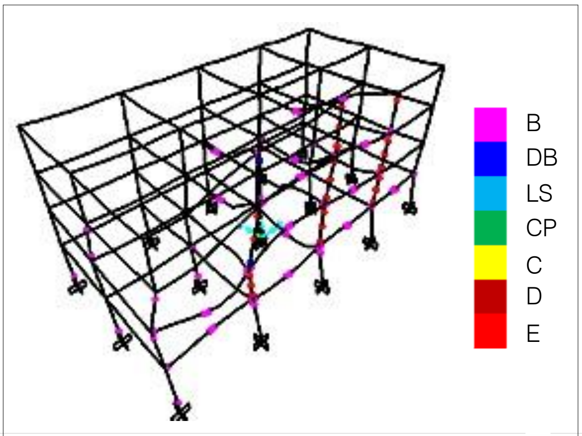
With the above-noted assumptions and computational procedures, Newmark's average acceleration numerical integration method was used with $\beta = 0.25$ and $\gamma = 0.5$ [16]. Geometric nonlinearity has been considered by incorporation the P-Delta option into the model. The range of important natural frequencies has been identified during the modal analysis and this was used to identify the two frequencies needed for SAP2000 to calculate Rayleigh damping coefficients. The maximum time step used was 0.001sec for all cases. Furthermore FEMA-356 [2] hinge property was assigned for each design section. Moments M2 and M3 were considered to cause a plastic hinge in flexural members and the axial-moment interaction (P-M2-M3) considered to cause a plastic hinge in a column. The analysis has been carried out with SAP2000 using UFC load combination and with the assumption that all beam and column have been adequately confined by shear reinforcement so that the strength of the beams may not be controlled by shear failure.

As part of the modeling process, blast loads must be simulated and imposed on the various structural elements. To this goal, load time history of blast loading for structural members has been calculated by dividing members into sub-sections and establishing a pressure time history for each small element [15]. The blast pressures applied to the members have been computed based on the radial stand-off distance from the point of explosion to the middle of each member. The blast loads are distributed uniformly along the elements length as shown in Fig. 4. The blast load parameters, i.e., pressure, time of arrival, impulse and load duration are calculated using A.T.-Blast software and they are given in Table 2 for the four stand-off distances chosen for this study.

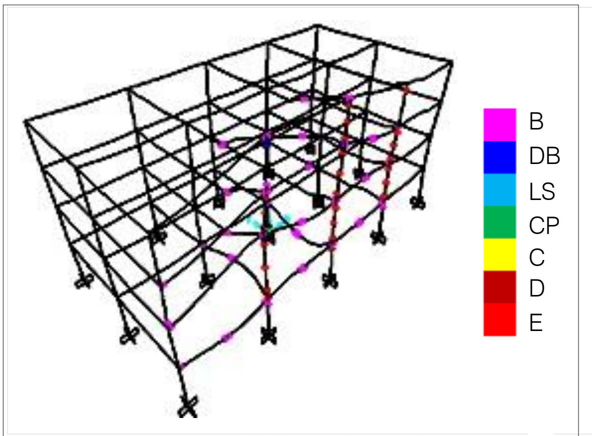
Blast Analysis Results Using SAP2000 Model

As noted earlier, an explosion yield of 113.5 kg (250 lb), assumed to occur at 10m, 7m, 5m and 3m standoff distances, has been considered for this study. The blast loads parameters applied on the structure for each case have been established as shown in Table 2.

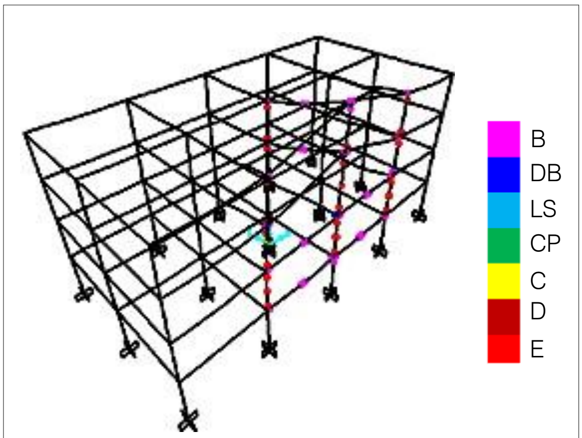
After performing a sequence of nonlinear static, nonlinear direct integration time history and free vibration analysis for each case, the final deformed shapes are as shown in Fig. 5 for the four stand-off distances.



Blast Distance: 3 m



Blast Distance: 5 m



Blast Distance: 7 m

Fig. 5: Hinges and Deformed Shape for Various Standoff Distances

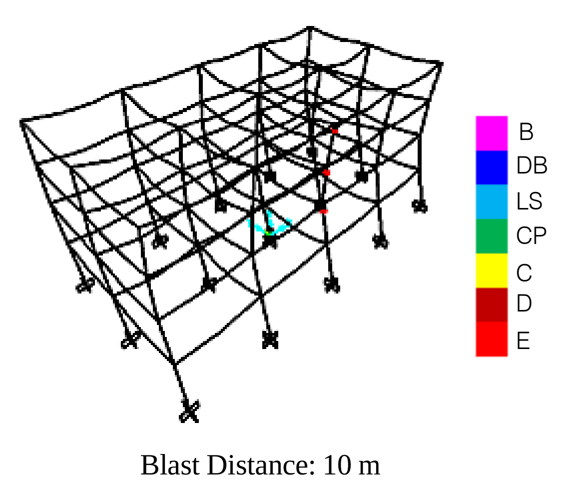


Fig. 5: (Cont…)

From the deformed shapes, it is observed that the building is susceptible to progressive collapse from detonation of 113.5 kg (250 lb) charge.

However, at 10m standoff distance, the building is safe to resist blast induced progressive collapse. weight at 3m, 5m and 7m stand-off distances.

Nonlinear SDOF Analysis of Building Components

In this section, the SDOF design approach is implemented on a typical structural element which was analyzed earlier using SAP2000. This method is being implemented extensively and refinements are made to further improve its capabilities [17]. A central exterior RC column from the previous building model, C1 in Fig. 3, has been subjected to blast loading and analyzed using a computer program developed for nonlinear SDOF systems [14]. In the program, dynamic equilibrium equation is solved by applying Newmark's numerical time integration method as noted earlier in Sec. 3 of this paper.

To simplify the analysis for a typical column element shown in Fig. 6, the column has been modeled as fixed at both ends. A *Smooth Resistance-Deflection* function is adopted from member analysis using Response-2000 [18] and equivalent structural damping of 5% has been adapted during the analysis

Table 2: Blast load parameters on structural elements for 10m, 7m, 5m and 3m standoff distances.

Element No.	Range [m]				Shock Velocity [m/msec]				Time of Arrival [msec]			
	10m	7m	5m	3m	10m	7m	5m	3m	10m	7m	5m	3m
1	10.44	7.62	5.83	4.24	0.59	0.77	1.00	1.36	9.45	5.23	3.18	1.79
2	13.45	11.40	10.30	9.49	0.50	0.55	0.60	0.64	14.98	11.11	9.21	7.91
3	10.11	7.16	5.22	3.35	0.60	0.82	1.11	1.68	8.90	4.67	2.60	1.19
4	11.76	9.34	7.95	6.87	0.54	0.64	0.74	0.85	11.75	7.68	5.68	4.33
5	15.69	13.97	13.09	12.46	0.46	0.49	0.51	0.52	19.64	16.03	14.26	13.05
6	10.86	8.19	6.56	5.20	0.57	0.72	0.89	1.12	10.16	6.00	3.96	2.57
7	13.78	11.79	10.72	9.95	0.49	0.54	0.58	0.61	15.65	11.81	9.93	8.64
8	10.97	8.32	6.73	5.41	0.57	0.71	0.87	1.07	10.34	6.19	4.16	2.77
9	12.50	10.26	9.01	8.08	0.52	0.60	0.66	0.73	13.13	9.15	7.19	5.85
10	16.26	14.60	13.76	13.16	0.45	0.48	0.49	0.50	20.87	17.32	15.59	14.41
11	12.04	9.70	8.37	7.35	0.53	0.62	0.71	0.80	12.27	8.23	6.25	4.90
12	14.73	12.88	11.92	11.22	0.48	0.51	0.54	0.56	17.59	13.87	12.04	10.80
13	12.50	10.26	9.01	8.08	0.52	0.60	0.66	0.73	13.13	9.15	7.19	5.85
14	13.87	11.88	10.83	10.06	0.49	0.54	0.58	0.61	15.81	11.99	10.10	8.82
15	17.33	15.79	15.01	14.47	0.44	0.46	0.47	0.48	23.27	19.85	18.18	17.04
16	13.78	11.79	10.72	9.95	0.49	0.54	0.58	0.61	15.65	11.81	9.93	8.64

Table 2: Blast load parameters on structural elements for 10m, 7m, 5m and 3m standoff distances (cont'd)

Element No.	Pressure [MPa]				Impulse [MPa- msec]				Load Duration [msec]			
	10m	7m	5m	3m	10m	7m	5m	3m	10m	7m	5m	3m
1	0.84	2.19	4.85	11.53	233.04	344.09	485.26	743.59	3.81	2.17	1.38	0.89
2	0.41	0.65	0.88	1.12	172.18	209.57	237.01	261.87	5.84	4.45	3.72	3.21
3	0.93	2.65	6.64	20.17	242.25	372.40	561.69	1032.42	3.60	1.95	1.17	0.71
4	0.60	1.18	1.92	2.99	202.00	266.95	325.88	392.25	4.69	3.13	2.34	1.82
5	0.27	0.37	0.44	0.50	143.92	164.70	177.88	188.51	7.35	6.20	5.59	5.16
6	0.75	1.76	3.44	6.72	222.14	314.37	416.76	564.95	4.09	2.47	1.68	1.16
7	0.38	0.59	0.78	0.97	167.35	201.35	225.59	247.09	6.07	4.71	4.00	3.50
8	0.73	1.68	3.19	6.02	219.57	308.00	403.05	536.12	4.16	2.54	1.75	1.23
9	0.50	0.89	1.31	1.84	187.77	238.05	278.93	319.70	5.18	3.70	2.93	2.41
10	0.25	0.33	0.38	0.43	138.18	156.43	167.74	176.68	7.72	6.62	6.05	5.64
11	0.56	1.05	1.65	2.44	196.34	255.01	305.96	360.08	4.88	3.34	2.57	2.04
12	0.32	0.46	0.57	0.68	154.86	181.20	198.77	213.52	6.71	5.45	4.79	4.33
13	0.50	0.89	1.31	1.84	187.77	238.05	278.93	319.70	5.18	3.70	2.93	2.41
14	0.37	0.58	0.76	0.94	166.21	199.39	223.00	243.66	6.12	4.77	4.07	3.57
15	0.21	0.27	0.30	0.33	128.40	142.91	151.53	158.18	8.40	7.41	6.89	6.53
16	0.38	0.59	0.78	0.97	167.35	201.35	225.59	247.09	6.07	4.71	4.00	3.50

4.5 Blast Analysis Results using Nonlinear SDOF Model

The blast load parameters for the selected typical structural element C1 as shown in Fig. 6 and established using A.T.-Blast are summarized in Table 3; these values have been extracted from Table 2. After performing SDOF blast analysis, the dynamic responses (deformation, velocity and acceleration) have been determined for each case. For brevity, only deformation time history results are shown in graphical form in Fig. 7.

4.6 Comparison of SAP20000 Analysis and A.T.-Blast SDOF Analysis Results

Figure 8 shows the maximum deflection resulting using three-dimensional SAP2000 and SDOF A.T.-Blast analysis approach for different stand-off distances. Through comparison of the analysis outcomes, one is able to assess the quality of blast load modeling for analysis and the influence of blast stand-off distances on the quality of these models.

From Fig. 8, it can be observed that as standoff distance is reduced, the SDOF A.T.-Blast analysis has given better results compared to SAP2000. The difference ratio for the 5m standoff distance is about 30 %, while for 10m the difference is close to zero. It can, thus, be observed that structural response to blast loads on the close proximity of the structure may not be captured well by general purpose analysis software systems such as SAP2000 although they are capable of handling dynamic loads.

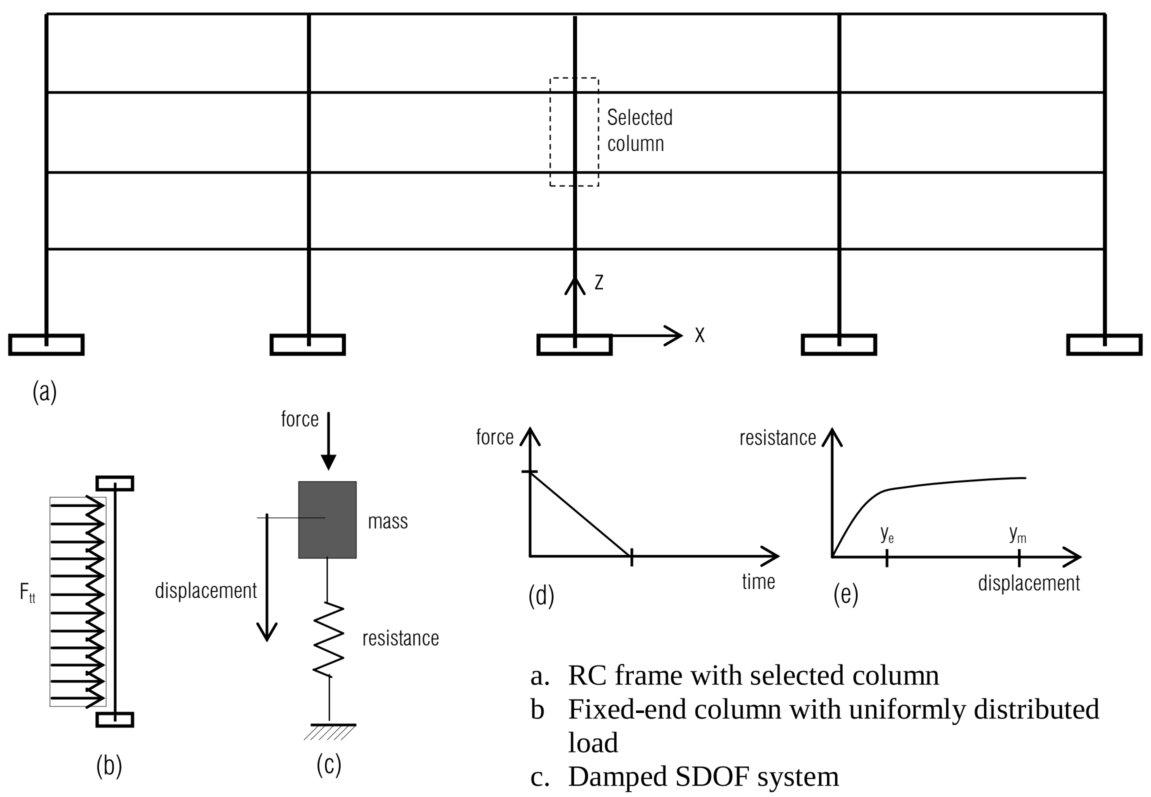
CONCLUSION

The study has shown that, in the process of modeling blast-susceptible structures for analysis and subsequent design, different analytical approaches produce similar and divergent results depending on the stand-off distance. From the report of this study, it is important to note that the quality of blast load simulation for non-linear dynamic response analysis of framed structures is dependent on the stand-off distance and the procedure used to determine the responses.

In a computerized environment for the analysis of such structural responses under blast loads, special-purpose software systems such as A.T.-Blast produce better quality results compared to general purposes analysis software although the latter is also capable of handling a variety of dynamic loads. With this latter group of products, the quality of analysis results under blast loading deteriorates as the standoff distance between the detonation and the structure under consideration diminishes. Accordingly, the analysis of structures under impulse loads, of which blast loads constitute a group, should be carefully modeled when using computerized approach to evaluate structural responses.

Acknowledgement

The first author gratefully acknowledges the facilitation of Visiting Professorship by the Mechanical Engineering Department, San Diego State University, California, USA during which the final draft of this paper was prepared. The second author expresses his humble gratitude to Ethiopian Institute of Technology, Mekelle University, Ethiopia for kindly sponsoring his research.



a. RC frame with selected column
b Fixed-end column with uniformly distributed load
c. Damped SDOF system
d. Applied force

Fig. 6 Equivalent SDOF Model for Dynamic Analysis.

Table 3: Blast Load Parameters on Design Column for Various Standoff Distances

Standoff Distance [m]	Shock velocity [m/msec]	Time of Arrival [msec]	Pressure [MPa]	Impulse [MPa-msec]	Load Duration [msec]
10m	0.57	10.34	0.73	219.57	4.16
7m	0.72	6.19	1.68	308.00	2.54
5m	0.88	4.16	3.19	403.05	1.75
3m	1.12	2.77	6.02	536.12	1.23

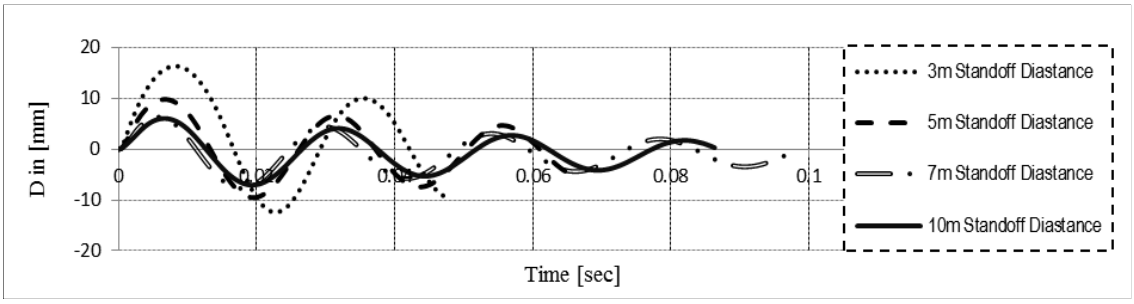


Fig. 7: SDOF Blast Analysis Deformation Response at Different Standoff Distances

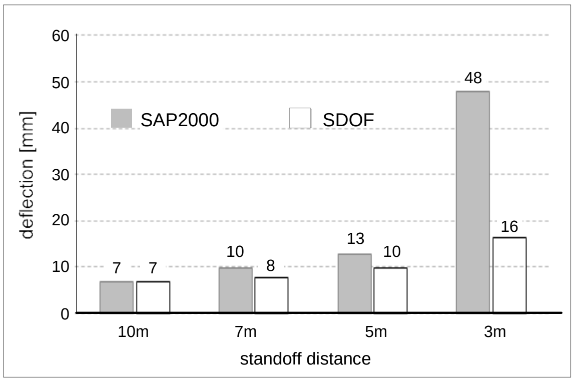


Fig. 8: SAP2000 and A.T.-Blast Comparison of Maximum Displacements

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