ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

BLAST LOADING AND BLAST EFFECTS ON RC FRAME BUILDINGS

A thesis submitted to the school of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structures)

by

Abdulaziz Kassahun

Adviser Dr. Shifferaw Taye

(March 2011)

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Special words of thanks due to all my instructors during the program each one of them has a special place in my heart. Finally I would like to express my deepest gratitude to my parents; for their patience and care throughout the duration of study, and to my friends who contributed to this thesis in one way or another.

Abstract

This thesis deals with the nature of blast loading and its effects on reinforced concrete frame structures, designed to withstand normal gravity and earthquake loads. Three dimensional and single degree of freedom (SDOF) nonlinear analysis programs have been used to determine the dynamic behavior of reinforced concrete elements under external blast loading.

Section design property has been employed to model reinforced concrete structural elements in SAP2000. Moments M3 and M2 are considered to cause plastic hinges in flexural members and the axial-moment interaction (P-M2-M3) is considered to cause plastic hinge in columns.

Equations of dynamic equilibrium have been solved using direct integration time history analysis. The solution algorithms are applied by utilizing Newmark's Average Acceleration Method with one millisecond time step size.

Numerical study has been carried out based on Unified Facility Criteria (UFC 2005) guideline from the US Department of Defense (DoD) to evaluate the performance of a building in resisting progressive collapse. It has been found that building designed according to the (EBCS) guidelines did not satisfied (UFC 2005) requirements to mitigate progressive collapse, and building designed according to the (UFC 2005) requirements could withstand blast loads result from detonation of 250 lbs (113.5 kg) at 10m standoff distance.

The dynamic response of typical RC column subjected to lateral blast loading has been examined by performing nonlinear SDOF blast analysis. It has been found that the maximum displacement result using SDOF approximation method is comparable with SAP2000 result for 250 lbs (113.5 kg) charge weight at 10m and 7m standoff distances. But the maximum displacement difference ratio of this analysis model increases as the standoff distance decreases.

Finally, it is recommended that guidelines on abnormal load cases and provisions on progressive collapse prevention should be included in the current Ethiopian Building Code Standard at least for special and important buildings.

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Notations

The major part of the symbols used in the text is listed below. These and others are defined when they first appear.

d	Differential symbol
[ ]	Matrix symbol
DLF	Dynamic Load Factor
DoD	Department of Defense
EBCS	Ethiopian Building Code Standard
Ec	Modulus of elasticity of concrete
Es	Modulus of elasticity of steel
f′c	Uniaxial compressive strength of concrete (cylinder test)
fy	Yield strength of steel
f'_{dc}	Design uniaxial compressive strength of concrete
f_{dy}	Design yield strength of steel
Is	Positive incident impulse
Is^-	Negative incident impulse
Ir	Positive normal incident reflected impulse
Ir^-	Negative normal incident reflected impulse
Pr	Peak positive normal reflected pressure
Pr^-	Peak negative normal reflected pressure
P_{so}	Peak positive incident pressure
P_{so}^-	Peak negative incident pressure
PC	Progressive Collapse
R	Standoff distance
T	Natural period of vibration
t_a	Time of arrival of blast wave
t_d	Time of duration
t_o	Positive duration of positive phase
t_o^-	Negative duration of positive phase
U	Shock front velocity
UFC	Unified Facilities Criteria
W	Charge weight
y	Particle velocity or absolute node displacement

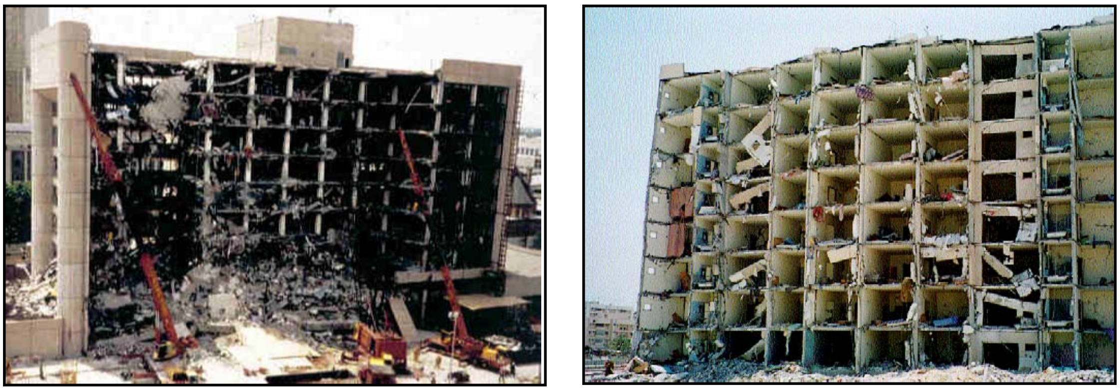
$\dot{y}$	Absolute node velocity
$\ddot{y}$	Absolute node acceleration
Z	Scaled distance
β	Newmark's factor for velocity
γ	Newmark's factor for acceleration
g	Newmark's factor for acceleration
ζ	Damping ratio
ω	Undamped natural frequency
ω_D	Damped natural frequency
Δt	Time incremental value
[C]	Viscous damping matrix
[M]	Mass matrix
[K]	Static stiffness matrix

CHAPTER ONE

INTRODUCTION TO BLAST EFFECTS ON CIVILIAN BUILDINGS

1.1 Introduction

Due to various accidental or intentional events related to important structures all over the world, explosive loads have received considerable attention in recent years. Disasters such as the bombings of the U.S. embassies in Nairobi, Kenya and Dares Salaam, Tanzania in 1998, the Khobar Towers military barracks in Dhahran, Saudi Arabia in 1996, the Murrah Federal Building in Oklahoma City in 1995, and the World Trade Center in New York in 1993 are the most notable recent examples [14]. Following those events there has been heightened interest among building owners and government entities in evaluating the progressive collapse potential of existing buildings, and in designing new buildings to resist this type of collapse [14]. Figure 1-1 (a) and (b) show the structural damage due to bomb attacks on the Murrah Federal Building Oklahoma City, 1995 and Khobar Towers ,Saudi Arabia 1996 respectively.



(a) Alfred P. Murrah Federal Building (b) Khobar Tower

Figure 1-1: Effects of bomb attacks on buildings [24]

Many military structures have been designed to resist explosive attacks. However, considering blast loading in the design of civilian building structures in the world is a relatively recent idea. While the type of loading may be the same for a military structure and a civilian building, both being external blast load, the magnitude of the load as well as architectural and economic aspects of a blast-resistant military structure are quite different from a civilian structure. In comparison with military structure the probability of explosive attack on civilian facilities is quite low and usually is in the form of an intentional sophisticated attack. Obviously, considering the very low probability of a particular civilian structure being exposed to an attack, one cannot afford to design all civilian facilities to withstand explosives. Therefore, the first step is to identify certain buildings and facilities that are critical for the functioning of society and its vital organizations such as the government, infrastructure and public facilities [1].

Blast loading and its effects on a structure is influenced by a number of factors including charge weight (W), location of the blast (or standoff distance), geometrical configuration and orientation of the structure (or direction of the blast). Structural response will differ 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.

Difficulties that arise with the complexity of the problem consequences from blast loading which involves time dependent finite deformations, high strain rates, and non-linear inelastic material behavior, have motivated various assumptions and approximations to simplify the load as well as structural models. These models span the full range of sophistication from single degree of freedom systems to more complex computer based applications [14].

1.2 Objective of the Thesis

The main objective is to study the effects of blast loads on RC framed structures and further, to see the dynamic properties of reinforced concrete structural elements under high strain rates typically produced by the blast loads.

Specific objectives

- To investigate the dynamic response and damage of RC framed structures subjected to external blast loading including; the threat definition, blast wave parameters and dynamic properties of materials.
- To look into available procedure for blast analysis on framed structures and to lay the basis for their consideration in the Ethiopian Building Code Standard.

1.3 Contents and Organization of the Thesis

The thesis is divided into five chapters:

- An introduction to the effect of blast loading in civilian building and difficulties that arise with the complexity of the problem is described in chapter one.
- Chapter Two explains the explosions definition, blast effects on buildings, evaluating the load time curve and the relation between the blast and progressive collapse. Furthermore design guidelines to mitigate progressive collapse and pervious research works are also reviewed in this Chapter.

- Chapter Three deals with the blast analysis methods ranging from simple hand calculation and SDOF approximation to more complex computer based application are explained.
- A numerical study on reinforced concrete structure subjected to blast loading is presented in Chapter Four. Beginning with a preliminary design, checking for (UFC 2005) requirements, and evaluation of the final design and determination of minimum standoff distance when subjected to blast loading after nonlinear dynamic analyses is presented in this chapter. The nonlinear SDOF analysis for typical column and comparison of results from 3D nonlinear analysis is also presented in this chapter.
- Finally, the conclusions drawn from this thesis work and the recommendations for the future work are presented in Chapter Five.

CHAPTER TWO

BLAST LOADING AND PROGRESSIVE COLLAPSE

2.1 Introduction

An explosion is a rapid and sudden release of stored potential energy characterized by a bright flash and an audible blast. Part of the energy is released as thermal radiation (flash); and part is coupled into the air as air blast and into the soil (ground) as ground shock, both as radially expanding shock waves [15].

Blast loading is unlike other types of severe loads caused by extreme events such as earthquake or high wind. These types of loads generate damage that is limited to a very few structural response mechanisms, and they are applied "globally" such that the entire structural system works to resist the load. Explosive blast activates many structural response mechanisms because of its extreme spatial and time variations in magnitude and time of application (duration).

2.2 Blast Loading

The blast loading on structures can be divided into two main groups based upon the confinement of the explosive charge [23].

2.2.1 Unconfined Explosions

This group can be divided into three types based upon the relative charge location and the blast loading produced on the structures.

2.2.1.1 Free Air Burst Loads

The blast wave propagates away from the center of the explosion striking the structures without intermediate amplification of the initial shock wave.

2.2.1.2 Air Burst Loads

The explosion is located at a distance away and above the structure so that ground reflections of the initial wave occur before the blast wave reaches the structure.

2.2.1.3 Surface Burst Loads

The explosion is located close to or on the ground so that the shock wave is amplified at the point of detonation due to ground reflection. Only this type of blast loads considered in this thesis.

2.2.2 Partially Confined Explosions

This can be divided into:

2.2.2.1 Exterior or Leakage Pressures

The detonation occurs near the ground surface and behind an obstruction so that the shock wave is interfered with before reaching the structure.

2.2.2.2 Interior or High Pressure Loads

The detonation is located within or immediately adjacent to barrier type structures and the blast pressure amplified due to their multiple reflections by the structure as a result of the closeness of the structure to the explosion.

2.3 Blast Wave Phenomena [23].

The violent release of energy from a detonation in a gaseous medium gives rise to sudden pressure increase in that medium. The pressure disturbance, termed the blast wave is characterized by an almost instantaneous rise from the ambient pressure to a peak incident pressure (Pso). Incident pressure is the pressure on a surface parallel to the direction of the blast wave.

This pressure increase or shock travels radially from the burst point with a diminishing velocity (U) which is always in excess of the sonic velocity of the medium. Gas Molecules, making up the front move at lower velocities (u). This latter particle velocity is associated with a dynamic pressure or the pressure formed by the winds produced by the shock fronts and it's a function of air density and wind velocity. As the shock front expands into increasingly larger volumes of the medium, the peak incident pressure at the front decreases and the duration of the pressure increases.

At any point from the burst, the pressure disturbance has the shape shown in Figure 2-1. The shock front arrives at time (t_A) and, after the rise to the peak value, the incident pressure decays to the ambient value in time (to) which is the positive phase duration. This is followed by a negative phase with duration (to^-) longer than the positive phase and characterized by a pressure below the pre-shot ambient pressure (maximum value of $P_{so}{}^-$) and a reversal of the particle flow.

The incident impulse associated with the blast wave is the integrated area under the pressure-time curve and is denoted as (I_s) for the positive phase and (I_s^-) for the negative phase as shown in Figure 2-1.

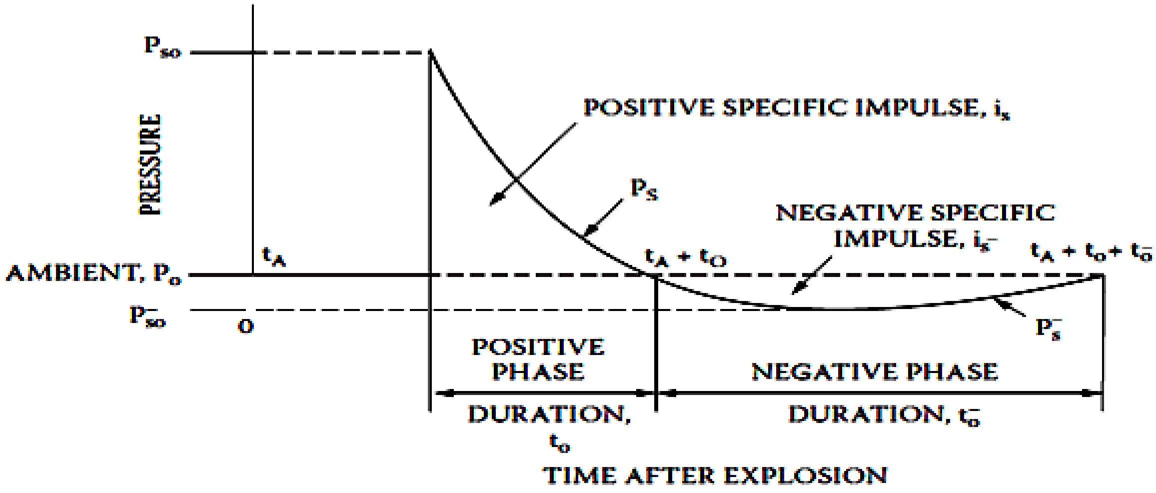


Figure 2-1: Typical blast-induced pressure-time history [21]

If the shock wave impinges on a rigid surface oriented at an angle to the direction propagation of the wave, a reflected pressure 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. The duration of the reflected pressure is controlled by the size of the reflecting surface. The peak positive reflected pressure is denoted as (Pr), the peak negative reflected pressure is (Pr $^-$), and the unit- impulses associated with a completely reflected incident wave are (Ir) for the positive phase and (Ir $^-$) for negative phase as shown in Figure 2-2. When the shock wave impings on a surface that is perpendicular to the direction of travel of the shock wave, then the point of the initial contact will be subjected to the maximum reflected pressure and impulse.

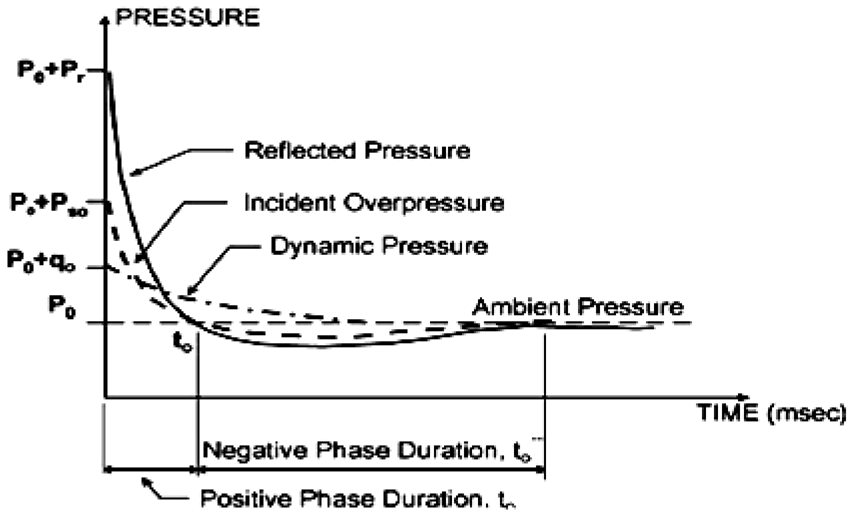


Figure 2-2: Complete overpressure - time profile [22].

2.4 Surface Burst Loading

The threat for a conventional bomb is defined by two equally important elements, the bomb size, or charge weight W, and the standoff distance (R) between the blast source and the target Figure 2-3. For example, the blast occurred at the basement of World Trade Centre in 1993 has the charge weight of 816.5 kg TNT. The Oklahoma bomb in 1995 has a charge weight of1814 kg at a stand-off of 5m [15]. As attacks may range from the small letter bomb to the gigantic truck bomb as experienced in Oklahoma City, the mechanics of a conventional explosion and their effects on a target must be addressed.

Throughout the pressure-time profile, two main phases can be observed; portion above ambient is called positive phase of duration (t_d), while that below ambient is called negative phase of duration (t_d^-). The negative phase is of a longer duration and a lower intensity than the positive duration. As the stand-off distance increases, the duration of the positive-phase blast wave increases resulting in a lower-amplitude, longer-duration shock pulse. Charges situated extremely close to a target structure impose a highly impulsive, high intensity pressure load over a localized region of the structure; charges situated further away produce a lower-intensity, longer-duration uniform pressure distribution over the entire structure. Eventually, the entire structure is engulfed in the shock wave, with reflection and diffraction effects creating focusing and shadow zones in a complex pattern around the structure. During the negative phase, the weakened structure may be subjected to impact by debris that may cause additional damage.

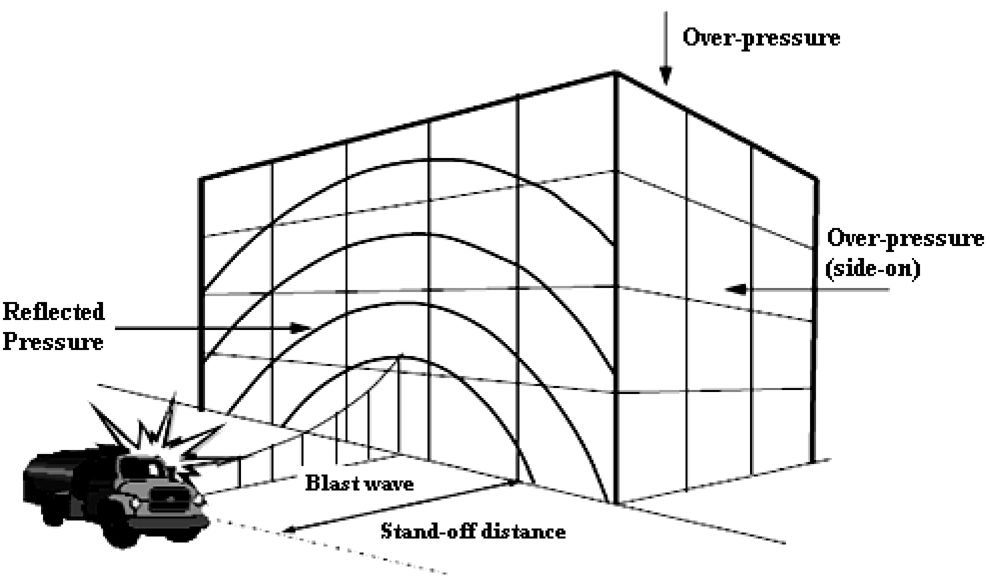


Figure 2-3: Blast Loads on a Building [22]

Here it is necessary to say that there are different types of explosives and they differ from each other in many properties such as: detonating rate, velocity, density, and heat production. *TNT* is used as the standard or reference explosive for equating the blast effects on structures. Other explosives can be converted to an equivalent weight of TNT. The equivalent weight of an explosive is based on blast pressure or impulse.

2.5 Prediction of Blast Pressure

The methods available for prediction of blast effects on buildings structures are [3]:

- Empirical methods
- Semi-empirical methods
- Numerical methods

Empirical methods are essentially correlations with experimental data. Most of these approaches are limited by the extent of the underlying experimental database. The accuracy of all empirical equations diminishes as the explosive event becomes increasingly near field.

Semi-empirical methods are based on simplified models of physical phenomena. The attempt is to model the underlying important physical processes in a simplified way. These methods are dependent on extensive data and case study. The predictive accuracy is generally better than that provided by the empirical methods.

Numerical (or first-principle) methods are based on mathematical equations that describe the basic laws of physics governing a problem. These principles include conservation of mass, momentum, and energy. In addition, the physical behavior of materials is described by constitutive relationships. These models are commonly termed computational fluid dynamics (CFD) models.

2.6 Evaluation of Load Time Shape

For most structures a triangular shape is assumed for the dynamic blast load with a sudden rise and linear decay as shown in Figure 2-5. The negative phase is neglected because it usually has little effect on the maximum response [5].

To evaluate the load time shape on any element the following procedure is used [15]:

1. Calculate the scaled distance:

$$\boldsymbol{Z} = \frac{\boldsymbol{R}}{\boldsymbol{W}^{1/3}} \qquad (1)$$

Where R is the distance from the center of explosion to the face of the element and W is the weight of the TNT explosive.

2. Peak pressure (incident or reflected), impulse, velocity, time of arrival, time of duration, and other parameters can be found from Figure 2-4.
3. The time of duration (t_d) can be calculated from the following relationship:

$$t_d = \frac{2I}{P} \tag{2}$$

Where I the impulse per unit of projected area obtained by integrating Ps (t) from t=0 to t= t_d

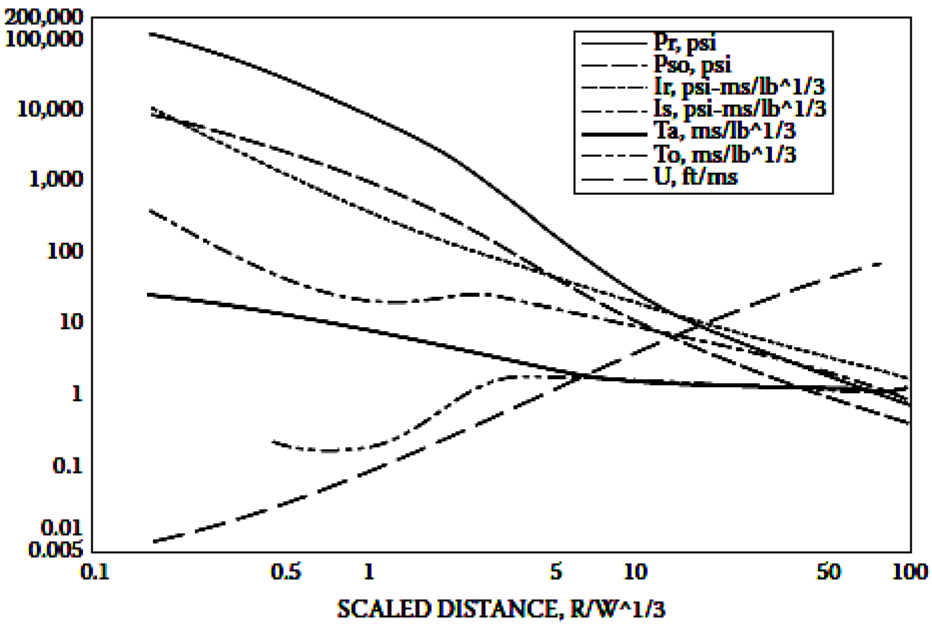


Figure 2-4: Shock wave parameters for surface bursts TNT at sea level [21]

A full discussion and extensive charts for predicting blast pressures and blast durations is given by TM5-1300. Some representative numerical values of peak reflected overpressure are given in Table 2-1. There is also publicly available software program A.T.-Blast [4] (Anti-Terrorism Blast) which is developed for blast load prediction according to TM5-1300. A polynomial fit-based equations, as well as equations that describe and account for angle of incidence and reflection coefficients, are incorporated in this software. This software is used for the purpose of estimating the blast pressure and impulse from a high explosive detonation as a function of standoff distance in this thesis.

Table 2-1: Peak reflected overpressures Pr (in MPa) with different W-R combinations [23]

Radius[m]	Weight[Kg]			
	100kg	**500kg**	**1000kg**	**2000kg**
1m	165.8	354.5	464.5	602.9
2.5m	34.2	89.4	130.8	188.4
5m	6.65	24.8	39.5	60.19
10m	0.85	4.25	8.15	14.7
15m	0.27	1.25	2.53	5.01
20m	0.14	0.54	1.06	2.13
25m	0.09	0.29	0.55	1.08
30m	0.06	0.19	0.33	0.63

For design purposes, reflected overpressure can be idealized by an equivalent triangular pulse of maximum peak pressure P_r and time duration t_d, which yields the reflected impulse i_r

$$\mathbf{i_r = \frac{1}{2} P_r t_d} \quad (3)$$

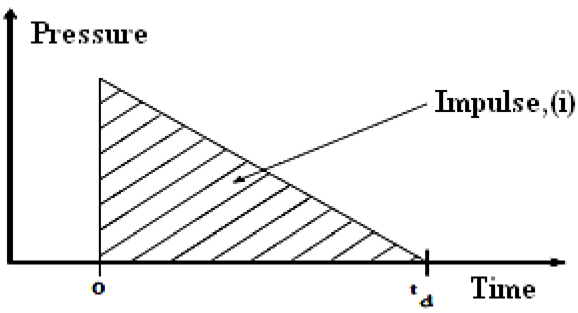


Figure 2-5: Load time shape.

2.7 Material Behaviors at High Strain Rate

Blast loads typically produce very high strain rates in the range of 10^2 - 10^4 s^{-1} [22]. This high straining (loading) rate would alter the dynamic mechanical properties of target structures and, accordingly, the expected damage mechanisms for various structural elements. Figure 2-6 shows the approximate ranges of the expected strain rates for different loading conditions. For reinforced concrete structures subjected to blast effects the strength of concrete and steel reinforcing bars can increase significantly due to strain rate effects.

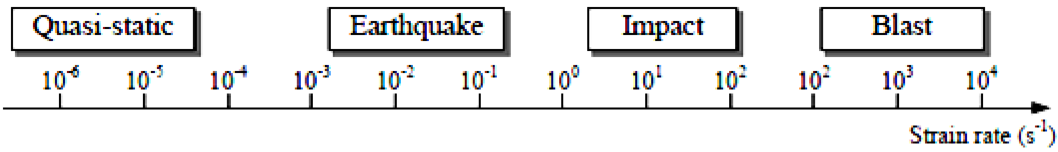


Figure 2-6: Strain rates associated with different types of loading [22]

It was shown experimentally that steel yield strength is more sensitive to rate effects than the ultimate strength, and one notes that the dynamic increase factors for the steel yield strength are much higher than for the ultimate strength [22]. This observation is also illustrated in Figure 2-7. For concrete, the entire static stress–strain curve is scaled by the appropriate dynamic increase factor. However, for steel, the yield strength is scaled by one factor, while the ultimate is scaled by another (see Figure 2-8). Additionally further refinements to the dynamic increase factor values cited in Table 2-2.

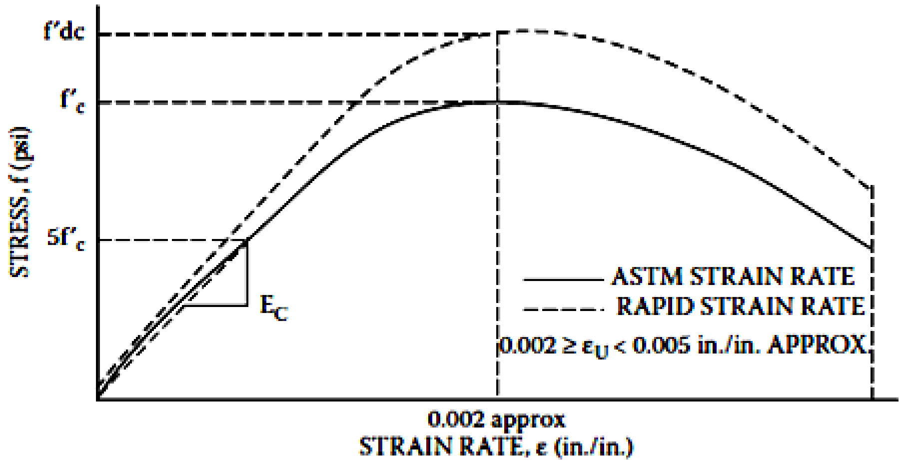


Figure 2-7: Effect of strain rate on stress-strain curve for concrete [21].

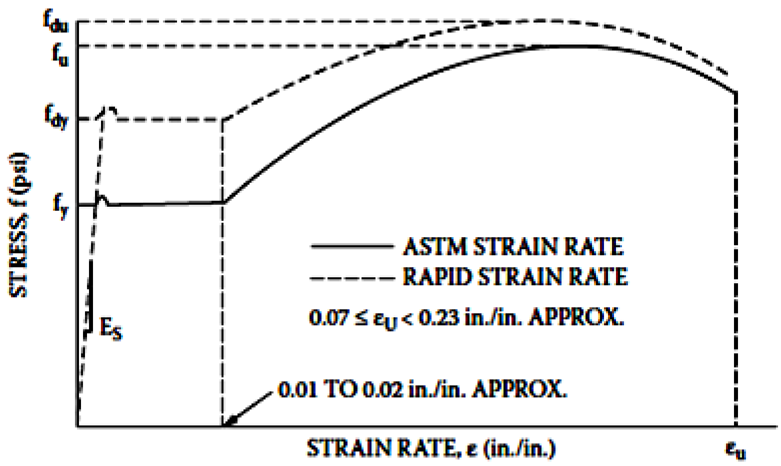


Figure 2-8: Effect of strain rate on Stress-strain curve for steel [21].

Table 2-2: Reinforced Concrete Design Dynamic Increase Factors [21]

Type of Stress	Far Design Range			Close-In Design Range		
	Reinforcing Bars		Concrete	Reinforcing Bars		Concrete
	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c
Bending	1.17	1.05	1.19	1.23	1.05	1.25
Diagonal tension	1	—	1	1.1	1	1
Direct shear	1.1	1	1.1	1.1	1	1.1
Bond	1.17	1.05	1	1.23	1.05	1
Compression	1.1	—	1.12	1.13	—	1.16

2.8 Progressive Collapse

Explosive loading incidents have become serious problems that must be addressed frequently. Besides the immediate and localized blast effects, one must consider the serious consequences associated with 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 are loaded beyond their capacity and fail. The residual structure is 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 [21].

It is estimated that at least 15 to 20% of the total number of building failures are due to progressive collapse [21]. A notable example of such a failure is the Ronan Point collapse. Since an explosion caused the progressive collapse at Ronan Point, a number of studies were devoted to include the relationships of abnormal loadings and progressive collapse in building standards. The British Standards employed three design approaches for resisting progressive collapse [7]:

1. *Tie Forces (TF).* This indirect design approach enhances continuity, ductility, and structural redundancy by requiring "ties" to keep the structure together in the event of an abnormal loading.
2. *Alternate Path (AP).* This direct method requires that the designer prove that the structure is capable of bridging over a removed structural element and that the resulting extent of damage does not exceed the damage limits. The missing structural element is any element that cannot provide an adequate vertical tie force.

3. *Specific Local Resistance (SLR).* This direct method requires that, for any structural element over which the building cannot bridge, the element must be designed as a "key" or "protected" element, capable of carrying a static pressure loading of 34 KN/m^2.

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. This Unified Facilities Criteria (UFC 2005) [7] provides the design requirements necessary to reduce the potential of progressive collapse for new and existing DoD facilities that experience localized structural damage through normally unforeseeable events.

The design requirements presented in this UFC were developed such that two structural response modes are available to provide different levels of resistance to progressive collapse. The first level of progressive collapse design employs Tie Forces, which are based on a "catenary" response of the structure. The second level employs the Alternate Path method, in which the structural mode is "flexural", as the building must bridge across a removed element.

The Progressive Collapse design using UFC is threat-independent and is not intended to address the hardening of a building that is exposed to a specific explosive threat .Instead level of progressive collapse design for a structure is correlated to the Level of Protection (LOP) that the Project Planning Team develops and provides to the designer as given in Table 2-3.

Table 2-3: Level of Protection and PC Design Requirements for new and existing Construction [24]

<table>
<tr><th>Level of Protection</th><th>PC Design Requirement</th></tr>
<tr><td>Very Low</td><td>Provide horizontal Tie Forces.</td></tr>
<tr><td>Low</td><td>Provide horizontal and Low vertical Tie Forces.</td></tr>
<tr><td>Medium</td><td rowspan="2">Satisfy the following three requirements:
A) Provide horizontal and vertical Tie Forces.
B) Apply the Alternate Path method.
C) Meet additional ductility requirements that effectively “harden” the perimeter, ground-floor load-bearing elements</td></tr>
<tr><td>High</td></tr>
</table>

For all Levels of Protection, all multistory vertical load-carrying elements must be capable of supporting the vertical load after the loss of lateral support at any floor level (i.e., a laterally unsupported length equal to two stories must be used in the design or analysis)[7].

There are three allowable analysis procedures for progressive collapse [7]: Linear Static, Nonlinear Static, and Nonlinear Dynamic. These methods are summarized as follows:

1. Linear Static. The geometric formulation is based on small deformations and the material is treated as linear elastic. The full load is applied at one time to the structure, from which a vertical load-bearing element has been removed.
2. Nonlinear Static: Both the material and geometry are treated as nonlinear. A load history from zero loads to the full factored load is applied to the structure with a removed vertical load-bearing element.
3. Nonlinear Dynamic: The material and geometry are treated as nonlinear. A dynamic analysis is performed by instantaneously removing a vertical load-bearing element from the fully loaded structure and analyzing the resulting motion.

2.9 Previous Work

T. Ngo, et al. [22] for their study on "Blast loading and Blast Effects on Structures" gives an overview on the analysis and design of structures subjected to blast loads phenomenon for understanding the blast loads and dynamic response of various structural elements. This study helps for the design consideration against extreme events such as bomb blast, high velocity impacts.

E. Agnew, et al. [8] study on "Dynamic analysis procedures for progressive collapse" discuses general idea on progressive collapse phenomena and dynamic model of progressive collapse. It provides the step by step analysis procedure for linear and nonlinear dynamic analysis.

A.M. Remennikov [3] studied on "A review of the methods for predicting bomb blast effects on buildings". When a single building is subjected to blast loading produced by the detonation of high explosive device. Simplified analytical techniques used for obtaining conservative estimates of the blast effects on buildings. Numerical techniques including Lagrangian, Eulerian, Euler-FCT, ALE, and finite element modelling used for accurate prediction of blast loads on commercial and public buildings.

TM 5-1300 (UFC 3-340-02) [23] is a manual titled "structures to resist the effects of accidental explosions" is the best known source in literature among several methods for establishing blast load parameters. It provides guidance to designers, the step-to-step analysis and design procedure, including the information on such items (1) Blast, fragment and shock loading. (2) Principle on dynamic analysis. (3) Reinforced and structural steel design and (4) A number of special design considerations.

CHAPTER THREE

ANALYSIS METHODES FOR BLAST LOADING

3.1 Introduction

Several analysis methods are used for blast resistance design ranging from simple hand calculations and graphical solutions to more complex computer dynamic based applications. Common methods/approaches for blast analysis are discussed below

3.2 Equivalent Static Method

One method of blast analysis which had been commonly used in the past, but which is no longer advocated is the equivalent static method [5]. The method employs a static analysis with an approximate applied load to simulate the dynamic response. Dynamic parameters such as time varying loads, rapid strain rate material strengths, load amplification factors, mass, stiffness, period of vibration, and allowable plastic deformations are not used. The primarily difficulty with this method is determining an appropriate static loading which will yield reasonable results. This method is not recommended for general use except for cases where the structure is far removed from the blast source, such that the blast loading resembles a wind gust.

3.3 Single Degree Of Freedom System (SDOF)

Complexity in analyzing the dynamic response of blast-loaded structures involves the effect of high strain rates, the non-linear inelastic material behavior, the uncertainties of blast load calculations and the time-dependent deformations. Therefore, to simplify the- analysis, a number of assumptions related to the response of structures and the loads has been proposed and widely accepted. To establish the principles of this analysis, the 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 simplifies the classification of the blast loading regimes [22].

3.3.1 Elastic SDOF Systems

The simplest discretization of transient problems is by means of the SDOF approach. The actual structure can be replaced by an equivalent system of one concentrated mass and one weightless spring representing the resistance of the structure against deformation. Such an idealized system is illustrated in Figure 3-1. The structural mass, M, is under the effect of an external force, $F(t)$, and the structural resistance, R, is expressed in terms of the vertical displacement, y, and the spring constant, K.

The blast load can also be idealized as a triangular pulse having a peak force F_m and positive phase duration t_d (see Figure 3-1). The forcing function is given as

$$\boldsymbol{F(t) = F_m\left(1 - \frac{t}{t_d}\right)} \qquad (4)$$

The blast impulse is approximated as the area under the force-time curve, and is given by

$$\mathbf{I} = \frac{1}{2}\boldsymbol{F_m}\mathbf{t_d} \qquad (5)$$

The equation of motion of the un-damped elastic SDOF system for a time ranging from 0 to the positive phase duration, t_d, is given by Biggs [13] as:

$$\boldsymbol{M\ddot{y} + Ky = F_m\left(1 - \frac{t}{t_d}\right)} \qquad (6)$$

The general solution for displacement y (t) and velocity ý (t) can be expressed as

$$\boldsymbol{y(t) = \frac{F_m}{K}(1 - \cos\omega t) + \frac{F_m}{Kt_d}\left(\frac{\sin\omega t}{\omega} - t\right)} \qquad (7)$$

$$\boldsymbol{\dot{y}(t) = \frac{dy}{dt} = \frac{F_m}{K}\left[(\omega \sin\omega t) + \frac{1}{t_d}(\cos\omega t - 1)\right]}$$

In which ω is the natural circular frequency of vibration of the structure and T is the natural period of vibration of the structure which is given by equation 8.

$$\boldsymbol{\omega = \frac{2\pi}{T} = \sqrt{\frac{K}{M}}} \qquad (8)$$

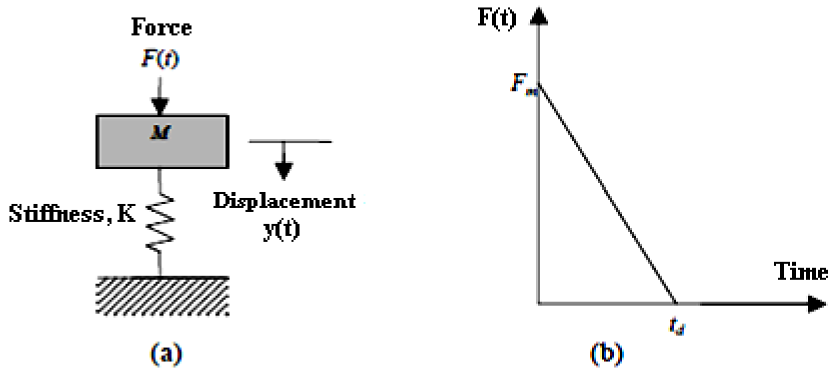


Figure 3-1: (a) SDOF system and (b) blast loading [22]

The maximum response is defined by the maximum dynamic deflection y_m which occurs at time t_m. The maximum dynamic deflection y_m can be evaluated by setting dy/dt in Equation 7 equal to zero, i.e. when the structural velocity is zero. The dynamic load factor, DLF, is defined as the ratio of the maximum dynamic deflection y_m to the static deflection y_{st} which would have resulted from the static application of the peak load F_m, which is shown as follows:

$$\mathbf{DLF} = \frac{\boldsymbol{y_{max}}}{\boldsymbol{y_{st}}} = \frac{\boldsymbol{y_{max}}}{\boldsymbol{F_m/k}} = \boldsymbol{\psi}(\boldsymbol{\omega t_d}) = \boldsymbol{\psi}\left(\frac{\boldsymbol{t_d}}{\boldsymbol{T}}\right) \tag{9}$$

The structural response to blast loading is significantly influenced by the ratio t_d/T or ωt_d ($t_d/T=\omega t_d/2\pi$). Three loading regimes are categorized as follows:

- $\omega t_d < 0.4$: impulsive loading regime.
- $\omega t_d > 40$: quasi-static loading regime.
- $0.4 < \omega t_d < 40$: dynamic loading regime.

3.3.2 Elasto-Plastic SDOF Systems

Structural elements are expected to undergo large inelastic deformation under blast load or high velocity impact. Exact analysis of dynamic response is then only possible by step-by-step numerical solution requiring nonlinear dynamic finite- element software. However, the degree of uncertainty in both the determination of the loading and the interpretation of acceptability of the resulting deformation is such that solution of a postulated equivalent ideal elasto-plastic SDOF system is commonly used [22]. Interpretation is based on the required ductility factor $\mu = y_m/y_e$ (Figure 3-2).

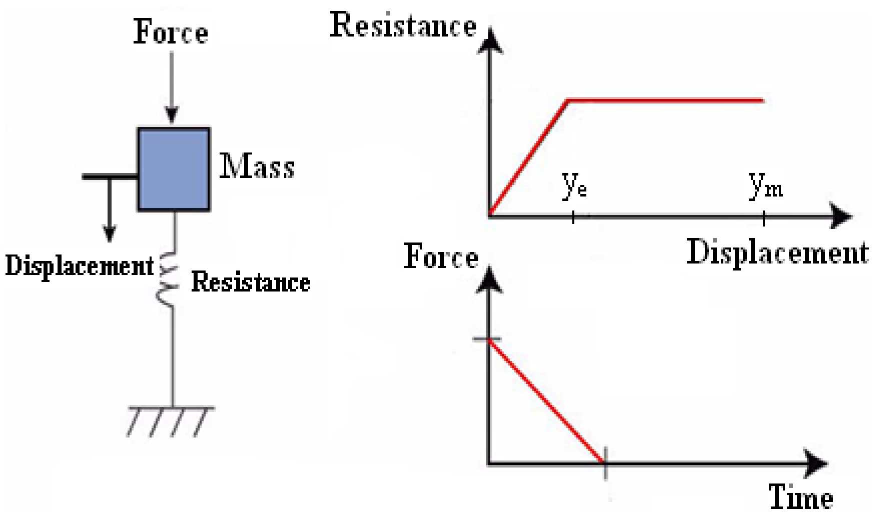


Figure 3-2: Simplified elasto-plastic SDOF model for blast load [11].

The response of the ideal bilinear elasto-plastic system can be evaluated using:-

1. *Graphical Presentation of Solutions*: - Analytical solutions for SDOF systems may be very cumbersome, even for simple loading functions. Such computations become much harder for nonlinear resistance functions and complicated loading pulses. To simplify such computations in support of design activities, one may use dynamic response charts that enable an analyst to estimate the values of key parameters for assessing the suitability of a tentative structural design. For the triangular load pulse comprising rapid rise and linear decay, with maximum value F_1 and duration t_d. The result for the maximum displacement is generally presented in chart form (TM 5-1300), as a family of curves for selected values of R_u/F_1 showing the required ductility μ as a function of t_d/T, in which R_u is the structural resistance of the beam and T is the natural period (Figure 3-3).

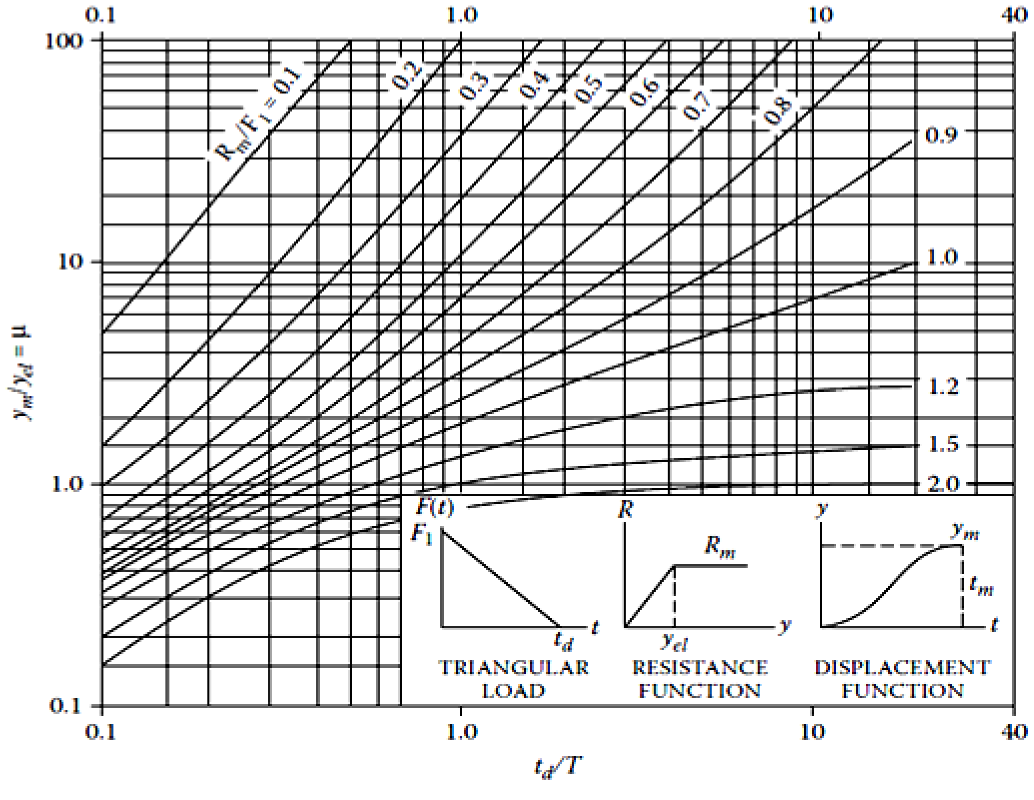


Figure 3-3: Maximum response of elasto-plastic SDOF systems under triangular load pulse with zero rise time [21].

2. *Closed Form Solutions*: - They are available only for some simple loading cases of SDOF systems. Published solutions exist for both elastic and elastic-plastic responses, and for triangular and rectangular load pulses. The analysis can also be greatly simplified when the duration of the loading, t_d, is either very short or extremely long compare to the period, t_n [5].

3. *Numerical Integration:* - The closed form solution of the equation of motion by the approach described earlier may not be possible for highly nonlinear cases. In this case numerical time integration method can be used. This method is also known as the time history method. Most text on structural dynamics (Bigges 1964, Chopra 2001, Paz 1991) provides extensive coverage on numerical solution methods for nonlinear SDOF systems.

It is necessary to derive an efficient numerical integration procedure to solve dynamic equation that will be valid for a wide range of cases. One such technique is the Newmark numerical integration method, which is commonly used to obtain the time history response for nonlinear SDOF systems. For a dynamic equilibrium equation N.M. Newmark developed a family of time-stepping solution based on the following equations [2]:

$$[\boldsymbol{M}]\ddot{\boldsymbol{y}} + [\mathbf{C}]\dot{\boldsymbol{y}} + [\boldsymbol{K}]\boldsymbol{y} = \boldsymbol{F_t} \tag{10}$$

$$\dot{\boldsymbol{y}}_{\boldsymbol{i+1}} = \dot{\boldsymbol{y}}_{\boldsymbol{i}} + [(\mathbf{1} - \boldsymbol{\gamma})\boldsymbol{\Delta}\mathbf{t}]\ddot{\boldsymbol{y}} + (\boldsymbol{\gamma\Delta}\mathbf{t}) + \ddot{\boldsymbol{y}}_{\boldsymbol{i+1}} \tag{11}$$

$$\boldsymbol{y}_{\boldsymbol{i+1}} = \boldsymbol{y}_{\boldsymbol{i}} + (\boldsymbol{\Delta}\mathbf{t})\dot{\boldsymbol{y}}_{\boldsymbol{i}} + [(\mathbf{0.5} - \boldsymbol{\beta})(\boldsymbol{\Delta}\mathbf{t}^{\mathbf{2}})]\,\ddot{\boldsymbol{y}}_{\boldsymbol{i}} + [\boldsymbol{\beta}(\boldsymbol{\Delta}\mathbf{t}^{\mathbf{2}})]\ddot{\boldsymbol{y}}_{\boldsymbol{i+1}} \tag{12}$$

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.

On the other hand, the accuracy obtainable from a SDOF approximation depends on how well the deformed shape of the structure and its resistance can be represented with respect to time [5]. The properties of the equivalent SDOF system are also based on load and mass transformation factors, which are calculated to cause conservation of energy between the equivalent SDOF system and the component assuming a deformed component shape and that the deflection of the equivalent SDOF system equals the maximum deflection of the component at each time.

In this thesis an MS Excel® based Computer Program is developed for the design of structural components subjected to blast loads using Single Degree Of Freedom (SDOF) methodology. Dynamic equilibrium equation is solved by applying numerical time integration method according to Newmark. Newmark's steps are easily programmed for *general resistance-deflection function* using VBA programming language. This code is used for blast analysis in chapter 4. (Source code: Refer to appendix)

The algorithm for the step by step Newmark's linear acceleration method of a single degree of freedom system is outlined below [5]:

- Determine the stiffness, K, mass, M, damping coefficient, C, force function F(t), Maximum restoring force , R_t, and time increment, Δt.
- At each time step (step =0 to end),determine the value of the forcing function, $F_o \ldots F_{end}$.
- For the intial time step (step=0) , intialize the displacement,velocity and acceleration.
  - $y_o=0$; $v_o=0$; $a_o=F_o/M$
- For each time (step =i ,beginning with i=0)
  - Caculate the effective stiffness,

    $K_i' = K +(6/\Delta t^2)M+(3/ \Delta t)C$
  - Caculate the effective incremental force,

    $\Delta F_i'=(F_{i+1}-Fi)+[(6/ \Delta t)M+(3)C]v_i+[(3)M+(\Delta t/2)C]a_i$
  - Solve for the incremetal displacement,

    $\Delta y_i= \Delta F_i /K_i'$
  - Calculate the incremetal velocity,

    $\Delta v_i= (3/ \Delta t)\Delta y_i-(3)a_i-(\Delta t /2)a_i$
  - Calculate the displacement , and velocity at the next time step (step=i+1)

    $y_{i+1}=y_i+ \Delta y_i$

    $v_{i+1}=v_i+ \Delta v_i$
  - Calculate the accelaration a_{i+1} at the end of the time interval using the dynamic equation of equilibrium :

    $a_{i+1=}(1/M)[(F_{i+1})-(C_{i+1})(v_{i+1})-[R_t-(yi- y_{i+1})K]$
- Repetition for the next time steps

3.4 Finite Element Analysis Method

A finite element analysis method is recommended when one or more of the following conditions exist [5]:

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 no uniform mass and stiffness characteristics.

Many commercial finite element programs are available for nonlinear dynamic analysis. Computational methods of computer modelling used by those software's for blast analysis can be categorized as coupled or uncoupled analysis [22]. In coupled analysis, both blast load prediction and structural modelling are achieved together in one model. Uncoupled analysis models analyses structures by applying pre-determined loads separately. Coupled analysis tends to be less accurate due to software limitations [22]. In this thesis uncoupled analysis commercially available finite element structural software is used i.e. SAP2000 to perform nonlinear dynamic analysis for greater approximation.

CHAPTER FOUR

CASE STUDTY ON REINFORCED CONCRETE FRAME BUILDING SUBJECTED TO EXTERNAL EXPLOSION.

4.1 Introduction

This chapter illustrates a theoretical study that was carried out on reinforced concrete frame building subjected to blast loads. First, the building is designed for dead, live, and earthquake loads according to the specification of EBCS 1995 (Ethiopian Building Code Standard) [9] guidelines. Then the design was checked for UFC 2005 requirements. Then the structure was subjected to blast loads resulting from detonation of 250 lb (113kg) weight at different standoff distances to investigate the minimum standoff distance to resist blast induced progressive collapse . Along this a typical column subjected to blast loads is selected from the frame and analyzed using Computer Program written for SDOF systems. Finally its results are obtained and compared with previously available SAP2000 outputs.

4.2 Preliminary Design

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 and each bay 6 m in length. (Figure 4-1)

The structure was assumed to have the following properties:

1. All connections were assumed to be moment resistance.
2. Column to foundation connections are considered fixed.
3. Material properties: concrete strength (f'_c) = 20 MPa, rebar yield strength (f_y) = 300 MPa. Modulus of elasticity of concrete (E_c) = 28000 MPa and modulus of elasticity of rebar (E_s) = 200000 MPa.

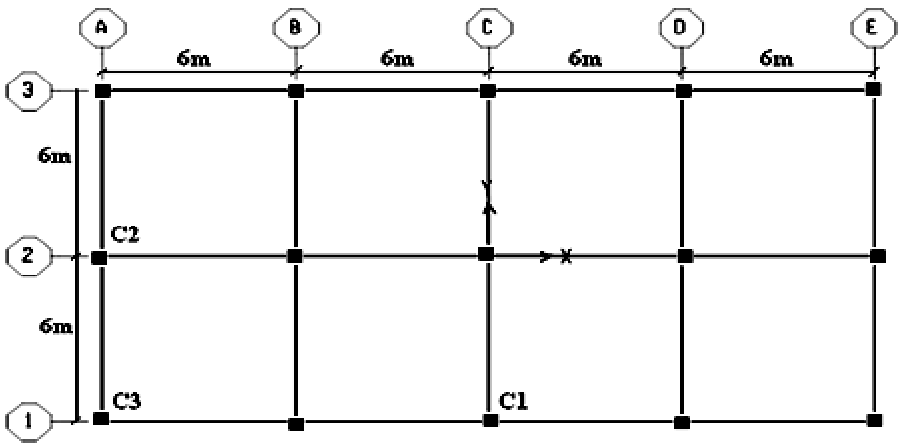


Figure 4-1: Reinforced Concert Building Plan and Location of the Removed Columns.

For the static loads, the following was assumed:

- Dead Load (DL) is equal to 3.225 kPa without self weight.
- Live Load (LL) is equal to 2 kPa.

Regarding earthquake the loads have been calculated and included as lateral loads. Equivalent static force is applied by considering the following basic assumption:

- Location of building in Zone 2.
- Importance factor, I = 1.4.
- Ductility factor, $k_D = 1$ for Ductility class "H".

Once the earthquake forces are determined, linear elastic structural analysis is used to determine the axial forces, bending moments and shear forces on the structure. Then members were designed using the most severe design requirements for any member in a group. Preliminary design for member sizes and their reinforcement is shown below in Table 4-1.

Table 4-1: Preliminary design parameters

Member Group.	**Dimensions.**	**Top Reinforcement.**	**Bottom Reinforcement.**
Beams	0.3×0.4 m	1200 mm^2	1200 mm^2
Columns	0.4×0.4 m	2500 mm^2	

4.3 Analysis of the Building Model for UFC 2005 Requirements [7]

According to the (UFC 2005) requirements for building safety against progressive collapse, the following steps shall be traced:

1. Remove the same column(s) as shown in Figure 4-1.
2. Apply 25% increasing factor for the material strength.
   (f′c) = 1.25* 20 = 25 MPa.
   (fy) = 1.25* 300 = 375 MPa.
3. Per the requirements of the UFC, the following load combination is required for analysis:
   (1.2Dead load + 0.5Live load).
4. Formation of Plastic Hinges.

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 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.

Theoretically hinges can occur anywhere along the beam. However, hinges are allowed to occur at the ends of each member and at the mid span of the flexural members. This simplifies the model by placing hinges in the most probable locations.

As shown in Figure 4-2 concrete moment hinge and concrete P-M-M hinge features is as follows:

a. Slope between points B and C is taken as 10 % total strain hardening for steel.
b. $\theta_y = 0$, since it is not needed.
c. Points C, D and E based on FEMA-356 [10].
d. M_y based on reinforcement provided.
e. P-M-M curve taken to be same as the moment curve in conjunction with the definition of Axial–Moment interaction curves.

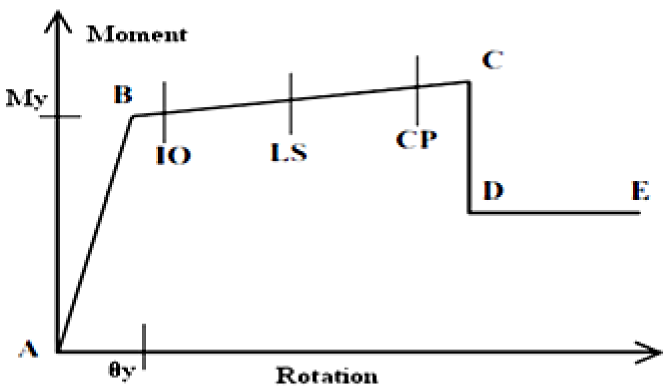


Figure 4-2: Plastic hinge properties

The qualitative performance levels of Federal Emergency Management Agency (FEMA) 356 [10] superimposed on a global force-displacement relationship for a sample building. Brief descriptions of the building damage and business interruption (downtime) for the three performance levels are given in Table 4-2.

Table 4-2: Building performance levels [10]

Performance level.	Damage description.	Down time.
Immediate occupancy.	Negligible structural damage; essential systems operational; minor overall damage	24 hours
Life safety.	Probable structural damage; no collapse; Minimal falling hazards; adequate emergency egress.	Possible total loss
Collapse prevention.	Severe structural damage; incipient collapse; probable falling hazards; possible restricted access	Probable total loss

5. Analyze the structure using nonlinear static analysis.

The building was analyzed, using SAP2000, a popular and widely used software package for conventional structural design. To simulate the column removal, the “non-linear staged construction” feature in SAP2000 was used. The model was analyzed in two stages using maximum of 100 steps per stage. In the first stage, the total load was applied to all elements; in the second, the column was removed and the analysis was run until the building settles. After the building has settled, the maximum plastic hinge rotations were observed. If the maximum plastic rotations were exceed the limit, the members must be redesign and repeat the analysis, until the plastic rotations from the analysis is within a limit.

By performing these procedures the analysis results for the deformed shapes and hinge formation are as shown in Figure 4-3, Figure 4-4 and Figure 4-5, for columns C1, C2, and C3 removal respectively.

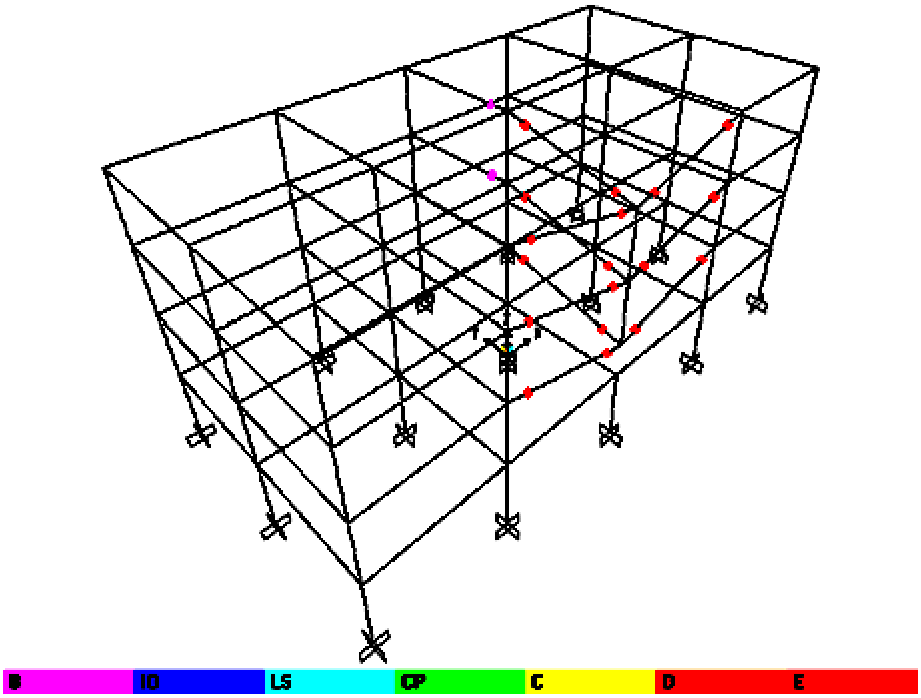


Figure 4-3: Hinges and deformed shape when C1 removed.

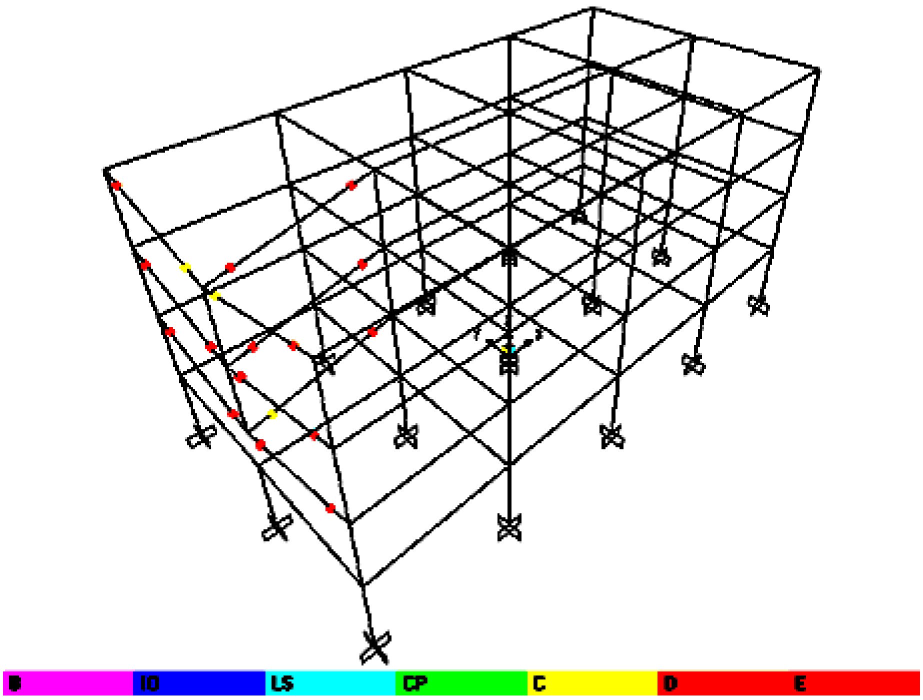


Figure 4-4: Hinges and deformed shape when C2 removed.

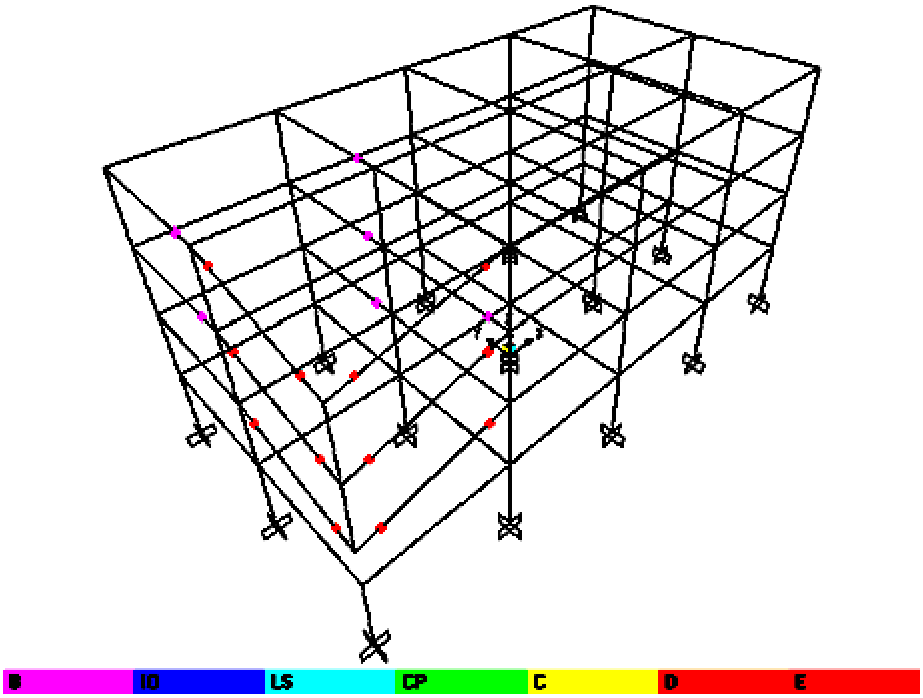


Figure 4-5: Hinges and deformed shape when C3 removed.

From the hinges and deformed shapes it is seen that the building is not safe according to the (UFC 2005) requirements, because the plastic hinge state exceeds the allowable rotation. Any hinge that is orange (D) or red (E) has failed. So the members must be redesigned.

6. Redesigned members.

The beam depth and steel ratio is increased and for the columns it is assumed to have only larger steel ratio as shown in Table 4-3.

Table 4-3: Properties of redesigned members.

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

Then, the same procedure is applied to the redesign building. The analysis results for the deformed shapes and hinge formation are as shown in Figure 4-6, Figure 4-7 and Figure 4-8, for columns C1, C2, and C3 removal respectively.

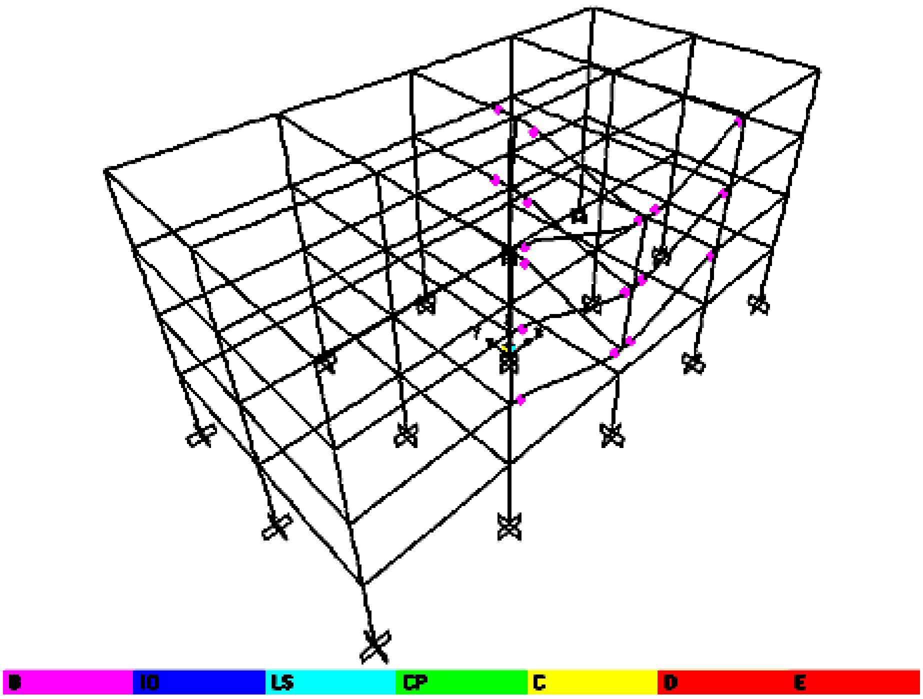


Figure 4-6: Hinges and deformed shape when C1 removed.

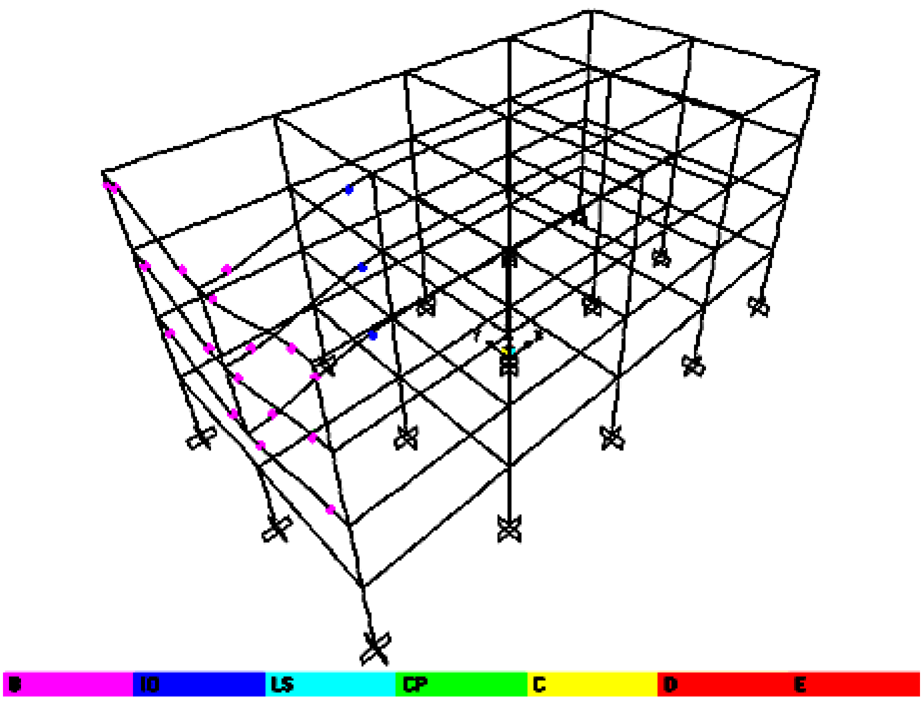


Figure 4-7: Hinges and deformed shape when C2 removed.

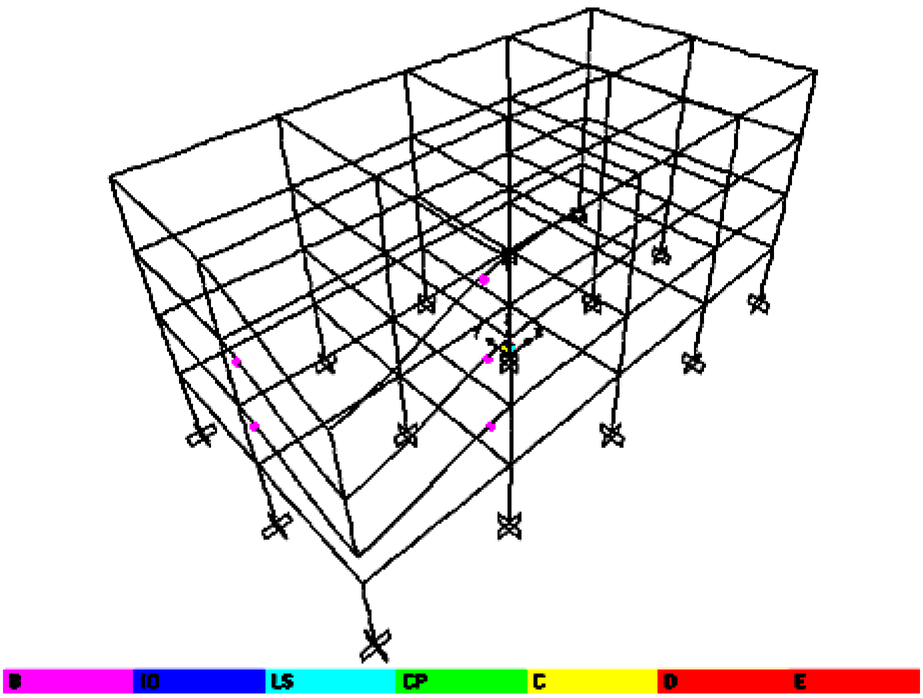


Figure 4-8: Hinges and deformed shape when C3 removed.

From the deformed shapes it is seen that no plastic hinges are failed that means the building has satisfied progressive collapse requirements per the (UFC 2005).

4.4 Analysis of the Building Model Using Nonlinear Dynamic Procedure.

The idea of removing a single column as a damage scenario adopted in various design requirements like UFC 2005 while leaving the rest of the building undamaged is unrealistic [21]. An explosive loading event near a building will cause extensive localized damage affecting more than a single column. Therefore, it is critical to assess accurately the behaviors of structural elements that were not removed from the building by the blast loads in their corresponding damaged states. This requires first a fully nonlinear blast–structure interaction analysis, then determining the state of the structural system at the end of this transient phase, and then proceeding with a fully nonlinear dynamic analysis for the damaged structure subjected to only gravity loads [21].

In this section the building is analyzed using three dimensional nonlinear analysis program (SAP2000). The building first analyzed using nonlinear static case as an initial state, since it is often necessary to start from a nonlinear static state when performing a nonlinear time-history analysis [20] . Then the building is analyzed using nonlinear direct integration time history analysis during external explosion events, and then the building is allowed for free vibration after the blast events. These successive steps of analysis simulate the real event, when an explosion may cause progressive collapse.

4.4.1 Modeling and Assumptions

The nonlinear dynamic analysis is done by a step-by-step integration of the equilibrium equations in the time domain. SAP2000 offers a number of standard direct integration methods. Newmark's method of numerical integration was used with gamma = 0.5 and beta = 0.25. With these parameters, the method is equivalent to the average acceleration method and is unconditionally stable with no energy dissipation. Geometric nonlinearity is considered by the selection of P-delta option 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. The range of important natural frequencies was identified during the modal analysis and was used to identify the two frequencies needed for SAP2000 to calculate Rayleigh damping coefficients. These coefficients were then used throughout the time history analyses. The maximum time step used was 0.001sec for all cases. The time step was usually automatically reduced by SAP2000 if the relative unbalance is greater than one.

Furthermore the following assumptions are made to simplify the analysis while illustrating the dynamic analysis procedure:

1. FEMA-356 [10] 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 adequately confined by shear reinforcement so that beams are not shear controlled.
4. The analysis is carried out using UFC load combinations, outlined in section 4.3.

4.4.2 Blast Loads on RC Frame

Load time history of explosive device for building and structural members may be calculated by dividing members in to sub sections and calculate a pressure time history for each small element [17]. 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 as shown in Figure 4-9. Each distributed load is a function of time .The blast load parameters i.e. Pressure, Time of arrival, Impulse and Load Duration are calculated using A.T.-Blast software as mentioned before in section 2.6.

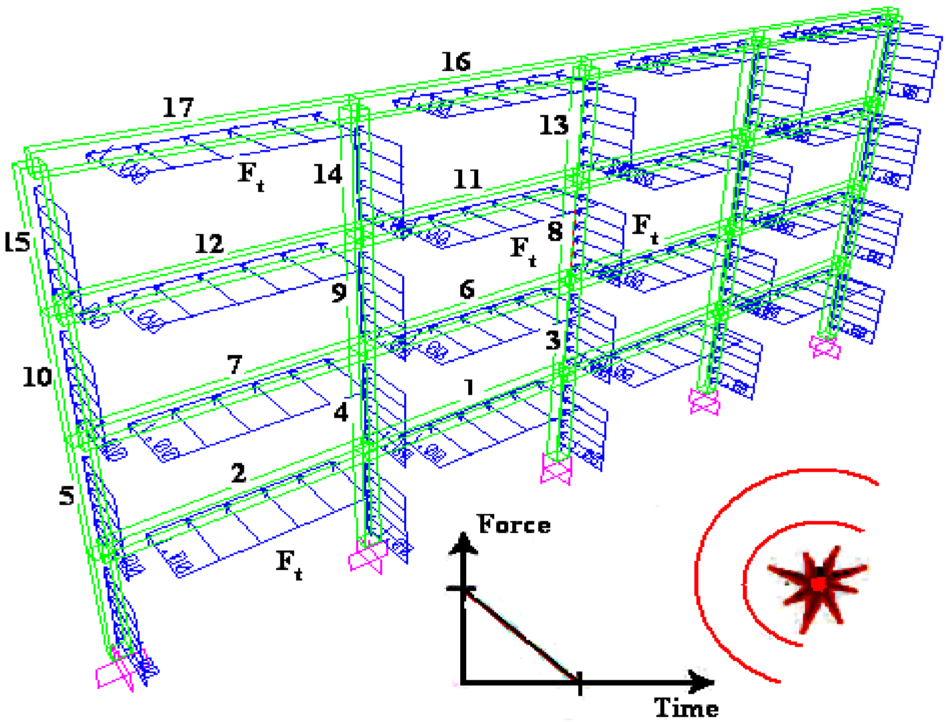


Figure 4-9: Distribution of the blast loads along the structural elements.

4.4.3 Blast Analysis Results of Case Study

An explosion yield of 250 lb (113.5 kg) TNT corresponding to a compacted truck is considered [24] . An Explosion is assumed to occur at 10m, 7m, 5m and 3m standoff distance from the center of a building. The blast loads parameters applied on the structure for each case are shown in Table 4-4, Table 4-5 , Table 4-6 and Table 4-7.

Table 4-4: Blast load parameters on structural elements for 10m standoff distance.

Element Number	Range (m)	Shock Velocity (m/msec)	Time of Arrival (msec)	Pressure (Mpa)	Impulse (Mpa-msec)	Load Duration (msec)
1	10.44	0.59	9.45	0.84	233.04	3.81
2	13.45	0.50	14.98	0.41	172.18	5.84
3	10.11	0.60	8.90	0.93	242.25	3.60
4	11.76	0.54	11.75	0.60	202.00	4.69
5	15.69	0.46	19.64	0.27	143.92	7.35
6	10.86	0.57	10.16	0.75	222.14	4.09
7	13.78	0.49	15.65	0.38	167.35	6.07
8	10.97	0.57	10.34	0.73	219.57	4.16
9	12.50	0.52	13.13	0.50	187.77	5.18
10	16.26	0.45	20.87	0.25	138.18	7.72
11	12.04	0.53	12.27	0.56	196.34	4.88
12	14.73	0.48	17.59	0.32	154.86	6.71
13	12.50	0.52	13.13	0.50	187.77	5.18
14	13.87	0.49	15.81	0.37	166.21	6.12
15	17.33	0.44	23.27	0.21	128.40	8.40
16	13.78	0.49	15.65	0.38	167.35	6.07
17	16.19	0.45	20.72	0.25	138.85	7.67

Table 4-5: Blast load parameters on structural elements for 7m standoff distance.

Element Number	Range (m)	Shock Velocity (m/msec)	Time of Arrival (msec)	Pressure (Mpa)	Impulse (Mpa-msec)	Load Duration (msec)
1	7.62	0.77	5.23	2.19	344.09	2.17
2	11.40	0.55	11.11	0.65	209.57	4.45
3	7.16	0.82	4.67	2.65	372.40	1.95
4	9.34	0.64	7.68	1.18	266.95	3.13
5	13.97	0.49	16.03	0.37	164.70	6.20
6	8.19	0.72	6.00	1.76	314.37	2.47
7	11.79	0.54	11.81	0.59	201.35	4.71
8	8.32	0.71	6.19	1.68	308.00	2.54
9	10.26	0.60	9.15	0.89	238.05	3.70
10	14.60	0.48	17.32	0.33	156.43	6.62
11	9.70	0.62	8.23	1.05	255.01	3.34
12	12.88	0.51	13.87	0.46	181.20	5.45
13	10.26	0.60	9.15	0.89	238.05	3.70
14	11.88	0.54	11.99	0.58	199.39	4.77
15	15.79	0.46	19.85	0.27	142.91	7.41
16	11.79	0.54	11.81	0.59	201.35	4.71
17	14.53	0.48	17.16	0.33	157.41	6.57

Table 4-6: Blast load parameters on structural elements for 5m standoff distance.

Element Number	Range (m)	Shock Velocity (m/msec)	Time of Arrival (msec)	Pressure (Mpa)	Impulse (Mpa-msec)	Load Duration (msec)
1	5.83	1.00	3.18	4.85	485.26	1.38
2	10.30	0.60	9.21	0.88	237.01	3.72
3	5.22	1.11	2.60	6.64	561.69	1.17
4	7.95	0.74	5.68	1.92	325.88	2.34
5	13.09	0.51	14.26	0.44	177.88	5.59
6	6.56	0.89	3.96	3.44	416.76	1.68
7	10.72	0.58	9.93	0.78	225.59	4.00
8	6.73	0.87	4.16	3.19	403.05	1.75
9	9.01	0.66	7.19	1.31	278.93	2.93
10	13.76	0.49	15.59	0.38	167.74	6.05
11	8.37	0.71	6.25	1.65	305.96	2.57
12	11.92	0.54	12.04	0.57	198.77	4.79
13	9.01	0.66	7.19	1.31	278.93	2.93
14	10.83	0.58	10.10	0.76	223.00	4.07
15	15.01	0.47	18.18	0.30	151.53	6.89
16	10.72	0.58	9.93	0.78	225.59	4.00
17	13.67	0.49	15.43	0.39	168.92	5.99

Table 4-7: Blast load parameters on structural elements for 3m standoff distance.

Element Number	Range (m)	Shock Velocity (m/msec)	Time of Arrival (msec)	Pressure (Mpa)	Impulse (Mpa-msec)	Load Duration (msec)
1	4.24	1.36	1.79	11.53	743.59	0.89
2	9.49	0.64	7.91	1.12	261.87	3.21
3	3.35	1.68	1.19	20.17	1032.42	0.71
4	6.87	0.85	4.33	2.99	392.25	1.82
5	12.46	0.52	13.05	0.50	188.51	5.16
6	5.20	1.12	2.57	6.72	564.95	1.16
7	9.95	0.61	8.64	0.97	247.09	3.50
8	5.41	1.07	2.77	6.02	536.12	1.23
9	8.08	0.73	5.85	1.84	319.70	2.41
10	13.16	0.50	14.41	0.43	176.68	5.64
11	7.35	0.80	4.90	2.44	360.08	2.04
12	11.22	0.56	10.80	0.68	213.52	4.33
13	8.08	0.73	5.85	1.84	319.70	2.41
14	10.06	0.61	8.82	0.94	243.66	3.57
15	14.47	0.48	17.04	0.33	158.18	6.53
16	9.95	0.61	8.64	0.97	247.09	3.50
17	13.08	0.51	14.24	0.44	178.04	5.58

After performing a sequential nonlinear static, nonlinear direct integration time history and free vibration analysis for each case the final deformed shapes are as shown in Figure 4-10, Figure 4-11, Figure 4-12 and Figure 4-13.

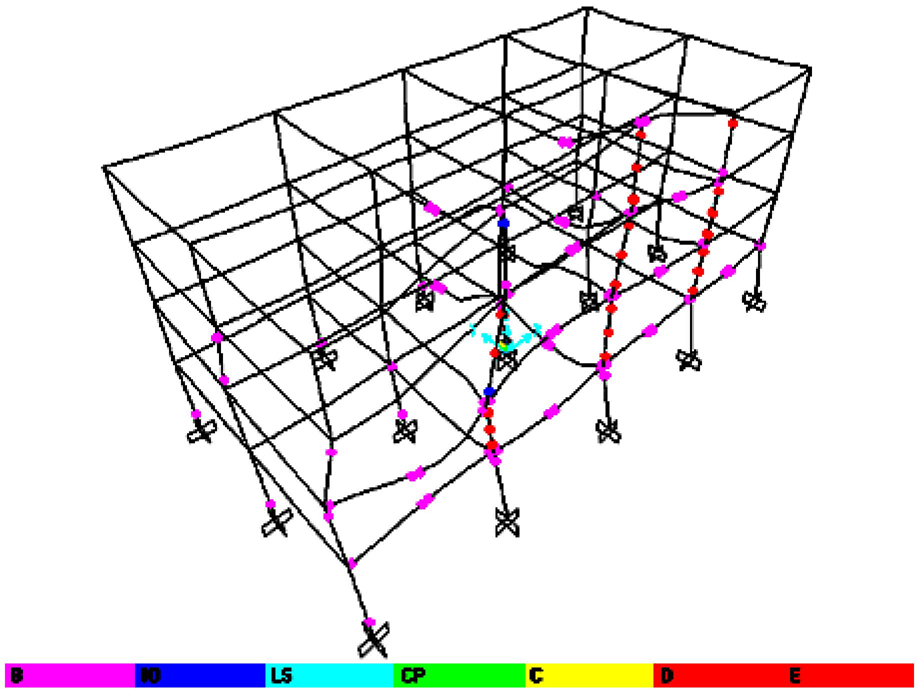


Figure 4-10: Final stage hinges and deformed shape for blast analysis at 3m standoff distance.

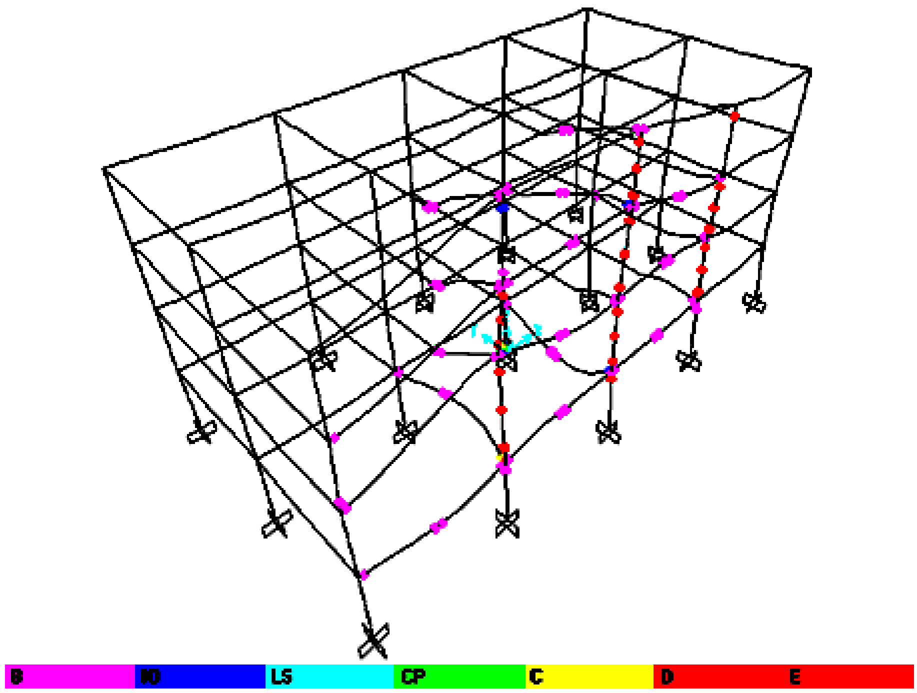


Figure 4-11: Final stage hinges and deformed shape for blast analysis at 5m standoff distance.

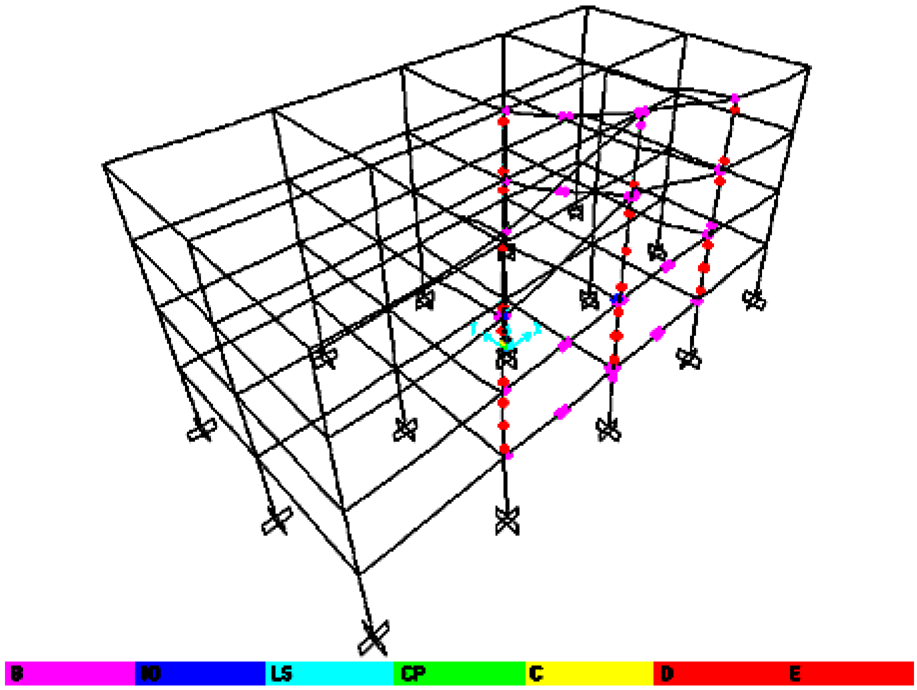


Figure 4-12: Final stage hinges and deformed shape for blast analysis at 7m standoff distance.

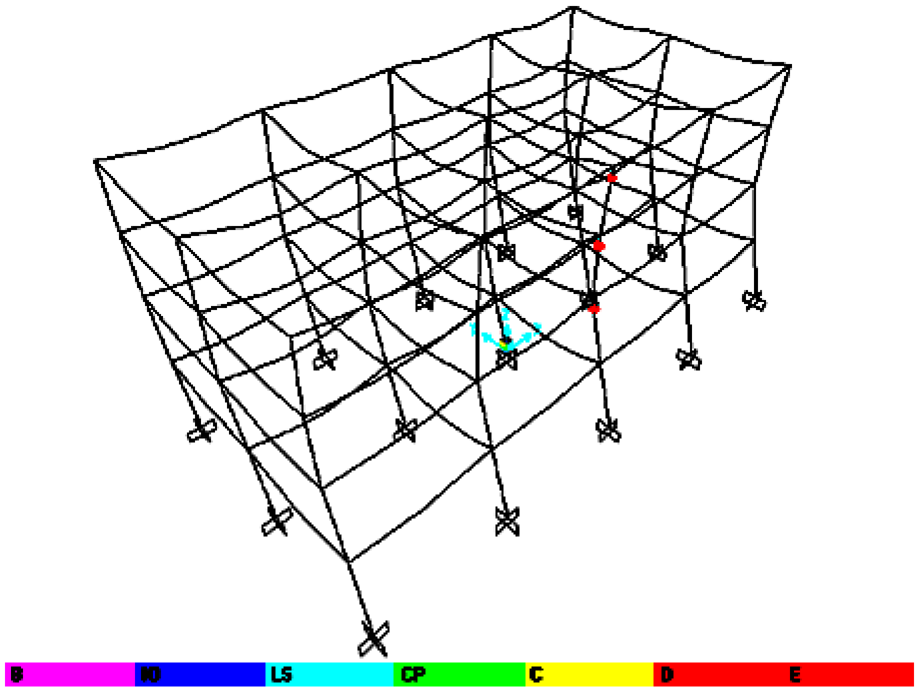


Figure 4-13: Final stage hinges and deformed shape for blast analysis at 10m standoff distance.

From the deformed shapes it is observed that the building is susceptible to progressive collapse from detonation of 250lb charge weight at 3m, 5m and 7m standoff distances. However at 10m standoff distance the building is safe to resist blast induced progressive collapse.

4.5 Single-Degree-Of-Freedom (SDOF) Analysis of Building Component Response to Blast Load

While real structures have infinite degrees of freedom, it is often advantageous to model a structure (usually specific structural elements) as a single-degree-of freedom (SDOF) system. Also it is necessary to consider the time and cost of the analysis when choosing design procedures. Because the design process is a sequence of iterations, the cost of analysis must be justified in terms of benefits to the project and increased confidence in the reliability of the results. Although SDOF may not adequately describe the detailed response of the structure, SDOF formulation can provide a rapid and easy solution and often gives designer valuable (but limited) information about dynamic characteristics (fundamental frequency, maximum response, etc.) of the system[21].

In this section SDOF design approach is implemented on a chosen structural element which is already analyzed using SAP2000. A central exterior RC column from the previous building model is subjected to blast loading and analyzed using a Computer Program written in VBA for nonlinear SDOF systems. In the Program inelastic material behavior and dynamic loading are considered in solving the dynamic equation. Finally the acceptability of SDOF approach is verified by comparing numerical solution results of SDOF approach with SAP2000 outputs.

To simplify the analysis for the selected column element shown in Figure 4-14 , while illustrating the analysis procedure, the following properties and assumptions are made:

1. The column is fixed at both end and modeled as two-dimensional.
2. Effects of large deflections are neglected.
3. A *Smooth Resistance-Deflection* function is adopted from member analysis using Response-2000 [19].
4. Equivalent structural damping of 5% is used throughout the analysis.
5. A triangular blast loads are assumed to be distributed on the element face [17].

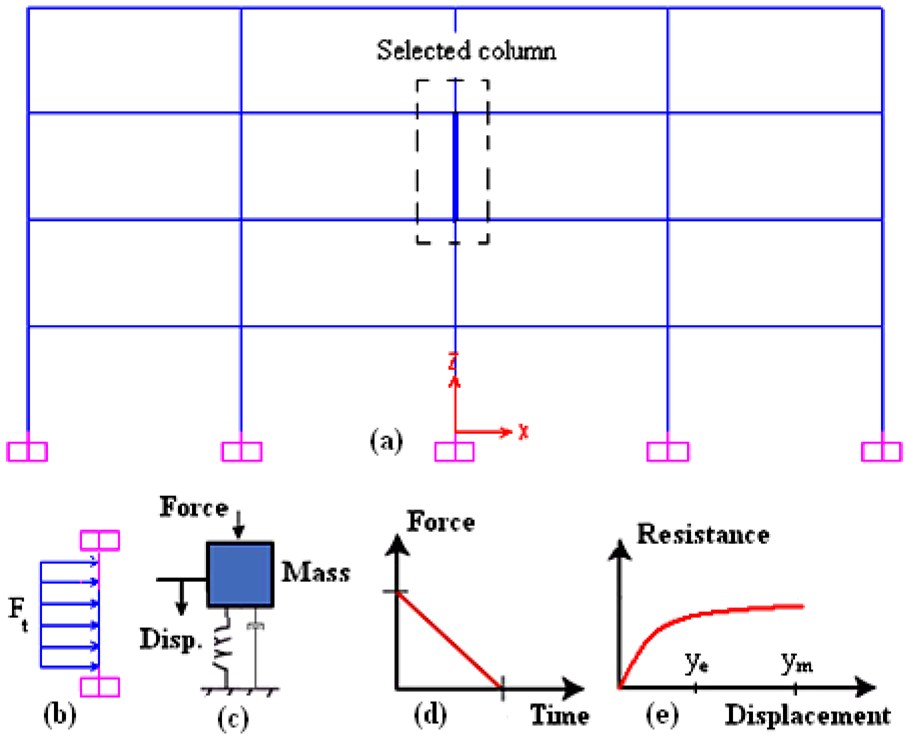


Figure 4-14: Equivalent SDOF model for dynamic analysis (a)RC frame with selected column (b)Fixed end column with uniformly distributed load (c) Damped SDOF system (d) Applied blast force (e) Resistance function

4.5.1 Design Loads and Resistance Function

The design loads for each case is taken from that calculated in section 4.4.3 and summarized in Table 4-8 for the selected column. In order to determine the dynamic response of a structural member using SDOF method, it is necessary to develop resistance –deflection function. The Resistance-Deflection relation for the column member is analyzed using Response-2000 software for its accuracy and easy feasibility. Response-2000 considers both material and geometric nonlinearity so that it is possible to consider P-M interaction of the section. The load maximum relation result of Response-2000 for given section is shown Figure 4-15 .After getting the load and resistance function the mass and the resistance functions are multiplied by mass and load factors (usually 0.72 for fixed end members) which estimate the actual portion of the mass or load participating in the deflection of the member along its span.

Table 4-8: Blast load parameters on design column for 3m,5m,7m and 10m standoff distance

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	0.71	6.19	1.68	308.00	2.54
5m	0.87	4.16	3.19	403.05	1.75
3m	1.07	2.77	6.02	536.12	1.23

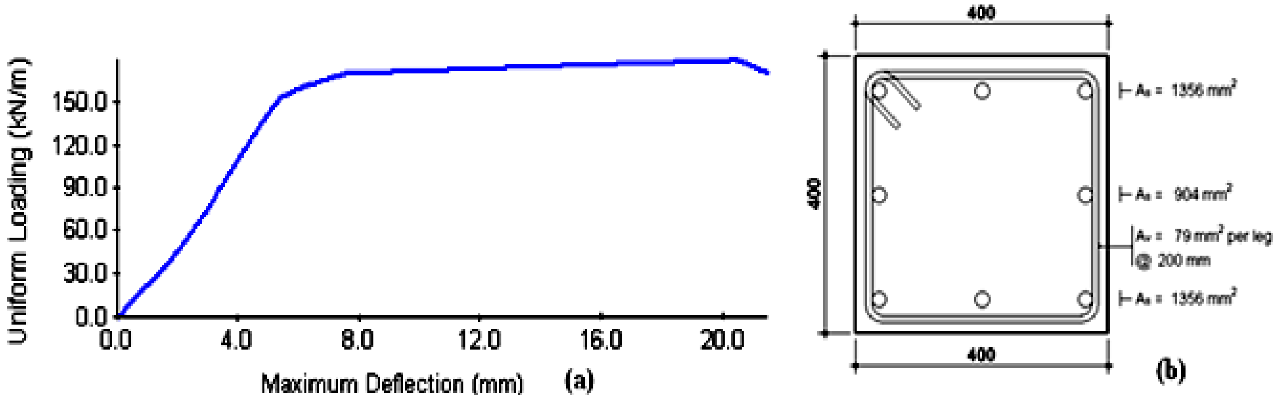
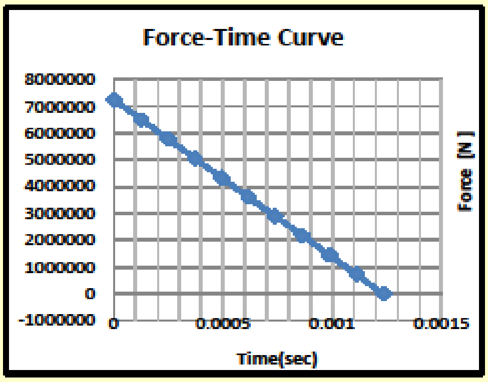


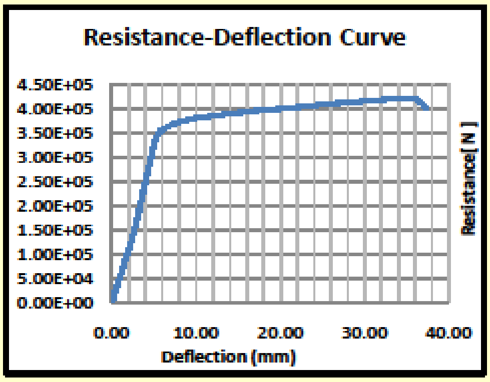
Figure 4-15: Load – Maximum deflection relation for column section [19]

4.5.2 SDOF Blast Analysis Results

After performing SDOF blast analysis the dynamic responses (Deformation, Velocity and Acceleration) are determined for each case. Figure 4-16 shows the interface of the program for the first case input data i.e., 250 lb TNT at 3m standoff distance. Besides giving numerical integration solutions, the Force-Time and Resistance-Deflection curve are shown clearly in the interface. For brevity only deformation time history results are shown in graphical form in Figure 4-17, Figure 4-18, Figure 4-19 and Figure 4-20 for each case. (See appendix for the source code of the program.)

NONLINEAR SDOF BLAST ANALYSIS





INPUT

Mass	10^3 (kg)	0.82944
Elastic stiffness	(kN/m)	55024.54
Damping Coefficient	%	5
Time Increment Δt	(sec)	
Diflection Increment Δl		0.1

Resistance-Deflection Input		Force-Time history 1	
Deflection	Resistance	Time	Force
(mm)	(N)	(sec)	(N)
0.00	0.00E+00	0	7226130.00
0.38	2.11E+04	0.001234	0
0.77	4.21E+04		
1.16	6.32E+04		
1.55	8.43E+04		
1.94	1.05E+05		
2.30	1.26E+05		
2.65	1.48E+05		
2.95	1.69E+05		
3.23	1.90E+05		
3.50	2.11E+05		
3.77	2.32E+05		
4.04	2.53E+05		
4.31	2.74E+05		
4.59	2.95E+05		
4.85	3.16E+05		
5.21	3.37E+05		

OUTPUT

Time	Deflection	Velocity	Acceleration
(sec)	(mm)	(mm/sec)	(mm/sec^2)
0.0000	0.000	0.000	2170138.889
0.0006	0.534	2429.002	*4656521.767*
0.0011	2.637	*3934.196*	261723.982
0.0017	4.527	3678.207	-602888.528
0.0022	6.680	3302.671	-593216.080
0.0028	8.238	3011.012	-585703.973
0.0034	9.984	2651.758	-576450.867
0.0039	11.223	2368.836	-569163.818
0.0045	12.577	2020.235	-560185.079
0.0050	13.506	1745.338	-553104.714
0.0056	14.478	1406.623	-544380.640
0.0062	15.243	1073.280	-535794.912
0.0067	15.708	810.416	-529024.463
0.0073	16.108	486.527	-520682.253
0.0079	16.285	231.119	-514103.835
0.0084	*16.330*	-83.583	-505982.407
0.0090	16.227	-331.705	-499340.121
0.0095	15.928	-637.161	-490739.387
0.0101	15.554	-877.643	-483628.732
0.0107	14.921	-1173.229	-474463.976
0.0112	14.285	-1405.570	-466919.348
0.0118	13.329	-1690.684	-457233.785
0.0123	12.199	-1969.739	-447277.684

Figure 4-16: Nonlinear SDOF Blast Analysis Interface

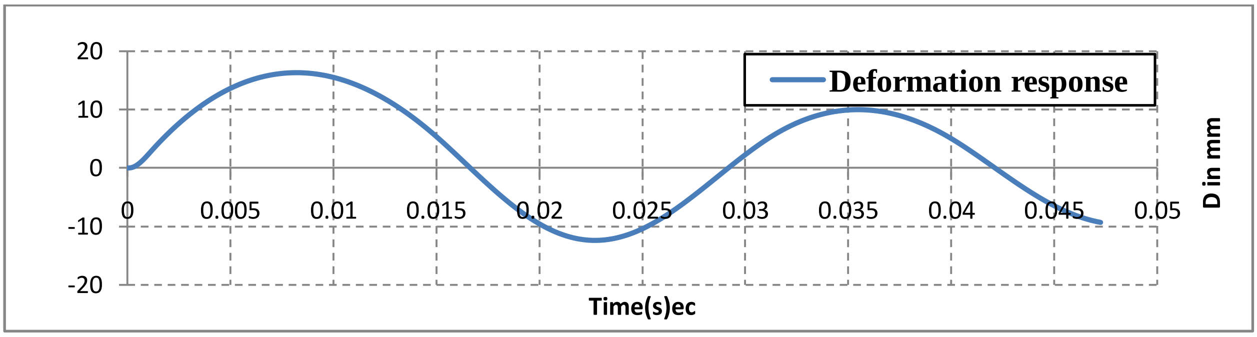


Figure 4-17: SDOF Blast Analysis Deformation Response for 3m standoff distance

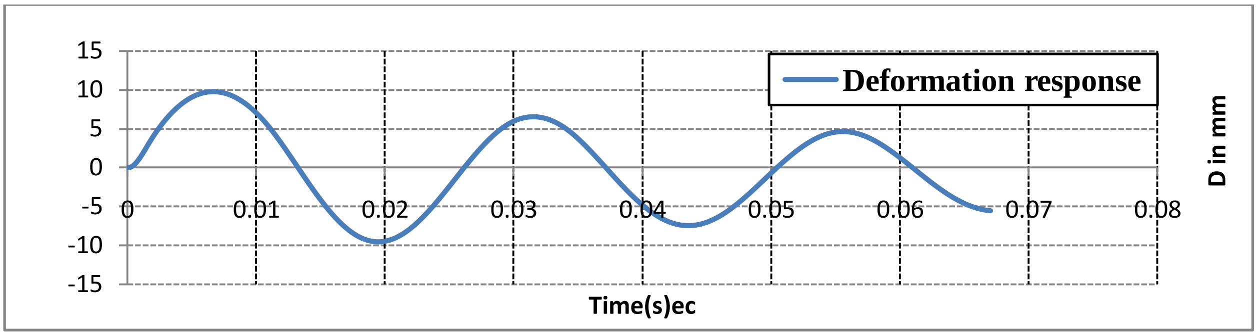


Figure 4-18: SDOF Blast Analysis Deformation Response for 5m standoff distance

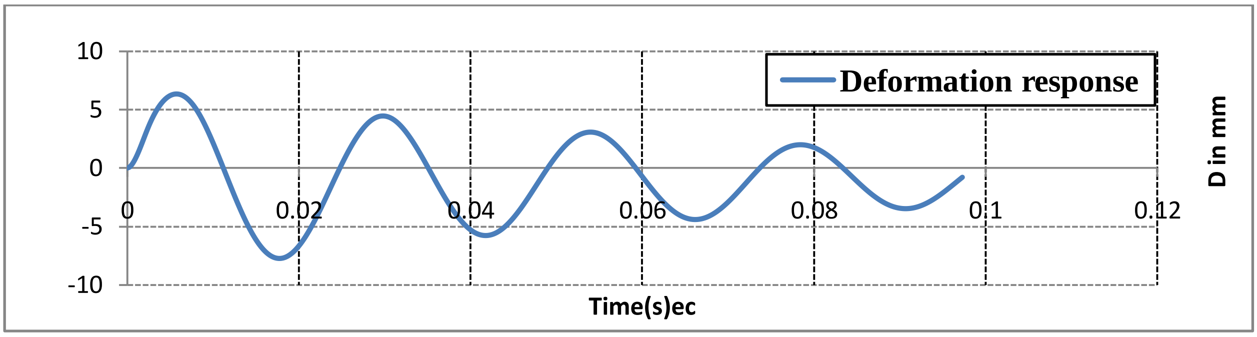


Figure 4-19: SDOF Blast Analysis Deformation Response for 7m standoff distance

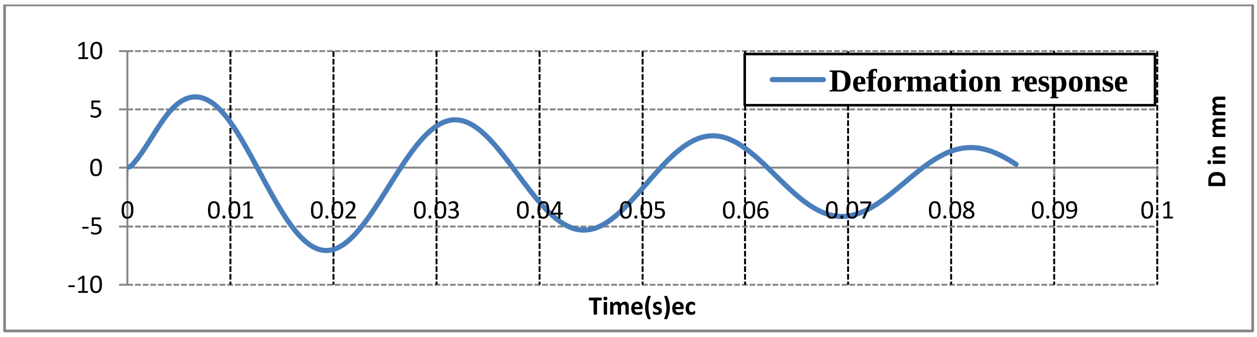


Figure 4-20: SDOF Blast Analysis Deformation Response for 10m standoff distance

4.5.3 Comparison with SAP2000 Result.

The maximum displacement response is calculated using nonlinear direct integration procedure using SAP2000 application software and SDOF approach for different cases. Table 4-9 and Figure 4-21 shows the value of each method and the difference ratio. The difference ratio raw refers to the percentage variation of the two analyses and is calculated as given below:

$$\text{Difference ratio } (\%) = \frac{|y_{max\ SAP2000} - y_{max\ SDOF}|}{y_{max\ SDOF}} \tag{13}$$

Table 4-9: Maximum displacement and difference ratio

Analysis Program used.	Standoff distance			
	10m	7m	5m	3m
SAP2000 Maximum displacement [mm]	6.97	9.82	12.74	47.8
SDOF Maximum displacement [mm]	6.96	7.66	9.77	16.33
Difference ratio	0.08%	28.12%	30.36%	192.71%

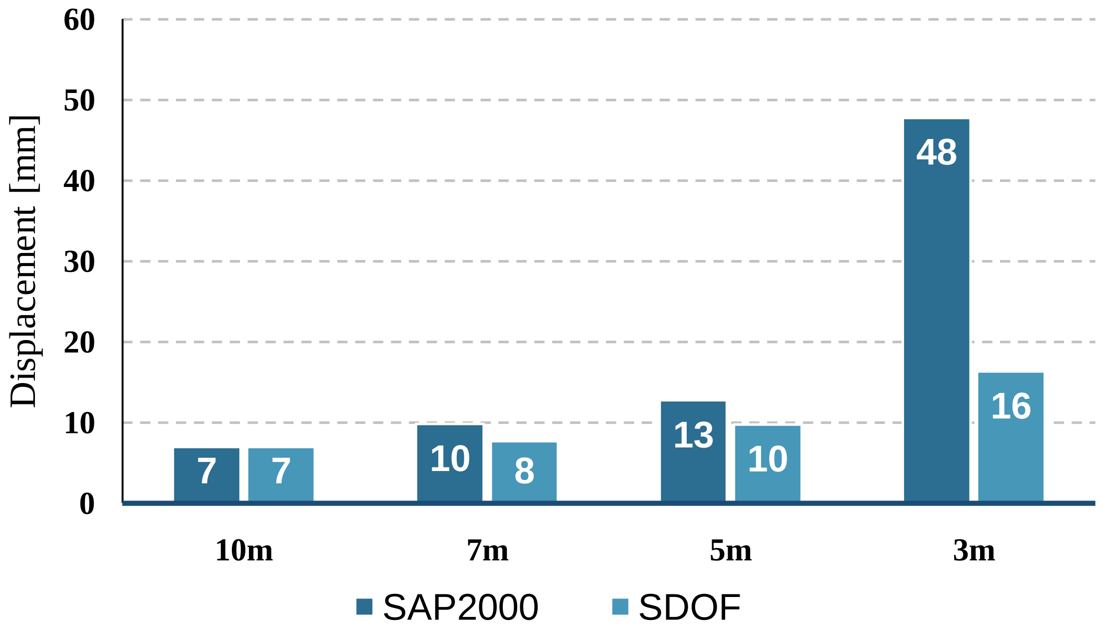


Figure 4-21: SAP2000 and SDOF system maximum displacement comparison

From Table 4-9 it's observed that as standoff distance is increased SDOF blast analysis gives good result compared to SAP2000 output. The difference ratio for the 5m standoff distance is about 30 %, while for 10m the difference is close to zero.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

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

In this paper it is tried to investigate the dynamic response and damage of RC framed structure under external blast loading using recommended procedures and code provisions. Blast analysis was performed based on sequences of nonlinear analysis using SAP2000 and SDOF approximation technique at different standoff distances. Based on the nonlinear dynamic analysis carried out on RC frame building the following observations and conclusions can be drawn:

1. Buildings designed according to the (EBCS 8) guidelines did not satisfy the requirements of (UFC 2005).
2. Buildings designed according to the requirements of (UFC 2005) are able to resist a blast load of 250lb at a standoff distance of 10m. But the design is unsafe when the same explosion may happen at 7m. Therefore, it is necessary to have limited use for (UFC2005), when the blast is the cause of the probable progressive collapse.
3. Design considerations against extreme events (bomb blast, high velocity impact) are very important. It is recommended that guidelines on abnormal load cases and provisions on progressive collapse prevention should be included in the current Ethiopian Building Code Standard.

A nonlinear computer program was developed to analyze SDOF systems subjected to blast loading .This program was used to compute dynamic response of a typical column subjected to blast loading at selected standoff distances. The maximum deflection results for each scenario are compared with the values obtained from the finite element package SAP2000. Based on the nonlinear SDOF blast analysis carried out on RC column the following conclusions can be drawn:

1. SDOF blast analysis gives good result for explosion at 10m and 7m distance. But as the standoff distance decreases the approximation also diminishes.

2. SDOF approach is very attractive in supporting the design activities because it requires less computational resources. One can assess very quickly a tentative design, modify it, and reanalyze it within a very short time.

In general SDOF approach can be used for the preliminary design, once the preliminary design is done; one may use a more sophisticated approach for the final verification of the design to obtain high accuracy and more information.

5.2 Recommendation

The following recommendations are suggested for future work:

1. Development of methods available for evaluating of blast load parameters in order to find a high accuracy method.
2. Application of the nonlinear dynamic analysis to different types of structures i.e. different bay length, different number of stories, etc.
3. To check the numerical solution from analytical method with experimental investigations.

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Appendix

```
'***************************************************************************************
                                        'INPUT PART
'***************************************************************************************
Sub RUNNSDOF( )

   Dim M, T, Da, N, LT, Dl, gama, beta, Kt, Dt, Tno, TMax, RMax, y, a1, a2, a3, a4 As Double
   Dim p(1000, 2), u(1000), uu(1000), uuu(1000), pk(1000), k(1000), f(1000) As Double
   Const pi As Double = 3.141592654
   'On Error GoTo ErrorHandler
   M = Worksheets("Sheet1").Cells(20, 4):            k(0) = Worksheets("Sheet1").Cells(21, 4)
   Da = (Worksheets("Sheet1").Cells(22, 4)) / 100: LT = Worksheets("Sheet1").Cells(28, 3)
   Dl = Worksheets("Sheet1").Cells(24, 4):           x1 = Worksheets("Sheet1").Cells(28, 3)
   x2 = Worksheets("Sheet1").Cells(29, 3):           y1 = Worksheets("Sheet1").Cells(28, 4)
   y2 = Worksheets("Sheet1").Cells(29, 4):           xmax = Worksheets("Sheet1").Cells(45, 3)
   ymax = Worksheets("Sheet1").Cells(45, 4):        TMax = Worksheets("Sheet1").Range("A50")
   RMax = Worksheets("Sheet1").Range("B50")

   'READ FROM THE GIVEN FILE
If Worksheets("Sheet1").Range("C25") = "Force-Time history 1" Then
   y = 100
   Dt = xmax / 10
   Worksheets("Sheet1").Cells(23, 4) = Dt
   For I = 0 To 10 Step 1
      Worksheets("Sheet1").Cells(8 + I, 28) = (x2 / 10) * I
      p(I, 1) = (x2 / 10) * I
      Worksheets("Sheet1").Cells(8 + I, 29) = ((y2 - y1) / (x2 - x1)) * p(I, 1) + y1
      p(I, 2) = ((y2 - y1) / (x2 - x1)) * p(I, 1) + y1
   Next I
   For I = 0 To 90 Step 1
      p(11 + I, 1) = x2 + I * Dt: p(11 + I, 2) = 0
      Worksheets("Sheet1").Cells(19 + I, 28) = x2 + I * Dt
      Worksheets("Sheet1").Cells(19 + I, 29) = 0
   Next I
End if

For I = 0 To 22 Step 1
   Worksheets("sheet1").Cells(22 + I, 6) = ((x2 + 90 * Dt) / 22) * I
Next I
```

```
'*******************************************************************************
                                  'ANALYSIS PART
'*******************************************************************************
    u(0) = 0
    uu(0) = 0
    pk(0) = Worksheets("Sheet1").Cells(30, 4)
    f(0) = Worksheets("Sheet1").Cells(30, 4)
    uuu(0) = (pk(0) - Da * uu(0) - k(0) * u(0)) / M
    a1 = 3 / Dt
    a2 = 6 / Dt
    a3 = Dt / 2
    a4 = 6 / (Dt ^ 2)
    'Coefficients
    wn = (k(0) / M) ^ 0.5
    T = (2 * 3.141592654) / wn
    C = 2 * M * wn * Da
    'Stiffness
    For x = 0 To (TMax + 1) Step Dl
        y = (1 / Dl) * x
        Worksheets("Sheet1").Cells(7, 19) = x
        Worksheets("Sheet1").Cells(9 + y, 19) = x
        R(y) = Worksheets("Sheet1").Cells(7, 20)
        Worksheets("Sheet1").Cells(9 + y, 20) = R(y)
        If x <= 0 Then GoTo 3:
        k(x) = (R(y) - R(y - 1)) / Dl
3:      Ki = k(x)
        Worksheets("Sheet1").Cells(9 + y, 21) = Ki
    Next x
    For x = 0 To y
        Worksheets ("Sheet1").Cells (8, 19) = u(x)
        If u(x) < 0 Then
            Ki = k(0)
        Else
            Ki = Worksheets("Sheet1").Cells(8, 21)
        End If
        Kt = Ki + (a4 * M + a1 * C)
        pk(x) = (p(x + 1, 2) - p(x, 2)) + (a2 * M + 3 * C) * uu(x) + (3 * M + a3 * C) * uuu(x)
        deu = pk(x) / Kt
        deuu = (3 * deu / Dt) - 3 * uu(x) - 0.5 * Dt * uuu(x)
        f(x + 1) = f(x) + Ki * deu
        If f(x + 1) >= RMax Then f(x + 1) = RMax
        u(x + 1) = u(x) + deu
        uu(x + 1) = uu(x) + deuu
        uuu(x + 1) = (p(x + 1, 2) - C * uu(x + 1) - f(x + 1)) / M
```

```
        Worksheets("sheet1").Cells(8 + x, 23) = Dt * x
        Worksheets("sheet1").Cells(8 + x, 24) = u(x)
        Worksheets("sheet1").Cells(8 + x, 25) = uu(x)
        Worksheets("sheet1").Cells(8 + x, 26) = uuu(x)
    Next x
Exit Sub

    'ERROR HANDLER
ErrorHandler:
    Select Case Err.Number
        Case 53, 76
            MsgBox "File or Path not found. Try Again."
        Case 71
            MsgBox "The drive door might be open - please check."
        Case 13
            MsgBox "Type Mismach, Check the inputs you gave."
        Case Else
            Err.Raise Err
    End Select
End Sub

Function Linterp(R As Range, x As Double) As Double
    ' linear interpolator / extrapolator
    ' R is a two-column range containing known x, known y
    Dim lR As Long, l1 As Long, l2 As Long
    Dim nR As Long
    nR = R.Rows.Count
    If nR < 2 Then Exit Function

    If x < R(1, 1) Then ' x < xmin, extrapolate
        l1 = 1: l2 = 2: GoTo Interp
    ElseIf x > R(nR, 1) Then ' x > xmax, extrapolate
        l1 = nR - 1: l2 = nR: GoTo Interp
    Else
        ' a binary search would be better here
        For lR = 1 To nR
            If R(lR, 1) = x Then ' x is exact from table
                Linterp = R(lR, 2)
                Exit Function
            ElseIf R(lR, 1) > x Then ' x is between tabulated values, interpolate
                l1 = lR: l2 = lR - 1: GoTo Interp
            End If
        Next
End If
```

```
Interp:
    Linterp = R(l1, 2) _
            + (R(l2, 2) - R(l1, 2)) _
            * (x - R(l1, 1)) _
            / (R(l2, 1) - R(l1, 1))
End Function
```

Declaration

I hereby declare that the work which is being presented in this thesis entitles "**Blast Loading and Blast Effects on RC Frame Buildings**" is original work of my own, has not been presented for a degree in any other university and that all sources of material used for the thesis have been duly acknowledged.

-- --------------------------

Abdulaziz Kassahun Date

(Candidate)

This is to certify that the above declaration made by the candidate is correct to the best of my knowledge.

-- --------------------------

Dr. Shifferaw Taye Date

(Thesis Advisor)