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Torsional Response of Reinforced Concrete Frame Buildings Subjected to Blast Loading

By

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Thesis submitted to the Faculty of the Graduate and Postdoctoral Studies
in Partial fulfillment of the requirements for the M.A.Sc. degree
in Civil Engineering

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Dedicated to my family

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Abstract

This study is intended to contribute to the understanding of the torsional behaviour of buildings subjected to blast loading. The scope of the investigation involves a 10-storey symmetrical reinforced concrete frame structure designed in accordance with CSA Standard A23.3 (2004), and the provisions of the National Building Code of Canada (2005). The building was analyzed under different magnitudes of explosions, triggered at different distances and location such that the building would be subjected to lateral impulsive forces causing torsion. Elastic dynamic time-history analyses were conducted using software ETABS under impulsive forcing functions caused by the detonations of 100 kg, 200 kg, 300 kg, 500 kg, and 1000 kg TNT at distances of 5 m, 10 m, and 20 m from the building, at three different eccentricities within the plane of building floor to create torsional eccentricities relative to the centre of rigidity. The performance of structure was evaluated by considering interstorey drift, floor rotations, lateral displacements, and P-M capacity/demand ratios for columns.

The results indicate that the perimeter columns, especially the corner columns are affected most when the building is subjected to blast loadings. The torsional building response increases with the amount of TNT. It is related to the location of the explosion relative to the building, and increases with torsional eccentricity. On the other hand, the torsional response decreases with distance from explosion to building. Maximum interstorey drifts are closely related to the eccentricity of blast loads. This effect can be neglected in practical applications for small size charges and distant explosions. For example, an explosion caused by less than 500 kg TNT at distances of 20 m and longer would produce very small interstorey drifts and the effects of the eccentricity of blast loads could be neglected in such buildings.

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Chapter 1

Introduction

1.1 Background

In today's politically turbulent environment many embassies, commercial centers, and buildings (government, office and residential buildings) have become targets for terrorist attacks that are often executed by explosives. This necessitates the development of design information for blast resistant buildings.

Terrorist attacks can be divided into three classes; i) ballistic attacks, ii) explosions, and iii) forced-entry. Among them explosive attacks are of considerable interest due to their potentially devastating consequences. Explosive attacks can be realized by different methods, including external vehicle attacks (car or truck bombs), stand-off weapon attacks, and suicide or parcel bombs. The external vehicular bomb attacks have become a common technique employed by terrorists in recent years because it is easy to deliver a large quantity of explosives (as much as the load capacity of vehicle) to targeted buildings which can be detonated relatively easily. In November 1995, a car bomb exploded in Riyadh, Saudi Arabia, killing five people in the offices of Saudi Arabian National Guard. The spectacular car bombing of the World Trade Center in New York City on February 26, 1993 resulted in 6 deaths and many injures. The most notable car bomb attack was executed on the Murrah Building in Oklahoma City on April 19, 1995,

killing 168 people and resulting in approximately 50 million dollars of damage. Detonations caused by such car-bombs, placed outside of buildings, may result in catastrophic structural collapses. In contrast to seismic and wind loads, blast loads occur within milliseconds, generating impulsive dynamic loads. Concerns of the structural engineering community include not only the complexities of blast pressures but also the structural response of buildings when subjected to blast loads.

Compared to the extensive previous research available on seismic and wind load effects on structures, much less research has been done on the behaviour of structures subjected to blast loading. With increased awareness of blast hazards on critical infrastructure, increased number of research programs have begun with the objectives of developing blast resistant design methodologies for new buildings, while developing similar guidelines for retrofitting existing buildings. The majority of research findings on blast loading remain to be classified for military use. However, some useful information is available in the literature to formulate blast pressures as a function of time.

The majority of current research on blast performance of building structures involves the investigation of charge-standoff distance relationships through planar structural analysis. Another concentration of research topic is the investigation of progressive collapse triggered by a column loss. However, there is lack of research on torsional response of buildings resulting from the eccentricity of lateral impulsive forces relative to the centre of rigidity of buildings. In a reinforced concrete structure experiencing torsional motion, the combination of axial loads and twisting moments may

generate instability in columns. Furthermore, the combined effects of shear and torsion may result in diagonal tension distress which may result in a brittle failure. Therefore, torsional effects are very important for safety and serviceability of structures. Because bomb detonations can occur in any arbitrary location within a three dimensional building layout, it is conceivable that a significant torque may be imposed on the structure. This aspect of dynamic response needs to be studied before a rational design approach can be developed for building structures.

1.2 Objective and Scope

The objective of the current research program is to investigate the effects of blast-induced torsion on structural response. More specifically, the objective includes the investigations of;

- the significance of torsional effects on reinforced concrete frame buildings subjected to blast pressures.
- the relationships between torsional response and explosive charge, as well as the location of explosion (i.e. frontal and lateral distance of explosion centre relative to the building).

The scope includes;

- Literature review
- Selection of a typical 10-storey reinforced concrete frame building.
- Modelling of the selected building and the impulsive blast pressures.

- Elastic, dynamic time history analysis.
- Parametric investigation under variables that consist of charge, standoff distance and level of eccentricity of blast loading.
- Presentation of results in the form of a thesis.

1.3 Literature Survey

Most of the research related to the effects of bomb blasts on structures is conducted for military purposes. The documents and computer programs generated by this research are classified and restricted for military use only. Therefore, the work presented in this thesis is primarily based on unrestricted government documents and publications from non-military sources. The most important references used in this study are presented briefly hereafter.

The information in Manual 42 (ASCE 1985), in TM 5-1300 (U.S. Department of the Army 1990), and in TM 5-855-1 (U.S. Department of the Army 1986) provides background material on structural consideration and design for blast loading. The works of Mays and Smith (1995), the National Research Council (1995), and Smith and Hetherington (1994) address the general concepts that pertain to weapons, blast effects on buildings, and protection against such events. The protection of buildings against vehicle bomb attacks is specifically discussed by Longinow and Mniszewski (1996).

Hopkinson (1915) and Cranz (1926) formulated the so called “cube-root scaling” relationship between distance and blast charge (i.e., weight of explosive). This

relationship, which is referred to in the literature as the Hopkinson-Cranz relationship, is commonly used for determining the properties of the waves from explosions at different distances and different explosive weights. The Hopkinson-Cranz relationship has been thoroughly verified by many experiments conducted for a large range of explosive charge energies. Baker (1973) summarizes the derivation of laws for scaling of blast wave properties and demonstrates its applicability.

Compiling and analyzing the data resulting from tests in free-air and hemispherical TNT surface bursts, Kingery and Bulmash (1984) developed a set of airblast curves for the positive-phase blast parameters. These curves were published in the TM5-1300 Manual entitled "Structures to resist the effects of accidental explosions," prepared by the U.S. Departments of Army, Navy and Air Force (1990) which can be used in defining the pressure-time relationships for different types of explosive detonations. In addition to research on blast load characterization, significant research was conducted on various aspects of blast effects on buildings. Mays and Smith (1995) edited a book on blast effects on buildings discussing the characteristics of blast loading, structural response to blasts, and basic guidelines for enhancing resilience of reinforced concrete and steel buildings. Gross and McGuire (1983), and Similowitz (2003) presented basic principles of progressive collapse resistant design, illustrating a method of analysis for multistory frame buildings that may suffer from loss of some elements due to blast effects. Similarly, another analysis technique for blast triggered progressive collapses was discussed by Choi and Krauthammer (2003). These researchers illustrated the use of computer

software employed for progressive collapse analysis with potential consequences of losing vertical structural elements in multistory buildings.

Biggs (1964) introduced the principles of dynamic structural analysis under blast-induced impulsive loads and structural design against such loads. In addition to the foregoing literature, documents such as “Structural design for physical security, state of the practice” (ASCE, 1999), and “Protection of buildings from blast effects, a state-of-the-art report” (Naumoski and Balazic, 2004) summarize general methods, guidance, and references for bomb blasts. These publications also discuss the characteristics of blast loads, the formulations of blast wave parameters, and the calculation of loads on different faces of buildings due to bomb explosions.

Lack of sufficient research on blast-resistant design of structures forced researchers to identify similarities with earthquake-resistant design. A workshop was organized on the application of seismic rehabilitation technologies to mitigate blast-induced progressive collapse jointly by the National Institute of Standards and Technology (NIST) and the General Services Administration (GSA) of U.S. to generate technical information that compared blast and earthquake effects on buildings (Carino et al. 2001). The workshop resulted in an overview of seismic design approaches that may be adopted for blast-resistance. Subsequently, the Multihazard Mitigation Council of the National Institute of Building Sciences organized a workshop which resulted in recommendations for further research on the prevention of progressive collapse triggered by blast (Burns et al. 2003, Dusenberry 2003, Crawford 2003). These workshops generated technical

information on the potential use of seismic design procedures which have matured over the last three decades, for blast-resistant design with a view of improving inelastic building response through structural integrity and deformability.

In addition to the above research publications, a number of standards and guidelines have addressed blast performance and blast-resistant design of structures, including the requirements for blast-induced progressive collapse. The U.S. Department of Defence (DOD) produced minimum antiterrorism standards for buildings (DOD 2002). Similarly, GSA produced facilities standards for the public building service (GSA 2005). Furthermore, GSA issued guidelines for progressive collapse analysis and design, addressing security issues for U.S. government buildings (GSA 2003). The required loads and load combinations for building designs, including those for progressive collapse analysis, were developed by the American Society of Civil Engineers and published as ASCE 7-02 document (ASCE 2002). The U.S. Federal Emergency Management Agency (FEMA) published a reference manual to mitigate potential terrorist attacks against buildings (FEMA 386-7 2003, FEMA 426 2003), covering risks associated with such attacks, the characteristics of such blasts and designing sites and buildings against blast effects. FEMA also published a primer to design safe school projects (FEMA 428 2003) and commercial buildings (FEMA 427 2003) in case of terrorist attacks.

Another useful reference for this study is the research paper published by Ozbakkaloglu, Naumoski, and Saatcioglu (2005). They investigated the responses of a 10-storey reinforced concrete frame building due to blast loads from explosions at

different distances from the building with different weights of explosives. Two dimensional time-history analysis was used in the study, i.e. no torsional effects were considered. This is done in the current investigation which represents a continuation of the research conducted by Ozbakkaloglu, Namouski and Saatcioglu (2005).

1.4 Outline of Thesis

The thesis consists of 5 chapters. Chapter 1 provides introduction, while identifying research needs, presenting the objectives and scope of research and also providing literature survey. Chapter 2 provides the definitions and characteristics of parameters that define the blast load, while also presenting the details of the building selected for investigation. Chapter 3 presents the details of structural analyses conducted and the specifics of the blast load scenarios considered. Chapter 4 provides the analysis results and major observations made during analyses under different explosion scenarios. The chapter also includes discussions on the effects of the location of explosion, the distance to explosion, and the amount of explosives on structural response. Major findings and conclusions based are summarized in Chapter 5.

Chapter 2

Blast Loads and the Selected Building

2.1 Characteristics of Bomb Blast

An explosion is a very rapid release of a large amount of energy characterized by a very loud blast. Part of the energy is released as thermal energy, and part is released into the air (air-blast) and into soil (ground-shock) as rapidly expanding shock waves. A shock wave consists of highly compressed air expanding in all directions from the explosion centre at a very high speed until reaching equilibrium with the surrounding air. Both the pressure and the velocity of the front of blast wave (called shock front) decrease rapidly as the shock wave propagates outward. The initial pressure resulting from an explosion, which is normally greater than the ambient pressure, is referred to as the *overpressure*.

An air blast occurs within milliseconds. The shock wave produced by a blast is shown schematically in Figure 2.1. It consists of two phases, i) positive pressure and ii) negative pressure. During the positive phase, the overpressure rises very rapidly from ambient (atmospheric) to the peak value, and then decreases slowly reaching the ambient pressure once again. In the negative phase, a vacuum is created and the air is sucked in rather than being pushed away as in the case of positive overpressure. Consequently, the wind blows towards the point of the detonation during the negative phase. The peak value of the negative phase (i.e., underpressure) is much smaller than that of the positive phase.

Given this, the negative phase is normally neglected in the design of blast load resisting structures. In practical applications, the overpressure due to blast is usually idealized as shown in Figure 2.2. The overpressure is approximated by a triangular impulse with zero rise time and a linear decay from the beginning to the end of impulse as illustrated in the figure.

When a blast wave strikes on a solid surface, the wave is reflected from the surface. When reflection occurs, the surface is subjected to a much higher overpressure than that caused of the incident wave in free air. This pressure is referred to *reflected pressure*.

2.2 Blast Wave Parameters for Blast Loading

It is important to acknowledge that blast loads are short-duration dynamic loads. Their duration is about 1000 times shorter than the duration of typical earthquakes. Structural response under short-duration dynamic loading could be significantly different than that under much longer duration loading such as an earthquake-induced dynamic excitation. For analysis of buildings under blast loading, the first step involves the determination of blast pressures and the duration blast loads. The characteristics of blast loading are affected by the following factors:

- * Type and size (TNT equivalent weight) of bomb
- * Location of the explosion centre relative to the structure (e.g. internal or external explosion),
- * Distance from the explosion centre to the structure,

- * Distance from the explosion centre to the structure,
- * Location of the explosion centre inside the building (basement or at higher floors, close to walls or columns, etc),
- * Type of the structural material (reinforced concrete, steel, or masonry structure),
- * Type of the structure system (moment resisting frame, shear wall, or combined frame-shear wall structural system, etc),
- * Height of the structure,
- * Design considerations for blast and/or other dynamic effects during original design or strengthening of the structure.

The properties of blast waves can be established through charts developed by the U.S. Department of the Army using the parameter Z where, $Z=R/W^{1/3}$ (TM5-1300 manual, 1990). R is the distance from the center of explosive source to the structure in ft., and W is the TNT equivalent charge weight in lb. The value of W for different TNT types of explosives can be obtained by multiplying the mass of the explosive by a corresponding conversion factor. The charts are given for two types of explosions, i.e., the spherical and hemispherical TNT explosions. The spherical TNT is an explosion in air, i.e., far from nearest reflecting surface. On the other hand, the hemispherical explosion is called the explosion at or very near the ground surface at sea level conditions of $P_0 = 101$ kPa and $a_0 = 340$ m/sec, where P_0 is atmospheric pressure and a_0 is sound speed in air. Figure 2.3 shows blast parameters for an explosive detonated on free ground surface (hemispherical explosion) as a function of scaled distance $Z=R/W^{1/3}$. The blast wave

parameters indicated in the figure are described below:

- P_{so} : Peak positive overpressure, also called peak side-on overpressure or incident overpressure in psi.
- i_s : Side-on specific impulse, defined as the area under the positive overpressure versus time curve in psi-ms.
- t_a : Shock arrival time in ms.
- t_d : Duration of the positive phase in ms.
- P_r : Peak normally reflected overpressure in psi.
- i_r : Normally reflected specific impulse, defined as the area under the positive normally reflected overpressure-time curve, in psi-ms.
- U : Shock front velocity in ft/ms.
- L_w : Wave length of positive phase in ft.

The parameters for normally reflected blast waves, P_r , and i_r , shown in Figure 2.3, are applicable for those cases when the structural elements are perpendicular to the direction of shock waves. Otherwise, the incident angle at which the blast wave strikes the structure should be taken into account to determine the parameters of reflected waves.

2.3 Blast Loads on the Front Face of a Building

When blast waves strike a building at zero angle of incidence relative to the front face the front wall, side walls, the roof, and the rear wall of the building are also loaded with corresponding loads due to blast. In this study, the blast loads on roof and side walls

are not included because their contributions to torsional response of the building considered are negligible compared to those acting on the front face. Furthermore, the blast loads acting on the rear wall acting in the opposite direction are quite small. Their effects are normally neglected in practice. Therefore, the loads acting on the front face are the only loads that were considered in this investigation. This simplified approach provides approximate but sufficiently accurate results for the purpose of evaluating the overall impact of blast on torsional response.

For the front face of a box-shape building, it is usually conservative to neglect the influence of the angle of incidence at which a blast wave strikes the building surface. Thus, the normally reflected parameters obtained from Figure 2.3 were directly used in the current investigation.

When a blast wave strikes a building, a diffraction of the shock wave occurs, causing interaction between the blast wave and the building. A decrease of the overpressure occurs at the edge of the front wall due to the diffraction of shock waves (Naumoski 2004). Consequently, the relationship between load per unit area and duration shown in Figure 2.4(a) is somewhat different from the relationship illustrated in Figure 2.2. The reflected overpressure vs. time can be simplified as illustrated in Figure 2.4, which is based on the idealized shock wave illustrated in Figure 2.2.

In Figure 2.4(a), P_s is the stagnation overpressure, t_a is the shock arrival time and t_c is the clearing time defined as $t_c = 3S/U < t_d$. In this equation, t_d is the positive phase duration, U is the shock front velocity, and S is the clearing distance, which is defined as

the lesser of the height and one half of the building width. For $t_c > t_d$, the pressure vs. time relationship for the front face of a building can be approximated to be linear as illustrated in Figure 2.4(b).

2.4 Description and Design of the Selected Building

A 10-storey reinforced concrete office building was selected for investigation of torsional effects under blast loading. The building was selected and designed as a frame building in Ottawa, with a total height of 40 m. Figure 2.5 illustrates the plan and elevation views of the building. The frames are considered to be the only structural elements without any participation from non-structural elements. The floor plan is symmetric in both directions.

The building was originally designed by Dincer (2003) according to the 1995 edition of the National Building Code of Canada (NBCC 1995), and 1984 edition of the CSA Standard A23.3 (1984). The design was subsequently adapted to the requirements of the 2005 edition of NBCC (2005) and 2004 edition of CSA Standard A23.3 (2004). The original design was conducted using a 2-D analysis computer software DRAIN-2D (1973). Some details in design were slightly changed in the current study based on the results of a 3-D elastic analysis using computer software ETABS (CSI 2002). Accordingly, the dimensions of beam cross sections, except for the roof, were changed from 300 mm x 500 mm (width x height) to 300 mm x 450 mm. The beam dimensions used for the roof remained as 300 mm x 400 mm. Concrete compressive strength and

reinforcement yield strength used were 30 MPa and 400 MPa, respectively.

The building was designed for seismic effects in addition to the gravity loads. The seismic analysis was conducted using ETABS (CSI 2002) in accordance with the provisions of Clause 4.1.8.12 and Clause 4.1.8.4 of NBCC 2005. The modal response spectrum method was used in design. The design response spectrum is shown in Figure 2.6. The design spectral acceleration values, $S(T)$, shown in the figure were determined as follows:

$$\begin{aligned} S(T) &= F_a S_a(0.2) \text{ for } T \leq 0.2s \\ &= F_v S_a(0.5) \text{ or } F_a S_a(0.2), \text{ whichever is smaller for } T = 0.5 \text{ s} \\ &= F_v S_a(1.0) \text{ for } T = 1.0s \\ &= F_v S_a(2.0) \text{ for } T = 2.0s \\ &= F_v S_a(2.0)/2 \text{ for } T \geq 4.0s \end{aligned}$$

where,

F_a : Acceleration-based site coefficient, which was obtained from Table 4.1.8.4.B of NBCC 2005. Since the soil type was selected as very dense soil and soft rock (Site Class C), $F_a = 1.0$ in the current study.

F_v : Velocity-based site coefficient, which was taken as 1.0 based on Table 4.1.8.4.C of NBCC 2005 for site class C.

T : Period of structure in seconds.

$S_a(T)$: 5% damped spectral response acceleration values for the reference ground conditions (Site Class C) for periods; T of 0.2 s, 0.5 s, 1.0 s, and 2.0 s, obtained

from Appendix C of NBCC 2005.

Because the building is located in Ottawa, $S_a(0.2)$, $S_a(0.5)$, $S_a(1.0)$, and $S_a(2.0)$ values are taken as 0.66g, 0.32g, 0.13g, and 0.044g. There were 30 vibration modes included in the modal response spectrum analysis although the effects of higher modes beyond the first modes associated with horizontal displacements did not contribute in any significant manner. The fundamental period of building was computed to be 2.98 s.

Three-dimensional elastic static analyses were also conducted using computer software ETABS (CSI 2002). The gravity loads used in design are:

- * Floor Dead loads: 5 kN/m^2
- * Floor Live loads: 2.4 kN/m^2
- * Roof Dead loads: 3.5 kN/m^2
- * Roof Live Loads: 2.2 kN/m^2

Bending moments, axial forces and shear forces were computed for all beams and columns. The design was based on the combined effects of: Dead load \pm Seismic load + 0.5(Live load), as required by CSA Standard A23.3-04 (CSA 2004). The reinforcement and cross sectional details of the beams and columns for each floor are shown in Figure 2.7 and Figure 2.8. The thickness of slab for all floors is 125 mm.

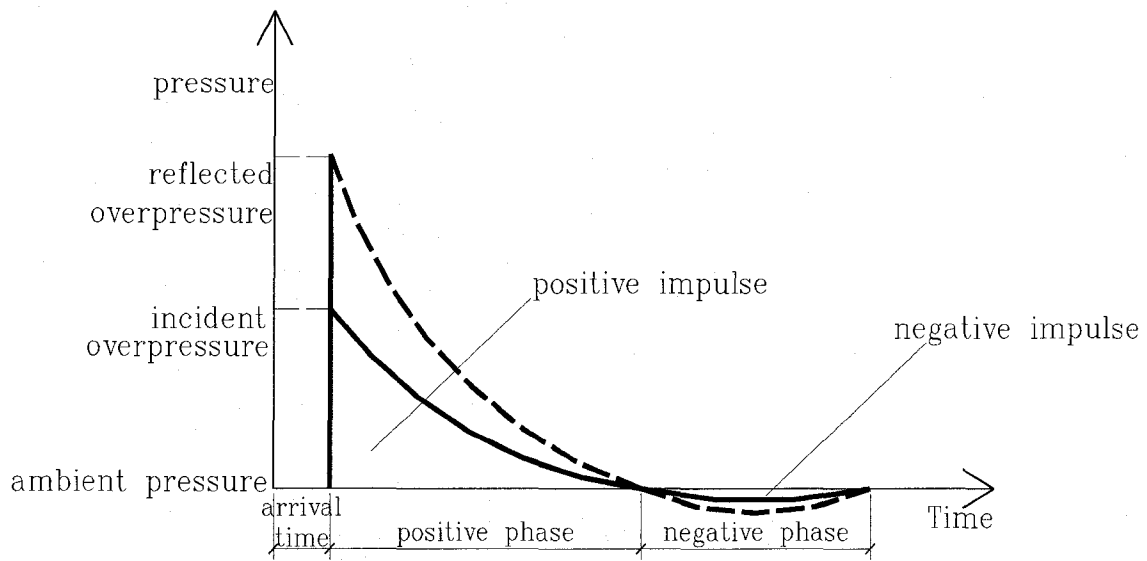


Figure 2.1 Qualitative pressure-time history. Adopted from “Structures to Resist the Effects of Accidental Explosions,” (NAVFAC 1990)

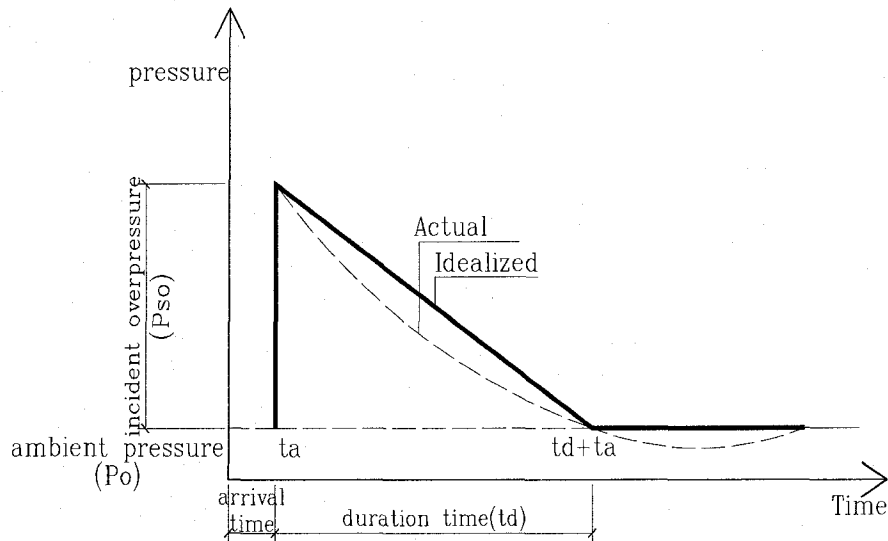


Figure 2.2 Idealization of pressure-time history. Adopted from “protection of buildings from blast effects, a state-of-the-art report” (Naumoski, 2004)

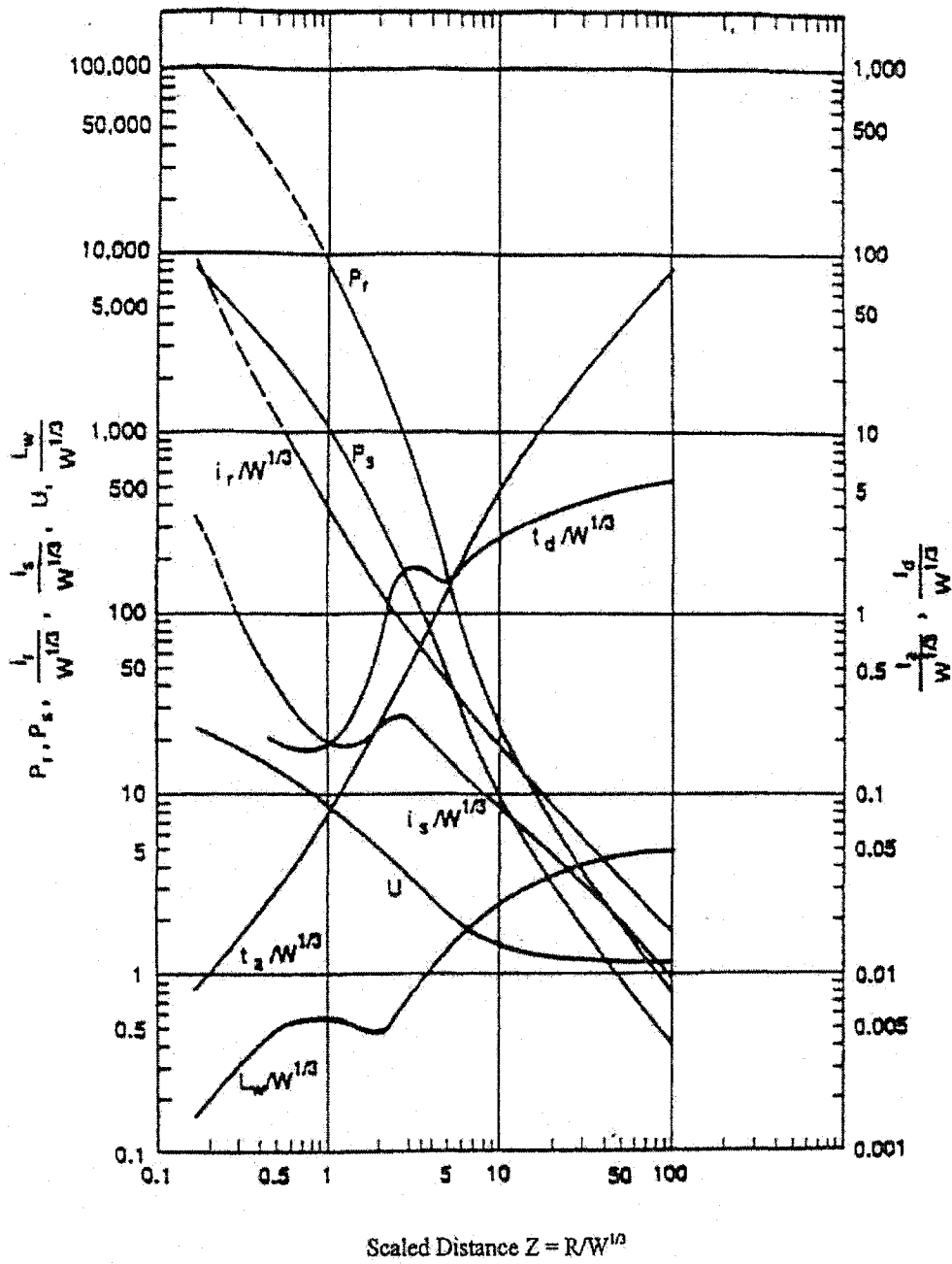


Figure 2.3 Positive phase air blast parameters for hemispherical TNT detonation on the surface at sea level. (U.S. Department of the Army, Navy, and Air Force 1990)

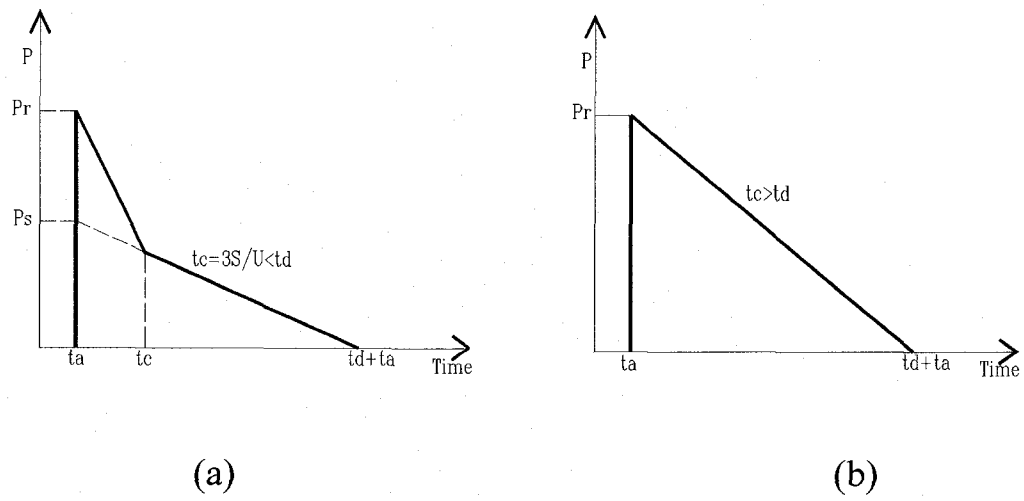


Figure 2.4 Idealized blast pressure on front face of a building

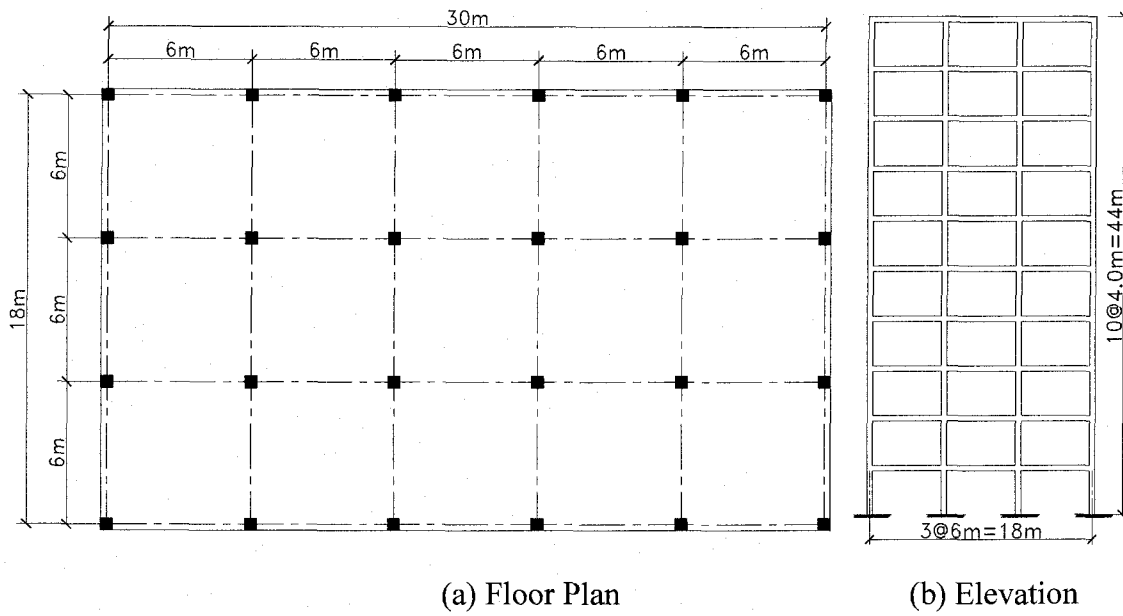


Figure 2.5 Reinforced concrete frame structure selected for analysis

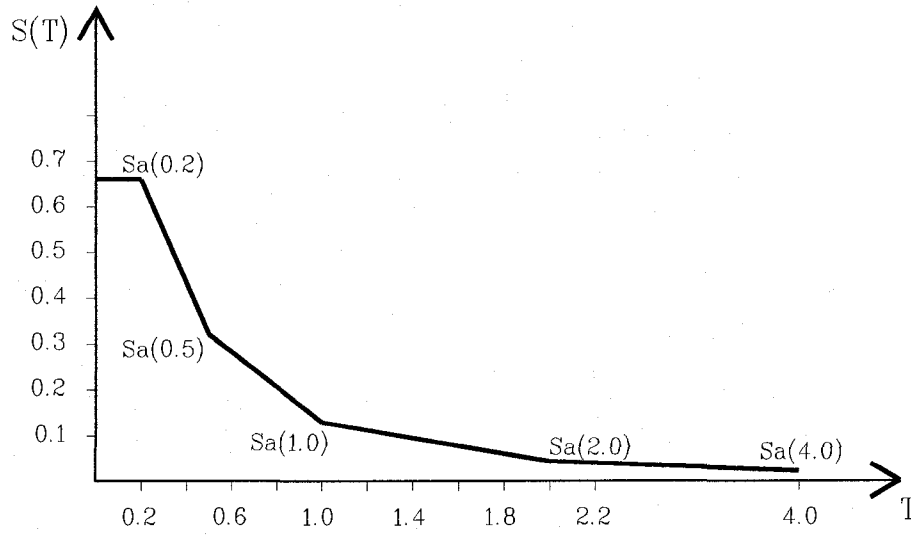


Figure 2.6 Seismic design spectrum for Ottawa, for Site Class C

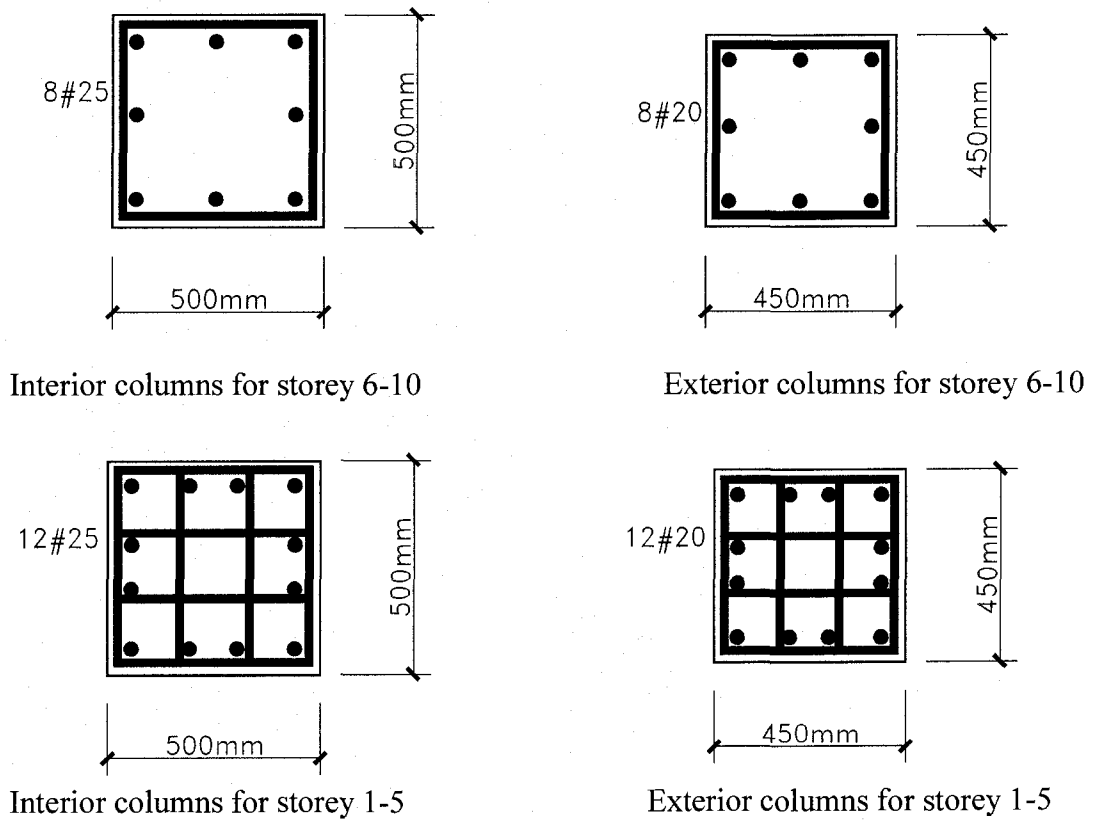
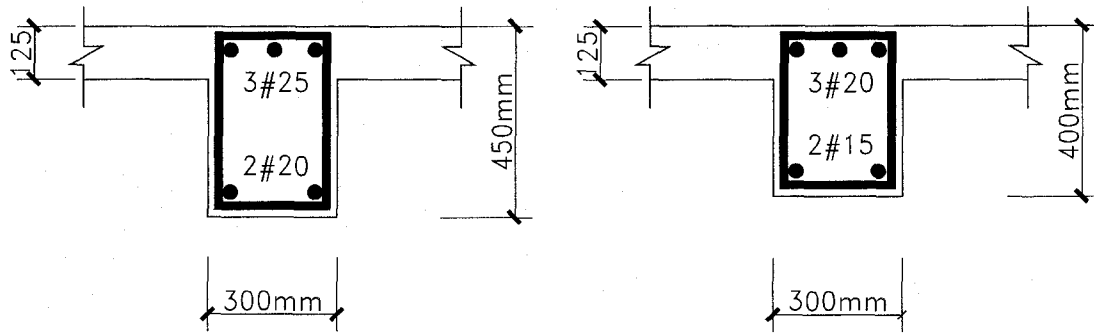


Figure 2.7 Design details of columns



Beam section for all levels except the roof

Beam section at the roof level

Figure 2.8 Design details of beams

Chapter 3

Dynamic Analysis

3.1 Computer Program for Dynamic Analysis

The evaluation of torsional response of a building requires 3 dimensional (3-D) analyses. The evaluation of blast response under impulsive dynamic loading requires dynamic analysis. Therefore computer program ETABS (CSI 2002), with 3-D dynamic analysis capabilities was selected. This software is a well known computer program, which has been widely used in research and practical applications.

ETABS is part of well-known SAP series of structural analysis software. It provides a fully integrated system for modeling, analyzing, designing, and optimizing building structures. A frame element is used to model beam-column behavior, which is represented by a straight line connecting two points (i.e. joints). The frame element uses a general three-dimensional beam-column formulation, which includes the effects of biaxial bending, torsion, axial deformation, and biaxial shear deformations. All six degrees of freedom are considered at each joint of frame elements. Constraints and restraints are used to decrease the number of unknown degrees of freedom to reduce the number of equations in the system for computational efficiency.

ETABS consists of two types of analyses; i) linear and ii) nonlinear. Linear analysis is used in this investigation. Therefore, the structural properties (stiffness, damping, etc.)

remain constant during response and the results of different analyses under different types of loads can be superposed, enabling the superposition of load effects for required load combinations.

For reinforced concrete structural elements the program has the ability to compute sectional and member capacities based on material properties, geometric data and the specified reinforcement configuration.

3.2 Input Data and Analysis Options Employed

For a given structure, ETABS requires the following input data:

- i. Geometric properties to define structural nodes and elements, as well as geometric element properties.
- ii. Joint constraints.
- iii. External loads (static or dynamic).
- iv. Type of analysis to be conducted.
- v. Time span of dynamic analysis (if selected).

Two types of analyses were required in the current investigation; i) modal analysis and ii) time-history dynamic analysis. *Modal analysis* was used to determine the vibration modes of the structure, which was required for mode superposition in dynamic analysis. The eigenvector analysis was chosen for modal analysis. This type of analysis was used for the seismic design of building as per NBCC (2005). *Linear time-history analysis* was used to investigate structural response to blast loading. A step-by-step linear time

integration technique was used for this purpose. Modal solution method was selected instead of the Direct-integration method for this computation. The load, $p(t)$, applied in a given time-history case can be a function of space and time. It can be written as a finite sum of spatial load vectors, r_i , multiplied by time functions, $f_i(t)$, as:

$$p(t) = \sum_i f_i(t)r_i. \quad (\text{Eq. 3.1})$$

$f_i(t)$ was taken as the linear time function corresponding to joint i , and r_i consisted of specified forces directed to an axis at joint i .

The time-history analysis was performed with discrete time steps. The time span of the analysis (i.e., the duration of the analysis) was 5 s. This time span is larger than the natural period of the fundamental mode of 2.98 sec. The integration time step in the analysis was 0.001s. Zero initial conditions were assumed, i.e., displacement, velocity, and acceleration were all assigned zero values at time $t = 0$.

3.3 Modelling of Building

A three dimensional building model was generated by specifying the locations of beam-column joints (nodes) and element lengths. Each element was modeled as a “stick element” with beam-column properties, i.e., with abilities to simulate flexural, shear and axial deformations.

The blast loads are specified similar to the approach used in wind load analysis. This

implies that the blast loads acting on the exposure area were transferred to act as concentrated loads on corresponding joints connecting the columns and the beams as illustrated in Figure 3.1. Load per unit area of the front face varies over the tributary area of each joint, designated as A_t in Figure 3.2. For simplicity, however the force acting on a given joint was assumed to represent an average value of the blast loads on the tributary area associated with that joint. It was also assumed that 50% of the front face of a building resists blast loads. This is an approximation and is based on the assumption that 50% of the front face area is not damaged during the blast and can transfer loads to the joints. Therefore, the effective load area corresponding to each joint is designated as A_e , and can be written as $A_e = A_t/2$.

Given the forgoing assumption, the maximum dynamic load acting on a joint can be expressed as the product of the effective area (A_e) and the peak normally reflected overpressures (p_r). The design loads linearly decreases with the increase in time, as shown in Figure 2.4(b). In this figure, P_r (peak reflected overpressure), t_a (shock arrival time), and t_d (duration of the positive phase) of the joint i were obtained from Figure 2.3.

As discussed in the previous chapter, the building is completely symmetric, and therefore there is no eccentricity between the center of mass and the center of rigidity. The concrete floors (assumed as rigid diaphragms) constrain the joints of beams and columns at floor levels to move together as planar diaphragms. Given this, the translations of the geometric centre of each floor, i.e., the translation U_{x0} parallel to the X axis, the translation U_{y0} parallel to the Y axis, and the rotation R_z about the vertical axis Z,

are uncoupled from each other in each storey.

3.4 Blast Load Scenarios

Explosions in this investigation are assumed to occur outside the structure on the ground surface. The locations of centres of explosion, relative to the centre of rigidity of the building, as well as the proximity to the building are illustrated in Figure 3.3. The locations of explosions along the X axis (longitudinal direction of the building) are designated as HD, and the locations from the front face of the building along the Y axis (transverse direction of the building) are designated as VD, as illustrated in the Figure 3.3.

Both the explosive charge and detonation location are key factors for evaluating blast load effects. In general, the charge and the location of an explosion are very uncertain for a given attack scenario. For example, the charge depends on the vehicle used during the attack. It can be in the order of 100 kg for a car-bomb, and 1000 kg or more for a truck-bomb. The location of explosion relative to building depends on the accessibility to building, i.e. whether a certain stand of distance is provided or not. Given these uncertainties, a range of explosive charges, varying between 100 kg and 1000 kg detonated at distances of 5m to 20m from the building are used in this study to conduct a parametric investigation. Each charge-distance combination is labeled for easy referencing. The symbol W represents the TNT charge weight, and VD and HD represent transverse and longitudinal distances, respectively, relative to the centre of rigidity in

transverse and longitudinal building plan directions. The values for W, VD, and HD used in this study provide many charge-location combinations. A total of 21 cases, designated in the form of VD-W-X were considered. As an example, the VD-W-X represented by 5m-100kg-E represents an explosion resulting from a detonation of 100 kg TNT at 5 m away from the building in the transverse direction on Line-E in Figure 3.3 (indicating location in the longitudinal direction). The charge-location combinations considered in this study are listed in Table 3.1. It can be seen that charges of W = 100 kg, 200 kg, 300 kg, 500 kg, 1000 kg, at VD = 5m, 10 m, 20 m and HD = 3 m, 9 m, 15 m were considered. It should be noted that in all cases, the explosions were assumed to occur along column lines D, E, and F (see Figure 3.3). The triangular impulsive forcing function specified at each node was computed from Figure 2.3 with due considerations given to charge weight W (in lbs) and R (in ft) as the distance to the joint of the structure in hand. The maximum and minimum values of P_r (peak reflected overpressure) and t_d (duration of positive impulse) for the building analyzed for different explosion scenarios are shown in Table 3.2. The maximum values of triangular impulse correspond to the closest first storey node to the centre of explosion and the minimum values of triangular impulse correspond to the farthest top corner node.

Table 3.1 Explosion scenarios considered

TNT weight, W, kg	Distance, VD, m	Location, HD (Line*), m		
		15m (Line-F)	9m (Line-E)	3m (Line-D)
100kg	5m	5m-100kg-F	5m-100kg-E	5m-100kg-D
200kg	5m	5m-200kg-F	5m-200kg-E	5m-200kg-D
	10m	10m-200kg-F	10m-200kg-E	10m-200kg-D
300kg	10m	10m-300kg-F	10m-300kg-E	10m-300kg-D
500kg	10m	10m-500kg-F	10m-500kg-E	10m-500kg-D
	20m	20m-500kg-F	20m-500kg-E	20m-500kg-D
1000kg	20m	20m-1000kg-F	20m-1000kg-E	20m-1000kg-D

* For line designations refer to Figure 3.3.

Table 3.2 Maximum and minimum reflected overpressure (P_r) and duration of positive phase (t_d) for triangular blast impulsive forcing functions

Case	Maximum Values of Triangular Impulse		Minimum Values of Triangular Impulse	
	Reflected overpressure (P_r) in kpa	Duration of positive phase (t_d) in ms	Reflected overpressure (P_r) in kpa	Duration of positive phase (t_d) in ms
5m-100kg-F	3864	9.63	26.9	24.58
5m-100kg-E	3864	9.63	29.1	24.44
5m-100kg-D	3864	9.63	31.2	24.33
5m-200kg-F	6389	11.43	36.7	29.92
5m-200kg-E	6389	11.43	46.6	27.74
5m-200kg-D	6389	11.43	51.8	26.70
10m-200kg-F	1736	10.90	34.6	30.40
10m-200kg-E	1736	10.90	44.2	28.26
10m-200kg-D	1736	10.90	50.4	26.96
10m-300kg-F	2642	13.01	51.1	30.70
10m-300kg-E	2642	13.01	56.1	29.65
10m-300kg-D	2642	13.01	60.9	28.76
10m-500kg-F	4020	16.59	65.0	33.32
10m-500kg-E	4020	16.59	71.4	32.20
10m-500kg-D	4020	16.59	74.5	31.90
20m-500kg-F	535	19.60	60.2	34.28
20m-500kg-E	535	19.60	65.4	33.24
20m-500kg-D	535	19.60	70.3	32.36
20m-1000kg-F	1240	18.93	76.8	40.04
20m-1000kg-E	1240	18.93	79.6	39.87
20m-1000kg-D	1240	18.93	82.1	39.71

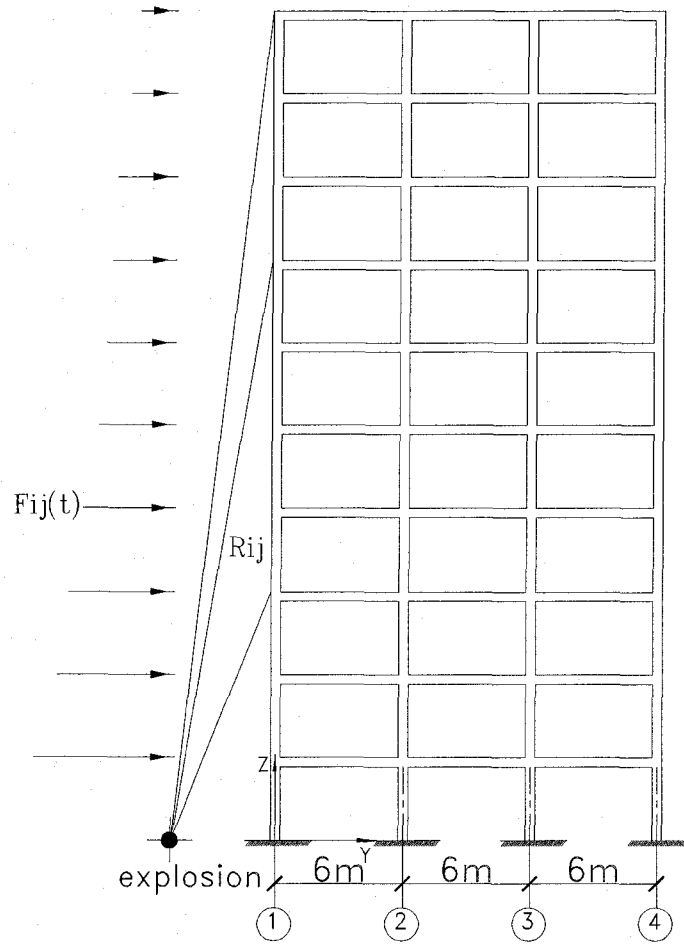


Figure3.1 Schematic illustration of blast loads along building height

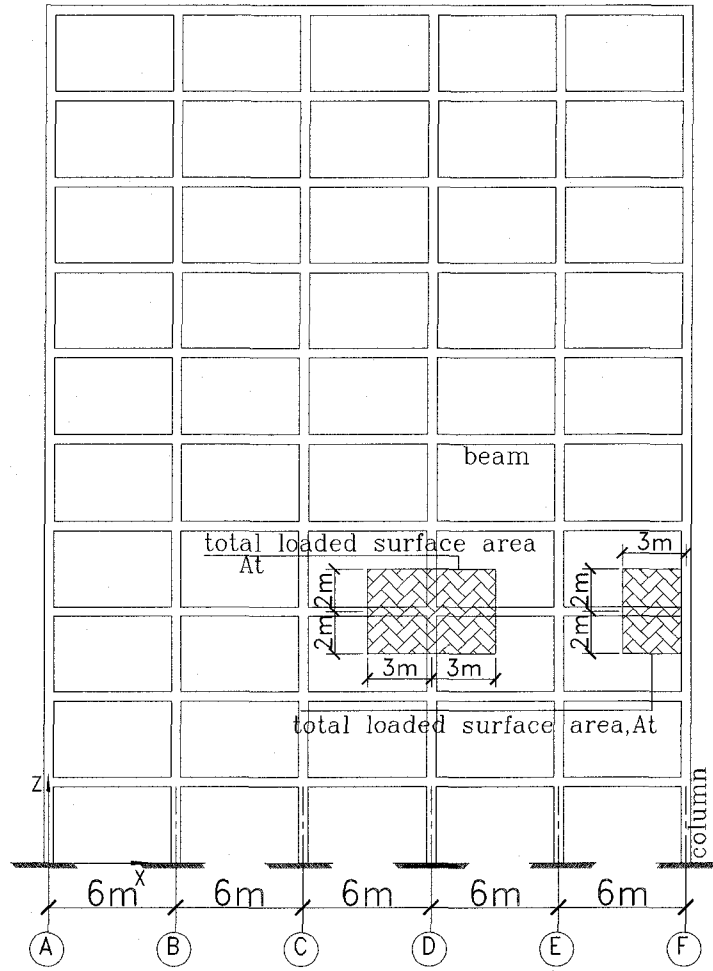


Figure 3.2 Schematic illustration of tributary area of exposed elements

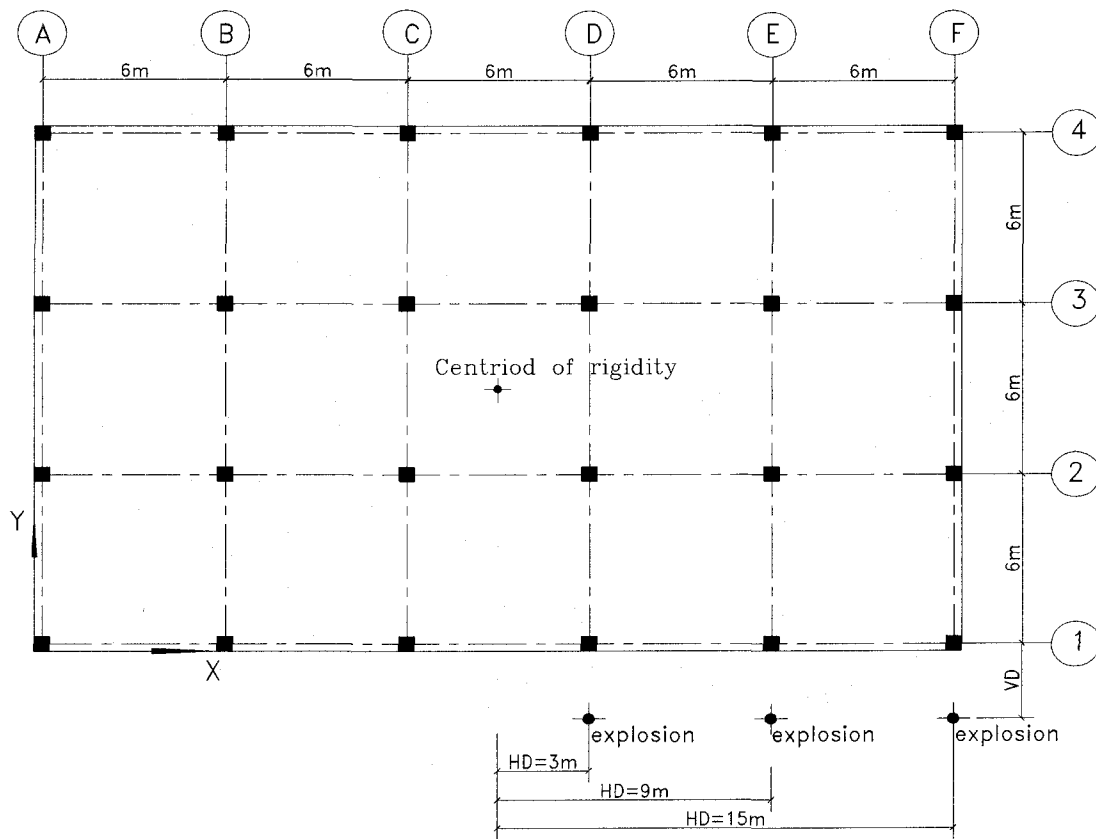


Figure 3.3 Locations of explosion centres relative to building plan

Chapter 4

Results of Dynamic Analysis

4.1 Performance Parameters

The 10-storey building selected and presented in Chapter 2 is analyzed under impulsive blast loads, applied at different levels of eccentricity relative to the centre of rigidity. The results of one of the analyses are shown in Figure 4.1 as an example. As can be seen, even a symmetrical building undergoes a torsional motion, in addition to the translational motions in three orthogonal directions.

A parametric investigation was conducted using 21 different blast loads, generating different degrees of torsion. The results were evaluated in terms of a number of selected performance parameters. These parameters are summarized below:

* *Maximum storey drift.* Storey drift ratio is a global response parameter and represents the ratio of the difference in the horizontal displacements of the bottom and top floors of a given storey to the height of the storey. The maximum values of storey drift ratios for different cases of blast loads were computed from the response time histories of displacements.

* *Maximum rotation, R_z .* Rotation is an appropriate parameter for measuring the torsional response of buildings. In this study, since the centre of rigidity coincides with the geometrical centre of each floor, the value of the rotation of the building, R_z (about Z

axis), is due to the eccentricity between the blast load (at each floor) and the centre of rigidity.

* Maximum displacement in Y direction. U_{y0} represents the Y displacement of the centre of mass (i.e. geometrical centre) in the Y direction, and U_y represents the Y displacement of joints of the building. ETABS provides both U_{y0} and U_y displacements. U_{y0} displacements were used to estimate the pure translation of building, and U_y displacements were used to determine the torsional response. In this study, the displacements in Y direction are much larger than those in X-direction, and therefore only the displacements in the Y direction are discussed.

* N-M Demand/Capacity ratio for columns. The Demand/Capacity ratio (DCR), is expressed as $DCR = D/C$, where, D = Demand (i.e., moment and axial force acting on the column) resulting from elastic analysis, and C = Capacity of the column (i.e., moment and axial force that the column can resist). The load combination $1.0DL + 0.5LL + 1.0BL$ was used for calculating demands, where DL is dead load, LL is live load and BL is blast load. The capacity calculations included the strain rate effect by increasing material strengths by 25%. This is consistent with the requirements of GSA Progressive Collapse Document (2003). Hence, a strength increase factor of 1.25 was used to determine dynamic strengths of both concrete and reinforcing steel. This was achieved by applying $\phi_s = 1.25$ and $\phi_c = 1.25$ as material resistance factors. The DCR values greater than 1.0 indicate that the column is overstressed.

Based on the comparison of the foregoing response parameters, the following topics

are discussed in this chapter:

** Effects of the location of explosion centre along the building (designated HD in Figure 3.3).*

** Effects of the distance from explosion centre to the front face of building (designated VD in Figure 3.3).*

** Effects of the explosive charge, W (kg).*

4.2 Effects of the Location of Explosion Centre

A total of 21 analyses were conducted in 7 groups, classified in terms of the amounts of explosive and distance to the explosion centre of the building. Each group comprised of 3 cases of different detonation location. The comparison of response quantities is carried out within each group. The classification of the groups is as follows:

- * Group 1: 100kg TNT detonation at a distance of 5m on Lines D, E, and F;*
- * Group 2: 200kg TNT detonation at a distance of 5m on Lines D, E, and F;*
- * Group 3: 200kg TNT detonation at a distance of 10m on Lines D, E, and F;*
- * Group 4: 300kg TNT detonation at a distance of 10m on Lines D, E, and F;*
- * Group 5: 500kg TNT detonation at a distance of 10m on Lines D, E, and F;*
- * Group 6: 500kg TNT detonation at a distance of 20m on Lines D, E, and F;*
- * Group 7: 1000kg TNT detonation at a distance of 20m on Lines D, E, and F.*

4.2.1 Effects of the Location of Explosion Centre on Storey Drift Ratio

The effects of the detonation location on storey drift ratios are illustrated in Figures

4.2, through 4.4 and in Table 4.1. Each figure contains three curves, corresponding to interstorey drifts due to the detonation of the same explosive charge at a fixed distance along column lines D, E, and F. The results indicate that the distribution of drift along building height indicates a similar trend. For explosions at a distance of 5m (i.e. Group 1 and Group 2), the maximum drift occur at the first storey level. For example, an explosion of 200 kg explosives at a distance of 5 m along Line E produces a maximum interstorey drift of 1.86% at the 1st floor level. On the other hand, for detonations at distances of 10 m and 20m, shown in Figure 4.3 and Figure 4.4, maximum interstorey drifts occur at the second storey level, though first storey drifts remain to be high.

Figures 4.2 to 4.4, illustrate that interstorey drifts are related to the location of the explosion. An explosion at Line-E produces higher interstorey drift than those at Lines D and F. However, the effects of the location of explosion could be negligible in some cases. For example, at a distance of 20 m, when explosive weighs are less than 500 kg, the maximum difference between interstorey drifts for explosions at Lines E and F is approximately 0.0404%, which is very small and could be neglected for all practical purposes. Among the cases considered, the maximum interstorey drift ratio of approximately 2.3% occurs when 500 kg of explosives is located 10 m away from the building. This level of drift may be too high for a gravity load buildings, without seismic design and detailing.

4.2.2 Effects of the Location of Explosion Centre on Building Rotation

Figures 4.5 to 4.7 show the effects of the locations of detonation on building rotation. Maximum rotations occur consistently at the roof level with the smallest rotation taking place at the first floor level. The explosion caused by 500 kg of explosives 10 m away on Line F produces a maximum rotation of 0.00644 rad at the roof level.

Explosions at Line D produce less torsional motion as expected since Line D is closer to the centre of rigidity. Figures 4.6 and 4.7 illustrate that at the same floor level, the rotations increase with an increase in distance HD. Figure 4.5 indicates that rotations caused by explosions at Line F (i.e. HD = 15 m) are smaller than those triggered by explosion at Line E (i.e. HD = 9 m). This can be explained with the reduced tributary area in the former case, though the eccentricity of load is higher. In spite of this difference in tributary areas, relatively small differences in rotations are observed between explosions taking place along Lines E and F. In general, however, torsional response increases with increasing eccentricity of load, as expected.

4.2.3 Effects of the Location of Explosion Centre on Displacements

The maximum displacement at the centre of mass of each floor (i.e. U_{y0}) is shown in Tables 4.2 to 4.8. These displacements represent pure translational response of the building under blast loading. A general observation from these tables indicates that the effect of the eccentricity of load is to reduce the displacement of the centre of rigidity. Therefore, when HD is equal to 3.0 m, the horizontal displacement is higher than that for

HD = 9.0 m, which is higher than that having HD = 15.0 m. In addition to the translational motion, the rotational motion also contributes to displacements at joints (i.e. U_y), and therefore the values of U_y shown in Figure 4.8, 4.9 and 4.10 depend on the resultants of translational and rotational response. In all cases, the roof experiences maximum displacements.

4.2.4 Effects of the Location of Explosion Centre on Nominal N-M Demand/Capacity Ratio

The effects of the location of the detonation centre on nominal demand/capacity ratios are shown in Figures 4.11 through 4.13 for Group1, 4, and 7. The figures include the nominal N-M DCRs for the bottom section of the first storey columns. These columns are considered to be the most critical for structural design since they are subjected to the largest forces. The three plans in each of these figures show the DCR results for explosion at Lines D, E, and F. It can be observed from these figures that the maximum DCR always corresponds to the corner columns, i.e., where Line F crosses Line 1. Also, it can be seen that the DCR values for the columns on the same longitudinal line (i.e. X-X line) decrease from the right side to the left side of building (i.e. from Line-F to Line-A). Therefore, the columns of the sideline closer to the centre of explosion, especially the corner columns, are the most critical elements to protect.

Figure 4.12 (a), (b), and (c), for Group 4 loading indicates that the explosion at Line-E generates a maximum DCR of 2.93, which is larger than the DCR of 2.48 observed

when the explosion is at Line-D. Similarly, the explosion at Line-E produces the smallest DCR value of 1.12 which is much smaller than the DCR of 1.58 created by the explosion at Line-D. This behaviour is attributed to torsional effects which could significantly amplify structural response to blast loading for columns that are closer to the centre of explosion. It is interesting to note that torsional effects caused by the same eccentric loading reduce structural response for columns near the far end of the building plan. Therefore, the number of overstressed columns (i.e. strength demands larger than the capacity, i.e., $DCR > 1.0$), are substantially influenced by torsional effects. As seen in Figure 4.12(a) and (b), for the cases of TNT 300 kg at a distance of 10 m, the detonation at Line-D produces 24 overstressed columns, more than the 20 overstressed columns caused by the detonation at Line-E. This is because of reductions in demands caused by torsional effects in columns on Line-A and Line-B. The reductions in force demands due to torsion are less when the explosion is at Line-D than those due to the explosion at Line-E.

Table 4.9 shows the maximum nominal demand/capacity ratios for columns and the number of first-storey columns with DCR larger than 1.0. This table demonstrates that in general, the number of overstressed bottom-storey columns decreases with eccentricity of blast load.

4.3 Effects of the Distance of Explosion Centre from the Building

Six groups of analyses were conducted to investigate the effects of the distance of explosion to the building. Two cases in each group, within which the comparison of response quantities is conducted, have an identical amount of explosives detonated along the same column line but with different distances from the building. The classification of the groups is as follows:

- * Group I: 200 kg TNT detonation on Line F at distances of 5 m and 10 m;
- * Group II: 200 kg TNT detonation on Line E at distances of 5 m and 10 m;
- * Group III: 200 kg TNT detonation on Line D at distances of 5 m and 10 m;
- * Group IV: 500 kg TNT detonation on Line F at distances of 10 m and 20 m;
- * Group V: 500 kg TNT detonation on Line E at distances of 10 m and 20 m;
- * Group VI: 500 kg TNT detonation on Line D at distances of 10 m and 20 m;

4.3.1 Effects of Explosion Distance to Building on Structural Response

Maximum Storey Drift Ratio: Graphs (a), (b), and (c) of Figures 4.14 and 4.15, include two curves each, showing maximum interstorey drifts along the building height for those cases with identical explosive charge placed at 5 m and 10 m away from the building on Line F, E, and D, respectively. The figures show that the interstorey drift dramatically decreases with distance from the explosion. Figure 4.14(a), (b), and (c) show that for the explosion of 200 kg of TNT (i.e. Group I, II, and III), the maximum drifts induced by an explosion at 5 m is approximately 2 to 3 times larger than those caused by an explosion 10 m away from the building. Similarly, Figures 4.15(a), (b), and (c), show

that the explosion of 500 kg TNT at 10 m causes approximately 3 to 4 times larger maximum interstorey drifts than those induced by an explosion 20 m away from the building. The comparison of results indicates that for two explosions of certain charge, detonated at two different distances to the building, the ratio of the interstorey drifts at any floor is approximately equal to the ratio of the detonation distances multiplied by a constant. In the current investigation, the constant was found to be within the range of 4 to 6 for explosions of 200kg TNT, and within the range of 6 to 8 for explosions of 500 kg TNT.

Maximum Rotation: Figures 4.16 and 4.17 show torsional rotations induced along the building height by explosions at distances of 5 m, 10 m and 20 m. The results indicate that closer explosions generate higher torsional response. The explosion of 200 kg TNT would cause much larger rotational response at 5 m than at a distance of 10 m. The variation of torsional rotations along the building height is small for a distant explosion. Close explosions result in a larger variation of rotations between the roof and the first storey level.

Maximum Displacement: Graphs (a), (b), and (c) of Figure 4.18 and Figure 4.19 show the effects of the distance of explosion to maximum building displacements. These figures illustrate that the displacement response increases significantly with the proximity of explosions to the building.

4.3.2 Effects of Explosion Distance on N-M Demand/Capacity Ratios

The computed demand-capacity ratios (DCR) under different charge and distance combinations are summarized in Table 4.10. The results indicate that the structure may remain undamaged after blast when explosives of 500 kg or less are detonated at distances longer than 20 m. This means that exceeding 20 m could be considered safe for buildings with similar characteristics as those considered in the current investigation under the explosion of 500 kg TNT. However, when the distance is reduced to 10 m, the maximum DCR increases over 5.0 indicating significant damage to first-storey columns and potential collapse of the building. Table 4.10 also indicates that the explosion of 200 kg of TNT within 5 m results in the entire first-storey columns to be overstressed with DCRs exceeding 1.0. This indicates potential collapse of the building. However, when the distance is increased to 10 m, less than 10 first-storey columns (42 % of bottom-storey columns) developed DCR of larger than 1.0. This observation confirms once again the importance of the distance of explosion on structural response and building damage.

4.4 Effects of TNT Charge Weight

The effect of the amount of explosives, expressed in terms of TNT equivalence, was investigated through a series of building analyses. These analyses are put in nine groups, as indicated below:

- * Group a: 100kg and 200kg TNT on Line F at a distance of 5m;
- * Group b: 100kg and 200kg TNT on Line E at a distance of 5m;

- * Group c: 100kg and 200kg TNT on Line D at distances of 5m;
- * Group d: 200kg, 300kg and 500kg TNT on Line F at a distance of 10m;
- * Group e: 200kg, 300kg, and 500kg TNT on Line E at a distance of 10m;
- * Group f: 200kg, 300kg, and 500kg TNT on Line D at a distance of 10m;
- * Group g: 500kg and 1000kg TNT on Line F at a distance of 20m;
- * Group h: 500kg and 1000kg TNT on Line E at a distance of 20m;
- * Group i: 500kg and 1000kg TNT on Line D at a distance of 20m;

4.4.1 Effects of TNT Charge Weight on Structural Response

Maximum Storey Drift Ratio: Figures 4.20, 4.21 and 4.22 illustrate interstorey drifts for different weight of explosives detonated at the same location and distance. The results indicate an increase in storey drift with increasing explosive weight. The results indicate that doubling the charge weight results in approximately twice the storey drift in building response. It appears that the ratio of two charge weights is approximately the same as the ratio of structural response attained through dynamic analysis.

Maximum Rotation: Figures 4.23 through 4.25 show the rotational response of building as a function of charge weight. The results indicate that an increase in charge weight produces approximately a proportional increase in building rotation. It is also observed that low explosive weights result in approximately constant rotation throughout the height of the building, though higher charges produce higher rotations at the roof level.

Maximum Displacement: Graphs (a), (b), and (c) of Figures 4.26 through 4.28, show

the effects of explosive charge on maximum displacement of the building. It can be seen in these figures that horizontal displacements in the direction of blast wave pressure increase with increasing charge weight proportionately.

4.4.2 Effects of Charge Weight on N-M Demand/Capacity Ratio

Table 4-11 provides a summary of nominal demand-capacity ratios (DCR) for columns. Maximum values of DCR and the number of overstressed first-storey columns increase with the explosive weight. When the building is subjected to an explosion of 500 kg TNT at 20m on Line D, the maximum DCR of the columns is 1.0 and none of the columns is overstressing, indicating elastic structural response. When the explosive weight is doubled to 1000 kg, the maximum DCR increases to 2.67 and the entire first-storey columns are overstressed.

Table 4.1 Maximum interstorey drifts

Group	Case	Storey drift on Y-direction (%)
Group 1	5m-100kg-F	0.6078
	5m-100kg-E	0.8190
	5m-100kg-D	0.6571
Group 2	5m-200kg-F	1.3960
	5m-200kg-E	1.8617
	5m-200kg-D	1.5343
Group 3	10m-200kg-F	0.5329
	10m-200kg-E	0.6342
	10m-200kg-D	0.5924
Group 4	10m-300kg-F	0.8737
	10m-300kg-E	1.0496
	10m-300kg-D	0.9689
Group 5	10m-500kg-F	1.7449
	10m-500kg-E	2.1398
	10m-500kg-D	1.9018
Group 6	20m-500kg-F	0.5646
	20m-500kg-E	0.6050
	20m-500kg-D	0.5955
Group 7	20m-1000kg-F	1.1836
	20m-1000kg-E	1.3001
	20m-1000kg-D	1.2765

Table 4.2 Maximum Y-displacement at the centre of mass for detonation of 100kg TNT at distance of 5m (i.e. Group 1)

Floor	Uyo (units: mm)		
	5m-100kg-F	5m-100kg-E	5m-100kg-D
1	11.3	18.3	20.6
2	18.6	27.8	31.3
3	22.8	32.5	36.3
4	25.5	35.1	38.9
5	27.4	36.7	40.7
6	28.3	37.1	40.9
7	31.9	38.8	42.0
8	37.3	45.2	48.9
9	44.6	54.8	59.6
10	52.1	65.8	72.0

Table 4.3 Maximum Y-displacement at the centre of mass for detonation of 200kg TNT at distance of 5m (i.e. Group 2)

Floor	Uyo (units: mm)		
	5m-200kg-F	5m-200kg-E	5m-200kg-D
1	26.2	41.9	48.1
2	41.7	63.4	72.2
3	49.5	72.2	81.6
4	53.6	76.1	85.3
5	56.5	78.1	87.0
6	57.6	78.2	86.7
7	60.5	78.3	86.5
8	70.6	88.8	96.6
9	85.8	109.8	120.0
10	102.7	135.1	148.7

Table 4.4 Maximum Y-displacement at the centre of mass for detonation of 200kg TNT at distance of 10m (i.e. Group 3)

Floor	Uyo (units: mm)		
	10m-200kg-F	10m-200kg-E	10m-200kg-D
1	10.8	15.0	17.2
2	21.1	28.1	31.8
3	28.1	36.2	40.5
4	32.8	41.1	45.6
5	36.8	44.9	49.3
6	39.4	46.8	51.1
7	46.0	53.2	57.1
8	54.1	62.5	67.0
9	64.1	75.0	81.0
10	73.2	87.1	94.5

Table 4.5 Maximum Y-displacement at the centre of mass for detonation of 300kg TNT at distance of 10m (i.e. Group 4)

Floor	Uyo (units: mm)		
	10m-300kg-F	10m-300kg-E	10m-300kg-D
1	17.8	25.4	29.1
2	34.0	46.2	52.3
3	44.2	58.2	65.3
4	50.8	65.2	72.7
5	55.8	70.0	77.5
6	58.4	72.3	79.6
7	66.4	78.6	85.1
8	78.2	92.6	100.4
9	93.8	112.6	122.6
10	108.8	132.8	145.4

Table 4.6 Maximum Y-displacement at the centre of mass for detonation of 500kg TNT at distance of 10m (i.e. Group 5)

Floor	Uyo (units: mm)		
	10m-500kg-F	10m-500kg-E	10m-500kg-D
1	35.8	52.4	60.5
2	63.9	90.3	104.1
3	81.1	110.8	126.3
4	91.5	121.9	137.9
5	98.1	128.0	143.9
6	103.5	130.1	145.5
7	124.6	142.5	149.1
8	137.5	162.9	176.0
9	157.5	196.0	217.2
10	185.5	234.3	260.8

Table 4.7 Maximum Y-displacement at the centre of mass for detonation of 500kg TNT at distance of 20m (i.e. Group 6)

Floor	Uyo (units: mm)		
	20m-500kg-F	20m-500kg-E	20m-500kg-D
1	12.0	14.1	15.0
2	26.8	30.6	32.5
3	37.1	42.7	45.5
4	45.3	51.7	55.0
5	52.1	58.7	62.2
6	58.5	64.5	67.8
7	69.1	76.0	79.8
8	81.6	90.2	94.9
9	95.6	106.1	111.9
10	106.7	119.0	125.6

Table 4.8 Maximum Y-displacement at the centre of mass for detonation of 1000kg TNT at distance of 20m (i.e. Group 7)

Floor	Uyo (units: mm)		
	20m-1000kg-F	20m-1000kg-E	20m-1000kg-D
1	24.7	29.8	33.2
2	54.6	64.0	70.4
3	76.3	89.1	97.2
4	93.0	107.2	115.9
5	105.6	120.8	129.9
6	115.7	130.0	138.3
7	136.5	152.9	162.4
8	161.9	181.3	192.6
9	190.4	214.6	228.7
10	213.9	242.4	259.3

Table 4.9 Effects of the location of explosion on maximum Nominal Demand/Capacity ratio of first-storey columns

Group	Case	Maximum N-M capacity ratio	The Number of Overstressed Bottom Columns
Group 1	5m-100kg-F (HD=15m)	1.891	10
	5m-100kg-E (HD=9m)	2.681	14
	5m-100kg-D (HD=3m)	1.922	20
Group 2	5m-200kg-F (HD=15m)	4.577	20
	5m-200kg-E (HD=9m)	6.233	24
	5m-200kg-D (HD=3m)	4.992	24
Group 3	10m-200kg-F (HD=15m)	1.330	6
	10m-200kg-E (HD=9m)	1.645	10
	10m-200kg-D (HD=3m)	1.352	8
Group 4	10m-300kg-F (HD=15m)	2.371	14
	10m-300kg-E (HD=9m)	2.915	20
	10m-300kg-D (HD=3m)	2.484	24
Group 5	10m-500kg-F (HD=15m)	5.440	24
	10m-500kg-E (HD=9m)	6.949	24
	10m-500kg-D (HD=3m)	5.928	24
Group 6	20m-500kg-F (HD=15m)	1.000	0
	20m-500kg-E (HD=9m)	1.078	2
	20m-500kg-D (HD=3m)	1.000	0
Group 7	20m-1000kg-F (HD=15m)	2.576	24
	20m-1000kg-E (HD=9m)	2.889	24
	20m-1000kg-D (HD=3m)	2.671	24

Table 4.10 Effects of the distance of explosion on maximum nominal Demand/Capacity ratio of first-storey columns

Group	Case	Maximum N-M capacity ratio	The Number of Overstressed Bottom Columns
Group I	5m-200kg-F	4.577	20
	10m-200kg-F	1.330	6
Group II	5m-200kg-E	6.233	24
	10m-200kg-E	1.645	10
Group III	5m-200kg-D	4.992	24
	10m-200kg-D	1.352	8
Group IV	10m-500kg-F	5.440	24
	20m-500kg-F	1.000	0
Group V	10m-500kg-E	6.949	24
	20m-500kg-E	1.078	2
Group VI	10m-500kg-D	5.928	24
	20m-500kg-D	1.000	0

Table 4.11 Effects of the explosive charge on maximum nominal Demand/Capacity ratio of first-storey columns

Group	Case	Maximum N-M capacity ratio	The Number of Overstressed Bottom Columns
Group-a	5m-100kg-F	1.891	10
	5m-200kg-F	4.577	20
Group-b	5m-100kg-E	2.681	14
	5m-200kg-E	6.233	24
Group-c	5m-100kg-D	1.922	20
	5m-200kg-D	4.992	24
Group-d	10m-200kg-F	1.330	6
	10m-300kg-F	2.371	14
	10m-500kg-F	5.440	24
Group-e	10m-200kg-E	1.645	10
	10m-300kg-E	2.915	20
	10m-500kg-E	6.949	24
Group-f	10m-200kg-D	1.352	8
	10m-300kg-D	2.484	24
	10m-500kg-D	5.928	24
Group-g	20m-500kg-F	1.000	0
	20m-1000kg-F	2.576	24
Group-h	20m-500kg-E	1.078	2
	20m-1000kg-E	2.889	24
Group-i	20m-500kg-D	1.000	0
	20m-1000kg-D	2.671	24

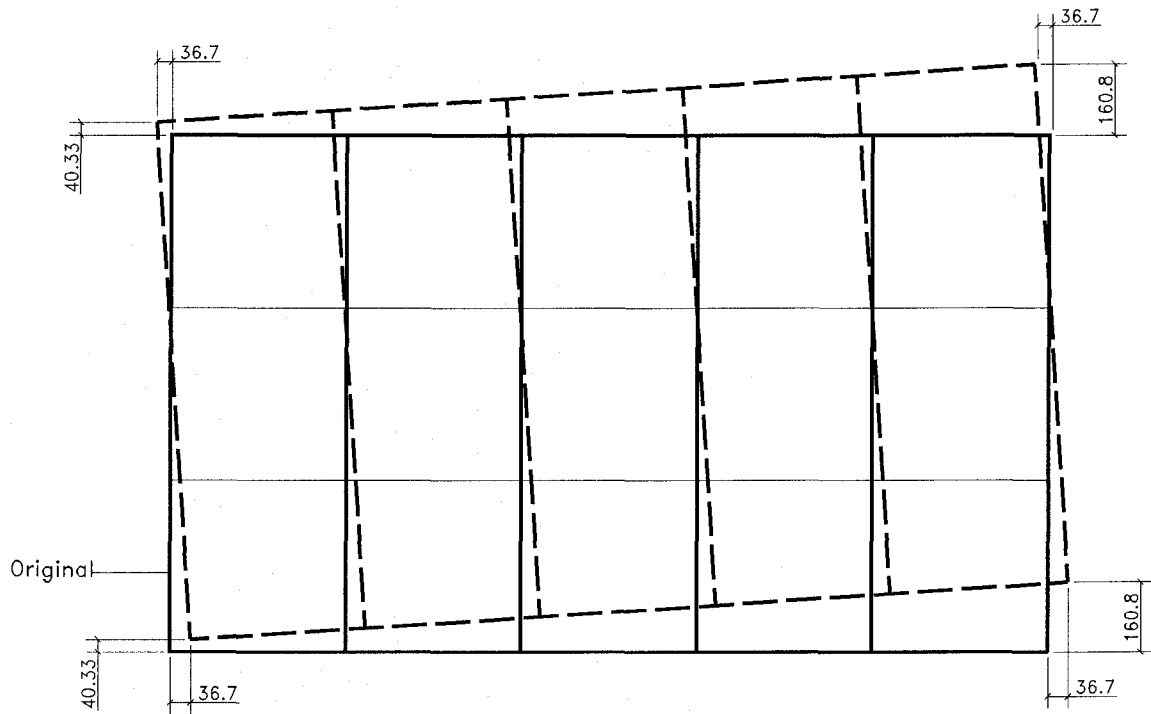


Figure 4.1 Maximum Y-displacement of the roof for the case of 5m-200kg-E
(units in mm)

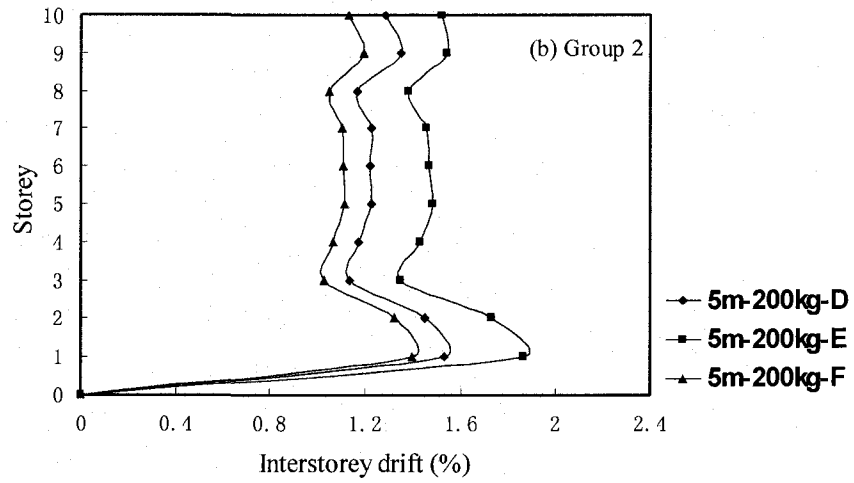
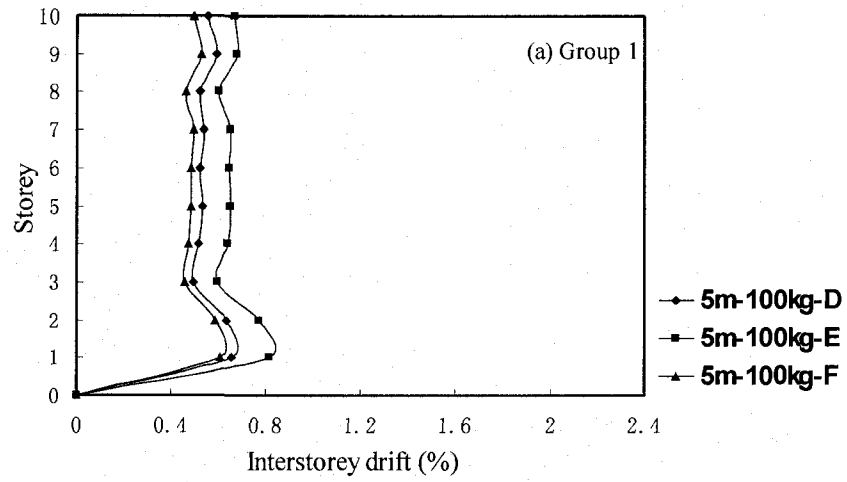


Figure 4.2 Effects of the eccentricity of blast loads on interstorey drifts (Explosion at 5 m)

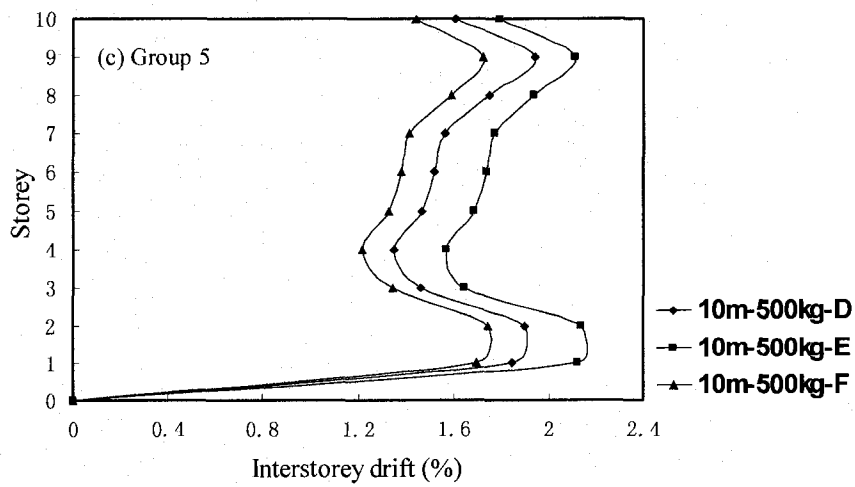
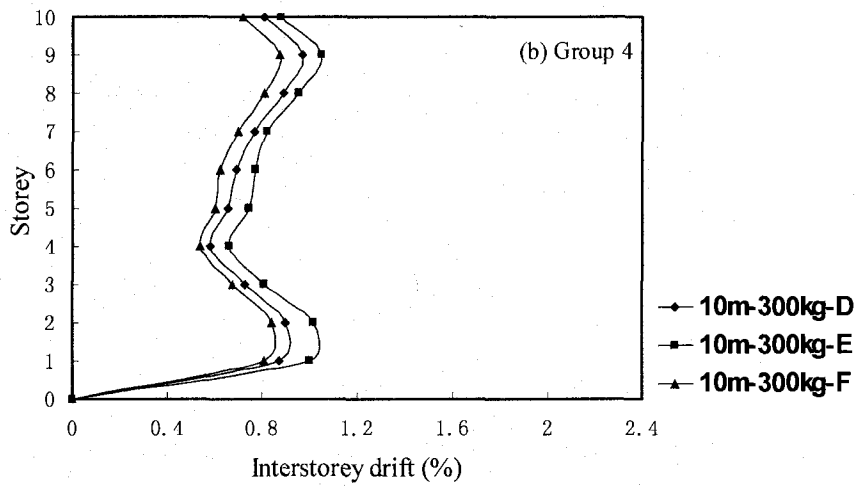
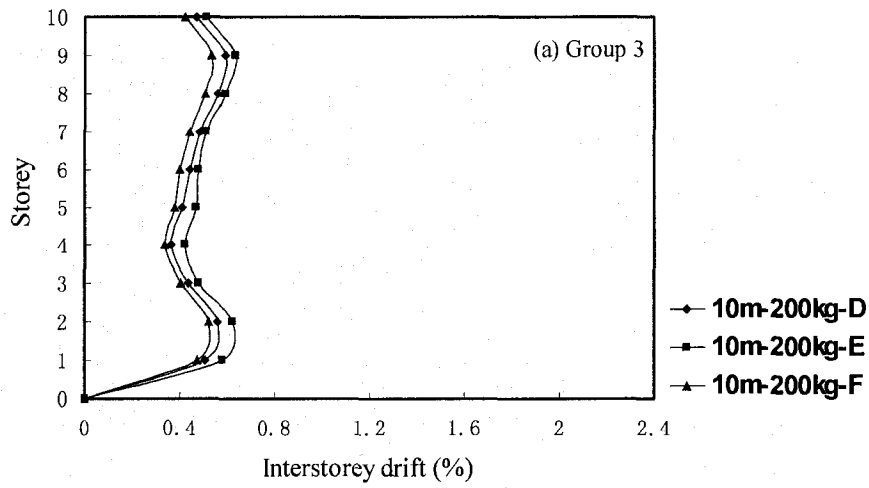


Figure 4.3 Effects of the eccentricity of blast loads on interstorey drifts (Explosion at 10 m)

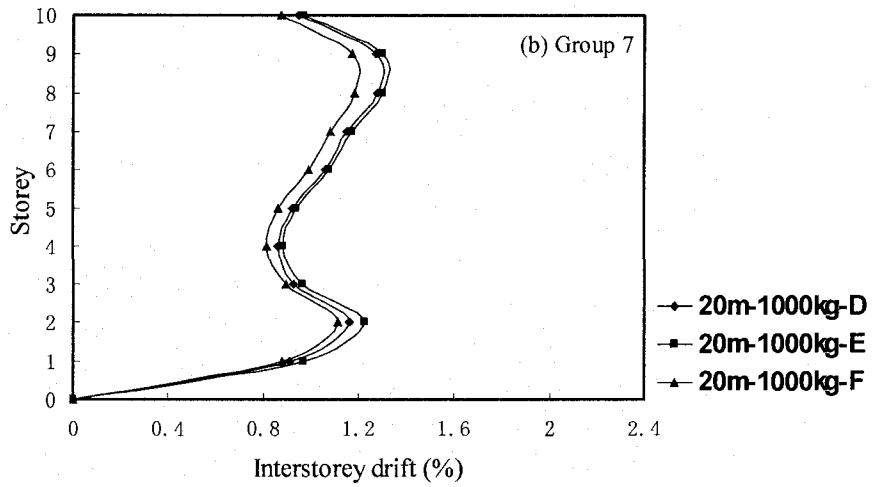
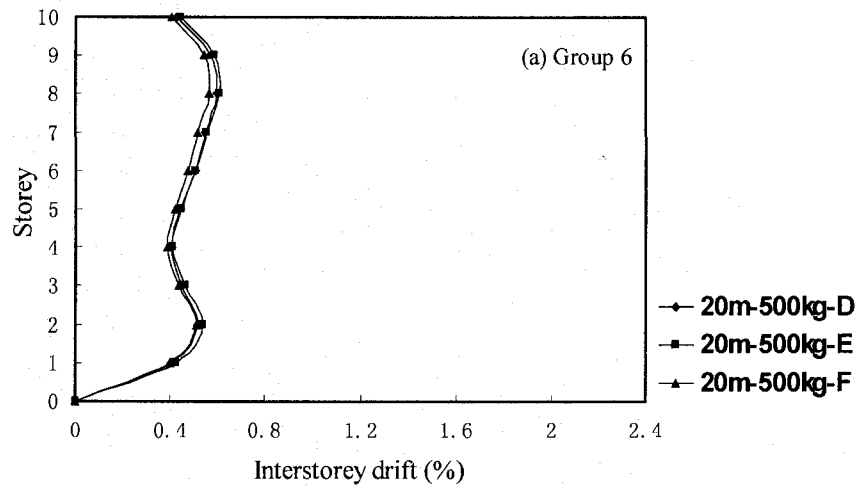


Figure 4.4 Effects of the eccentricity of blast loads on interstorey drifts
(Explosion at 20 m)

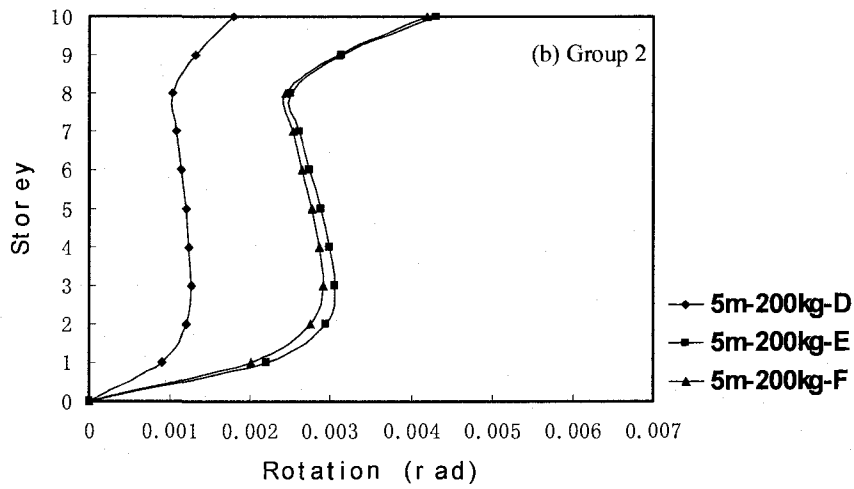
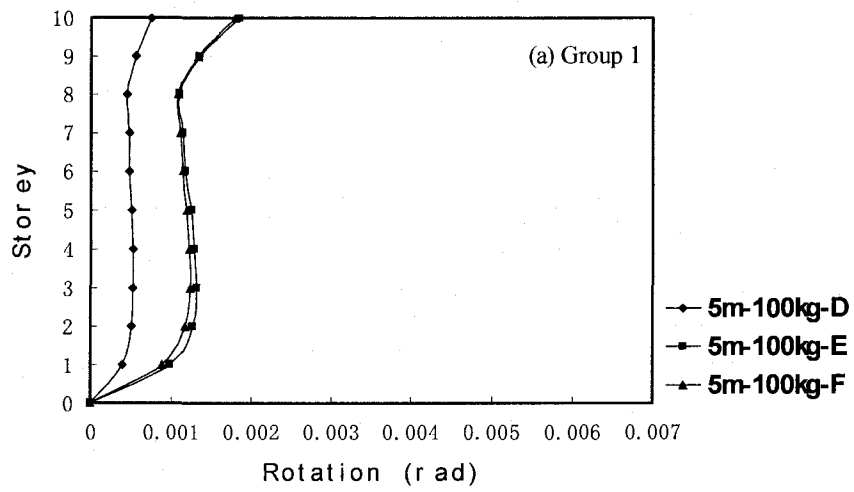


Figure 4.5 Effects of the eccentricity of blast loads on maximum rotation in radians (Explosion at 5 m)

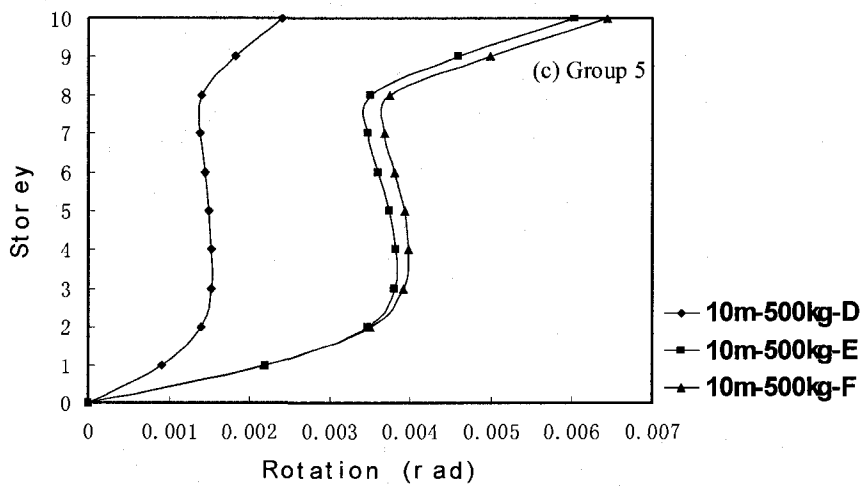
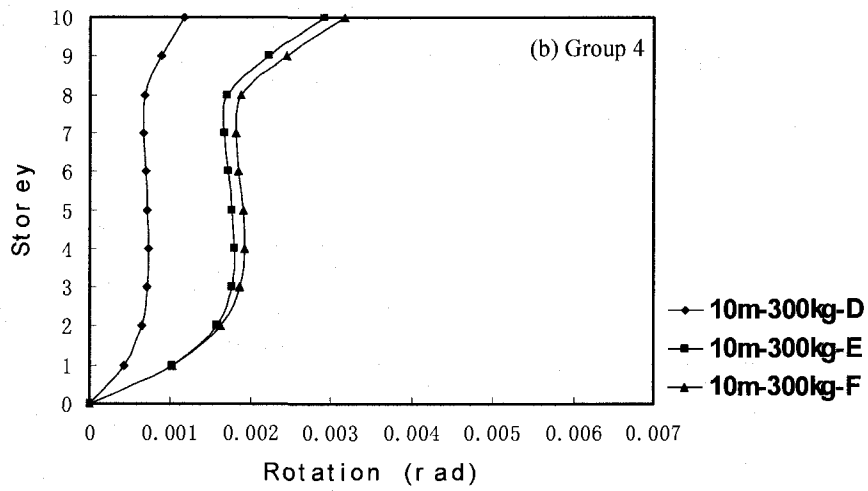
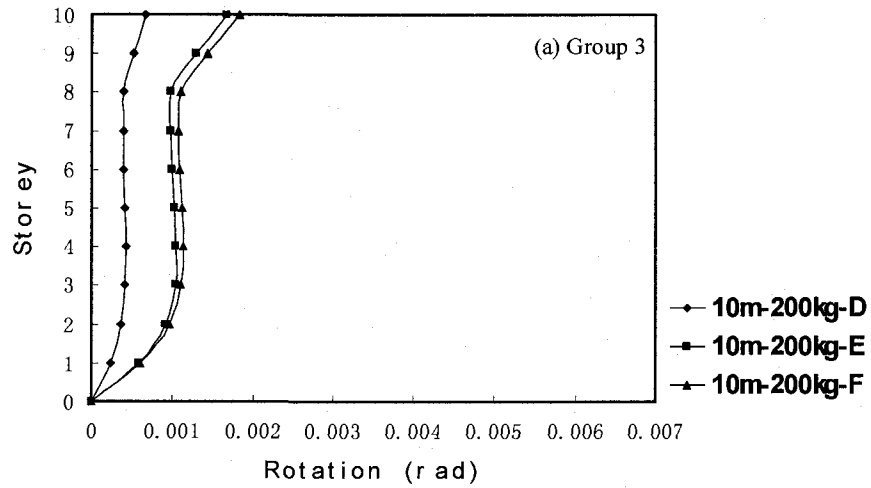


Figure 4.6 Effects of the eccentricity of blast loads on maximum rotation in radians (Explosion at 10 m)

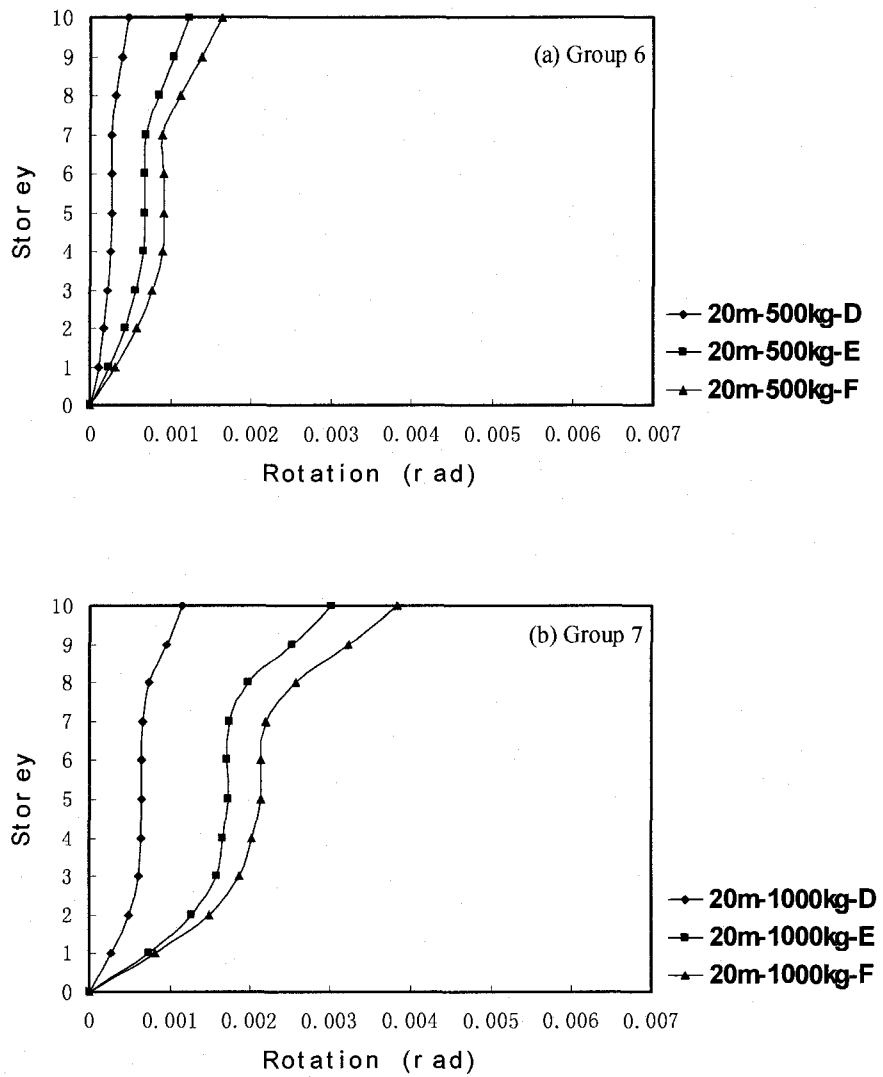


Figure 4.7 Effects of the eccentricity of blast loads on maximum rotation in radians (Explosion at 20 m)

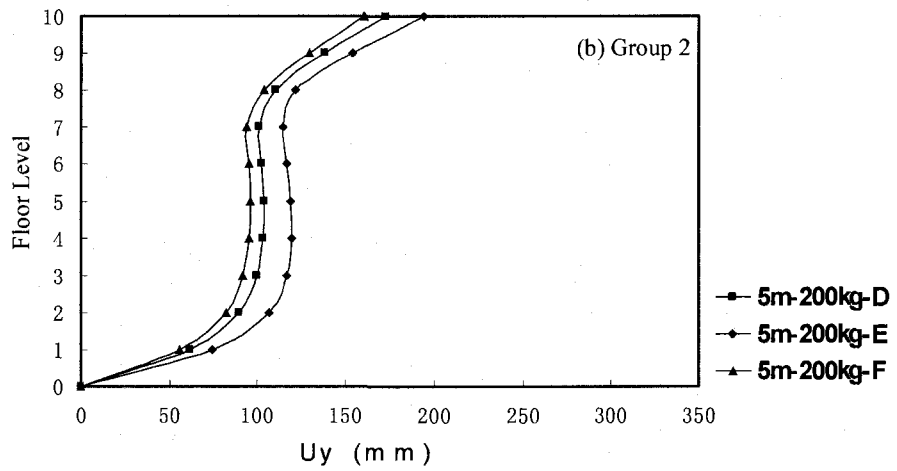
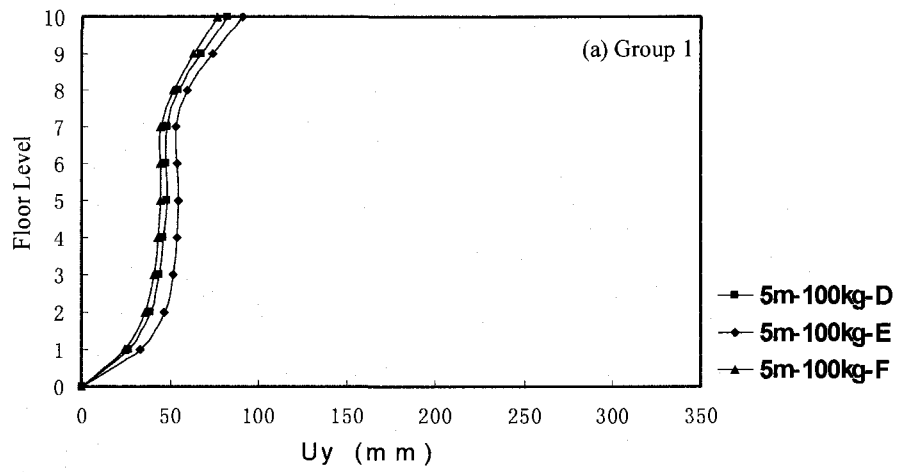


Figure 4.8 Effects of the eccentricity of blast loads on maximum displacement of joints (Explosion at 5 m)

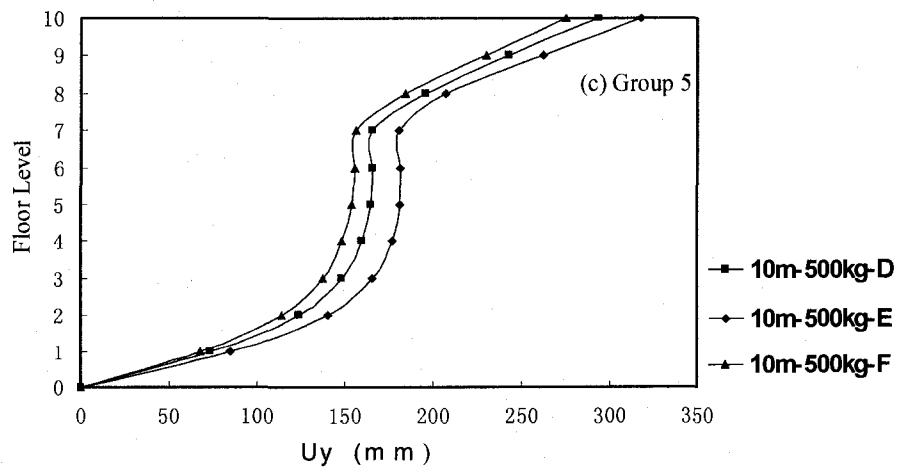
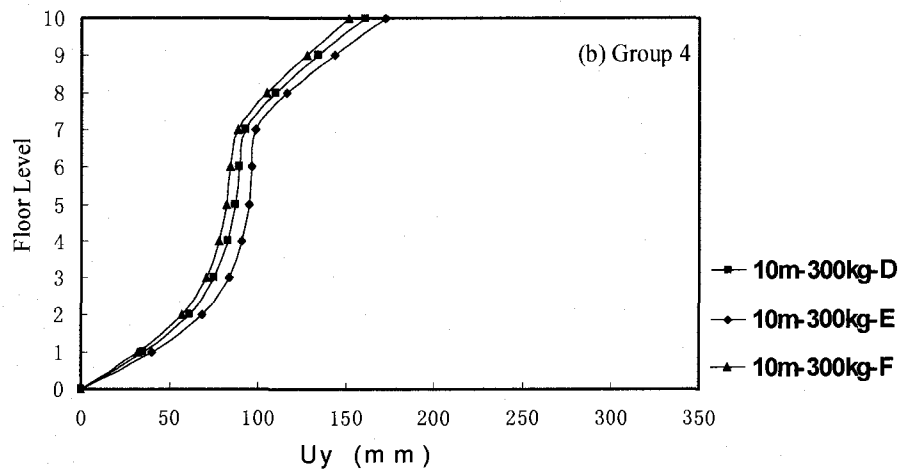
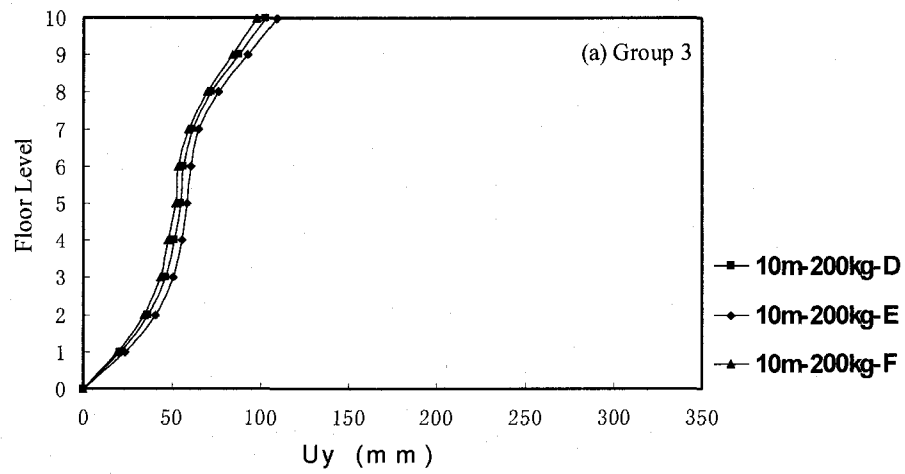


Figure 4.9 Effects of the eccentricity of blast loads on maximum displacement of joints (Explosion at 10 m)

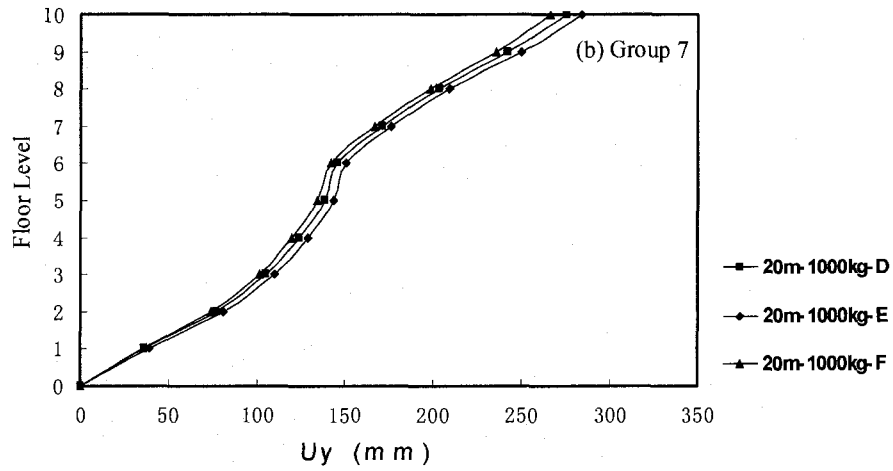
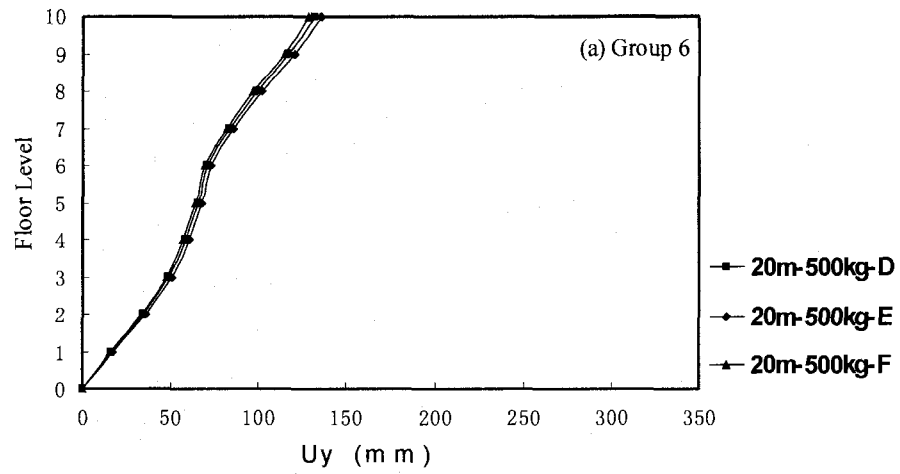
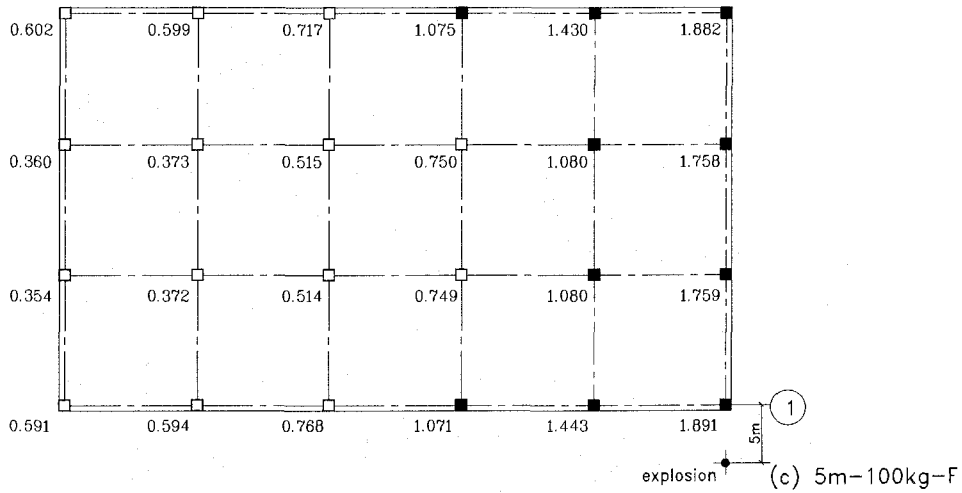
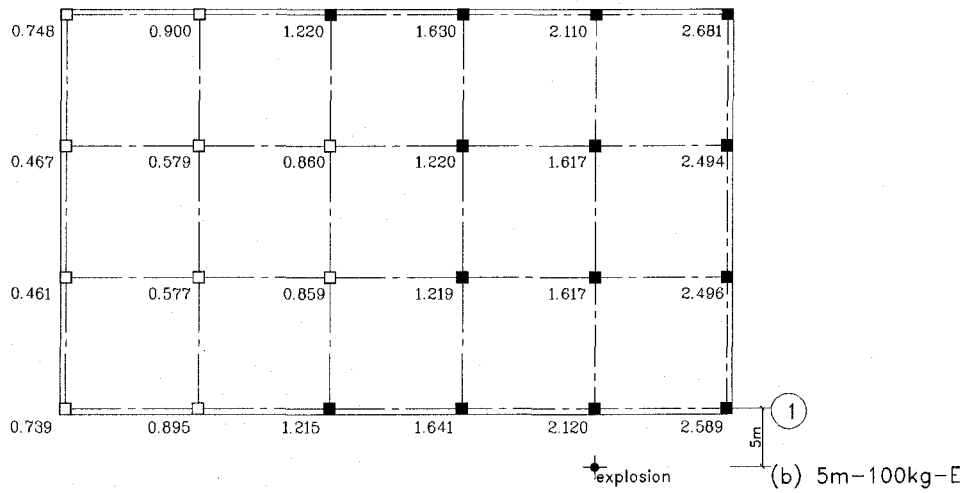
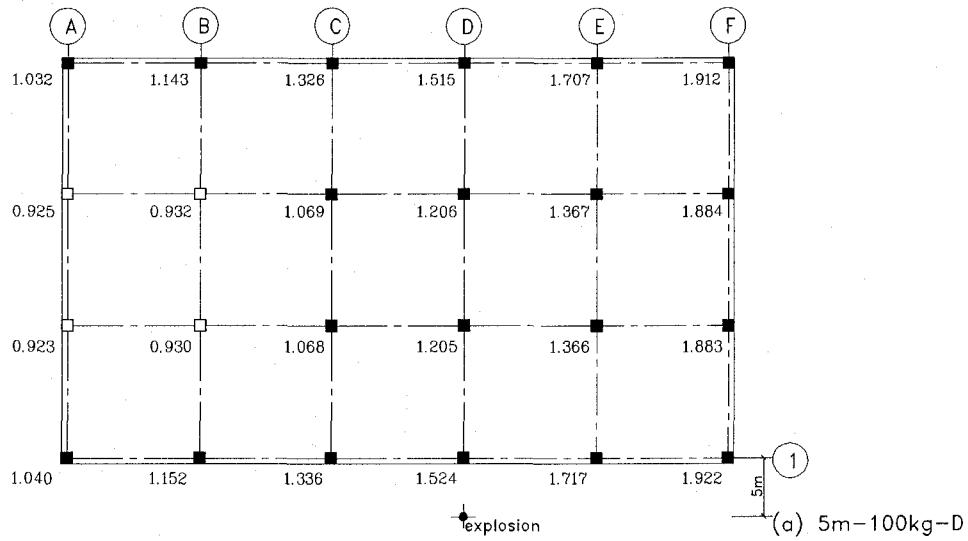
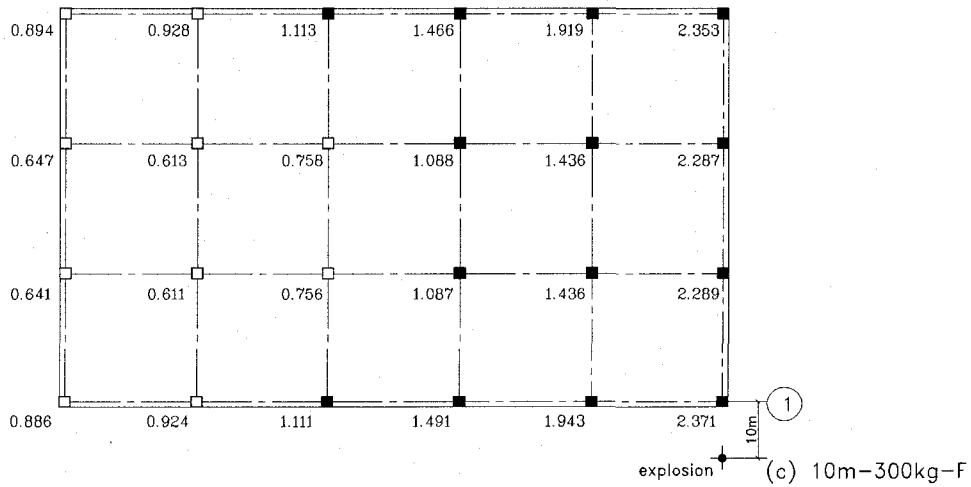
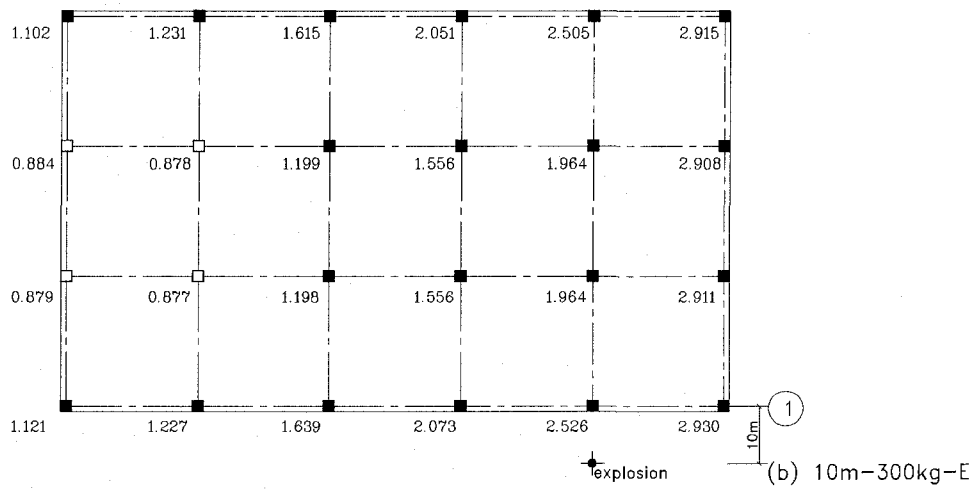
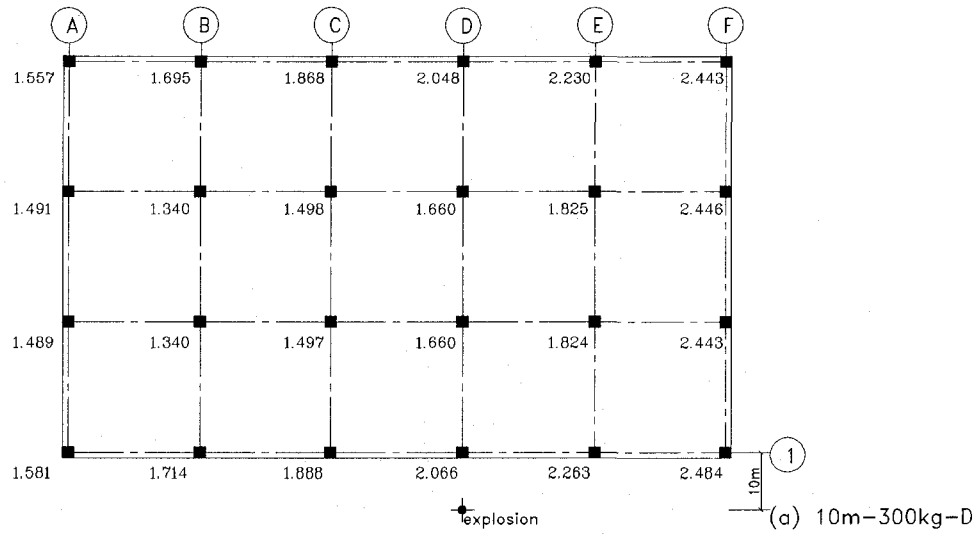


Figure 4.10 Effects of the eccentricity of blast loads on maximum displacement of joints (Explosion at 20 m)



□ D/C ≤ 1.0
 ■ D/C > 1.0

Figure 4.11 Maximum nominal N-M Demand/Capacity ratio of first-storey columns when 100 kg of explosive charge detonated at 5 m (i.e. Group 1)



□ D/C ≤ 1.0
 ■ D/C > 1.0

Figure 4.12 Maximum nominal N-M Demand/Capacity ratio of first-storey columns when 300 kg of explosive charge detonated at 10 m (i.e. Group 4)

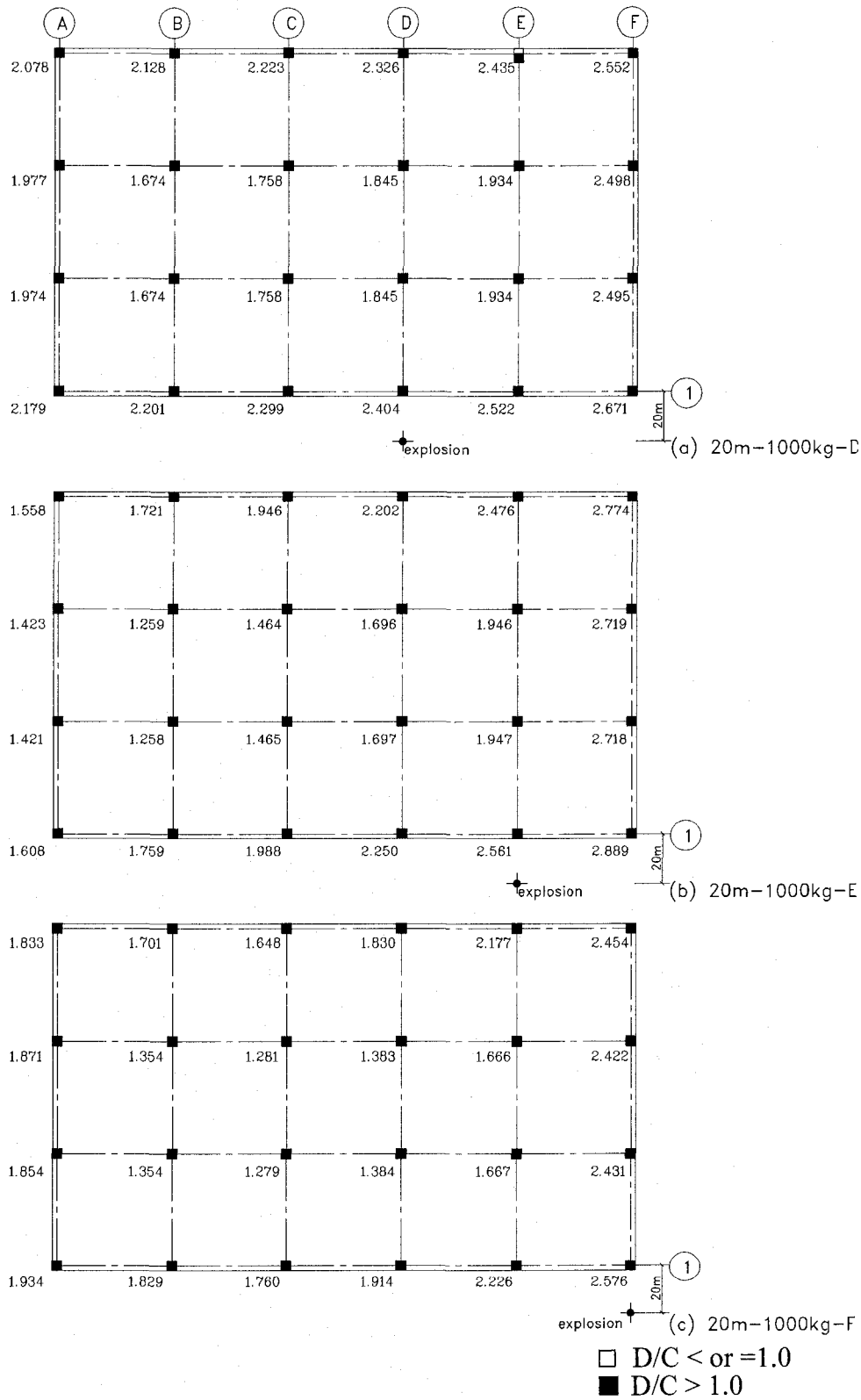


Figure 4.13 Maximum nominal N-M Demand/Capacity ratio of first-storey columns when 1000 kg of explosive charge detonated at 20 m (i.e. Group 7)

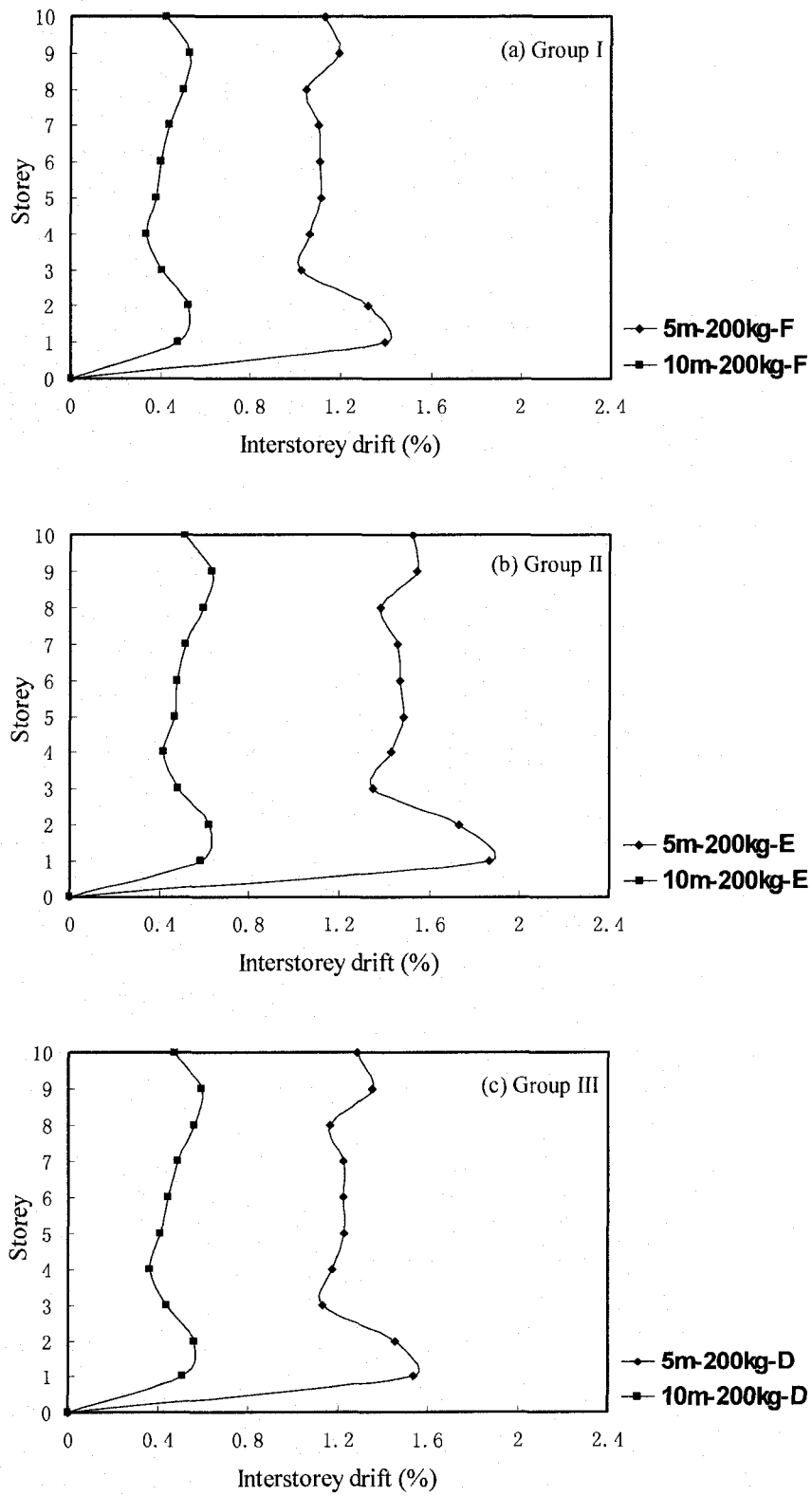


Figure 4.14 Effects of the distance from explosion centre to building on interstorey drift for charge weight of 200 kg

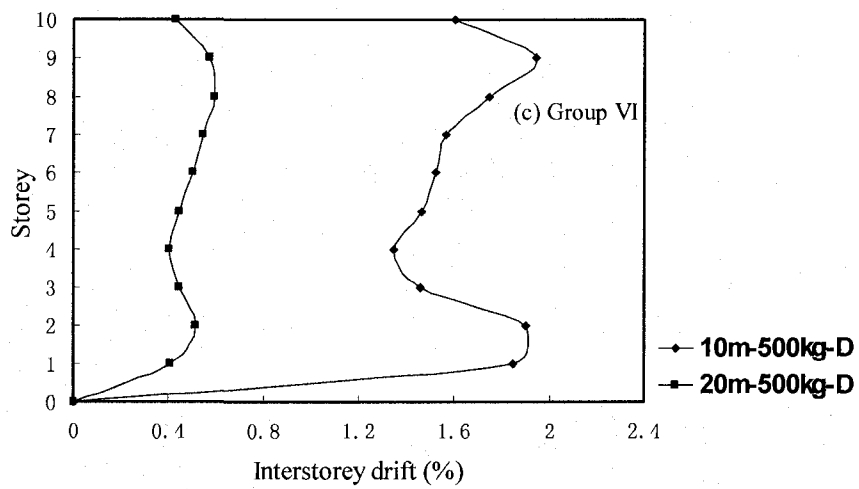
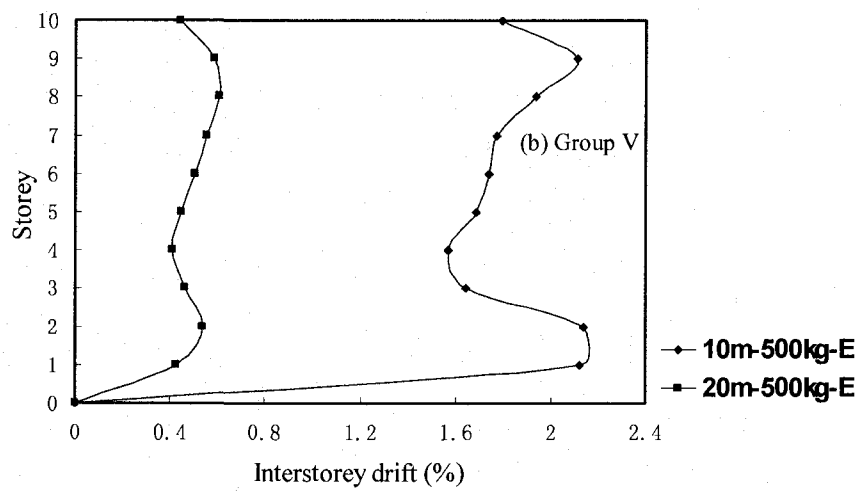
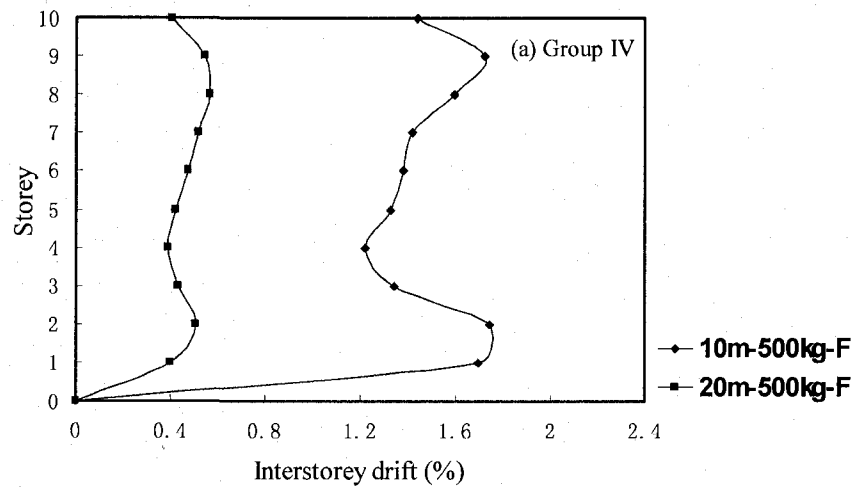


Figure 4.15 Effects of the distance from explosion centre to building on interstorey drift for charge weight of 500 kg

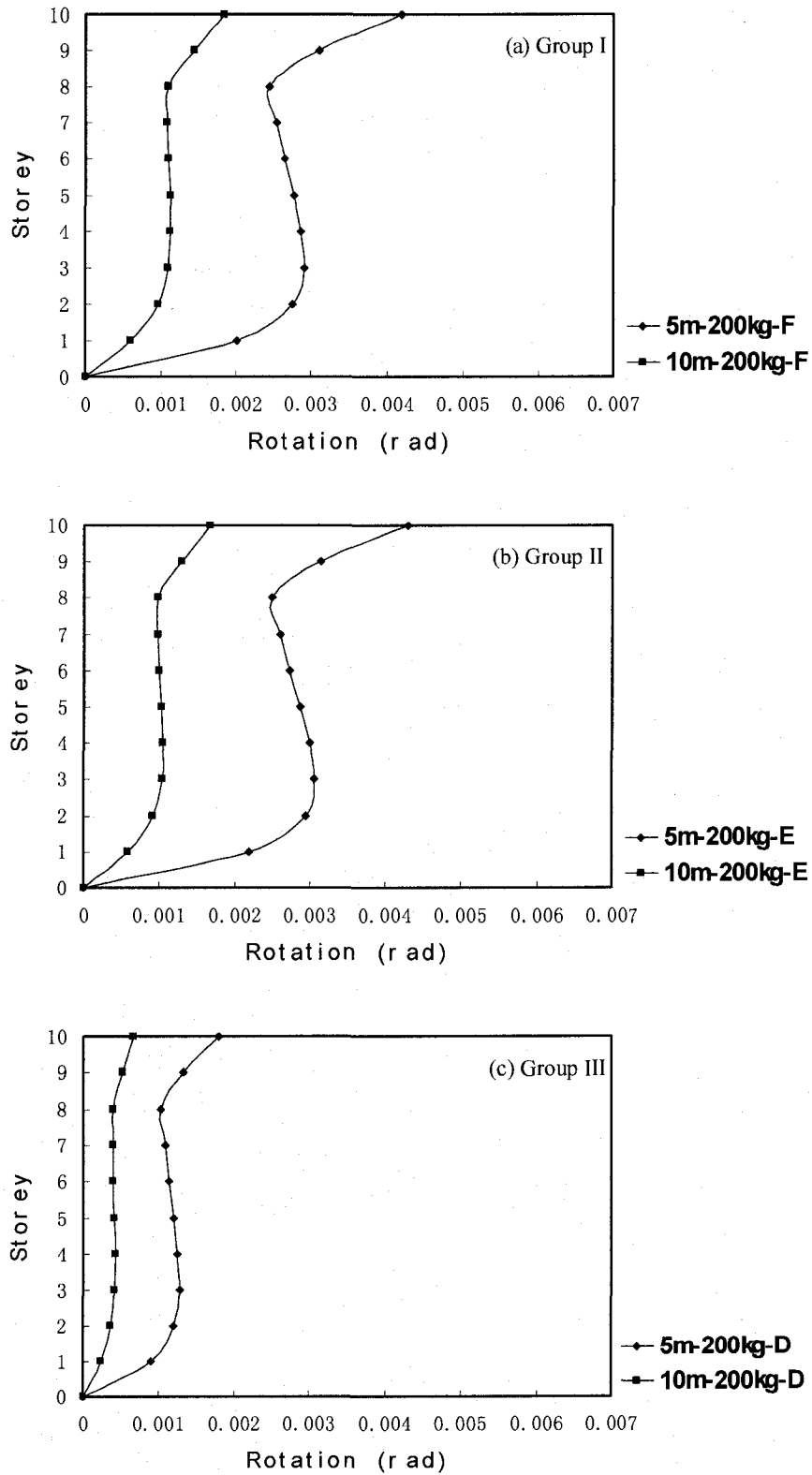


Figure 4.16 Effects of the distance from explosion centre to building on building rotation in radians for charge weight of 200 kg

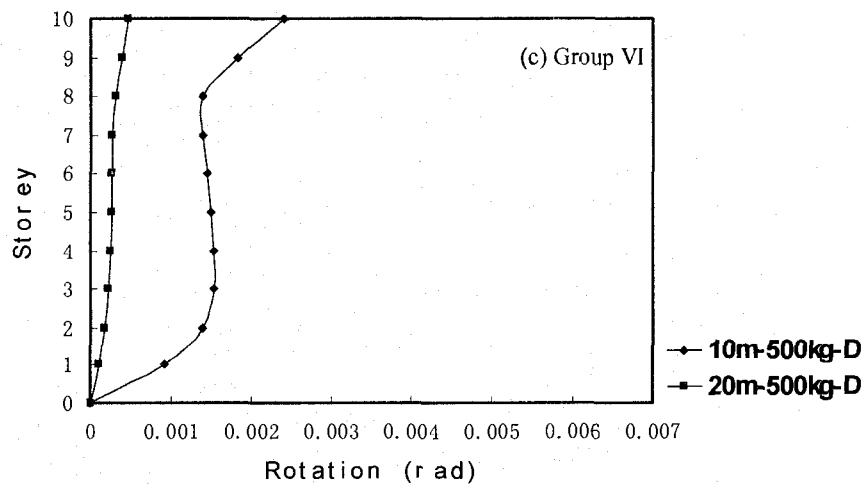
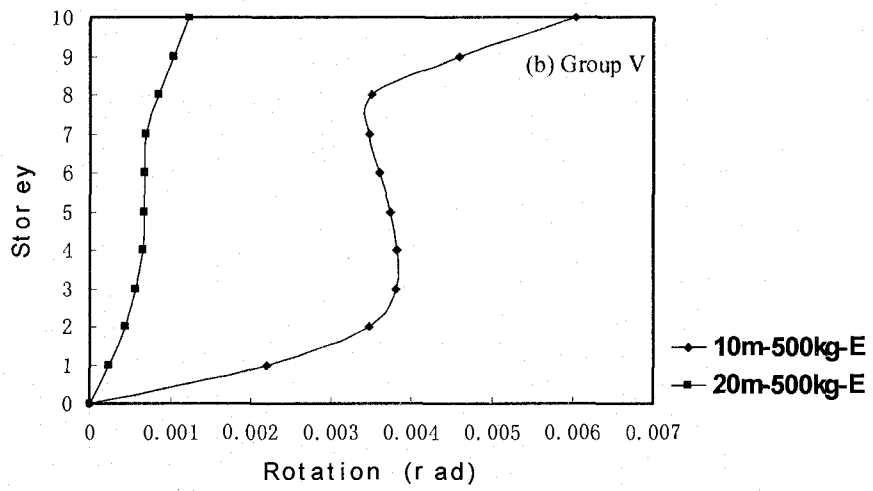
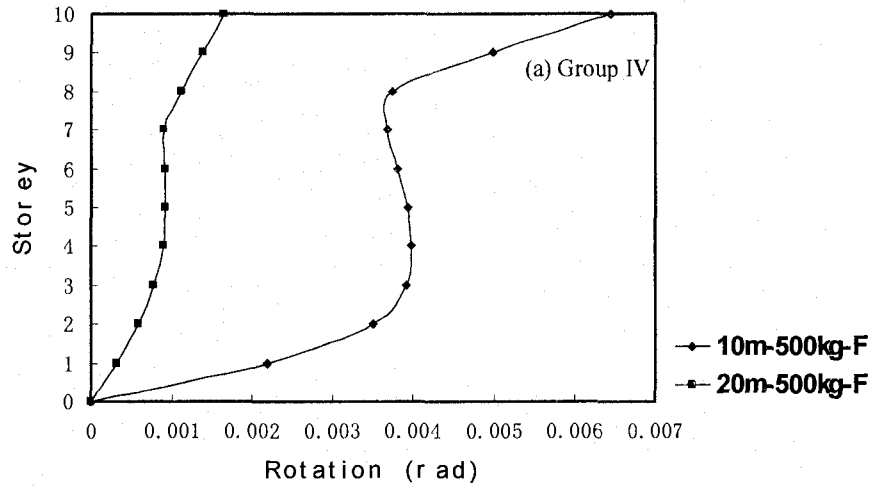


Figure 4.17 Effects of the distance from explosion centre to building on building rotation in radians for charge weight of 500 kg

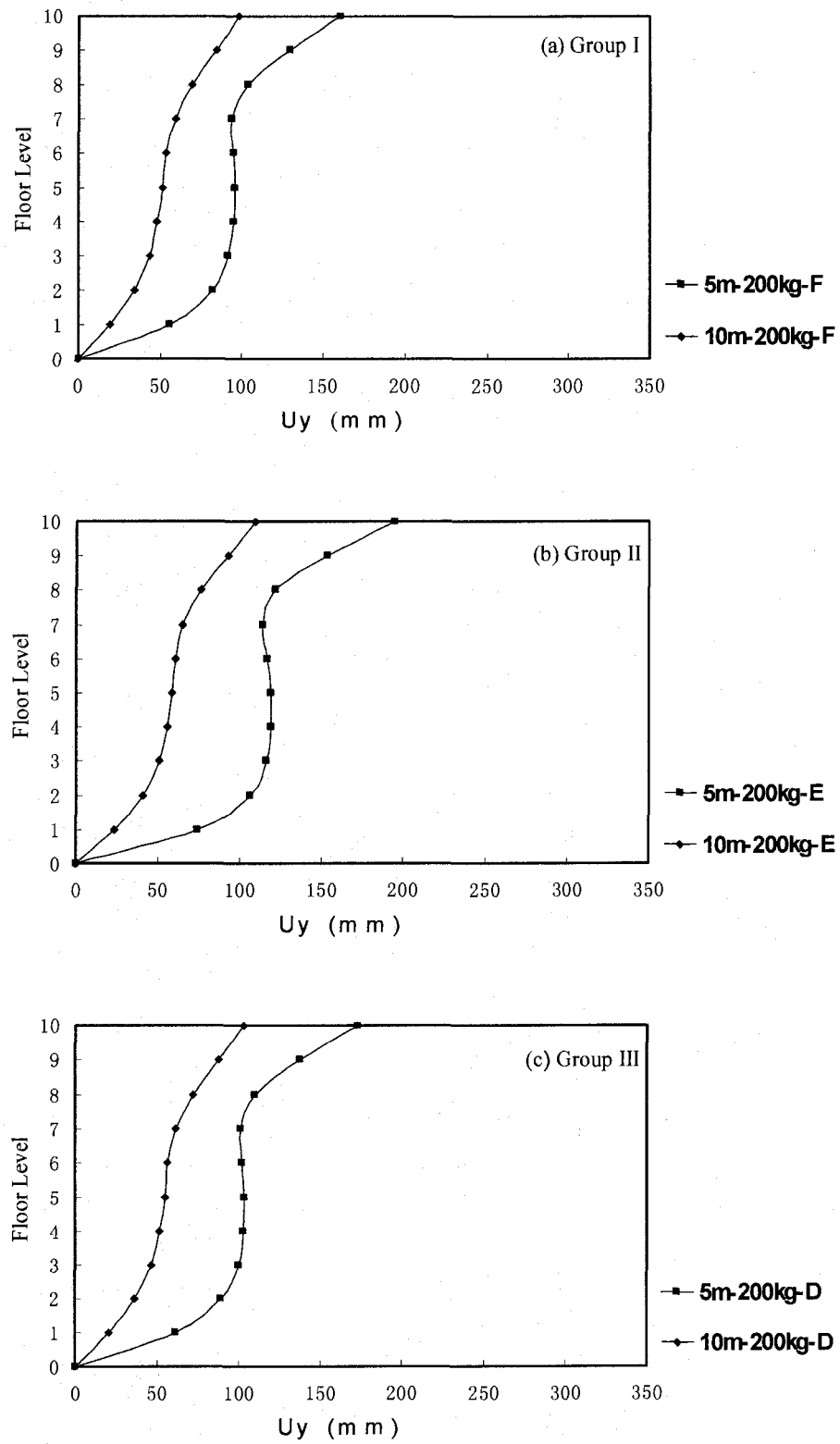


Figure 4.18 Effects of the distance from explosion centre to building on maximum horizontal displacement of joints for charge weight of 200 kg

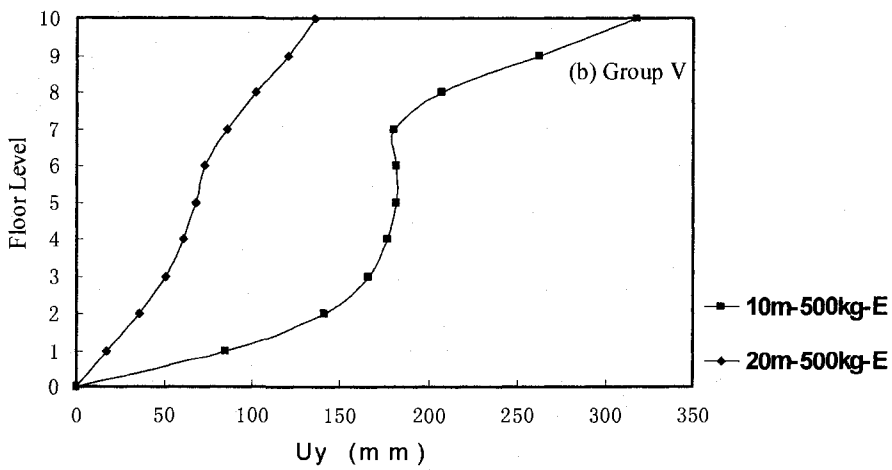
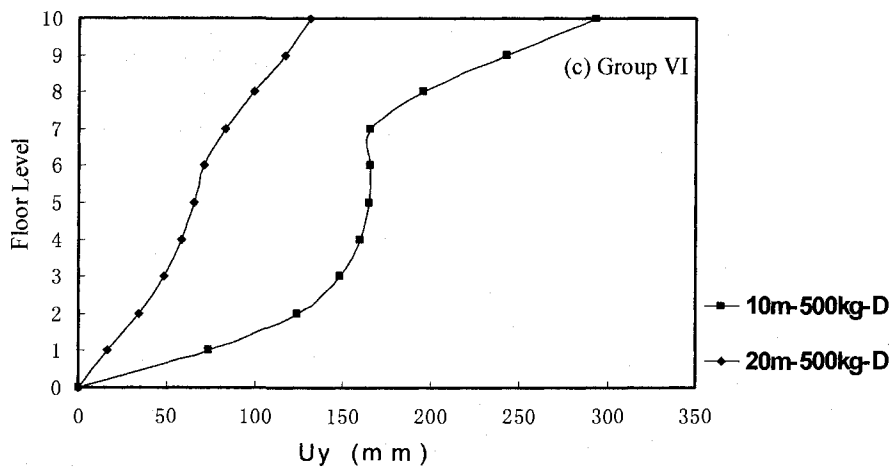
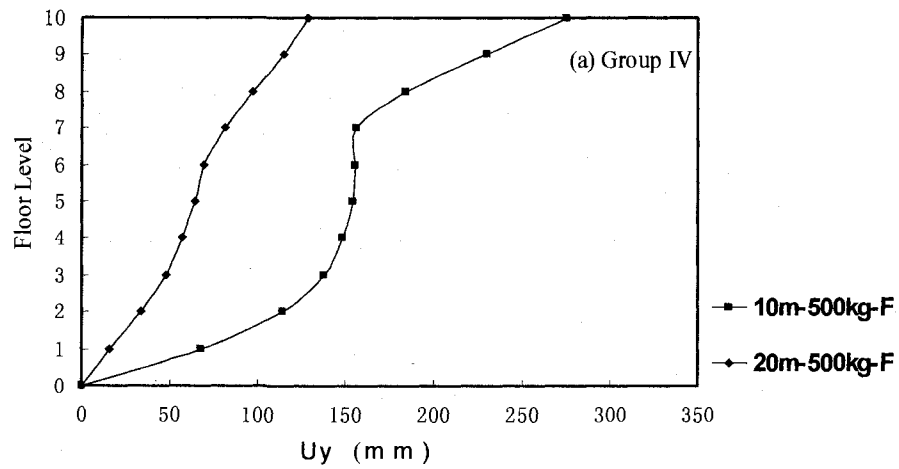


Figure 4.19 Effects of the distance from explosion centre to building on maximum horizontal displacement of the centre of rigidity for charge weight of 500 kg

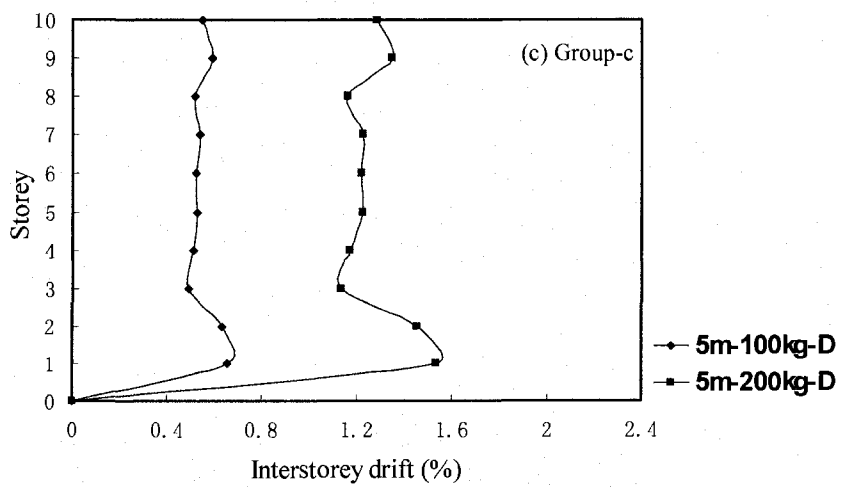
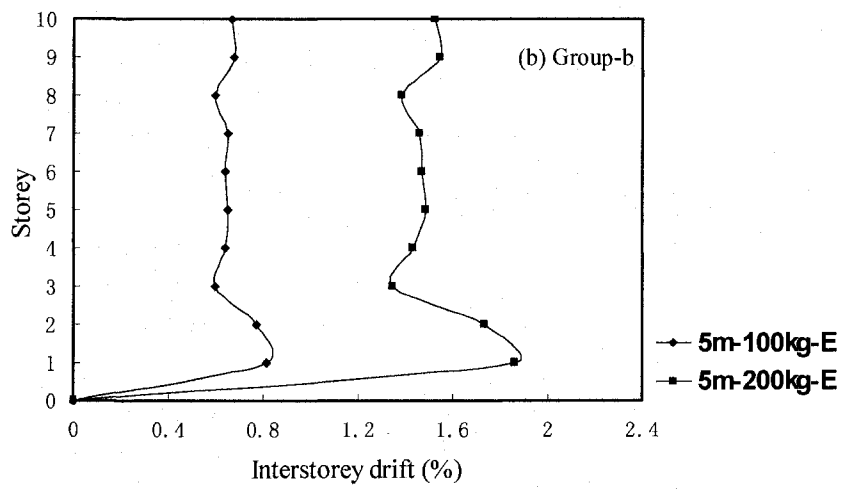
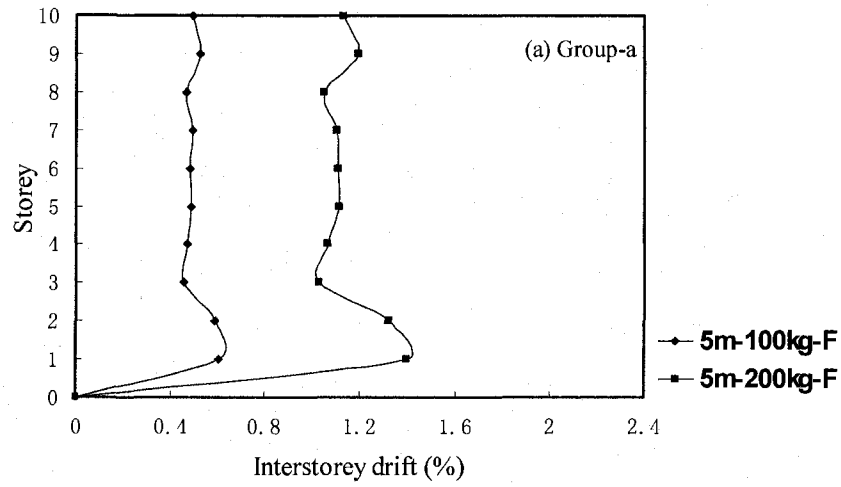


Figure 4.20 Effects of charge weight on interstorey drifts (explosion at 5 m)

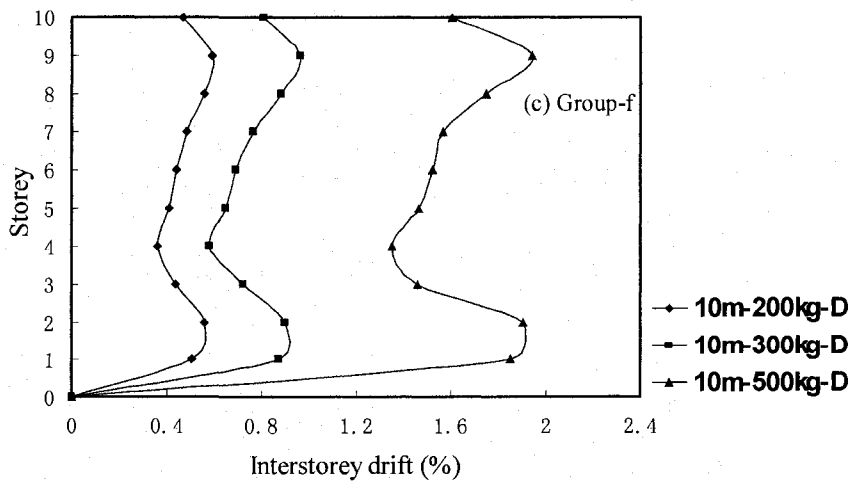
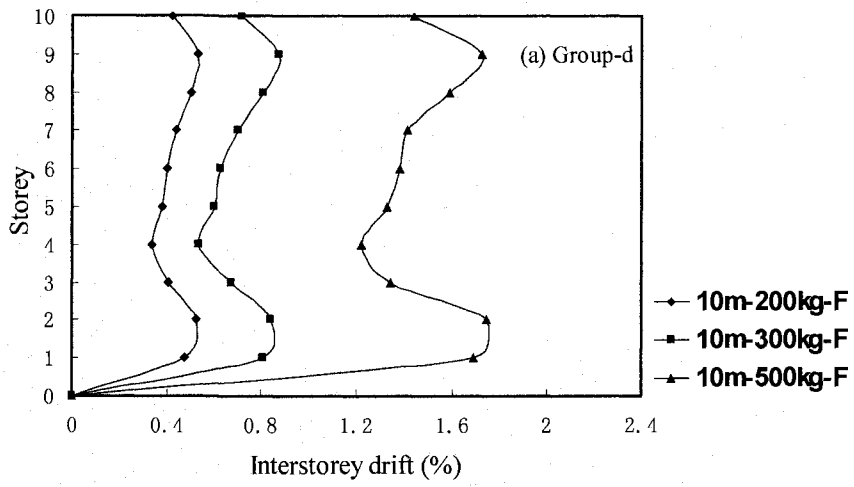
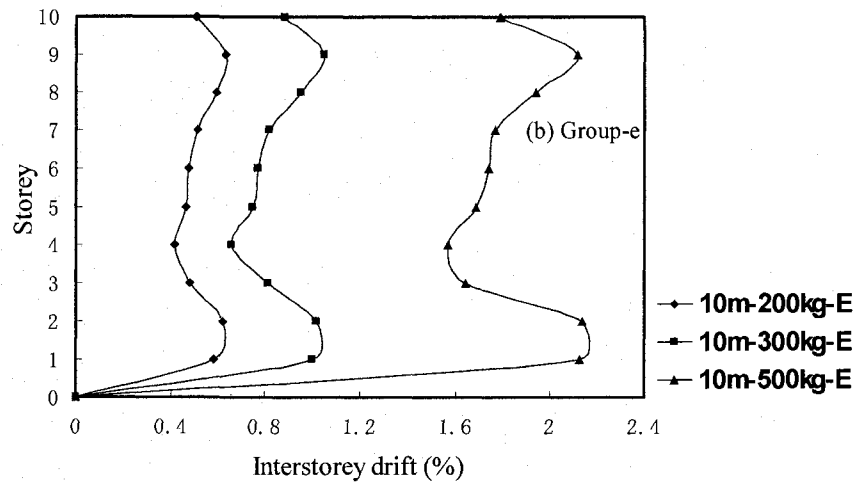


Figure 4.21 Effects of charge weight on interstorey drifts (explosion at 10 m)

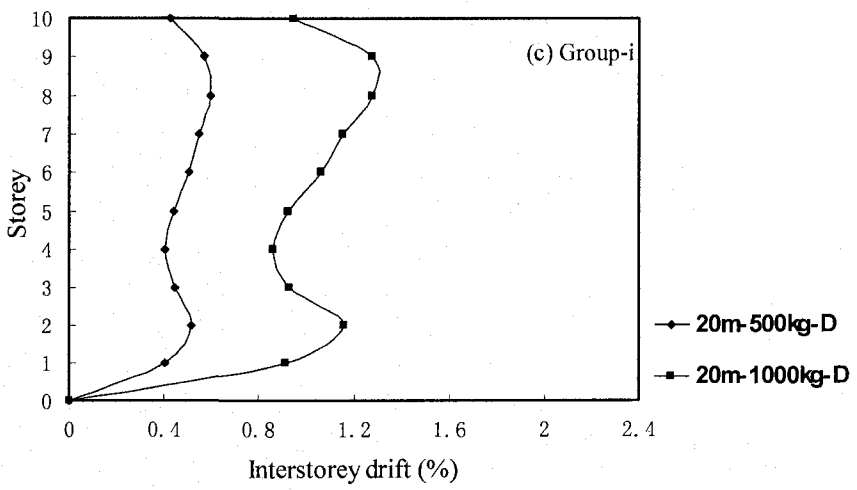
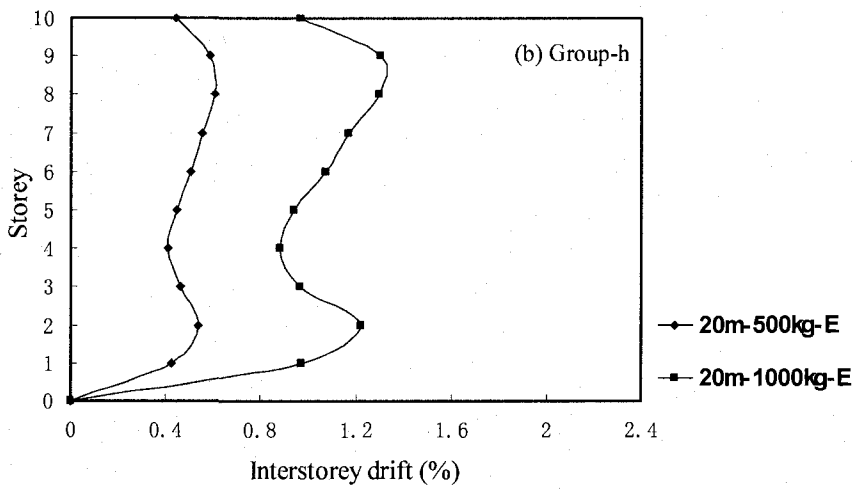
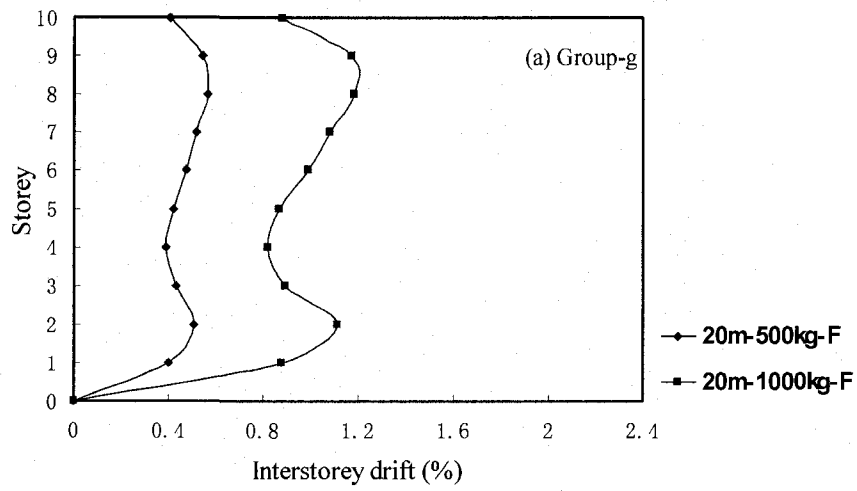


Figure 4.22 Effects of charge weight on interstorey drifts (explosion at 20 m)

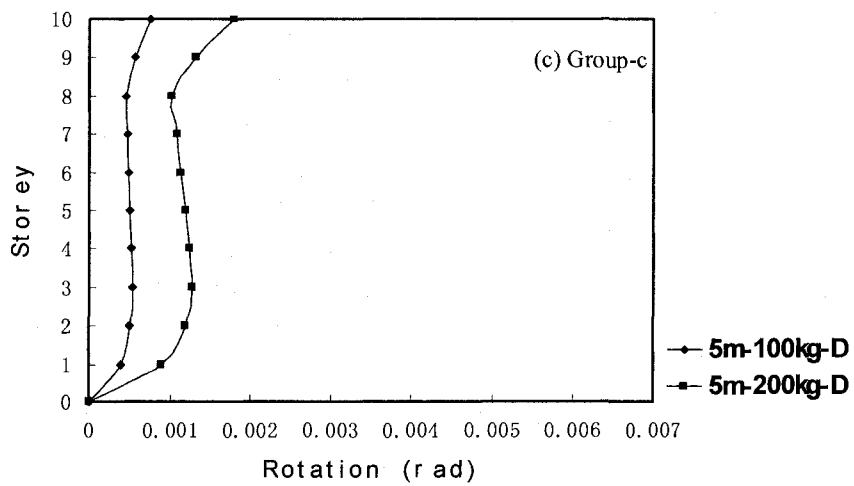
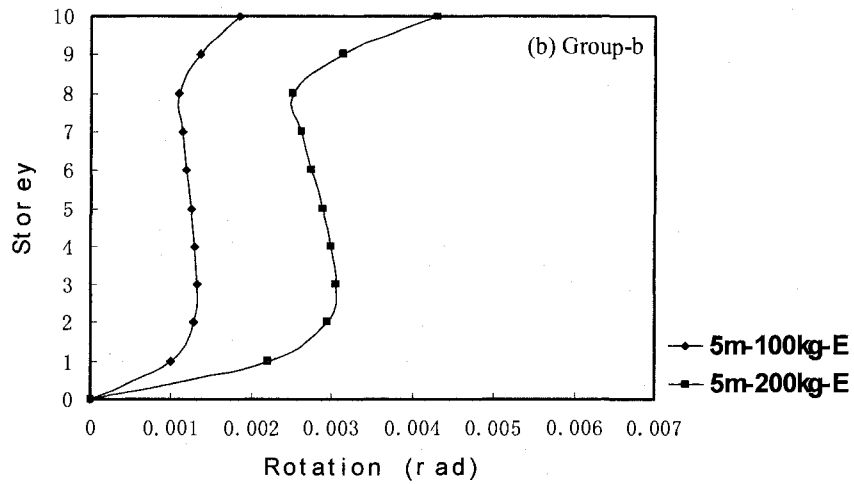
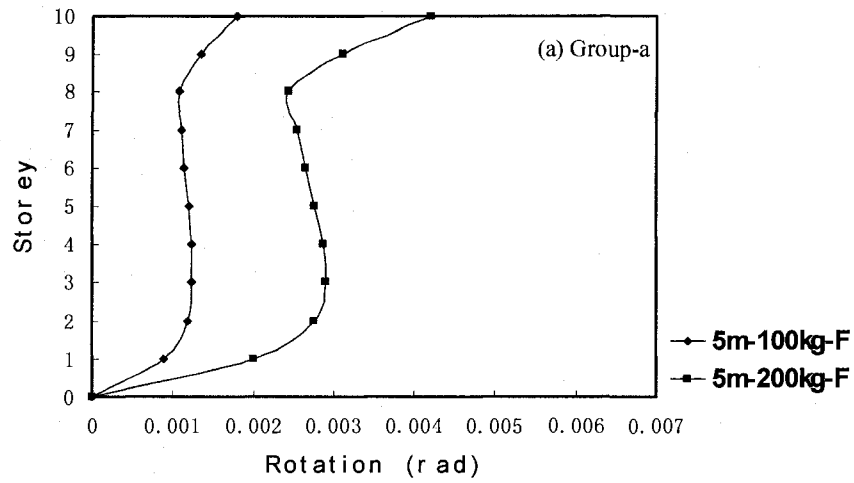


Figure 4.23 Effects of charge weight on maximum building rotation in radians (explosion at 5 m)

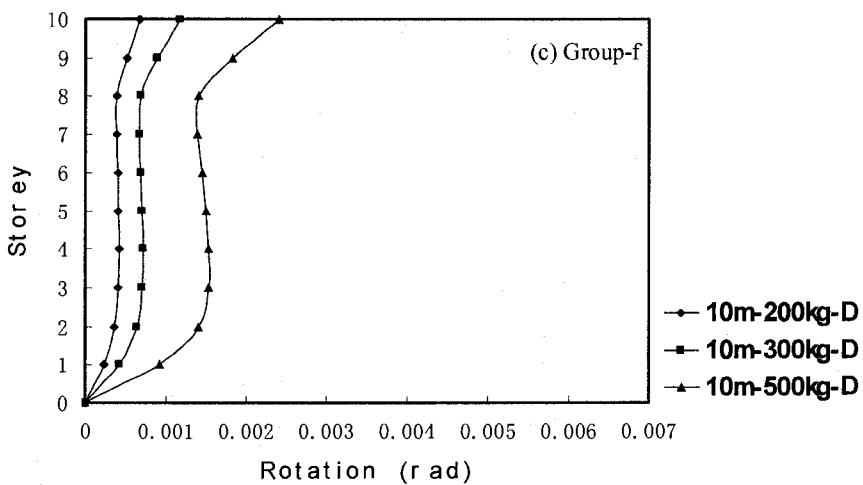
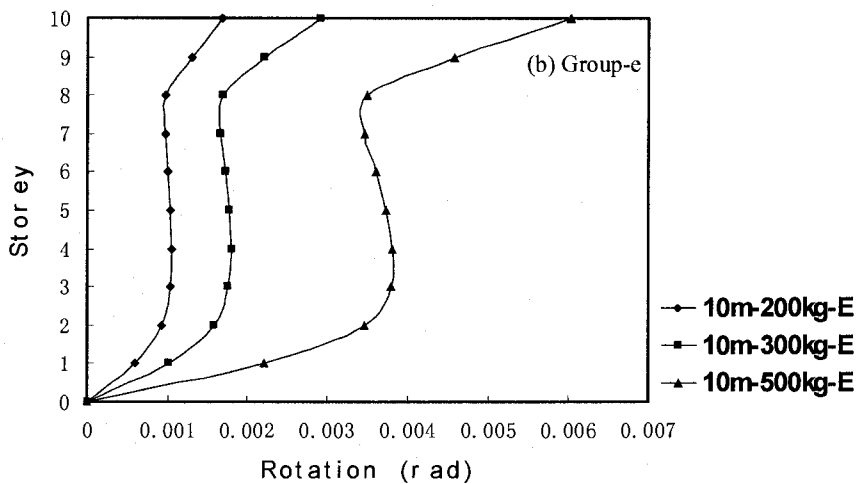
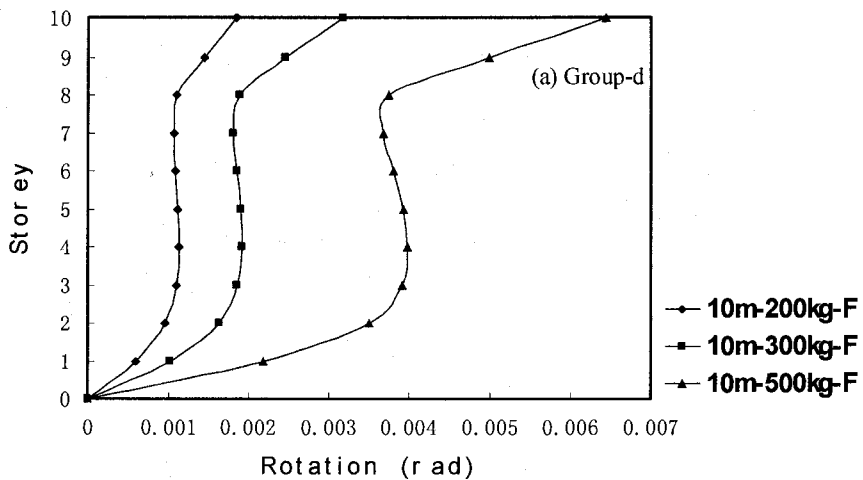


Figure 4.24 Effects of charge weight on maximum building rotation in radians (explosion at 10 m)

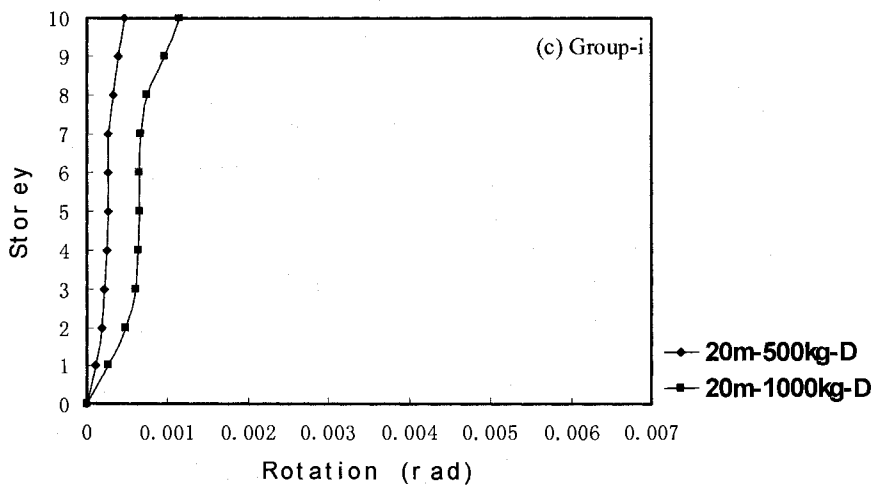
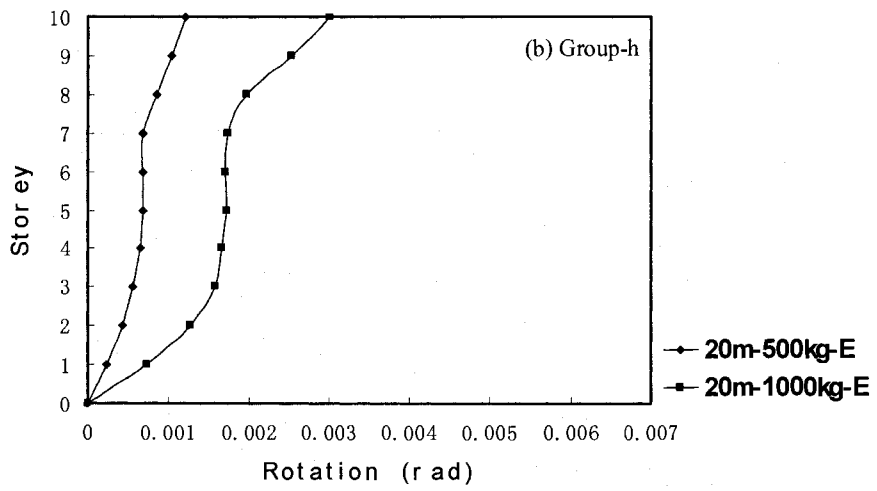
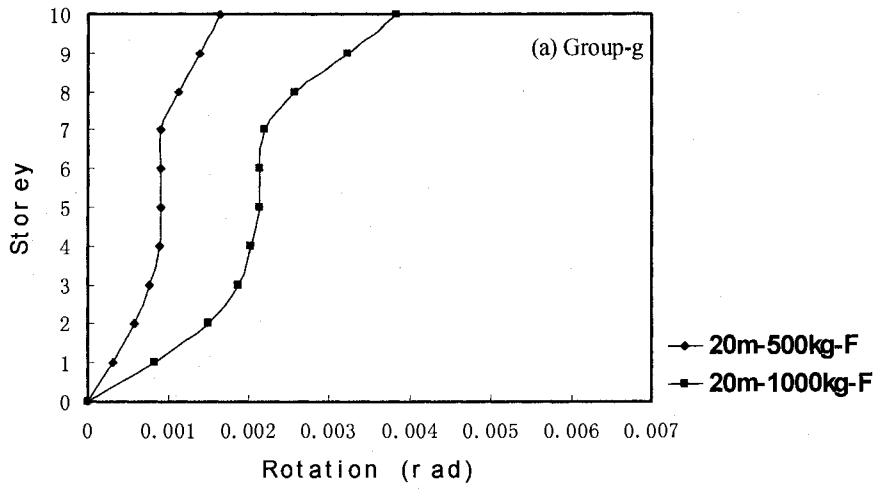


Figure 4.25 Effects of charge weight on maximum building rotation in radians (explosion at 20 m)

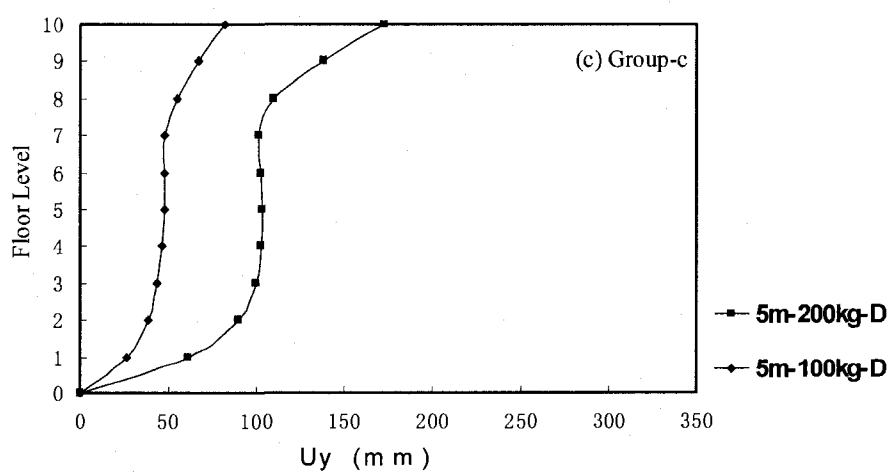
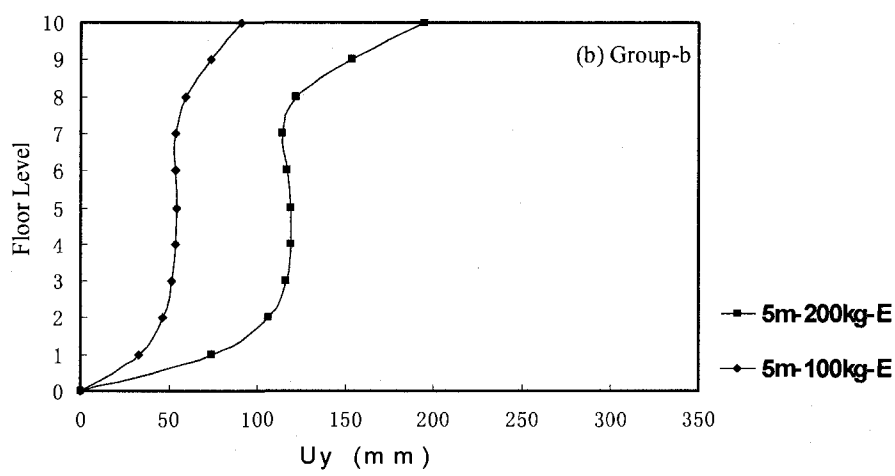
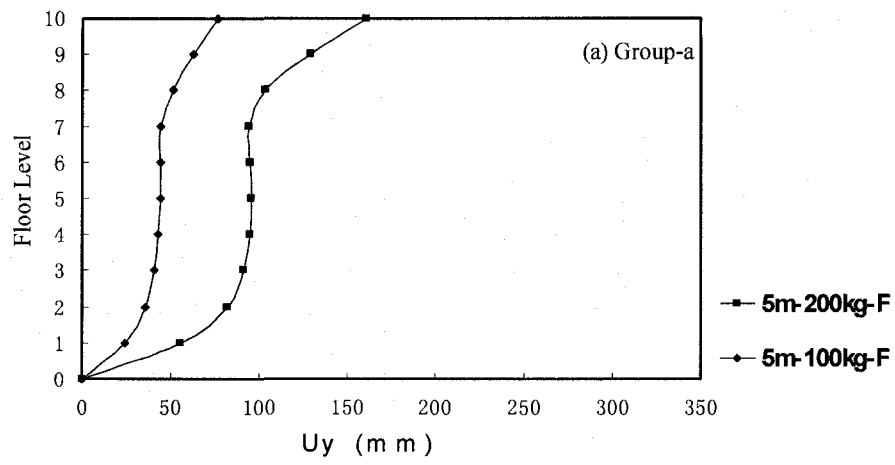


Figure 4.26 Effects of charge weight on maximum displacement of joints (explosion at 5 m)

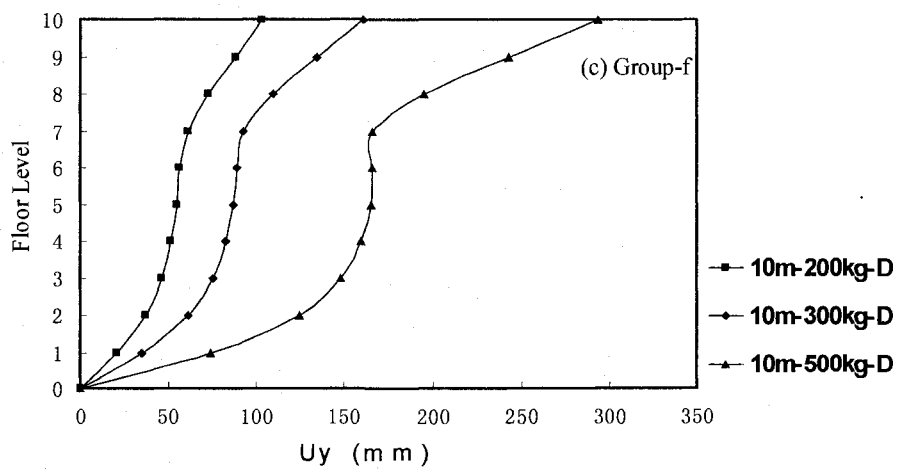
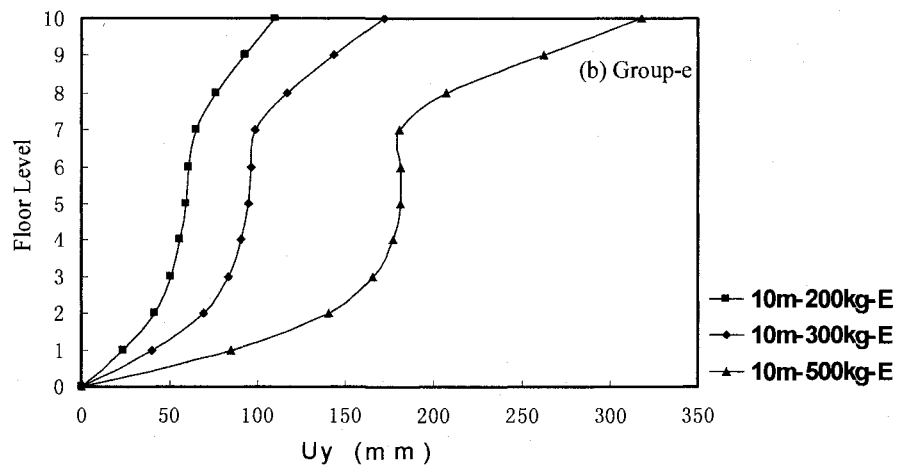
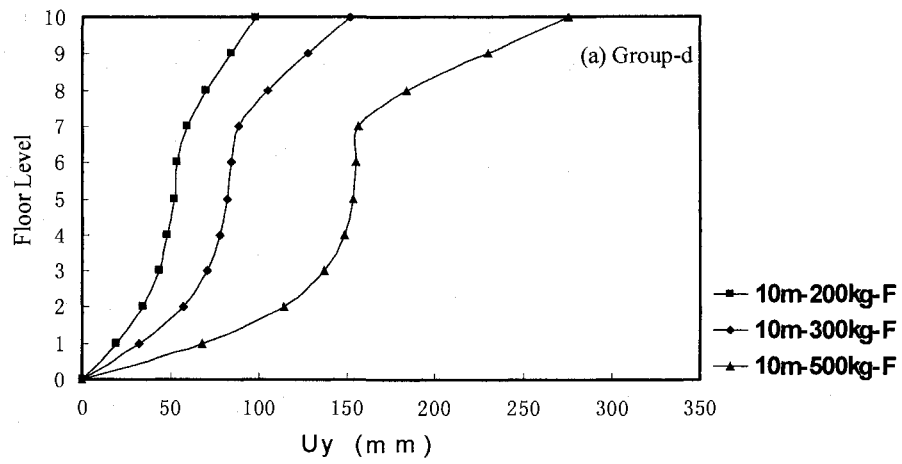


Figure 4.27 Effects of charge weight on maximum displacement of joints (explosion at 10 m)

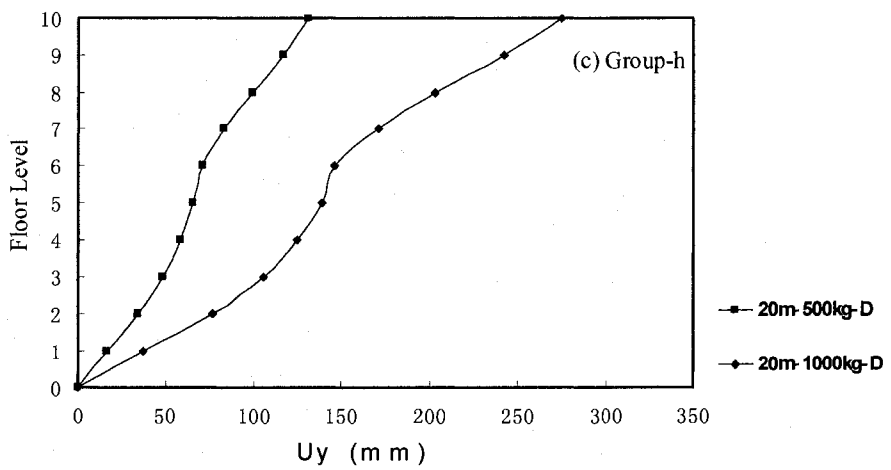
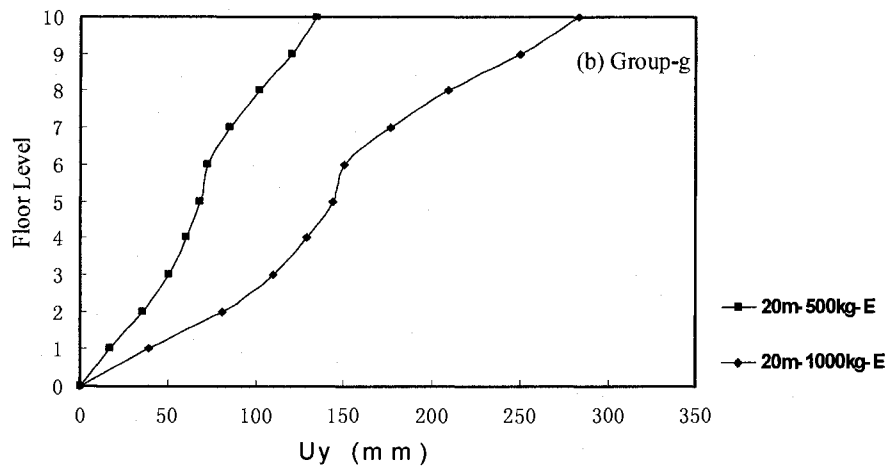
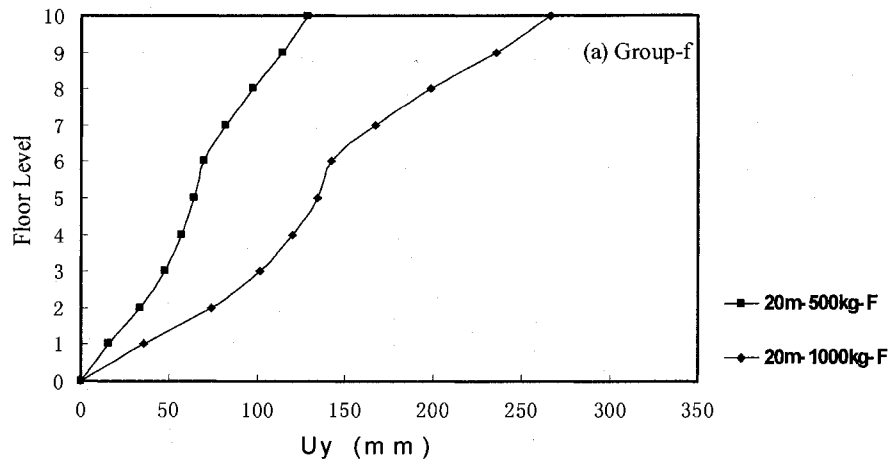


Figure 4.28 Effects of charge weight on maximum displacement of joints (explosion at 20 m)

Chapter 5

Summary and Conclusions

5.1 Summary and Conclusions

Torsional response of buildings occurs due to the eccentricity of resultant blast loads. This is also true for buildings with symmetric floor plans. The current investigation is intended to explore the significance of torsional response under different explosive charge, proximity to building, and load eccentricity combinations. Three dimensional dynamic time history analyses were conducted for this purpose. The blast load was idealized as a triangular impulsive forcing function. The analyses were limited to elastic response. A 10-storey reinforced concrete building was designed according to the 2005 edition of the National Building Code of Canada and the 2004 edition of the CSA Standard A23.3. The performance of the building was evaluated by considering interstorey drift, building rotation, horizontal displacement, and N-M Capacity/Demand ratios for columns.

The building was subjected to blast loads resulting from the detonations of 100 kg, 200 kg, 300 kg, 500 kg, and 1000 kg TNT (W), at distances of 5 m, 10 m, and 20 m away from the building (VD). The centre of explosion was placed at different eccentricities relative to the centre of rigidity of the building (HD). These locations coincided with column lines F, E, and D, shown in Figure 3.3.

The following conclusions can be drawn from this analytical research project:

- Maximum interstorey drifts are closely related to the eccentricity of blast loads. This effect can be neglected in practical applications for small size charges and distant explosions. For example, an explosion caused by less than 500 kg TNT at distances of 20 m and longer would produce very small interstorey drifts so that the effects of the eccentricity of blast loads could be neglected in practical applications.
- The eccentricity of blast load plays a significant role on torsional response. The rotation of the building caused by a detonation with 9 m of eccentricity relative to the centre of rigidity is significantly larger than that caused by a detonation at 3m of eccentricity.
- The pure translational response, represented by the displacement of the geometric centroid of building, decreases with the eccentricity of load. An explosion closer to the centre of rigidity (considered in the plan of a building), generates much larger translational deflections than an explosion farther from the centre of rigidity.
- The maximum displacement of each joint (i.e. U_y) varies with the eccentricity of load. The rate of increase in joint displacements is different at different locations of beam-column joints within the building plan because of the interaction of translational and rotational response.
- The perimeter columns, especially the corner columns are the most critical under

blast loads with centre of explosion having eccentricity relative to the centre of rigidity of the building. The most critical location of explosion centre is near column line E, affecting full tributary area of transverse bay. As the eccentricity of load increases further (say to column line F) the surface area that is affected by the explosion, the structural response reduces even though the torsional effects increase.

- The relative location of explosion (eccentricity of the resultant blast load) has a significant effect on the magnitude of torsional response. This can be estimated by evaluation demand-capacity ratios (DCR). The number of overstressed columns with DCR exceeding 1.0 is substantially influenced by the eccentricity of blast loads.
- Structural response decreases drastically with the increase of distance of explosion from the building. For the reinforced concrete building considered in this study, a detonation distance of 20 m and larger could be considered as a secure distance for bombs with equivalent TNT mass of 500 kg and smaller. However, when the distance decreases to 10 m, the same size bomb-blast can result in a total structural collapse because of the damage of entire first storey columns. The same destruction can be expected under reduced charge mass of 200 kg if detonations take place at 5m or closer distances. A 200 kg bomb at 10 m can produce partial damage or local failures in structures with similar characteristics as the 10-storey reinforced concrete frame building considered.

- The increase in interstorey drift ratio due to a decrease in distance of explosion can be estimated by finding the ratio of distances multiplied by a constant. This constant was found to vary between 4.0 and 6.0 for 200 kg charge weight and 6.0 to 8.0 for 500 kg charge weight.
- For a given detonation distance, the ability of the building considered in this investigation to resist bomb blast is strongly affected by the amount of explosives. The structural response may be in the elastic range when a 500 kg explosive is detonated at 20m of distance. If the weight of explosive increases to 1000 kg (while detonating the same distance of 20 m), the entire first-storey columns are overstressed, potentially resulting in total structural collapse.
- Maximum structural response (maximum interstorey drifts, maximum transverse deflections and maximum torsional rotations) is approximately proportional to the amount of explosives detonated. For example, the detonation of 1000 kg TNT at 20 m distance produces approximately twice the response when compared with the detonation of 500 kg TNT at the same 20 m distance.

Based on the above observations, it is important to note that structural response is not only affected by the charge weight and distance of explosion, but also by the torsional effects. Because each structure has its unique structural properties, behave differently, it is important to conduct three-dimensional analyses to assess the vulnerability of buildings to blast loads.

5.2 Recommendation for Future Research

The investigation of the vulnerability of buildings to blast loading is a new field, and extensive research is needed. While many available documents and advanced computer programs are mainly restricted to military use only, substantial research is necessary to satisfy the requirements for the design of civilian structures. Based on the literature review and the experience gained from this study, the following topics are recommended for the near future research.

* A substantial number of additional analyses on the torsional behaviour of buildings under blast loading need to be conducted by considering buildings with different structural systems and configurations to establish relationships between structure and the three factors which directly determine the quantities of blast loads, including; i) explosion location relative to building, ii) distance of explosion to building, and iii) TNT charge weight.

* Software and the methods for analyzing the vulnerability of buildings to blast loads should be developed. Under intensive blast loading, most elements of a structure designed for earthquake areas would likely perform elasto-plastic or inelastic, but the existing methods and corresponding commercial software only can deal with elastic response in 3D or the inelastic response in 2D. Neither of these two capabilities truly simulate structural behaviour beyond the elastic stage.

Reference

ASCE.1985. *Design of blast structures to resist nuclear weapons effects*, Manual No.42. American Society of Civil Engineers, New York.

ASCE.1997. *Design of blast resistant building in petrochemical facilities*. American Society of Civil Engineers, Reston, Virginia.

ASCE.1999. *Structural design for physical security- state of the practice*. American Society of Civil Engineers, Reston, Virginia.

ASCE, 2002. *Structural Design for Physical Security*. American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, Virginia, 20191-4400, U.S.A..

Baker, W.E., 1983. *Explosions in Air*. Wilfred Baker Engineering, San Antonio, TX, 2nd printing.

Baker, W.E., Cox, P.A., Westin, P.S., Kulesz, J.J., and Strehlow, R.A., 1983. *Explosions Hazards and Evaluation*, Elsevier, New York, USA.

Biggs, J.M., 1964. *Introduction to Structural Dynamics*. McCraww-Hill, New York, USA.

Burns, J., Abruzzo, J., and Tamaro, M, 2003. *Structural Systems for Progressive Collapse Prevention*. The National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C..

Carino, J. Nicholas and Lew, H.S, Sept. 10, 2001. *Application of Seismic Rehabilitation Technologies to Mitigate Blast-Induced Progressive Collapse*. The Workshop organized jointly by the National Institute of Standards and Technology (NIST) and the General Services Administration (GSA). U.S. Department of Commerce, Oakland Ca.

Choi, H-J. and Krauthammer, T., December 2003. *Investigation of Progressive Collapse Phenomena in a Multistory Building*. Proc. First International Conference on Design and

Analysis of Protective Structures against Impact/Impulsive/Shock Loads, Tokyo.

Clough, R.W., and Penzien, J., 1993. *Dynamics of Structures*, 2nd ed., McCraw-Hill, New York, USA.

Crawford, John, 2003. *Retrofit Methods to Mitigate Progressive Collapse*. The National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C..

CSA A23.3. 2004. *Standard-Design of concrete structures*. Canadian Standards Association, Mississauga, Ontario, Canada.

CSI, ETABS V8.0, 2002. *Concrete Frame Design Manual*, Computer and Structures, Inc, Berkeley, California, USA.

Dincer, E.2003. Seismic drift demands in reinforced concrete structures. M.A.Sc. thesis, Department of Civil Engineering, University of Ottawa, Ottawa, ON.

DOD, July 2002. *DOD Minimum Antiterrorism Standards for Buildings*. UFC 4-010-01 31, Department of Defence of the United States of America.

Dusenberry, D. O, 2003. *Review of Existing Guidelines and Provisions Related to Progressive Collapse*. The National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C..

FEMA 386-7, September 2003. *Integrating Mad-Made Hazard into Mitigation Planning*. Federal Emergency Management Agency (FEMA). The U.S. Department of Homeland Security.

FEMA 426, December 2003. *Reference Manual to Mitigate Potential Terrorist Attacks Against Buildings*. Federal Emergency Management Agency (FEMA). The U.S. Department of Homeland Security.

FEMA 428, December 2003. *Primer to Design Safe School Projects in Case of Terrorist Attacks*. Federal Emergency Management Agency (FEMA). The U.S. Department of

Homeland Security.

FEMA 427, December 2003. *Primer for Design of Commercial Buildings to Mitigate Terrorist Attacks*. Federal Emergency Management Agency (FEMA). The U.S. Department of Homeland Security.

Gross, J. and McGuire, W., Jan. 1983. *Progressive Collapse Resistant Design.*” Journal of Structural Engineering, ASCE., Vol.109, No.1, Jan. 1983, pp.1-15.

GSA, June 2003. *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects*. The U.S. General Services Administration, the Office of the Chief Architect,.

GSA, March 2005. *Facilities Standard for the Public Buildings Service*. The U.S. General Services Administration, the Office of the Chief Architect.

Jagmohan L. Humar, 2002. *Dynamics of Structures*. Book News, Inc., Portland, OR.

Kanaan, A.E., Powell, G.H., April 1973. “DRAIN-2D: General Purpose Computer Program for Dynamic Analysis of Inelastic Plane Structures” University of California, Berkeley, California.

Longinow, A., and Mniszewski, K.R., 1996. *Protecting buildings against vehicle bomb attacks; practice periodical on structural design and construction*. ASCE, Reston, VA, Feb. 1996, pp. 51-53.

Mays, G.C., P.D.(editors) 1995. *Blast effects on building- Design of Buildings to Optimize resistance to blast loading*. Thomas Telford Service ltd., London , England.

National Research Council, 1995. *Protecting Buildings from bomb damage*, National Academy Press, Washington, D.C..

National Academy of Science, National Research Centre. 1995. *Protecting building from bomb damage- Transfer of blast-effects mitigation technologies from military to civilian application*. National Academy Press, Washington, D.C..

NAVFAC. 1990. Army Manual TM5-1300, *Structures to resist the effects of accidental explosions*. Department of the Army, the Navy, and the Air Force, Washington, D.C..

NBCC. 2005. *National Building Code of Canada*. Associate Committee on the National Building Code. National Research Council of Canada, Ottawa, Canada.

Nove Naumoski and John Balazic, 2004, *Protection of buildings from blast effects, A State-of-the-Art Report*, Architectural and Engineering Resources Directorate, Architectural and Engineering Services Real Property Branch, Public Works and Government Services Canada, Gatineau, Quebec, Canada.

Smilowitz, R., 2003. *Analytical Tools for Progressive Collapse Analysis*. The National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C..

T. Ozbakkaloglu, N. Naumoski and M. Saatcioglu, 2005, *Response of reinforced concrete frame buildings to blast loading*, 1st CSCE Specialty Conference on Infrastructure Technologies, Management and Policy, Toronto, Ontario, Canada.