



uOttawa

L'Université canadienne
Canada's university

FACULTÉ DES ÉTUDES SUPÉRIEURES
ET POSTDOCTORALES



FACULTY OF GRADUATE AND
POSTDOCTORAL STUDIES

Sahar Mirhosseini

AUTEUR DE LA THÈSE / AUTHOR OF THESIS

M.A.Sc. (Civil Engineering)

GRADE / DEGREE

Department of Civil Engineering

FACULTÉ, ÉCOLE, DÉPARTEMENT / FACULTY, SCHOOL, DEPARTMENT

Progressive Collapse Analysis of Reinforced Concrete Building under Blast Loading

TITRE DE LA THÈSE / TITLE OF THESIS

Dr. Murat Saatcioglu

DIRECTEUR (DIRECTRICE) DE LA THÈSE / THESIS SUPERVISOR

CO-DIRECTEUR (CO-DIRECTRICE) DE LA THÈSE / THESIS CO-SUPERVISOR

EXAMINATEURS (EXAMINATRICES) DE LA THÈSE / THESIS EXAMINERS

Dr. S. Foo

Dr. D. Palermo

Dr. B. Isgor

Dr. N. Naumoski

Gary W. Slater

Le Doyen de la Faculté des études supérieures et postdoctorales / Dean of the Faculty of Graduate and Postdoctoral Studies

Progressive Collapse Analysis of Reinforced Concrete Buildings under Blast Loadings

by

Sahar Mirhosseini

In partial fulfillment
Of the requirements for the degree of
Master of Applied Science
in
Structural Engineering



Department of Civil Engineering
University of Ottawa
Ottawa, Canada
September 2007



Library and
Archives Canada

Bibliothèque et
Archives Canada

Published Heritage
Branch

Direction du
Patrimoine de l'édition

395 Wellington Street
Ottawa ON K1A 0N4
Canada

395, rue Wellington
Ottawa ON K1A 0N4
Canada

Your file Votre référence
ISBN: 978-0-494-41673-0
Our file Notre référence
ISBN: 978-0-494-41673-0

NOTICE:

The author has granted a non-exclusive license allowing Library and Archives Canada to reproduce, publish, archive, preserve, conserve, communicate to the public by telecommunication or on the Internet, loan, distribute and sell theses worldwide, for commercial or non-commercial purposes, in microform, paper, electronic and/or any other formats.

The author retains copyright ownership and moral rights in this thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without the author's permission.

AVIS:

L'auteur a accordé une licence non exclusive permettant à la Bibliothèque et Archives Canada de reproduire, publier, archiver, sauvegarder, conserver, transmettre au public par télécommunication ou par l'Internet, prêter, distribuer et vendre des thèses partout dans le monde, à des fins commerciales ou autres, sur support microforme, papier, électronique et/ou autres formats.

L'auteur conserve la propriété du droit d'auteur et des droits moraux qui protègent cette thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

In compliance with the Canadian Privacy Act some supporting forms may have been removed from this thesis.

Conformément à la loi canadienne sur la protection de la vie privée, quelques formulaires secondaires ont été enlevés de cette thèse.

While these forms may be included in the document page count, their removal does not represent any loss of content from the thesis.

Bien que ces formulaires aient inclus dans la pagination, il n'y aura aucun contenu manquant.


Canada

Acknowledgements

Especial thanks to my supervisor, Professor Murat Saatcioglu, for his support, guidance and encouragement during the period of the course of this study.

I would like to express my grateful appreciation to my husband Farhad Khosrojerdi and my son Ali for their patience and their understanding during these years. I also want to thank my parents for their nonstop support and encouragement in my entire life. Many thanks are extended to my parents-in-law for their support and encouragement.

Abstract

During the last two decades, a significant number of embassies, commercial centers, governmental structures, industrial facilities, and residential buildings have been attacked by explosives. Consequently, blast-resistant design of structures has gained importance in recent years to minimize structural damage and threat to life safety. Some guidelines have been developed for this purpose, based on previously conducted research. The U.S. General Services Administration (GSA) report is one of these guidelines that provide practical solutions to blast-related engineering problems, including requirements for analysis and design for prevention of progressive collapse in structures. The guidelines also help establish the potentials for progressive collapse in existing buildings.

Three multi-storey reinforced concrete frame buildings were considered in the current research project to investigate their vulnerability against blast-triggered progressive collapse. Computer software DRAIN R/C was employed for modeling and structural analysis of buildings. Among the parameters considered was building height (or number of stories). This was done by considering a 5-storey and a 10-storey R/C building with 6m span length in each bay. In addition, the effect of span length on progressive collapse was investigated by considering a 5-storey building with 6m and 9m spans. All buildings consisted of a symmetrical floor plan and moment resisting frames. Two types of analysis were conducted under gravity loads according to the GSA report specifications, i) elastic analysis; where the members were assumed to remain elastic by specifying an unrealistically high moment capacity and ii) inelastic analysis; where the members were allowed to yield shortly before reaching their nominal (design) capacities. The analyses were conducted by removing a first-storey column and investigating the consequential increase in force and deformation demands in adjacent members. The results from linear analysis indicate that the loss of a corner column may not necessarily trigger progressive collapse, while the removal of an exterior edge column and interior column would result in structural collapses. Results from nonlinear analysis indicate total collapse of all the

buildings if the total load of $2(DL + 0.25LL)$ is applied. But if the loading is reduced to $(DL + 0.5LL)$, the collapse occurs only in a building with an interior first-storey column removed.

Table of Content

<i>Acknowledgements</i>	II
<i>Abstract</i>	III
<i>Table of Content</i>	V
<i>List of Figures:</i>	VII
<i>List of Tables:</i>	XII
CHAPTER 1:	1
Introduction	1
1.1 Introduction	1
1.2 Objective	2
1.3 Scope	2
1.4 Literature Review and Lessons Learned from Previous Events	3
CHAPTER 2:	23
Selection and Design of Buildings	23
2.1 Description of Structures	23
2.1.1 Plan and Elevation Views and the Loading	24
2.1.2 Member Design	24
2.1.3 Requirements for Non-Seismic and Seismic Design in CSA A23.3	27
CHAPTER 3:	38
Analysis Procedure	38
3.1 DRAIN – R/C	38
3.2 Modeling	39
CHAPTER 4:	43
Elastic and Inelastic Progressive Collapse Analyses	43
4.1 General	43
4.2 General Services Administration (GSA) Criteria and Elastic Analysis	44
4.3 Inelastic Analysis	47

CHAPTER 5:	94
Summary and Conclusion	94
5.1 Summary	94
5.2 Conclusion	95
5.3 Recommendations for Future Research	97
REFERENCES:	98
APPENDIX	102
Input Data for DRAIN-R/C	102

List of Figures:

Figure 1.1: 1993 bombing of the World Trade Centre in New York City.....	17
Figure 1.2: 1995 bomb attack of the Murrah Federal Building in Oklahoma City	17
Figure 1.3: Schematic view of blast effect and resulting pressures on a building (Adopted from Ref. 21).....	18
Figure 1.4: Variation of blast pressure with time (Adopted from Ref. 21)	18
Figure 1.5: Variation of blast pressure with distance (Adopted from Ref. 21)	19
Figure 1.6: Cases to be considered for a frame building for collapse analysis (Adopted from Ref. 16).....	20
Figure 1.7: Cases to be considered for a building with walls for collapse analysis (Adopted from Ref. 16).....	21
Figure 1.8: Maximum allowable collapse area (Adopted from Ref. 16).....	22
Figure 2.1: Plan and elevation views of 5 storey reinforced concrete building with 6.0 m spans.....	32
Figure 2.2: Plan and elevation views of the 10 storey reinforced concrete building.....	33
Figure 2.3: Plan and elevation views of 5 storey reinforced concrete buildings with 9.0 m spans.....	34
Figure 2.4: Design details for frame elements of 5-storey and 10-storey reinforced concrete buildings with 6.0 m spans.....	35
Figure 2.5: Design details for frame elements of 5-storey reinforced concrete building with 9.0 m spans	36
Figure 2.6: Effect of loosing first storey column and change in moment demands	37
Figure 2.7: Drift capacities for ordinary and seismic designs	37
Figure 3.1: Create a file in DRAIN R/C	41
Figure 3.2: Defining distributed load.....	41
Figure 3.3: Defining stiffness properties in DRAIN R/C	42
Figure 3.4: Defining hinge properties in DRAIN R/C.....	42
Figure 4.1: Moment-Rotation Relationship for Elastic Analysis.....	52

Figure 4.2: Location of the removed column in each scenario; exterior column removal, corner column removal, and interior column removal.....	53
Figure 4.3: Linear response of 5-storey building for exterior column removal in short (above) and long (below) directions	54
Figure 4.4: Linear response of 5-storey building for corner column removal in short (above) and long (below) directions	55
Figure 4.5: Linear response of 5-storey building for interior column removal in short (above) and long (below) directions	56
Figure 4.6: a) Linear response of 10-storey building for exterior column removal in short direction	57
Figure 4.6 (Cont'd): b) Linear response of 10-storey building for exterior column removal in long direction	58
Figure 4.7: a) Linear response of 10-storey building for corner column removal in short direction	59
Figure 4.7 (Cont'd): b) Linear response of 10-storey building for corner column removal in long direction	60
Figure 4.8: a) Linear response of 10-storey building for interior column removal in short direction	61
Figure 4.8 (Cont'd): b) Linear response of 10-storey building for interior column removal in long direction	62
Figure 4.9: Linear response of 5-storey building with 9m span length for exterior column removal in short and long directions.....	63
Figure 4.10: Linear response of 5-storey building with 9m span length for corner column removal in short and long directions.....	64
Figure 4.11: Linear response of 5-storey building with 9m span length for interior column removal in short and long directions.....	65
Figure 4.12: Element modeling – Inelastic hinge for flexure	66
Figure 4.13: Moment – Rotation relationship for inelastic analysis.....	66
Figure 4.14: Inelastic analysis of 5-storey reinforced concrete building with 6.0 m spans under the removal of an exterior edge column	67

Figure 4.15: Inelastic analysis of 5-storey reinforced concrete building with 6.0 m spans under the removal of a corner column	68
Figure 4.16: Inelastic analysis of 5-storey reinforced concrete building with 6.0 m spans under the removal of an interior column.....	69
Figure 4.17: Inelastic analysis of 10-storey reinforced concrete building with 6.0 m spans under the removal of an exterior edge column	70
Figure 4.18: Inelastic analysis of 10-storey reinforced concrete building with 6.0 m spans under the removal of a corner column	71
Figure 4.19: Inelastic analysis of 10-storey reinforced concrete building with 6.0 m spans under the removal of an interior column.....	72
Figure 4.20: Inelastic analysis of 5-storey reinforced concrete building with 9.0 m spans under the removal of an exterior edge column	73
Figure 4.21: Inelastic analysis of 10-storey reinforced concrete building with 9.0 m spans under the removal of a corner column	74
Figure 4.22: Inelastic analysis of 10-storey reinforced concrete building with 9.0 m spans under the removal of an interior column.....	75
Figure 4.23: Formation of plastic hinges due to the loss of an exterior edge column in the short direction of 5-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection	76
Figure 4.24: Formation of plastic hinges due to the loss of an exterior edge column in the long direction of 5-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection	77
Figure 4.25: Formation of plastic hinges due to the loss of a corner column in the short direction of 5-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	78
Figure 4.26: Formation of plastic hinges due to the loss of a corner column in the long direction of 5-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	79
Figure 4.27: Formation of plastic hinges due to the loss of an interior column in the short direction of 5-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	80

Figure 4.28: Formation of plastic hinges due to the loss of an interior column in the long direction of 5-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	81
Figure 4.29: Formation of plastic hinges due to the loss of an exterior edge column in the short direction of 10-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection	82
Figure 4.30: Formation of plastic hinges due to the loss of an exterior edge column in the long direction of 10-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection	83
Figure 4.31: Formation of plastic hinges due to the loss of a corner column in the short direction of 10-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	84
Figure 4.32: Formation of plastic hinges due to the loss of a corner column in the long direction of 10-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	85
Figure 4.33: Formation of plastic hinges due to the loss of an interior column in the short direction of 10-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	86
Figure 4.34: Formation of plastic hinges due to the loss of an interior column in the long direction of 10-storey building with 6.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	87
Figure 4.35: Formation of plastic hinges due to the loss of an exterior edge column in the short direction of 5-storey building with 9.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection	88
Figure 4.36: Formation of plastic hinges due to the loss of an exterior edge column in the long direction of 5-storey building with 9.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection	89
Figure 4.37: Formation of plastic hinges due to the loss of a corner column in the short direction of 5-storey building with 9.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection.....	90

Figure 4.38: Formation of plastic hinges due to the loss of a corner column in the long direction of 5-storey building with 9.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection..... 91

Figure 4.39: Formation of plastic hinges due to the loss of an interior column in the short direction of 5-storey building with 9.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection..... 92

Figure 4.40: Formation of plastic hinges due to the loss of an interior column in the long direction of 5-storey building with 9.0m spans at: a) First yield b) 2% Deflection c) 4% Deflection..... 93

List of Tables:

Table 1.1: Levels of protection – new buildings (Adopted from Ref.14).....	14
Table 1.2: Levels of protection – existing buildings (Adopted from Ref. 14).....	15
Table 1.3: Minimum standoff distances (Adopted from Ref. 14).....	16
Table 2.1: Data used for Design of Buildings.....	29
Table 2.2: Design details for 5-storey moment resisting frame building with 6.0 m spans	29
Table 2.3: Design details for 10-storey moment resisting frame building with 6.0 m span	30
Table 2.4: Distribution of seismic force in each floor for both directions.....	30
Table 2.5: Load combinations (Adopted from NBCC 2000, Table 4.1.3.2.)	30
Table 2.6: Design load combinations.....	31
Table 2.7: Design details for 5-storey moment resisting frame building with 9.0 m span	31
Table 4.1: Drift Ratio from nonlinear analysis based on (DL + 0.5LL) loading.....	52
Table A.1: Distributed load pattern for 5-storey and 10-storey reinforced concrete buildings with 6m span length.	103
Table A.2: Distributed load pattern for 5-storey reinforced concrete buildings with 9m span length.	103
Table A.3: Stiffness properties of 5-storey reinforced concrete building:.....	103
Table A.4: Stiffness properties of 10-storey reinforced concrete building:.....	104
Table A.5: Stiffness properties of 5-storey reinforced concrete building with 9m span:	104
Table A.6: Hinge properties and rigid segments for 5-storey R/C building.....	105
Table A.7: Hinge properties and rigid segments for 10-storey R/C building.....	107
Table A.8: Hinge properties and rigid segments for 5-storey R/C building-9m span length	109

CHAPTER 1:

Introduction

1.1 Introduction

The majority of civil engineering infrastructure is not explicitly designed for blast loading. Therefore, they are vulnerable to blast-induced air pressures. Threats to civilian infrastructure, on the other hand, have increased during recent years due to increased frequency of terrorist bomb attacks. A significant number of embassies, commercial centers, governmental structures, industrial facilities, and residential buildings have been attacked by explosives during the last two decades. Consequently, the blast-resistant design of structures has gained importance in recent years to minimize structural damage and threat to life safety. One of the major consequences of bomb attacks, from structural performance perspective, is the possibility of progressive collapse. Progressive collapse is defined as the domino effect in failure that results from the failure of a single or a group of elements that propagates to other structural elements, precipitating a major or total collapse of the building. First-storey building columns are especially vulnerable to car or truck bombs. While the characteristics of explosions and their effects on buildings are complex topics that are currently being investigated, conventional structural analysis techniques can be employed to assess the impact of losing a vertical element such as a

first-storey building column on structural performance. The U.S. General Services Administration (GSA) has developed design guidelines that provide practical solutions to blast-related engineering problems, including the requirements for analysis and design against progressive collapse. The guidelines also help establish the potentials for progressive collapse in existing buildings. The research project presented in this thesis deals with progressive collapse potentials of typical reinforced concrete frame structures under blast-induced column damage scenarios.

1.2 Objective

The objective of the research project is to investigate the vulnerability of multi-storey reinforced concrete buildings to blast-induced progressive collapse. The objective includes the assessment of the feasibility of elastic and inelastic structural analyses techniques in establishing vulnerabilities associated with progressive collapse while also understanding the effects of the location of damaged columns, building height and span length as parameters.

1.3 Scope

The scope consists of analytical research involving the selection, design and analysis of building structures under different column damage scenarios triggered by explosions. Specifically the following steps constitute the scope of research:

- Literature review and understanding of the blast phenomenon and its effects on building structures with specific emphasis on blast-induced progressive collapse.
- Selection and design of 5 and 10-storey reinforced concrete frame structures, with different span lengths, for progressive collapse analyses.
- Modelling of frames for computer analysis.
- Review and use of computer program DRAIN-RC with abilities to handle inelastic force-deformation response.

- Elastic analyses of 5 and 10-storey buildings due to the removal of a first-storey i) corner column, ii) exterior edge column, and ii) interior column to assess the applicability of the GSA guidelines.
- Repeat of the above analyses for the 5-storey structures with different span lengths to assess the effect of span length (and hence the tributary gravity load) on the applicability of the GSA criterion.
- Modelling and inelastic analyses of structures to investigate inelastic deformation and strength requirements associated with first-storey column removal.
- Comparison of building performance between seismically designed and ordinary gravity frames in terms of their progressive collapse potentials.
- Preparation of a thesis and the presentation of results.

1.4 Literature Review and Lessons Learned from Previous Events

Many civilian facilities such as embassies, commercial centres, governmental buildings, industrial facilities and residential buildings have been targets for bomb attacks during the last two decades. The first event that attracted the attention of the structural engineering community occurred in 1968 in the UK, involving a precast concrete apartment building (Ronan Point Apartment Building). Accidental gas explosion in this high-rise building triggered the collapse of the corner portion of the building along its entire height. Although the Ronan Point event is an example of unintentional explosion, most previous devastations were caused by explosive devices loaded on vehicles, i.e. car or truck bombs, detonated remotely or by suicide drivers. The most notable attacks using car bombs include the attack on the World Trade Centre Tower in New York City on February 26, 1993, and on the Alfred P. Murrah Federal Building in Oklahoma City on April 19, 1995. The detonation of explosives in the underground garage, two levels below grade of the World Trade Centre killed six people and injured many others. The destruction was more extensive in Oklahoma City where the explosion resulted in 168 deaths and hundreds of injuries. In this attack, the bomb was placed three to five meters away from the building and caused progressive collapse of the entire front side of the building, which accounted for 80% of the deaths. The damage to the Murrah Federal

Building, and about 75 other buildings in the surrounding area was estimated to be 50 million U.S. dollars. These events led to the extensive investigation of building structures subjected to blast-induced impulsive loads.

According to Mallonee, et al., (1996), the total collapse of one side of the Murrah Building caused the majority of fatalities, whereas other injuries were caused mostly by flying debris. The observation made from other events, i.e. Khobar Towers and U.S. Embassy in Nairobi, indicated that flying debris could be the primary cause for deaths and injuries during explosions (U.S. Department of State 1999). These are the main reasons for the committee on Blast Mitigation for Structures Program (BMSP) to focus on lessening the probability of collapse in structures and reducing number of fatalities and injuries from flying debris. The same committee recommended a dual approach, consisting of testing and analysis, to investigate the phenomenon of blast-induced progressive collapse. These approaches involve the investigation of either total structural behaviour or components of structures. Because of the inherent complexity of building systems, it is usually not sufficient to rely on component testing for understanding the mechanism of progressive collapse. The building system may not behave the same as individual test specimens. The Blast Mitigation for Structures: 1999 Status Report (DTRA/TSWG Program) suggested that the phenomenon of structural collapse was a stochastic procedure and the estimation of the probability of failure in a real structure would involve an experienced engineer and engineering judgement. It was also indicated that conducting a full-scale test would be very costly. However, the importance of experimental research was recognized in the 1999 DTRA/TSWF Status Report.

The previous research indicates that the effects of blast loading depend on several factors. The primary factors are indicated below:

- *Type and size of bomb.*
- *Location of explosion.*
- *Distance of explosion (stand off distance).*
- *Type of structure (reinforced concrete, steel, or masonry structure)*
- *Type of structural system.*

Although, the first two factors are considered as major parameters that effect building damageability, Scheffiner (1998) advanced other views. Accordingly, the number of deaths has been linked to structural integrity and consideration of blast effects in structural design. Scheffiner's views were supported by observations made during the World Trade Centre attack in New York City and the Murrah Federal Building in Oklahoma City. The structural design of these two buildings explains the difference in casualties in each event. The World Trade Centre was designed following the principles of blast-resistant design. The Murrah Federal Building on the other hand was designed primarily for gravity loads, with little seismic considerations as required by the local building code for regions of low seismicity. As a result, the fatality was much less in New York City than that in Oklahoma City. The Murrah Building and the Ranon Point Apartment Building show examples of structural behaviour in buildings lacking design against progressive collapse. The extensive damage incurred in these two buildings is shown in Figures 1.1 and 1.2.

Design information and guidelines are available in the literature for blast resistance and progressive collapse prevention of structures. A summary of these guidelines is provided below:

U.S. Department of Defense (DOD) Minimum Antiterrorism Standards for Buildings

This document is part of the Unified Facilities Criteria (UFC) gathered by the U.S. Department of Defense for use in its projects. The objective of the document is to minimize mass casualties in DOD buildings. The recommendations are applicable to both new and existing buildings. The levels of protection for nonstructural elements, such as glazing of windows, are also specified in the guidelines (Table 1.1 and Table 1.2). Another design strategy that was recommended in the "DOD Minimum Antiterrorism Standards for Buildings" is the use of appropriate stand off distance, as outlined in Table 1.3.

U.S. Department of Army (DA) FM 3-19.30 Physical Security Document

Most features of physical security (such as, design of building, barriers, access, security system, etc.) are covered in this document for US Department of Army buildings. Different types of blockades and different levels of protection (magnitude of standoff distance) against bomb attacks are recommended. Also, a rather comprehensive coverage of general design requirements for walls, roofs and floors are provided.

Federal Emergency Management Agency (FEMA) 427 Primer for Design of Commercial Buildings to Mitigate Terrorist Attacks

This document explains the effect of bomb detonations on buildings and the nature of different types of shock waves generated during explosions. It also highlights the effects of two primary factors of blast loading; weight of the charge, and standoff distance. Figure 1.3, adopted from the FEMA 427 document, demonstrates a schematic view of blast effects and resulting pressures on buildings. The variation of pressure with time and distance are demonstrated in Figures 1.4 and 1.5, respectively, also adopted from the same document.

The FEMA document indicates that the pressure from a blast explosion can cause the failure of elements that carry vertical loads, such as columns and walls. This may trigger the structure to develop progressive collapse either in vertical or horizontal direction. The focus of FEMA 427 is primarily on prevention of progressive collapse. Lateral building loads exhibiting dynamic characteristics are compared. Three differences are identified between blast loads and seismic or wind loads. First, the magnitude of the explosive loads is much higher than earthquake or wind loads. Second, blast loads have a very short duration of time (micro seconds) in comparison of seismic or wind loads (seconds and minutes). Finally, the pressure from blast loads decreases with distance quite rapidly and causes localized damage as opposed to other dynamic loads which produce more global damage. The FEMA 427 document recommends three approaches to design buildings for preventing progressive collapse; i) indirect method ii) alternate load path method and iii) specific local resistance method.

- *Indirect Method:* This method ensures protection against progressive collapse through the selection of appropriate structural system, layout of vertical load

bearing elements (columns and walls) as well as designing and detailing individual members for strength, ductility and continuity.

- *Alternate-Load-Path Method:* This method of progressive collapse prevention requires designing structures to carry loads that may be induced on members resulting from the removal of a load bearing element. It requires structural analysis of buildings after the removal of specific elements, such as columns or walls. This approach does not require the characterization of blast loading. It is the preferred method of design in the General Services Administration (GSA) recommendations.
- *Specific Load-Resistance Method:* Involves explicit design of structural components against blast loading. Therefore, the design-level explosive forces must be computed for an identified blast threat. Blast-resistant design or retrofitting of existing structures focus on structural members in lower stories.

The following specific recommendations are made for structural design aspects of blast-resistant design of buildings:

Structural Characteristics

- Mass:** Lightweight structural members are not resistant to blast pressures. A building with steel deck without the concrete fill will not have blast resistance.
- Shear Capacity:** Shear failure is often more brittle than flexural failure. Therefore, it is recommended to prevent brittle shear failure in all elements and their connections to promote ductile flexural behaviour.
- Capacity for Load Reversals:** The structural elements should be designed for load reversals and should resist uplift pressures resulting from air blasts. In addition, care should be exercised in utilizing members that are not intended to resist load reversals. For example; prestressed concrete elements, seated connections and concrete fill over steel decks (unless attached with headed studs), should not be used.
- Redundancy:** Alternate load paths should be provided in vertical load carrying members.

- v) **Ties:** An integrated system of ties, in perpendicular direction can be used to help distribute loads to alternate paths in the event of the failure of a vertical load carrying element.
- vi) **Ductility:** Blast loading is strong enough to force structural elements to go into the inelastic range of deformations. These members should possess sufficient deformability beyond their elastic limits to experience inelastic deformations while maintaining their strength. This is ensured through proper design and detailing of elements and their connections, similar to those employed for seismic design.
- vii) **Construction Material:** Cast-in-place monolithic reinforced concrete has qualities that make it the preferred material for blast resistant construction. Concrete structures have a significant mass which improves response to explosions, since the mass is mobilized only after the initial pressure wave is significantly diminished. Furthermore, cast in place concrete can be designed and detailed to have continuity and ductility. Another advantage of reinforced concrete buildings is the stability of their columns which are less susceptible to global buckling upon losing a floor and associated lateral support.

Structural Layout

- i) **Column Spacing:** Column spacing should be limited to enable redistribution of loads in the event of a column failure.
- ii) **Exterior Bay:** Exterior bays are susceptible to blast attack. They are not able to redistribute forces in two directions. Therefore, it is advisable to have a shallow bay adjacent to the building exterior to limit the extent of damage.
- iii) **Transfer Girders:** Transfer girders should not be used in blast resistant buildings as their failure can lead to the collapse of a significant portion of the building.

Direct Design Method

Direct design method involves explicit analysis of the building for a design-level blast load. The blast load is characterized as an impulsive dynamic load, acting only a fraction

of a second on the building. Dynamic analysis, similar to seismic analysis of buildings may be used for design.

Federal Emergency Management Agency (FEMA 428) Primer for Design Safe School Projects in Case of Terrorist Attacks

This document is especially prepared for protecting schools against terrorist attacks. The structural design recommendations are the same as those presented in FEMA 427 and described above. The main focus of the guideline is placed on progressive collapse prevention as in the case of FEMA 427.

Federal Emergency Management Agency (FEMA 426) Reference Manual to Mitigate Potential Terrorist Attacks Against Buildings

This guideline provides a discussion of general concepts for mitigating risks associated with terrorist attacks. The structural and nonstructural design requirements are similar to those discussed above for FEMA 427.

U.S. General Services Administration (GSA) Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects

The U.S. General Services Administration guidelines provide a comprehensive coverage of blast-resistant design with specific recommendations for progressive collapse prevention. The guidelines also help establish the potentials for progressive collapse in existing buildings. The first edition of the guidelines was published in 2000 with emphasis on reinforced concrete buildings. The subsequent edition in 2003 included other types of structures.

The GSA guidelines specify exemption criteria for structures to be checked against. If the building meets any of these exemptions there is no need for further evaluation or design against progressive collapse. The following shows the exemption criteria:

- i. *The building is classified for agricultural use, or for occupancy of less than two hours per day.*
- ii. *The building is a detached one-or two-family dwelling.*

- iii. *The structure is not intended for occupancy, i.e., bridge, transmission tower, hydraulic structure etc.*
- iv. *The building is one-storey light steel frame or wood construction with less than 280 m² living area.*
- v. *The remaining useful life of the building is less than 5 years.*
- vi. *The building is a one-storey un-reinforced masonry, wood, or adobe structure.*
- vii. *The building has been designed and constructed to meet the progressive collapse requirements of GSA or ISC Security Criteria and these requirements are clearly documented.*
- viii. *The building meets the minimum standoff distance requirements specified in a Table provided and has sufficient redundancy to avoid single point failure and at the most 10 stories high without public areas and/or controlled parking.*
- ix. *The building meets the minimum standoff distance requirements and has sufficient redundancy to avoid single point failure and at the most 10 stories high with controlled public areas and/or parking, designed to conform to at least Seismic Zone 3 of UBC-1997 or Seismic Design Category D or E of IBC-2000.*

If the building under consideration does not fall within one of the criteria listed above, then it should be designed for progressive collapse.

The design principles for progressive collapse prevention in reinforced concrete buildings include redundancy, continuity, ductility, load reversal capacity, and the elimination of premature brittle shear failure. Redundancy in a structural system facilitates alternative load paths to develop upon the loss of one or more elements in the structure. This important factor should be kept in mind during the design process. Moreover, continuity in structural elements should be achieved by tying up the elements together properly, i.e. certain continuity of beam to beam across a failed column should be ensured. It is also suggested that, for prevention of progressive collapse sufficient inelastic deformability should be ensured (through appropriate detailing of reinforcement) to permit adequate redistribution of stresses. According to the guidelines, flexural failure must occur before the brittle shear failure takes place (one of the concepts also employed in seismic design).

Once the design process is completed, it is recommended that the building is analyzed for future deformations and higher load capacities. The use of elastic load analysis is recommended and supported for progressive collapse analysis. The GSA guidelines recommend static analysis under the following load combination, applied on the structure in the downward direction:

$$\text{Load} = 2(\text{DL} + 0.25\text{LL}) \quad (1.1)$$

Where, DL stands for dead load and LL stands for live load.

Three different scenarios are recommended for estimating the vulnerability of regular structures to instantaneous loss of a column. The first two scenarios are based on an external attack, which causes the loss of exterior and/or corner columns. The third scenario includes an internal attack which results in the loss of an interior column in the building. In all cases, the analysis should be done for both orthogonal directions of the building. Similarly, where building system consists of shear walls, the above three analysis scenarios should be considered. Figures 1.6 and 1.7 demonstrate these three analysis considerations for buildings with moment resisting frame and load bearing walls, respectively. Another important issue, which is illustrated in Figure 1.8, is the allowable collapse area for buildings with unusually long bays.

Examination of linear elastic analysis results shall be performed to identify the magnitudes and distribution of potential demands on both the primary and secondary structural elements for quantifying potential collapse areas. The magnitude and distribution of these demands will be indicated by **Demand-Capacity Ratios (DCR)** and it is expressed as:

$$\text{DCR} = Q_{UD} / Q_{CE} \quad (1.2)$$

Where,

Q_{UD} = Applied force (demand) on a component or connection/joint (moment, axial force, shear, and possible combination of forces)

Q_{CE} = Expected ultimate, un-factored capacity of the component and/or connection/joint (moment, axial force, shear and possible combination of forces)

The acceptable value for DCR, for typical and atypical structures is defined to be equal or smaller than 2 and 1.5, respectively. These values and capacities are established based on the approaches described in FEMA 274 and FEMA 356.

British Standards Institution (BSI) 8110: Part 2: 1985, Structural Use of Concrete

This standard was published for blast resistant design after the progressive collapse of the Ronan Point apartment building in May 1968, in London, England. This design provision provides specific guidelines to withstand the removal of critical elements in the structure to prevent progressive collapse. It also presents vertical and horizontal ties in design to improve the survivability of the building under extreme loads. Provisions of this standard are intended for design of five storey and higher structures.

American Concrete Institute (ACI) Building Code Requirements for Structural Concrete (ACI 318-05)

The main requirement of this code is to ensure structural integrity between the structural elements leading to higher robustness in reinforced concrete buildings. This integrity is achieved by tying the members together sufficiently. This is achieved by ensuring the continuity of bottom reinforcement in beams through their negative moment regions.

Canadian Standards Association (CSA A23.3 – 2004) Standard for “Design of Concrete Structures”

The Canadian standard for concrete building design provides only a very limited coverage of progressive collapse. The standard does not make reference to a procedure for blast load investigation, but provides a clause on structural integrity, which is limited to providing additional slab reinforcement to maintain structural integrity.

In addition to the above guidelines, some studies were conducted to assess the performance of buildings under blast loading, including potentials for progressive collapse. Burns (2003) provided a critical review of GSA requirements discussed above. Cagley (2003) explained concerns over progressive collapse in design and proposed mitigation strategies. Corley (2003) conducted a comprehensive investigation of the Murrah Federal Building performance after the bomb attack and discussed the importance of seismic design requirements on blast performance of similar buildings. The author indicated that reinforced concrete response could improve substantially if continuity and ductility requirements of seismic design are implemented.

Crawford (2003) provided a description of a retrofit method for progressive collapse mitigation. He explained the causes of progressive collapse and highlighted the importance of redistribution of forces following the loss of a column in the structural system. He attributed the reasons for progressive collapse to insufficient frame capacity and lack of continuity between frames and other load carrying systems. The author indicated the need for suitable retrofit techniques for prevention of progressive collapse, though expressed concerns over lack of engineering knowledge, awareness of policy makers, building response and performance.

Dusenberry (2003) provided a review of available guidelines and provisions for progressive collapse. He provided a comparative evaluation of existing design guidelines, many of which are summarized in this section. Ellingwood (2003) paper expressed the importance of design approaches employed in mitigating the probability of gradual failure caused by blast loading. He emphasized the importance of going beyond the minimum design requirements covered in building codes. It was further indicated that structural engineers should be aware of issues that surround design loads and appreciate the behavior of systems rather than design of elements.

Table 1.1: Levels of protection – new buildings (Adopted from Ref.14)

Level of Protection	Potential Structural Damage	Potential Door and Glazing Hazards	Potential Injury
Below AT standards	Severely damaged. Frame collapse/massive destruction. Little left standing.	Doors and windows fail and result in lethal hazards	Majority of personnel suffer fatalities
Very Low	Heavily damaged – onset of structural collapse: Major deformation of primary and secondary structural members, but progressive collapse is unlikely. Collapse of non-structural elements.	Glazing will break and is likely to be propelled into the building, resulting in serious glazing fragment injuries, but fragments will be reduced. Doors may be propelled into rooms, presenting serious hazards.	Majority of personnel suffer serious injuries. There are likely to be a limited number (10% to 25%) of fatalities.
Low	Damaged–unrepairable. Major deformation of non-structural elements and secondary structural members and minor deformation of primary structural members, but progressive collapse is unlikely.	Glazing will break, but fall within 1 meter of the wall or otherwise not present a significant fragment hazard. Doors may fail, but they will rebound out of their frames, presenting minimal hazards.	Majority of personnel suffer significant injuries. There may be a few (<10%) fatalities.
Medium	Damaged – repairable. Minor deformations of non-structural elements and secondary structural members and no permanent deformation in primary structural members.	Glazing will break, but will remain in the window frame. Doors will stay in frames, but will not be reusable.	Some minor injuries, but fatalities are unlikely.
High	Superficially damaged. No permanent deformation of primary and secondary structural members or nonstructural elements.	Glazing will not break. Doors will be reusable.	Only superficial injuries are likely.

Table 1.2: Levels of protection – existing buildings (Adopted from Ref. 14)

Level of Protection	Potential Structural Damage	Potential Door and Glazing Hazards	Potential Injury
Below AT standards	Severely damaged. Frame collapse/massive destruction. Little left standing.	Doors and windows fail and result in lethal hazards	Majority of personnel suffer fatalities.
Very Low	Heavily damaged – onset of structural collapse: Major deformation of primary structural members, but progressive collapse is unlikely. Collapse of secondary structural members and nonstructural elements.	Glazing will break and is likely to be propelled into the building, resulting in serious glazing fragment injuries, but fragments will be reduced. Doors may be propelled into rooms, presenting serious hazards.	Majority of personnel suffer serious injuries. There are likely to be a limited number (10% to 25%) of fatalities.
Low	Damaged – unrepairable. Major deformation of secondary structural members and minor deformation of primary structural members, but Progressive collapse is unlikely. Collapse of non-structural elements.	Glazing will break and is likely to be propelled into the building, but should result in survivable glazing fragment injuries. Doors may fail, but they will rebound out of their frames, presenting minimal hazards.	Majority of personnel suffer significant injuries. There may be a few (<10%) fatalities.
Medium	Damaged – repairable. Minor deformations of secondary structural members and no permanent deformation in primary structural members. Major deformation of nonstructural elements.	Glazing will break, but will remain in the window frame. Doors will stay in frames, but will not be reusable.	Some minor injuries, but fatalities are unlikely.
High	Superficially damaged. No permanent deformation of primary and secondary structural members or nonstructural elements.	Glazing will not break. Doors will be reusable.	Only superficial injuries are likely.

Table 1.3: Minimum standoff distances (Adopted from Ref. 14)

Location	Building Category	Standoff Distance or Separation Requirements			
		Applicable Level of Protection	Conventional Construction Standoff Distance	Effective Standoff Distance ⁽¹⁾	Applicable Explosive Weight ⁽²⁾
Controlled Perimeter or Parking and Roadways without a Controlled Perimeter	Billeting	Low	45 m ⁽⁴⁾ (148 ft.)	25 m ⁽⁴⁾ (82 ft.)	I
	Primary Gathering Building	Low	45 m ⁽⁴⁾⁽⁵⁾ (148 ft.)	25 m ⁽⁴⁾⁽⁵⁾ (82 ft.)	I
	Inhabited Building	Very Low	25 m ⁽⁴⁾ (82 ft.)	10 m ⁽⁴⁾ (33 ft.)	I
Parking and Roadways within a Controlled Perimeter	Billeting	Low	25 m ⁽⁴⁾ (82 ft.)	10 m ⁽⁴⁾ (33 ft.)	II
	Primary Gathering Building	Low	25 m ⁽⁴⁾⁽⁵⁾ (82 ft.)	10 m ⁽⁴⁾⁽⁵⁾ (33 ft.)	II
	Inhabited Building	Very Low	10 m ⁽⁴⁾ (33 ft.)	10 m ⁽⁴⁾ (33 ft.)	II
Trash Containers	Billeting	Low	25 m ⁽⁴⁾ (82 ft.)	10 m ⁽⁴⁾ (33 ft.)	II
	Primary Gathering Building	Low	25 m ⁽⁴⁾ (82 ft.)	10 m ⁽⁴⁾ (33 ft.)	II
	Inhabited Building	Very Low	10 m ⁽⁴⁾ (33 ft.)	10 m ⁽⁴⁾ (33 ft.)	II
Building Separation (for new buildings only)	Billeting	Low	10 m ⁽⁴⁾ (33 ft.)	No antiterrorism minimum	III ⁽³⁾
	Primary Gathering Building	Low	10 m ⁽⁴⁾ (33 ft.)	No antiterrorism minimum	III ⁽³⁾
	Inhabited Building	Very Low	No antiterrorism minimum	No antiterrorism minimum	Not applicable

(1) Even with analysis, standoff distances less than those in this column are not allowed for new buildings, but are allowed for existing buildings if constructed/retrofitted to provide the required level of protection at the reduced standoff distance.

(2) See UFC 4-010-10, for the specific explosive weights (kg/pounds of TNT) associated with designations – I, II, III. UFC 4-010-10 is For Official Use Only (FOUO)

(3) Explosive for building separation is an indirect fire (mortar) round.

(4) For existing buildings, see paragraph B-1.1.2.2.

(5) For existing family housing, see paragraph B-1.1.2.2.3.



Figure 1.1: 1993 bombing of the World Trade Centre in New York City



Figure 1.2: 1995 bomb attack of the Murrah Federal Building in Oklahoma City

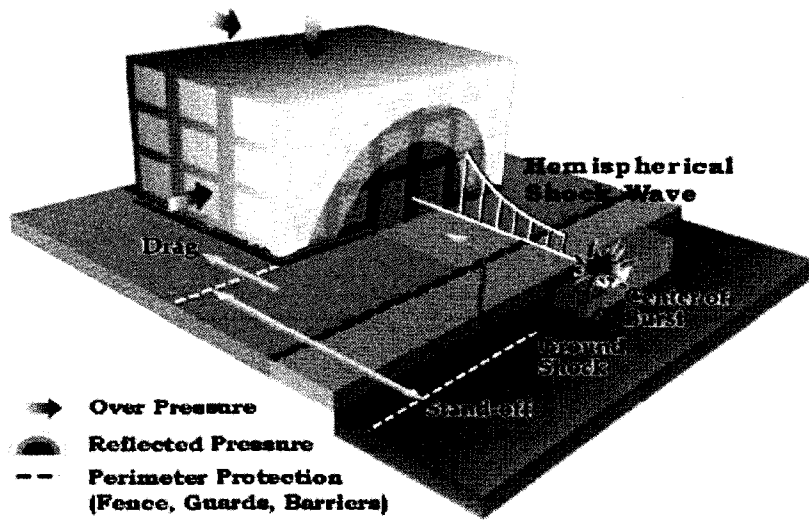


Figure 1.3: Schematic view of blast effect and resulting pressures on a building
(Adopted from Ref. 21)

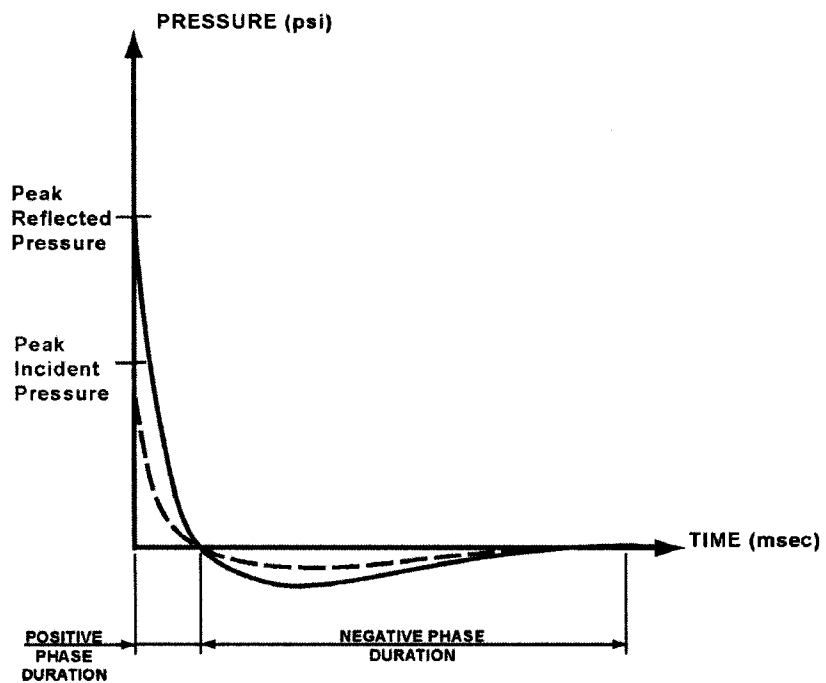


Figure 1.4: Variation of blast pressure with time (Adopted from Ref. 21)

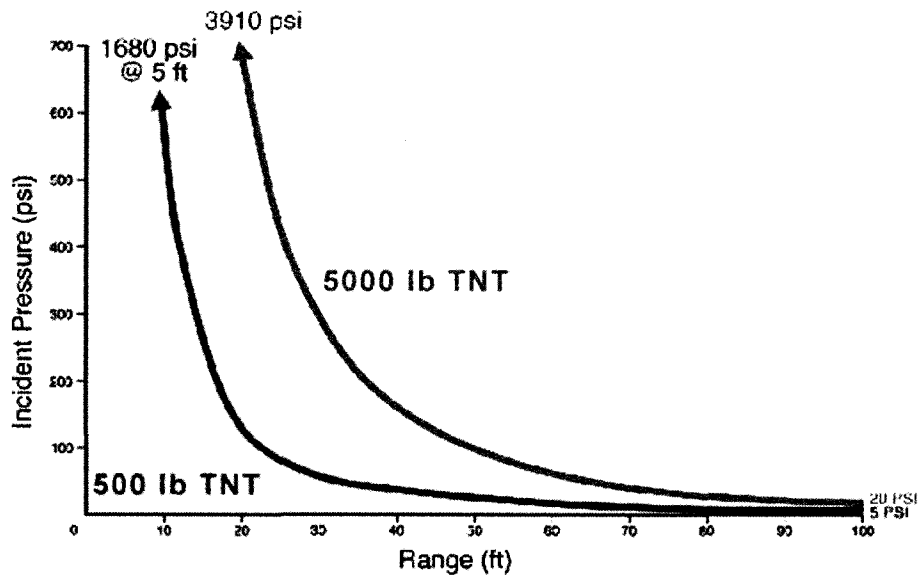
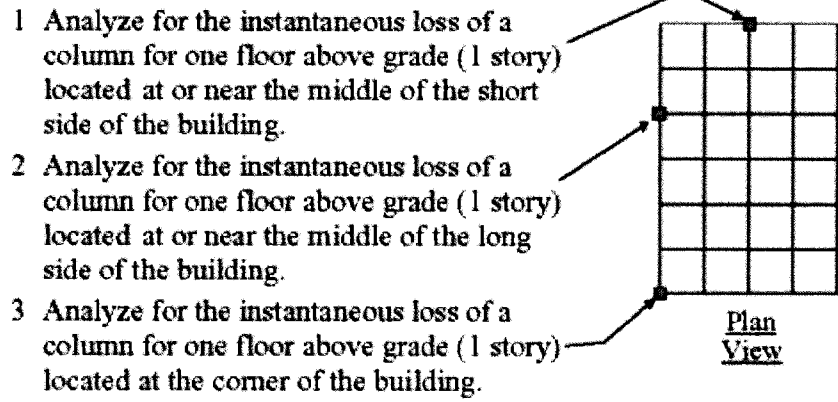
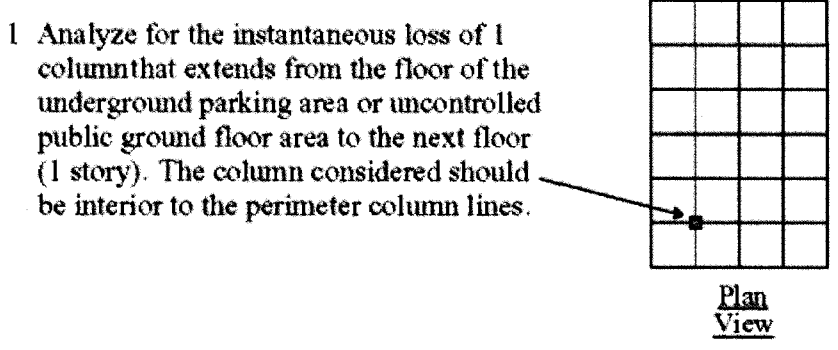


Figure 1.5: Variation of blast pressure with distance (Adopted from Ref. 21)



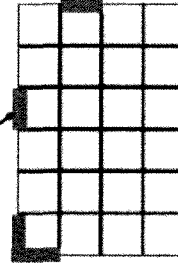
a) Exterior considerations



b) Interior consideration

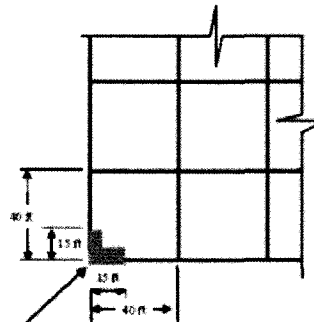
Figure 1.6: Cases to be considered for a frame building for collapse analysis
(Adopted from Ref. 16)

- 1 Analyze for the instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above grade, located at or near the middle of the short side of the building.
- 2 Analyze for the instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above grade, located at or near the middle of the long side of the building.
- 3 Analyze for the instantaneous loss of the entire bearing wall along the perimeter at the corner structural bay or for the loss of 30 linear feet of the wall (15 ft in each major direction) (whichever is less) for one floor above grade*.



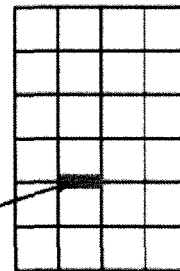
Plan View

- * The loss wall section for the corner consideration must be continuous and include the corner. For example, if the structural bay of a facility is 40 ft by 40 ft, the wall section that would require removal consists of 30 ft of the wall beginning at the corner and extending 15 ft in each major direction.



a) Exterior considerations

- 1 Analyze for the instantaneous loss of one structural bay or 30 linear feet of an interior wall section (whichever is less) at the floor level of the underground parking area and/or uncontrolled ground floor area. The wall section considered should be interior to the perimeter bearing wall line.



Plan View

b) Interior consideration

Figure 1.7: Cases to be considered for a building with walls for collapse analysis

(Adopted from Ref. 16)

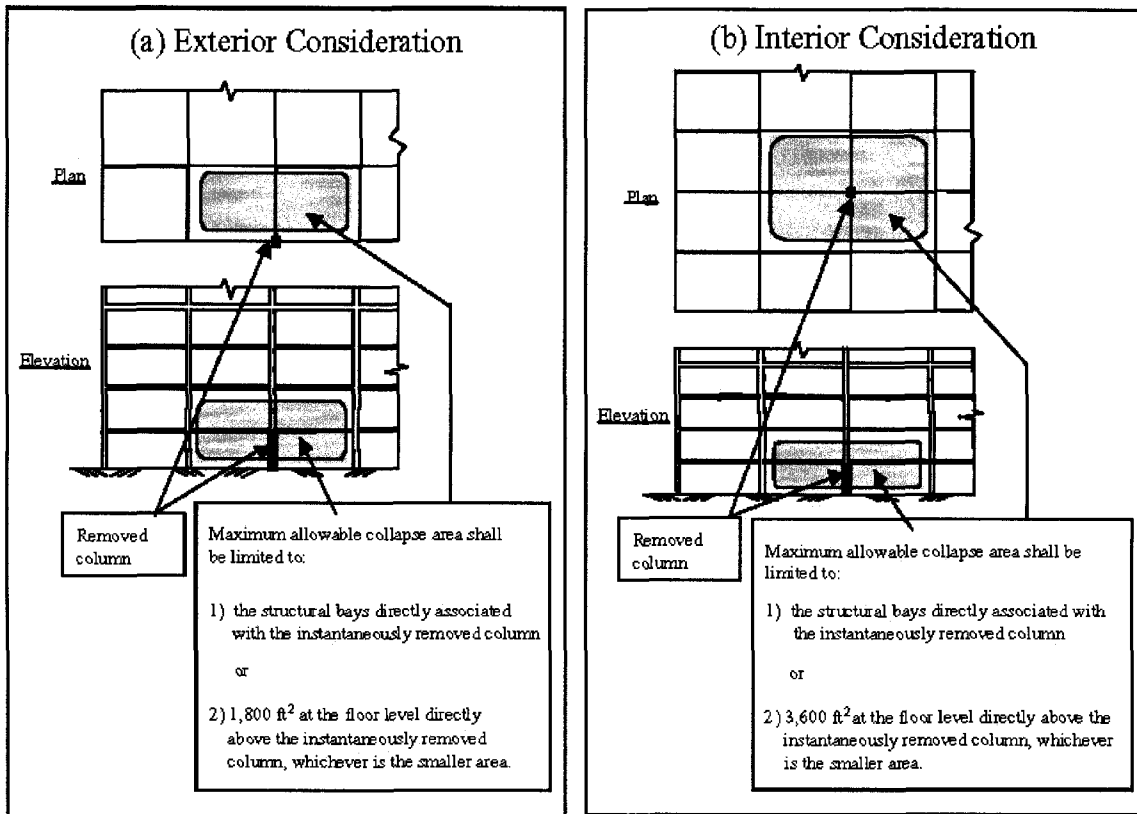


Figure 1.8: Maximum allowable collapse area (Adopted from Ref. 16)

CHAPTER 2:

Selection and Design of Buildings

2.1 Description of Structures

Three multi-storey reinforced concrete frame buildings with a regular floor plan were selected for investigation. A five-storey and a ten-storey building with 6.0 meter span lengths, plus a five-storey building with 9.0 meter span lengths were selected. All three structures are made of reinforced concrete frames and the lateral load resisting system comprises of moment resisting frames. These buildings are considered as office buildings for designation of loading in design. The floor plan consists of five bays in the long direction and three bays in the short direction with equal spans. The column height for each storey is 4.0 m. in all buildings.

The buildings are assumed to be located in Ottawa for their seismic design. The buildings with 6.0 m span were designed by E. Dincer (2003) in an earlier research program at the University of Ottawa. The structure with 9.0 m span was designed as part of the current research project reported in this thesis. The buildings were designed by following the CSA Standard A23.3 (1994) and CSA Standard A23.3 (2005), with seismic design provision of the National Building Code of Canada (NBCC 2005).

2.1.1 Plan and Elevation Views and the Loading

The structures have regular floor plans, with the same layout for all three buildings. Three structures with different number of stories and span lengths were considered to investigate the effects of building height and increased span length under blast loading. One of the 5-storey buildings and the 10 storey building had 6.0 m span in both directions and 4.0 m columns at each floor level. The remaining 5-storey building had the same characteristics except for the span length increased to 9.0 m. All buildings had two-way floor slabs, with equal amounts of loads transferred to beams in longitudinal and transverse directions due to the symmetry of the floor plan. Figures 2.1, 2.2, and 2.3 illustrate the plan and elevation views of buildings.

The concrete used in design was 30 MPa ($f'_c = 30$) and the reinforcement used was of Grade 400 ($f_y = 400$). The occupancy load was determined, assuming office buildings. Table 2.1 shows dead loads and live loads used in design, as well as the material properties used in design.

2.1.2 Member Design

Buildings with 6.0 m spans were designed previously by Dincer (2003) based on the requirements of CSA Standard A23.3–1984 and NBCC-1995. These designs were subsequently checked for conformity with CSA Standard A23.3–2004 and NBCC-2005. It was found that the strength of members and detailing of beams and columns were not affected sufficiently by the new standard and the code to justify a change. Hence the original design was used in the current project. Table 2.2 and 2.3 show the design details of frame elements for 5-storey and 10-storey R/C buildings with 6.0 m span lengths, respectively.

The design of 5-storey building with 9.0 m spans was performed based on the requirements of CSA Standard A23.3–2004 and NBCC-2005. Different dead load but the

same live loads that were used in the previous building designs were also used in this building. SAP 2000 computer software was used for the purpose of analysis and design.

The seismic provisions of NBCC 2005 were used to compute lateral earthquake loads. Since the building met the requirements of Clause 4.1.8.7 part (b) of NBCC-2005, “Equivalent Static Force Procedure” was used. The base shear force, V , was computed as per Clause 4.1.8.11 of NBCC 2005 as illustrated below:

$$V = S(T_a) M_v I_E W / (R_d R_o)$$

Where;

$$S(2.0) M_v I_E W / (R_d R_o) \leq V \leq 2/3 S(0.2) I_E W / (R_d R_o)$$

T_a is the fundamental period of the structure with empirical code equation given below for reinforced concrete frame structures:

$$T_a = 0.075 (h_n)^{3/4} \quad (h_n \text{ is in meters}) \text{ [Clause 4.1.8.11.(3)(a)(ii)]}$$

The height of each floor is 4.0 m then:

$$T_a = 0.075 (20.0)^{3/4} \rightarrow T_a = 0.7 \text{ sec}$$

$S(T_a)$ is the design spectral acceleration and can be determined as indicated below and through interpolation, as per Clause 4.1.8.4.(6) of NBCC 2005:

$$S(T_a) = F_v S_a(0.5) \text{ or } F_a S_a(0.2), \text{ whichever is smaller for } T = 0.5 \text{ s}$$

$$S(T_a) = F_v S_a(1.0) \text{ for } T = 1.0 \text{ s}$$

Since the computed T_a is 0.7 sec, the above equations can be used. F_a is acceleration-based site coefficient and F_v is velocity-based site coefficient and they are defined in Tables 4.1.8.4.B and 4.1.8.4.C of NBCC 2005 for **Site Class C** respectively. Uniform Hazard Spectral accelerations for Ottawa can be obtained From Table C-2 as, $S_a(0.2) =$

0.66, $S_a(0.5) = 0.32$, $S_a(1.0) = 0.13$ and $S_a(2.0) = 0.044$. From Table 4.1.8.4.B and Table 4.1.8.4.C, it is observed F_a and F_v are equal to **1** for Site Class C.

Therefore;

$$S(T_a) = 0.24$$

M_v is the factor to account for higher mode effects on base shear. It is equal to **1.0** for this particular building. This value is obtained from Table 4.1.8.11 of NBCC 2005.

I_E is the building importance factor and is equal to 1.0 for office buildings from Table 4.1.8.5 of NBCC 2005.

W is the total weight of building (i.e. total dead loads). It can be calculated as shown below:

$$W = \sum_i^n W_i \quad \rightarrow \quad W = 45562.5 \text{ KN}$$

The last two factors to be determined are R_d , and R_o , which are the ductility and overstrength-related force modification factors, respectively. The frame was considered as a ductile moment-resisting frame, therefore the value for these two factors from Table 4.1.8.9 are: $R_d = 4.0$ and $R_o = 1.7$.

The above factors are then used to calculate the lateral seismic force as shown below for both directions of the structure (short and long directions):

$$V = 0.24 * 1 * 1 * 45562.5 / (4 * 1.7) \quad \rightarrow \quad V = 1608 \text{ KN}$$

This value should satisfy the limits for V :

$$V \geq S(0.2) M_v I_E W / (R_d R_o) = 185 \text{ KN} \quad \text{OK}$$

$$V \leq 2/3 S(0.2) I_E W / (R_d R_o) = 1848 \text{ KN} \quad \text{OK}$$

Therefore the V used for design is **1608 KN**.

According to the Clause 4.1.8.11.(6) of the National Building Code of Canada (2005), the above value for lateral earthquake force should be distributed along the height of the structure in accordance with the equation shown below:

$$\mathbf{F}_x = (\mathbf{V} - \mathbf{F}_t) \mathbf{W}_x \mathbf{h}_x / \left(\sum_{i=1}^n W_i h_i \right)$$

where \mathbf{F}_x is the lateral force applied to each level and \mathbf{F}_t (portion of \mathbf{V} to be concentrated at the top of the building) is equal to $0.07\mathbf{T}_a\mathbf{V}$ but should not be greater than $0.25\mathbf{V}$ and can be **zero** if \mathbf{T}_a is less than or equal to **0.7 sec**. Since the fundamental period of this structure is 0.7 sec, \mathbf{F}_t is zero. Consequently the above formula reduces to;

$$\mathbf{F}_x = \mathbf{V} \mathbf{W}_x \mathbf{h}_x / \left(\sum_{i=1}^n W_i h_i \right)$$

Table 2.4 shows the distribution of seismic design forces applied at each floor level. Note that these forces were applied to the centre of gravity of each floor in x and y directions, in SAP2000 computer program. Since each floor was defined as a rigid diaphragm in the computer program, the earthquake forces were distributed through the frames evenly. The load combination used in design was based on the requirements of CSA A23.3-04 and NBCC 2005 as shown in Table 2.5. The applicable load combinations are shown in Tables 2.6 and the final design details obtained are summarized in Table 2.7.

2.1.3 Requirements for Non-Seismic and Seismic Design in CSA A23.3

There are remarkable differences between gravity load design of ordinary buildings and those designed against seismic forces. The principal concepts of seismic design include continuity, ductility and inelastic energy dissipation capacity. In ordinary buildings, the continuity of reinforcement into regions where they are not needed under gravity loads is not implemented. This means that beams are designed to carry only negative moments

within their support regions and positive moments within the mid-span regions. During seismic or blast-induced lateral loads, however, these beams experience positive moments in their support regions and possibly some negative moment in their positive moment regions. Lack of appropriate reinforcement for these additional moments results in immediate collapse of members. Furthermore, the structure should be equipped with sufficient integrity so that the failure of a particular element does not lead to progressive collapse.

When a column is removed from an ordinary gravity load resisting frame, the direction of bending changes and the negative moment region becomes a positive moment region. Since there is little or no positive reinforcement within the support region, the beam behaves as an unreinforced member and fails. Figure 2.6 shows effect of losing a lower storey column in an ordinary frame.

Another important difference between gravity and seismic designs is the requirement for ductility for the latter. Buildings designed for extreme loads, such as those caused by earthquakes and bomb blasts result in significant inelasticity in critical regions of structural elements. These regions need to be designed and detailed for ductile behaviour to avoid premature brittle failures. A structure with sufficient ductility can deform excessively without a significant loss of strength. These types of structures absorb energy and sustain inelastic forces without failure. This implies that they develop high deformation (or drift) capacities. According to NBCC seismic requirements the buildings are designed to be able to sustain 2.5% of their height (2.5 % drift ratio capacity) within the inelastic range of deformations without local or complete collapse. Ordinary gravity frames may have drift capacities limited to 1% drift ratio. This is illustrated schematically in Figure 2.7.

Table 2.1: Data used for Design of Buildings

	Floor	Roof
Dead Loads (kN/m ²) (6m – 9m)	5 - 8	3.5 – 5.5
Live Loads (kN/m ²)	2.4	2.2
Concrete Compressive Strength (MPa)	30	
Reinforcement Yield Strength (MPa)	400	

Table 2.2: Design details for 5-storey moment resisting frame building with 6.0 m spans

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1- 4	300*450 3#25 top, 2#20 bottom	450*450 8#25	400*400 8#20
	5	300*400 3#20 top, 2#15 bottom	450*450 8#25	400*400 8#20
Exterior	1- 4	300*450 3#25 top, 2#20 bottom	400*400 8#20	300*300 8#20
	5	300*400 3#20 top, 2#15 bottom	400*400 8#20	300*300 8#20

Table 2.3: Design details for 10-storey moment resisting frame building with 6.0 m span

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1- 5	300*450 3#25 top, 2#20 bottom	500*500 12#25	400*400 8#25
	6- 9	300*450 3#25 top, 2#20 bottom	500*500 12#20	400*400 8#20
	10	300*400 3#20 top, 2#15 bottom	500*500 12#20	400*400 8#20
Exterior	1- 5	300*500 3#25 top, 2#20 bottom	400*400 8#25	350*350 8#25
	6- 9	300*450 3#25 top, 2#20 bottom	400*400 8#20	350*350 8#20
	10	300*400 3#20 top, 2#15 bottom	400*400 8#20	350*350 8#20

Table 2.4: Distribution of seismic force in each floor for both directions

Floors	W (KN)	h (m)	W*h	V (KN)	V*W*h	F _x (EQ)
5	6682.5	20	133650	1608	214909200	411.3
4	9720	16	155520	1608	250076160	478.7
3	9720	12	116640	1608	187557120	359.0
2	9720	8	77760	1608	125038080	239.3
1	9720	4	38880	1608	62519040	119.7

Table 2.5: Load combinations (Adopted from NBCC 2000, Table 4.1.3.2.)

Case	Load Combination ⁽¹⁾	
	Principal Loads	Companion Loads ⁽²⁾
1	1.4D	
2	(1.25D or 0.9D) + 1.5L	0.5S or 0.4W
3	(1.25D or 0.9D) + 1.5S	0.5L or 0.4W
4	(1.25D or 0.9D) + 1.4W	0.5L or 0.5S
5	1.0D + 1.0E	0.5L + 0.25S

Table 2.6: Design load combinations

No.	Combo
1	1.4 DL
2	1.25DL + 1.5LL
3	1.0DL + 0.5LL + 1.0EQ
4	1.0DL + 1.0EQ

Table 2.7: Design details for 5-storey moment resisting frame building with 9.0 m span

Frame	Floor	Beam Section and Reinforcement	Column Section and Reinforcement	
			Interior	Exterior
Interior	1	600*500 10#25 top, 7#20 bottom	700*700 24#25	600*600 16#25
	2- 4	600*500 10#25 top, 7#20 bottom	650*650 16#25	600*600 16#25
	5	600*500 8#25 top, 7#20 bottom	600*600 16#25	600*600 16#25
Exterior	1	600*500 6#25 top, 4#20 bottom	600*600 16#25	500*500 16#25
	2- 4	600*500 6#25 top, 4#20 bottom	600*600 16#25	500*500 16#25
	5	500*400 6#25 top, 4#20 bottom	600*600 16#25	500*500 16#25

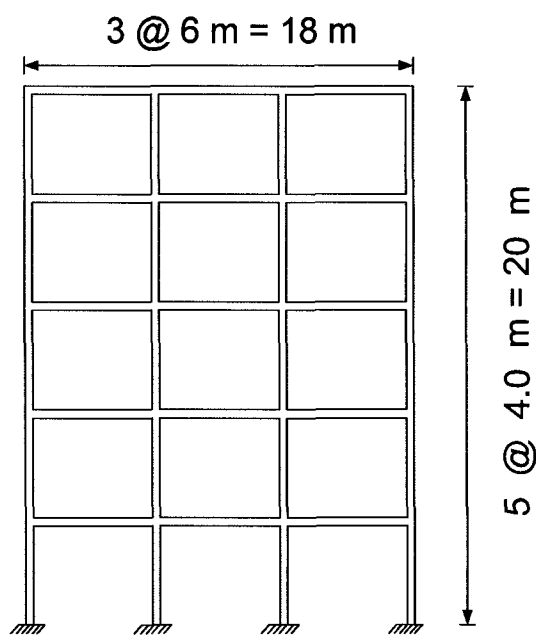
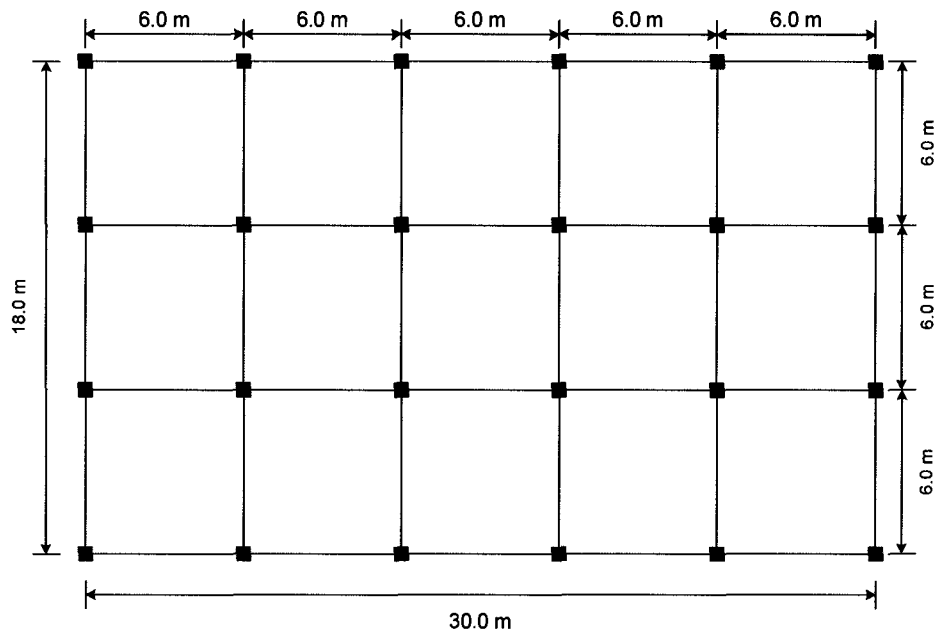


Figure 2.1: Plan and elevation views of 5 storey reinforced concrete building with 6.0 m spans

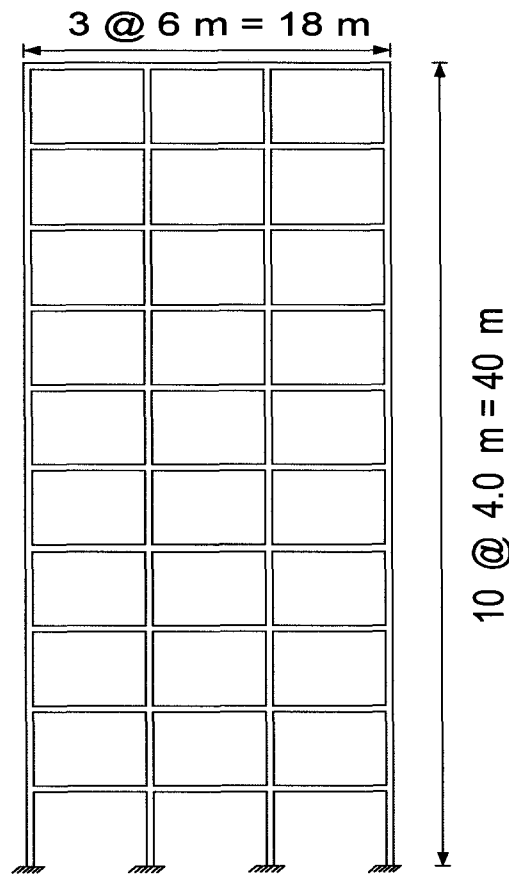
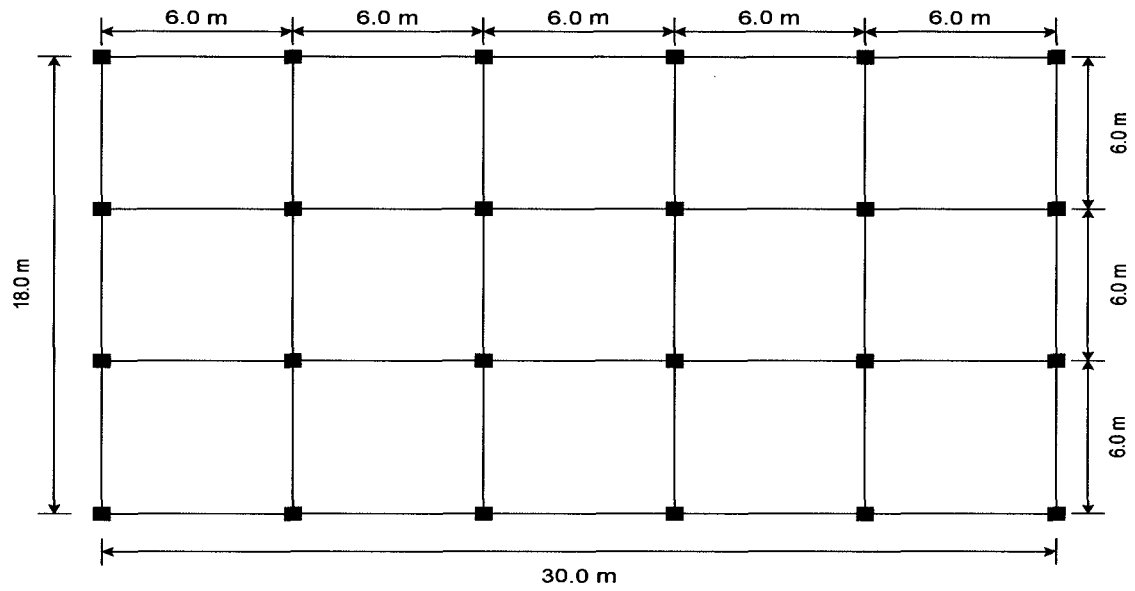


Figure 2.2: Plan and elevation views of the 10 storey reinforced concrete building

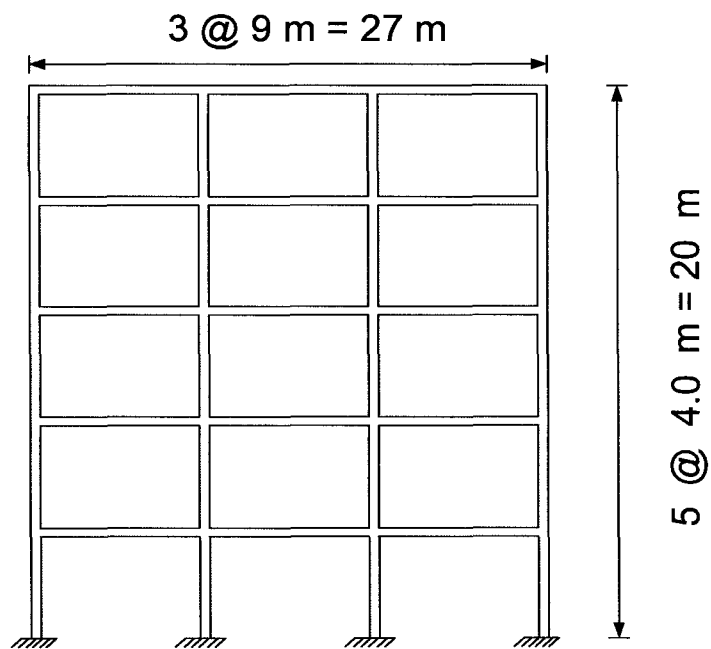
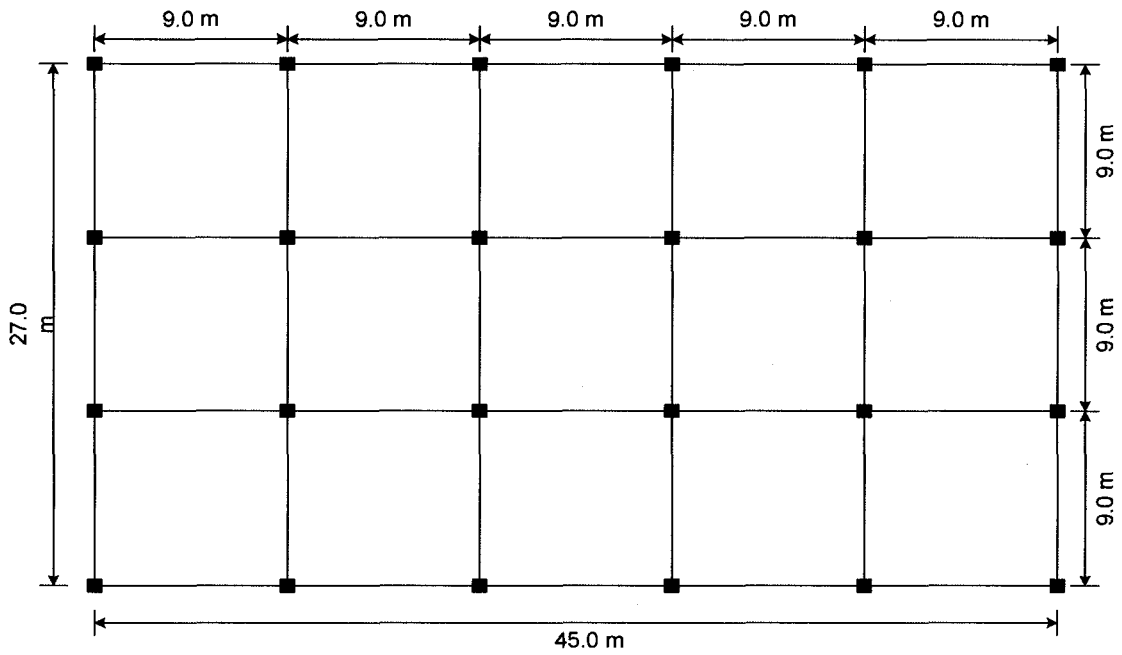


Figure 2.3: Plan and elevation views of 5 storey reinforced concrete buildings with 9.0 m spans

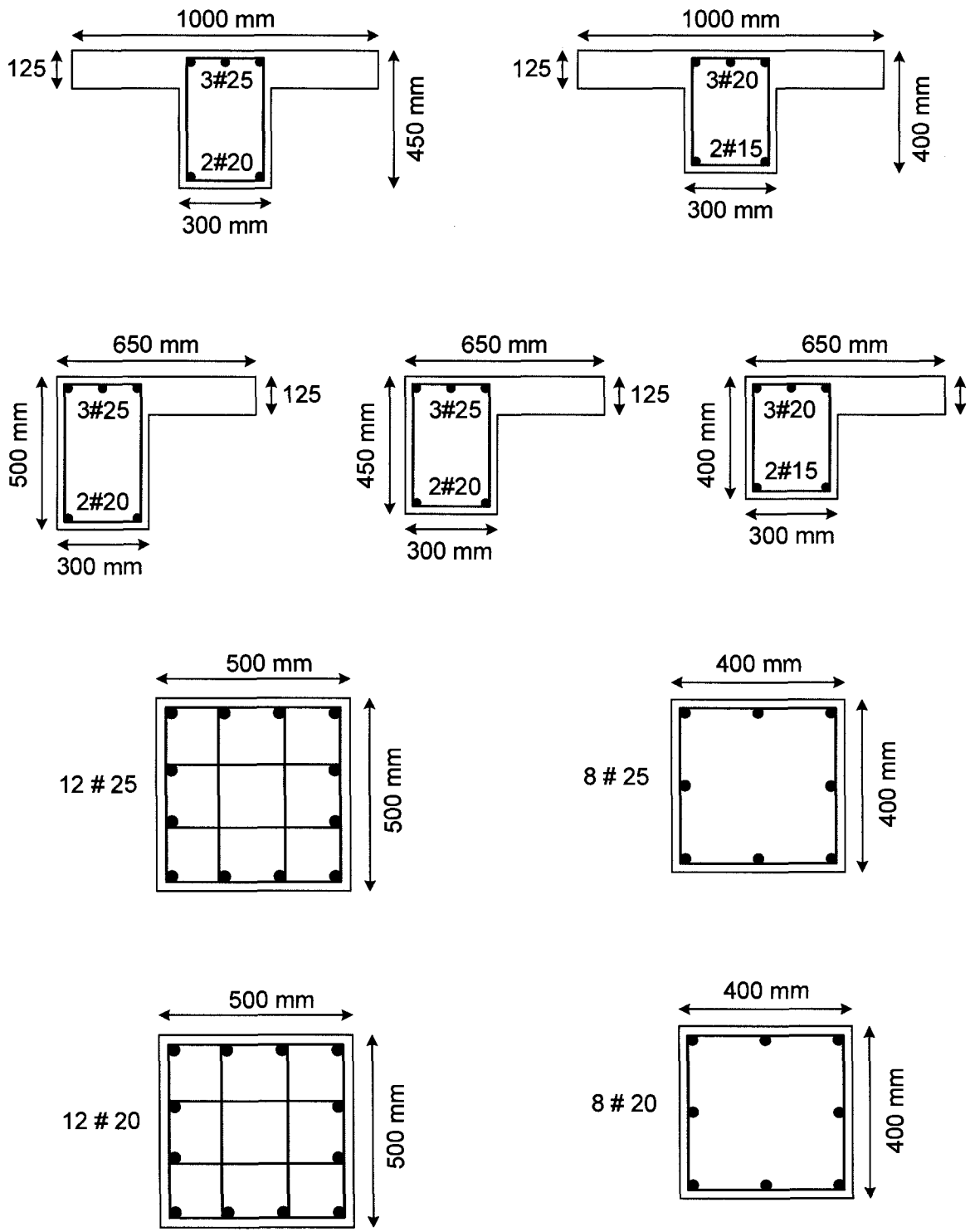


Figure 2.4: Design details for frame elements of 5-storey and 10-storey reinforced concrete buildings with 6.0 m spans

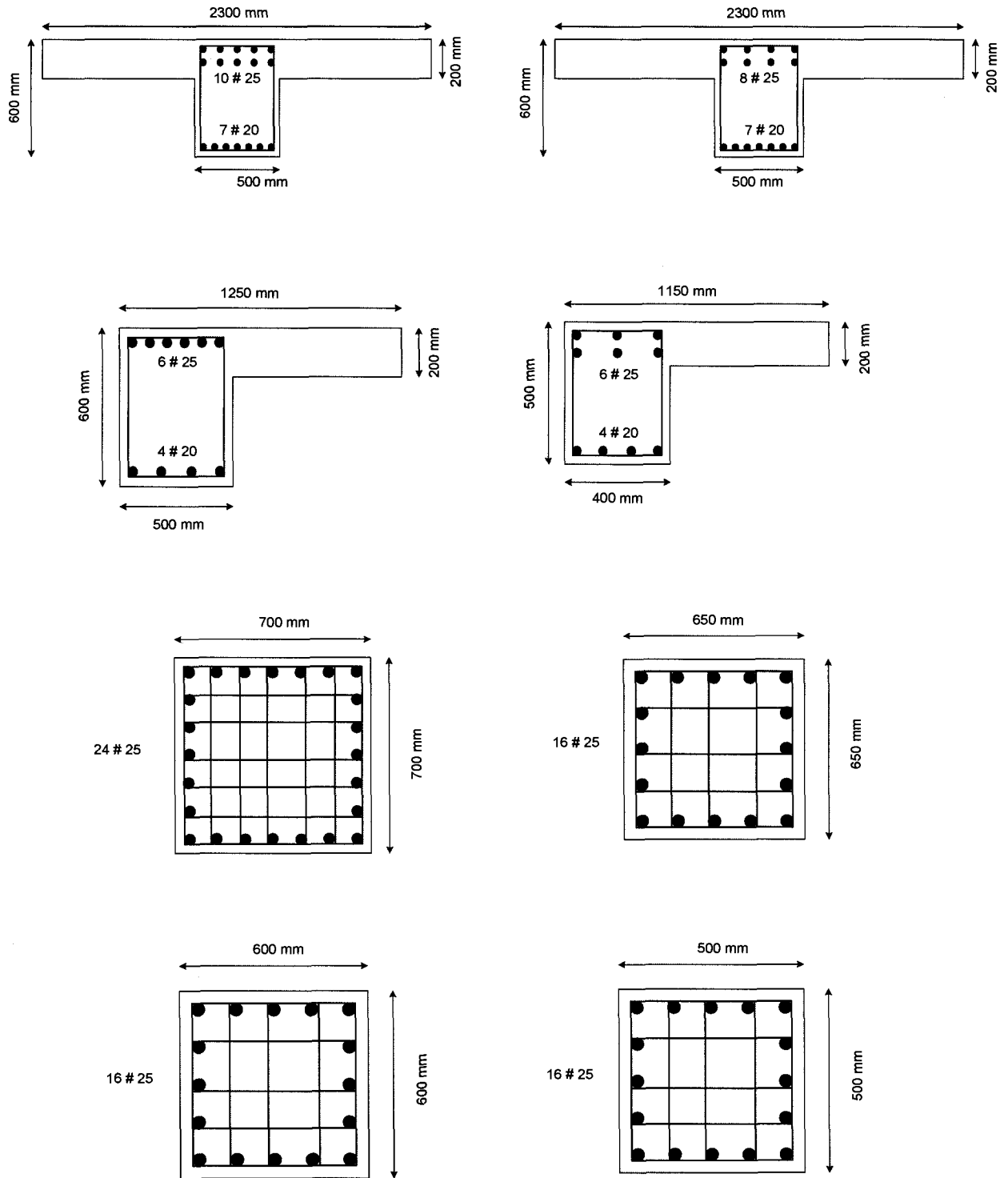


Figure 2.5: Design details for frame elements of 5-storey reinforced concrete building with 9.0 m spans

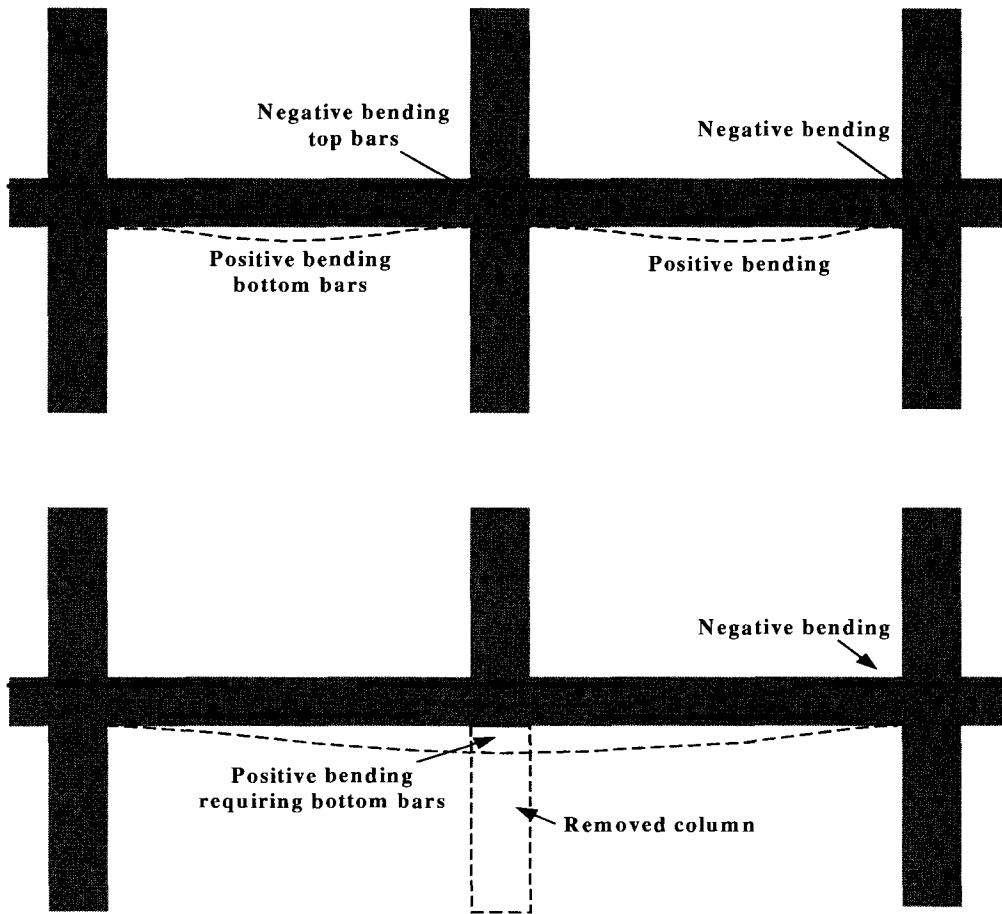


Figure 2.6: Effect of losing first storey column and change in moment demands

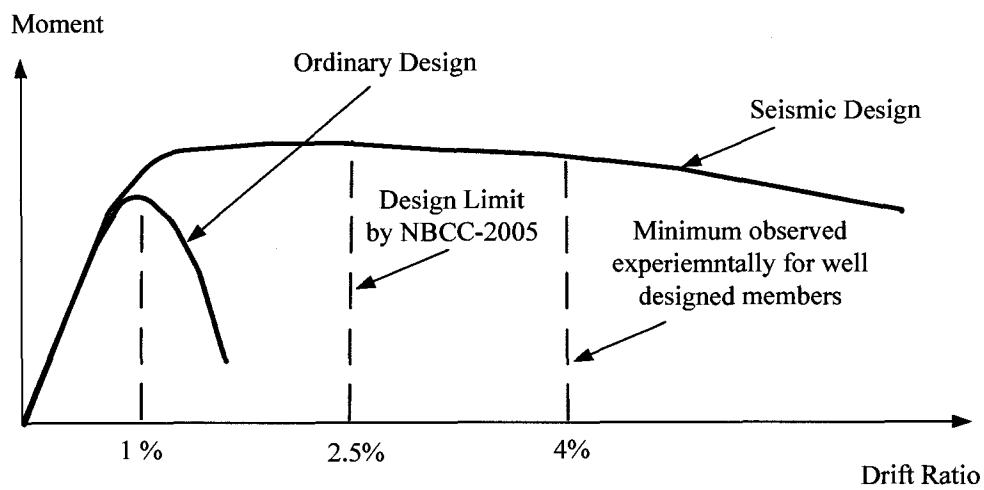


Figure 2.7: Drift capacities for ordinary and seismic designs

CHAPTER 3:

Analysis Procedure

3.1 DRAIN – R/C

Computer software “DRAIN R/C” was used for the progressive collapse analysis of buildings for both elastic and inelastic analyses. DRAIN R/C is a 2-dimensional plane-frame analysis program with static and dynamic elastic and inelastic analysis capabilities. The program was developed originally at the University of California at Berkely as a general purpose dynamic analysis program for seismic analyses (Kanaan and Powell, 1983). It has been modified at the University of Ottawa by Alsiwat (1993), Shooshtari (1998) and Shooshtari (2005) to incorporate features relevant to reinforced concrete structures (hence labeled DRAIN-R/C). The features introduced at the University of Ottawa include:

- i) Hysteretic model that incorporates axial force-flexure interaction.
- ii) Hysteretic model for inelastic shear.
- iii) Hysteretic model for anchorage slip.
- iv) Hysteretic model for infill panels.
- v) P- Δ effects.

- vi) Static inelastic analysis under incremental loading (push-over analysis).
- vii) Blast loading as time varying impulsive load.
- viii) Improved load specification, incorporating variable member loads.
- ix) User-friendly windows-based interactive input-output options.

The input is entered by filling structural geometry, member stiffness properties and yield strength for each member (for inelastic analysis). The loading for static analysis is specified either as constant gravity or nodal loads (in either direction) or incrementally increasing static nodal loads (for static inelastic analysis). Time-varying dynamic loads can be entered either in the form of earthquake ground motion records (ground accelerations at time increments) or impulsive blast loading, specified as a function of time, applied at nodes. Figures 3.1 through 3.4 illustrate data entry for various input items.

After the execution, output files are stored with file extensions “.STA”, “.YIE” and “.INP” for static analysis results, yield results and input file, respectively. The data entered can be verified by checked against the data file with extension “.INP.” The file with extension “.STA” contains all the static results such as static nodal displacements (horizontal displacements, vertical displacements, and rotations), bending moments, shear forces, and axial forces.

3.2 Modeling

The three buildings analyzed using computer program DRAIN-R/C consisted of 3 interior and six exterior frames in the short direction and three interior and six exterior frames in the long direction. The frames affected by column removal were modeled in two orthogonal directions separately as plane frames. The frames were symmetric, with equal spans. The geometry was specified in separate files for longitudinal and transverse frames. A total of eighteen frames were modeled for analysis, three frames for each direction of each structure. The columns were modeled to have full fixity at the base.

Uniformly distributed downward loads were applied on the beams. The load patterns assigned to the beams are indicated in Tables A.1 and A.2, included in the Appendix.

Each frame element in the model was assigned element properties consisting of element stiffnesses (modulus of elasticity, moment of inertia and area), hinge properties (cracking moment, yield moment and nominal moment capacity) and rigid segments at member ends simulating those segments that are built integrally with the adjoining members. Effective moment of inertia “ I_{eff} ” assigned to each member was as per CSA A23.3–2004 requirements, incorporating the effects of cracking. This effect reduces the gross moment of inertia (I_g) of beams and columns. The reduction factors, which are indicated in CSA 2004, are 0.35 for beams and 0.7 for columns.

$$(I_{\text{eff}})_{\text{Beam}} = 0.35(I_g)_{\text{Beam}}$$
$$(I_{\text{eff}})_{\text{Column}} = 0.70(I_g)_{\text{Column}}$$

According to Clause 8.6.2.3 of CSA Standard A23.3-04 the modulus of elasticity, E_c , is taken as indicated in the following equation for normal density concrete with compression strengths between 20 and 40 MPa ($f'_c = 30$ MPa):

$$E_c = 4500 \sqrt{f'_c}$$

Tables A.3 through A.5, included in the Appendix, show stiffness properties of structural elements. Element yield strengths were specified through member-end hinges, for use in non-linear analyses. These quantities, as well as post-yield stiffnesses were computed through sectional analysis, conducted by using computer program RC-Section (Saatcioglu 2002). Because only the flexural effects were considered in this study, the anchorage slip and inelastic shear springs were not activated. The properties for member-end hinges and rigid end segments are specified as input items in Tables A.6 through A.8 in the Appendix.

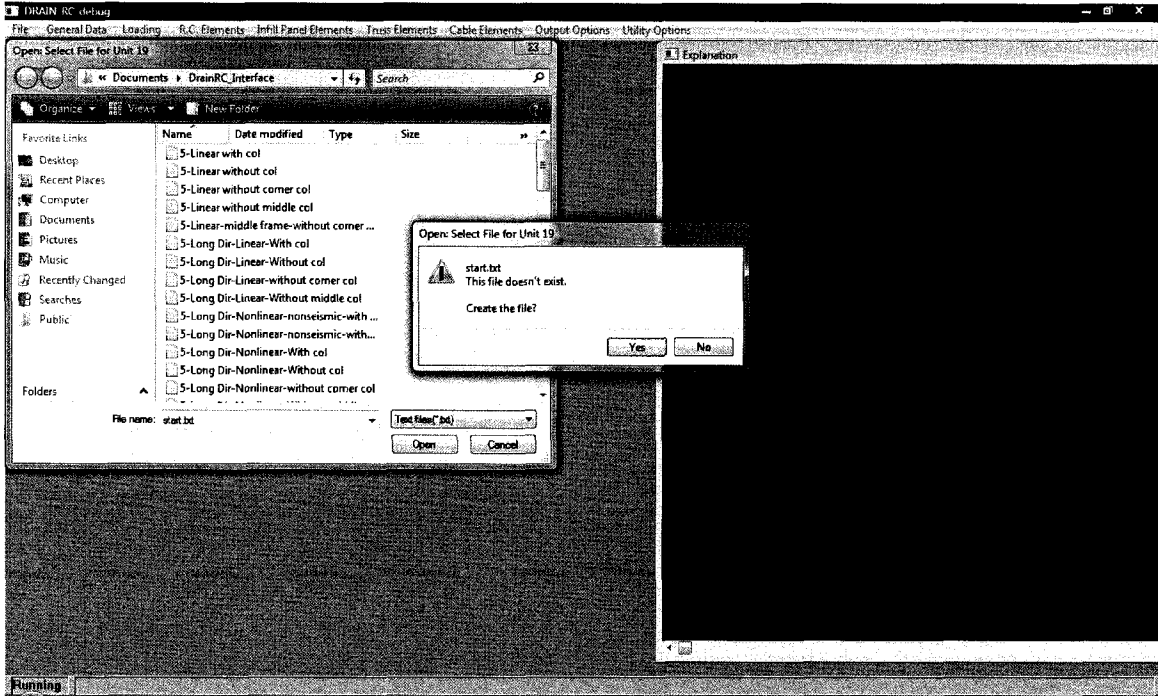


Figure 3.1: Create a file in DRAIN R/C

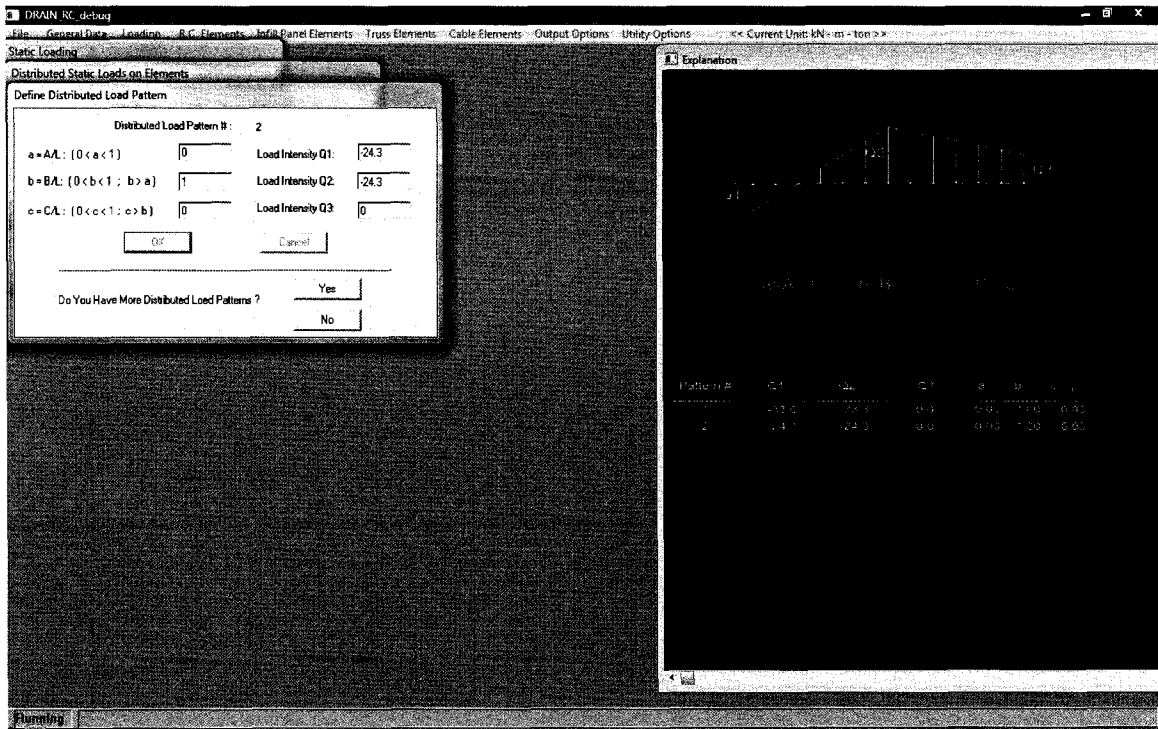


Figure 3.2: Defining distributed load

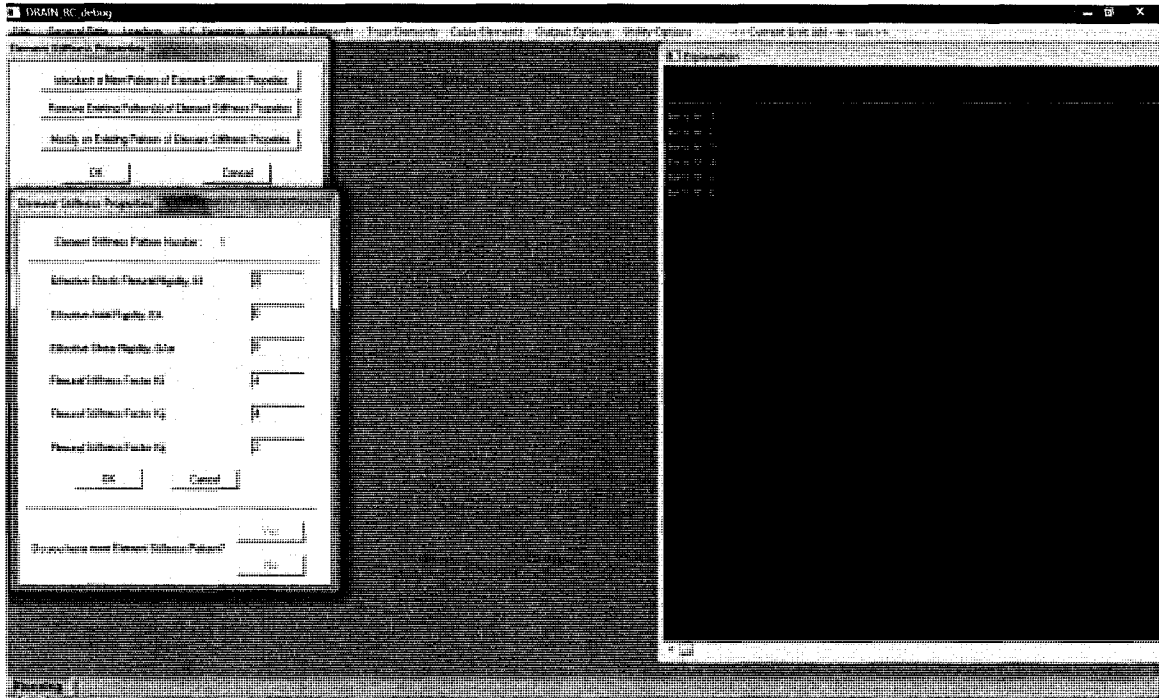


Figure 3.3: Defining stiffness properties in DRAIN R/C

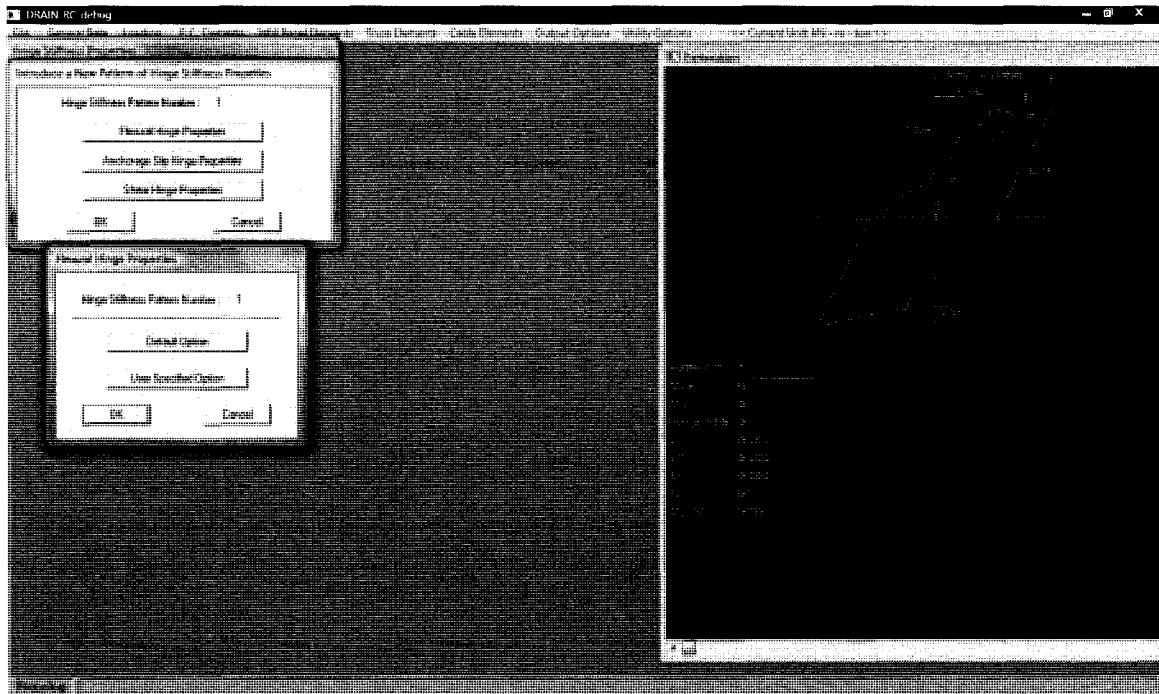


Figure 3.4: Defining hinge properties in DRAIN R/C

CHAPTER 4:

Elastic and Inelastic Progressive Collapse Analyses

4.1 General

The progressive collapse potential of the three buildings considered were established through elastic and inelastic analyses under three different first-storey column loss scenarios. These scenarios include the loss of a corner column, exterior edge column and an interior column. Frames in both short and long directions were analyzed to assess the performance of frames in both orthogonal directions. Elastic analysis results were checked against the GSA specified allowable demand-capacity ratios. Inelastic analysis results were evaluated on the basis of established inelastic deformation capacities of elements for ordinary gravity load carrying frames and earthquake resistant frames.

4.2 General Services Administration (GSA) Criteria and Elastic Analysis

The progressive collapse analysis based on GSA approach (2002) can be performed using elastic analysis under different column removal scenarios. These analyses are conducted for buildings that are found to be potentially vulnerable as a result of initial screening. The recommended vertical load to be used in such analysis is indicated below:

$$\text{Load} = 2(\text{DL} + 0.25\text{LL})$$

Where; DL and LL represent dead and live loads, respectively. The GSA specifies three analysis scenarios, two for the loss of exterior columns and one for interior column. The effects of each scenario are to be investigated in both orthogonal directions of buildings (short and long sides of structures). The analysis scenarios include; i) exterior considerations (immediate loss of an exterior column and a corner column) and ii) interior considerations (instantaneous loss of an interior column). Figures 1.6 and 1.7 in Chapter 1 illustrate the cited analysis considerations. Chapter 1 includes a detailed overview of GSA guidelines against progressive collapse. In these guidelines the Demand Capacity Ratio (DCR) is used to indicate the vulnerability of a building to progressive collapse. This factor is computed as follows:

$$DCR = Q_{UD} / Q_{CE}$$

where,

Q_{UD} = Demand force determined in member from elastic analysis (moment, axial force, shear, and possible combined forces).

Q_{CE} = Capacity of the element (moment, axial force, shear and possible combined forces). These capacities (dynamic capacities) incorporate a strain rate effect of 1.17 and a material over-strength factor of 1.1. Consequently, the dynamic capacities are computed by increasing the reinforcement yield strength by 25% ($1.25 f_y$) and the concrete compressive strength by 25% ($1.25 f'_c$)

The acceptable value of DCR for either primary or secondary structural elements is 2.0 or less for regular building configurations. If an irregular building configuration is investigated the limit for this value is equal to or less than 1.5. If the Demand Capacity Ratio exceeds these specified limits, the building is considered to have high risk of damage for progressive collapse.

The use of a well-established computer program is recommended by GSA for elastic analysis of structures. In order to avoid conservative solutions, a 3-dimensional model is suggested. However a 2-dimensional model is acceptable. Computer program DRAIN-R/C, with its planar analysis capability was selected for use in this investigation.

The elastic analysis was conducted by assigning unrealistically high flexural capacities to beams and columns. Figure 4.1 illustrates the actual moment rotation relationship for a typical member, in comparison with that of elastic behaviour. The structures are assumed to have sufficiently high shear capacities so that the premature shear failure is avoided. This assumption can be justified as it reflects the current design practice. Furthermore, the column removal increases the beam span length and makes flexural behaviour more critical.

Figure 4.2 illustrates first-storey floor plans for the buildings analyzed, with assumed locations of column damage due to bomb blast indicated as missing columns. The frames with missing columns were modeled in each direction and loaded with GSA specified uniformly distributed load. The load was increased incrementally to its full value and elastic moment demands were established. These demands were compared with moment capacities to determine demand-capacity ratios (DCR). A total of 18 analyses were conducted using the two 5-storey and one 10-storey frame buildings under investigation, incorporating three column loss scenarios in two directions.

The results are shown in Figures 4.3 through 4.11 in the form of DCR's for beam moments. It should be noted that the buildings analyzed were all designed for seismic forces as required by NBCC-2005 and seismic detailing as required by CSA A23.3-2004

as fully ductile structures. This implies that the beams had continuous bottom reinforcement, satisfying the seismic requirement of having positive moment capacity in support regions (at beam ends, including the location of column removal) equal to or greater than 50% of the maximum negative moment capacity. Therefore, these structures were able to provide some resistance to column removal. Buildings designed without seismic considerations would have no significant positive moment capacity at beam ends and collapsed immediately after the removal of columns.

The results shown in Figures 4.3 to 4.11, from linear analysis indicate that the loss of a lower storey column due to a nearby explosion is likely to trigger progressive collapse in multi-storey reinforced concrete frame structures, except for the damage to a corner or an exterior column in a short-span (6.0 m span) frame building. Buildings do not survive the loss of an interior column as indicated by higher than 2.0 values for DCR. The analysis results show buildings with 6.0 m span length may survive the loss of a corner or an exterior column. The 5-storey building with 6.0 m spans developed DCR's of less than 2.0 in all beams upon removal of the corner and exterior edge columns, though the 10-storey building indicated a slightly higher than 2.0 DCR value for one of the beams, and may still be considered to meet the GSA criterion.

The removal of an exterior edge column in the 5-storey building with 9.0 m spans developed significantly higher DCR values (2.04 to 2.12) than the companion building with 6.0 m spans (showing DCR values of approximately 1.70). This comparison signifies that long span buildings are more vulnerable to progressive collapse than buildings with shorter spans, as expected. In all cases the removal of an interior column creates a more severe condition, with DCR values in 5-storey frames reaching approximately to 2.70 in buildings with 6.0 m spans and 2.39 in the companion structure with 9.0 m spans. The criticality of interior column removal increases as the number of stories increases. The 10-storey building showed high DCR values of 2.82 and above due to interior column removal. In all cases the frames in the short direction were affected more by column removal than those in the long direction. This is expected since the

frames in the long direction were able to distribute stresses better than those in the short direction with fewer bays.

A similar study was done by Omer Salem Ahmed Yagob (2006) at University of Ottawa to investigate the vulnerability of the same ten storey reinforced concrete building (nominal ductile) to progressive collapse. In Yagob's study, in order to conduct elastic analysis (DCR values) a 3-dimensional program, SAP 2000, was employed. Comparison of results shows very small variation in DCR values which indicates a 2-dimensional program is conservative.

4.3 Inelastic Analysis

Elastic analysis of structures is a standard technique that can be conducted without much difficulty. This is why it is recommended as one of the techniques in the GSA document for progressive collapse analysis. However, it may not be an accurate representation of structural behaviour under extreme conditions. The removal of a support does create a severe condition that can easily generate inelastic deformations in structural elements. Therefore, nonlinear static analysis was conducted as another approach to assess the vulnerability of structures for progressive collapse. Unlike elastic analysis, the beams are allowed to yield in non-linear analysis shortly before they develop their design capacities. As the beams start yielding, they become softer and redistribution of forces to nearby more rigid (elastic) elements takes place. Hence, inelastic analysis allows redistribution of forces just as they take place in actual buildings suffering a column loss.

An important aspect of inelastic analysis is the ability to assess structural performance and potential member failure more accurately, instead of utilizing empirical rules, such as the computation of DCR as recommended by GSA to decide whether the structure can survive the loss of a column or not. Inelastic capacities of structural members can be assessed by using experimental data obtained from seismic research. It has been established through extensive experimental research that ordinary reinforced concrete elements, without special seismic detaining (without the confinement of concrete) can

sustain about 2% lateral drift ratio (displacement equal to 2% of member length) under flexure before they start developing significant strength decay. The same drift capacity reduces to 1% if the reinforcement is spliced within the potential plastic hinge region without proper detailing. Shear dominant members also show reduced deformability of 1% lateral drift ratio. Hence it is reasonable and somewhat conservative to assume the onset of failure in flexure dominant beams to be at vertical deflection approximately corresponding to 1% of its original length. The deflection capacity increases substantially in structures with seismic resistant design and detailing practices. Earthquake resistant building components are confined significantly by transverse reinforcement in their critical regions. This increases flexural deformation capacity to 4% and higher, especially if the members are subjected to low axial compression, as in the case of beams. In the beam of the frame structures considered in this investigation it may be conservative to assume the onset of strength decay to be at 4% deflection, though most flexure dominant elements can develop up to 6% deflection in the absence of axial compression. The effect of axial compression is to reduce inelastic deformability.

The computer program DRAIN-R/C is equipped with inelastic hinges that can simulate inelastic behaviour of structural elements as illustrated in Figure 4.12. Though the program was essentially developed for seismic investigations under deformation reversals with appropriate hysteretic models, it can be used to take advantage of the non-linear primary force-deformation relationship incorporated as part of the hysteretic model. The flexural yield capacities of individual members can be computed based on the design details presented in Chapter 2. The yield moments are assigned to plastic hinges at member ends. The primary force-deformation relationships of the hysteretic models in DRAIN-R/C have bi-linear idealization, as shown in Figure 4.13. The initial ascending branch of this relationship is linear up to the yield point, slope of which gives the effective elastic stiffness, incorporating the effects of concrete cracking. The post yield stiffness is assumed to be 5% of the effective elastic stiffness for each element. The Appendix provides the details of the input data entered for analysis.

The same building models used earlier for elastic analysis were also used for conducting inelastic progressive collapse analysis under incrementally increasing loads. The load that was used was that recommended by GSA guidelines, i.e., $2(DL+0.25LL)$. The analyses were conducted for the same column removal scenarios that were considered for elastic analysis, i.e., exterior corner column removal, exterior edge column removal and interior column removal in both directions of frames.

The vertical gravity load, as defined in GSA recommendations was applied incrementally and the corresponding lateral (vertical) deflections of beams were recorded in percentages of beam span length. It was observed that the buildings could not carry the GSA recommended loads without developing excessive deformation demands. Deflection demands were in excess of 20% of beam length, which signifies the failure of these beams. Excessive plastification was observed, with virtual the beams yielding.

Next, the analysis results were examined to identify the loads at which initial yielding takes place, 2% deflection takes place as the failure deformation of ordinary beams and 4% deflection takes place as the onset of strength decay in seismically designed beams. The results are presented in Figures 4.14 to 4.22 for different column removal scenarios for all buildings. The results show that the 5-storey building with 6.0 m spans was able to carry approximately 50% of GSA load ($2DL + 0.5LL$) when a corner column was removed. The same building was able to resist additional 10 % of load as the deflection demand increased to 2%, corresponding to the capacity of ordinary building frames. The same building resisted additional 10% of load at 4% deflection, assuming the beams were confined for seismic requirements. When the companion 5-storey building with 9.0 m spans was analyzed it was observed that the increase in span length resulted in approximately 10 to 15% drop in load carrying capacity at each of the three deformation levels.

Examination of the results for exterior edge column removal indicates that initial yielding occurred at about 40% of the GSA load. The load resistance increased by 10% at 2% deflection and another 10% at 4% deflection, indicating that the loss of an edge column

results in a more severe condition than the loss of a corner column. The performance of the 5-storey building with 9.0 m spans showed similar behaviour, implying that the effect of span length was small under the exterior column loss scenario.

The analysis results clearly showed that most severe case of loading was generated by the removal of an interior column. All buildings had limited load carrying capacities, corresponding to 20% of GSA load at first yielding, 32% at 2% deflection and 39% at 4% deflection. This implies that an ordinary 10-storey building can only resist 32% of GSA load prior to developing progressive collapse, whereas seismically designed 10-storey building can resist approximately 40% of the GSA load. In general the building height did not affect the progressive collapse performance of frames when triggered by the loss of an interior column.

The sequence of plastification under incrementally increasing loads is indicated in Figures 4.23 through 4.40. In general yielding began in the beams of the bay that lost a column. Initial yielding is limited to the beam ends immediately above the lost column. Further increase in loading increased the number of hinges formed at beam ends. When the beam deflection reached 2%, all the beams in the bay that suffered from column loss yielded at both ends. Further increase in load resulted in increased inelasticity in already yielded regions of beams, while resulting in some column yielding in exterior columns when the column loss was in the perimeter of the building (exterior edge column loss). Increased span length, as in the case of 9.0m span 5-storey beam did not result in column yielding, though produced increased inelastic deformations at beam ends. There was no column yielding observed when an interior column was lost.

The above analyses were repeated under (DL + 0.5LL), without the impact factor of 2.0 included in GSA Guidelines. The results are tabulated in Table 4.1 in terms of vertical beam deflections, expressed as percentages of beam length. Accordingly, the 6.0 m span buildings survived corner and exterior edge column losses, developing less than 2% deflection in beams. However, the same structures required deformability of approximately 8% vertical deflection when an interior column is removed. When the

span length was increased to 9.0 m, the deformation demands increased to approximately 3%, 5% and 9% for exterior, corner and interior column removals, respectively, indicating imminent progressive collapse, unless the beams are confined as seismic-resistant elements, in which case the collapse can be prevented under corner and external column removal scenarios.

The comparison of the results obtained from elastic and inelastic analyses indicate good agreement in terms of the locations of critical joints. At failure loads, the areas where significant yielding was observed coincided with high DCR values computed through elastic analysis.

Table 4.1: Drift Ratio from nonlinear analysis based on (DL + 0.5LL) loading

DL + 0.5LL		Drift Ratio (Vertical Displacement)		
		Exterior Column	Corner Column	Interior Column
5-Storey	Short Side	1.6%	1.8%	8.2%
	Long Side	1.6%	1.7%	8.0%
10-Storey	Short Side	0.7%	0.9%	8.4%
	Long Side	0.6%	0.8%	7.6%
5-Storey 9m span	Short Side	3.0%	5.3%	9.2%
	Long Side	3.0%	5.2%	9.0%

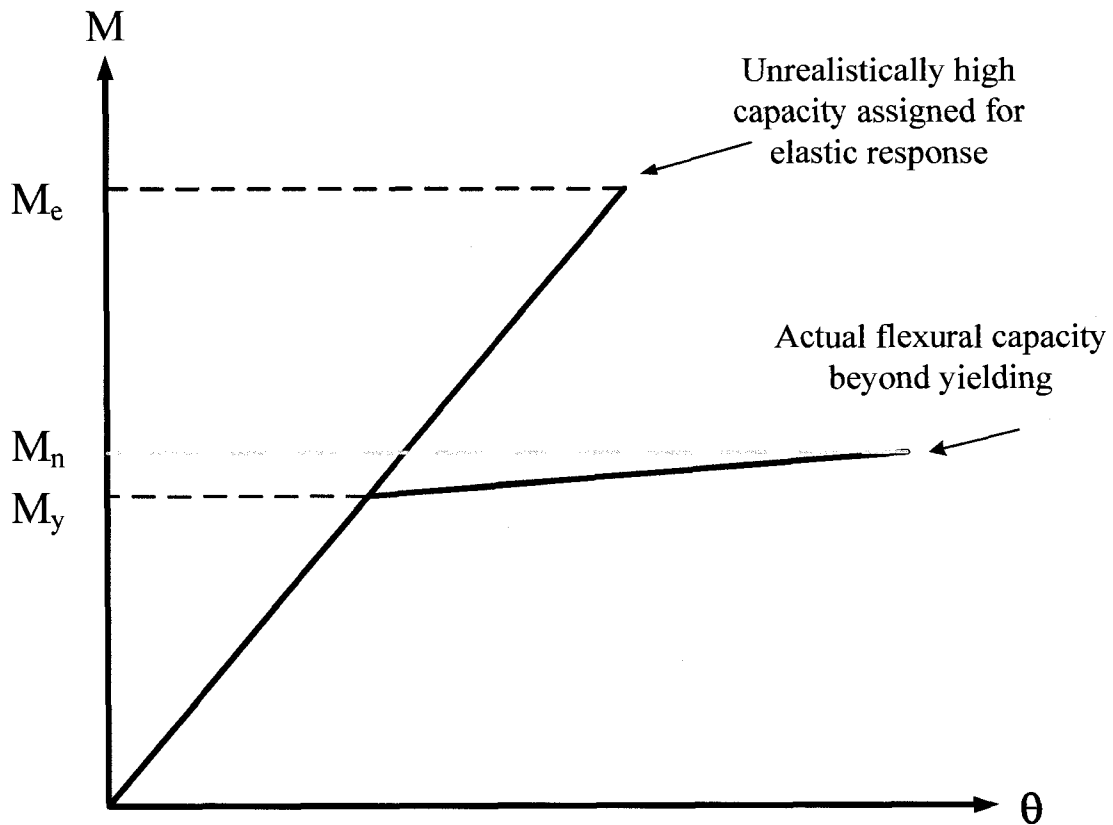


Figure 4.1: Moment-Rotation Relationship for Elastic Analysis

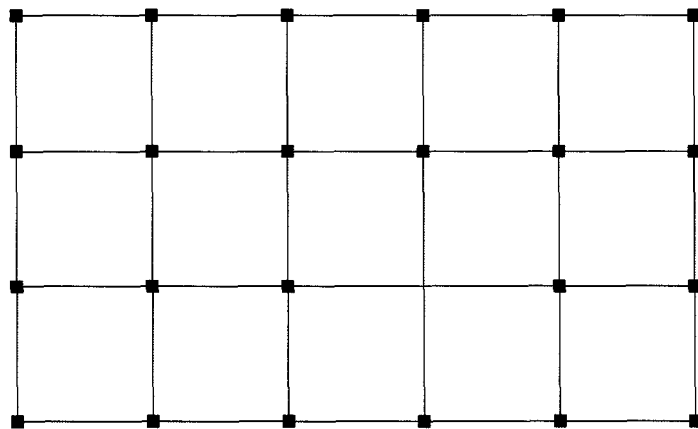
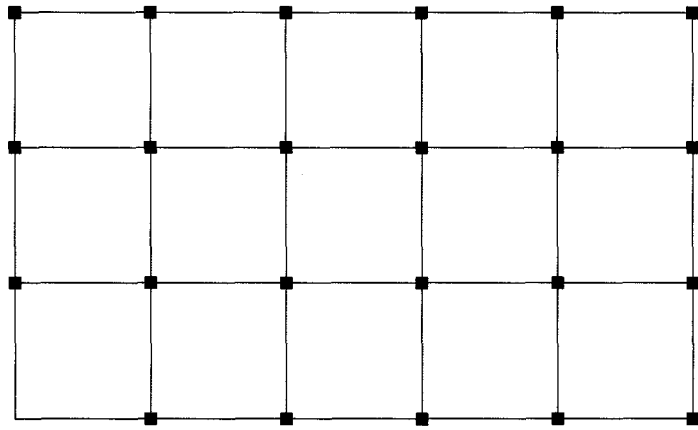
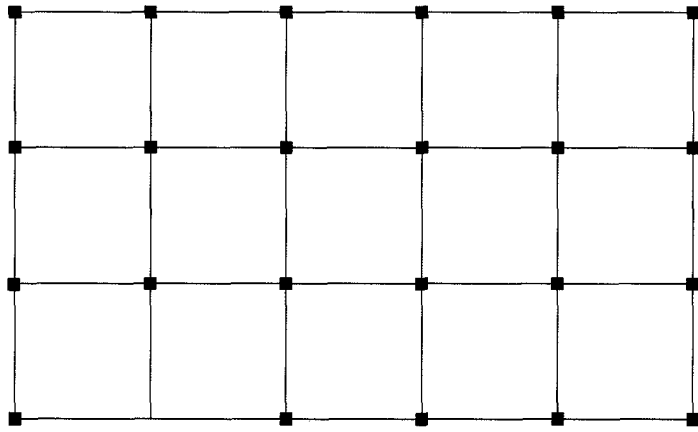


Figure 4.2: Location of the removed column in each scenario; exterior column removal, corner column removal, and interior column removal

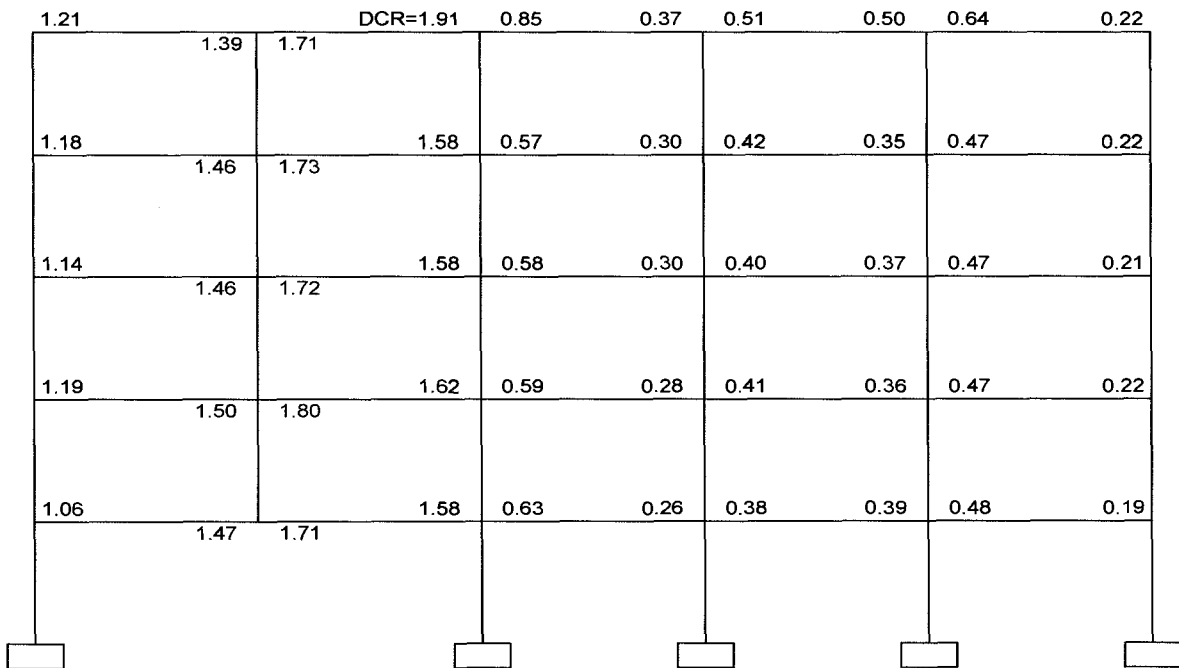
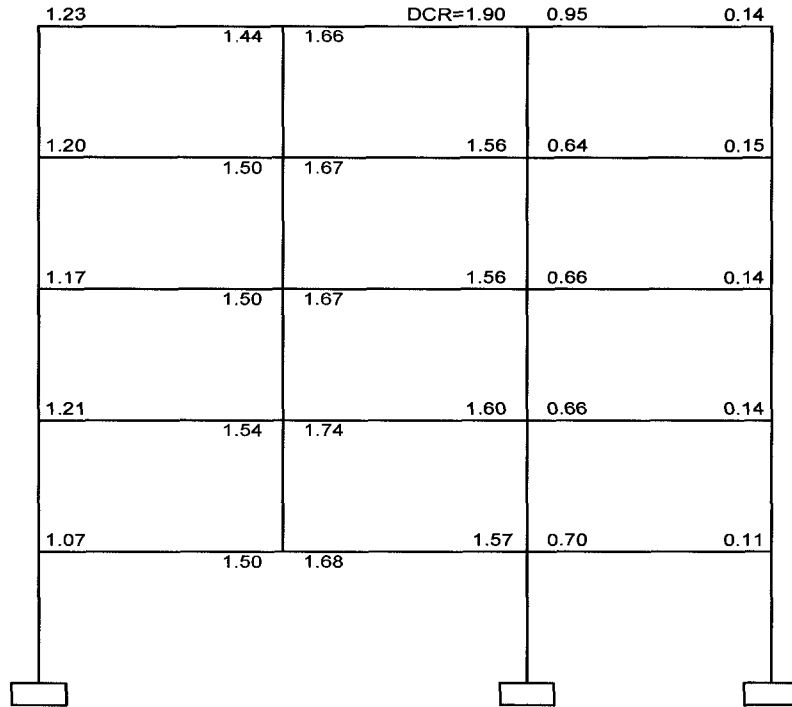


Figure 4.3: Linear response of 5-storey building for exterior column removal in short (above) and long (below) directions

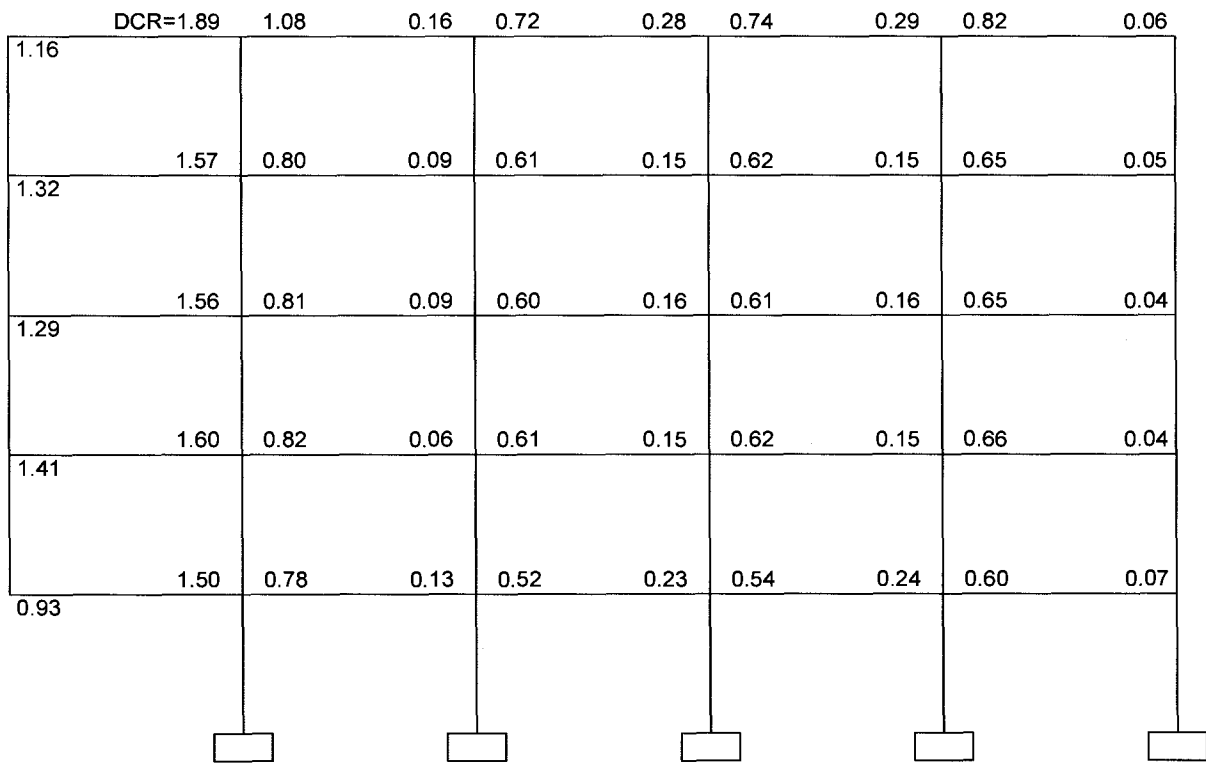
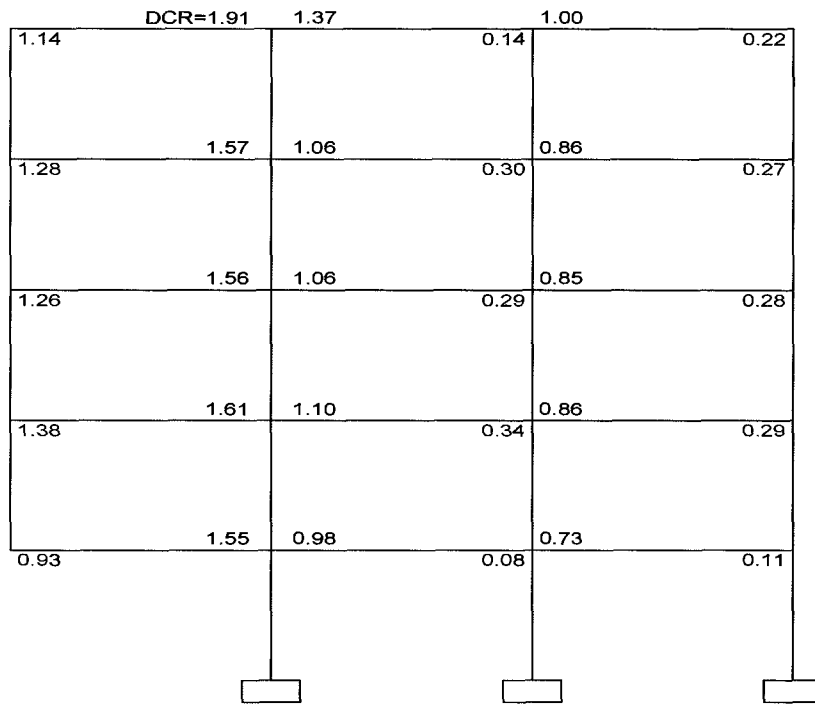


Figure 4.4: Linear response of 5-storey building for corner column removal in short (above) and long (below) directions

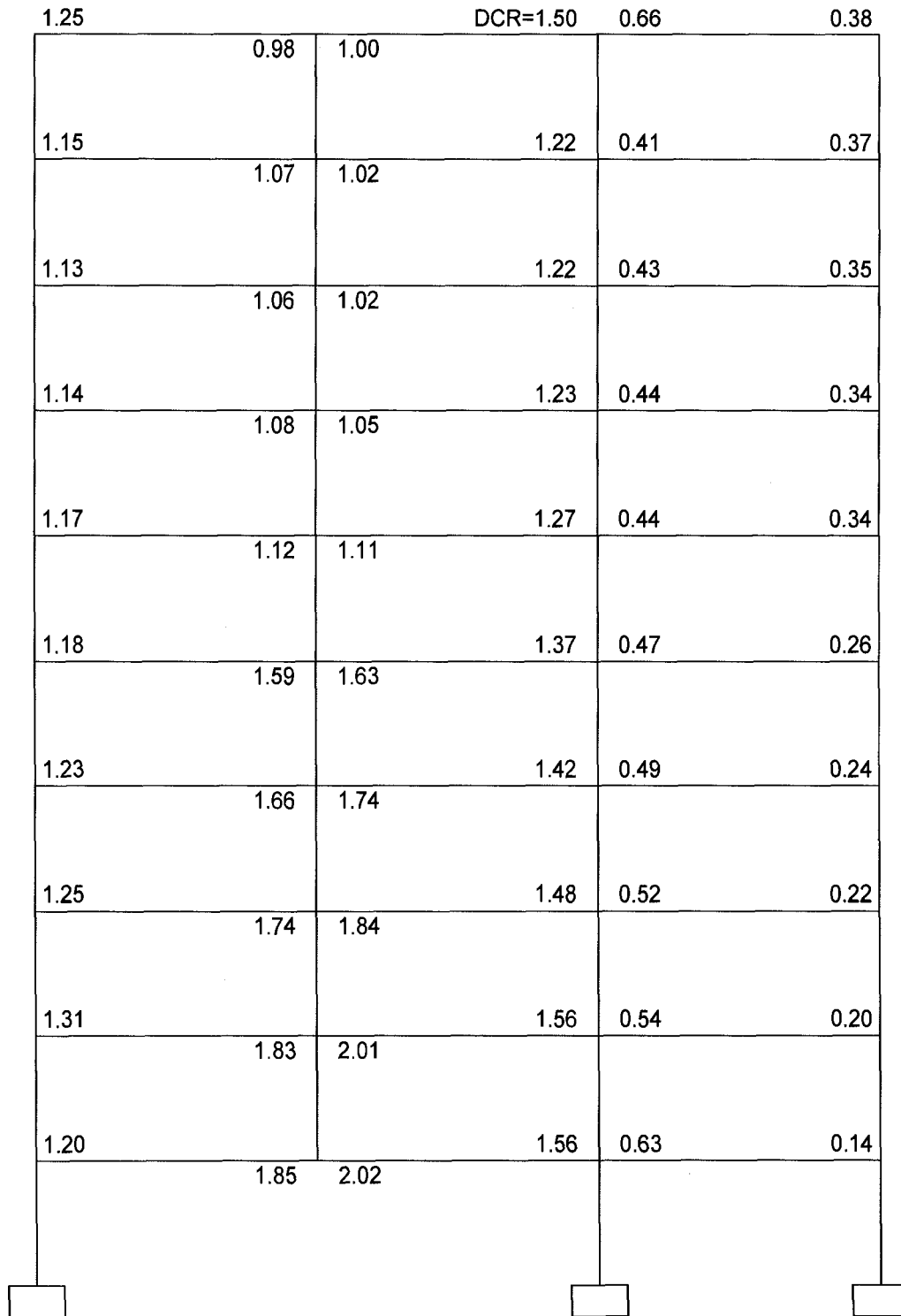


Figure 4.6: a) Linear response of 10-storey building for exterior column removal in short direction

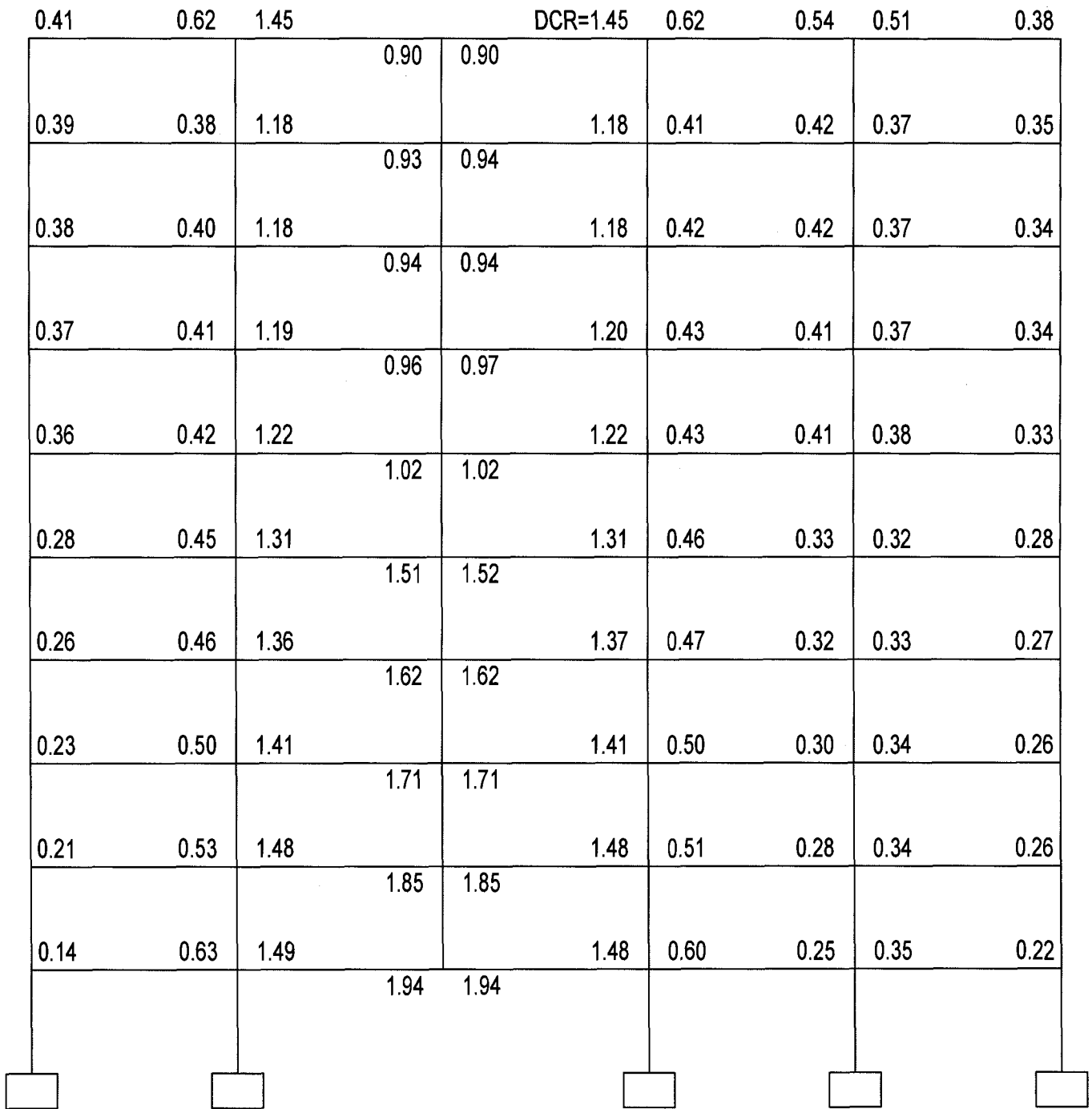


Figure 4.6 (Cont'd): b) Linear response of 10-storey building for exterior column removal in long direction

	DCR=1.58	1.16	0.07	0.93
0.84				0.10
	1.28	0.89		0.78
0.92			0.04	0.14
	1.28	0.90		0.77
0.91			0.02	0.14
	1.30	0.90		0.78
0.94			0.04	0.15
	1.33	0.92		0.79
1.02			0.08	0.19
	1.42	0.99		0.75
1.30			0.29	0.34
	1.46	1.02		0.78
1.42			0.34	0.39
	1.51	1.05		0.79
1.47			0.38	0.42
	1.58	1.11		0.82
1.70			0.49	0.48
	1.56	1.02		0.68
1.22			0.24	0.24

Figure 4.7: a) Linear response of 10-storey building for corner column removal in short direction

	DCR=1.56	0.90	0.30	0.74	0.27	0.74	0.28	0.71	0.17
0.87	1.28	0.66	0.21	0.62	0.15	0.61	0.15	0.56	0.14
0.96	1.28	0.66	0.21	0.61	0.16	0.60	0.16	0.56	0.14
0.96	1.30	0.67	0.20	0.61	0.16	0.60	0.16	0.56	0.14
0.98	1.33	0.68	0.18	0.62	0.15	0.61	0.15	0.58	0.12
1.06	1.40	0.73	0.10	0.57	0.10	0.57	0.10	0.54	0.06
1.34	1.45	0.75	0.07	0.58	0.10	0.58	0.09	0.56	0.04
1.45	1.49	0.78	0.05	0.57	0.10	0.58	0.10	0.58	0.02
1.50	1.57	0.83	0.01	0.58	0.09	0.58	0.09	0.60	0.00
1.70	1.50	0.80	0.06	0.48	0.18	0.50	0.18	0.54	0.04
1.21									

Figure 4.7 (Cont'd): b) Linear response of 10-storey building for corner column removal in long direction

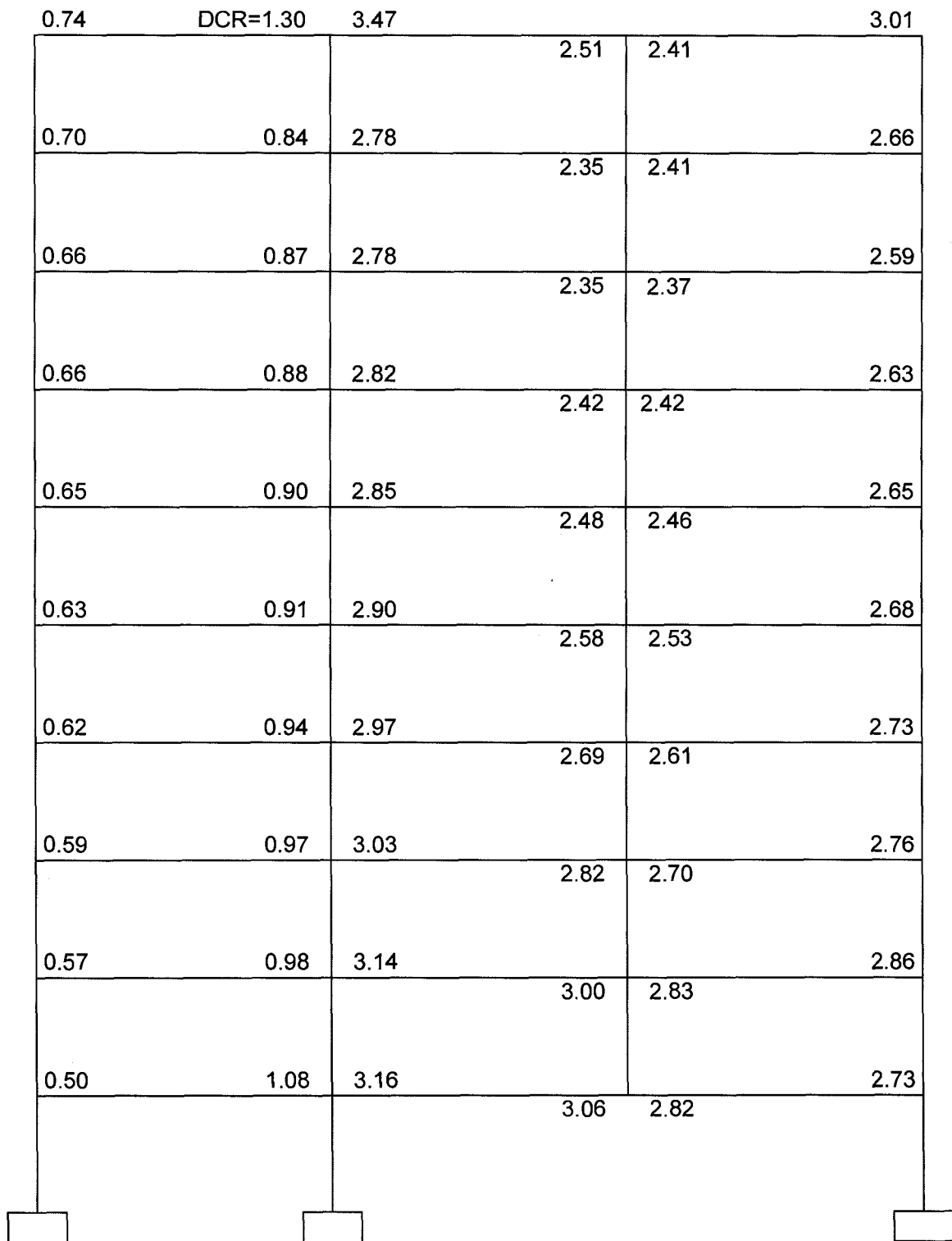


Figure 4.8: a) Linear response of 10-storey building for interior column removal in short direction

0.85	1.19	3.40		DCR=3.39	1.14	1.06	1.06	0.81
			2.43	2.42				
0.78	0.74	2.72		2.73	0.76	0.84	0.77	0.70
			2.24	2.24				
0.74	0.78	2.71		2.72	0.78	0.83	0.77	0.68
			2.25	2.25				
0.74	0.79	2.74		2.74	0.79	0.82	0.78	0.69
			2.30	2.30				
0.73	0.81	2.78		2.78	0.80	0.82	0.78	0.68
			2.35	2.36				
0.70	0.83	2.82		2.82	0.82	0.80	0.78	0.67
			2.43	2.44				
0.68	0.86	2.87		2.88	0.84	0.78	0.79	0.66
			2.54	2.54				
0.65	0.90	2.94		2.94	0.86	0.76	0.80	0.66
			2.65	2.65				
0.63	0.92	3.02		3.02	0.88	0.74	0.81	0.66
			2.79	2.79				
0.53	1.04	3.05		3.04	0.97	0.69	0.83	0.60
			2.89	2.89				

Figure 4.8 (Cont'd): b) Linear response of 10-storey building for interior column removal in long direction

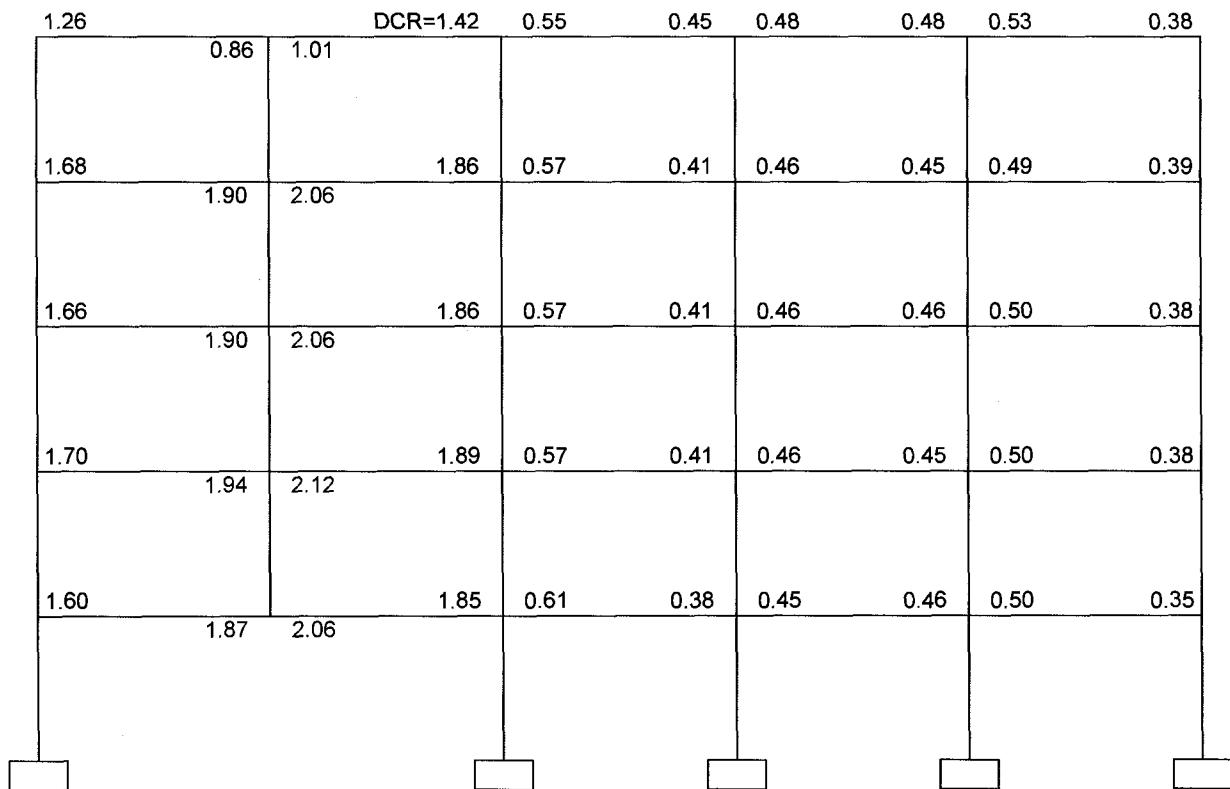
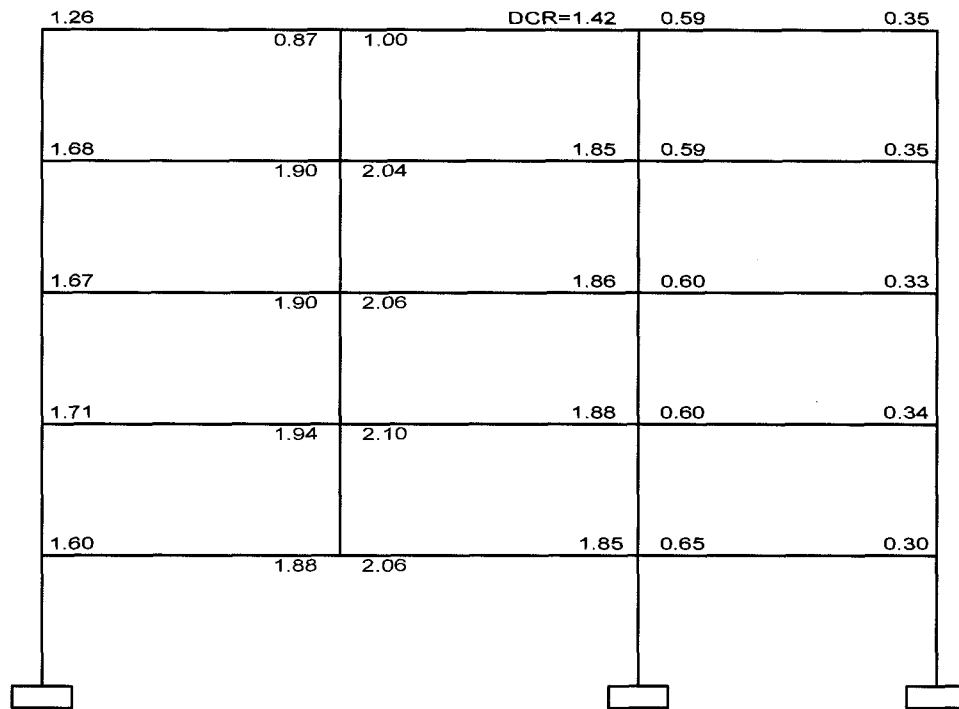


Figure 4.9: Linear response of 5-storey building with 9m span length for exterior column removal in short and long directions

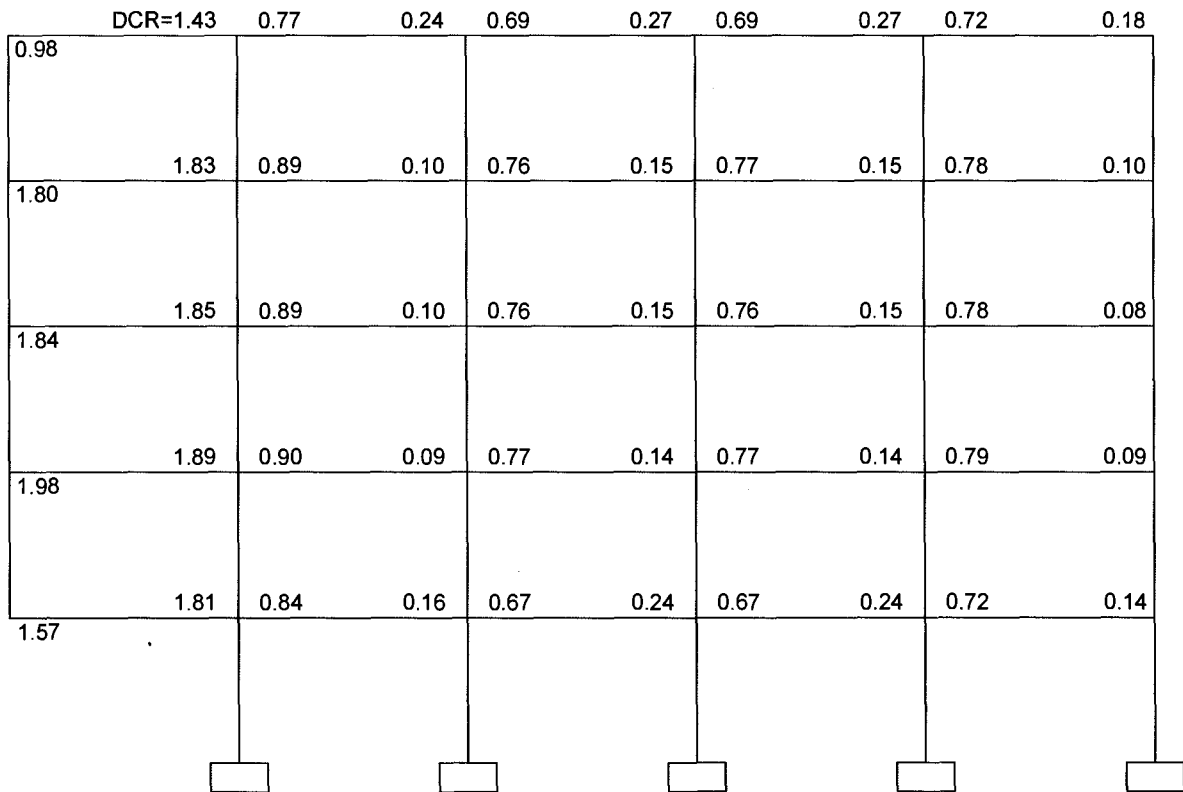
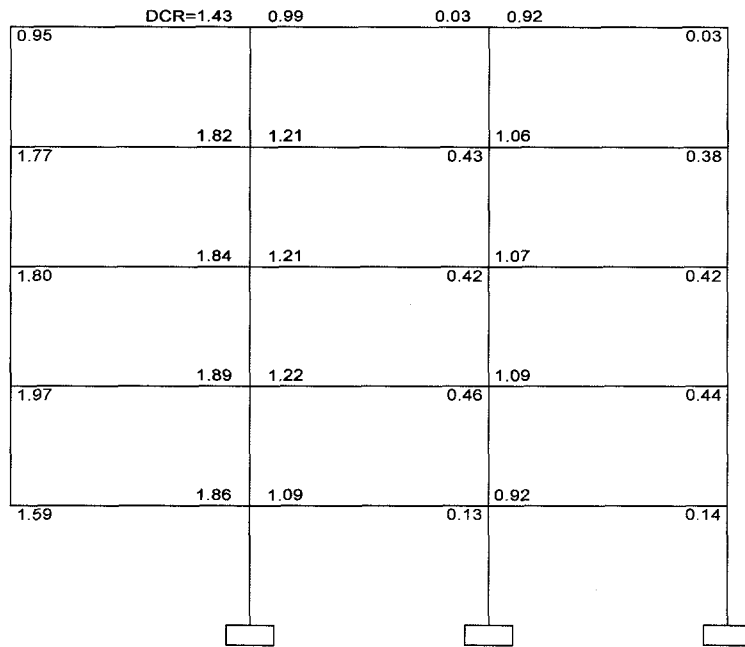


Figure 4.10: Linear response of 5-storey building with 9m span length for corner column removal in short and long directions

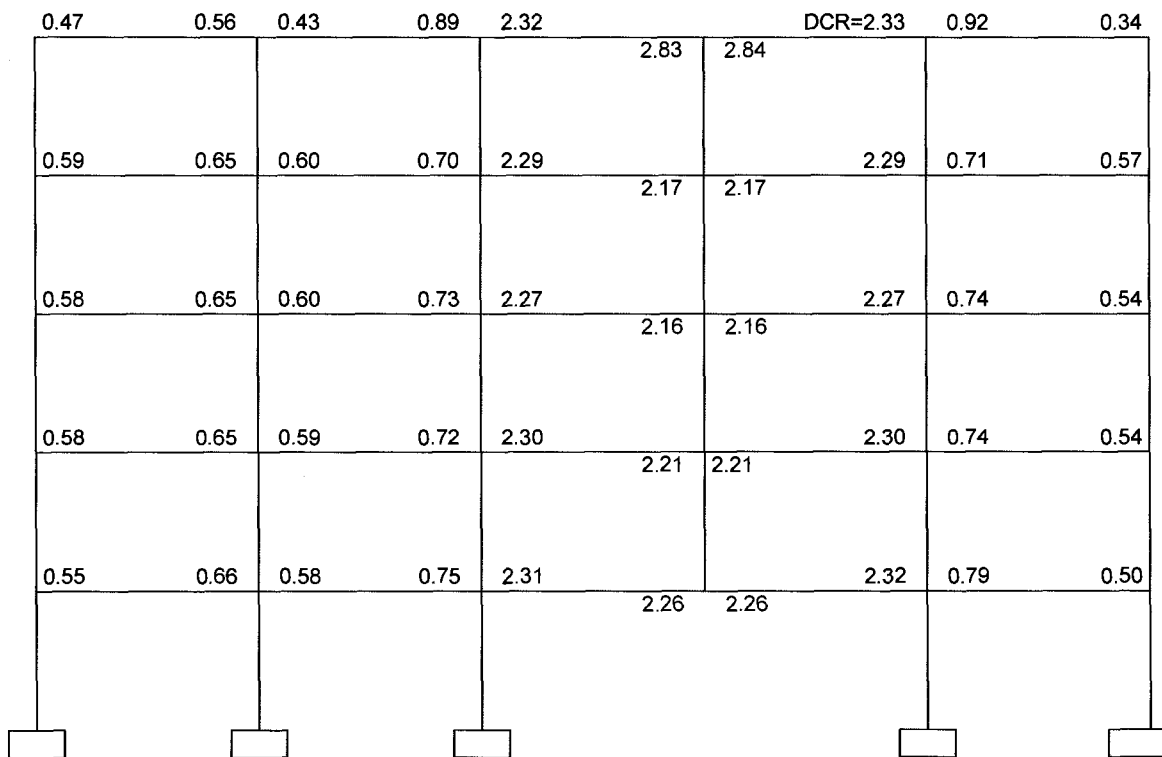
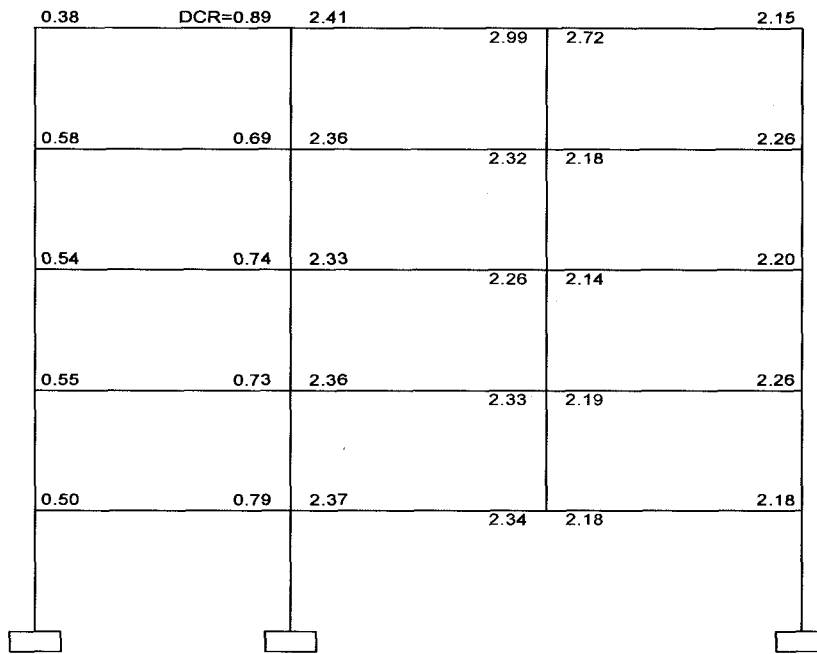


Figure 4.11: Linear response of 5-storey building with 9m span length for interior column removal in short and long directions

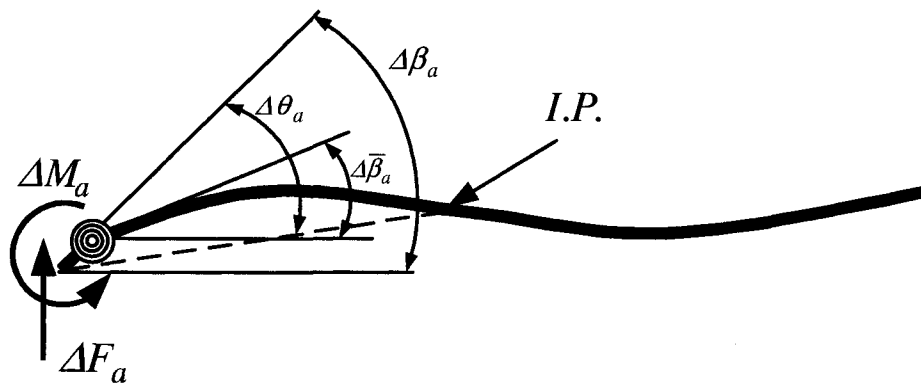


Figure 4.12: Element modeling – Inelastic hinge for flexure

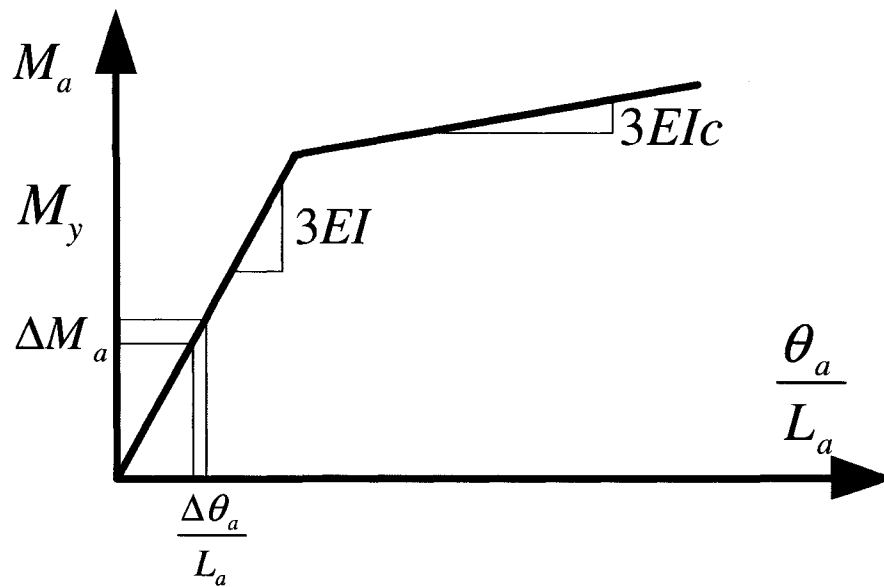


Figure 4.13: Moment – Rotation relationship for inelastic analysis

Exterior Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	40	38
2% Drift Ratio	53	60
4% Drift Ratio	60	72

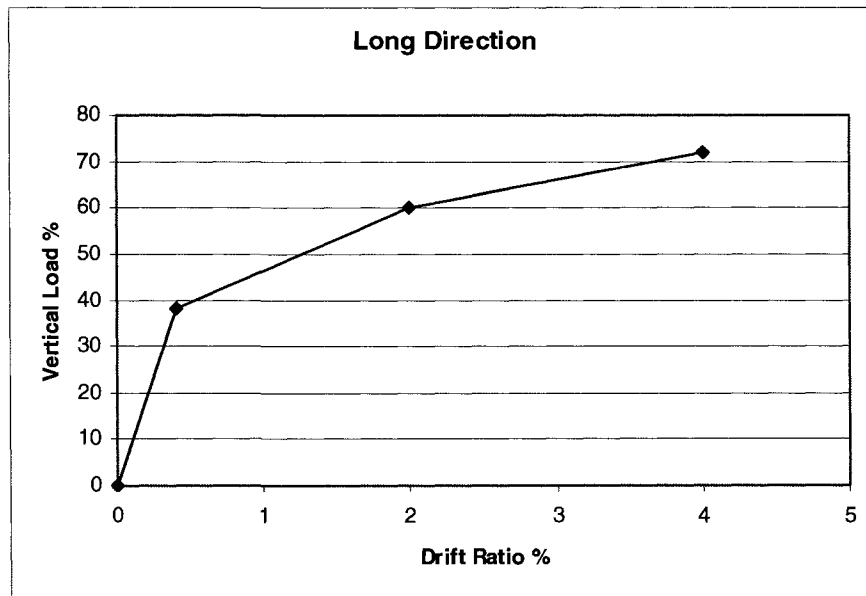
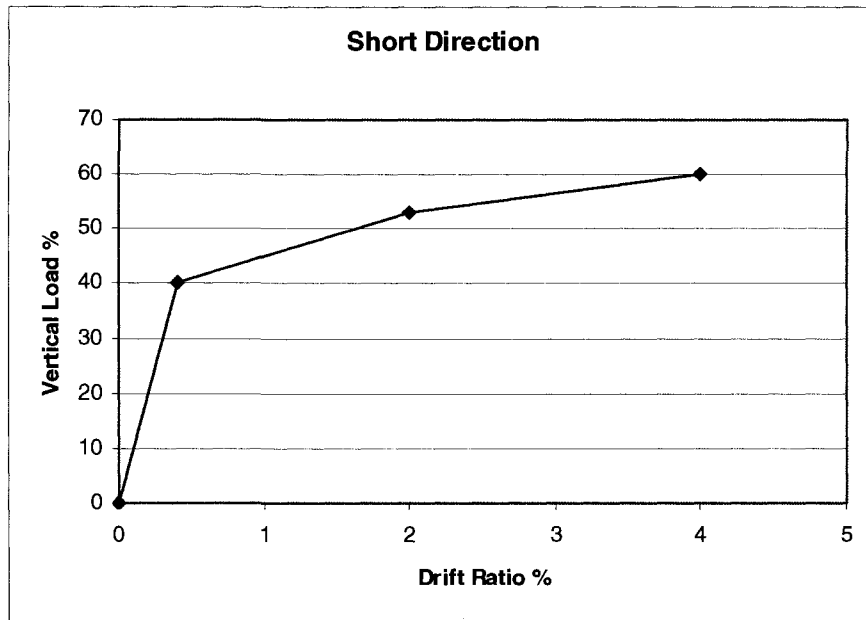


Figure 4.14: Inelastic analysis of 5-storey reinforced concrete building with 6.0 m spans under the removal of an exterior edge column

Corner Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	51	51
2% Drift Ratio	60	60
4% Drift Ratio	71	72

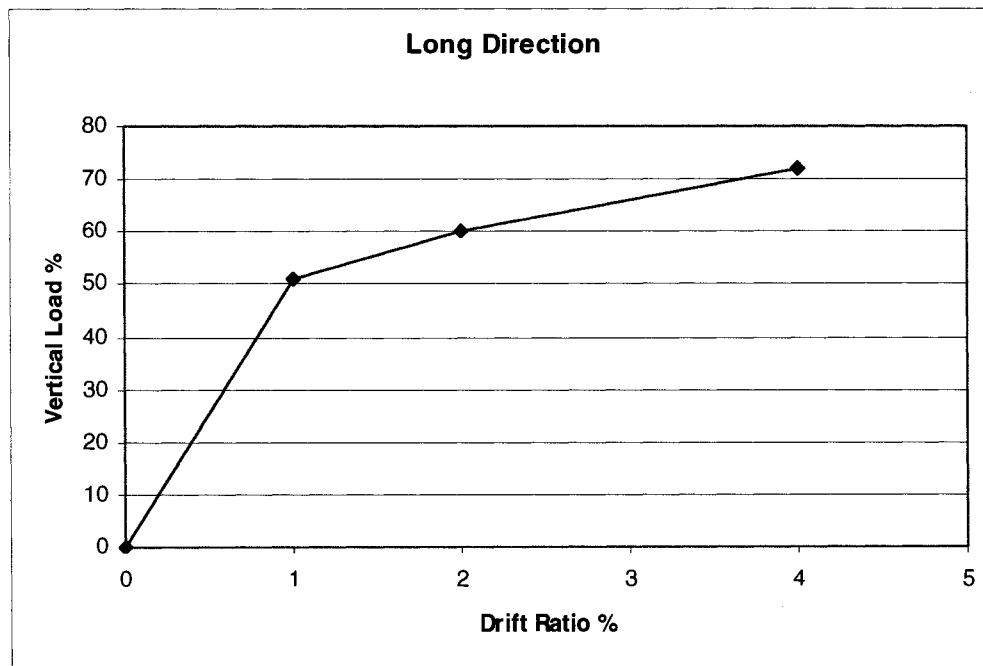
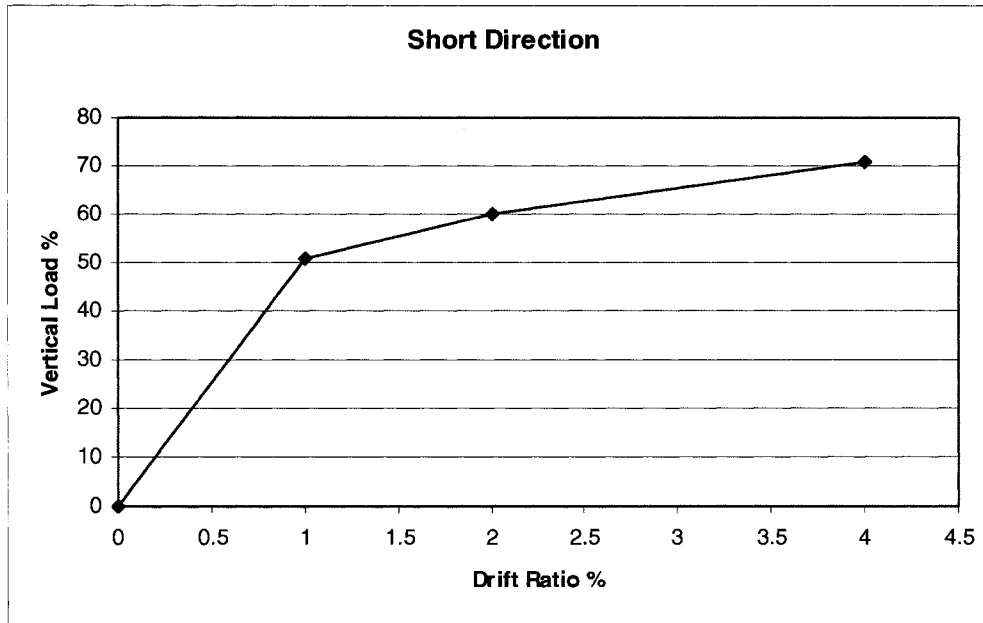


Figure 4.15: Inelastic analysis of 5-storey reinforced concrete building with 6.0 m spans under the removal of a corner column

Interior Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	22	22
2% Drift Ratio	32	32
4% Drift Ratio	39	40

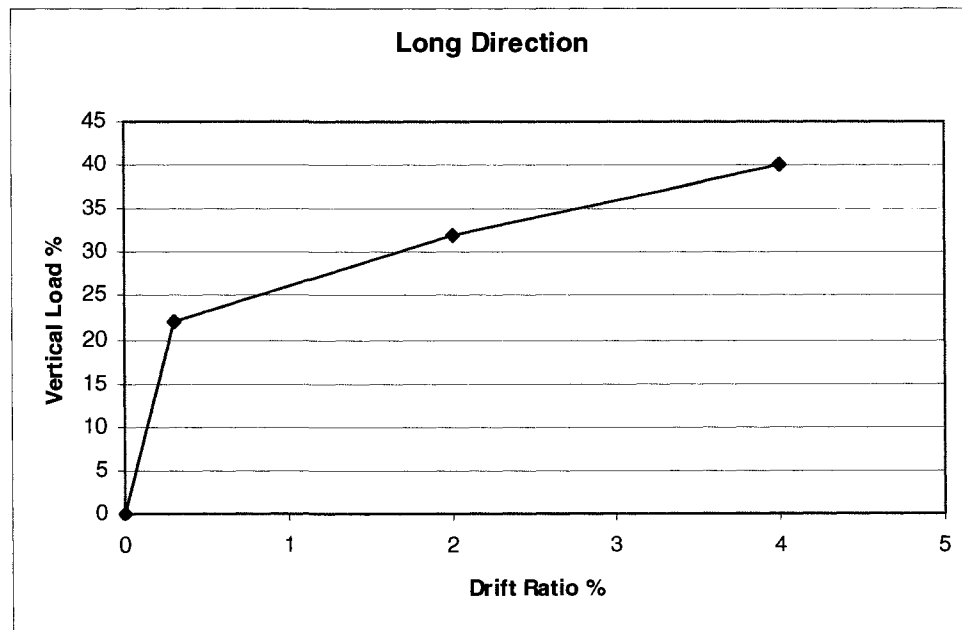
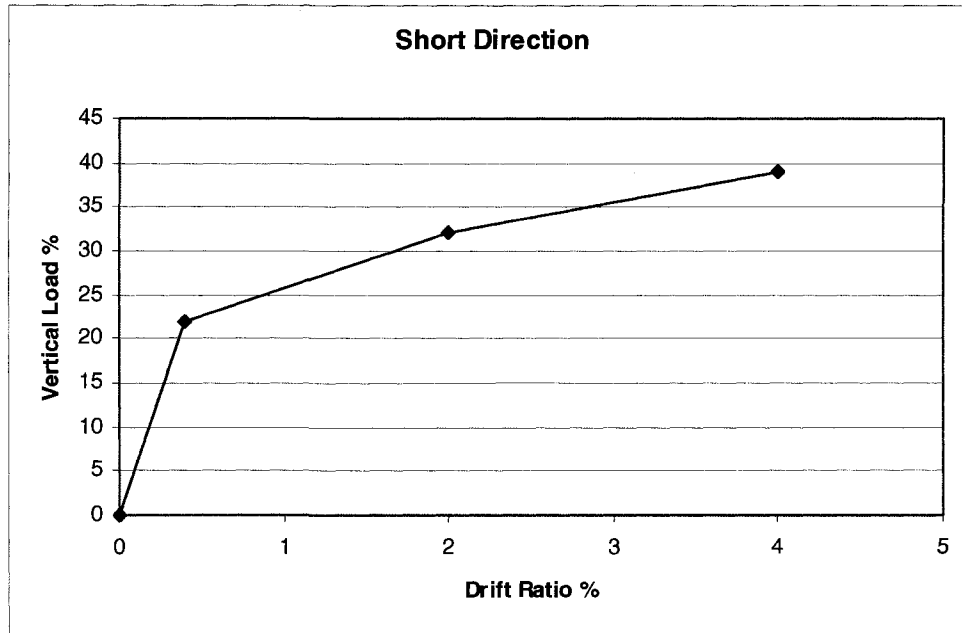


Figure 4.16: Inelastic analysis of 5-storey reinforced concrete building with 6.0 m spans under the removal of an interior column

Exterior Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	34	35
2% Drift Ratio	68	69
4% Drift Ratio	84	87

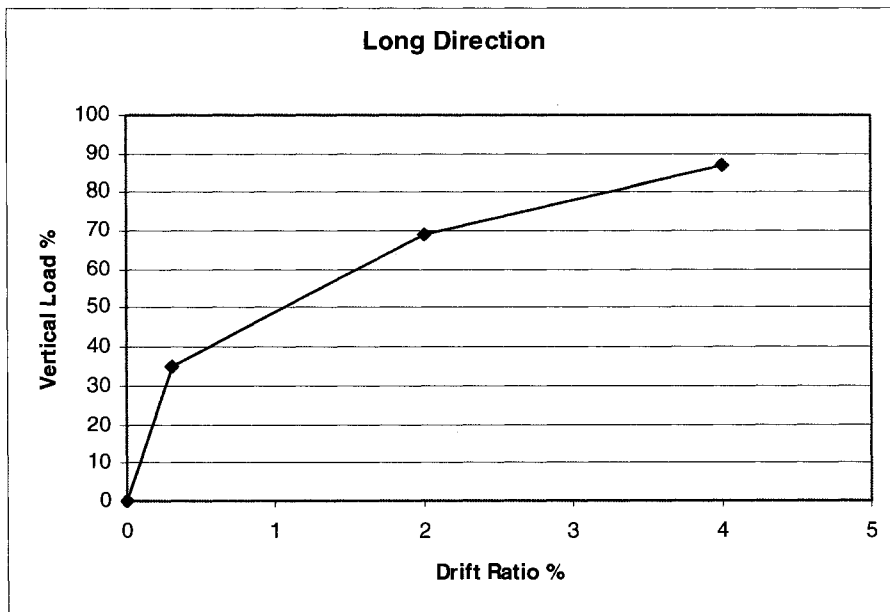
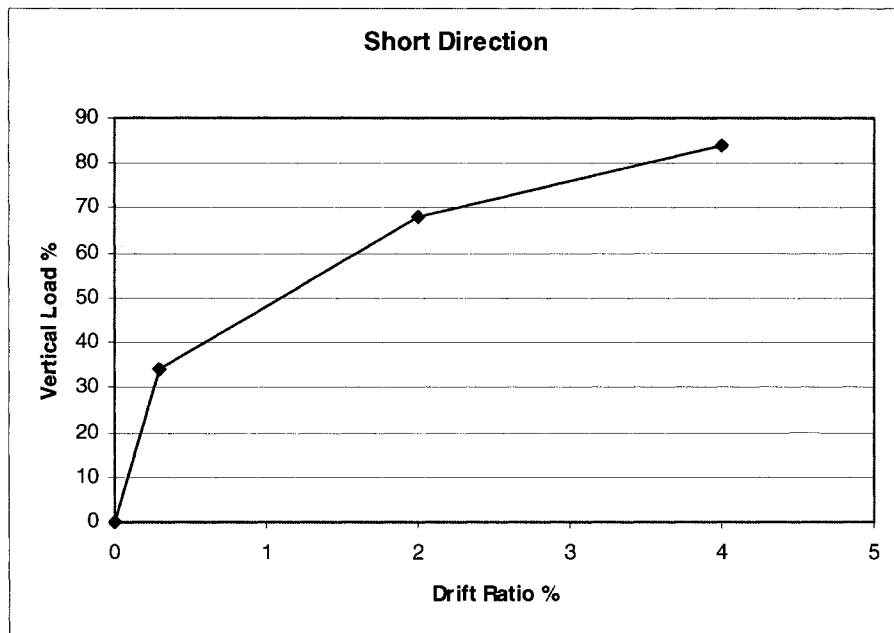


Figure 4.17: Inelastic analysis of 10-storey reinforced concrete building with 6.0 m spans under the removal of an exterior edge column

Corner Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	54	54
2% Drift Ratio	65	66
4% Drift Ratio	80	80

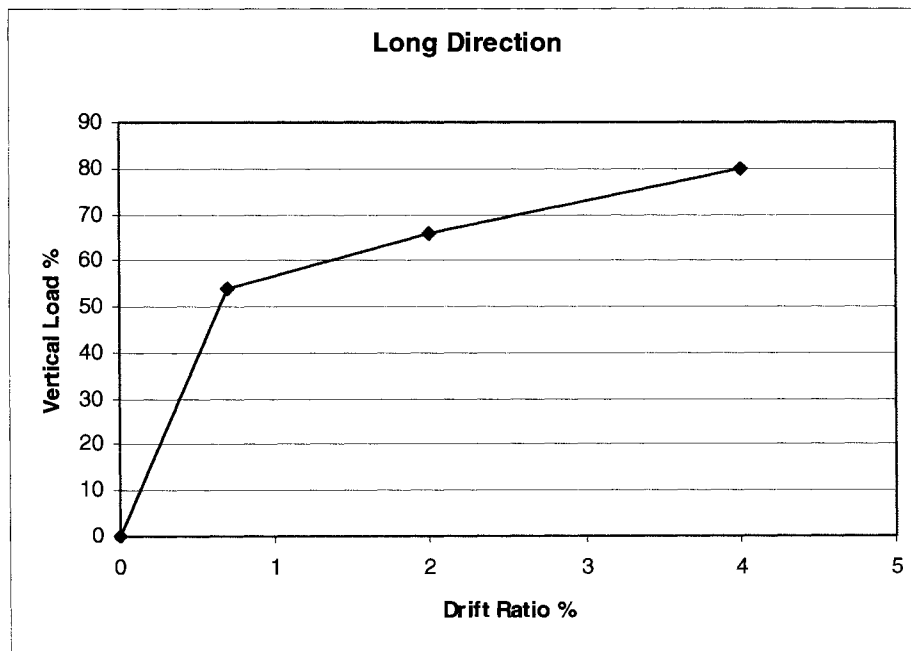
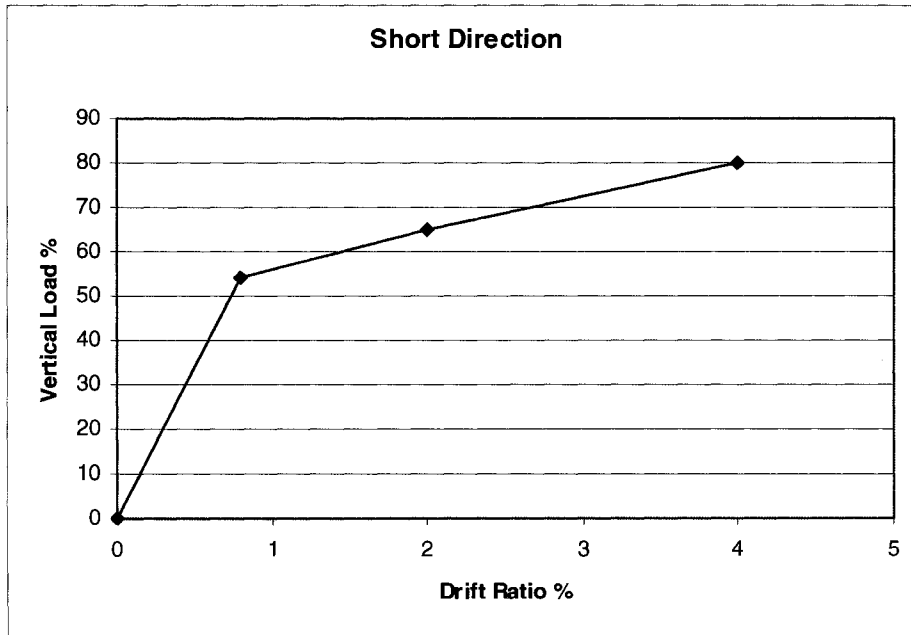


Figure 4.18: Inelastic analysis of 10-storey reinforced concrete building with 6.0 m spans under the removal of a corner column

Interior Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	20	20
2% Drift Ratio	32	32
4% Drift Ratio	38	41

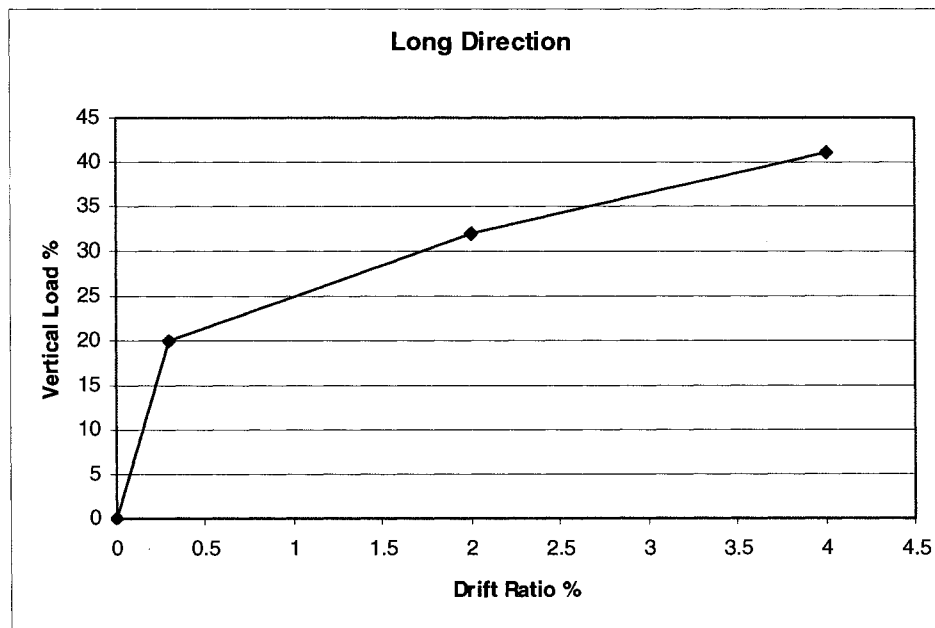
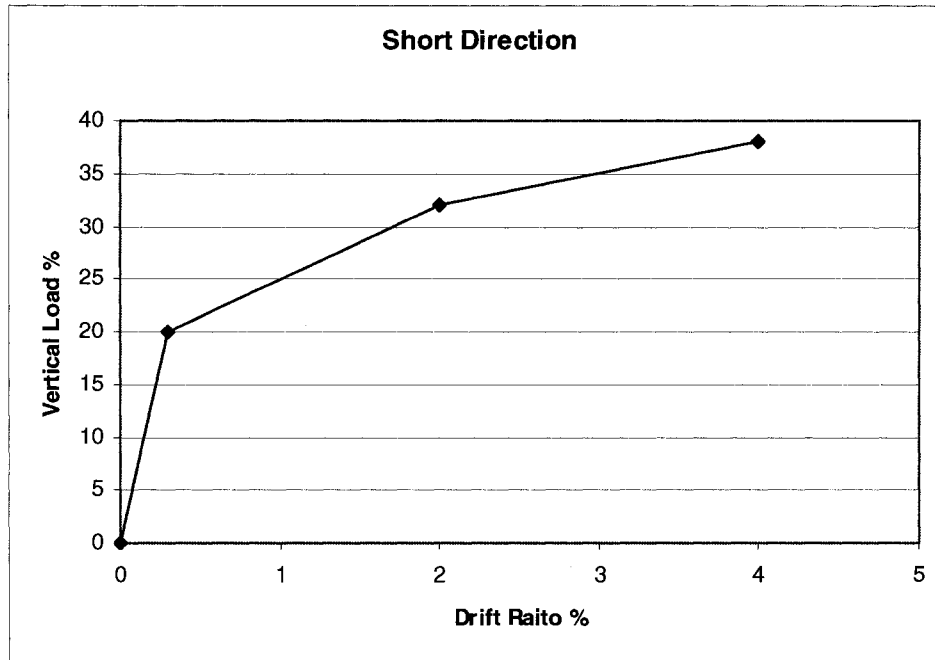


Figure 4.19: Inelastic analysis of 10-storey reinforced concrete building with 6.0 m spans under the removal of an interior column

Exterior Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	32	33
2% Drift Ratio	49	49
4% Drift Ratio	62	62

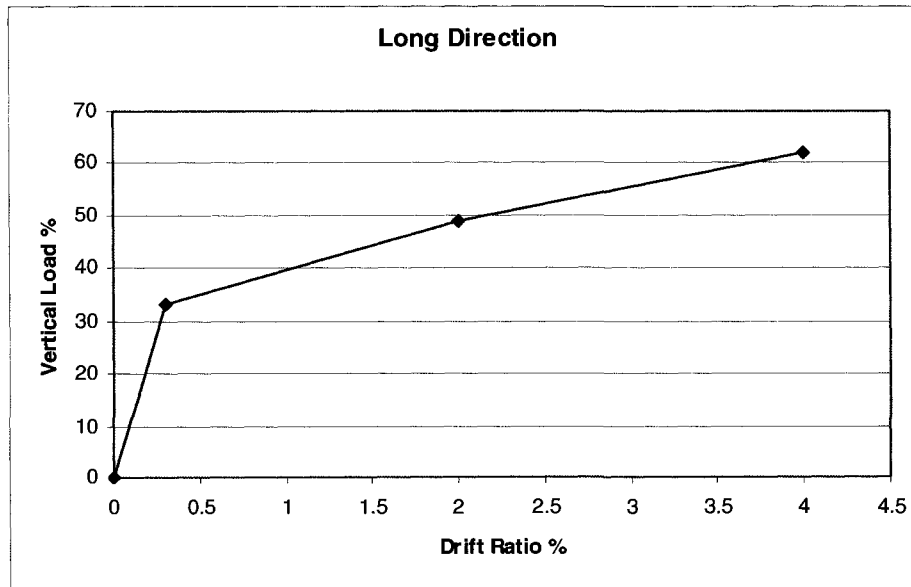
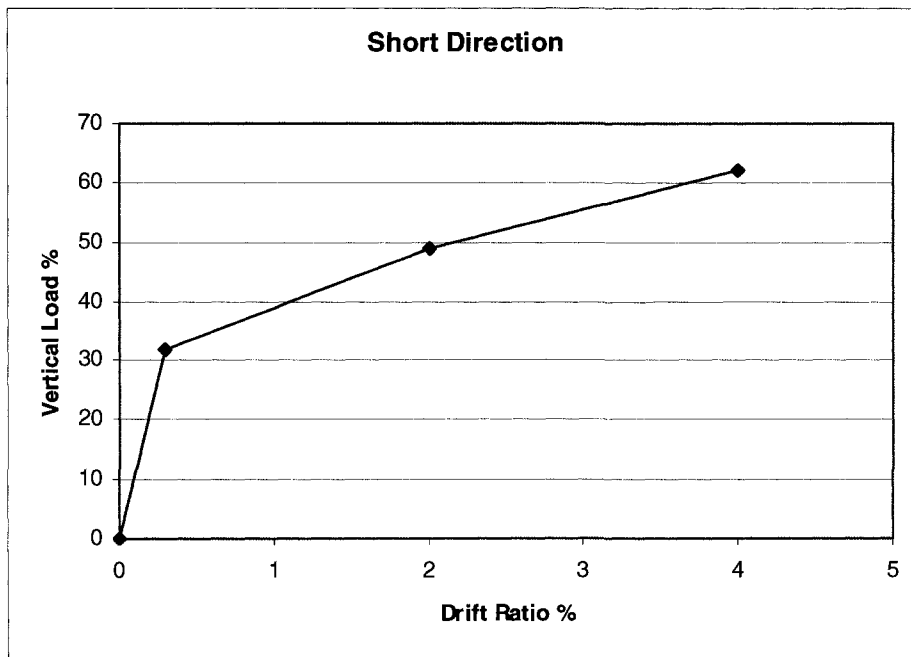


Figure 4.20: Inelastic analysis of 5-storey reinforced concrete building with 9.0 m spans under the removal of an exterior edge column

Corner Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	39	40
2% Drift Ratio	48	49
4% Drift Ratio	55	58

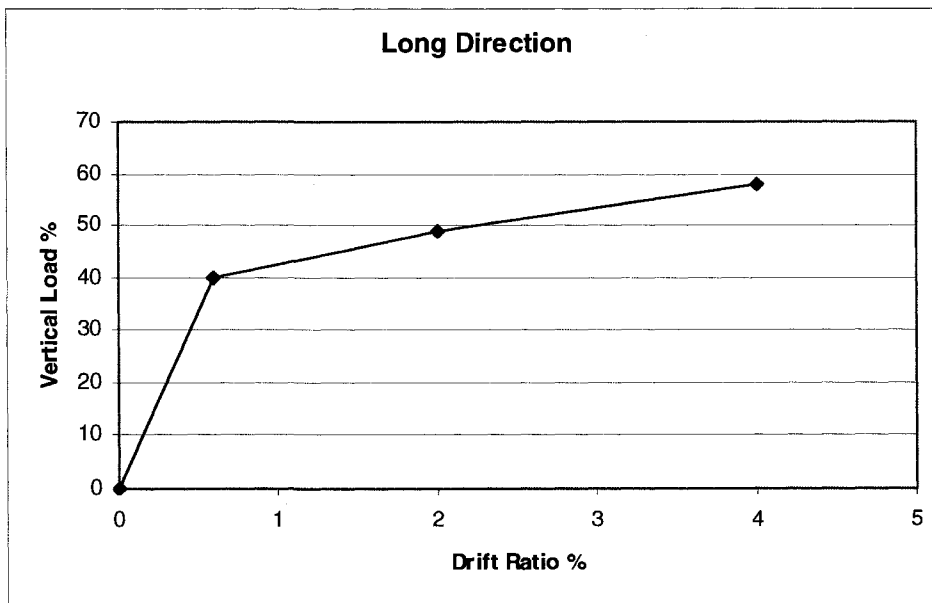
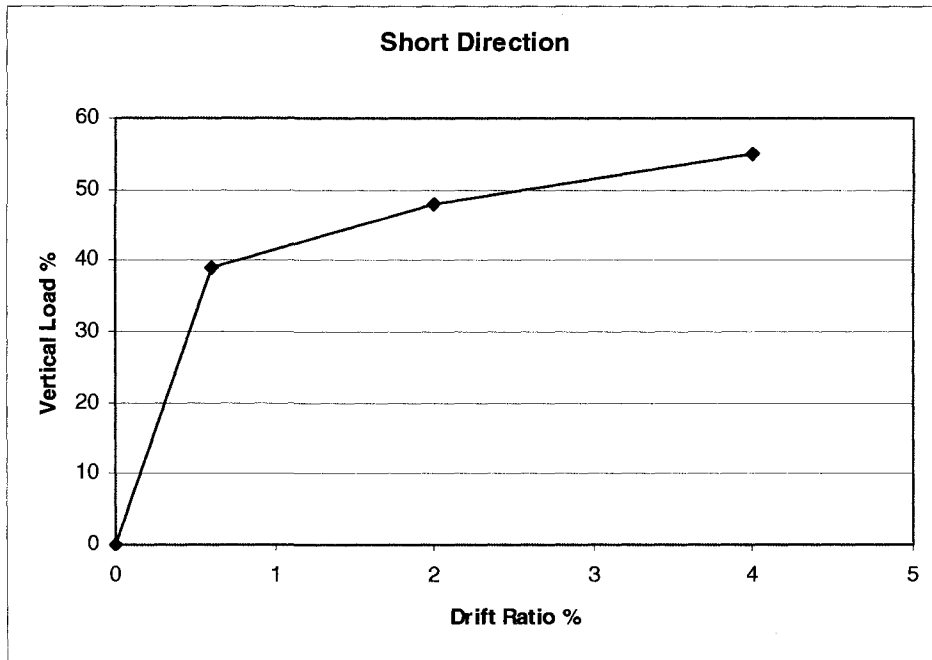


Figure 4.21: Inelastic analysis of 10-storey reinforced concrete building with 9.0 m spans under the removal of a corner column

Interior Column	Load Percentage (%)	
	Short Direction	Long Direction
First Yielding	21	22
2% Drift Ratio	32	32
4% Drift Ratio	39	39

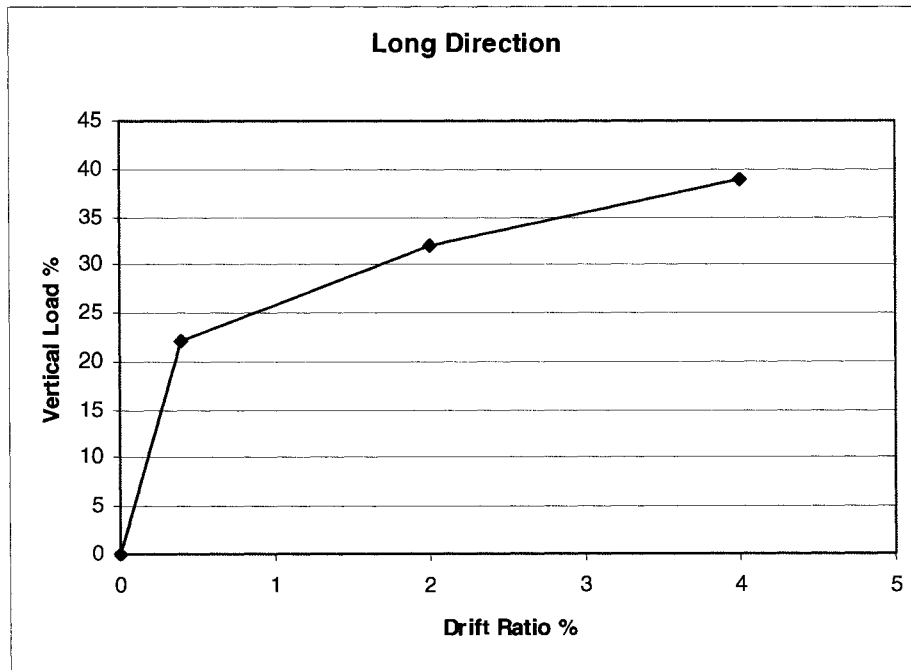
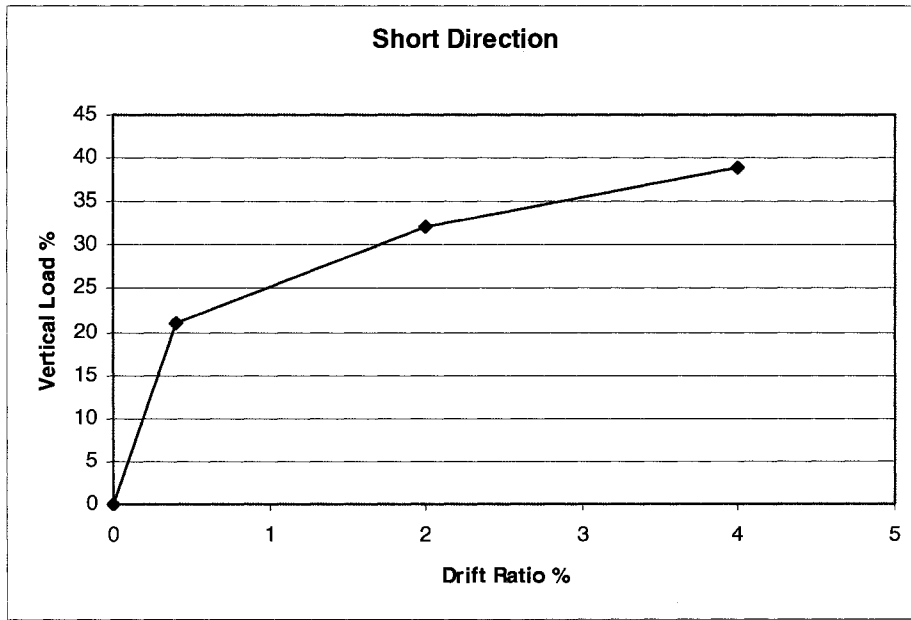


Figure 4.22: Inelastic analysis of 10-storey reinforced concrete building with 9.0 m spans under the removal of an interior column

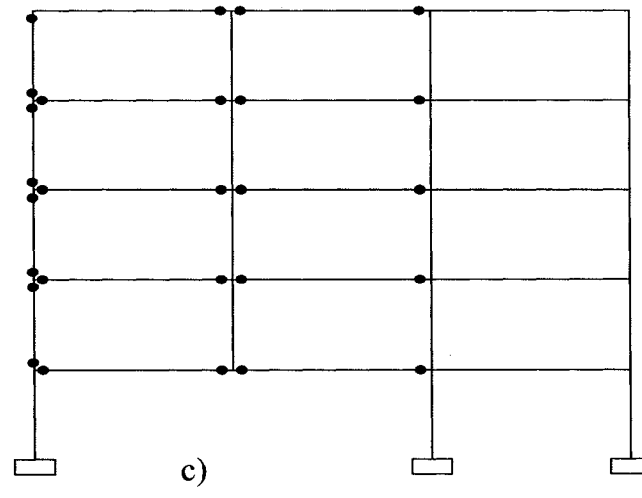
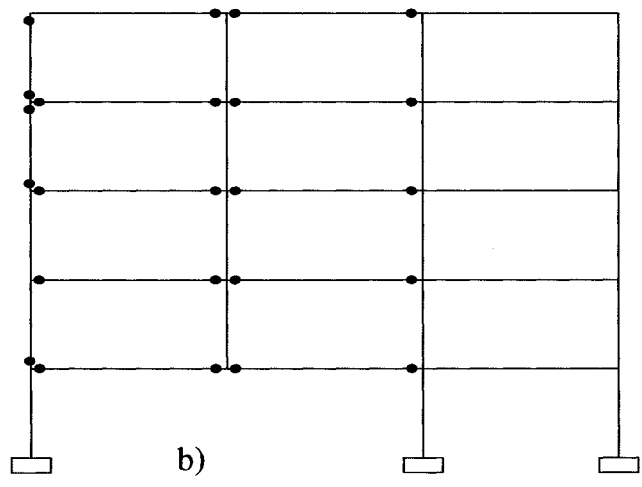
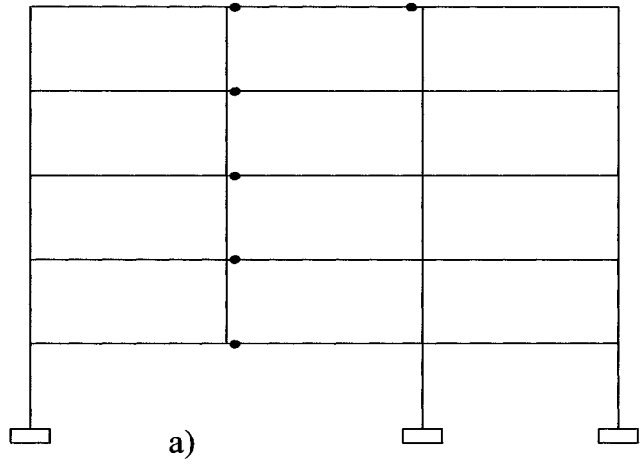


Figure 4.23: Formation of plastic hinges due to the loss of an exterior edge column in the short direction of 5-storey building with 6.0m spans at:
 a) First yield b) 2% Deflection c) 4% Deflection

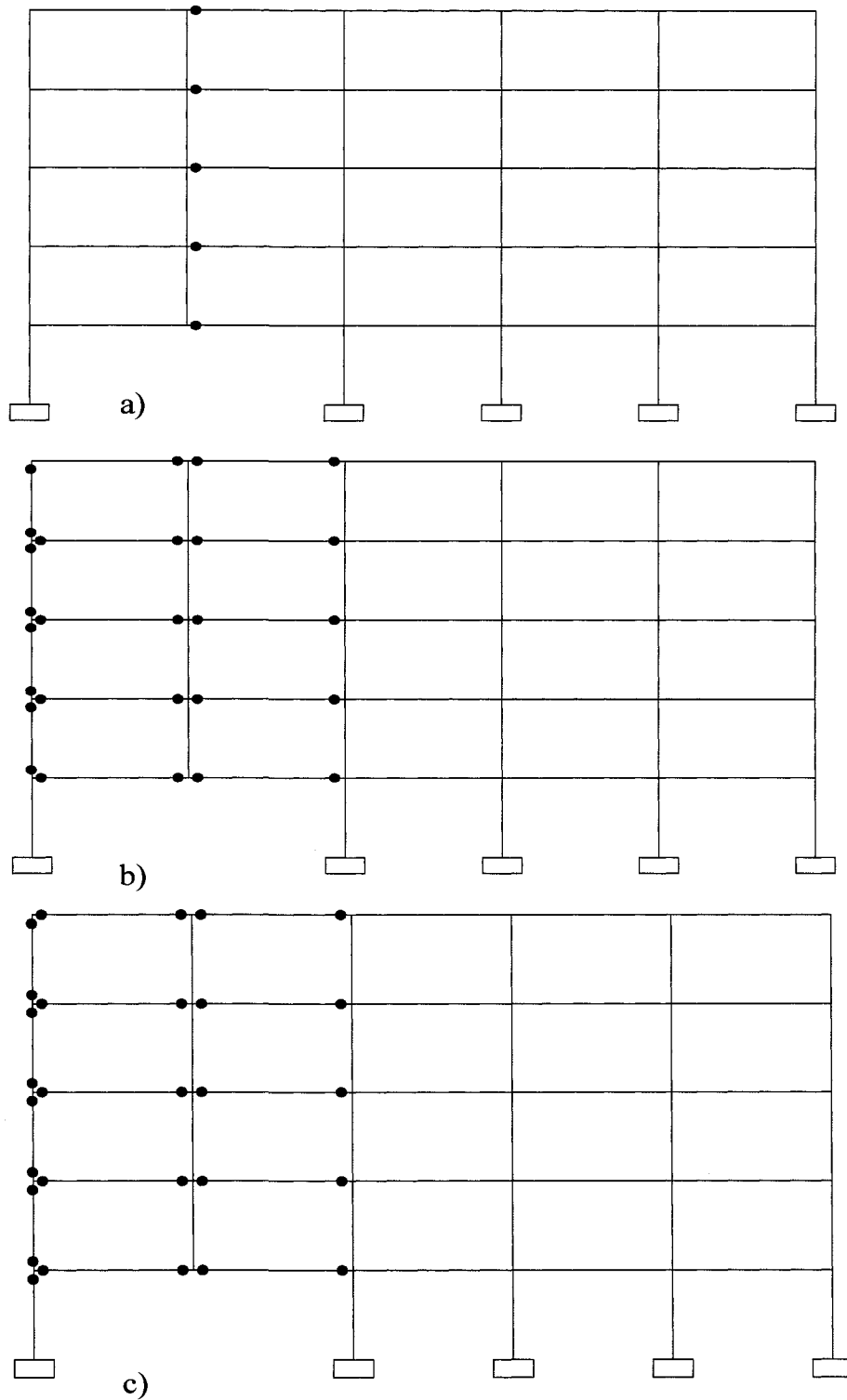


Figure 4.24: Formation of plastic hinges due to the loss of an exterior edge column in the long direction of 5-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

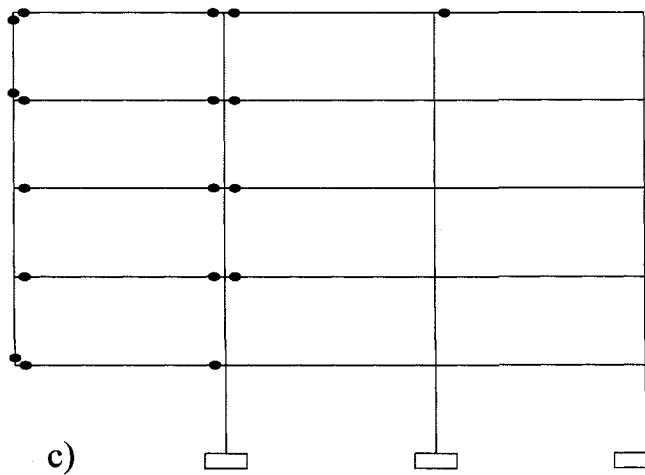
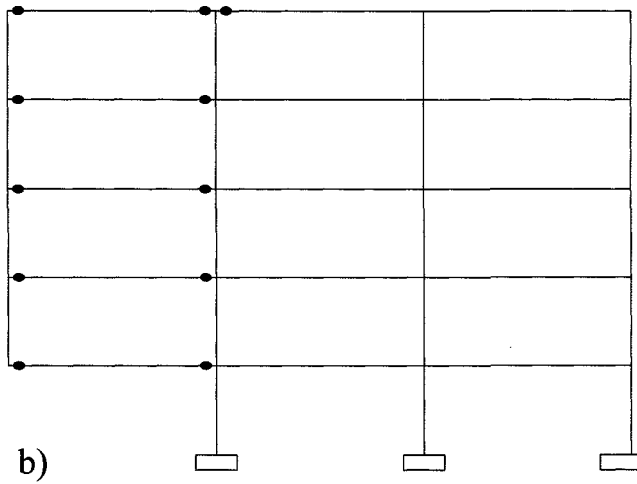
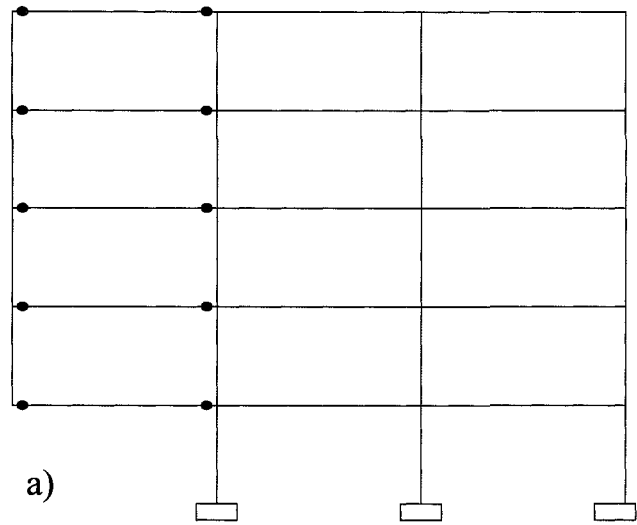


Figure 4.25: Formation of plastic hinges due to the loss of a corner column in the short direction of 5-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

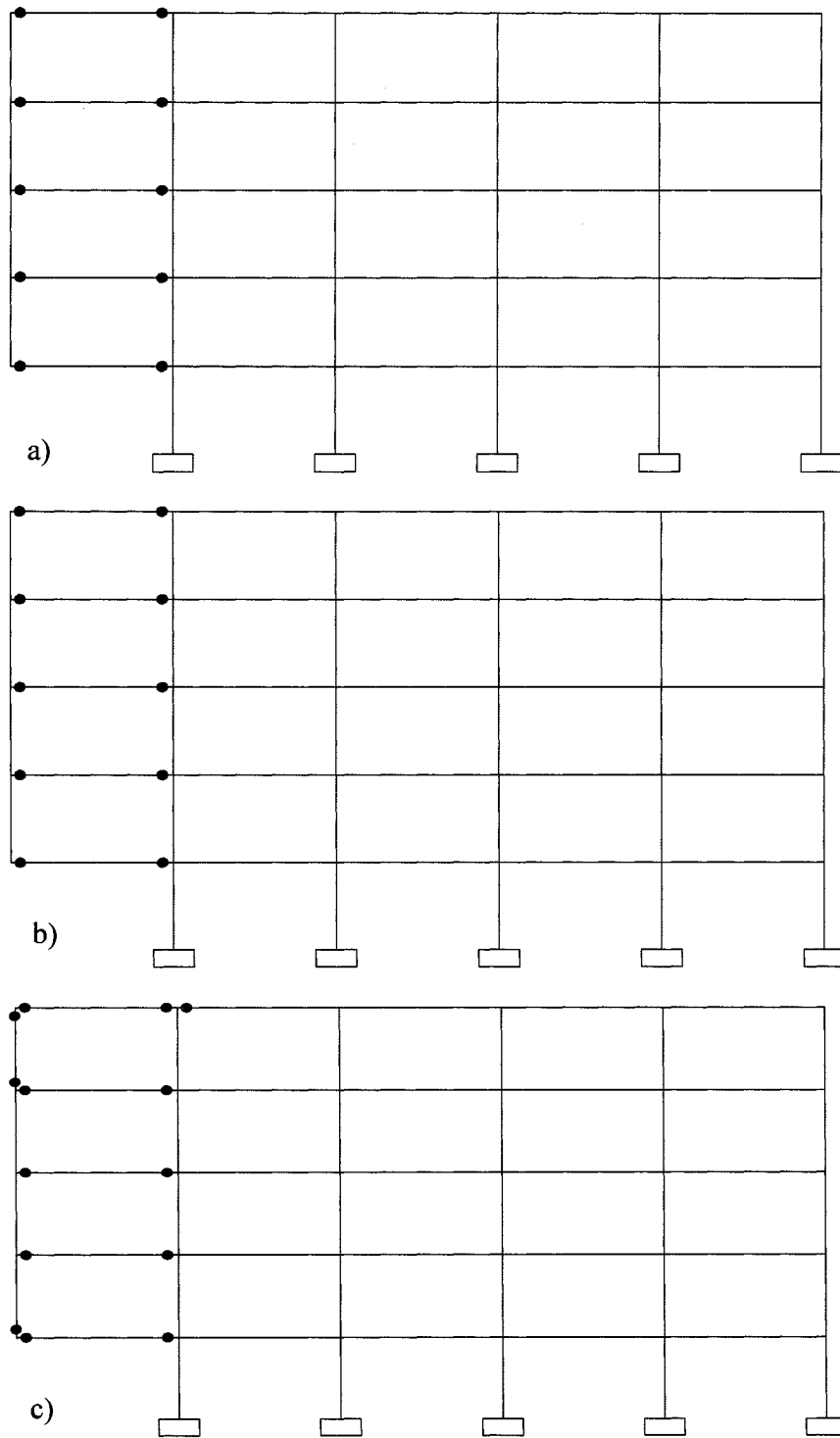


Figure 4.26: Formation of plastic hinges due to the loss of a corner column in the long direction of 5-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

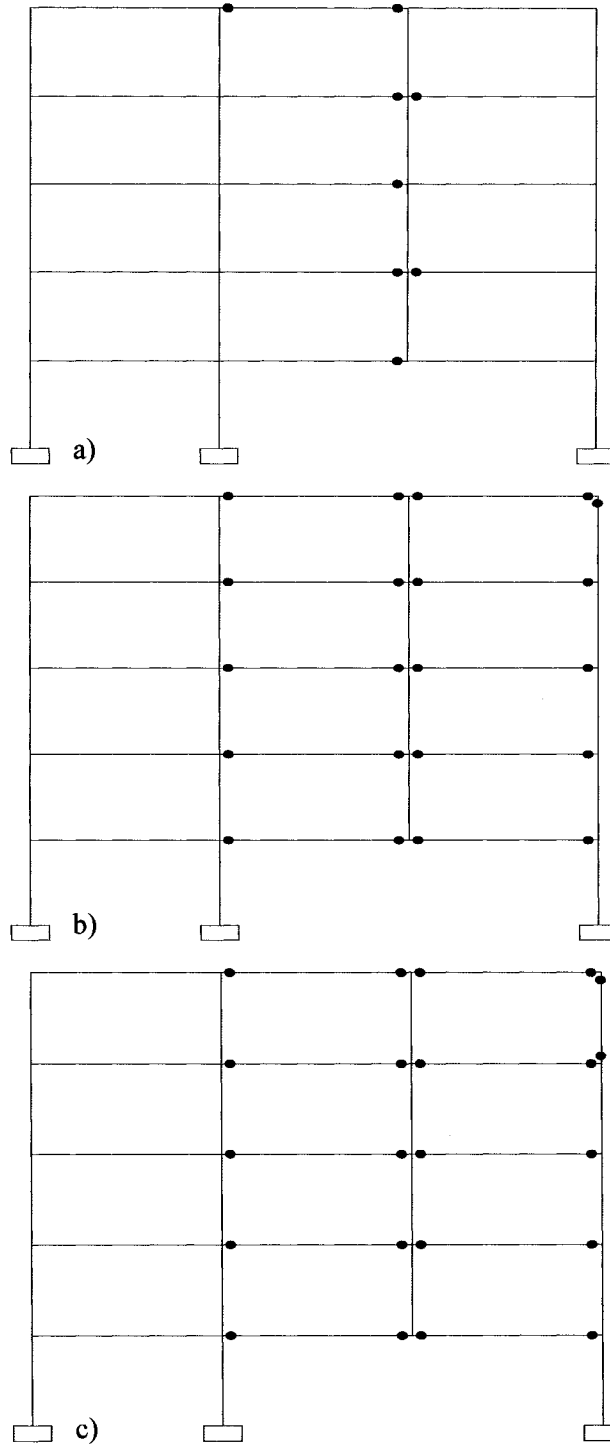


Figure 4.27: Formation of plastic hinges due to the loss of an interior column in the short direction of 5-storey building with 6.0m spans at:
 a) First yield b) 2% Deflection c) 4% Deflection

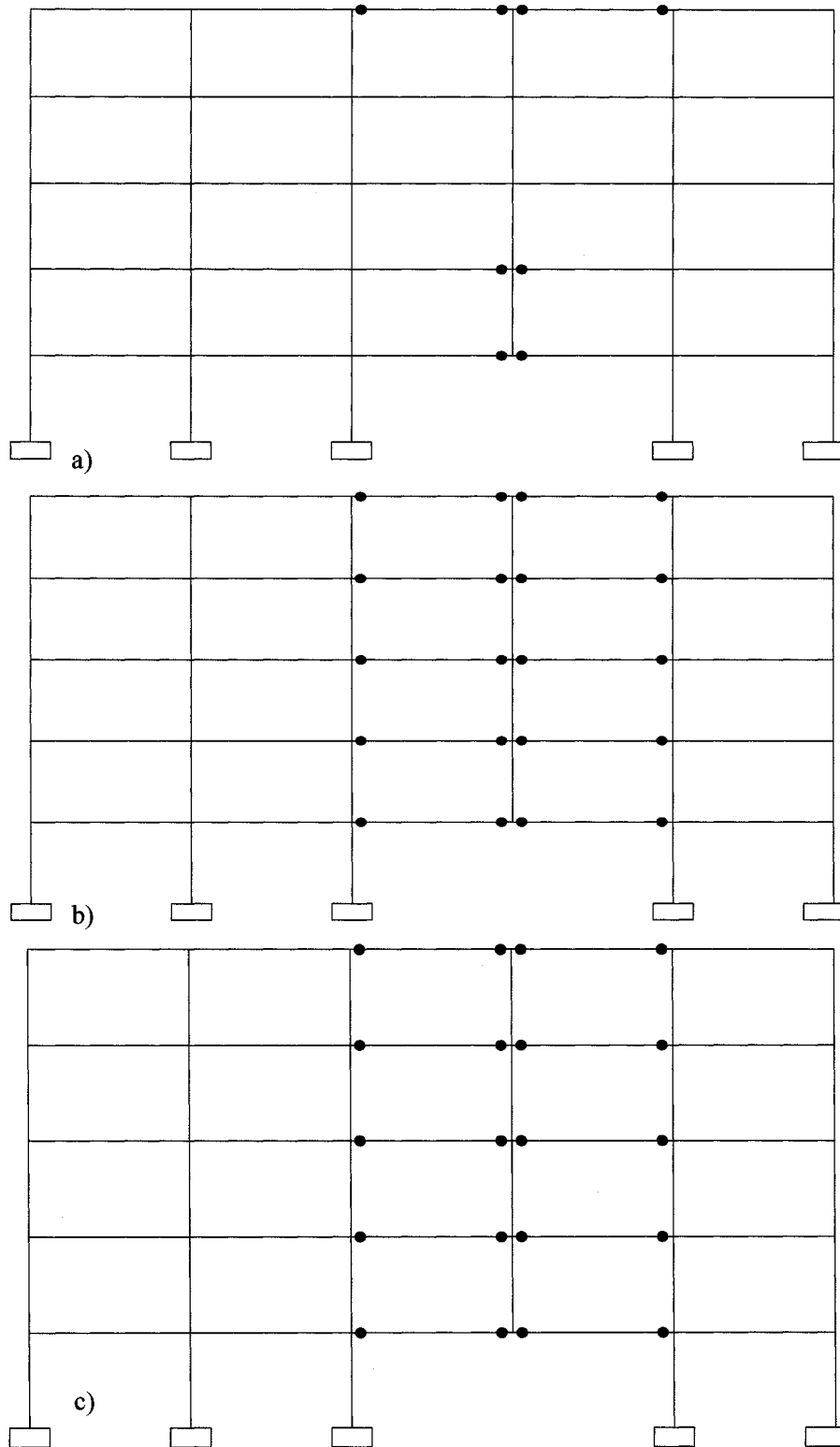


Figure 4.28: Formation of plastic hinges due to the loss of an interior column in the long direction of 5-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

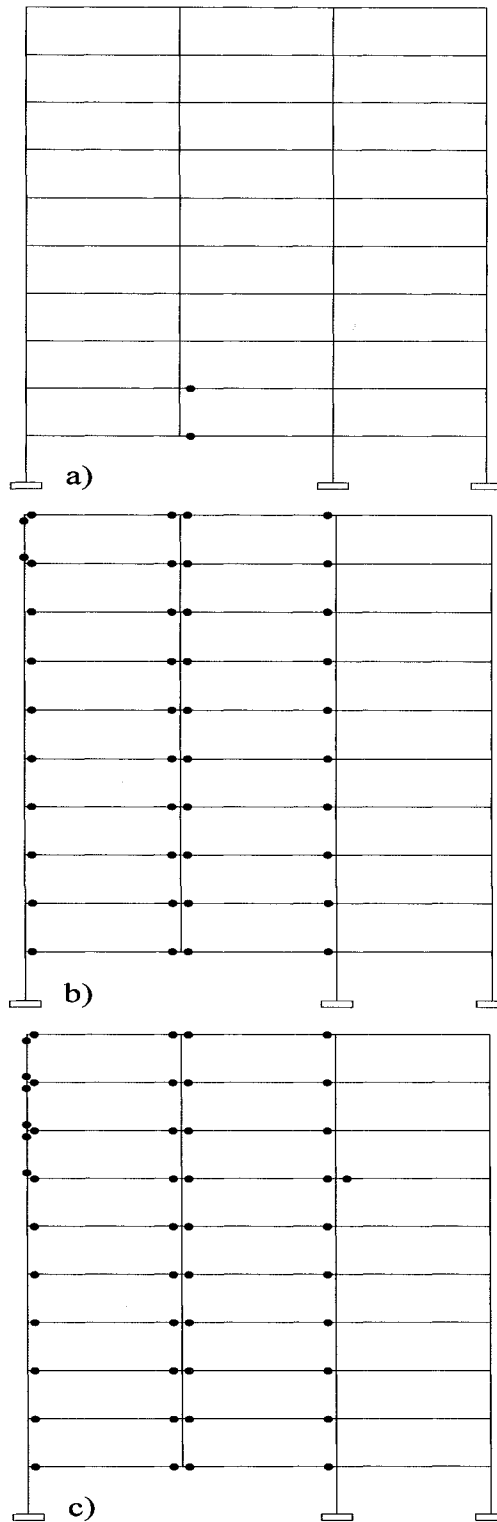


Figure 4.29: Formation of plastic hinges due to the loss of an exterior edge column in the short direction of 10-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

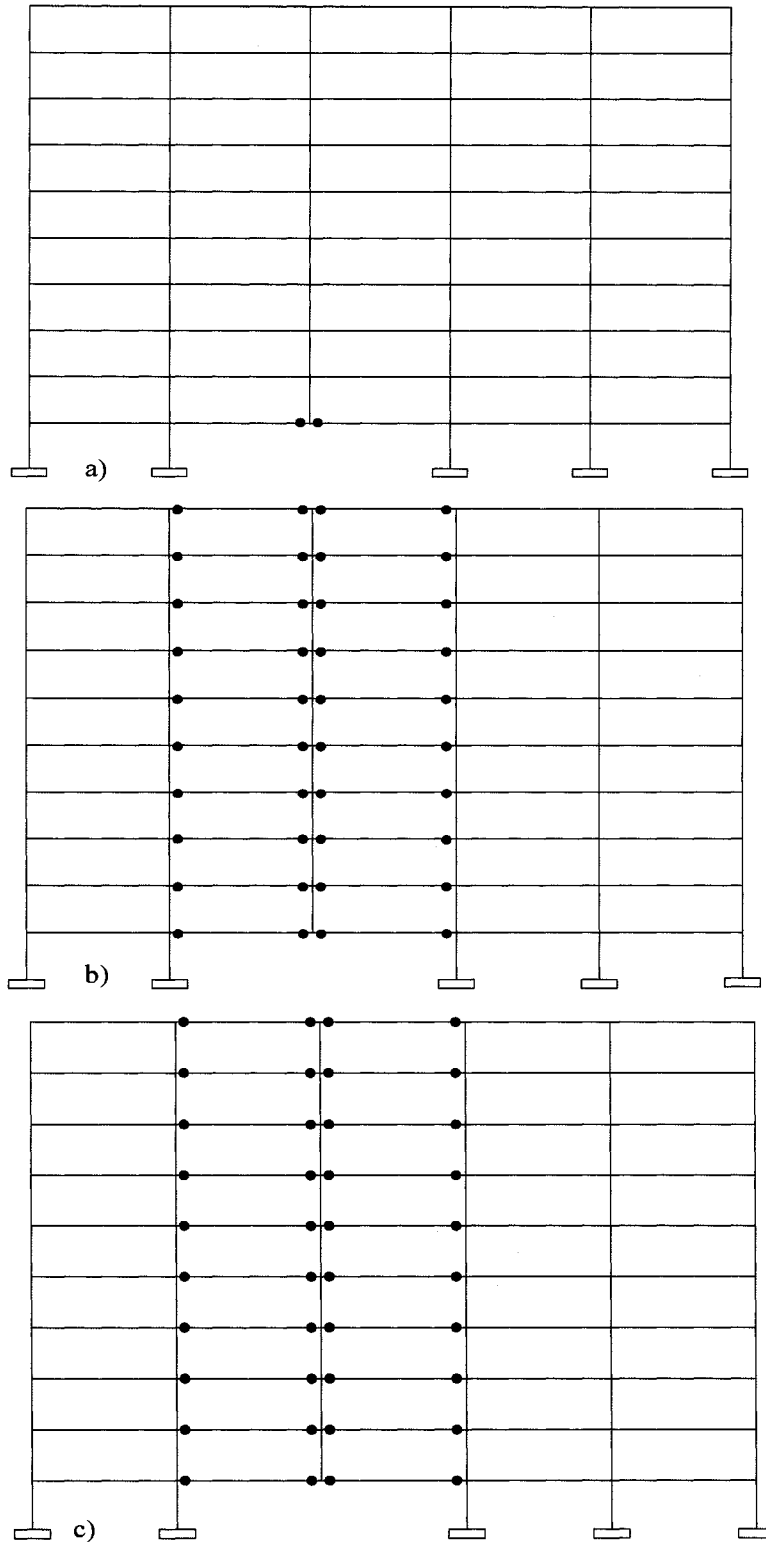


Figure 4.30: Formation of plastic hinges due to the loss of an exterior edge column in the long direction of 10-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

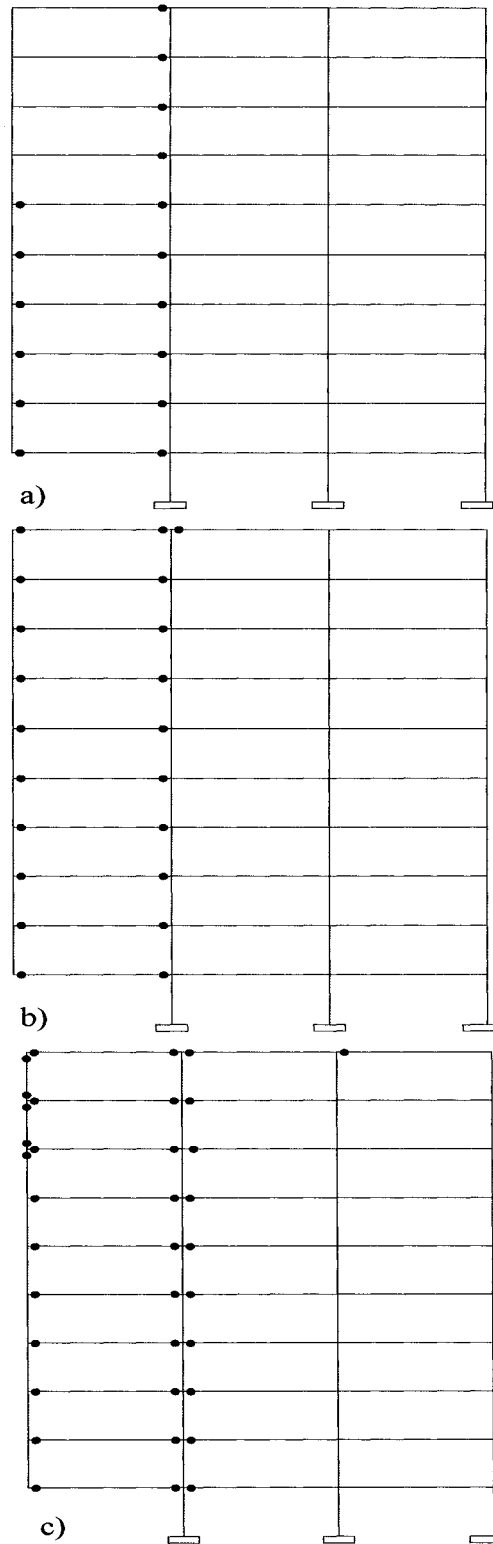


Figure 4.31: Formation of plastic hinges due to the loss of a corner column in the short direction of 10-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

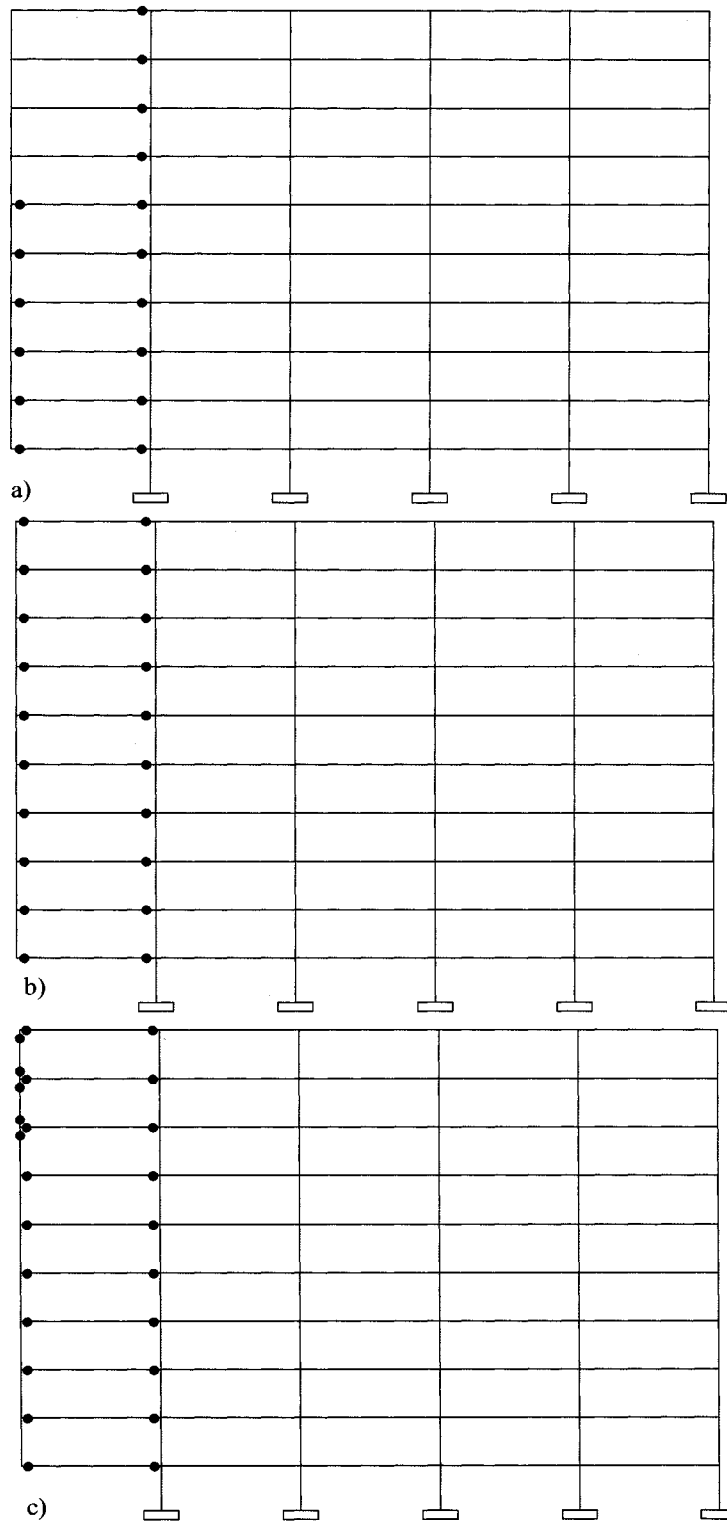


Figure 4.32: Formation of plastic hinges due to the loss of a corner column in the long direction of 10-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

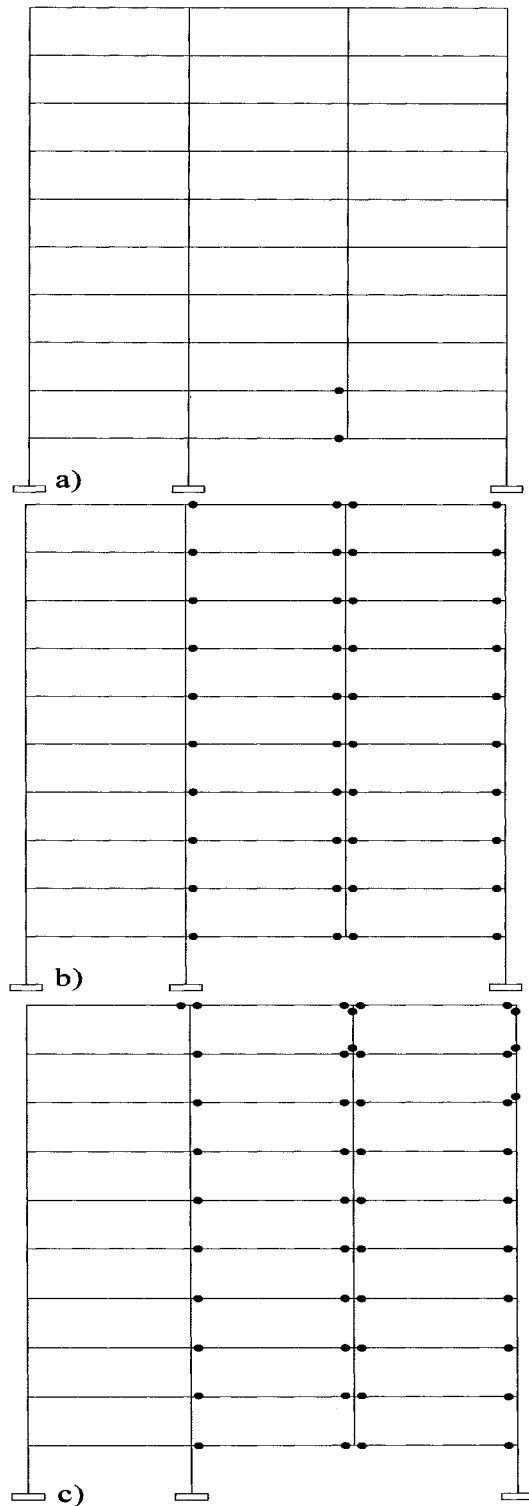


Figure 4.33: Formation of plastic hinges due to the loss of an interior column in the short direction of 10-storey building with 6.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

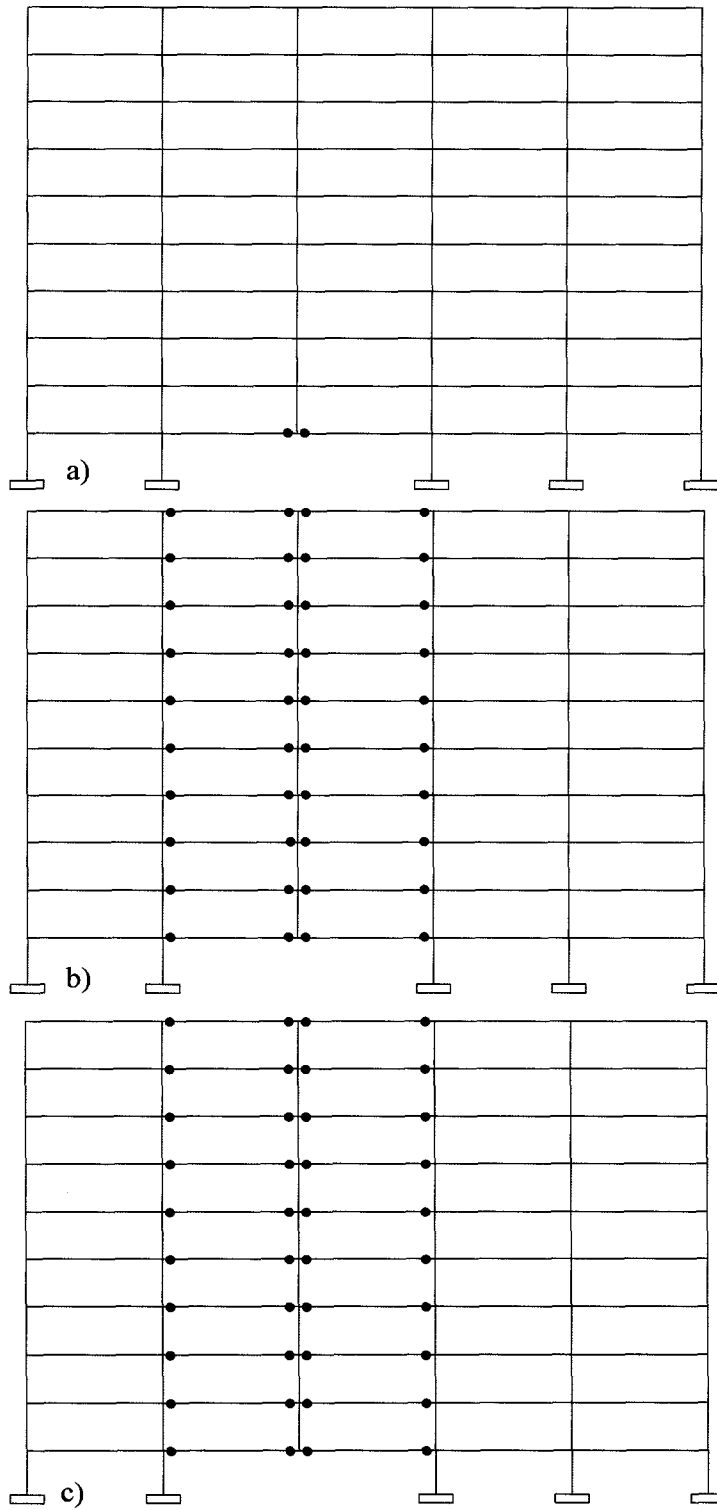


Figure 4.34: Formation of plastic hinges due to the loss of an interior column in the long direction of 10-storey building with 6.0m spans at:
 a) First yield b) 2% Deflection c) 4% Deflection

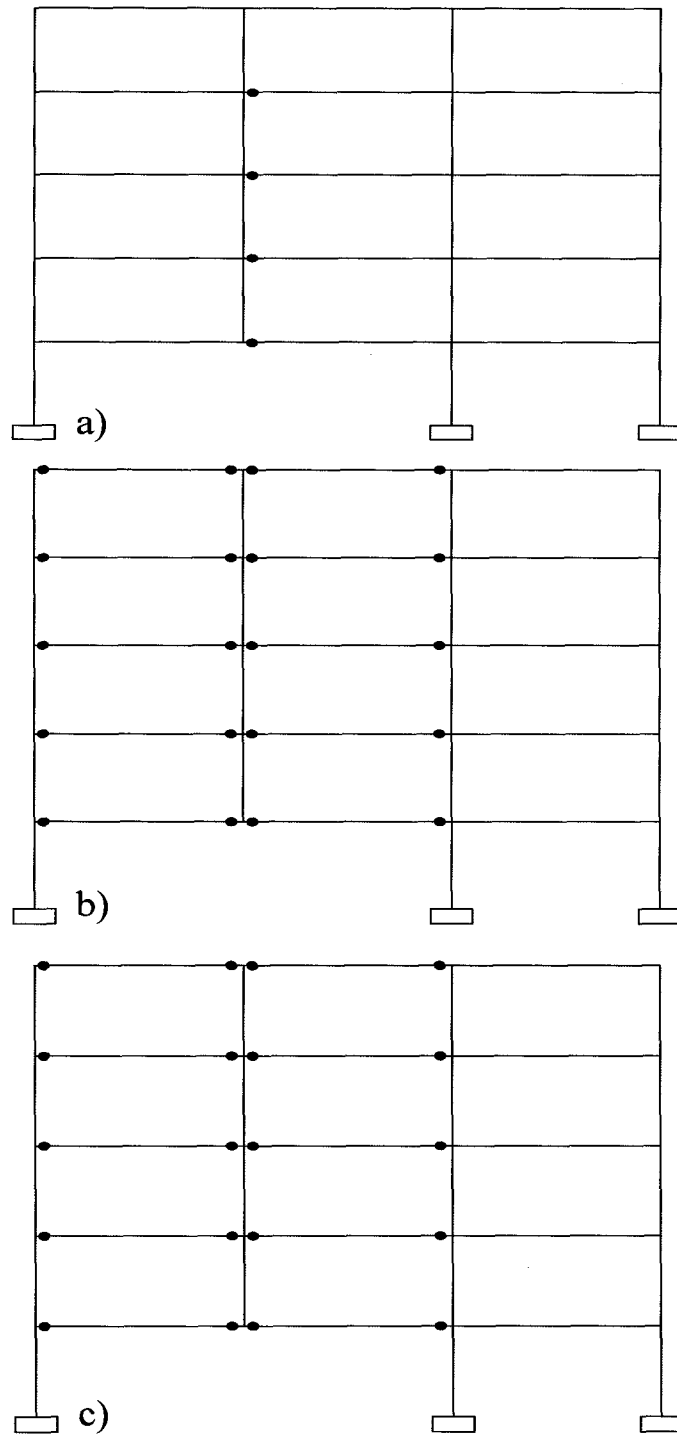


Figure 4.35: Formation of plastic hinges due to the loss of an exterior edge column in the short direction of 5-storey building with 9.0m spans at:

a) First yield b) 2% Deflection c) 4% Deflection

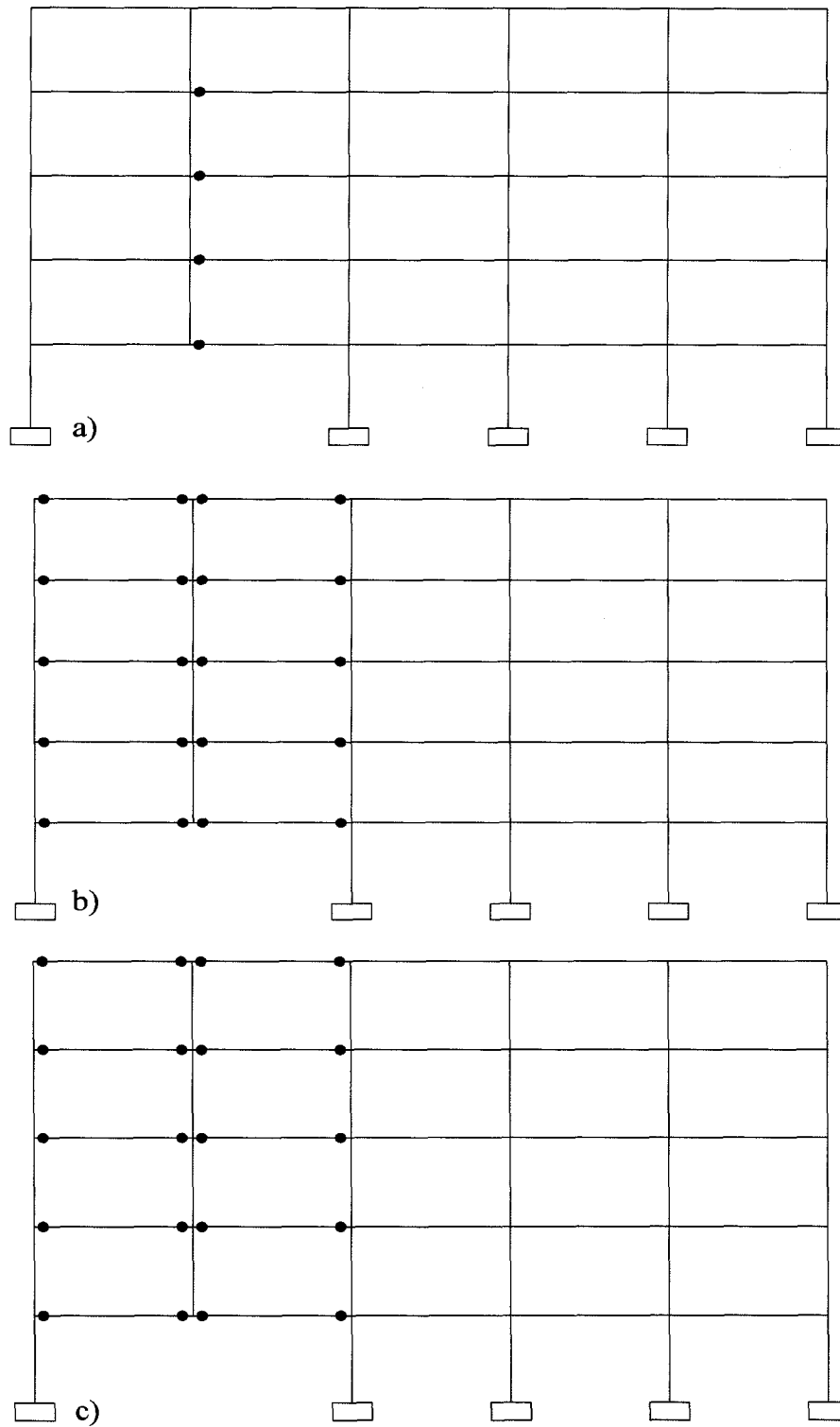


Figure 4.36: Formation of plastic hinges due to the loss of an exterior edge column in the long direction of 5-storey building with 9.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

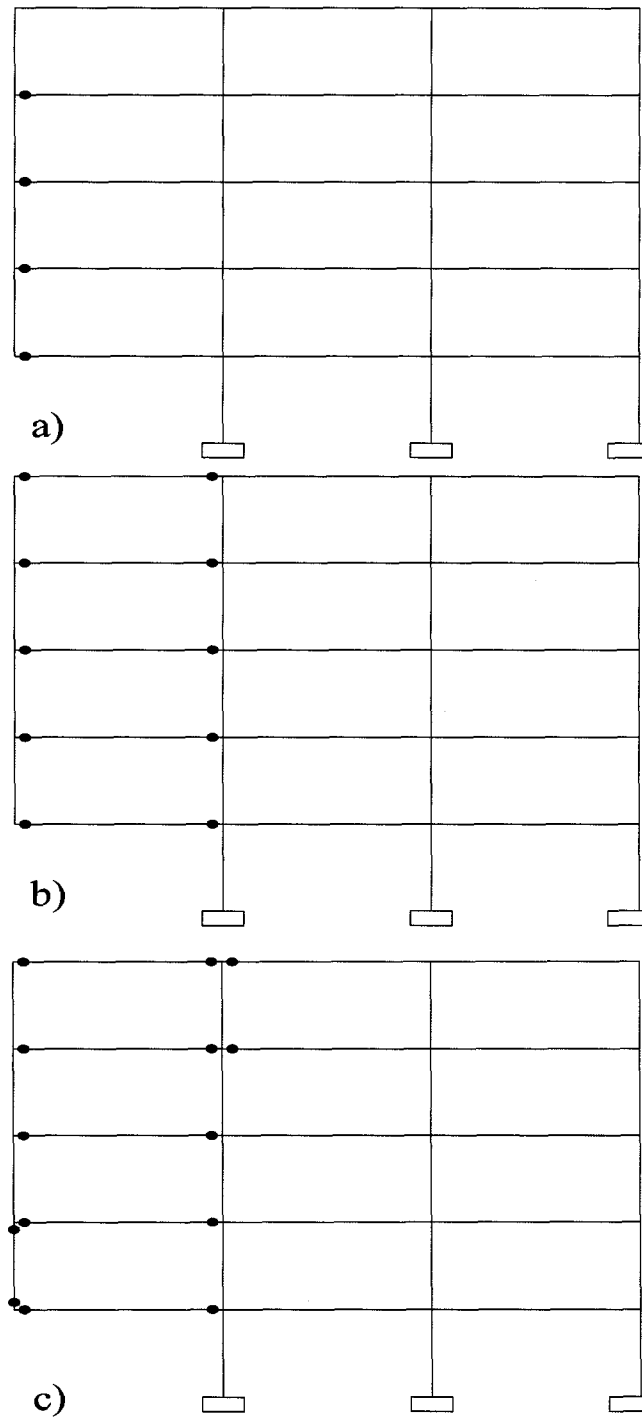


Figure 4.37: Formation of plastic hinges due to the loss of a corner column in the short direction of 5-storey building with 9.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

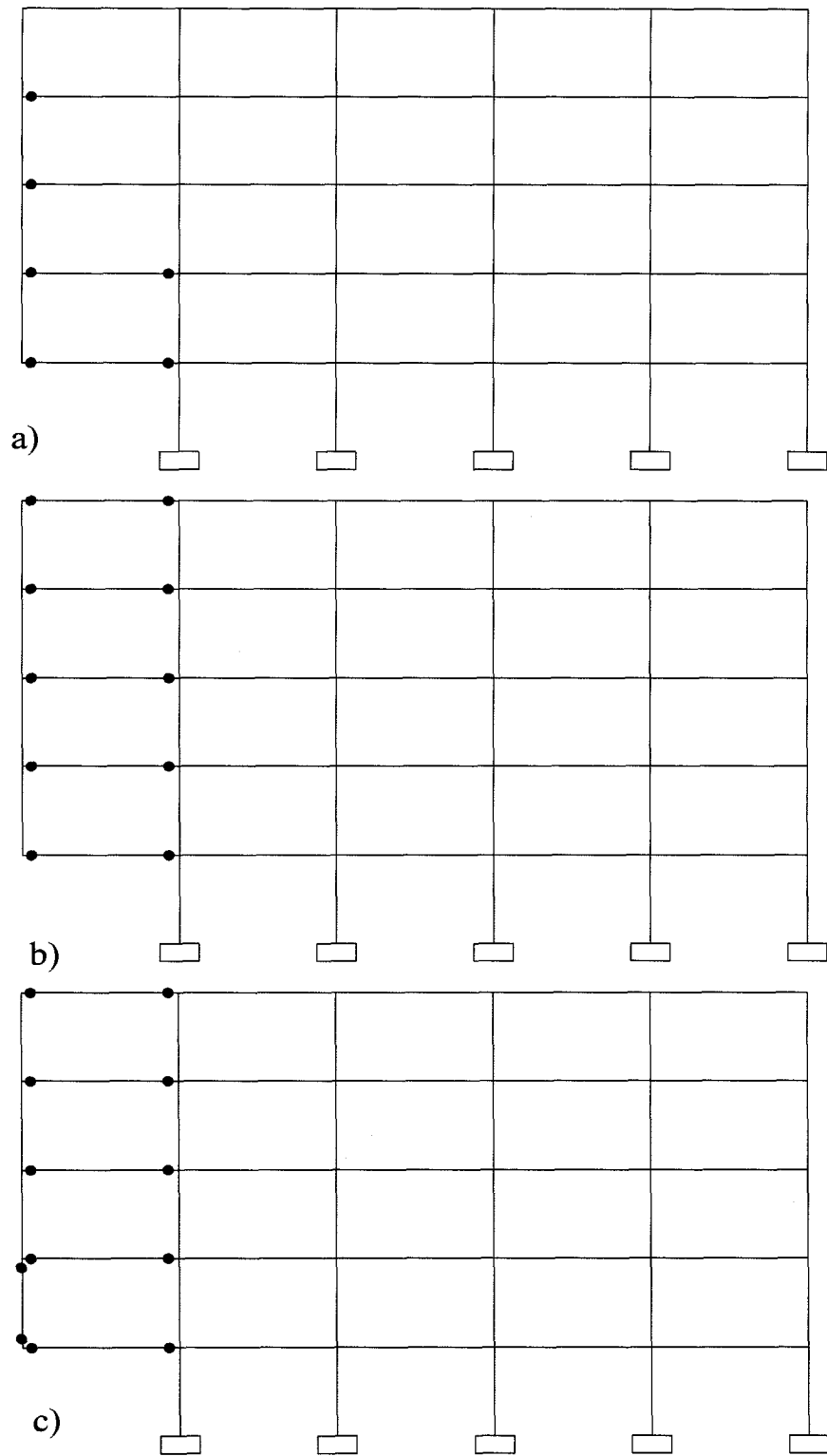


Figure 4.38: Formation of plastic hinges due to the loss of a corner column in the long direction of 5-storey building with 9.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

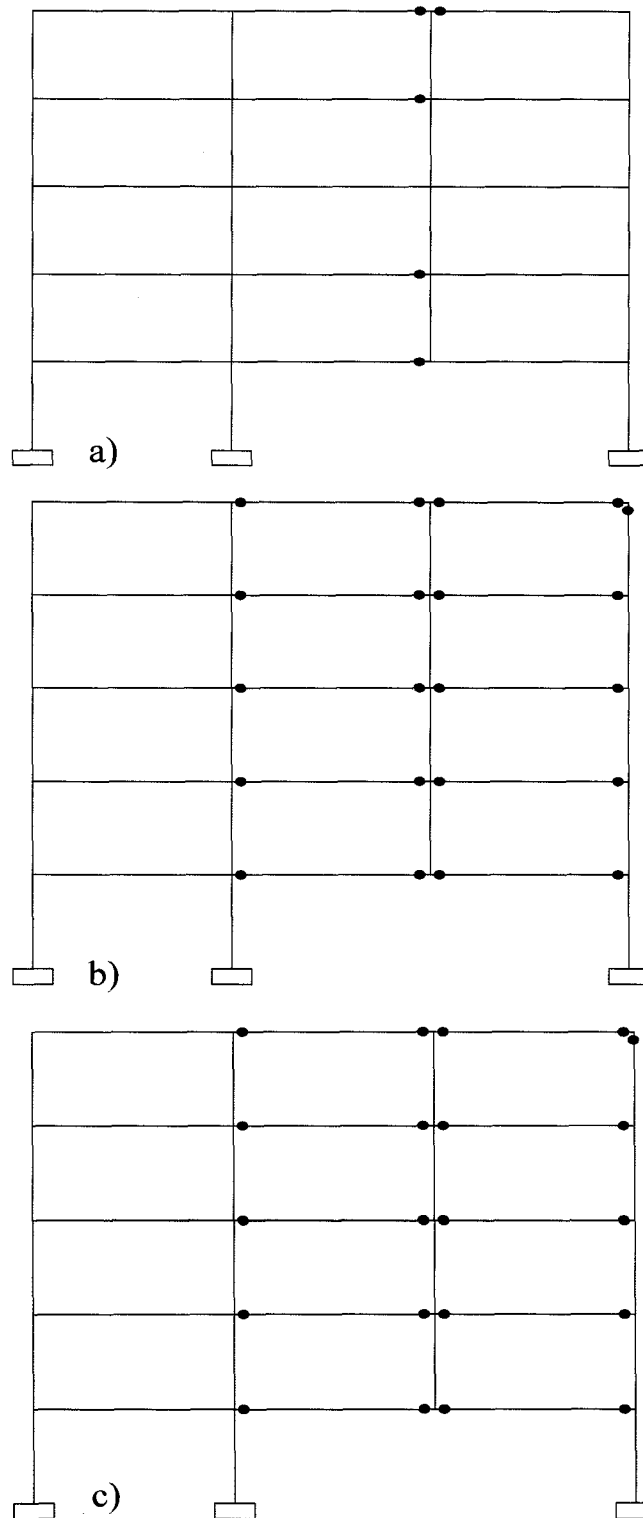


Figure 4.39: Formation of plastic hinges due to the loss of an interior column in the short direction of 5-storey building with 9.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

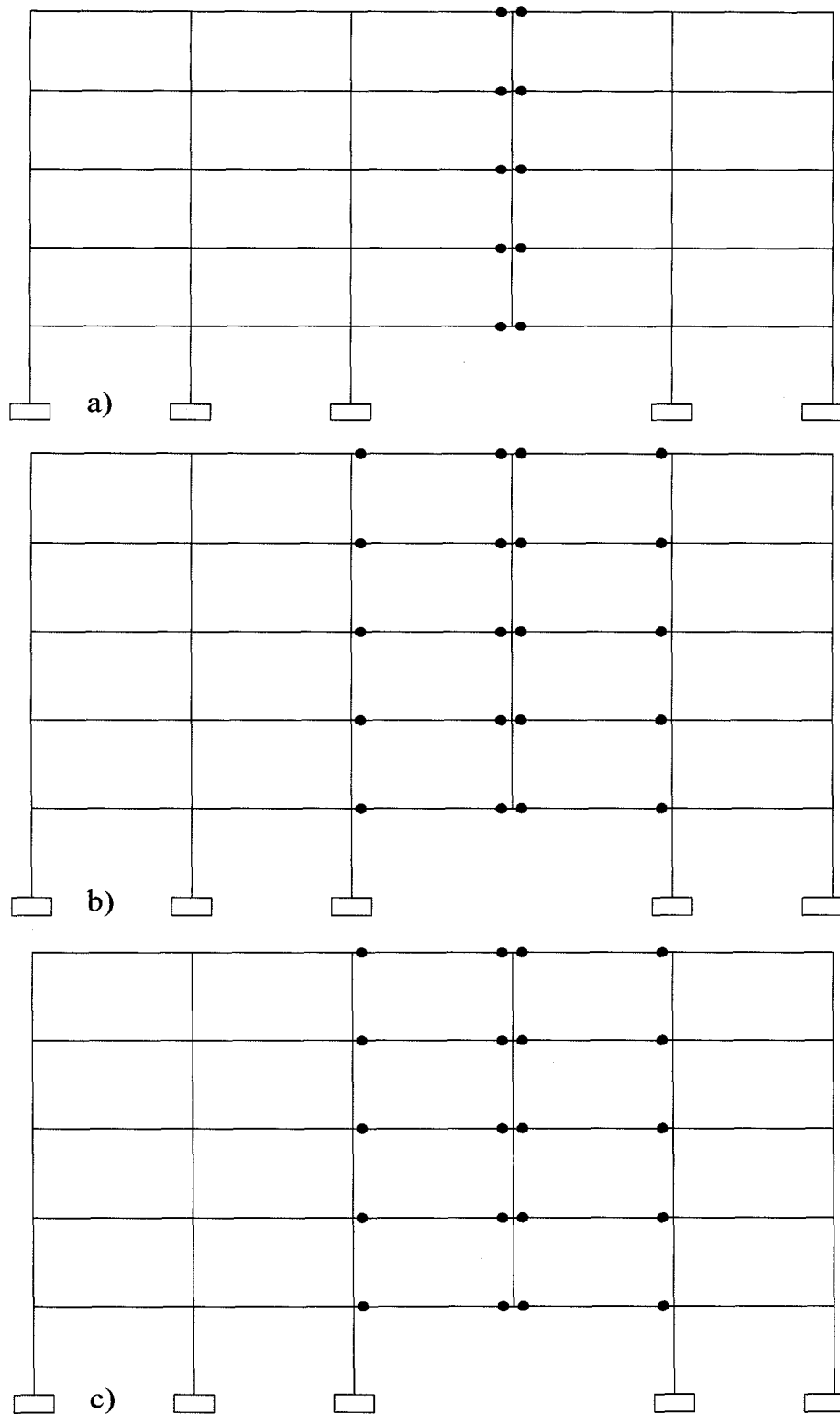


Figure 4.40: Formation of plastic hinges due to the loss of an interior column in the long direction of 5-storey building with 9.0m spans at:
a) First yield b) 2% Deflection c) 4% Deflection

CHAPTER 5:

Summary and Conclusion

5.1 Summary

Many industrial, commercial, federal and residential buildings have been attacked during the last two decades by bomb explosions. These attacks resulted in significant fatalities and partial or total structural collapses of buildings. The lethal hazards caused by bomb detonations turned the attention of the structural engineering community to progressive collapse prevention. Because of lack of sufficient research in the field, particularly experimental research, the majority of previous studies concentrated on the investigation of buildings that suffered from bomb attacks (e.g., Murrah Building). Some analytical research was also conducted. The knowledge gathered was disseminated to produce analysis and design guidelines. The General Services Administration (GSA) guideline is one of these documents. The GSA procedure is employed as part of the progressive collapse analysis conducted in the current investigation.

Three multi-storey reinforced concrete buildings were selected designed and analyzed for assessment of their progressive collapse potential resulting from column losses at selected locations. The buildings were designed based on the National Building Code of

Canada (NBCC-2005) and CSA Standard A23.3-04. The building frames were modeled and analyzed in two orthogonal directions using computer software DRAIN-R/C. Both elastic and inelastic analyses were conducted to assess demand capacity ratios and inelastic deformation demands for critical elements.

5.2 Conclusion

The following conclusions can be obtained from the analytical research conducted in this study pertaining to the vulnerability of reinforced concrete frame buildings to blast-induced progressive collapse.

From elastic analysis:

- The loss of a lower storey column due to a nearby explosion is likely to trigger progressive collapse in multi-storey reinforced concrete structures. Buildings designed as gravity load resisting structures (without seismic design and detailing practices) can not withstand a column loss and fail immediately upon losing their supports. This abrupt failure is due to lack of continuity in such buildings and lack of positive moment capacity in support regions.
- Buildings designed following seismic design requirements have continuity and ductility that may prevent progressive collapse under certain conditions. Frame buildings with spans of less than 6.0m may survive corner and exterior column removal due to bomb explosion. However, the same type of building may not survive if an interior column is damaged and lost.
- Unlike buildings with short spans (6m and shorter), the chance of progressive collapse due to the loss of an exterior edge column is much higher in buildings with longer spans (9.0m and longer).
- Reinforced concrete frame structures may not survive the removal of first storey columns from the interior frames. Interior columns have high tributary areas and their removal triggers progressive collapse, even in short-span buildings.
- The GSA progressive collapse requirements can be applied relatively easily by conducting elastic analysis and computing Demand Capacity Ratios for critical beam elements.

Results from inelastic analysis:

- Reinforced concrete frame buildings develop high ductility demands under GSA specified load combinations upon removal of first storey columns. These vertical displacement demands can be as high as 20% of beam length, which is not possible to sustain even by seismically designed and tailed elements.
- 5-storey reinforced concrete building with 6.0m span can carry 60 to 70% of GSA specified loads when a corner column or an exterior edge column is removed. This percentage decreases to 40% when building experiences an interior column failure. The increase in span length results in lower load carrying capacity, as expected. When the span length is increased from 6.0m to 9.0m the load capacity decreases by additional 5% to 10%.
- Reinforced concrete frame buildings have limited load carrying capacity if an interior column is lost due to a bomb explosion. The frames resist about 20% of GSA load at initial yield, about 30% at 2% deflection and about 40% at 4% deflection level. This implies that earthquake resistant buildings can carry approximately 40% of GSA loads prior to failure, while ordinary building frames develop progressive collapse at about 30% of GSA load.
- The building height does not play a major role in load carrying capacity of beams following a column loss. However, as the level of redundancy increases better distribution of forces among members improve load carrying capacity within the elastic range and during limited inelasticity.
- If the performance of buildings is checked against (DL + 0.5LL) gravity load, earthquake resistant buildings are likely to survive bomb blasts under exterior and corner column loss scenarios. However, the same building may not sustain loads resulting from the removal of an interior column.

5.3 Recommendations for Future Research

The analytical research conducted in this investigation has a limited scope. Further analyses of reinforced concrete buildings are recommended as part of future research. Specifically, the following efforts are recommended for future research:

- Investigation of frame-shear wall interactive systems and their vulnerabilities to progressive collapse.
- Consideration of strain rate effects in specifying member capacities.
- Consideration of a wider range of parameters, in terms of span length and building height.
- Three dimensional analyses of buildings, bringing the contribution of two-way slab into progressive collapse analysis.

REFERENCES:

1. Carino, J. Nicholas and Lew, H.S. "Application of Seismic Rehabilitation Technologies to Mitigate Blast-Induced Progressive Collapse." Proceedings of a Workshop organized jointly by the National Institute of Standards and Technology (NIST) and the General Services Administration (GSA). U.S. Department of Commerce, Sept. 10, 2001, Oakland Ca.
2. Multihazard Mitigation Council of the National Institute of Building Sciences. "Prevention of Progressive Collapse: Report on the July 2002 National Workshop and Recommendations for Future Efforts," Washington, D.C. 2003.
3. Burns, J., Abruzzo, J., and Tamaro, M. "Structural Systems for Progressive Collapse Prevention." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
4. Cagley, J. R. "The Design Professional's Concerns Regarding Progressive Collapse Design." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
5. Dusenberry, D. O. "Review of Existing Guidelines and Provisions Related to Progressive Collapse." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
6. Corley, G. "Applicability of Seismic Design in Mitigating Progressive Collapse." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
7. Crawford, John. "Retrofit Methods to Mitigate Progressive Collapse." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard

- Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
8. Ellingwood, Bruce R. "Load and Resistance Factor Criteria for Progressive Collapse Design." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
 9. Krauthammer, T. "Development of Progressive Collapse Analysis Procedure and Condition Assessment for Structures." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
 10. Lew, H.S. "Analysis Procedures For Progressive Collapse of Buildings." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
 11. Moore, D. B. "The UK and European Regulations for Accidental Actions." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
 12. Scott, D. Lane, B. Gibbons, C. "Fire Induced Progressive Collapse." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
 13. Smilowitz, R. "Analytical Tools for Progressive Collapse Analysis." Proceedings of the National Workshop on Prevention of Progressive Collapse, Multihazard Mitigation Council of the National Institute of Building Sciences, Washington, D.C., 2003.
 14. DOD. "DOD Minimum Antiterrorism Standards for Buildings." UFC 4-010-01 31, Department of Defence of the United States of America. July 2002.
 15. GSA. "Facilities Standard for the Public Buildings Service." The U.S. General Services Administration, the Office of the Chief Architect, March 2005.

16. GSA. "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, June 2003.
17. ASCE. "Structural Design for Physical Security." American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, Virginia, 20191-4400, U.S.A.
18. FEMA 386-7. "Integrating Mad-Made Hazard into Mitigation Planning." Federal Emergency Management Agency (FEMA). The U.S. Department of Homeland Security, September 2003.
19. FEMA 426. "Reference Manual to Mitigate Potential Terrorist Attacks Against Buildings." Federal Emergency Management Agency (FEMA). The U.S. Department of Homeland Security, December 2003.
20. FEMA 428. "Primer to Design Safe School Projects in Case of Terrorist Attacks." Federal Emergency Management Agency (FEMA). The U.S. Department of Homeland Security, December 2003.
21. FEMA 427. "Primer *for* Design of Commercial Buildings to Mitigate Terrorist Attacks." Federal Emergency Management Agency (FEMA). The U.S. Department of Homeland Security, December 2003.
22. ACI 318-05. "Building Code Requirements for Structural Concrete." American Concrete Institute, 38800 Country Club Road, Farmington Hill, MI 48331, 2005, pp.430.
23. CSA A23.3-04. CSA, "Design of Concrete Structures for Buildings (CAN3-A23.2-04)." Canadian Standards Association, Rexdale, Ontario, 2005.
24. BS 8110. "Structural use of Concrete Part 1. Code of Practice for Design and Construction." The British Standards Institute, 1997.
25. Mays, G.C. and Smith, P.D. "Blast Effects on Buildings." Thomas Telford, London, 1995, pp.121.
26. Gross, J. and McGuire, W. "progressive Collapse Resistant Design." Journal of Structural Engineering, ASCE., Vol.109, No.1, Jan. 1983, pp.1-15.
27. Choi, H-J. and Krauthammer, T. "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, Japan, 15-19 December 2003.
28. Whittaker et al. "Performance-Based Engineering of Buildings and Infrastructure for Extreme Loadings."
 29. DoD. "Unified Facilities Criteria (UFC) 4-010-10, DoD Minimum Antiterrorism Standoff Distances for Buildings." Department of Defence of the United States of America.
 30. DoD. "DoD Ammunition and Explosive Safety Standards . 6055.9-STD." Department of Defence of the United States of America. July, 1999.
 31. U.S. Department of Army (DA) FM 3-19.30 "Physical Security" Document,
 32. ASCE 7-02. "Minimum Design Loads for Buildings and Other Structures." American Society of Civil Engineers, Reston, VA, 2002.
 33. UBC-1997. Uniform Building Code, Volume 2, International Conference of Building Officials, Whittier, California, U.S.A., 1997, p. 492.
 34. IBC-2000. International Building Code, International Conference of Building Officials, Whittier, California, U.S.A., 2000.
 35. "FEMA 274 – NEHRP Commentary on Guidelines for the Seismic Rehabilitation of Buildings." Federal Emergency Management Agency, 1997.
 36. "FEMA 356 – Prestandard and Commentary for the Seismic Rehabilitation of Buildings." Federal Emergency Management Agency, November 2000.
 37. BSI. "British Standard – Structural Use of Concrete Part 2. Code of Practice for Special Circumstances." British Standards Institution (BSI) 8110, 1985.
 38. Scheffner, L. "Structural Designs to Reduce the Effects of Terrorist car bombs". Wisconsin Engineer (Newsletter), University of Wisconsin-Madison, Wisconsin, 3pp, 1998.
 39. Previous research was done by E. Dincer (2003) and O. Yagob (2006).

APPENDIX

Input Data for DRAIN-R/C

Table A.1: Distributed load pattern for 5-storey and 10-storey reinforced concrete buildings with 6m span length.

Distributed Load Patterns	Exterior Frame	Interior Frame
Pattern #1 (Floors)	-33.6 KN/m	-67.2 KN/m
Pattern #2 (Roofs)	-24.3 KN/m	-48.6 KN/m

Table A.2: Distributed load pattern for 5-storey reinforced concrete buildings with 9m span length.

Distributed Load Patterns	Exterior Frame	Interior Frame
Pattern #1 (Floors)	-50.4 KN/m	-100.8 KN/m
Pattern #2 (Roofs)	-36.45 KN/m	-72.9 KN/m

Table A.3: Stiffness properties of 5-storey reinforced concrete building:

a) Exterior frames

Exterior Frame	EI	EA	GA _e
Pattern #1 (beams, floor 1-4)	39305.1	4405744.0	1560367.0
Pattern #2 (beams, floor 5)	27605.2	4036030.0	1429428.0
Pattern #3 (exterior-exterior columns)	11646.0	2218276.0	785639.6
Pattern #4 (exterior-interior columns)	36807.0	3943602.0	1396692.0

b) Interior frames

Interior Frame	EI	EA	GA _e
Pattern #1 (beams, floor 1-4)	39305.1	4405744.0	1560367.0
Pattern #2 (beams, floor 5)	27605.2	4036030.0	1429428.0
Pattern #3 (exterior-interior columns)	36807.0	3943602.0	1396692.0
Pattern #4 (interior-interior columns)	58957.6	4991122.0	1767689.0

Table A.4: Stiffness properties of 10-storey reinforced concrete building:

a) Exterior frames

Exterior Frame	EI	EA	GA _e
Pattern #1 (beams, floor 1-5)	53916.4	4775456.0	1691307.0
Pattern #2 (beams, floor 6-9)	39305.1	4405744.0	1560368.0
Pattern #3 (beams, floor 10)	27605.2	4036031.0	1429428.0
Pattern #4 (exterior-exterior columns)	21575.6	3019321.0	1069343.0
Pattern #5 (exterior-interior columns)	36807.0	3943602.0	1396693.0

b) Interior Frames

Interior Frame	EI	EA	GA _e
Pattern #1 (beams, floor 1-9)	39305.1	4405744.0	1560368.0
Pattern #2 (beams, floor 10)	27605.2	4036031.0	1429428.0
Pattern #3 (exterior-interior columns)	36807.0	3943602.0	1396693.0
Pattern #4 (interior-interior columns)	89860.7	6161879.0	2182332.0

Table A.5: Stiffness properties of 5-storey reinforced concrete building with 9m span:

a) Exterior frames

Exterior Frame	EI	EA	GA _e
Pattern #1 (beams, floor 1-4)	155279.3	11091382.0	3928198.0
Pattern #2 (beams, floor 5)	71888.6	8626630.0	3055265.0
Pattern #4 (exterior-exterior columns)	89860.7	6161879.0	2182332.0
Pattern #5 (exterior-interior columns)	186335.2	8873105.0	3142558.0

b) Interior Frames

Interior Frame	EI	EA	GA _e
Pattern #1 (beams, floor 1-5)	155279.3	16267360.0	5761356.0
Pattern #2 (exterior-interior columns) (interior-interior column 5 th floor)	186335.2	8873105.0	3142558.0
Pattern #3 (interior-interior columns 1 st floor)	345209.0	12077282.0	4277371.0
Pattern #4 (interior-interior columns 2 nd -4 th floor)	256651.2	10413575.0	3688141.0

Table A.6: Hinge properties and rigid segments for 5-storey R/C building

a) Exterior Frame - Inelastic Properties

Exterior Frame	M_{cr}	M_y	M_n
Pattern #1 (beams, floor 1-4)	+39.8	+89.3	+109.8
	-56.9	-191.6	-210.4
Pattern #2 (beams, floor 5)	+31.4	+51.1	+68.8
	-45.6	-108.1	-114.5
Pattern #3 (ext-ext 1st column)	14.8	121.8	132.9
Pattern #4 (ext-ext 2nd column)	14.8	113	128.4
Pattern #5 (ext-ext 3rd column)	14.8	105	123.6
Pattern #6 (ext-ext 4th column)	14.8	99.2	115.6
Pattern #7 (ext-ext 5th column)	14.8	87.5	107.1
Pattern #8 (ext-int 1st column)	35.1	241.4	270
Pattern #9 (ext-int 2nd column)	35.1	216.7	250.9
Pattern #10 (ext-int 3rd column)	35.1	190.5	227.3
Pattern #11 (ext-int 4th column)	35.1	164.3	201.6
Pattern #12 (ext-int 5th column)	35.1	140	174.3

b) Exterior Frame - End Eccentricity Pattern



Exterior frame	at end I	at end J
Pattern #1	0.15	0.2
Pattern #2	0.2	0.2
Pattern #3	0.2	0.15
Pattern #4	0	0.225
Pattern #5	0.225	0.225
Pattern #6	0.225	0.2

c) Interior Frame – inelastic Properties

Interior Frame	M_{cr}	M_y	M_n
Pattern #1 (beams, floor 1-4)	+43.2	+85.4	+118.6
	-77.4	-199.4	-210.4
Pattern #2 (beams, floor 5)	+34	+51.6	+71.8
	-61.8	-108.1	-114.5
Pattern #3 (ext-int 1st column)	35.1	241.4	270
Pattern #4 (ext-int 2nd column)	35.1	216.7	250.9
Pattern #5 (ext-int 3rd column)	35.1	190.5	227.3
Pattern #6 (ext-int 4th column)	35.1	164.3	201.6
Pattern #7 (ext-int 5th column)	35.1	140	174.3
Pattern #8 (int-int 1st column)	49.9	460.6	478.7
Pattern #9 (int-int 2nd column)	49.9	422.1	459.1
Pattern #10 (int-int 3rd column)	49.9	378.5	434.9
Pattern #11 (int-int 4th column)	49.9	320.4	386.2
Pattern #12 (int-int 5th column)	49.9	261.3	328.3

d) Interior Frame – End Eccentricity Pattern

Interior frame	at end I	at end J
Pattern #1	0.2	0.225
Pattern #2	0.225	0.225
Pattern #3	0.225	0.2
Pattern #4	0	0.225
Pattern #5	0.225	0.225
Pattern #6	0.225	0.2

Table A.7: Hinge properties and rigid segments for 10-storey R/C building

a) Exterior Frame - Inelastic Properties

Exterior Frame	M_{cr}	M_y	M_n
Pattern #1 (beams, floor 1-4)	+39.8	+89.3	+109.8
	-56.9	-191.6	-210.4
Pattern #2 (beams, floor 5)	+31.4	+51.1	+68.8
	-45.6	-108.1	-114.5
Pattern #3 (ext-ext 1st column)	14.8	121.8	132.9
Pattern #4 (ext-ext 2nd column)	14.8	113	128.4
Pattern #5 (ext-ext 3rd column)	14.8	105	123.6
Pattern #6 (ext-ext 4th column)	14.8	99.2	115.6
Pattern #7 (ext-ext 5th column)	14.8	87.5	107.1
Pattern #8 (ext-int 1st column)	35.1	241.4	270
Pattern #9 (ext-int 2nd column)	35.1	216.7	250.9
Pattern #10 (ext-int 3rd column)	35.1	190.5	227.3
Pattern #11 (ext-int 4th column)	35.1	164.3	201.6
Pattern #12 (ext-int 5th column)	35.1	140	174.3

b) Exterior Frame - End Eccentricity Pattern



Exterior frame	at end I	at end J
Pattern #1	0.15	0.2
Pattern #2	0.2	0.2
Pattern #3	0.2	0.15
Pattern #4	0	0.225
Pattern #5	0.225	0.225
Pattern #6	0.225	0.2

c) Interior Frame – inelastic Properties

Interior Frame	M_{cr}	M_y	M_n
Pattern #1 (beams, floor 1-4)	+43.2	+85.4	+118.6
	-77.4	-199.4	-210.4
Pattern #2 (beams, floor 5)	+34	+51.6	+71.8
	-61.8	-108.1	-114.5
Pattern #3 (ext-int 1st column)	35.1	241.4	270
Pattern #4 (ext-int 2nd column)	35.1	216.7	250.9
Pattern #5 (ext-int 3rd column)	35.1	190.5	227.3
Pattern #6 (ext-int 4th column)	35.1	164.3	201.6
Pattern #7 (ext-int 5th column)	35.1	140	174.3
Pattern #8 (int-int 1st column)	49.9	460.6	478.7
Pattern #9 (int-int 2nd column)	49.9	422.1	459.1
Pattern #10 (int-int 3rd column)	49.9	378.5	434.9
Pattern #11 (int-int 4th column)	49.9	320.4	386.2
Pattern #12 (int-int 5th column)	49.9	261.3	328.3

d) Interior Frame – End Eccentricity Pattern

Interior frame	at end I	at end J
Pattern #1	0.2	0.225
Pattern #2	0.225	0.225
Pattern #3	0.225	0.2
Pattern #4	0	0.225
Pattern #5	0.225	0.225
Pattern #6	0.225	0.2

Table A.8: Hinge properties and rigid segments for 5-storey R/C building-9m span
length

a) Exterior Frame - Inelastic Properties

Exterior Frame	M_{cr}	M_y	M_n
Pattern #1 (beams, floor 1-4)	+121 -190.1	+233.3 -562	+288.2 -594.9
Pattern #2 (beams, floor 5)	+69 -116.7	+117.6 -339.9	+200.9 -410.1
Pattern #3 (ext-ext 1st column)	68.5	629.5	742
Pattern #4 (ext-ext 2nd column)	68.5	598	719.1
Pattern #5 (ext-ext 3rd column)	68.5	568.9	688.9
Pattern #6 (ext-ext 4th column)	68.5	535.3	657.9
Pattern #7 (ext-ext 5th column)	68.5	500.5	625.1
Pattern #8 (ext-int 1st column)	118.3	988.5	1122.8
Pattern #9 (ext-int 2nd column)	118.3	913.4	1072.4
Pattern #10 (ext-int 3rd column)	118.3	835.8	1005.5
Pattern #11 (ext-int 4th column)	118.3	754.1	929.9
Pattern #12 (ext-int 5th column)	118.3	670.3	847.3

b) Exterior Frame - End Eccentricity Pattern



Exterior frame	at end I	at end J
Pattern #1	0.25	0.3
Pattern #2	0.3	0.3
Pattern #3	0.3	0.25
Pattern #4	0	0.3
Pattern #5	0.3	0.3
Pattern #6	0.3	0.25

c) Interior Frame – inelastic Properties

Interior Frame	M_{cr}	M_y	M_n
Pattern #1 (beams, floor 1-4)	+134.5 -288.3	+265.9 -729.6	+427.3 -863.1
Pattern #2 (beams, floor 5)	+134.5 -288.3	+203.8 -593.8	+343.4 -702.5
Pattern #3 (ext-int 1st column)	118.3	988.5	1122.8
Pattern #4 (ext-int 2nd column)	118.3	913.4	1072.4
Pattern #5 (ext-int 3rd column)	118.3	835.8	1005.5
Pattern #6 (ext-int 4th column)	118.3	754.1	929.9
Pattern #7 (ext-int 5th column)	118.3	670.3	847.3
Pattern #8 (int-int 1st column)	187.9	1897.2	2038.7
Pattern #9 (int-int 2nd column)	150.4	1309.8	1417.7
Pattern #10 (int-int 3rd column)	150.4	1158.2	1324.9
Pattern #11 (int-int 4th column)	150.4	994.6	1187.4
Pattern #12 (int-int 5th column)	118.3	731.1	906.7

d) Interior Frame – End Eccentricity Pattern

Interior frame	at end I	at end J
Pattern #1	0.3	0.35
Pattern #2	0.35	0.35
Pattern #3	0.35	0.3
Pattern #4	0.3	0.325
Pattern #5	0.325	0.325
Pattern #6	0.325	0.3
Pattern #7	0	0.3
Pattern #8	0.3	0.3