Progressive Collapse Analysis of Two Existing Steel Buildings Using Linear Static Procedure

Reza Jalali Larijani

Submitted to the
Institute of Graduate Studies and Research
in partial fulfillment of the requirements for the Degree of

Master of Science
in
Civil Engineering

Eastern Mediterranean University
January 2012
Gazimağusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

Prof. Dr. Elvan Yılmaz
Director

I certify that this thesis satisfies the requirements as a thesis for the degree of Master of Science in Civil Engineering.

Asst. Prof. Dr. Murude Çelikağ
Chair, Department of Civil Engineering

We certify that we have read this thesis and that in our opinion it is fully adequate in scope and quality as a thesis for the degree of Master of Science in Civil Engineering.

Asst. Prof. Dr. Murude Çelikağ
Supervisor

Examining Committee

1. Asst. Prof. Dr. Giray Ozay
2. Asst. Prof. Dr. Murude Çelikağ
3. Asst. Prof. Dr. Serhan Şensoy

ABSTRACT

There are numerous threats which could cause progressive collapse in a structure that may lead to fatality. After the incident in Oklahoma Murrah building and the recent terrorist attacks, such as WTC (World Trade Center) in 2001, it became more important to do assessment towards preventing the progressive collapse.

Although, there have been many researches carried out on progressive collapse, the increase in terrorist attacks, especially loss of lives (nearly 3000 died in the attacks of September 2001) in the World Trade Center case, lead to the development of new guidelines, such as General Services Administration (GSA), Department of Defense (DoD), and Unified Facilities Criteria (UFC) for assessing and preventing progressive collapse. In addition, a limited number of investigations were done on steel structure, especially on dual frame systems (moment frame with bracing system) so far, numerous investigations were carried out on reinforced concrete structure until now. The researches on the progressive collapse resistance of steel framed buildings are gradually increasing with the improvements on steel materials, technology and methods particularly in the developed countries.

Progressive collapse occurs when a primary structural component (s) of a building fail (s) to tolerate an accidental overloading. This failure leads to spreading of the forces to other neighboring weight bearing components (typically columns), if this distribution of loads go beyond the component (s) capacity then they may collapse. Hence, the intensity and coverage of the overall damage is disproportionate to the initial cause. In order to decrease this destructive incidents in buildings, NIST (National Institute of Standards and Technology) has published a list of potential load hazards generating progressive collapse as follows: accidental events, such as; airplane crashes, car crashes, errors in design or construction process, fire accidents, violent harsh change in air pressure (explosion), accidental over load, explosion caused by bombs, vehicular collision, and hazardous materials.

In this study the susceptibility of two different asymmetric existing steel building frames (nine-story building and six-story building before and after rehabilitation), with different frame systems, steel sections and number of stories, to progressive collapse has been assessed. For this, alternate path method with the linear static analysis is carried out according to GSA 2003 guidelines using software ETABS-3D. Demand Capacity Ratio (DCR) of each primary element (beams and columns) is given with its specific details in all frames. Comparison between nine-story and six-story building shows that the nine-story building with dual frame system (moment frame with bracing system) has lower susceptibility and more resistance to progressive collapse with respect to the six-story building with simple building frame system (gravity system with bracing system) when, in particular, the frame utilizes continual beams in connections (beam-column connection) or moment frame system in structural frame system. Also, implementing the built-up box sections for columns is a better choice than using built-up I-sections for columns since there is no weak axis for the box section.

Keywords: Progressive Collapse (PC), Demand Capacity Ratio (DCR), Alternate Load Path Method (APM), GSA guidelines, Yield Stress, Deflection.

ÖZ

Yapılarda ölümle sonuçlanan aşamalı çökmeye neden olabilecek tehlikeler vardır. Oklahoma Murrah Binasında meydana gelen olay ve son günlerde, örneğin 2001'de dünya ticaret merkezinde, meydana gelen terror saldırıları sonrası aşamayı çökmeyi önleyici değerlendirmelerin yapılması daha da önem kazanmıştır.

Bu güne kadar aşamalı göçme üzerine çok sayıda araştırma yapılmış olmasına rağmen, terror saldırılarındaki artış, özellikle Dünya Ticaret Merkezindeki terror saldırısı sonucu can kayıpları (Eylül 2001'deki saldırılarda yaklaşık olarak 3000 kişi ölmüştür) aşamalı göşmeyi önlemeyi değerlendirmek için Genel Hızmet İdaresi (GSA), Savunma Bakanlığı (DoD) ve Birleştirilmiş Tesisat Kriterleri (UFC) gibi yeni klavuzların geliştirilmesine neden olmuştur.

İlaveten, betonarme binalar üzerinde çok sayıda inceleme ve araştırma yapılmış olmasına karşın çelik yapılarda, özellikle de ikili çerçeve sisteminde sadece kısıtlı sayıda inceleme yapılmıştır. Özellikle gelişmekte olan ülkelerde çelik karkas binaların aşamalı göçmeye dayanımı konulu araştırmalar her geçen gün çelik malzemesi, teknoloji ve methodlarının gelişimiyle yavaş yavaş artmaktadır.

Aşamalı göçme, en önemli bir yapı elemanının kaza sonucu aşırı yüklemeyi tolere edemeyip başarısız olması sonucu oluşur. Bu başarısızlık oluşan yüklerin komşu taşıyıcı elemanlar (tipik olarak kolonlar) tarafından taşınabilmesini gerektirir, fakat bu yük

dağılımının taşıyıcı yapı elemanlarının kapasitesini aşması durumunda bu elemanlar çökebilir. Bundan dolayı, genel hasarın yoğunluğu ve etki alanı bunu başlatan nedene gore orantısızdır.

NIST (Ulusal Standard ve Teknoloji Enstitüsü) yapılarda bu tür yıkıcı olayları azaltma adına bir çalışma başlattı. NIST binalarda yıkıcı zararı azaltma adına zarar oluşturabilecek aşamalı göçmeye neden olabilecek bir dizi aktivite listelemiştir; örneğin, kazalar, araba kazaları, yangın, patlama sonucu oluşacak şiddetli hava basıncı değişimi, tasarım ve inşaat esnasında oluşabilecek hatalar, vs.

Bu çalışmada iki farklı çerçeve sistemi, çelik kesitleri, kat sayısı olan iki asimetrik mevcut bina çerçevesinin (altı ve dokuz kat rehabılıtasyon öncesi ve sonrası) aşamalı göçmeye karşı hassasiyeti incelenmiştir. Bu araştırmada GSA 2003 kılavuzu ve ETABS-3D alternatif yol metodu doğrusal static analiz kullanılarak yapılmıştır. Her ana eleman için (kiriş ve kolonlar) DCR yanında tüm çerçeveler için specific.detaylar verilmiştir. Altı ve dokuz katlı binalar karşılaştırıldığında dokuz katlı ve çift çerçeve sistemi olan binanın aşamalı göçmeye karşı daha dirençli olduğu gözlemlenmiştir. Bu çalışmada kolon elemanları için kaynaklı kutu kesitlerin kullanılması kaynaklı I-kesitlerinin kolon olarak kullanılmasında zayıf aksı olmadığı için avantaj sağlayacaktır.

Anahtar Kelimeler: Aşamalı göçme (PC), İstek kapasite oranı, Ratio (DCR), Alternatif Yük Yolu Metodu (APM), GSA kılavuzu, Akma basıncı, deformasyon.

This thesis is dedicated to my family who offered me constant support and unconditional love throughout the course of this dissertation.

ACKNOWLEDGMENT

I would like to express my gratitude to my supervisor, Assit. Prof. Dr. Murude Çelikağ, Chair of the Department of Civil Engineering, Eastern Mediterranean University, whose expertise, proficiency, sympathy and patience added to my knowledge and graduate experience. I appreciate all her efforts and assistance in many areas including methodology and editing of this thesis. Without her invaluable supervision and notification, I would not have finished this thesis. My appreciation also is extended to all the instructors in the Civil Engineering department who have similarly helped in the process of achieving success in this study.

I would like to declare my deepest appreciation to my family for supporting and encouraging me through my entire life and in this specific case. Their motivation and encouragement is so valuable for words to express.

I have to emphasize the great support I have received from all my friends in Civil Engineers department. I fully appreciate them sharing and exchanging ideas which truly contributed to this study.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1

INTRODUCTION

1.1 Preface

The progressive collapse of structures is commenced when the primary component (s), usually columns, is eliminated. When a column is suddenly removed as a result of a vehicle collision, explosion, terrorist attacks, earthquake and other natural or artificial hazards, gravity loads (Dead Load and Live Load) gets transmitted to adjoining columns in the structure. If these primary elements are not appropriately designed to bear and redistribute the overloading, that portion of the structure or the whole of the structure may collapse. The columns of a building persist to fail until the extra loading on the column becomes steady. Consequently, a significant portion of the building may fall down because of the larger and superior damage to the building than the preliminary impact (Kevin A. Giriunas, Dr. Halil Sezen, 2011).

1.2 Significance of Progressive Collapse

Although progressive collapse is generally a rare accident in developed countries, but its effect on buildings is very dangerous and costly.

Without significant consideration of adequate continuity, ductility and redundancy, the progressive collapse cannot be prevented. The progress of consecutive damage during the progressive collapse, which occurred in Alfred P Murrah building in Oklahoma City, in 1995, resulted in 168 fatalities. These huge fatal results may be continued in other similar buildings, unless effective measures are considered for preventing progressive collapse. Other similar accident was due to the collapse of twin towers of World Trade Center during the suicide attacks in New York City.

There are numerous severe threats which caused by progressive collapse in a structure that may lead to fatality. After the incidents, which are mentioned above, the demands on the assessment of buildings towards preventing the progressive collapse have increased.

Although, there have been many researches carried out on progressive collapse, the increase in terrorist attacks, especially loss of lives (nearly 3000 died in the attacks of September 2001) in the World Trade Center case, lead to the development of new guidelines, such as GSA, DoD, and UFC for assessing and preventing progressive collapse.

In addition, a limited number of investigations are done on steel structure, especially on dual frame systems (moment frame with bracing system) so far, lots of investigates are done on reinforced concrete structure until now. The researches on the progressive collapse resistance of steel framed buildings are gradually increasing with the improvements on steel materials, technology and methods particularly in the developed countries.

1.3 Objectives of this Study

This study aims to do a quantitative comparison between progressive collapse potential of two different asymmetric existing steel frame systems with different number of stories. The results will be compared from the point of structures vulnerability to progressive collapse, using alternate load path method and analyzed by linear static procedure based on GSA 2003 guidelines. Also, in case of the buildings failing due to progressive collapse they will be rehabilitated and the best recommendations for preventing progressive collapse will be presented.

So, the main objectives of this study are:

- To assess the susceptibility of two existing buildings (nine-story and six-story with dual frame system and simple building frame system respectively) to progressive collapse.

- To rehabilitate the structure (s) under consideration by using alternate load path method in case of high progressive collapse potentiality.

- To make a comparison between different steel frame systems with different number of stories and various sections (built-up I-section and built-up box-section) regarding to progressive collapse incident.

- To find the proper steel sections used in nine-story and six-story buildings, regarding to progressive collapse incidents.

- To determine the appropriate recommendation (s) for preventing progressive collapse in these structure.

It should be mentioned that the main objective of carrying out the above mentioned study is to protect lives of people in the event of considerable damage to the buildings.

1.4 Thesis Outline

This study includes six chapters.

Chapter two is comprised of literature review. This section is devoted to the general definition of progressive collapse, significance of progressive collapse, mechanism of progressive collapse, major structural sources of progressive collapse, a list of potential load hazards which generate progressive collapse, technical definition of progressive collapse, Tie Force, analytic methods for evaluating progressive collapse, analysis methods of progressive collapse with the explanation of their advantages and disadvantages, practical ways for decreasing the progressives collapse, the pass on some historical and important cases of progressive collapses, method used in standards and codes for preventing progressive collapse, introducing of standards and codes related to progressive collapse, experimental researches regarding to progressive collapse, progressive collapse criteria along with their objectives, application and important documents for minimizing and preventing progressive collapse and at the end, the description of GSA guidelines which has been used in this study for preventing progressive collapse and the description of linear static analysis are given in sections 2.1 to 2.17 respectively.

Chapter three is allocated to general description of structures. The outline of this chapter is first introduced in section 3.1. The geometry and the system of the building, design and analysis software, materials properties, definitions for steel sections, connections, loading of the structures and description of buildings are provided in sections 3.2 to 3.8 of this chapter respectively.

Methodology of linear static analysis along with choice of methods for preventing the progressive collapse (alternate load path method), load combination, calculation of the Demand Capacity Ratio, the selection of columns for removing based on GSA guidelines are given in chapter four.

Chapter five includes results and discussion. This chapter is divided into three sections. Modeling the building, removing the columns based on GSA guidelines, analyzing the structure and computing the Demand Capacity Ratio for beam and columns then drawing the considered frames with their DCRs for nine-story, six-story (before and after rehabilitation) building are given in sections 5.1, 5.2, and 5.3 respectively.

Chapter six includes summary and conclusion. A summary of what has been prepared and the significant results along with the comparison between case studies (Tables 6.1, 6.2, and 6.3) are given in sections 6.1 and 6.2 respectively. The final conclusion of the thesis is included in section 6.3. Section 6.4 introduces recommendations for future studies.

Chapter 2

LITERARTURE REVIEW

During the recent decades, a lot of attention has been paid to probable progressive collapse among the building owners in different parts of the world. This is because of the fact that progressive collapse is a potentially destructive event for huge buildings leading to significant number of casualties and injuries for their residents and also may lead to significant loss of properties.

2.1 Definition of Progressive Collapse

According to Allen and Schriever (1972), progressive collapse occurs when a primary structural element of a building fails to bear an accidental overload. This failure will be distributed to other neighboring weight bearing components. As a result, the intensity and coverage of the total damage is disproportionate to the original cause.

Progressive collapse, according to Song et al. (2010), is defined as an accidental event caused by a man made or natural disaster. This type of structural failure is mainly due to the result of the loss of one or a number of supporting elements in a building. At the present time, in order to prevent or minimize the potential hazards and destructive consequences of progressive collapse in the existing or future buildings, a significant number of approaches have been provided by authorized bodies, international and local centers and societies all over the world.

There have been a many studies for improving design and resistance of structural elements of buildings against progressive collapse. Finally, these studies have resulted some modified design codes and preventive technical measures against progressive collapse. Some computer modeling approaches have also been developed for simulation and cost estimation of progressive collapse. On the other hand significant full scale physical testing methods have yet to be developed for better understanding of progressive collapse.

2.2 Significance of Progressive Collapse

Although progressive collapse is generally a rare accident in developed countries, its effect on buildings is dangerous and costly. Without significant consideration of adequate continuity, ductility and redundancy progressive collapse cannot be prevented. In 1995, the progress of consecutive damage during the progressive collapse of the Alfred P Murrah building in Oklahoma City resulted in 168 fatalities. Such fatal results may continue unless effective measures are considered for preventing progressive collapse. The collapse of twin towers of World Trade Centre was another example to progressive collapse due to terrorist attack.

2.3 Mechanism of Progressive Collapse

Referring to ASCE 7 (2002), Lew defines the process of progressive collapse as the spread of an initial failure that occurs consecutively from an element to another one, leading to total or partial structural collapse. In other words, if the adjoining structural elements of a building are not able to stop further progressive failure, progressive collapse occurs. In the process of progressive collapse which occurs as a short time dynamic and non linear accidental event, structural members are predisposed to non linear deformation (Lew, 2005).

As an example, when an explosion destroys a column of a multi-story framed building, a significant displacement occurs among the structural elements situated above the damaged column. In this situation, if the beams and columns could be able to provide a cautionary response to prevent the collapse of the floor supported by the failed column, this progressive collapse will be prevented (Lew, 2005).

According to Kim and Kim (2009), during the process of progressive collapse, a series of constructional failure causes partial or complete collapse of the structure.

2.4 Primary Structural Sources of Progressive Collapse Defined by Applied Research Associate Inc

Progressive collapse is caused by abnormal loading condition based on the four primary sources:

Accidental impact, Faulty or defective construction practice, Foundation failure, and Violent change in air pressure or explosion (GSA, 2003).

The building foundation and foundation connection should be designed as such that for the case of a sudden removal of a primary load bearing elements, these components are competent to resist the potential redistribution of forces.

2.5 Potential Load Hazards Triggering Progressive Collapse

In order to decrease the destructive event in buildings, National Institute of Standards and Technology (NIST) has published a list of potential load hazards triggering progressive collapse as follows:

- Accidental events, such as; airplane crashes, car crashes, etc.
- Errors in design and/or construction process
- Fire accidents
- Violent and harsh change in air pressure (explosion).
- Accidental overload
- Explosion caused by bombs
- Vehicular collision
- Hazardous materials

According to NIST each of the above factors may lead a building to progressive collapses. Although these events may occur very rare, but unfortunately a common mistake among architects and building designers is that they don't pay attention to mentioned hazards in construction design and they don't consider protective strategies for them.

2.6 Technical Definition of Progressive Collapse

From the technical point of view, Sezen and Giriunas (2009), suggest that, when one or more vertical load carrying elements (typically columns) are removed as a result of a manmade or natural accident, the weight of the building is transferred to the nearby columns in the building.

Additionally, these researchers state that, if the resistance of these nearby columns is not enough to resist or transfer this accidentally over loaded gravity load, the structure related to this failure will eventually collapse, resulting more consecutive damage to the building in comparison with the initial damage.

2.7 Progressive Collapse Requirements based on UFC (2010)

UFC 4-010-01 needs to all existing and new buildings of three stories or higher be designed to resist progressive collapse. UFC 4-023-03 recommend two levels of design processes to avoid PC:

- The first level for designing the structure to resist PC employs the Tie Forces method, which is based on the membrane tension or chain (catenary) response of the structure. This design level can be utilized for structures assigned Very Low Level Of Protection and Low Level Of Protection (VLLOP and LLOP). Only horizontal ties are needed for buildings (structures) assigned VLLOP, whereas both horizontal and vertical ties are mandatory for buildings assigned LLOP.

- The second level for designing the structure to resist PC employs the alternate load path method based on flexural performance of the floor system, as the structure must bridge across eliminated load bearing components. This is mandatory to use mentioned design level for structure assigned Medium Level Of Protection and High Level Of Protection (MLOP and HLOP).

This is clear that alternate load path method relies to tie force method, since tie forces requirements which are necessary for VLLOP and LLOP, and additional ductility requirements must be applied for MLOP and HLOP. Where, a sufficient tie force cannot be applied in a vertical structural element, in that case the alternate load path method is allowed to be employed to confirm that alternate paths are available and the structure can bridge over removed component (Nabil A. Rahman et al., 2007).

2.7.1 TIE FORCES

This method (Tie Force) aims to tie the building together mechanically. Also, it enhances and develops the continuity, ductility, alternate load paths in structure. Tie forces should be applied by the existing structural components that have been designed based on conventional design methods to carry the standard loads which may be imposed upon the building. In horizontal dimension three ties are considered, longitudinal, transverse and peripheral. Vertical ties, on the other hand are required in columns and load-bearing walls. The Figure 1 shows the mentioned ties for a frame construction. It should be mentioned that these tie forces are different from "reinforcement ties" as described in ACI 318 Building Code Requirements for Structural Concrete (UFC, 2010).

The structural elements (beams, girders, spandrels) and their connections should be able to carry the required longitudinal, transverse, or peripheral tie force magnitudes.

2.7.1.1 Longitudinal and Transverse Ties

Designer should utilize the floor and roof system to supply the adequate longitudinal and transverse tie resistance. The structural components could be applied to provide some or even all of the required tie forces.

The longitudinal and a transverse tie force should be extended orthogonally to each other within the floor and roof system. This is mandatory to fasten the peripheral ties to these ties (longitudinal and transverse tie force) at each end.

2.7.1.2 Peripheral Ties

Designer should utilize the floor and roof system to supply the adequate peripheral tie strength. The structural components could be applied instead, if they can be demonstrated able to carry the peripheral tie force.

2.7.1.3 Vertical Ties

Designer should utilize the columns and load-bearing walls to carry the adequate vertical tie strength. Each of these elements (column and load-bearing wall) should be fasten continuously from the foundation to the roof level.

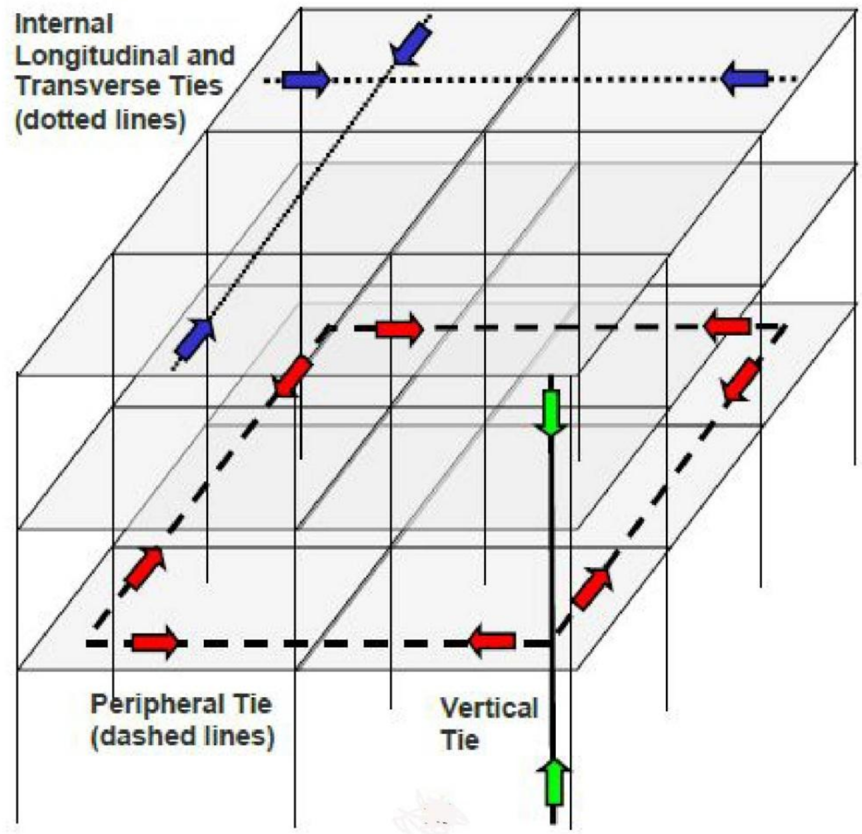


Figure 1: Tie Forces in a Frame Structure
(Source: UFC 2010)

Location restrictions for internal and peripheral ties are shown in Figure 2, below. They are parallel to the long axis of a beam.

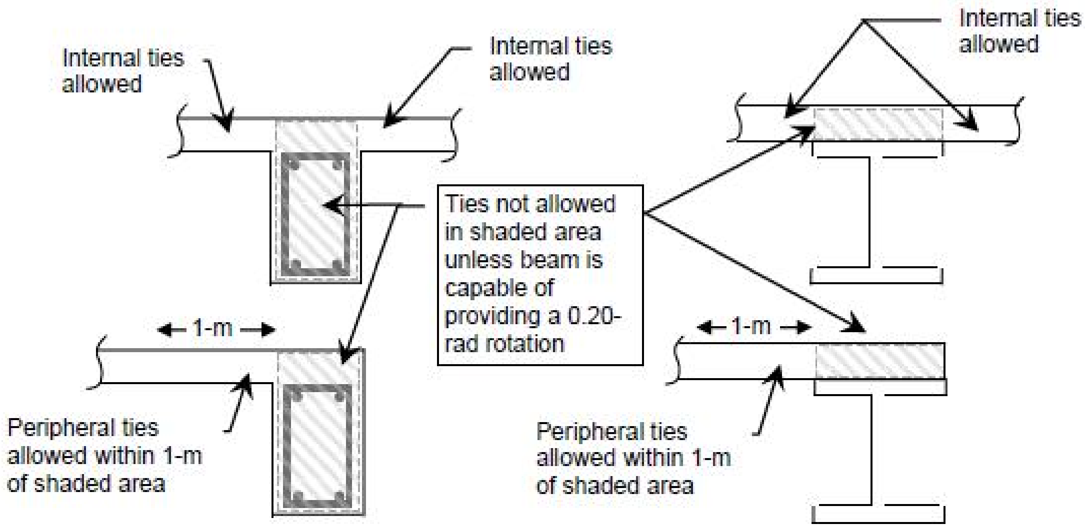


Figure 2: Location restrictions for internal and external peripheral Ties that is parallel to long axis of a beam, girder or spandrel.
(Source: UFC 2010)

2.8 Analytic Methods for Evaluating Progressives Collapse

A considerable amount of detailed technical data and guidelines have been proposed by standard authorized centers such as the General Services Administration (GSA) and Department of Defense (DoD) in USA.

2.8.1Alternate Path Method (APM)

This innovative method has recently been proposed by DoD. In this method, the designer of a building assumes alternative paths in the building. If one component fails to bear the accidental overload then the progressive collapse will occur. This alternative path is mainly designed for preventing the collapse. Alternate path method is commonly recommended by the US general service administration (GSA, 2003), especially for buildings with maximum ten stories high, based on a feasible framework. Additionally the inter agency security committee (ISC, 2001) encourages the researchers to use

Alternate path method for evaluating the susceptibility of buildings to progressive collapse.

2.8.2 Different Analysis Methods of Progressive Collapse in Alternate Path Method

The following analysis procedures are proposed for progressive collapse. These methods have also been suggested by FEMA 274 for seismic analysis:

- Linear Elastic static method (LS)
- Linear Dynamic method (LD)
- Non linear Elastic static method (NS)
- Non linear Dynamic method (ND)

2.8.2.1 Advantages and Disadvantages

Advantages and Disadvantages of the above methods have been investigated by different researchers. The above four methods were studied by Marjanishvili and Agnew (2006), through applying them in a sample building showing specific properties of each of them. They found that both of the static and dynamic analysis should be used for achieving the best results for progressive analysis.

On the other hand, Powell (2005), in his study concluded that the non linear analysis is the best method for designing the new building to resist progressive collapse. He came to this conclusion through comparing linear elastic static (LS) analysis, with non linear elastic static (NS) and non linear dynamic (ND) analyses. Regarding the two dimensional frame analysis, Kaewkulchai and Williamson (2003), reviewed these analysis procedures for progressive collapse in different structures. They concluded that, since the linear analysis may not be able to study the dynamic effect produced by the sudden exclusion of columns, then such type of analysis may provide non conservative results for designing the new structures. But for assessing and analyzing the vulnerability of existing buildings (structures) to progressive collapse and making comparison between two or more case studies linear analysis is a proficient procedure.

2.8.2.2 Disadvantage of Non Linear Dynamic Analysis

Generally, non linear analysis is conducted for defining the dissipation of energy, yielding of the materials and in order to reviewing inelastic deformations as well as cracking and fracture. One important disadvantage of this analysis method is that it is performed in a time consuming, step by step method. On the other hand, since the definition of structural behavior of connections between beam to column for steel and concrete is a very complicated issue, the analysis procedure is not suitable for assessing the vulnerability of existing mid-rise buildings (3-D models) in order to make comparison between two or more case studies. In this regard, Lew concludes that, for low and mid- rise building, this method is not performed routinely.

2.9 Practical Ways for Minimizing Progressives Collapse

Researchers have proposed three scientific methods for reducing the probability of disproportionate collapse in buildings.

- Alternate load path
- Improved local resistance for critical component
- Inter connection or continuity

2.9.1 Alternate Load Path

According to ASCE 7, the buildings will be enhanced in a way that if a primary component faces damage or collapse, progressive collapse would not occur. Although the "alternate load path" method is used for analysis, it is also used for preventing the collapse. This method is based on the redundancy improvement, ensuring that, the loss of any single component would not eventually lead to progressive collapse. In this method the designer tries to consider alternate paths when it seems that one or more components in the buildings may fail because of accidental over load or force. Most researchers believe that this is a simple and direct approach.

2.9.2 Improved Local Resistance

According to ASCE 7, the shear and flexural capacity of perimeter columns and walls will be enhanced in order to guarantee more protection through decreasing or limiting the progress and strength of the primary damage.

In this approach, additional resistance is considered and established for critical components of a building that might be subjected to accidental over load or explosion attacks. Shankar (2004) believes that continuity and inter connection in the whole structure will eventually lead to improvement of redundancy and local resistance. He believes that this method is more effective than increased redundancy alone. He also suggests that for reducing the susceptibility of buildings to disproportionate collapse, the best approach involves a suitable combination of improved redundancy, local resistance and inter connection.

2.9.3 Inter Connection or Continuity

This approach is generally a mixed approach for improving both redundancy and local resistance. This approach is based on the evidence that effective interconnection, although might be with the additional cost, will effectively prevent or reduce critical failures in building components (ACI, 2002).

2.10 Some Important Cases of Progressive Collapses

In these section important historical incidents of progressive collapse is given.

2.10.1 Progressive Collapses of Ronan Point Apartment

One of the most important accidents, which led to closer consideration of progressive collapse, was the disproportionate collapse of the Ronan point apartment tower in 1968, in England. Since then, analysis and prevention of progressive collapse has been considered as one of the most important challenges for code-writing and other responsible bodies in this field. They tried to develop design rules and criteria for preventing or minimizing susceptibility of future failures of building structures.

This event was occurred after a gas explosion in the kitchen of flat located at the 18th floor of the 22-story apartment tower in West London. This explosion knocked out load bearing pre-cast panels near the corner of this tower. The lack of support at the 18th floor led to the collapse of upper floors and finally this process continued as a chain reaction to the lower floor (four people died in this accident).

Figure 3: Progressive collapse in Ronan Point Building (16May 1968)
Source:http://www.emergencymgt.net/sitebuildercontent/sitebuilderfiles/ProgressiveCollapseBasics.pdf

2.10.2 Progressive Collapse in Murrah Federal Office Building (1995)

This type of progressive collapse was occurred after the explosion of a bomb installed in a truck, parked at the base of the building. Three main columns of the building were damaged leading to the failure of a transfer girder. This process ended with the collapse of columns supported by the girder and floor areas supported by the damaged columns.

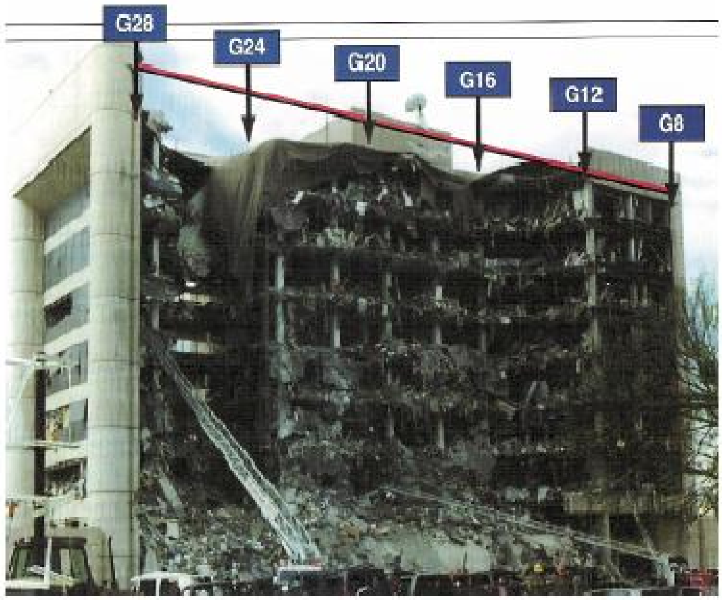


Figure 4: Murrah Federal
Source:http://www.emergencymgt.net/sitebuildercontent/sitebuilderfiles/ProgressiveCollapseBasics.pdf

2.10.3 Progressive Collapse of the Twin Towers of WTC

During the attacks performed by Boeing jetliner, the structure near the impact zone was damaged losing its supports to the above load. The weight of the collapsing upper part resulted in a downward progressive failure.

Figure 5: World Trade Center
Source: http://www.emergencymgt.net/sitebuildercontent/sitebuilderfiles/ProgressiveCollapseBasics.pdf
Source: Shankar Nair. R. Progressive collapse basics

2.11 Method Used in Codes and Standards for Preventing Progressive Collapse

The Table 1 summarizes the rules and instructions assumed for preventing collapse in various codes and standards.

Table 1: Design approaches for preventing collapse in various Codes and Standards
(Source: Shankar Nair. R. Progressive collapse basics).

Approaches for design against disproportionate collapse adopted in selected codes and standards	Redundancy	Local Resistance	Interconnection	Threat-dependent analysis
ASCE 7-02	•			
ACI 318-02			•	
GSA…PBS, 2000	•			
GSA…PBS, 2003				•
GSA PC Guidelines	•			

Table 2 also provides a summary on how to use the three methods for preventing the collapse of the three critical cases (Ronan point, Murrah Federal building explosion and Twin towers airplane crash).

Table 2: Summary of the contribution of various standards to the collapse prevention of three buildings
(Source: Shankar Nair. R. Progressive collapse basics).

Would use of these codes and standards in their design have improved the performance of Ronan Point, Murrah and WTC?	Redundancy	Local Resistance	Interconnection	Threat-dependent analysis	Ronan Point	Murrah Building	WTC 1 & 2
ASCE 7-02	•				?	N	N
ACI 318-02			•		Y	?	N
GSA…PBS, 2000	•				?	N	N
GSA…PBS, 2003				•	N	Y	N
GSA PC Guidelines	•				N	N	N

Table 2 shows that if these codes were used for the design of the three buildings considered then the damage would have been lower in some cases.

2.12 American Society of Civil Engineers Standard 7 (ASCE 7) for Preventing Progressive Collapse

An important definition provided by American Society of Civil Engineers standard 7 (ASCE 7), for minimum design load for buildings and other structures is as follows:

The spread of primary failure distributed from one element to another that finally result in the collapse of the whole structure or a significant part of it in an accident. In this reason (ASCE 7) reminds that buildings should be clearly designed in order to be competent against collapse, especially against disproportionate forces. Although it is impractical to design structures to resist general collapse produced by severe abnormal force on a large portion of a buildings, but these buildings can be designed to decrease the effects of over loading, injuries and to minimize progressive collapse.

2.13 Unified Facilities Criteria (UFC) for Preventing Progressive Collapse

Unified Facilities Criteria (UFC) refers to many standard publications for different strategies considered for minimizing or limiting the probability of progressive collapse in future building design processes. These strategies include many related items, such as, building type, story height, design approaches and many other critical issues. In its detailed publications entitled as “design of buildings to resist progressive collapse” published in 2010. This standard system will be used for all Department of Defense in

United States of America (DoD) projects. DoD is the responsible for safeguarding national security of the United States which has been founded in 1947.

2.14 Design Approaches for Decreasing the Possibility of Progressive Collapse

ASCE 7 provides two common scientific approaches for decreasing the probability of progressive collapse, including direct and indirect design (UFC, 2010).

2.14.1 Direct design

In this approach, many explicit items related to considering resistance of progressive collapse will be followed during the design process.

- **Alternate path (AP) method**: ASCE 7 states that the building should be designed considering bridging over missing structural elements as well as the extent and intensity of accidental or over loaded damage to be localized (UFC, 2010).

- **Load resistance method (SLR)**: This method stresses that the structure or a part of it should be designed for increasing the strength to resist against specific load or force.

2.14.2 Indirect design

Based on this approach, the designer tries to increase the resistance of the structure through considering adequate levels of strength, continuity and ductility. In this regard, UFC (2010) refers to ASCE 7 defining general design guidelines such as (1) suitable plan layout, (2) integrated systems of ties, (3) ductile detailing, (4) structural systems redundancy, (5) beam properties in walls, (6) catenary behavior of the floor slabs, (7) load bearing systems in interior partitions and many other important technical issues. In this approach, in order to tie structure together, designers should consider the continuity, ductility, structural redundancy, and the provision of minimum levels of strength.

2.15 Experimental Researches Relating to Progressive Collapse

There are limited studies relating to the actual full scale analysis of progressive collapse in the literature. One of them investigated progressive collapse experimentally and also through computational analysis relating to two existing buildings, Ohio union building and Bankers life and casualty company building. The following pictures show the experimental procedures in these two buildings (Song, Sezen and Giriunas, 2010).

Figure 6: Before and after removal of four first-story columns of the Ohio Union building and its subsequent demolition.

Figure 7: Before and after removal of four first-story columns of the north side of the Bankers Life and Casualty Company building.

The computational analysis was performed by SAP 2000, focused on linear static analysis of both buildings. Results showed that the columns in the top story were under self-weight pressure more than the other columns, as a result of a loss of columns. This failure referred to smaller cross section and lower moment of inertia was used. They concluded that, the Ohio union state building could satisfy the GSA progressive collapse criteria for all frame members. Only five columns failed in this building. On the other hand BLCC building may not be able to satisfy guidelines proposed by GSA criterion even after removal of the first columns. Calculation of demand capacity ratio (DCR) and maximum displacement showed that after the removal of the last columns, buildings were most susceptible to progressive collapse. The beams were more critical against impact loads than columns in this study.

Kim and Kim (2008) conducted a research focused on analysis of collapse process of buildings constructed by steel moment frames, through a scientific consideration of feminize seismic connections. Their special variables in this study included resisting capacities against progressive collapse such as RBS (reduced beam section), WUF-W (welded unreal forced flange–welded web connection) and WCPF (welded cover plated flange). They compared two kinds of buildings constructed through using steel moment frames. One of them was for high seismic load and the other was for the medium level seismic load. Through the implementation of alternative path load study, these researches evaluated the vertical displacement of elements of the level after removing the column. They also studied the rotation of plastic hinge at the end points of the beams. Finally, their study led to the conclusion that the most effective element was the cover plate connection against the progressive collapse, especially among the medium level seismic sites.

Khandelwal, EL-Tawil and Sadek (2009) performed a research for evaluating the progressive collapse of steel braced fames through using models based on validated computational simulation procedures through applying alternate path method (APM) they conducted their standards on a ten-story building by removing important load bearing column and adjacent braces, in order to define the ability of the structure to resist the member loss. They finally concluded that the frame that was braced eccentrically was more resistant to progressive collapse than from that was braced concentrically.

Sadek et al. (2009) studied the behavior of steel beam column structures based on two kinds of moment resisting connections. Their study considered the performance of a center column under the vertical displacement process, with a focus on two beams spans as well as three related columns. They applied a significant amount of load under displacement control, up to the level that led to connection failure. The main goal of this study was to define the behavior of the connections, as well as to study their resisting ability to resist against tensile forces occurred in beams. They finally found a significant agreement between their experimental and simulation methodology of research.

Fu (2009) developed a scientific computation model for twenty-story building to analysis the progressive collapse process. He used an ABAOUS package for this modeling procedure and showed the general behavior of twenty-story buildings when encountered eventual loss of their columns. There was a significant agreement between his modeling results and experimental data found by researcher.

Samuel Tan and Albolhassan Astareh-ASL (2003) from the University of California evaluated the efficiency of steel building floors equipped with cable based retrofit against progressive collapse. They performed three tests including (1) specimen without any mechanism to resist against PC, (2) and (3) included some steel cables on the web of beams that are connected to the side of the last column at the edge of the floor. They discovered that inclusion of cables significantly increased the resistance against progressive collapse.

Sadek et al. (2010) conducted a study comprising two experimental and computational methodologies relating to two steel framed structures including three columns and two beams. This study was performed on two ten-story buildings that were designed for eliminating the probability of progressive collapse. They eliminated the beam–column assemblies from the exterior frames. The first test specimen included connections with welded, unreinforced flange-bolted web and the second specimen was comprised of connections equipped with reduced beam sections. Researchers increased the vertical displacement of columns to define their reaction in a simulated system. After the development of the collapse process of each assembly and depleting the capacity of vertical load carrying members the test was finished. At specific locations, the horizontal and vertical displacements, as well as the rotation at the ends of beams were observed and the corresponding applied loads were calculated. The results of this study showed that the rotational capacities for both of the connections were twice as large as the values achieved from the seismic test.

Khandelwal et al. (2008) developed some scientific models for evaluating the resistance efficiency of steel framed buildings against progressive collapse. They finally found a higher level of resistance among frames specified for high seismic areas than those designed for moderate seismic loads through evaluating with alternative path method.

Lee et al. (2008) conducted two non linear analyses for evaluating the resistance of welded steel moment frames against progressive collapse. They also developed a small trainer's model for defining the vertical resistance versus chord rotation of beams with dual span. In order to assess the maximum deformation demands, the researchers also evaluated the relationship between the gravity load and the chord rotation process. They finally found that the ratio of beam span to its depth is the most important index for defining catenary behavior of double-span beams.

2.16 Progressive Collapse Criteria

In order to prevent the destructive consequences of progressive collapses in existing buildings and buildings to be designed in future, many authorized standard centers, such as General Service Administration of USA (GSA), American Society of Civil Engineering (ASCE) and the Defense Department of USA (DoD) have proposed progressive collapse criteria for governmental and federal buildings in USA. This criterion is also applied for many buildings in other developed and developing countries.

2.16.1 Main Objective of Progressive Collapse Criteria

The main objective of these criteria is to protect lives in the event of significant damage to the buildings.

2.16.2 Important Documents for Preventing Progressive Collapses

Applied Research Associates' Security Engineering & Applied Sciences Sector developed both Unified Facilities Criteria (design of building to resist progressive collapse) for DoD and progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects for GSA.

Designers and architects refer to GSA and UFC documents when designing new buildings and facilities in order to improve the quality of buildings and structures. They are encouraged to insure that problems related to progressive collapse are reasonable, considered and prevented in the design and implementation processes (Herrle, and McKay, 2005).

Generally it can be concluded that both of GSA and UFC guidelines help analysts and designers to identify and decrease the accidental occurrence of progressive collapse. These guidelines have been provided referring to critical needs of contractor in construction processes of each building. These guidelines updates periodically.

2.16.3 DoD Criteria

Based on the DoD criteria, all new and existing buildings of three stories or more should refer to the Unified Facilities Criteria (UFC-4-023-03) titled as "Design of buildings to resist progressive collapse-PC UFC."

This criterion covers all masonry, wood and cold framed steel constructions in addition to reinforced concrete and structural steel facilities. It should be stated that PC-UFC criteria are basically provided for decreasing the probability of mass casualties instead of directly eliminating the initial damage (www.ccb.org/UFC/4-023.pdf).

2.16.4 Different Application of PC-UFC

Four different levels of protection (LOP) are proposed in these criteria:

- **VLLOP (Very Low Level of Protection):** In this LOP, only indirect design is used through defining the required levels of Tie Forces.

- **LLOP (Low Level of Protection):** In LLOP, both the indirect and direct methods are used incorporating a combination of vertical and horizontal Tie Forces. According to this LOP, when the needed vertical tie force capacity cannot be provided by a vertical structural element, then this element should be designed again or the alternate path method should be used for evaluation of the bridging process over the element, when it is removed. But alternate path method cannot be used for element with inadequate horizontal Tie Force capacity.

- **MLOP (Medium Level of Protection), and HLOP (High Level of Protection):** For the above two mentioned LOPs, alternate path methods are used for defining the level of flexural resistance as well as defining the catenary resistance provided by the Tie Forces.

2.16.5 GSA guidelines for Preventing Progressive Collapse

GSA guidelines provide suitable methodology and application criteria for evaluating the predisposition of new structures to progressive collapse.

2.16.5.1 Exterior Considerations

In this step, the following processes are commonly followed based on GSA 2003:

1-Analyses of the result in the case of a removal and loss of a column for one floor located above grade, located at or near the middle of the long side of the building.

2-Analysis of the result in the case of a removal or loss of a column for one floor located above grade located at or near the middle of short side of the building.

3-Analysing the accidental loss of one floor above the grade (1st story) located at the corner of the building.

2.16.5.2 Internal Considerations

For buildings with underground parking areas, the analysis should be carried out for possible accidental loss of one column between the basement and the ground floor in the underground car parking. The researcher should carry out analysis for each separate case (Marjanishvili, 2004).

2.17 Linear Static Analysis

In this analysis method, the researcher removes the column that is under consideration and then carries out analysis to calculate the Demand Capacity Ratio (DCR). When DCR of a structural element is higher than the acceptable limit for shear and flexure, the failure of the element is occurs. This analysis procedure is given in more detail in chapter 4 of this thesis.

Chapter 3

DEFINITION OF MODEL STRUCTURES

This chapter focuses on the details of two steel braced buildings Building A and B selected from the Iranian cities of Mashhad and Amol respectively. The building A is a nine-story high and the building B is a six-story high building.

The units kg, cm and meter are used for analysis and design in Iran. Therefore, for the case studies investigated in this thesis, the same units were adopted.

3.1 Outline of Chapter

The geometry and the system of structures are described in section 3.2. Design and analysis software is introduced in section 3.3. Material properties and steel elements used in structures are provided in sections 3.4 and 3.5. Sections 3.6 to 3.8 are allocated to connections, loading of the structures and general description of the two buildings.

3.2 The Structural System and its Geometry

For assessment of progressive collapse potential of different structural systems, the first step is to choose different structural models with different structural systems. It is obvious that different systems will face different vulnerability which will be assessed in the next step. The choice of models and their system is very important for this study. Thus, analyzing and assessing of building susceptibility to PC is chosen to find new solution in case of high vulnerability of structure to PC. In this study, Alternate Load

Path Method (APM) based on linear static analysis, which is reliable and also the preferred method according to GSA guidelines, has been used to verify and analyze the process.

Since using existing buildings as case studies would increase the validity of this study, then two buildings have been chosen based on their site plans that may be threatened by internal and external factors. These threats may occur as a result of explosion in heating system (internal factors), car accidents, terrorist attacks and floods (external factors). It is also necessary to remember that all the above mentioned factors will force the first floor (based on GSA guidelines). Neither of the buildings have equal bays defined as X and Y directions. In other word, they have different number of bays (short and long side). Using three dimensional models of both buildings, two exterior frames (short and long side) located at the nearby roads have been analyzed by considering only gravity loads (amplified Live and Dead Load) or vertical loads.

This is based on the assumption that after sudden removal of a column which has high level of vulnerability against external factors the lateral load is not important.

The first case is a nine-story residential building located in Amol city in Iran with noticeable vulnerability against progressive collapse.

The second case is a six-story building located in Mashhad city in Iran. It is a residential building with a high possibility for progressive Collapse and completely different frame system than the nine-story building.

Designers of both buildings have followed the Iranian 2800 guidelines which is based on American code (AISC-ASD 89).

Geometrical information of these two models is as follows:

- The nine-story building has got a dual frame system, designed as a medium (high) rise building.

- The six-story building has got a simple building frame system (gravity frame with concentric bracing system) and it is designed as a medium (low) rise building.

- The nine-story building has a moment frame system with bracing system in both X and Y directions.

- The six-story building's system is based on gravity frame system with bracing system in both X and Y directions.

- The nine-story building has four and six bays in X and Y directions, respectively.

- The six-story building has two bays and four bays in X and Y directions, respectively.

- Both buildings have asymmetric shape.

- Roofs of both existing structures are in-situ concrete slab type.

- Steel sections in both structures are comprise of: Built-up I-section which looks like IPE or IPB section, double IPE, double IPE with two or several plates that are welded to flanges and web, and Box section.

- The design of foundation and the type of foundation is not considered in both buildings.

- There is no bracing system in short side (X direction), beside the road, in both buildings.

- In nine-story building 100% of lateral load is allocated to braces while the moment frame should resist 30% of lateral load.

- The six-story building structure is braced against lateral loading.

3.3 Software Selection

Both buildings have been analyzed and designed by using the software product of SCI Corporation, called ETABS-3D version 9.5.0 as one of the powerful finite element computer programs.

3.4 Material Properties

The steel properties which have been entered manually and used for both buildings based on Iranian code which has been extracted from AISC-ASD 89 are as follow:

- Modulus of Elasticity: E = 2.039E+10 kg/m^2
- Poisson's Ratio: $v = 0.3$
- Weight per Unit Volume: 7833 kg/m^3
- Mass per Unit Volume: 798.1kg/m^3
- Minimum Yield Stress: 24000000 kg/m^2
- Effective Tensile Stress: 37000000 kg/m^2

3.5 Description of Steel Sections

The most popular steel section in Iran is IPE especially for beams; however, when it's not suitable, bigger cross-section with higher level of load bearing capacity should be used and this is implemented through welding plates together or even by using beams with higher web height with holes on the web called castellated beam. This type of beam is called CPE in Iran.

In case of an earthquake in a building with I column section, critical damage is likely to happen in the direction which the columns are bent in their weak axes (around the web). That's why hollow sections (box sections) are used during the design of the columns. There are also two more solutions as (1) either to increase the strength of IPE section by using multiple plates or (2) by combining plates with double IPE sections.

For the braces, double channels have been used for this specific case.

In nine-story building for beams a combination of plates which looks like IPE and IPB section and for columns BOXES are used.

In six-story building for beams; built-up I-section with plates which have been welded to bottom and top of flanges and CPE are used. Meanwhile, for the columns; double IPE and double IPE with plates in bottom and top of the flange, double IPE with plates which have been attached to web, bottom and top of the flanges and BOX sections have been used (see APPENDIX).

3.6 Connections

For nine-story building beam-column connections are rigid. It means that beams are continuous. The columns are continuous between the two story levels. Brace connections are pinned as well. The brace connections are properly located in place. The building has a dual frame system (moment system with bracing system).

For six-story building the beam-column connections are pinned together. The columns are continues between the two story levels. Brace connections are pinned as well. The brace connections are properly sited in place. This means that the simple building frame system (gravity frame with concentric bracing system) has been used.

3.7 Loading

Both buildings are classified in residential group defined as a category II according to the Iranian Earthquake Code.

- **Building A (Nine-story Building):** Live load and dead load of nine-story building for floors are 500 kg / m^2 and 300 kg / m^2 respectively. The dead Load of the surrounding wall is 800 kg / m^2 and dead load of the stair box in X direction is 2000 kg /m^2. For the roof, the live load is 150 kg / m^2 and dead load is 300 Kg / m^2.

- **Building B (Six-story Building):** Live load and dead load of the six-story building for floors are 200 kg / m^2 of 370 kg / m^2 respectively. The dead Load of the surrounding wall is 1420 kg / m^2 and dead load of the stair box in X direction is 1420 kg/ m^2. For the roof the live load is 350 kg / m^2 and dead load is 320 kg / m^2.

3.8 Description of Buildings

According to the above two case studies, buildings with (A) Nine-story and (B) Six-story are described as follow:

3.8.1 Nine-story building (Building A) with Dual Frame System

Three-dimensional model of the nine-story steel building is shown in Figure 8.

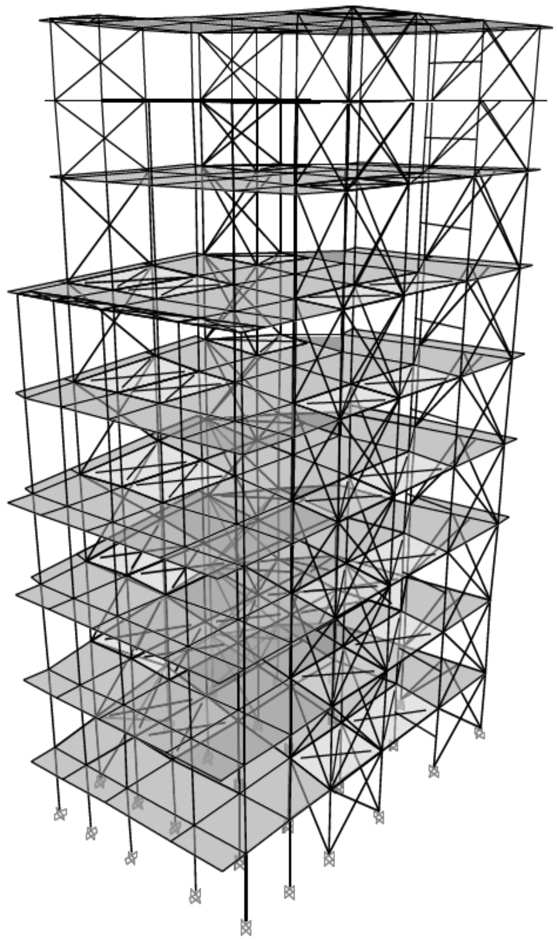


Figure 8: Three-dimensional model of the nine-story steel building.

The site plan of nine-story building is shown in Figure 9.

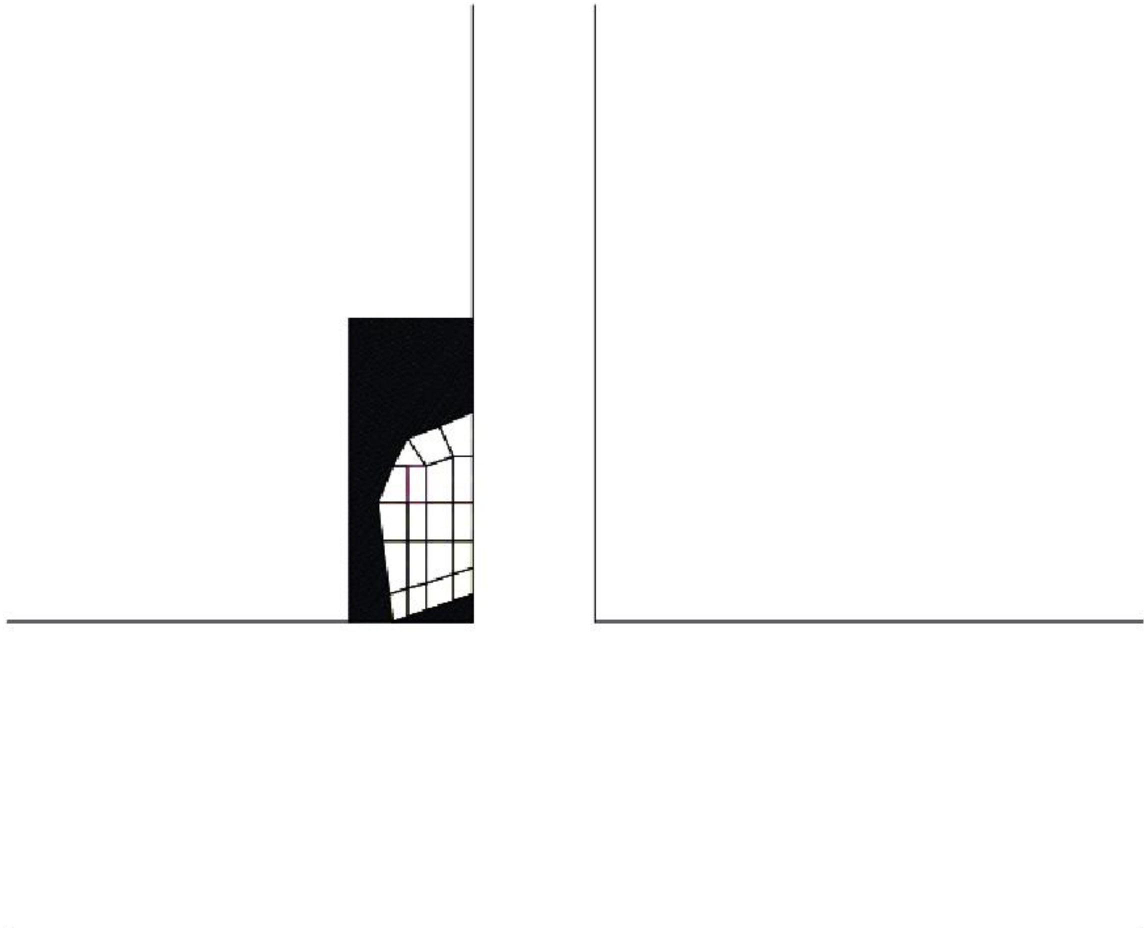


Figure 9: Site plan of nine-story building.

The plan (first floor plan) of nine-story building is shown in Figure 10.

In the plan, beams are shown with the letter B and columns are shown with the letter C.

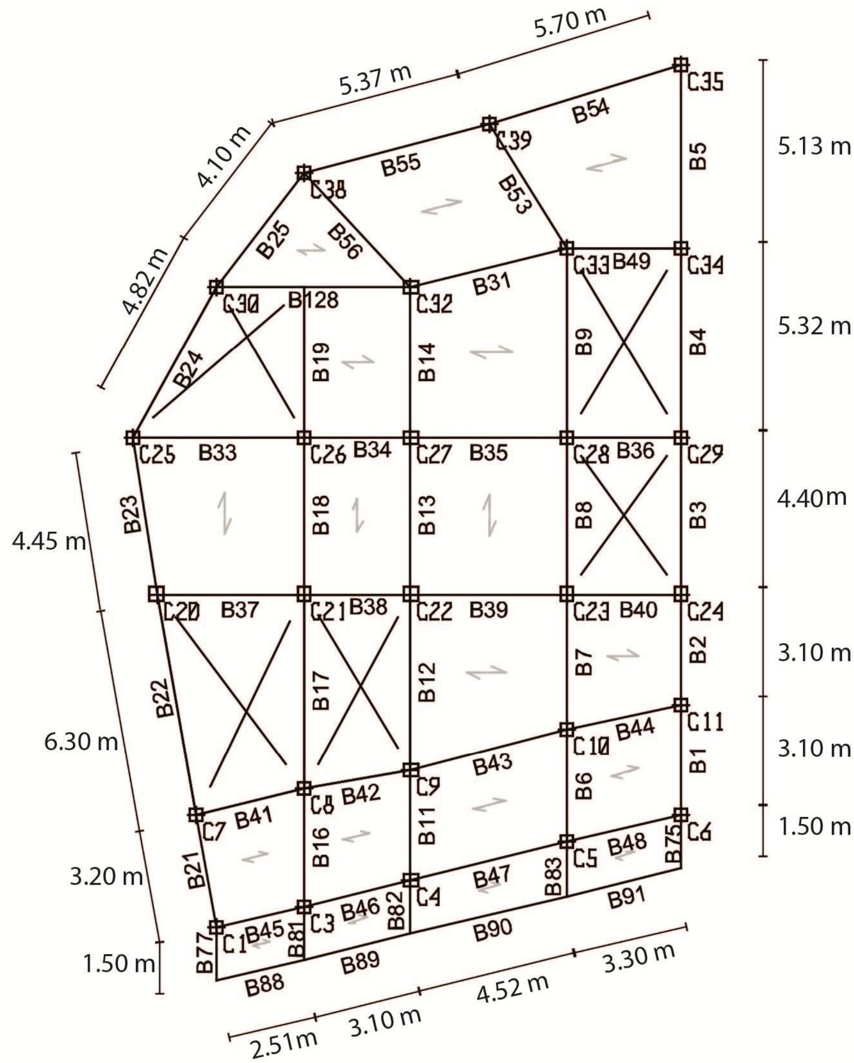


Figure 10: Two dimensional model of the nine-story steel building

Steel sections for the short side of building (A) (exterior frame, beside the road) are shown in Figure 11. The term BOX is referred to column sections. The first number represents the length and the second number represents the thickness of the boxes. The sections which are labeled as PG are built-up I sections which are made of combining three plates together (see APPENDIX).

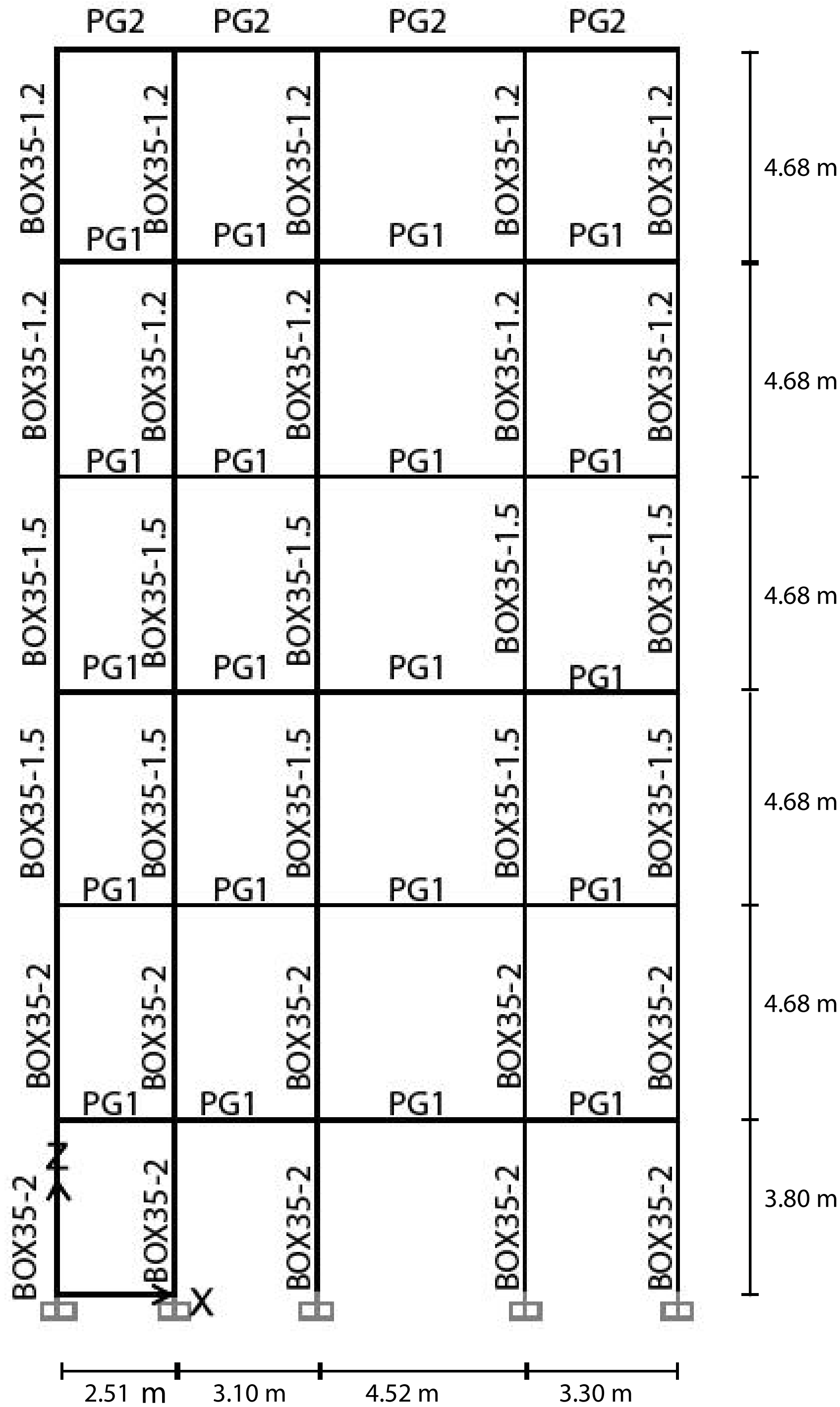


Figure 11: The sections label for short side of nine-story building

The steel sections for the long side of the building A (exterior frame, beside the road) are shown in Figure 12.

The term box is referred to the column members. The first number represents the length and the second number represents the thickness of the boxes. The steel sections which are labeled as PG are a built-up I sections by using three plates (see APPENDIX).

Braces made up of double channel sections, labeled with the letter “U”.

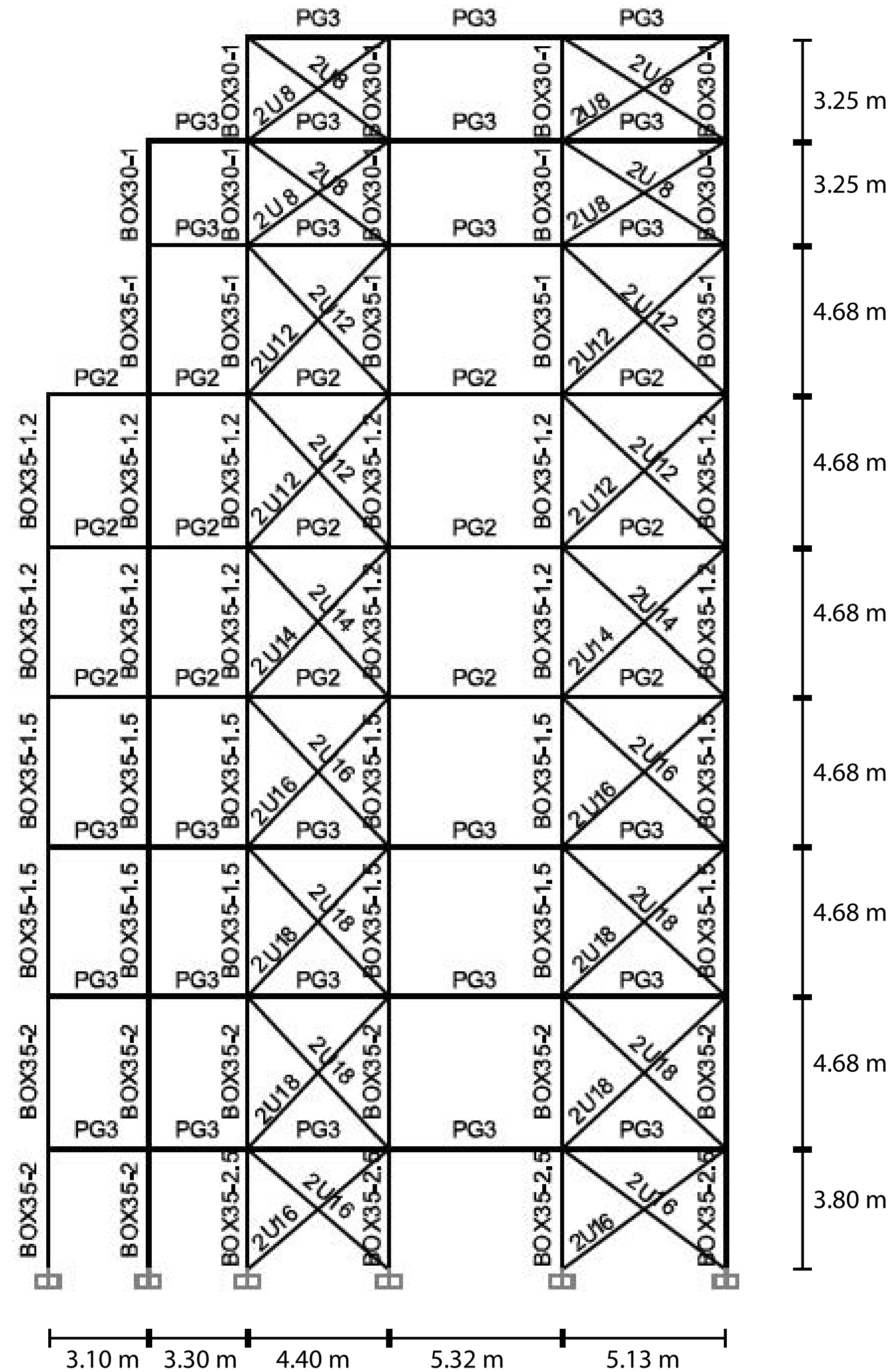


Figure 12: Sections label for long side of nine-story building

3.8.2 Six-story building (Building B) with Building Frame System

Three-dimensional model of the six-story steel building is shown in Figure 13.

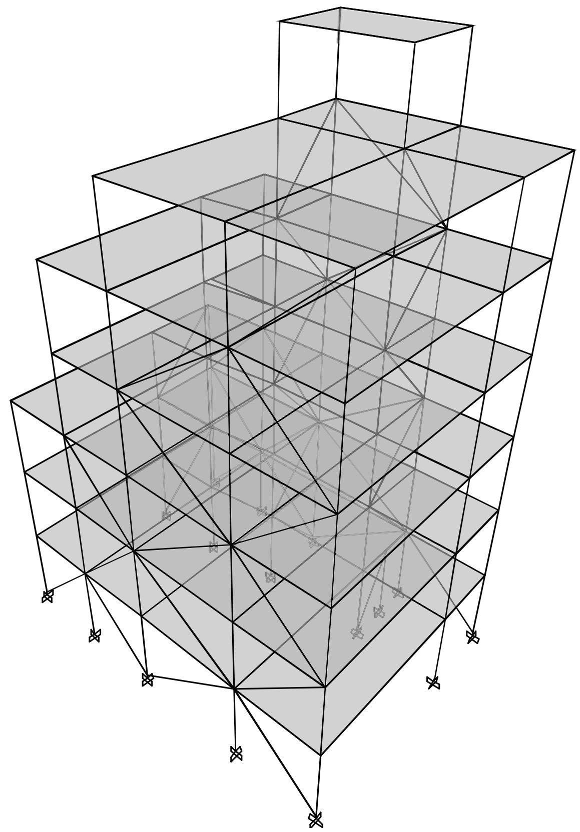


Figure 13: Three-dimensional model of the six-story steel building

The site plan of six-story building is shown in Figure 14.

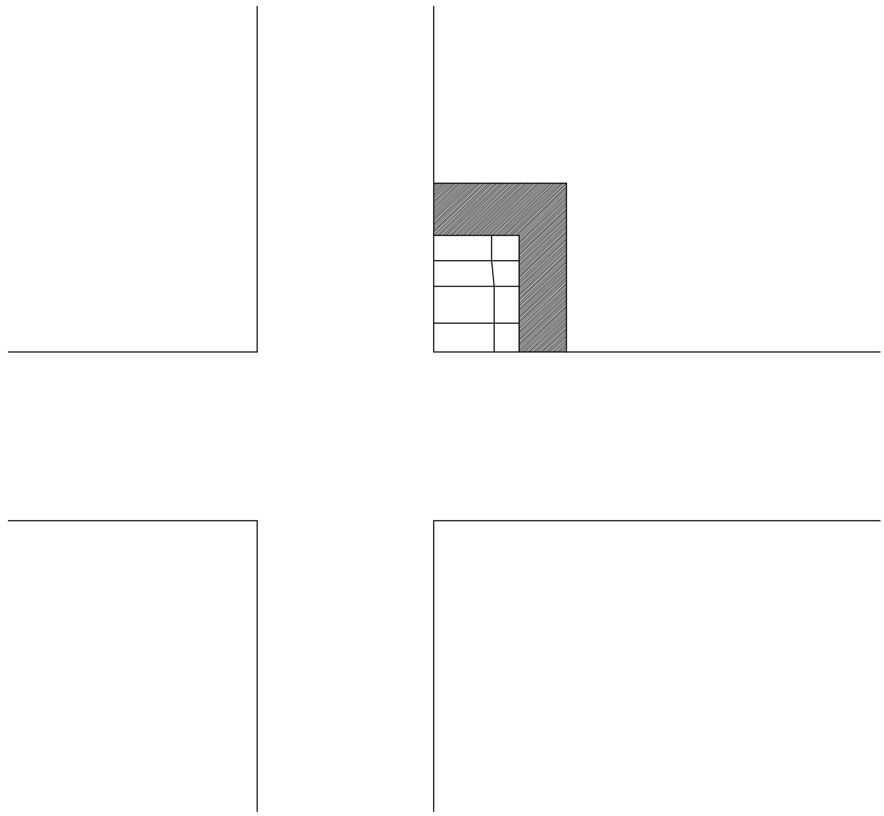


Figure 14: Site plan of the six-story building

The plan (first floor plan) of the six-story building is shown in Figure 15.

Beams are shown with the letter B and columns are shown with the letter C in the building plan.

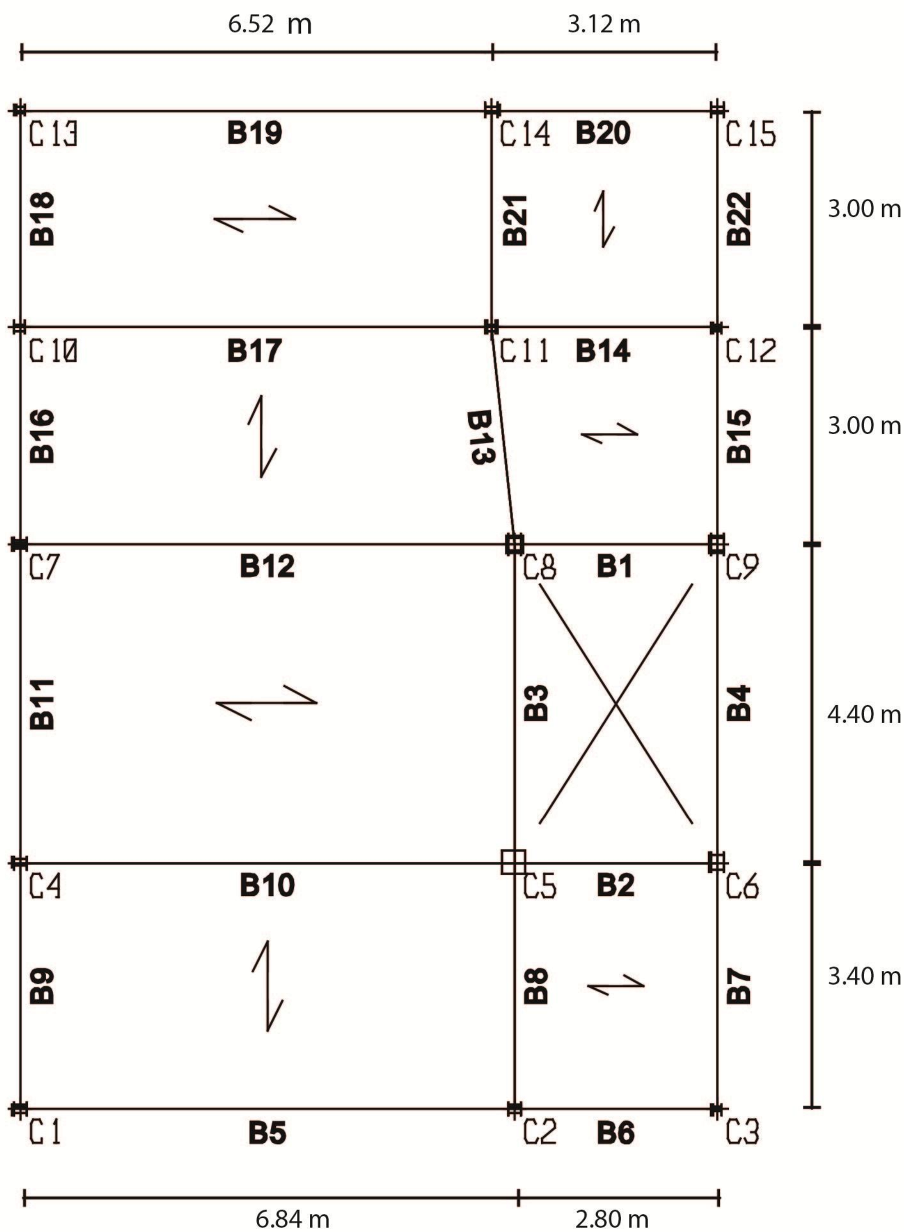


Figure 15: Two dimensional model of the six-story steel building

The sections label for short side of the six-story steel building (exterior frame, beside the road) is shown in Figure 16.

In six-story steel building, IPE, built-up I-section, built-up I-section with welded plates on top and bottom of flanges, built-up double I-section with welded plates on top and bottom of flanges, and castellated beam are used for the beams and for the columns; double IPE are used with welded plates on top and bottom and attached to the beam's web (see APPENDIX).

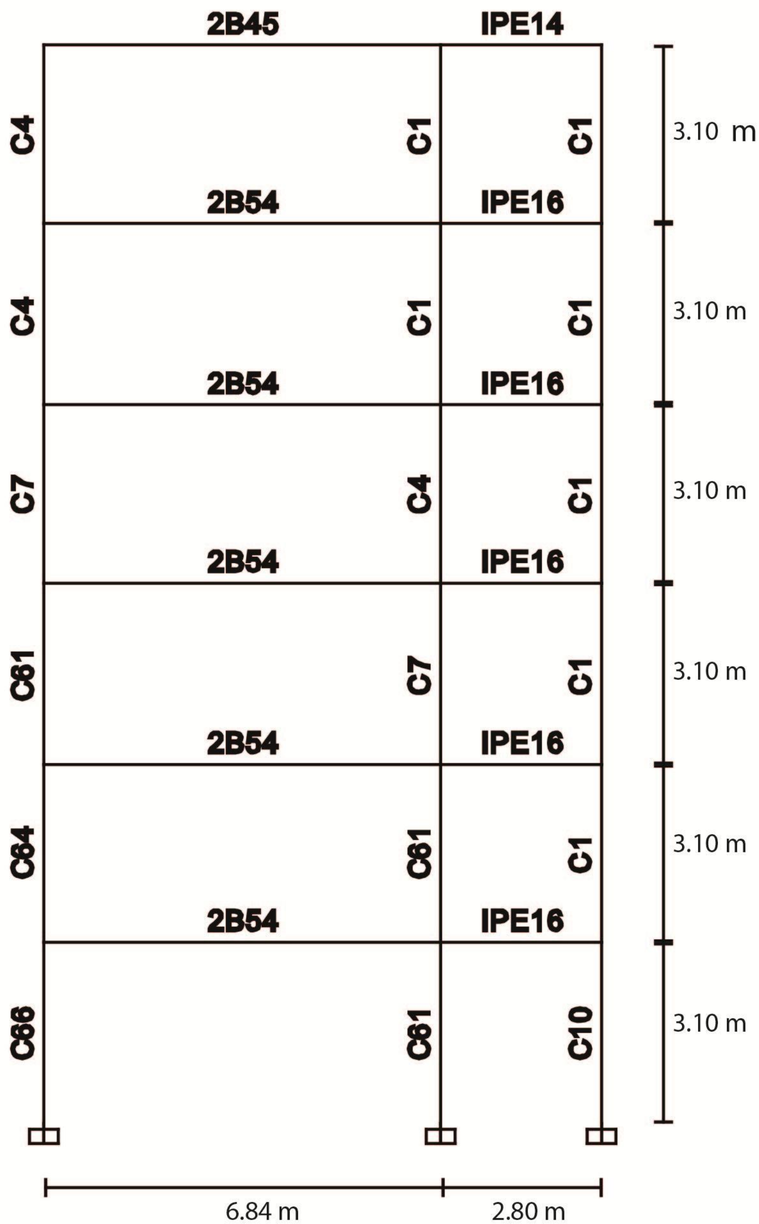


Figure 16: Section labels for the short side of the six-story steel building

The steel section designations for the long side of the six-story steel building (exterior frame, beside the road) are shown in Figure 17.

Braces made up of double channel sections, labeled with the letter “U”.

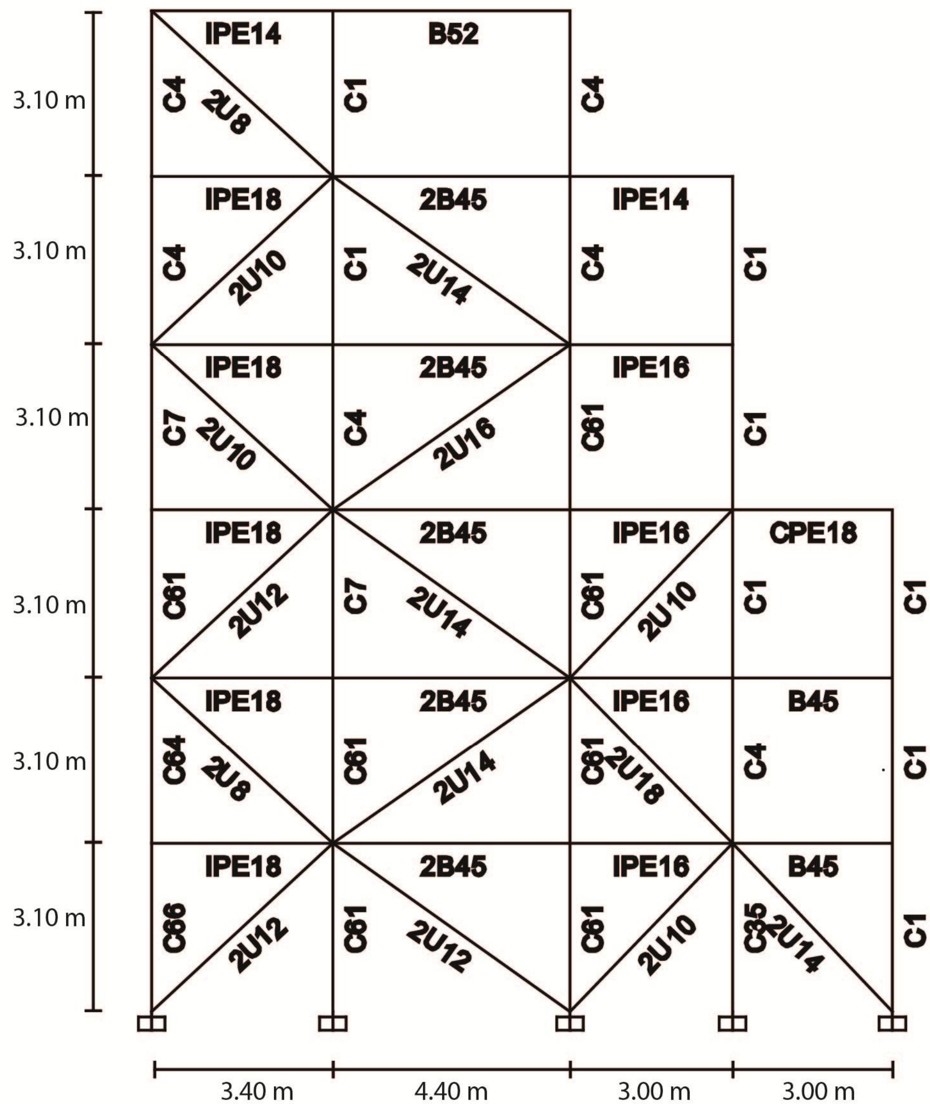


Figure 17: Steel sections labels for the long side of the six-story steel building

Chapter 4

CALCULATION OF DEMAND CAPACITY RATIO FOR PROGRESSIVE COLLAPSE

4.1 Flowchart Approach to Assessing the Progressive Collapse Potential

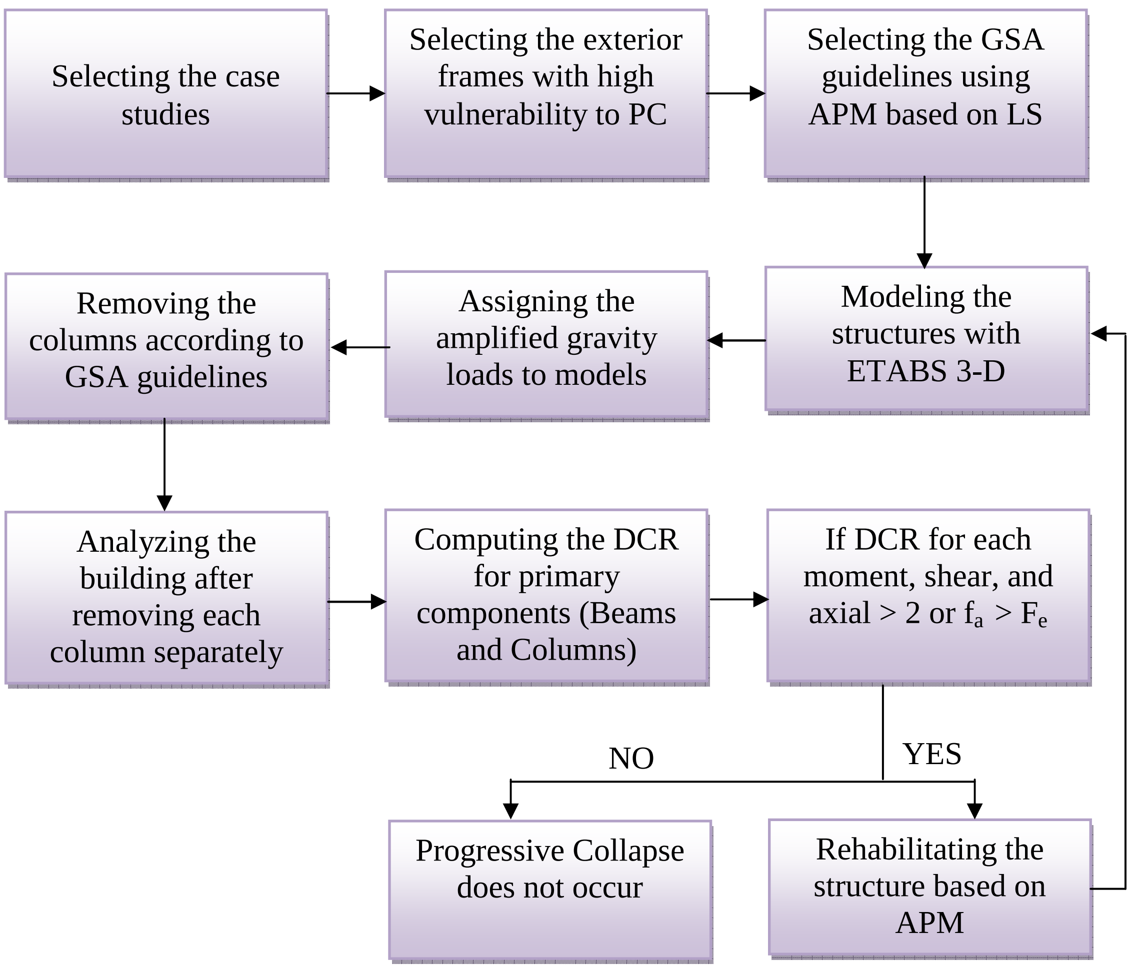


Figure 18: Flowchart approach to assessing progressive collapse potential

4.2 Choice of Guidelines

Different guidelines such as DoD, GSA, UFC, etc are being used for analyzing the process of progressive collapse. Among them GSA guidelines, which considers structures with less than ten-story, is the most compatible one for this case study.

The GSA guideline consists of four different methods for analysis as listed below:

- Linear Static Analysis
- Non Linear Static Analysis
- Linear Dynamic Analysis
- Non Linear Dynamic Analysis

According to GSA guidelines, "Linear static analysis" is the preferred method for analyzing the structures with potential for PC (GSA, 2003).

4.3 Methods for analyzing and Preventing the Progressive Collapse

As mentioned in literature, researchers have proposed three scientific methods for reducing the probability of disproportionate collapse in buildings consisted of alternate load path, improved local resistance for critical component, and inter connection or continuity. With regard to the U.S General Services Administration (GSA, 2003) and the interagency security committee (ISC, 2001), alternate load path is suitable method for evaluating and preventing the process of progressive collapse in buildings of up to ten stories (low to medium rise). Also, this study requires analysis of case studies, therefore, linear static analysis, which is a reliable method for assessing the vulnerability of buildings to PC. Thus, alternate load path based on linear procedure is used in this study according to GSA guidelines.

4.4 Choice of the Software for Computer Analysis

There are a variety of software that can be used for these analysis. In this specific case, reliable choices are SAP 2000, ETABS-3D, ASTAD Pro, DRAIN 2D-X and DRAIN 3D-X. For this study ETABS 3D was available and it is known to be fast, accurate and compatible with linear static analysis. Therefore, ETABS 3D has been used in this study.

4.5 Analysis of Loading

According to GSA guidelines, for static analysis procedures the below mentioned vertical load should be used for these case studies:

$$\text{Load} = 2(DL + 0.25LL) \tag{1}$$

Where:

DL = Dead Load and LL = Live Load

4.6 Calculation of Demand Capacity Ratio (DCR)

In order to determine the susceptibility of the building to PC, Demand Capacity Ratio should be calculated based on the following equation:

$$DCR=QUD/QCE \tag{2}$$

In which:

QUD= Acting force (Demand) determined or computed in element or connection/joint

QCE= Probable ultimate capacity (Capacity) of the component and/or connection/joint

Referring to DCR criteria defined through linear static approach, different elements in the structures and connections with quantities value less than 1.5 or 2 are considered not collapsed as follows:

DCR < 2.0: for typical structural configuration

DCR < 1.5: for atypical structural configuration (GSA, 2003)

Cases which have been chosen for this study have typical structural configuration.

Building structures in these case studies are dual frame system and simple building frame system where braces are designed for lateral load. Since the loading pattern used in this study for analysis is based on just gravity (amplified dead and live load), computation of DCR values for braces are neglected and according to past studies, DCR has been calculated only for beams and columns.

It should also be stated that by installing braces in structures for lateral loads, building resistance (columns) against progressive collapse will increase and DCR values would be so small while in case of omitting the braces the DCR values would be so high that the building may collapse.

In this study, Demand Capacity Ratio should be computed for moment, axial force, shear and possible combined forces (it has to be mentioned that DCR could get extracted from ETABS-3D).

4.6.1 DCR_{moment}

DCR for moment is calculated based on the equation (3) bellow:

$$DCR = M_{max}/M_p \text{ (Computed)} \tag{3}$$

Where:

M_{max}: Maximum actual (existing) moment

4.6.1.1 Plastic Moment

The plastic moment or simple plastic moment is the largest (maximum) bending moment that a section can resist. The formula for this plastic moment is:

$$M_p = F_y Z \tag{4}$$

M_p: Plastic moment capacity of the section when the axial force is absent

Z: Plastic modulus

F_y: Yield strength of material

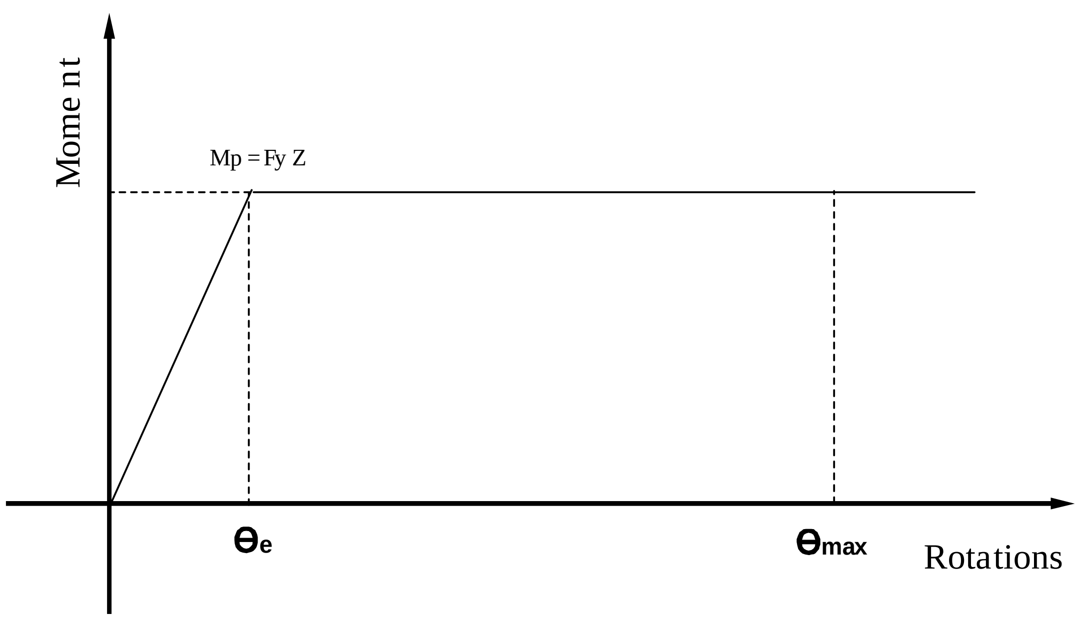


Figure 19: Moment curvature

M_p: Plastic Moment

θ_e: Elastic Rotation Limit

θ_{max}: Maximum Rotation

4.6.1.2 Influence of the Axial Force on M_p

Columns may carry considerable axial forces as well as bending moment. The axial force (P) tensile (compressive), reduces the M_p or plastic moment in columns. On the other hand, in many incidences this maximum value or maximum capacity needs to be reduced due to the existence of axial load.

Recommendation for considering axial compression on M_p (Bending + Axial Compression):

- If $P < 0.15\ P_y$, neglect the effect of axial compression or axial force on the plastic moment where:

  P: Actual axial force

  P_y: Maximum axial force or axial force causing yielding of the full cross section (corresponding to yielding)

  $$P_y = F_y A \tag{5}$$

  Where:

  A: Cross section area

  F_y: Yield stress of material

- Modify the plastic moment capacity (M_p) where $P > 0.15\ P_y$.

The formulas for calculation of this reduced plastic moment (by effect of axial forces) are listed below.

For rectangular cross section:

$$\frac{M}{M_P} = 1 - \left(\frac{P}{P_y}\right)^2 \tag{6}$$

For I-cross section subjected to bending according to its strong axis:

$$\frac{M}{M_P} = 1 - \left(\frac{P}{P_y}\right)^2 \frac{A^2}{4wZ_x} \tag{7}$$

Where:

$$\frac{P}{P_y} < \frac{A_w}{A}\ ; \tag{7.1}$$

$$\frac{M}{M_P} = \frac{A}{2Z_x}\left(1 - \frac{P}{P_y}\right)\{h - [A\left(1 - \frac{P}{P_y}\right)2b]\} \tag{8}$$

Where:

$$\frac{A}{A_w} \leq \frac{P}{P_y} \leq 1\ ; \tag{8.1}$$

For I-cross section subjected to bending according to its weak axis:

$$\frac{M}{M_P} = 1 - \left(\frac{P}{P_y}\right)^2 \frac{A^2}{4hZ_y} \tag{9}$$

Where:

$$0 \leq \frac{P}{P_y} \leq \frac{wh}{A}; \tag{9.1}$$

$$\frac{M}{M_P} = \frac{A^2}{8tZ_y}\left\{\left(\frac{4bt}{A}\right) - \left(1 - \frac{P}{P_y}\right)\right\}\left(1 - \frac{P}{P_y}\right) \tag{10}$$

Where:

$$\frac{wh}{A} \leq \frac{P}{P_y} \leq 1\ ; \tag{10.1}$$

Where:

M: Reduced plastic moment (modified moment)

M_p: Plastic moment when axial force is absent

P: Actual axial force

P_y: The axial force corresponding to yielding or maximum axial force

Z_X: Plastic section modulus (strong axis)

Z_y: Plastic section modulus (weak axis)

A: Cross section area

A_w: Web area or shear area

b,h,t,w : Cross section parameters shown in Figure 20.

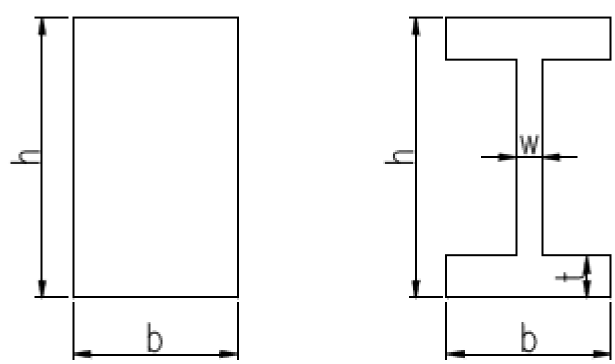


Figure 20: Cross sectional parameters

4.6.2 DCR_{shear}

DCR for shear is calculated using equation (11):

$DCR=V_{max}/V_p$ (Computed) (11)

Where:

V_{max}: Maximum actual (existing) shear

V_p: Plastic shear

Design for shear is represented in AISC as below:

LRFD Factored Design shear strength and ASD Service Allowable shear strength are presented here by (12) and (13):

LRFD Factored Design shear strength = $\varphi_v V_n$ (12)

ASD Service Allowable shear strength $=V_n/\ \Omega_v$ (13)

In which:

$\varphi_v = 1.00$ (LRFD)

$\Omega_v = 1.50$ (ASD)

V_n: Nominal shear strength

A_w: Area of web = $t_w d$

C_v: Web shear coefficient

1.0 for webs of rolled “I” – shaped sections (Conservative)

$V_p=\varphi_v\ V_n=Vn= 1.00(0.6F_yA_w)\ C_v$ (AISC Spec. G p. 16.1-64). (14)

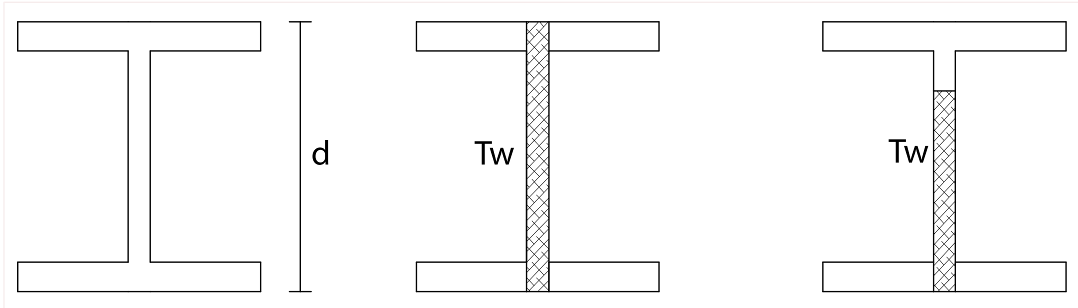


Figure 21: Aw configuration

4.6.3 DCR_{axial}

DCR for axial is calculated using equation (15):

$$DCR = Axial_{max}/Axial_{p} \text{ (Computed).} \tag{15}$$

Where:

$Axial_{max}$: Actual axial force

$Axial_{p}$: Axial force causing yielding of the full cross section

4.6.4 Selecting the Columns for Removing

To calculate DCR according to GSA guidelines, structures should be analyzed as below:

1. Analyzing the sudden removal of a column in one floor above the ground (1st story) which is located at or near the middle of the short side of the building. This situation will be assessed in case 1 (see Figure 22).

2. Analyzing the sudden removal of a column in one floor above the ground (1st story) which is located at or near the middle of the long side of the building. This situation will be assessed in case 2 (see Figure 22).

3. Analyzing the sudden removal of a column between the ground floor and the floor above the ground level (1st story) which is located at the corner of the building. This situation will be assessed in case 3 (see Figure 22).

Chapter 5

RESULTS AND DISCUSSIONS

The results of analysis and also the values of DCR for beams and columns are presented in this chapter. The susceptibility of two different case studies (nine-story building and six-story building before and after rehabilitate) with different frame systems against progressive collapse has been assessed. DCR of primary elements (beams and columns) are given with their specific details in all frames.

5.1 DCR for Nine-story Building

Located in Amol city in Iran, this nine-story steel building is constructed by dual frame system with X and inverted V Braces (moment frame with bracing system) in both X and Y directions based on Iranian Steel Standards that follow American or AISC-ASD 89 guidelines.

The sudden removals of the columns (Figure 22) from the nine-story building are analyzed according to GSA guidelines and also the building vulnerability against PC is assessed.

Using ETABS-3D, removal of columns and their consequences have been modeled through the following case studies:

- Case 1: removal of column in the middle of short side of the building.

- Case2: removal of column in the middle of the long side of the building.

- Case3: the removal of column in the corner of the building.

Referring to each case, the locations of the removed columns are shown in Figure 22.

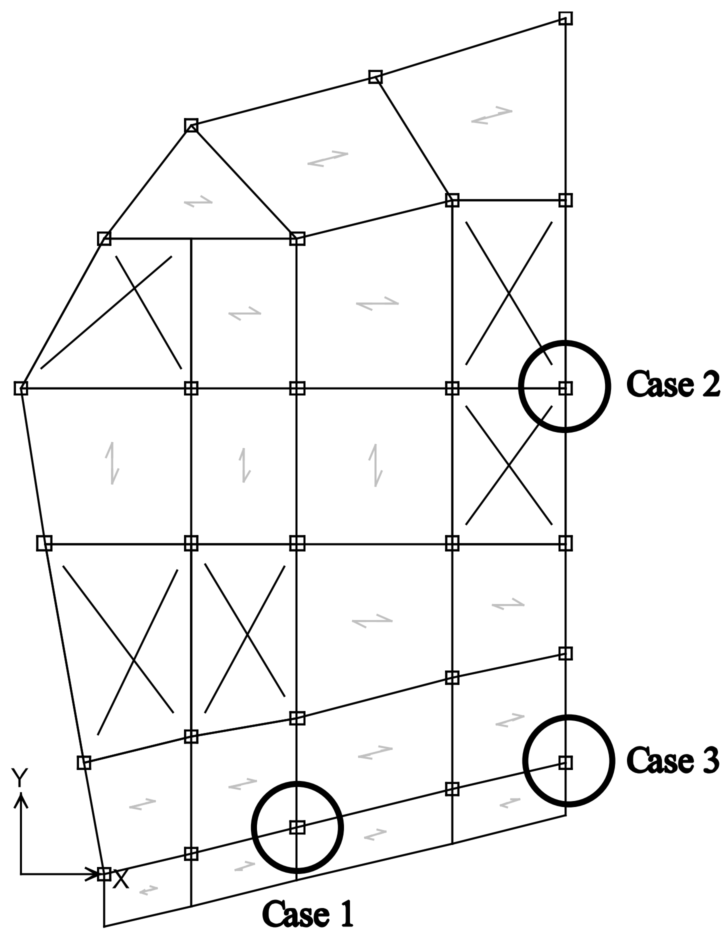


Figure 22: The location of the columns removed in the nine-story building according to GSA guidelines

Site plan of the nine-story building illustrates the geographic vulnerability of the building to any accidental damage, such as car accidents. For this reason, the resistance of the building in case of an accidental damage and possible progressive collapse should be assessed. In addition, the presence of "central heating and ventilation system" at the first floor of the building also indicates a possibility of a gas explosion which could be another reason for progressive collapse in this building.

Considering the above mentioned information, high vulnerability to PC following the sudden removal of column in the first floor according to GSA guidelines is analyzed.

5.1.1 Demand Capacity Ratio for Moment (Nine-story Building)

Figure 23 shows the computed DCR moment for the short side of nine-story building. Since none of the calculated DCRs (maximum is 1.087) are not even close to the limit 2, then it is concluded that, for the case of the nine-story building, the susceptibility of structure against progressive collapse is low.

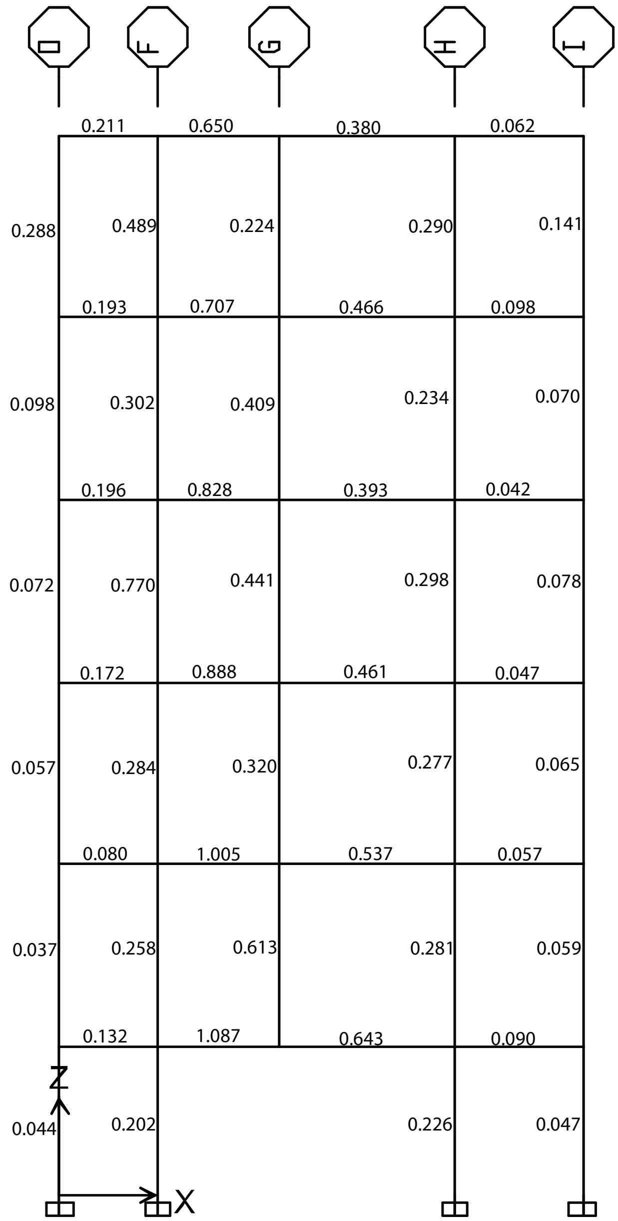


Figure 23: Demand Capacity Ratio's for flexure (DCR) short side of the nine-story building (middle column eliminated)

Figure 24 also shows that none of the computed DCRs after removing the middle column of the nine-story building in the long side, are more than 1 so, it shows that the building resistance against PC is even better than the case where of the removal of the middle column in the short side.

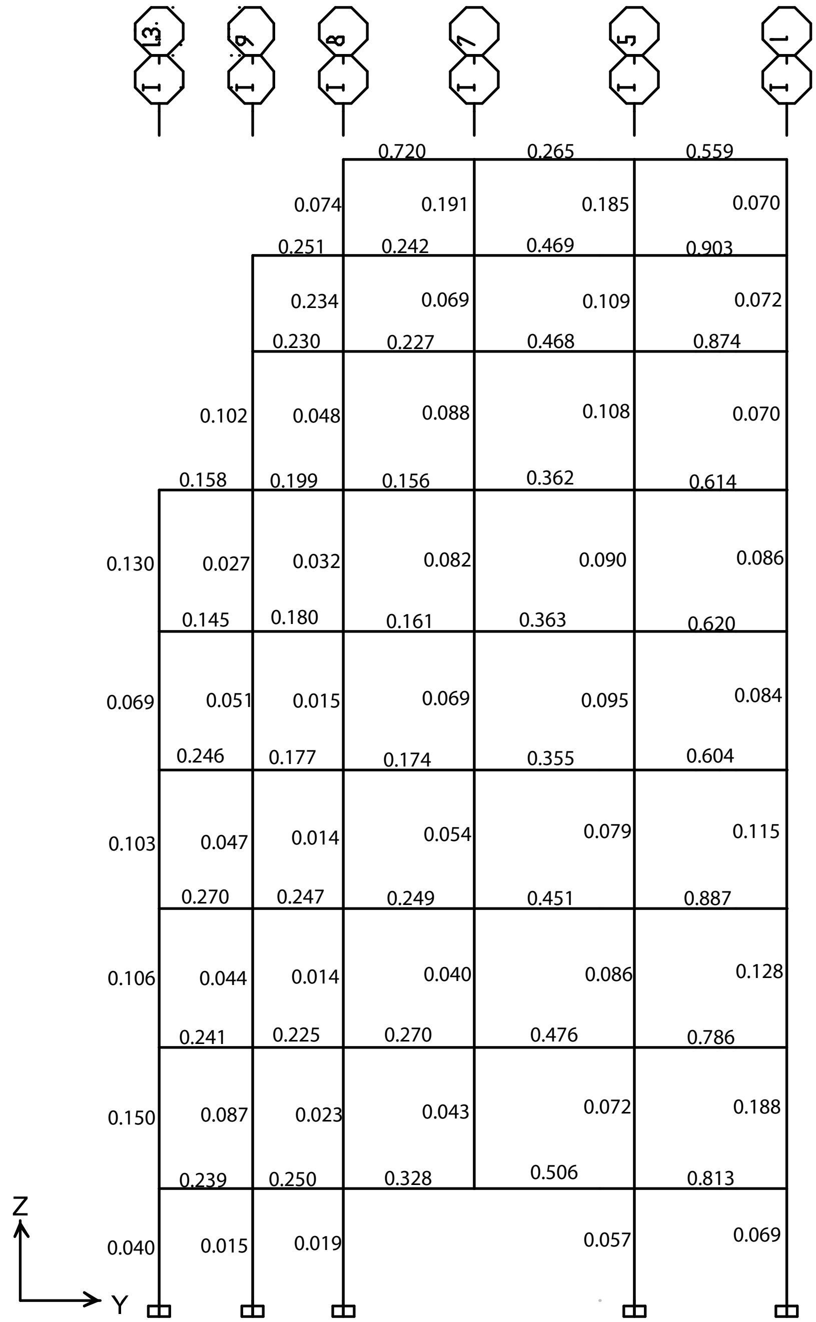


Figure 24: Demand Capacity Ratio's flexure (DCR) for the long side the nine-story building (middle column eliminated)

The consequences of the removal of the corner column in the short side of the nine-story building have been modeled using ETABS-3D and DCR results are shown in the Figure 25.

Results here (Figure 25) also show that all the DCRs are less than 1. This means that the nine-story building resistance against PC is better than the removal of the middle column in the short side of the building where the maximum DCR in moment is equal to 0.790.

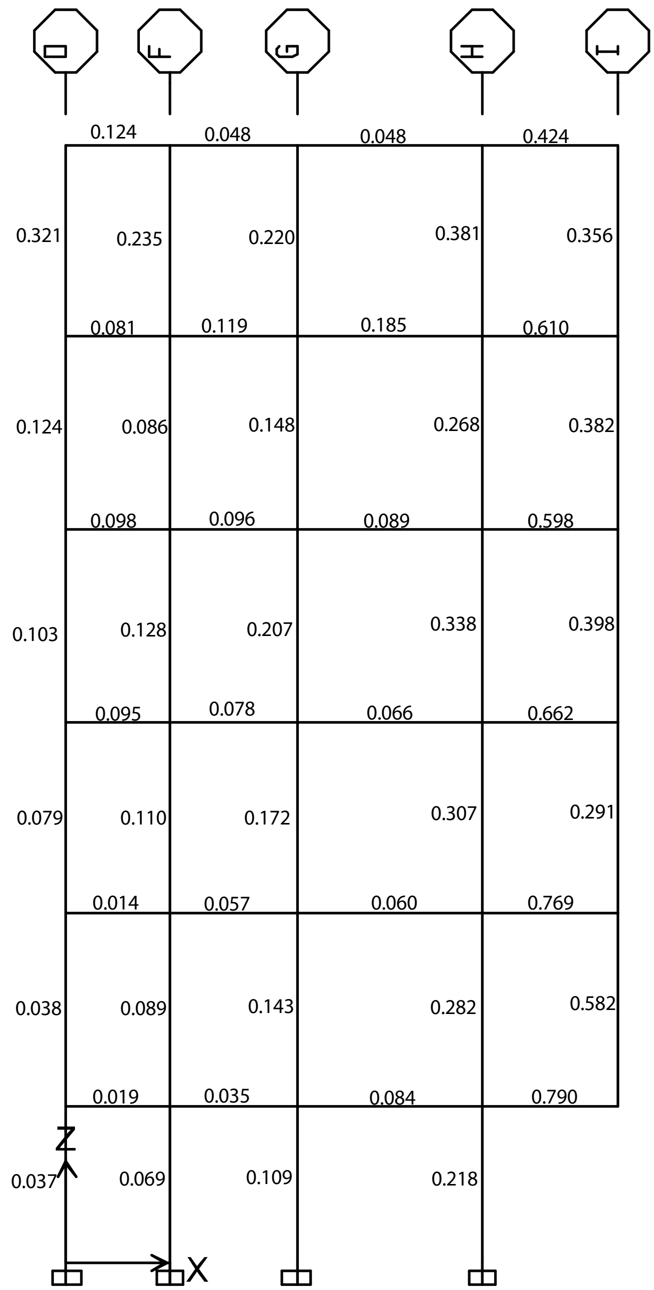


Figure 25: Demand Capacity Ratio's flexure (DCR) for short side of the nine-story building (corner column eliminated)

According to Figure 26, flexures DCRs for the long sides of nine-story building when the corner column is eliminated are lowers than 2 for all the beams and columns. Only for the beams above the eliminated column in first and second floors, DCRs are more than 1 (0.905).

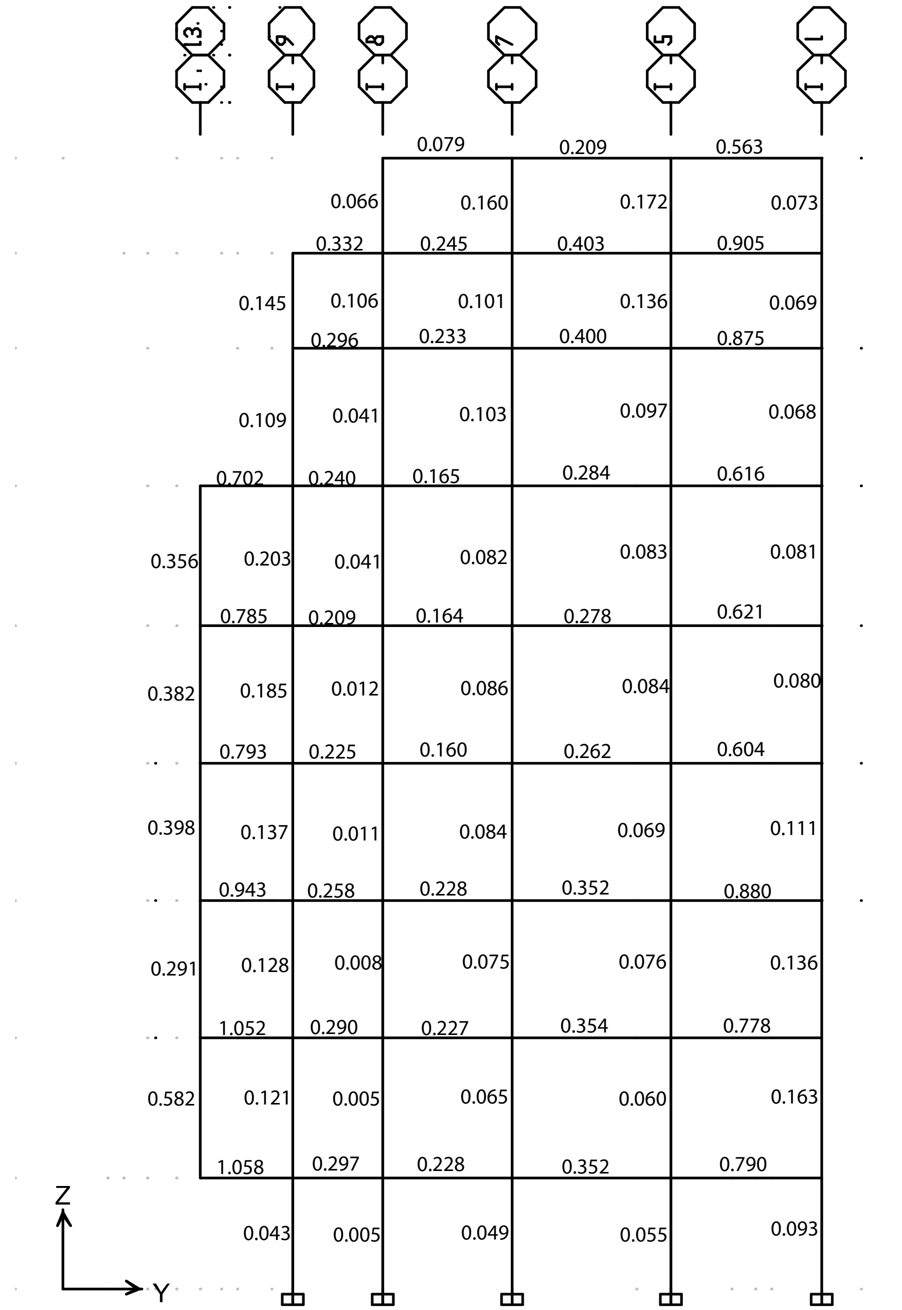


Figure 26: Demand Capacity Ratio’s flexure (DCR) for the long side of the nine-story building (corner column eliminated)

5.1.2 Demand Capacity Ratio for Shear (Nine-story Building)

In this section, Demand Capacity Ratio for Shear are modeled and the analysis for the nine-story building were carried out in the case of middle and corner columns being eliminated in the short and long sides of the building.

According to Figure 27, when the middle column is eliminated, almost all DCRs for the short side of the nine-story building are less than 0.511. It means that the susceptibility of structure to PC is very low.

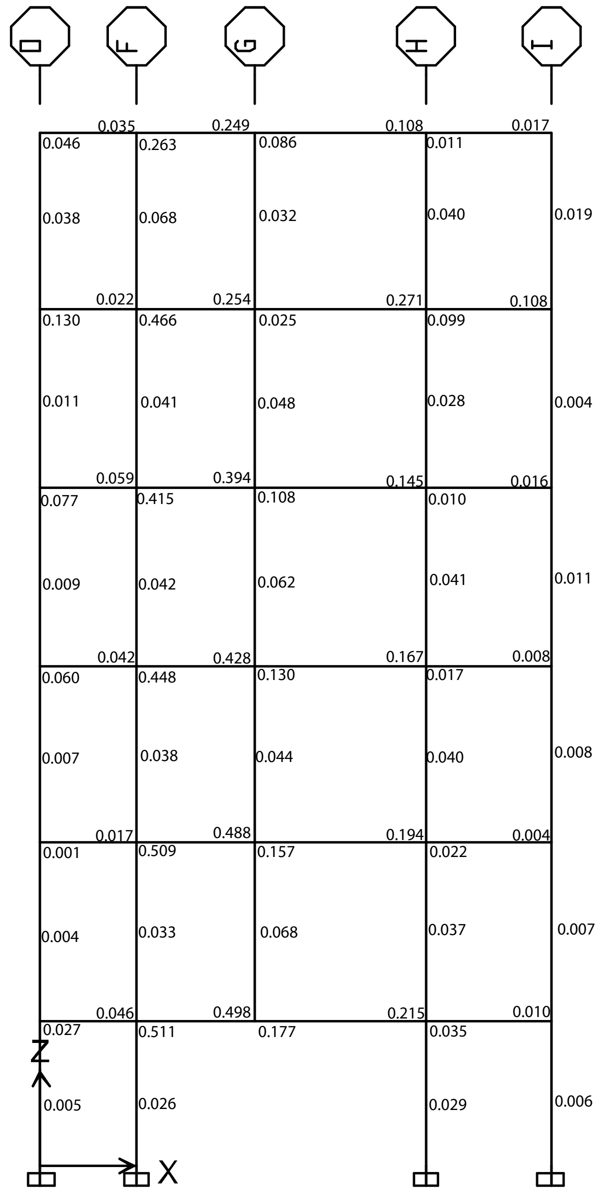


Figure 27: Demand Capacity Ratio's flexure (DCR) for the short side of the nine-story building (middle column eliminated)

According to Figure 28, almost all the DCRs for the long side of the nine-story building when middle column is eliminated are less than 0.6 (maximum DCR_{shear} is equal to 0.566) showing very low potential for the occurrence of PC.

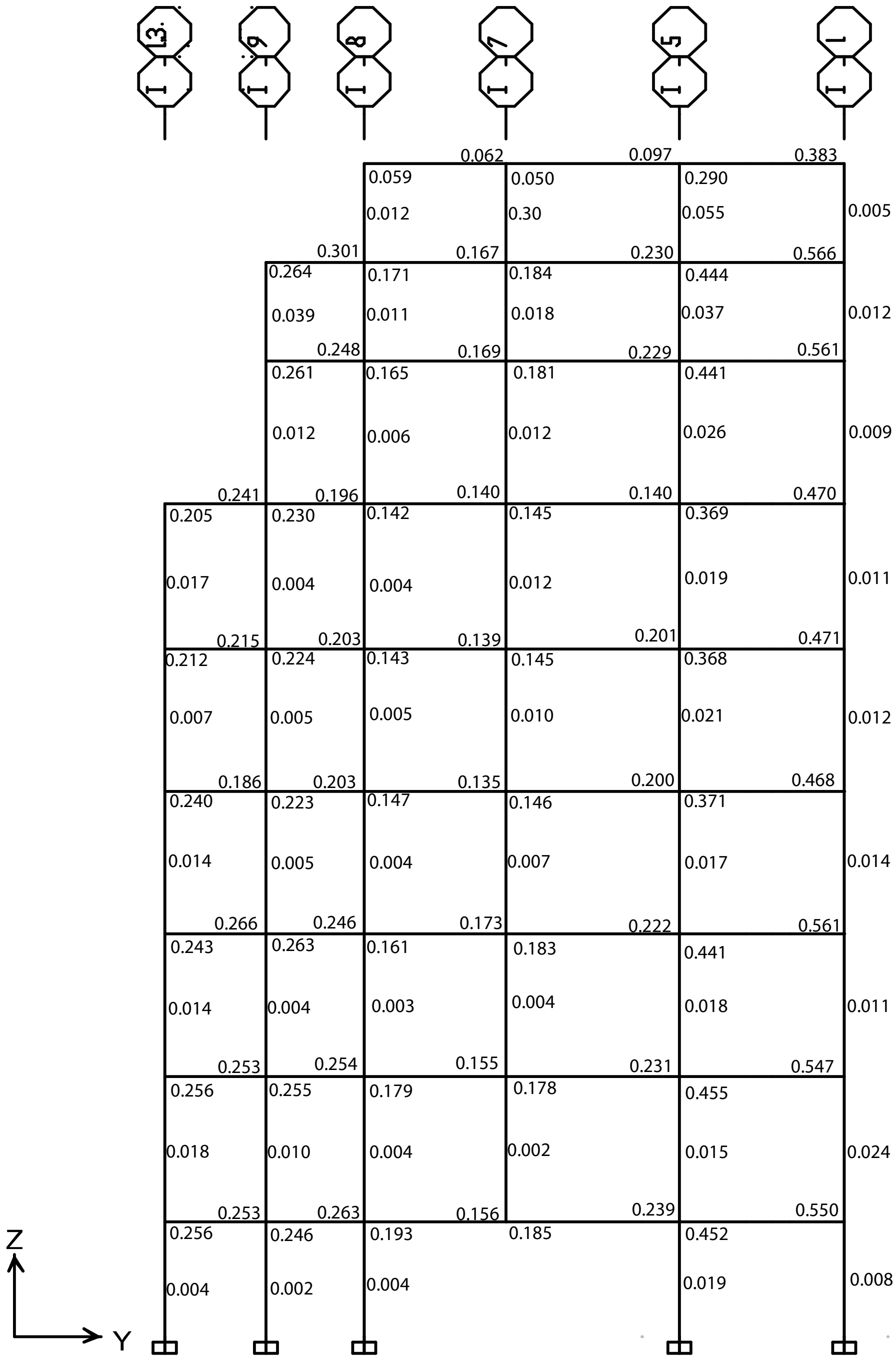


Figure 28: Demand Capacity Ratio's shear (DCR) for the long side of the nine-story building (middle column eliminated)

Figure 29 shows that all DCRs are less than 0.4 and the condition is relatively better than the case of the removal of middle column in the short side. The susceptibility of building against PC is low.

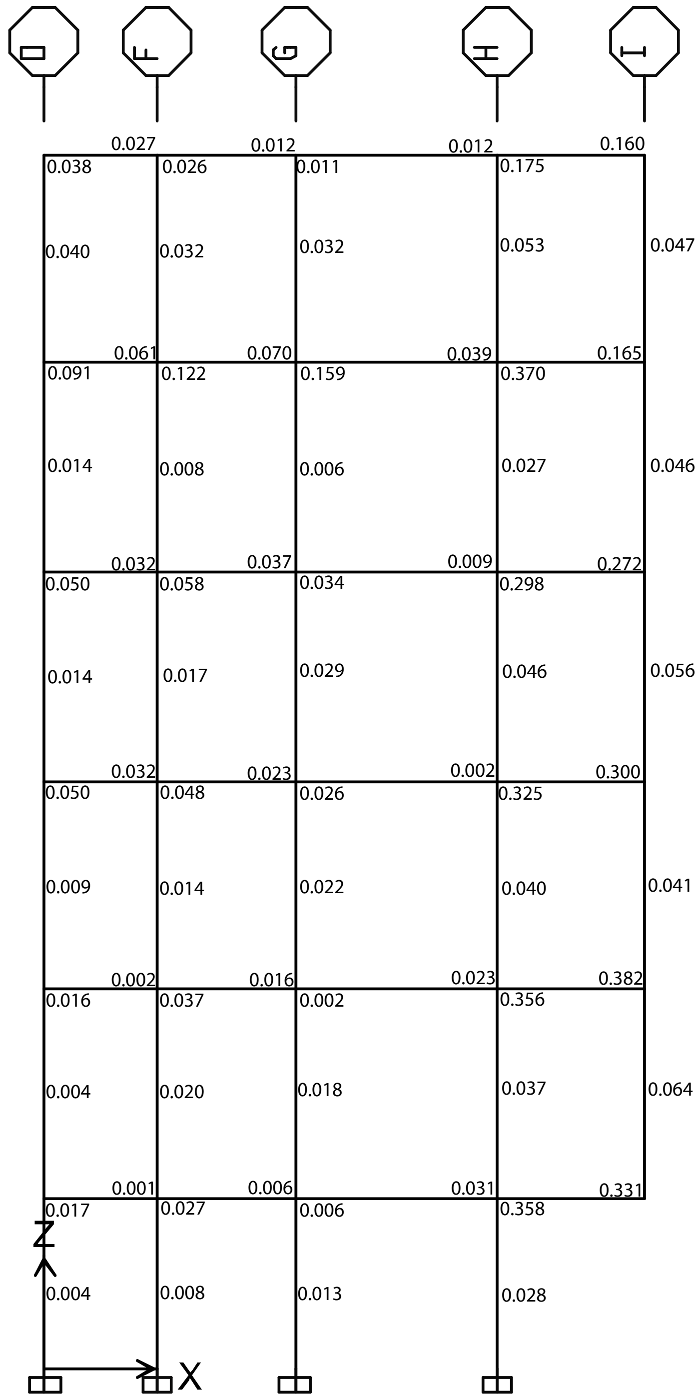


Figure 29: Demand Capacity Ratio's shear (DCR) for the short side of the nine-story building (corner column eliminated)

Figure 30 provides the DCRs for the case where corner column is eliminated. All DCRs for long side of the nine-story building are less than 0.6. This situation is the same as the removal of middle column in the long side of the building and there is no danger of progressive collapse in this case too.

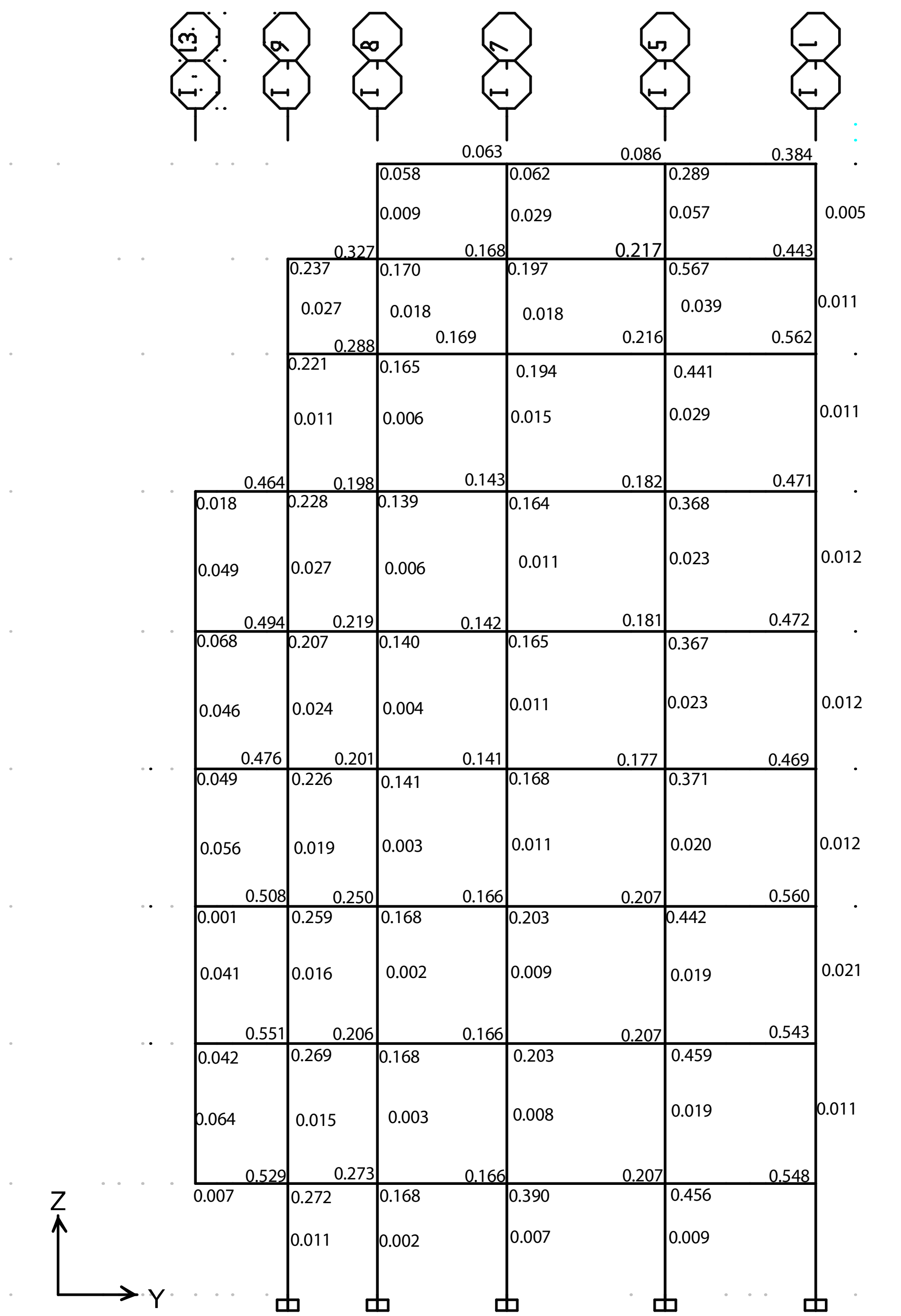


Figure 30: Demand Capacity Ratio's shear (DCR) for long side of the nine-story building (corner column eliminated)

5.1.3 Demand Capacity Ratio for Axial force

Note: Axial force is only being calculated for columns since its equal to zero for beams.

According to Figure 31, DCR of the nine-story building is calculated for axial force after removal of middle column in the short side. In all cases the DCR values are less than 0.41 which shows no possibility of PC for the building.

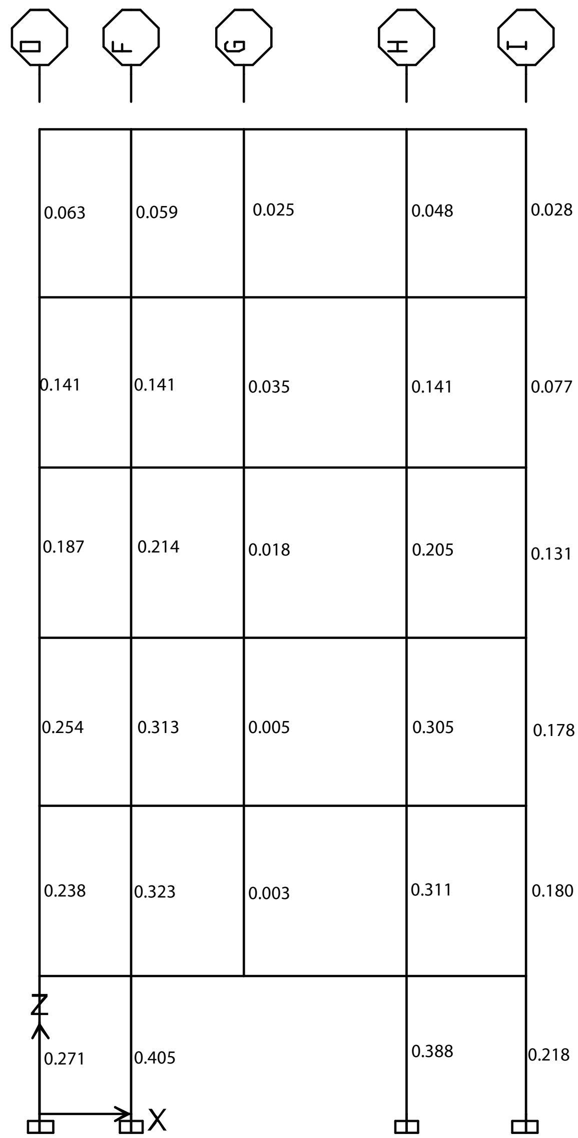


Figure 31: Demand Capacity Ratio's axial force (DCR) for the in short side of the nine-story building (middle column eliminated)

Figure 32 shows that, for all columns, DCR_{axial} of the long side of the nine-story building is less than 0.62, which indicates that the columns have the capacity to bear the existing axial forces and the building could resist the progressive collapse. DCRs of the long side are a little more when compared to the ones of short side for axial load.

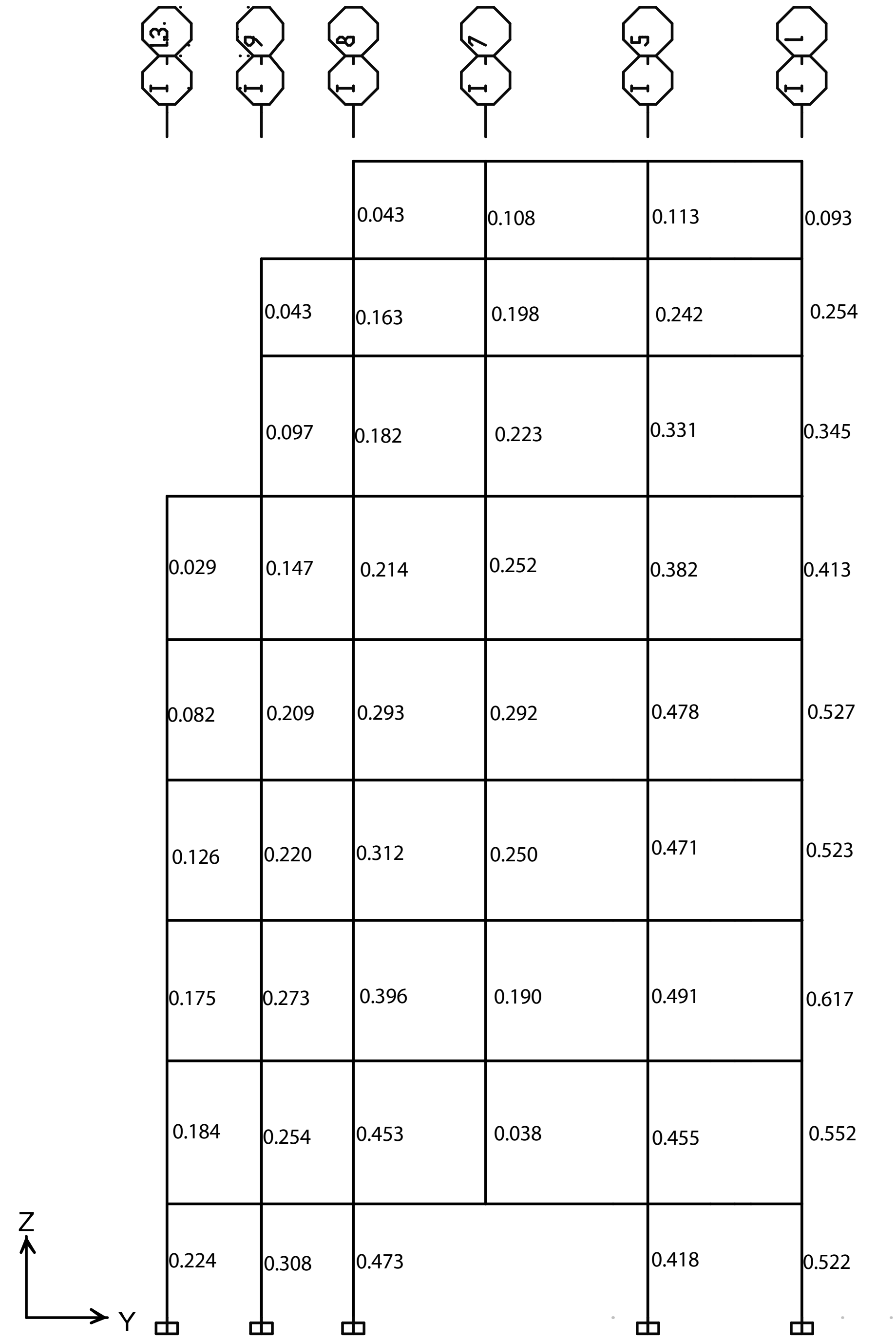


Figure 32: Demand Capacity Ratio's axial force (DCR) for the long side of the nine-story building (middle column eliminated)

When corner column is eliminated in the short side of the building (Figure 33), the DCRs for all the members are less than 0.440 which is well below the limit 2. This means that if the corner column in the short side is removed the whole building will resist progressive collapse.

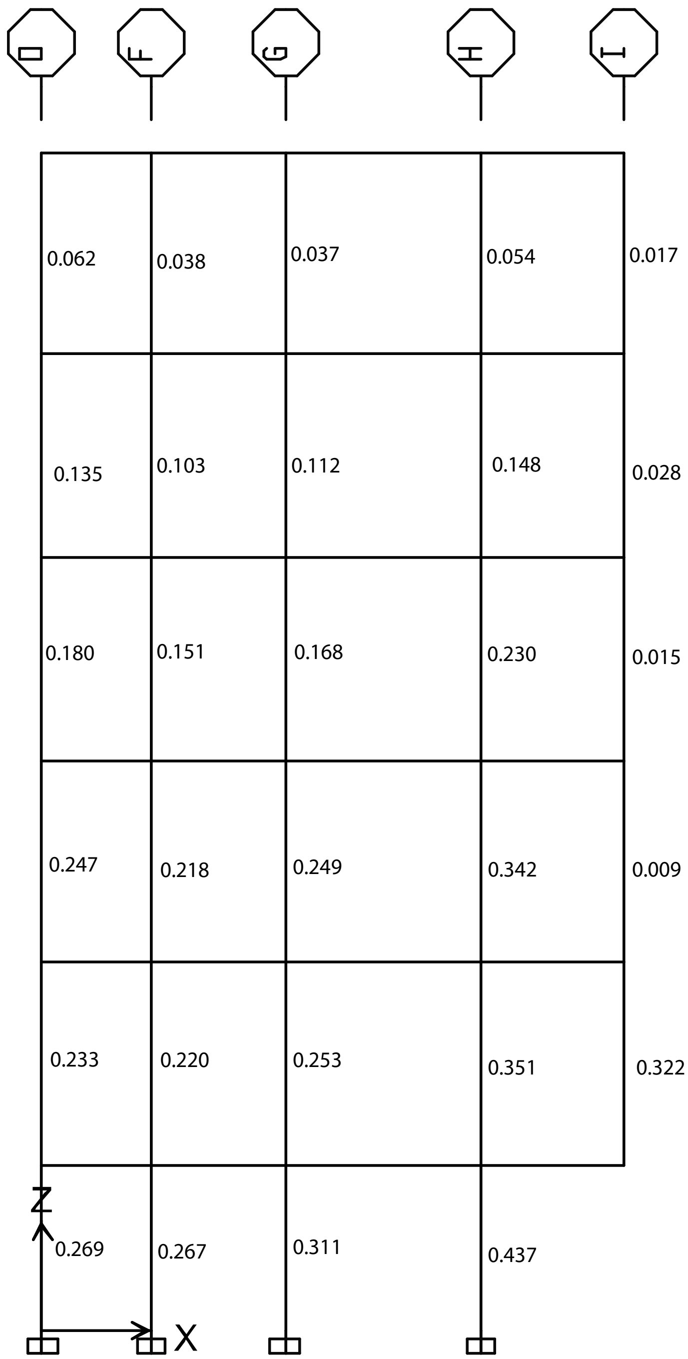


Figure 33: Demand Capacity Ratio's axial force (DCR) for the nine-story building (corner column eliminated)

When corner column is eliminated in the long side of nine-story building, DCRs axial are less than 0.591 for all members which indicates that the building will resist PC (Figure 34).

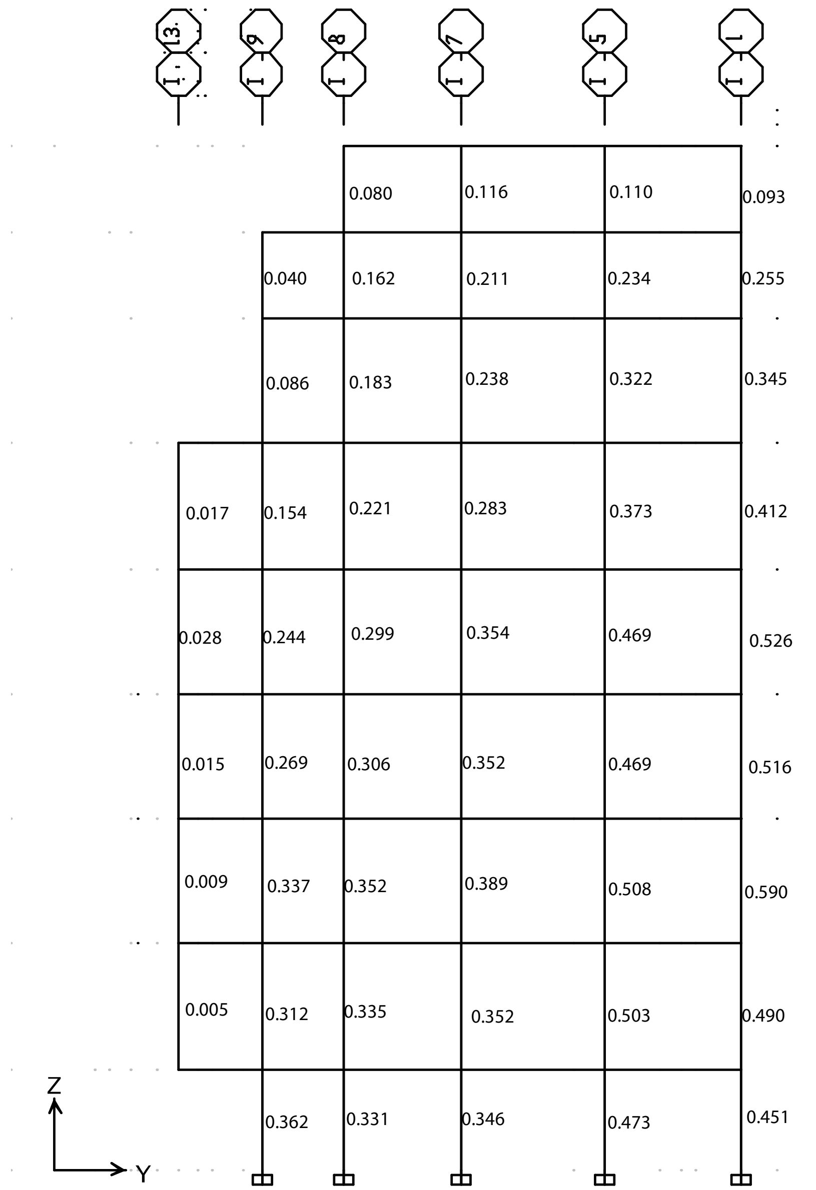


Figure 34: Demand Capacity Ratio's axial force (DCR) for the long side of the nine-story building (corner column eliminated)

5.2 DCR for Six-story Building

The second case study is a six-story gravity frame building with bracing system (simple building frame system). The building is located in Mashhad, north eastern part of Iran. This building was built based on Iranian steel standards which follows American guidelines (AISC-ASD 89). There is no external bracing system in the short side of the building. In this case study, removal of a column in a floor above the ground floor is modeled and analyzed according to GSA guidelines to measure the susceptibility of building in case of accidental damage and hence progressive collapse.

According to GSA guidelines, three different models have been planned for the study based on the removal of columns.

- Case 1: Removal of column in the middle of the short side of a six-story building (see Figure 35).

- Case2: Removal of column in the middle of the long side of a six-story building (see Figure 35).

- Case3: Removal of column in the corner of a six-story building (see Figure 35).

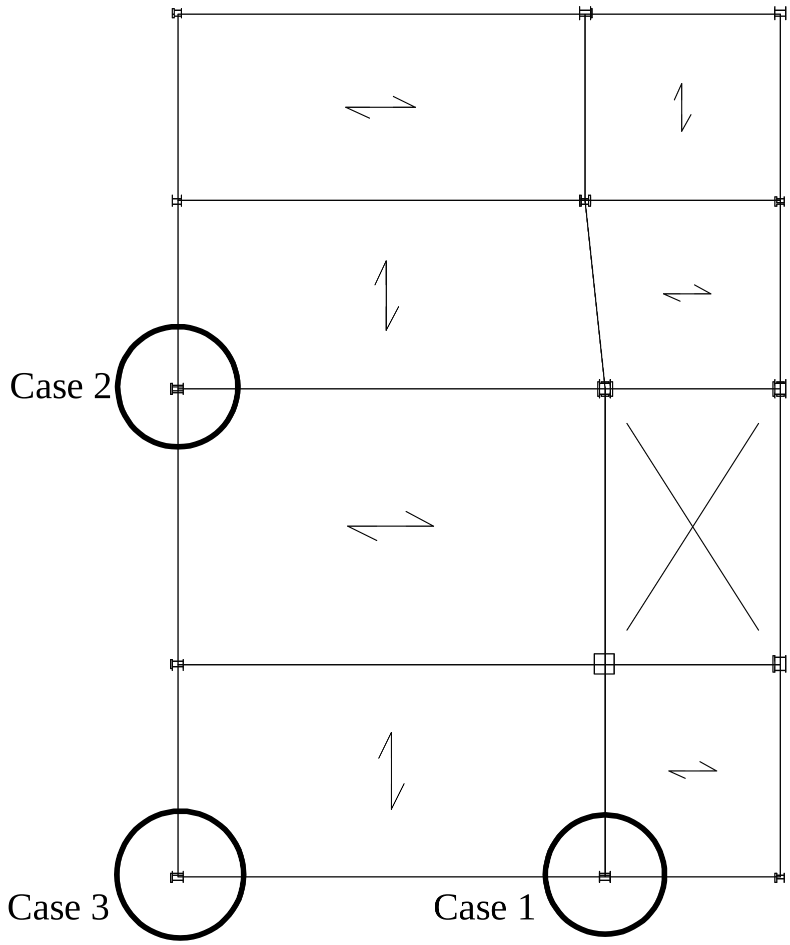


Figure 35: The location of column removing according to GSA guidelines

According to the site plan of the six-story building, the locations of building that is more likely to face with accidents were chosen.

Installation of central heating and air conditioning system in the first floor of the building increases the vulnerability of this floor against explosion. According to this risk, removal of a column from the floor with higher vulnerability is assessed based on GSA guidelines.

5.2.1 Demand Capacity Ratio for Moment in six-story building

According to Figure 36, DCR_{moment} for six-story building when middle column is eliminated is more than 2 (maximum DCR_{moment} in this side is 20.530). This means that the structure has high risk for PC.

It has to be mentioned that f_a is greater than F_e which shows that the structure cannot tolerate additional axial force that may be created as a result of an accidental overload.

Note:

f_a is computed axial stress.

F_e is allowable Euler stress.

When $f_a > F_e$ columns could not resist the existing axial force.

It shows that cooperation of axial force (compression) and bending moment simultaneously caused to columns failed due to considerable axial forces. This noticeable axial force was along with bending.

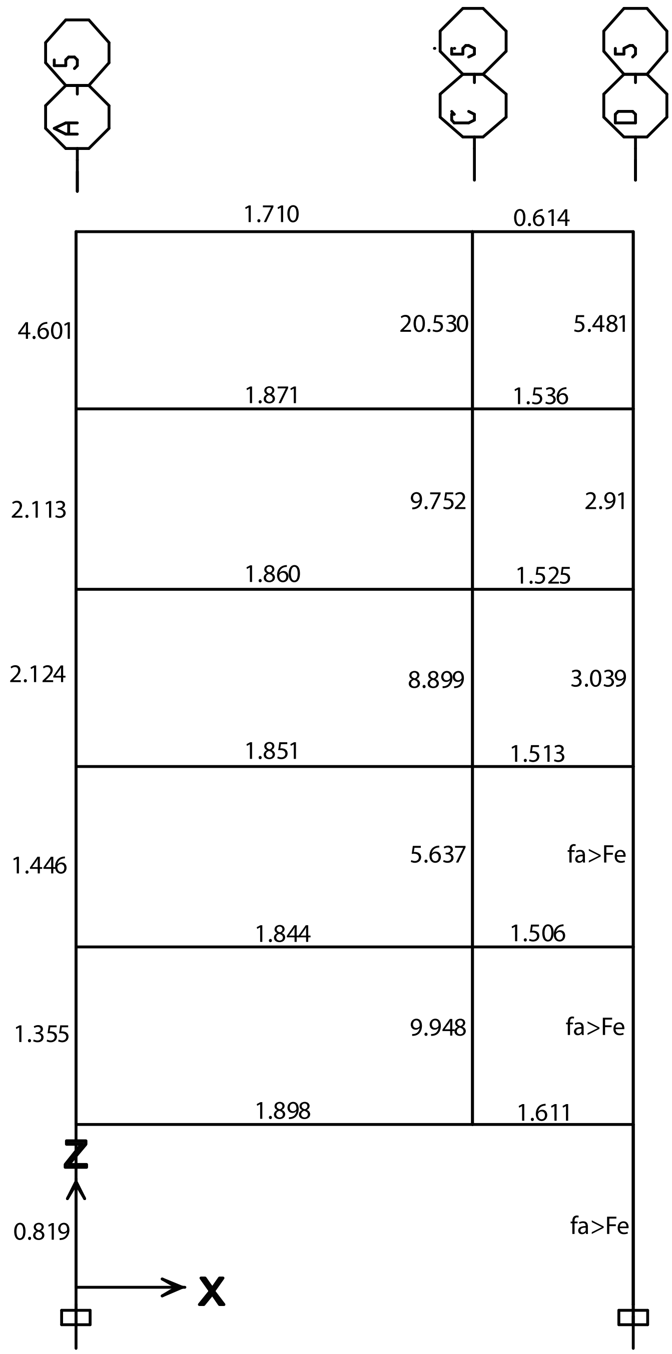


Figure 36: Demand Capacity Ratio's flexure (DCR) for short side of six-story building (middle column eliminated)

Figure 37 shows that for six-story building when middle column is eliminated, DCR_{moment} is less than 2 (1.849) in all elements but the two columns could not resist existing axial force. It means that $f_a > F_e$ but with respect to the short side where there is no bracing, the behavior of this case was better. DCRs for all elements in this side are less than 2 but computed axial stress for two columns are greater than allowable Euler stress yet ($f_a > F_e$). It means that PC will occur.

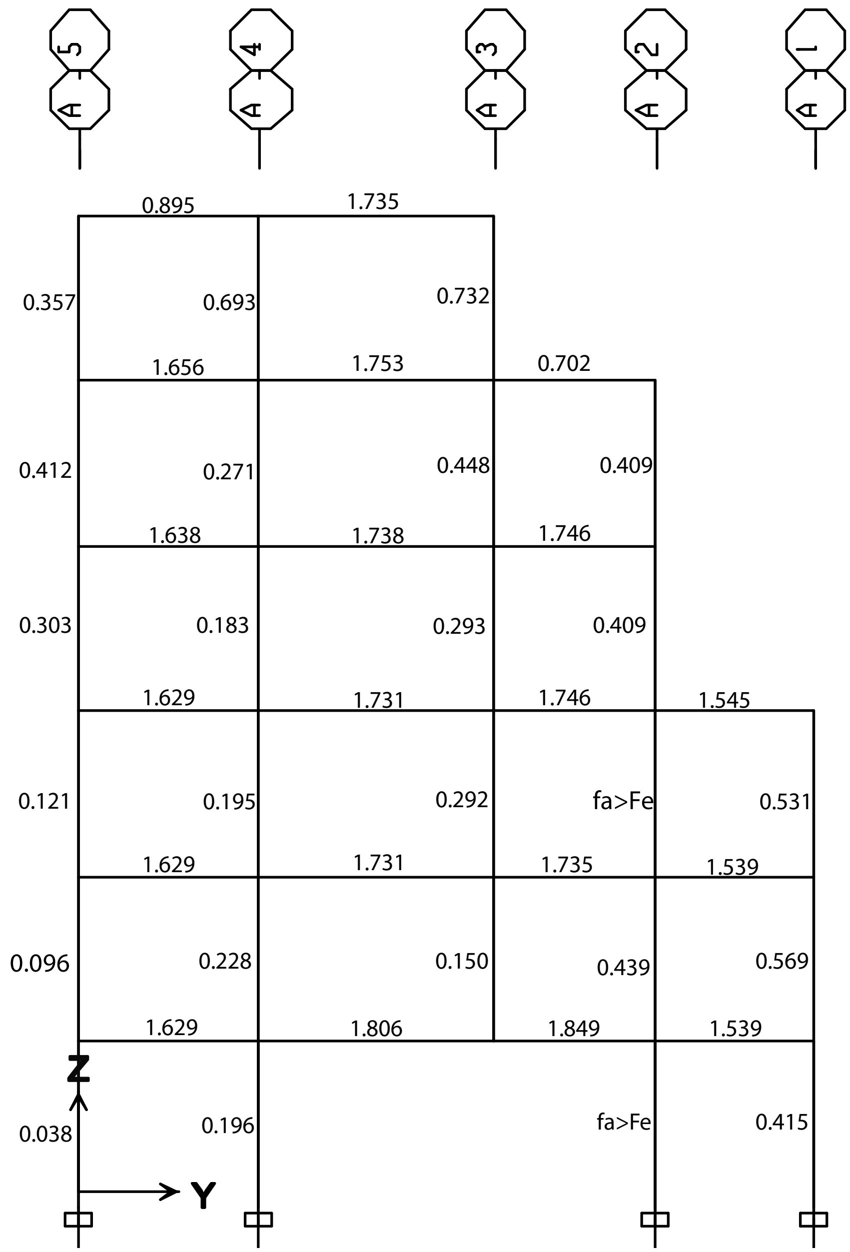


Figure 37: Demand Capacity Ratio's flexure (DCR) for long side of six-story building (middle column eliminated)

According to Figure 38, when corner column is eliminated, DCR's flexure is less than 2 (1.901) showing that progressive collapse may not happen in this case.

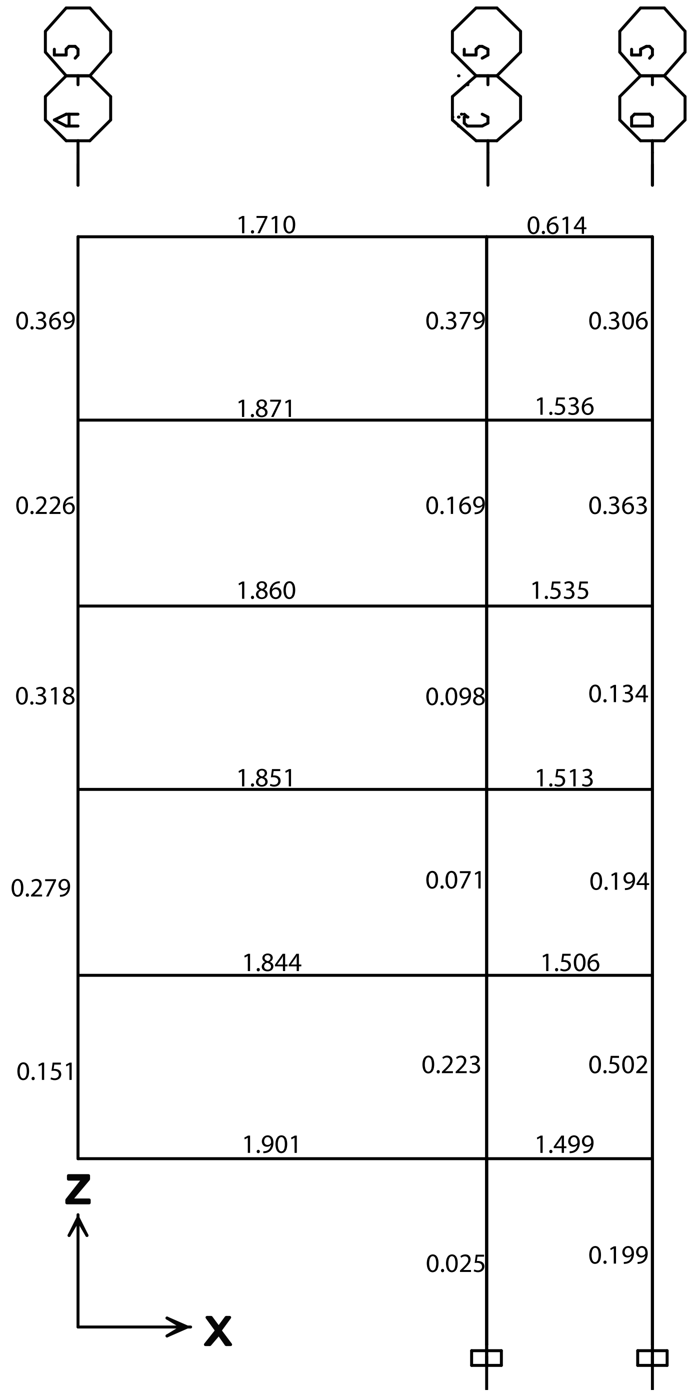


Figure 38: Demand Capacity Ratio's flexure (DCR) for short side of six-story building (corner column eliminated)

Referring to Figure 39, it shows that DCR's flexure for long side of six-story building when Corner column is eliminated has reached to an outstandingly high number of 44.778 which is well above 2. In this case, the structure will be highly likely to subject to PC. Also, F_e (allowable Euler stress) is less than f_a (computed axial stress) in two other columns. After assessing DCR_{moment} in this frame it is realized that this frame has got the worst behavior when compared to the rest of the frames and also it has very high susceptibility of progressive collapse in case of sudden removal of a column.

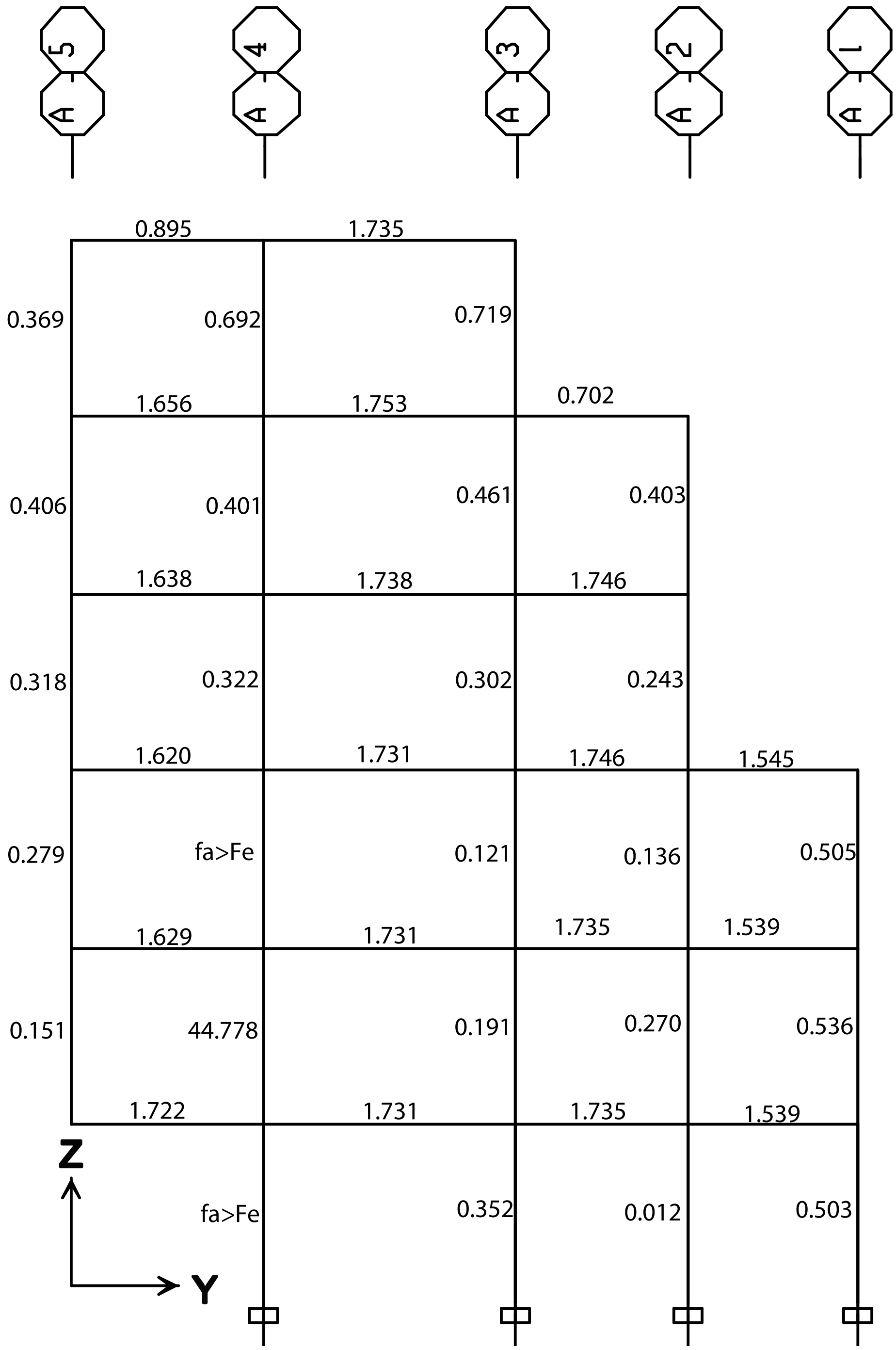


Figure 39: Demand Capacity Ratio's flexure (DCR) for long side of six-story building (corner column eliminated)

5.2.2 Demand Capacity Ratio for Shear

Calculation of DCR_{shear}, after removal of middle column in short side of six-story building, shows that progressive collapse will not occur in this case (Figure 40) but since three columns have got higher computed axial stress than the allowable Euler stress, then these columns could not bear the actual axial forces. Consequently, the progressive collapse will happen in case of a removal of the column.

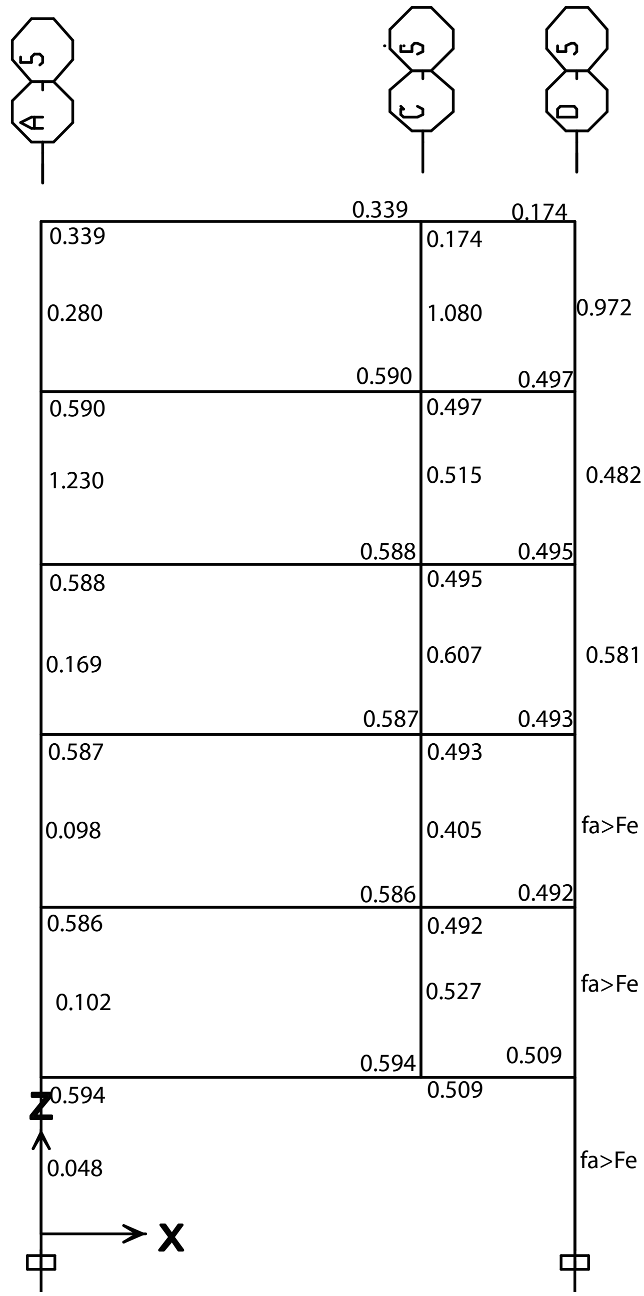


Figure 40: Demand Capacity Ratio's shear (DCR) for short side of six-story building (middle column eliminated)

According to Figure 41, DCRshear for long side of six-story building when middle column is eliminated, is less than 1.330 so in this case, the building has enough resistance against PC, but because of the two columns having higher computed axial stress than the allowable Euler stress ($f_a > F_e$), then they could not bear the existing axial forces and progressive collapse is highly likely to happen in this case too.

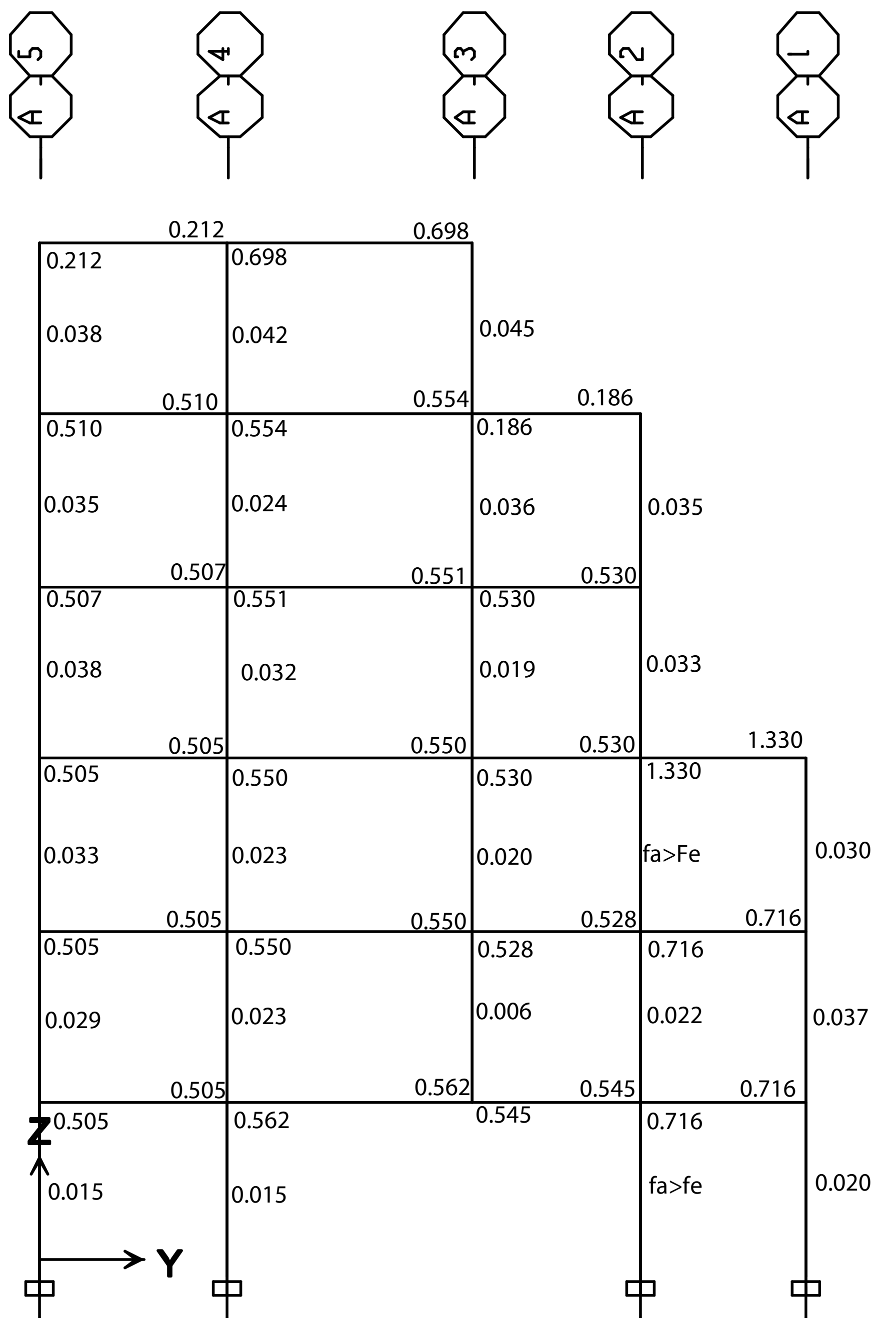


Figure 41: Demand Capacity Ratio's shear (DCR) for long side of six-story building (middle column eliminated)

DCRshear for short side of six-story building, when corner column eliminated (Figure 42), is less than 0.6 (0.595) for all members so this frame could endure the PC.

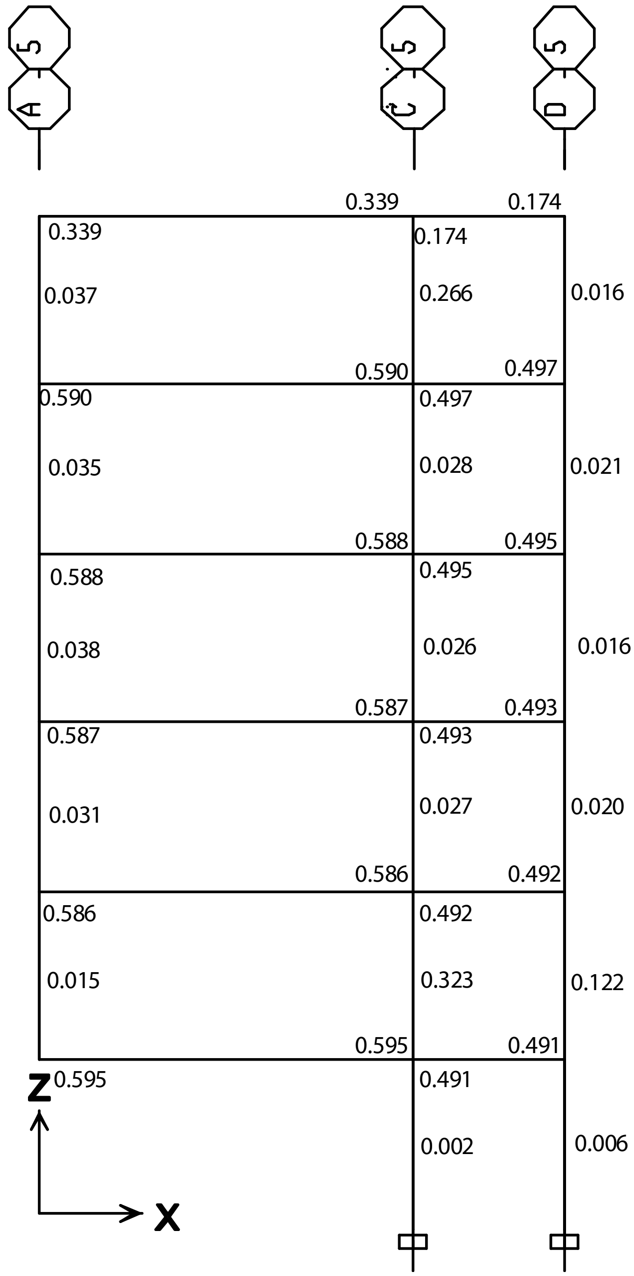


Figure 42: Demand Capacity Ratio's shear (DCR) for short side of six-story building (corner column eliminated)

DCR shear for long side of six-story building (Figure 43), when corner column eliminated, is less than 1.330, but because of the two columns having higher computed axial stress than the allowable Euler stress ($f_a > F_e$), then they will not be able to bear the existing axial forces and progressive collapse is very likely to occur.

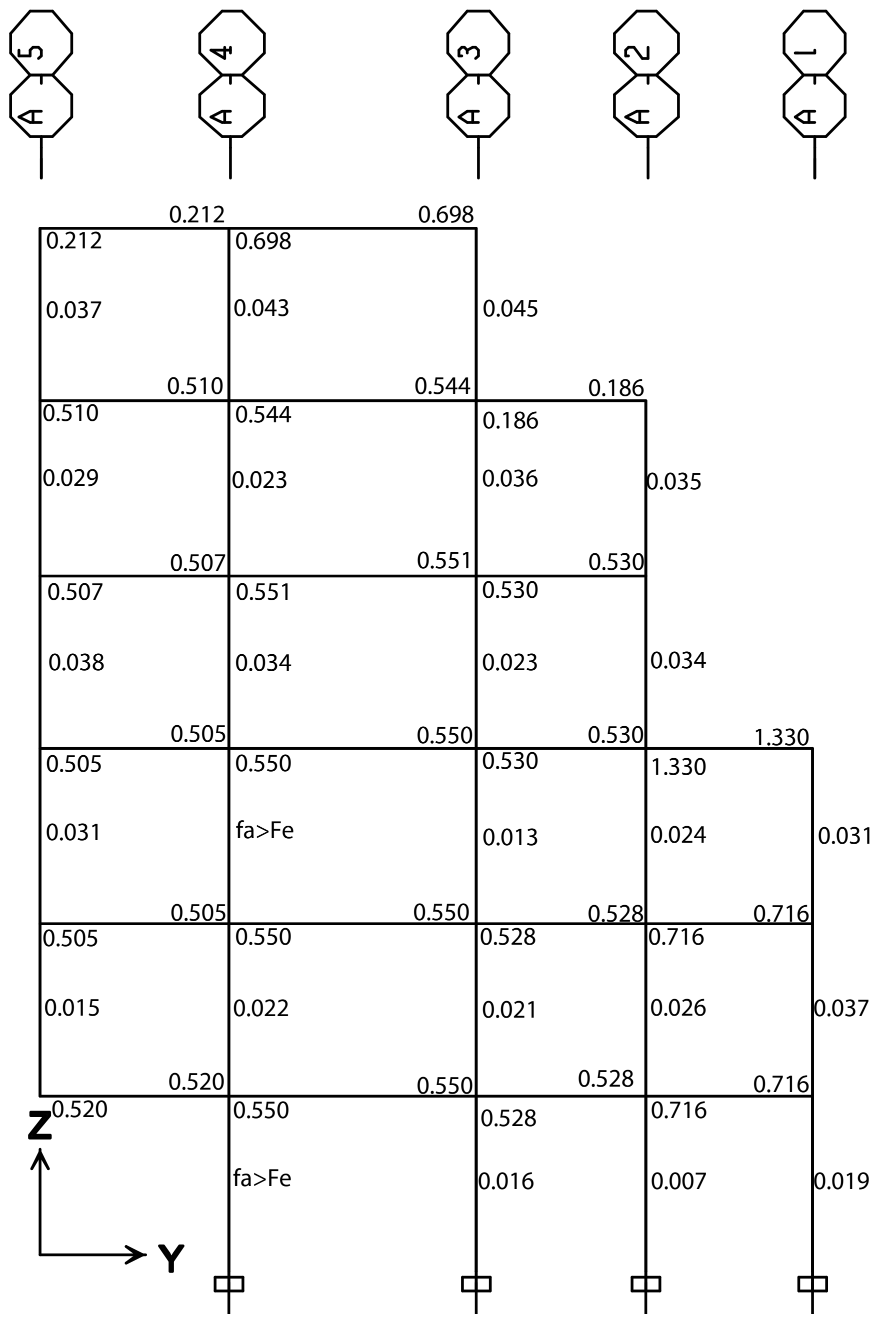


Figure 43: Demand Capacity Ratio's shear (DCR) for long side of six-story building (corner column eliminated)

5.2.3 Demand Capacity Ratio for Axial force

According to Figure 44, DCR's axial force for short side of six-story building when middle column eliminated, is less than 1.146. However, three columns have higher computed axial stress than the allowable Euler stress ($f_a > F_e$), so they could not bear the existing axial forces and the progressive collapse will happen.

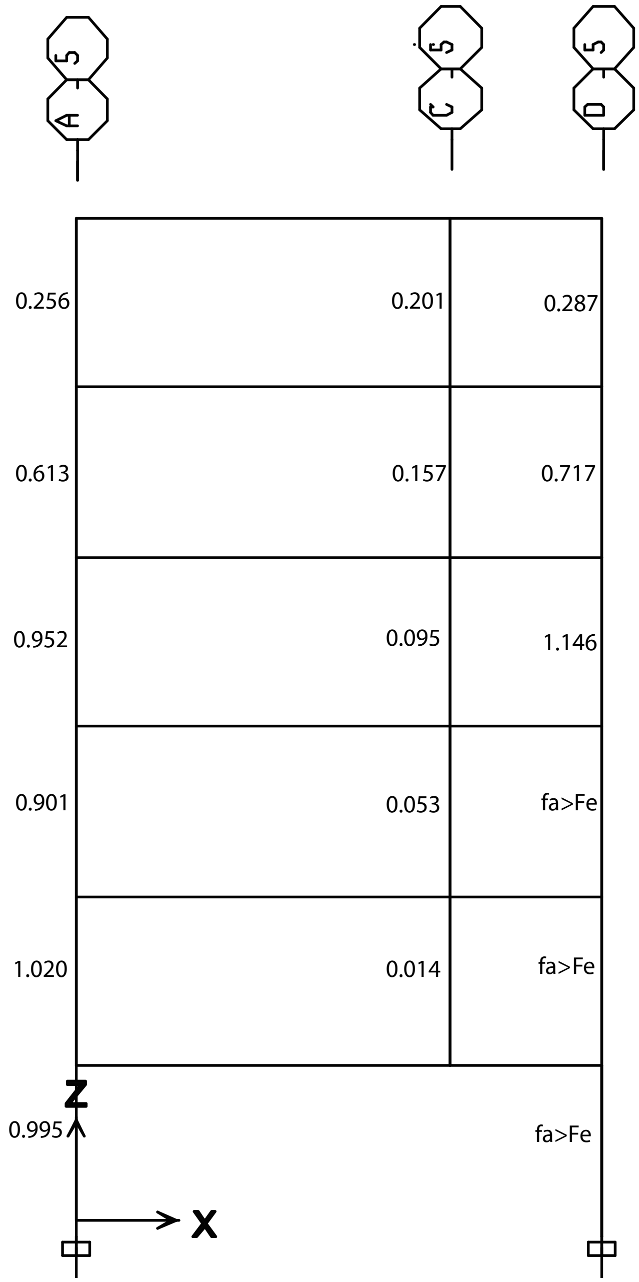


Figure 44: Demand Capacity Ratio's axial force (DCR) for short side of six-story building (middle column eliminated)

According to Figure 45, DCR's axial force for long side of the six-story building when the middle column eliminated, is greater than 2 (2.046). In addition, the computed axial stress is more than allowable Euler stress ($f_a > F_e$), hence, the PC will occur.

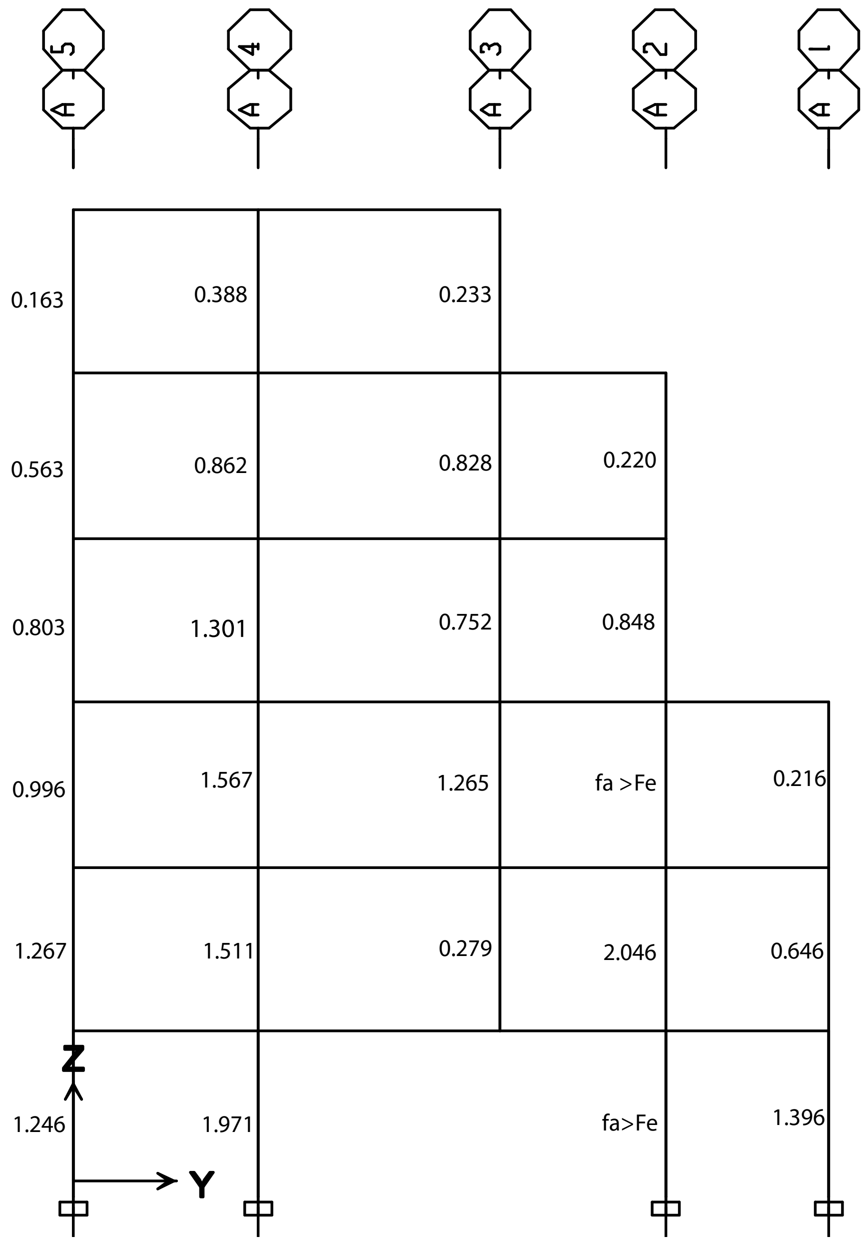


Figure 45: Demand Capacity Ratio's axial force (DCR) for long side of six-story building (middle column eliminated)

According to Figure 46, Demand Capacity Ratios for axial forces (DCR) in short side of six-story building when corner column eliminated, is less than 2 which shows that the susceptibility of structure for occurrence of progressive collapse is low.

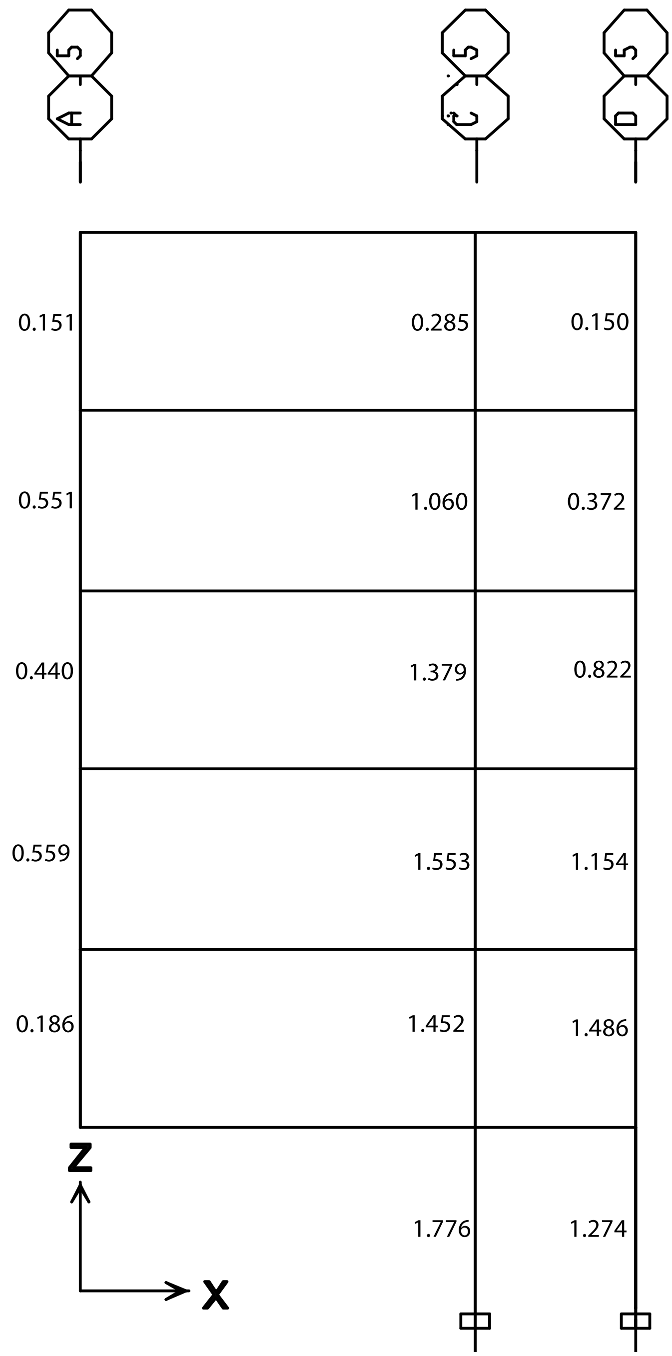


Figure 46: Demand Capacity Ratio's axial force (DCR) for short side of six-story building (corner column eliminated)

Figure 47 shows the Demand Capacity Ratio's for axial forces (DCR) in long side of six-story building when corner column is eliminated. DCR_{axial} has passed the limit 2 (2.592). Furthermore, in this frame, the computed axial stress is more than the allowable Euler stress ($f_a > F_e$) too. Therefore, the columns could not bear the existing axial forces and progressive collapse will happen.

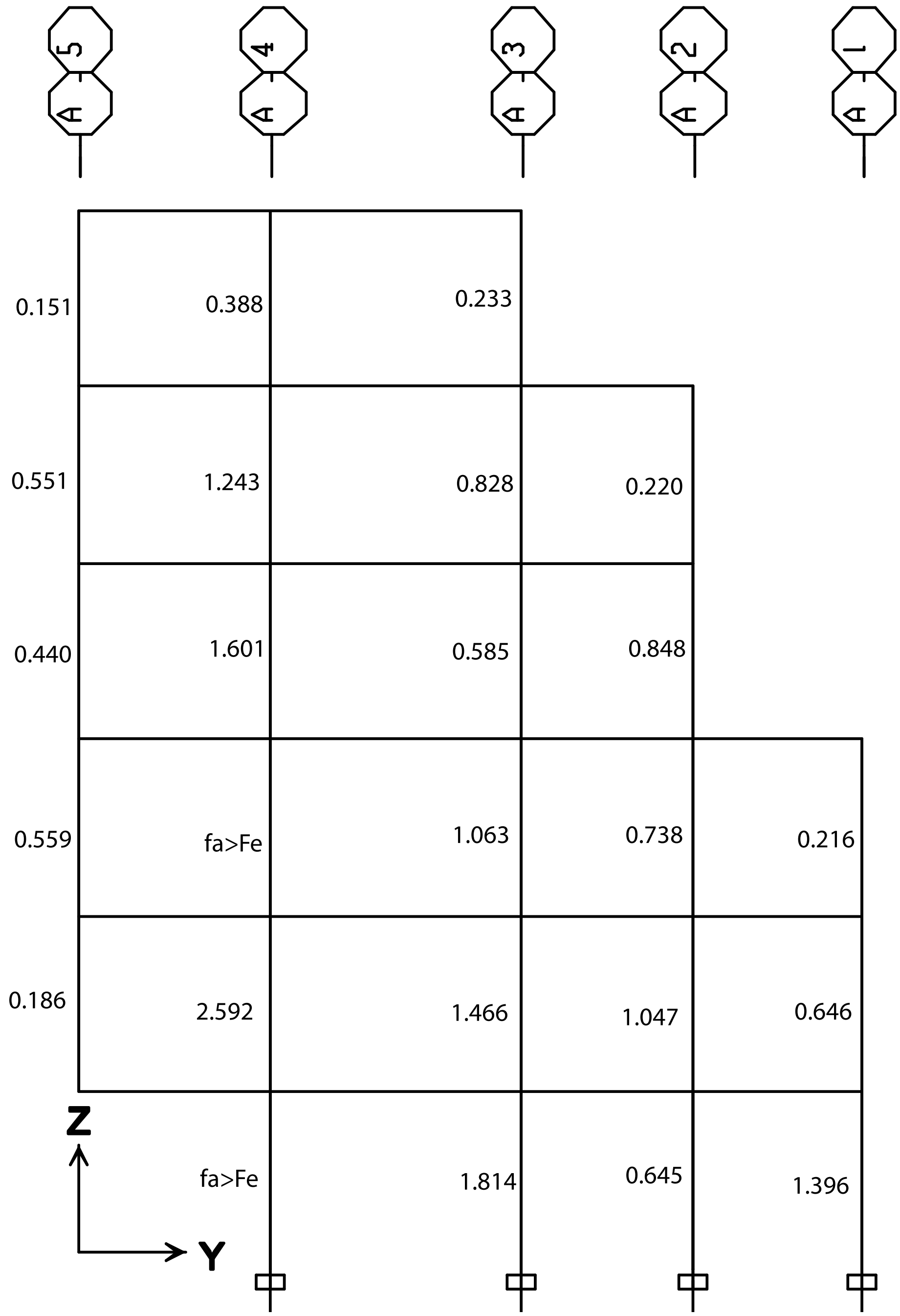


Figure 47: Demand Capacity Ratio's axial force (DCR) for long side of six-story building (corner column eliminated)

After assessing and analyzing the six-story building it was concluded that the structure will face progressive collapse in case of a sudden removal of columns thus the structure has been rehabilitated as follow:

5.3 DCR for Six-story Building after Rehabilitation

The steel cross sections for the short side (exterior frame, beside the road) before rehabilitation are shown in Figure 48.

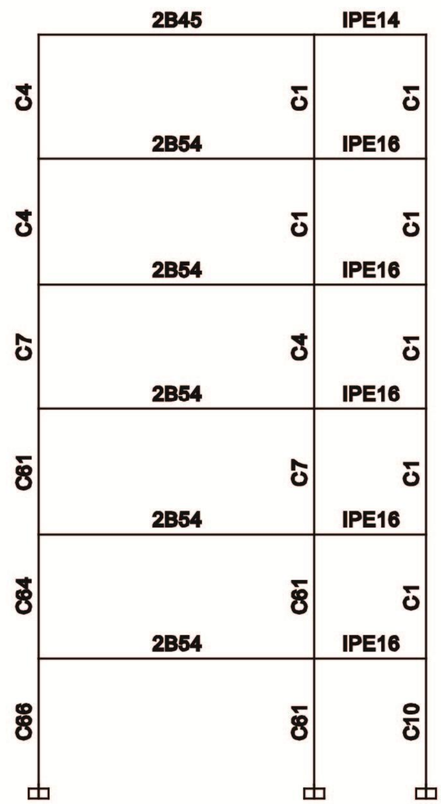


Figure 48: Short side (X direction) elevation of the six-story building before rehabilitation

One of the methods of rehabilitation of the structures (slender column) against progressive collapse is to add braces in the frame as detailed below:

Exterior frame in short side of the six-story building do not have lateral bracing system (it has gravity frame). Hence, by adding X braces in the first floor and diagonal braces in other floors, the exterior frame will be rehabilitated. In this way abnormal forces in some of the members get transferred to other members and this action will remove the slender column issue too. The steel sections used are double channel as detailed in Figure 49.

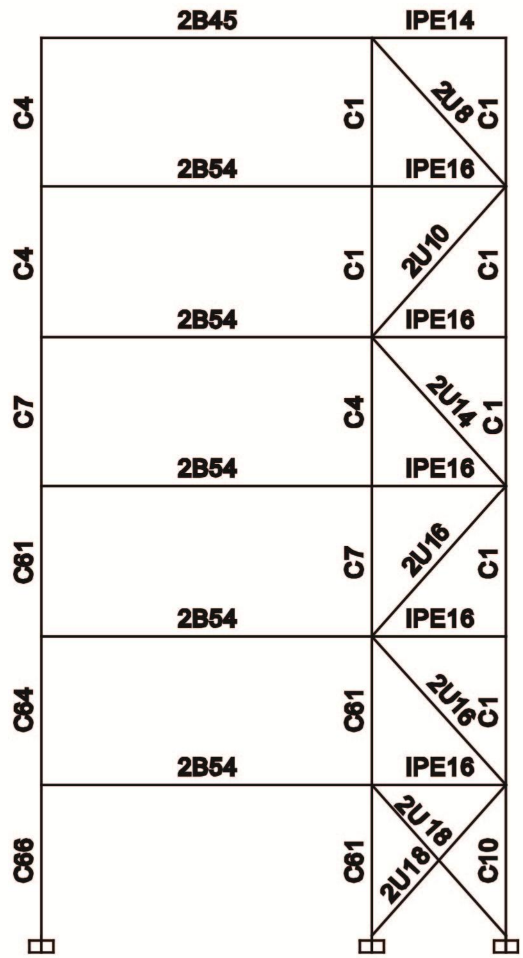


Figure 49: Short side (X direction) elevation of the six-story building after rehabilitation

As illustrated in Figure 50, in long side of the first floor, diagonal bracing system is used. This creates a high PC potential.

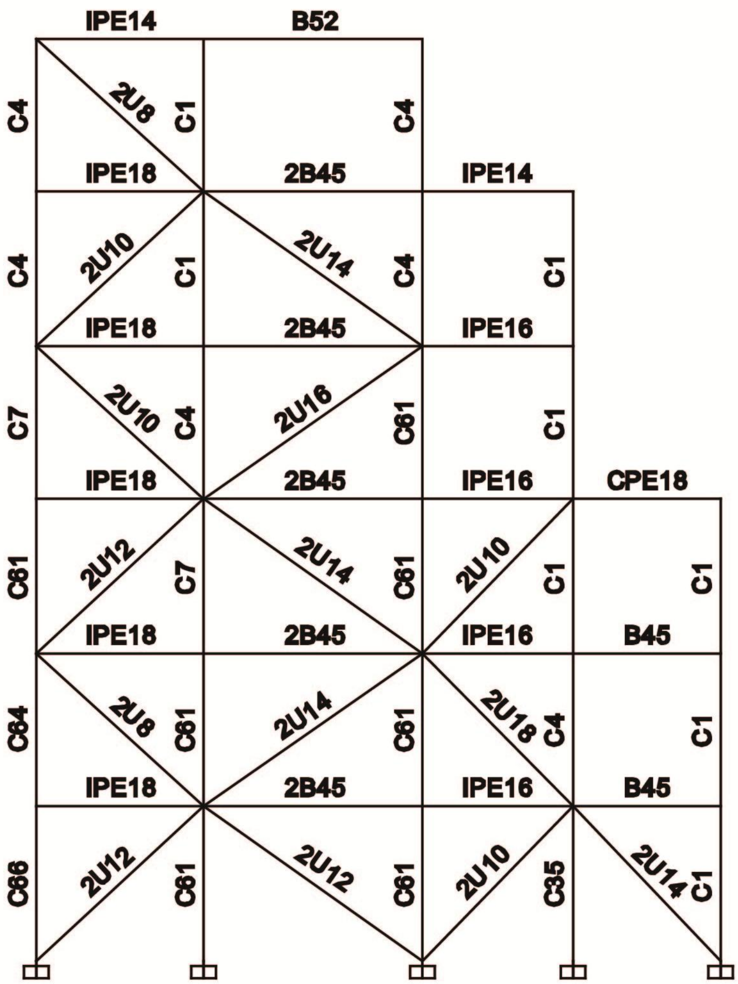


Figure 50: Long side (Y direction) elevation of the six-story building before rehabilitation

It can be observed from Figure 51 that diagonal braces in the first floor are reinforced by introducing additional diagonal braces to each of the existing diagonal braces and therefore forming cross-bracing system. The steel cross sections used are double channel as shown in Figure 51.

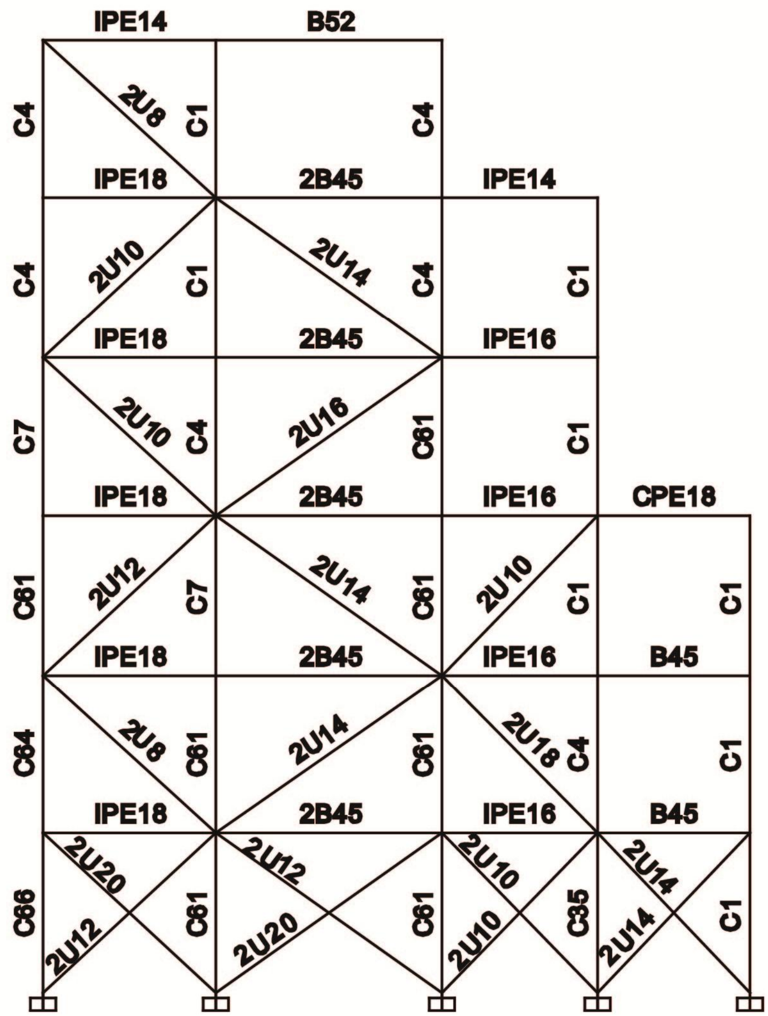


Figure 51: Long side (Y direction) elevation of the six-story building after rehabilitation

The DCRs after rehabilitation of six-story building are presented in section 5.3.1.

5.3.1 Demand Capacity Ratio for Moment after Rehabilitation of the Six-story Building

Figure 52 shows that, DCR for short side of six-story building, when middle column eliminated, is less than 2. In other word, maximum DCR_{moment} in this side is 1.898 for all elements. Then it is concluded that the structure has got low potentiality in case of occurring the progressive collapse. It has to mentioned that by rehabilitating the frame through adding braces, no effect will occur to beams DCRs but DCRs for columns are a little lower leading to lower computed axial stress (f_a is lower than F_e). This means that the progressive collapse is prevented in this model.

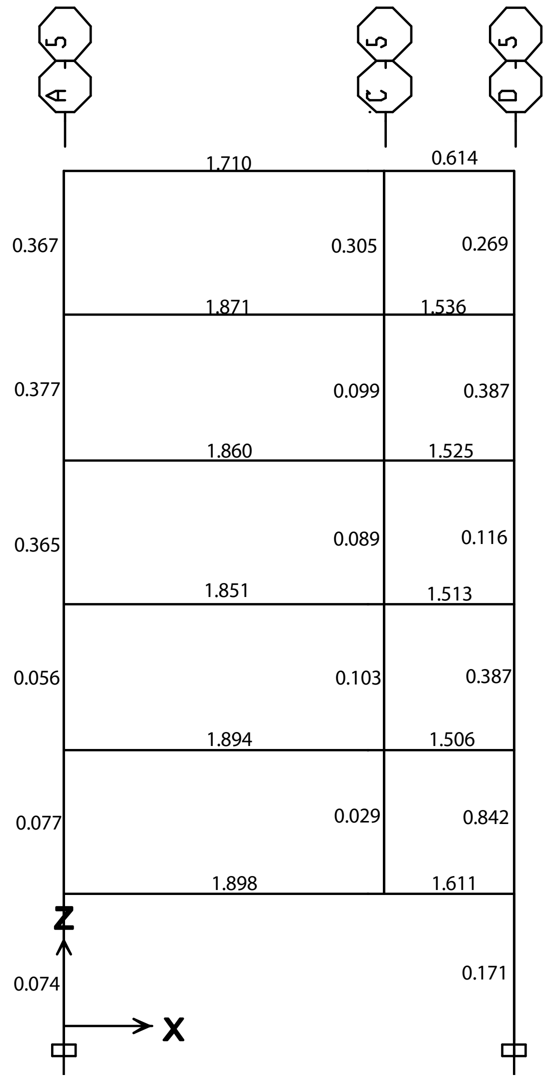


Figure 52: Demand Capacity Ratio's flexure (DCR) for six- story building (middle column eliminated)

In Figure 53, DCR_{moment} is less than 2 (maximum DCR_{moment} in this side is 1.849) for all elements which shows that the frame has got ability to be stable against the progressive collapse in case of removing the middle column from the long side of building.

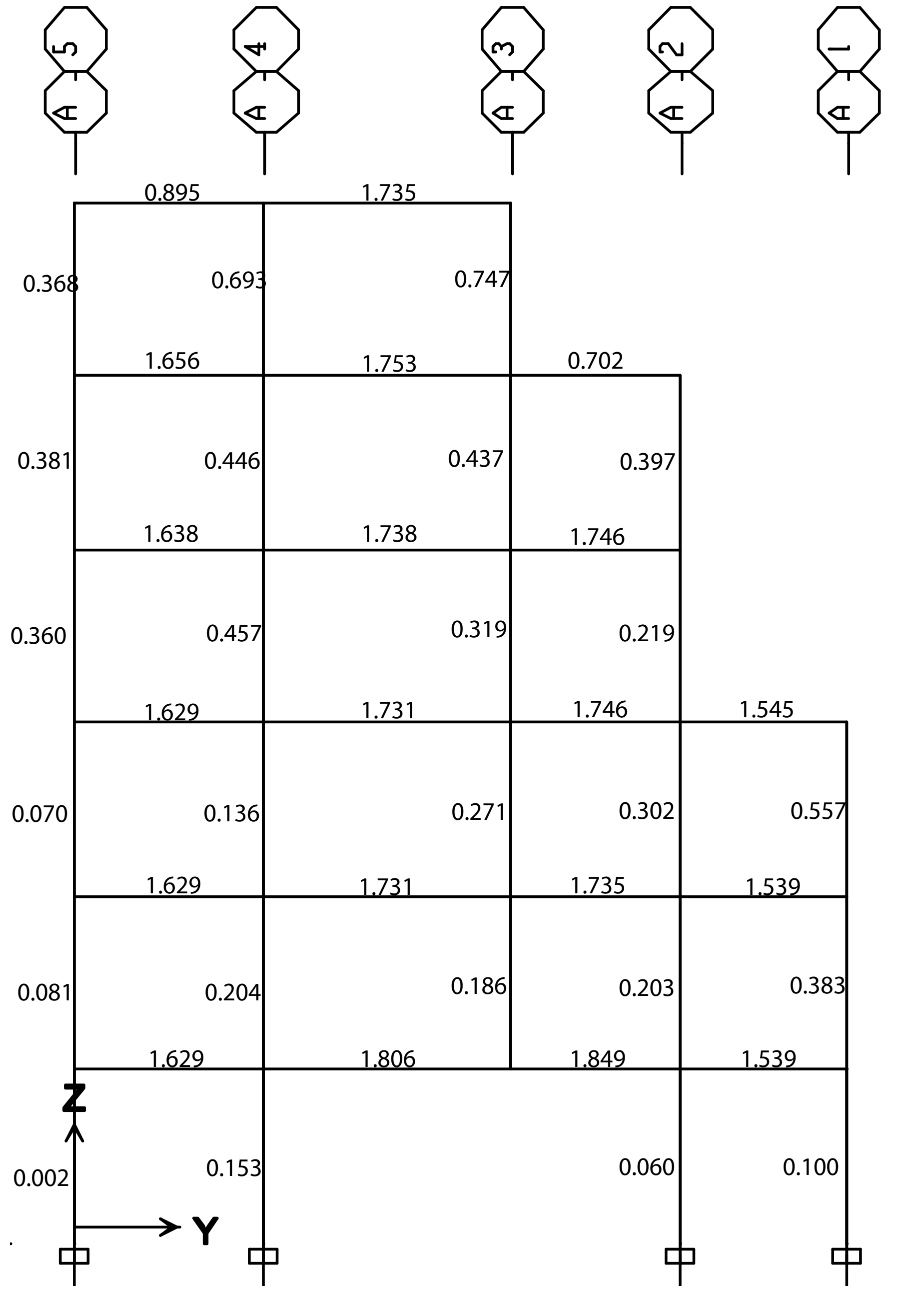


Figure 53: Demand Capacity Ratio's flexure (DCR) for the long side of six-story building (middle column eliminated)

It can be observed from Figure 54 that DCR_{moment} for short side of six-story building when corner column is eliminated is less than 2 for all members. In other words, maximum DCR_{moment} in this side is about 1.90 which shows that the frame is guarded against the progressive collapse in case of removal of the corner column from the short side.

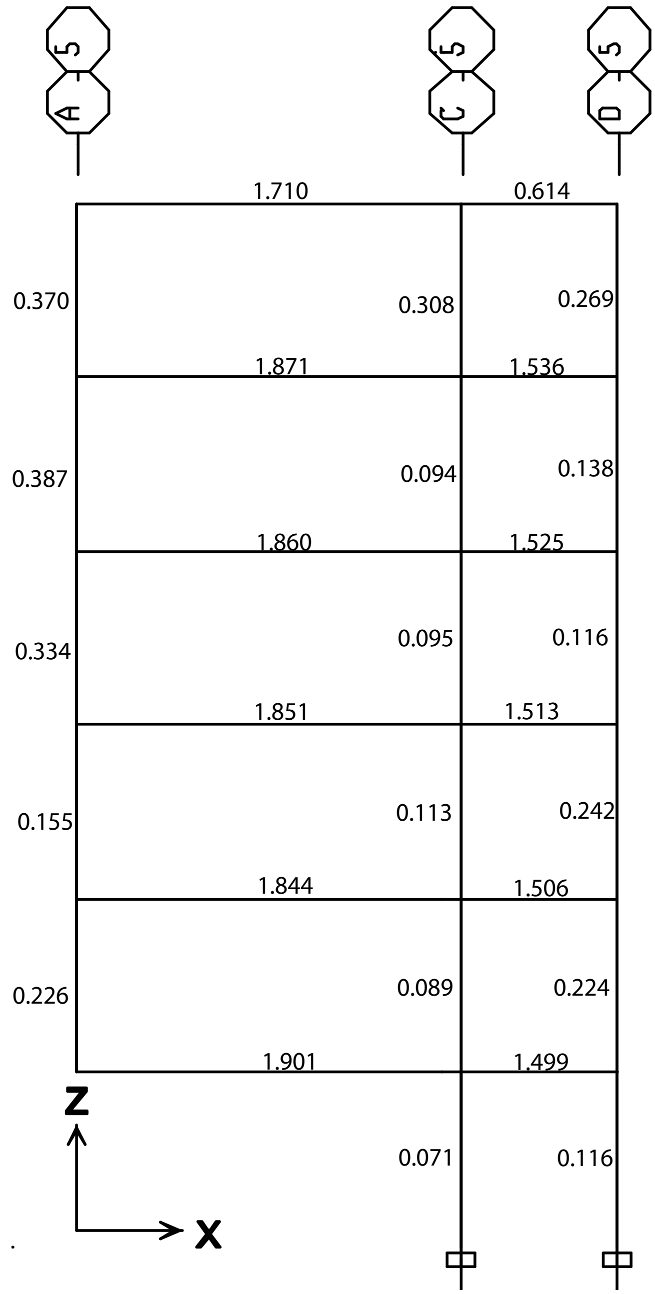


Figure 54: Demand Capacity Ratio's flexure (DCR) for short side of six-story building (corner column eliminated)

In Figure 55, eliminating the corner column leads to the DCR_{moment} values of less than 2 for the long side of six-story building. This means that the members could stand against the progressive collapse.

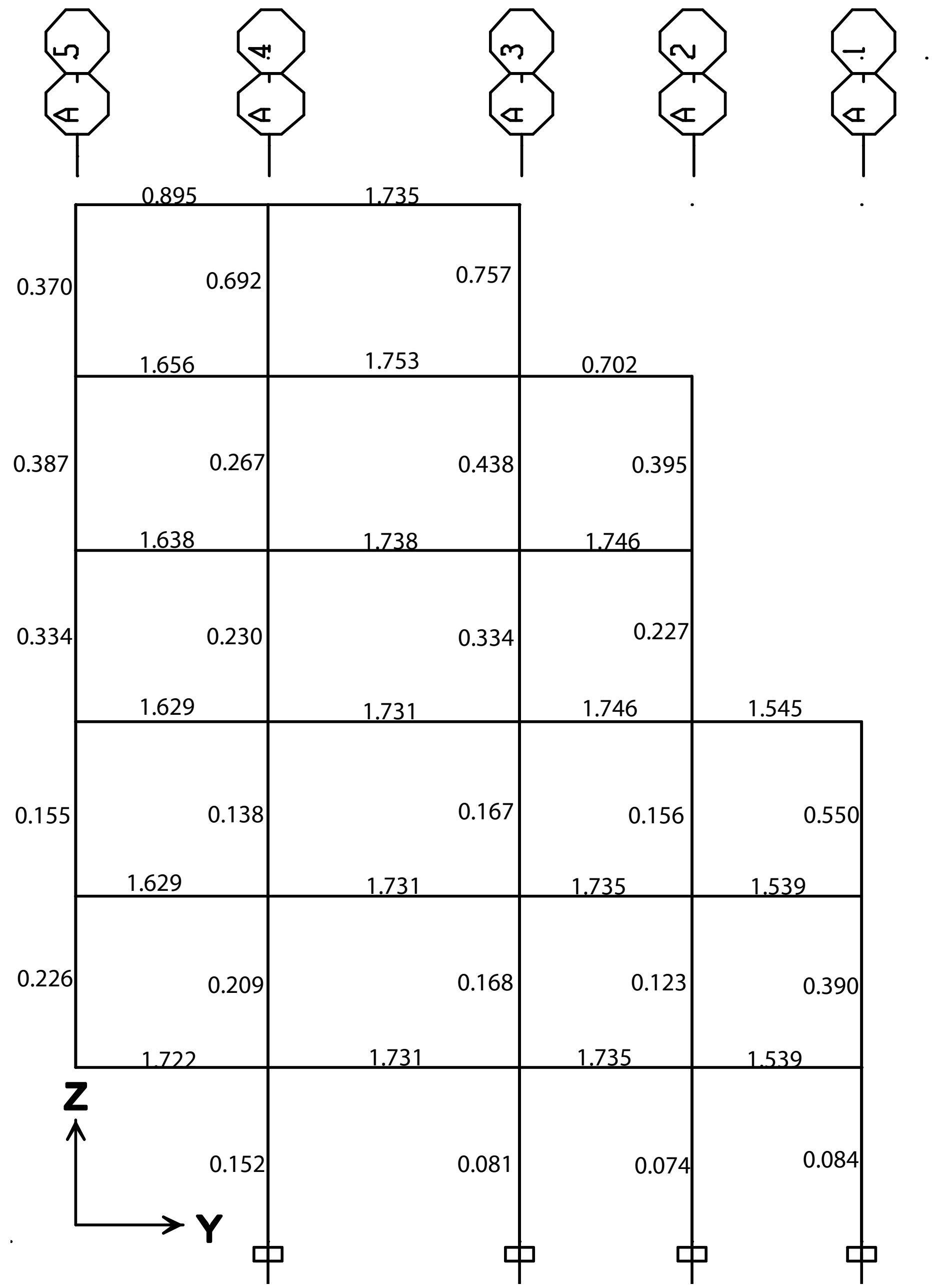


Figure 55: Demand Capacity Ratio’s flexure (DCR) for long side of six-story building (corner column eliminated)

5.3.2 Demand Capacity Ratio for Shear after Rehabilitation of the Six-story Building

Figure 56 is related to the short side of six-story building when middle column is eliminated. It should be mentioned that DCR for beams in shear did not change after rehabilitation and adding braces do not have any effect on beam's shear too. However, rehabilitation decreased columns DCR and also decreased computed axial stress due to axial force. This means that the columns now can bear the existing axial forces and the progressive collapse will not occur. After removal of middle column in short side maximum DCR_{shear} reduced to a maximum value of 0.590 which is well below the limit of 2 and the structure is guarded against progressive collapse.

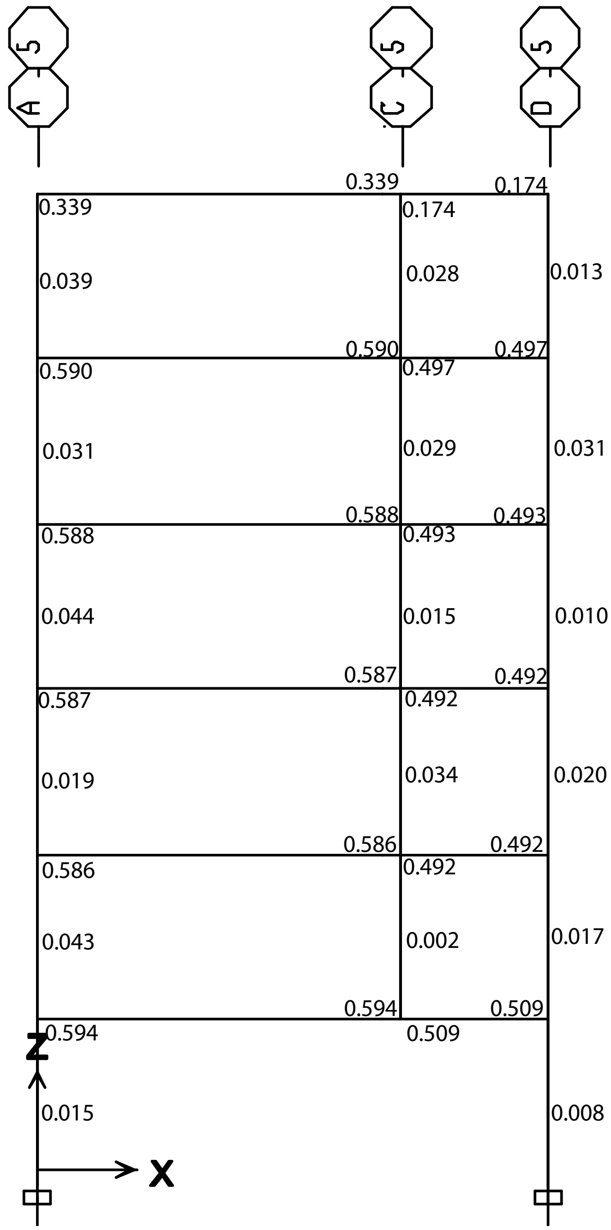


Figure 56: Demand Capacity Ratio's shear (DCR) for short side of six-story building (middle column eliminated)

Figure 57 also shows that DCR_{shear} for long side of six-story building when middle column is eliminated is lower than 2 (maximum DCR_{shear} is 1.330) for all members in this frame. This illustrates that the structure could tolerate shear force and the progressive collapse will not occur.

It should be noted that by adding new diagonal braces to the old diagonal braced system and turning the bracing into X bracing system, the f_a has become less than F_e which means that the columns could bear the existing axial forces (DCR_{shear} < 2).

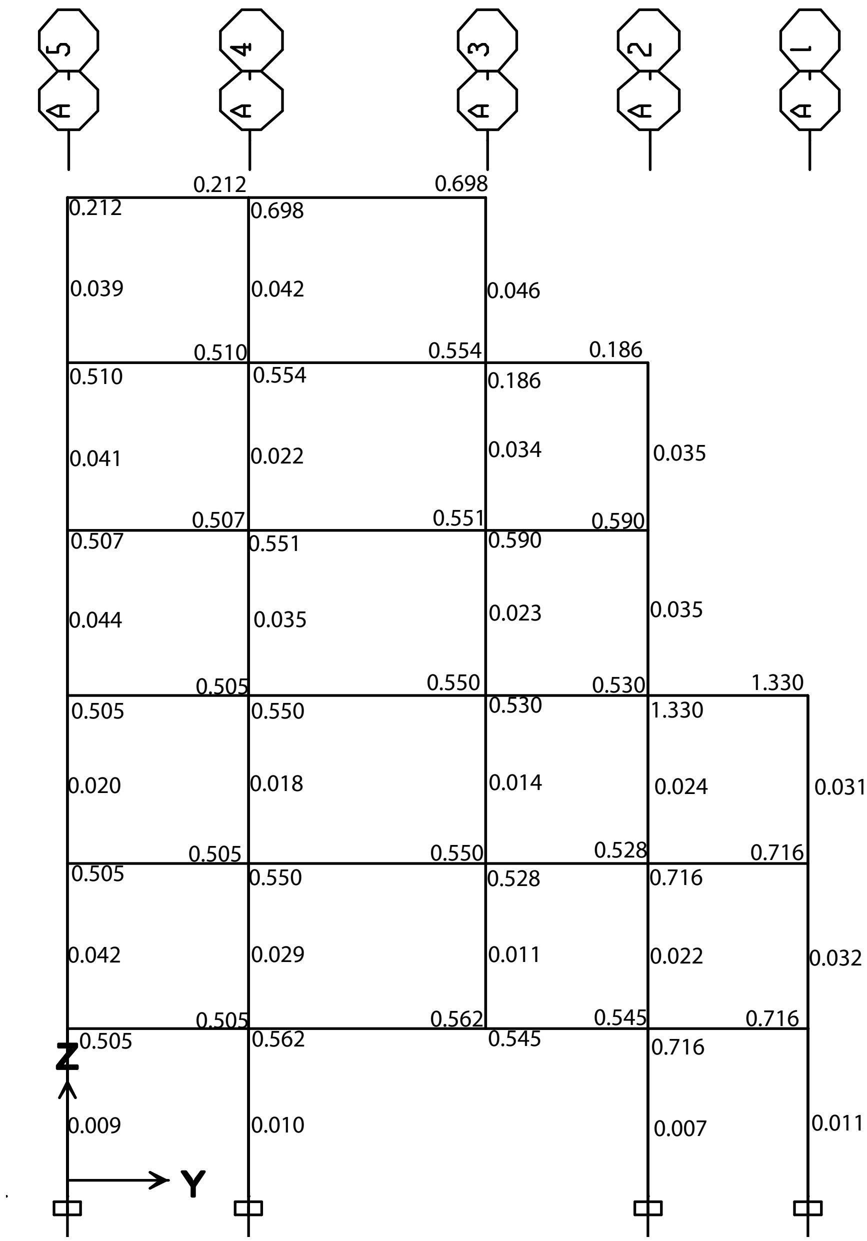


Figure 57: Demand Capacity Ratio's shear (DCR) for long side of six-story building (middle column eliminated)

DCR$_{shear}$ for the short side of the six-story building when corner column is removed is lower than 2 (maximum DCR$_{shear}$ is 0.595). Thus, there is low potential for progressive collapse to happen in this case (Figure 58).

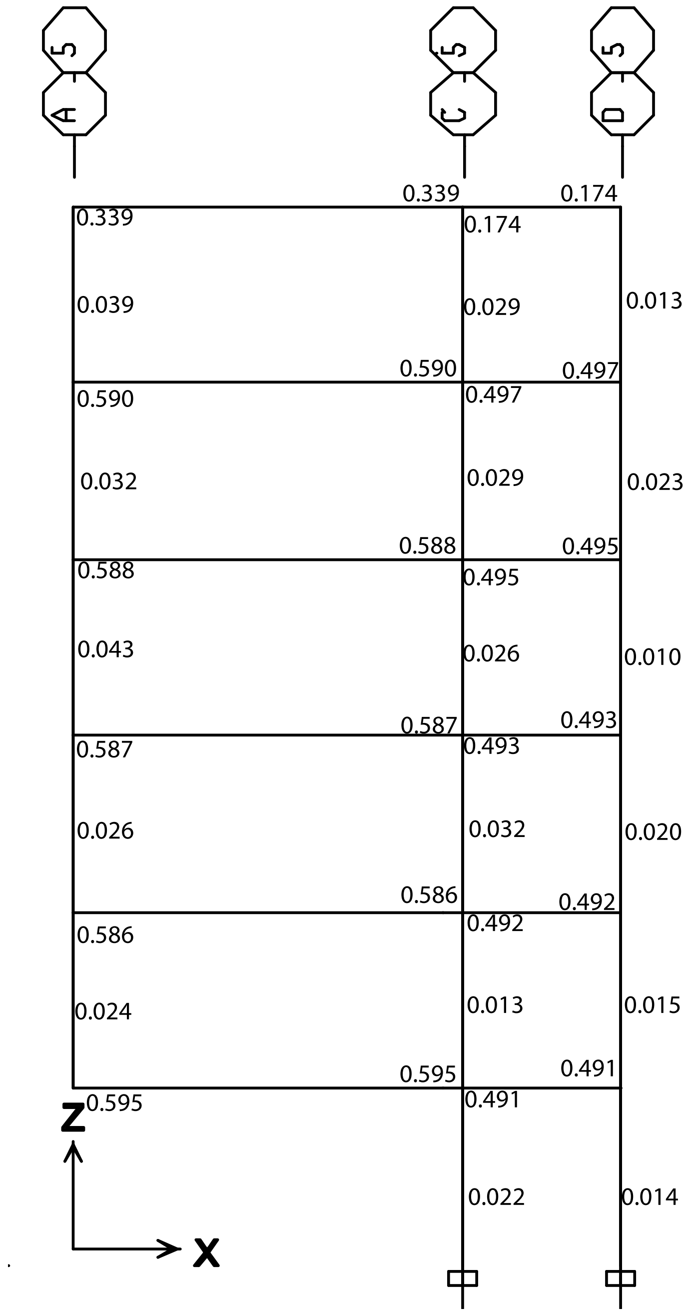


Figure 58: Demand Capacity Ratio's shear (DCR) for six-story building (corner column eliminated)

In Figure 59, DCR for long side of six-story building after removing the corner column is lower than 2 (maximum DCR_{shear} is 1.330). Hence, the structure is capable of resisting progressive collapse and the computed axial stress is less than the allowable Euler stress due to axial force and bending moment.

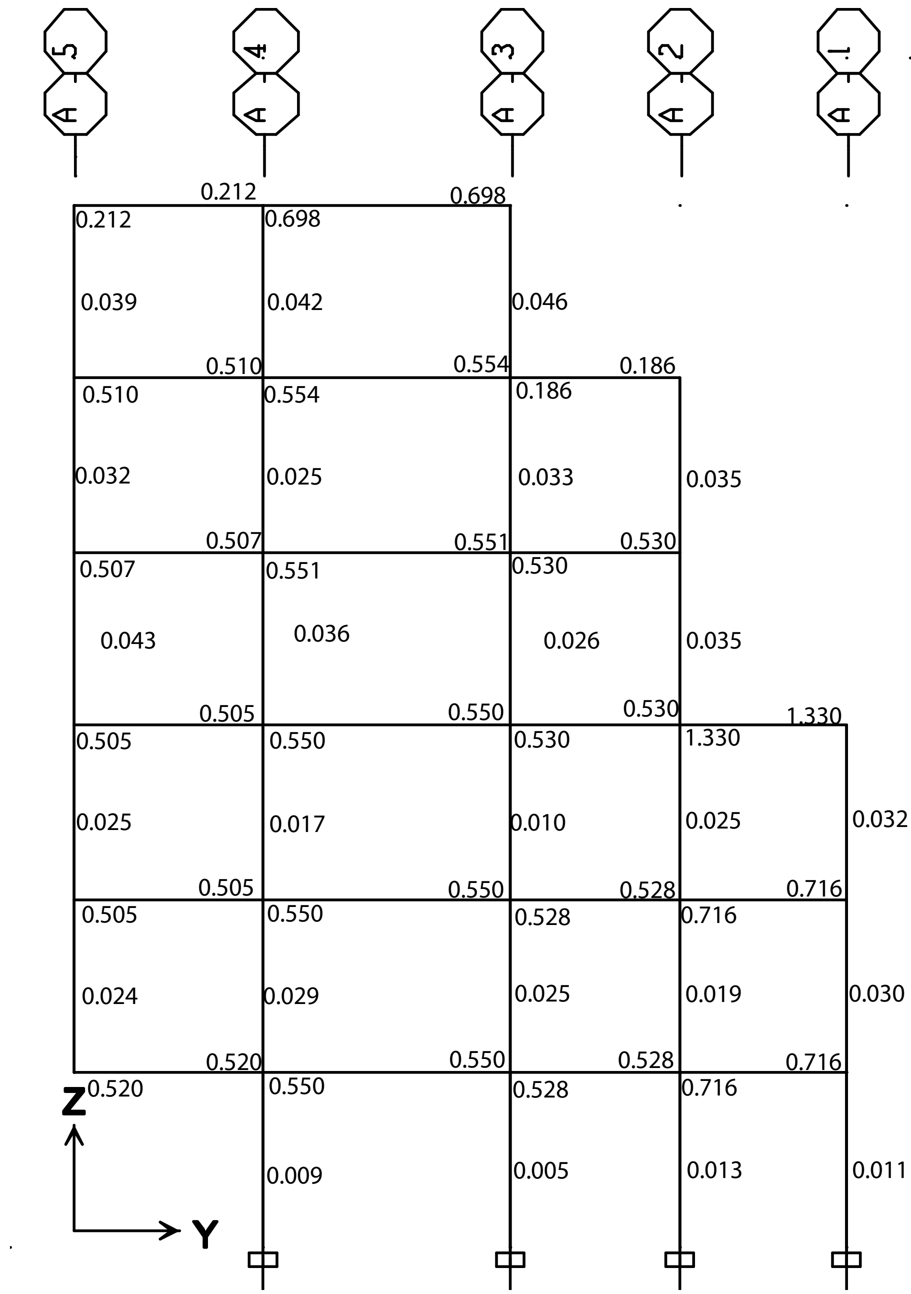


Figure 59: Demand Capacity Ratio's shear (DCR) for six-story building (corner column eliminated)

5.3.3 Demand Capacity Ratio for Axial force After Rehabilitation of the Six-story Building

Figure 60 shows that the DCR_{axial} for all members is lower than 2 (maximum DCR_{axial} is 1.284) so the susceptibility of structure against progressive collapse is very low.

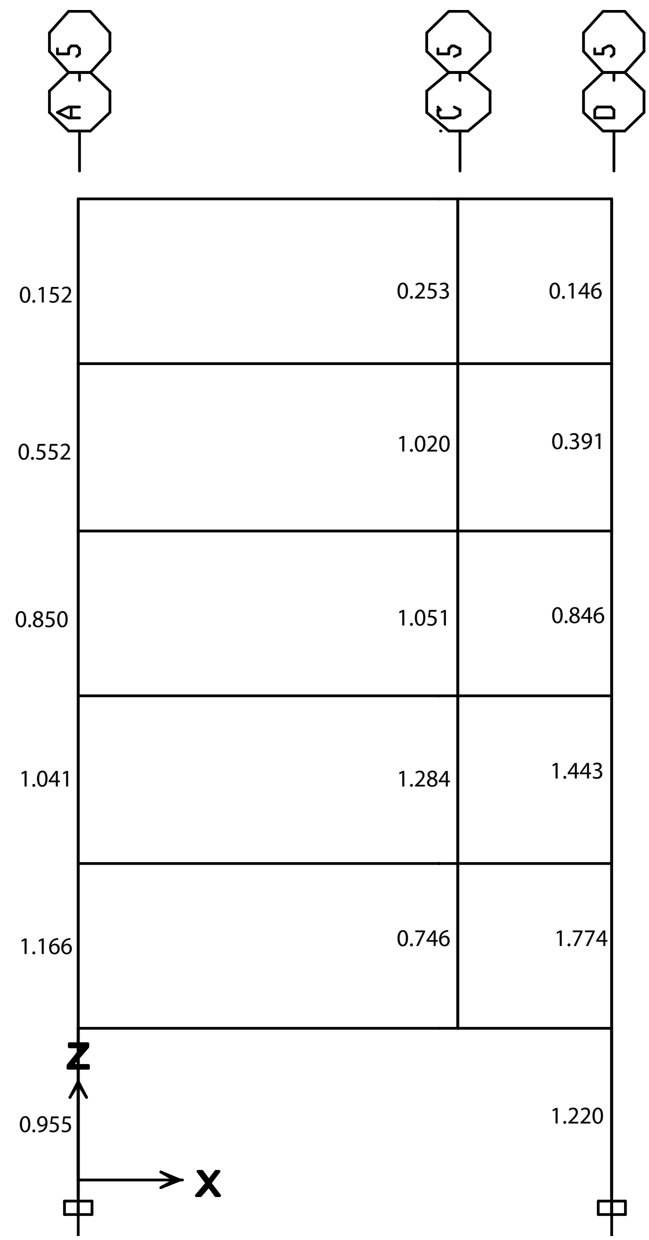


Figure 60: Demand Capacity Ratio's axial force (DCR) for six-story building (middle column eliminated)

After rehabilitation of the structure in Figure 61, DCR_{axial} is decreased a value less than 2 (maximum DCR_{axial} is 1.630) and the structure has gained enough resistance against the progressive collapse. On the other hand the computed axial stress of axial force became lower than the allowable Euler stress of axial force.

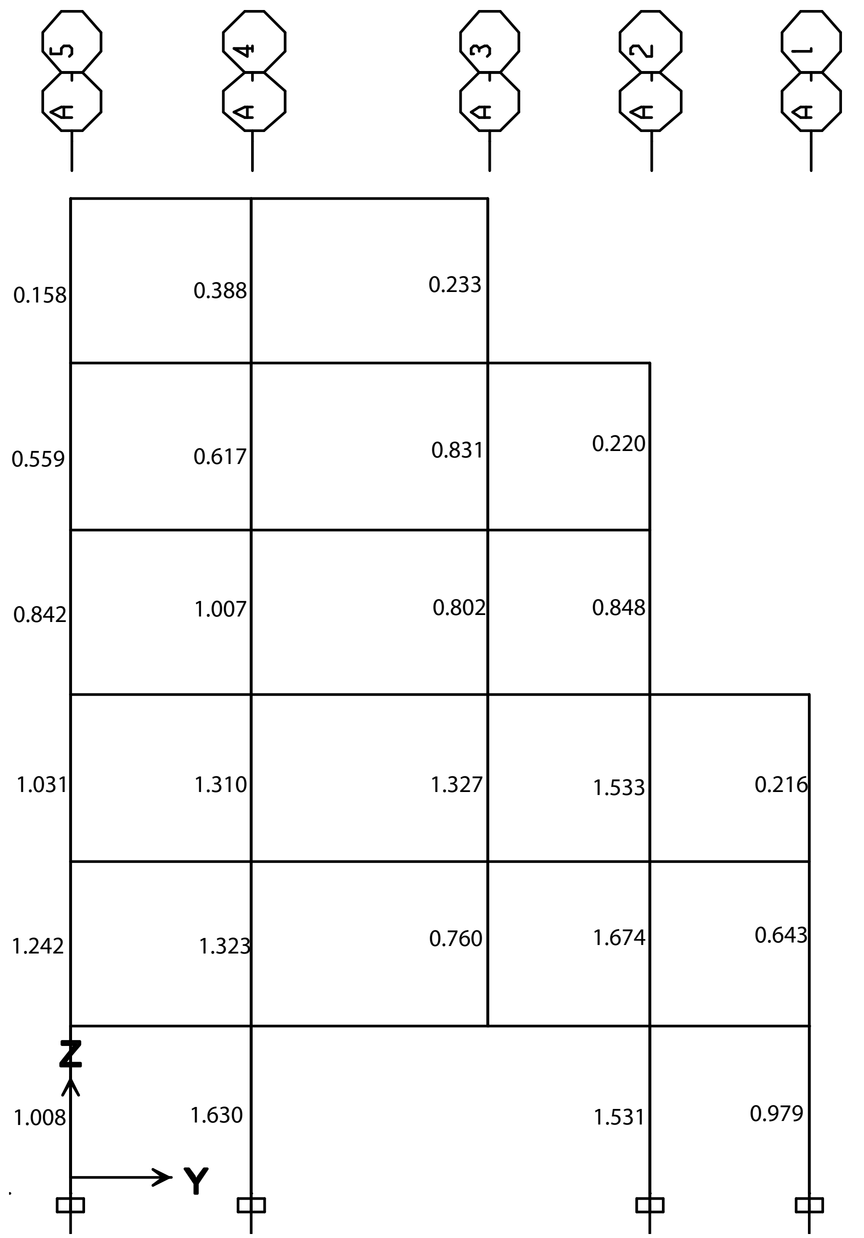


Figure 61: Demand Capacity Ratio's axial force (DCR) for six-story building (middle column eliminated)

In Figure 62, DCR_{axial} for the short side of six-story building is lower than 2 (maximum DCR_{axial} is 1.387) when corner column is eliminated. Thus there is low possibility for PC to happen.

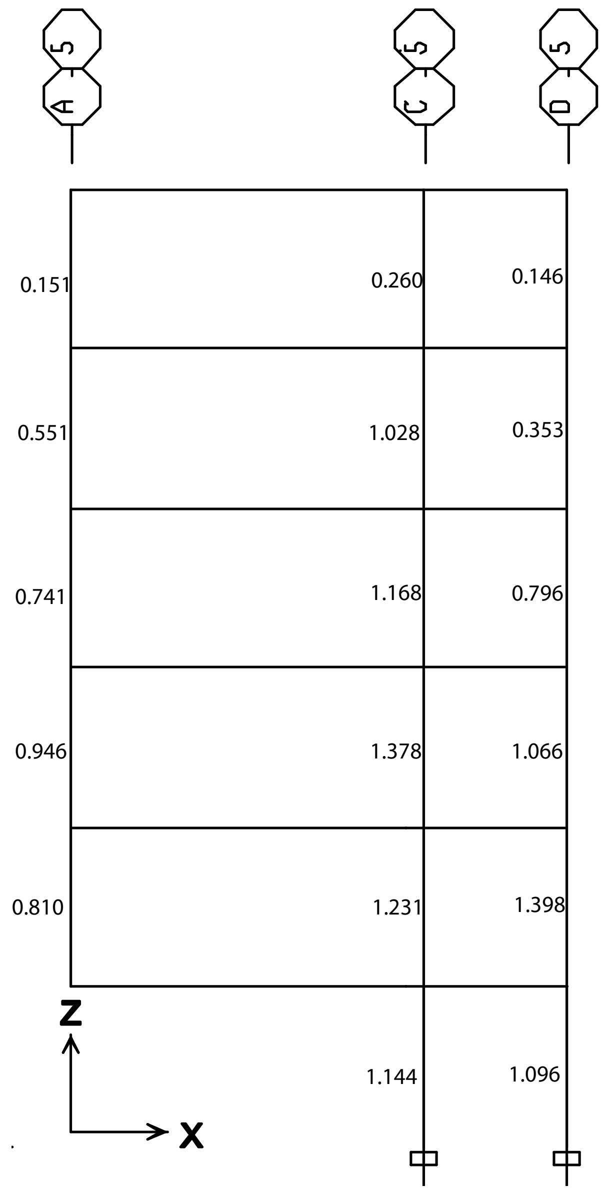


Figure 62: Demand Capacity Ratio's axial force (DCR) for six-story building (corner column eliminated)

By calculating the DCR_{axial} for the frame shown in Figure 63 (maximum DCR_{axial} is 1.975), it is realized that the structure has been guarded against the axial force in case of the occurrence of progressive collapse.

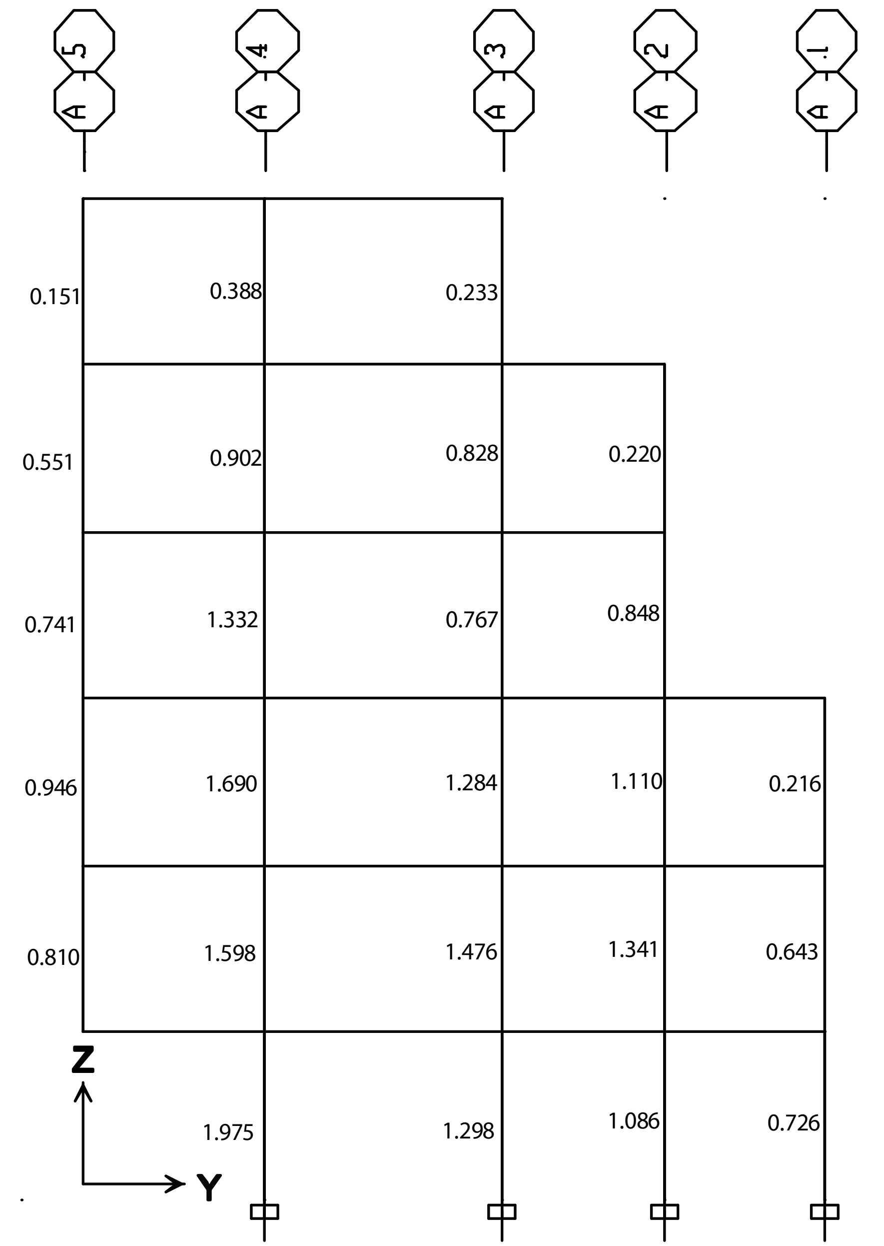


Figure 63: Demand Capacity Ratio's axial force (DCR) for six-story building (corner column eliminated)

Chapter 6

SUMMARY AND CONCLUSION

6.1 Summary

There are numerous serious threats which could cause progressive collapse in a structure that may result in loss of lives. After the incident in Oklahoma Murrah building and the recent terrorist attacks, such as WTC (World Trade Center) in 2001, demands on assessing progressive collapse have become more necessary.

Although, there have been a lot of research done on progressive collapse, the consistent increase in terrorist attacks during the past two decades and the loss of lives of hundreds of people demanded more action to be taken in line with having buildings that could withstand progressive collapse. Therefore, new guidelines, such as GSA, DoD, and UFC guidelines have been introduced for this matter. However, there has been limited research on steel structures, especially on dual frame system (moment frame with bracing system for resisting against lateral load).

There have been many researches on progressive collapse of reinforced concrete structure so far and nowadays the researches on the progressive collapse resistance of steel framed buildings are gradually increasing with the improvements on steel material, technology and methods particularly in the developed countries.

To assess the susceptibility of building to progressive collapse, linear static analysis (elastic behavior) which is a simple and conservative method has been used. Non-linear static analysis is a good choice for designing of the new buildings and two dimensional (2-D) models but for analyzing and assessing the existing buildings it would take considerably more time to carry out analysis and design. Therefore, for this research work linear static procedure which is reliable for evaluating the susceptibility of existing building to PC was more appropriate to use.

In other words, this work was aimed to compare the vulnerability of two different case studies with different number of stories and different framing systems for achieving the best result between two buildings. Also, it was intended to test the rehabilitation of frames that are likely to be subject to progressive collapse by using GSA guidelines.

Finally, the potential of each building against progressive collapse (before and after rehabilitation) has been calculated and results are reported in previous chapters in this thesis.

6.2 Major Findings

As a result of using procedures mentioned in section 6.1 the below given results were obtained.

6.2.1 Failure Progresses

Generally, sudden removal of column in a structure causes QUD (QUD is demand or acting forces for moment, shear and axial force) to increase. This means that QUD/QCE (QCE is ultimate capacity for moment, shear and axial force) will be close to or even greater than 2. As a result, this increases the Demand Capacity Ratio (demand forces or acting forces over ultimate capacity of members) which leads to the failure of structures primary members. Finally, the building became more prone to progressive collapse. It has to be noted that where Demand Capacity Ratio is more than the value of 2 (for typical structures) for axial force, moment and shear, then progressive collapse would happen.

In nine-story building with dual frame system (moment frame system with bracing system in both X and Y directions) building had a lower vulnerability towards progressive collapse when column is removed from the long side of the beam than the case of column removal from the short side of the building.

In six-story building with building gravity frame system (gravity system with diagonal bracing system) the degree of susceptibility against progressive collapse was variable. In other words sometimes short side had greater PC potentiality than the long side and vice versa. So it was unclear whether the removal of middle column in short and long side was worse than the removal of corner column or not.

For instance, in the short side, removal of middle column and in the long side the removal of corner column had worse effect on Demand Capacity Ratio. The latter caused the DCR to increase to a value of 44.778 which indicates that the building has got a very high vulnerability to PC. So rehabilitating of this building was unavoidable and after rehabilitation the potential of building having subject to PC considerably lowered.

6.2.2 The Effect of the Number of Stories

Despite of the nine-story building having more floors than the six-story building it was found to have more resistance against progressive collapse.

6.2.3 Summarizing and Comparing the Case Studies

Comparing the behavior between nine-story and six-story buildings it appears that the nine-story building with dual frame system has lower vulnerability to PC than the six-story building with gravity frame system. This also means that use continuous beam to column connections (rigid beam-column connection) or moment frame system in steel frame is better for the resistance of buildings against progressive collapse. For the nine-story building, box columns (square boxes) are used and as a result there were no weak axes for the mentioned columns. But in six-story building built-up beams, a combination of IPE with plates has been used. In case of progressive collapse the critical damage is likely to happen due to columns being bent in their weak axes (around the web). Therefore, it was also this reason that caused the six-story building not to withstand against progressive collapse.

According to section 6.1 the buildings were faced with progressive collapse occurrence and the problem (for the six-story building) was solved by inserting braces into the framing system. It should be emphasized that the maximum DCR and maximum deflection for the two buildings are detailed in Table 3 to 5, nine-story building and six-story building before rehabilitation and nine-story building after rehabilitation).

Below Tables show the maximum DCR and deflection after the removal of columns. (B and C are beams and columns respectively):

Table 3: Maximum DCR and deflection for nine-story building after the removal of columns, based on GSA guidelines

Title	DCR_{Moment}	DCR_{Shear}	DCR_{Axial}	DEFLECTION Beam (m)	DEFLECTION Column (m)
Middle of Short Side	1.087	0.511	0.405	0.008 at 0	0.0364
Middle of Long Side	0.903	0.566	0.617	0.005 at 2.565	0.0366
Corner of Short Side	0.790	0.382	0.437	0.007 at 3.289	0.0366
Corner of Long Side	1.058	0.567	0.590	0.007 at 0	0.0366

Table 4: Maximum DCR and deflection for six-story building before rehabilitation after removing columns, based on GSA guidelines

Title	DCR_{Moment}	DCR_{Shear}	DCR_{Axial}	DEFLECTION Beam (m)	DEFLECTION Column (m)
Middle of Short Side	20.530 and $f_a > F_e$	1.080 and $f_a > F_e$	1.146 and $f_a > F_e$	0.045 at 3.471	0.1144
Middle of Long Side	1.849 and $f_a > F_e$	1.330 and $f_a > F_e$	2.046 and $f_a > F_e$	0.018 at 2.006	0.0173
Corner of Short Side	1.901	0.595	1.776	0.045 at 3.372	0.0366
Corner of Long Side	44.778 and $f_a > F_e$	1.330 and $f_a > F_e$	2.592 and $f_a > F_e$	0.017 at 1.418	0.0366

Table 5: Maximum DCR and deflection for six-story building after rehabilitation and removal of the columns, based on GSA guidelines

Title	DCR_{Moment}	DCR_{Shear}	DCR_{Axial}	DEFLECTION Beam (m)	DEFLECTION Column (m)
Middle of Short Side	1.898	0.594	1.774	0.045 at 3.471	0.0240
Middle of Long Side	1.849	1.330	1.674	0.018 at 2.006	0.0169
Corner of Short Side	1.901	0.595	1.398	0. 045 at 3.372	0.0153
Corner of Long Side	1.753	1.330	1.975	0.017 at 1.418	0.0153

In the Tables 3 to 5 it can be perceived that the behavior of the nine-story structure in terms of progressive collapse is much better than that of the six-story structure even after execution of rehabilitation. It should be mentioned that, moment frames which are accompanied by bracing systems have been used in the nine-story building, while gravity frame with bracing system has been installed and implemented in the six-story building. The better performance of the nine-story structure is due to the difference in structural system, the kind of column sections and the disparity in bracing systems which have been installed on the selected exterior frames. Meanwhile it should be indicated that the bracing system of the nine-story building (in short and long side) was the X bracing system but in the six-story building merely in the long side, the diagonal bracing system was used.

6.3 Final Conclusion

The outcome of this thesis is associated with some prominent and important ideas such as:

- In a moment braced frame system, the resistance of structure against progressive collapse is comparatively much greater and better than gravity braced system.

- Usage and implementation of built-up box shaped sections (square boxes) especially for the frames which are expose to exterior or interior damages, resulted in more resistance against progressive collapse when compared with the built-up IPE section and its combinations and derivatives.

- The columns which are positioned on the periphery of the structure should be disposed and located in the direction so that the bending of the column (with IPE section and its combination with plate) occurs around the strong axes (flange) or in other word the moment pivots around the strong axes.

  The below case exemplifies the last result:

  In the six-story building in which the columns were combined by IPE and welded plates, the columns was bent around their minor axis (selected side in the study). Later the columns of exterior frame of long side, beside the road, were rotated 90 degrees and the bending of the column happened around the strong axes (flange). This increased their resistance against progressive collapse. The

DCRs (for columns) and maximum deflections (maximum deflection for columns or joint) has been decreased.

Figure 64 shows the columns axes direction in first floor for six-story building:

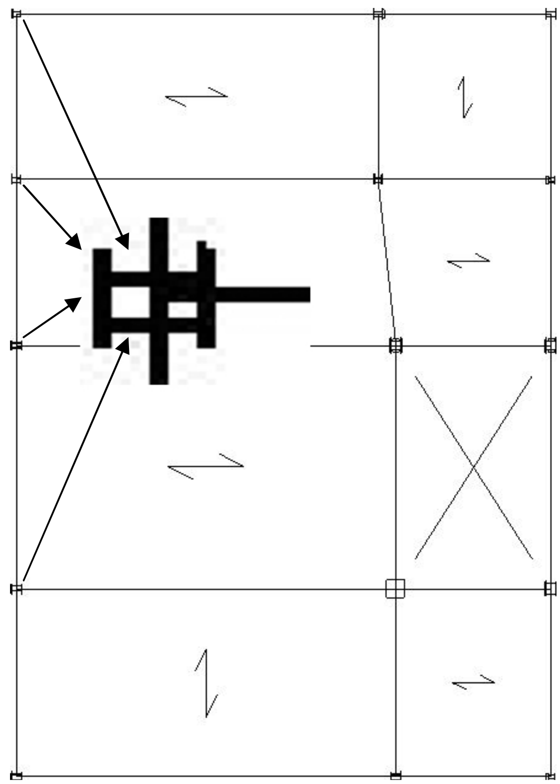


Figure 64: Exterior column direction for six-story building

Figure 65 shows the six-story building first floor plan after rotating the columns located in the exterior frame, beside the road by, 90 degrees.

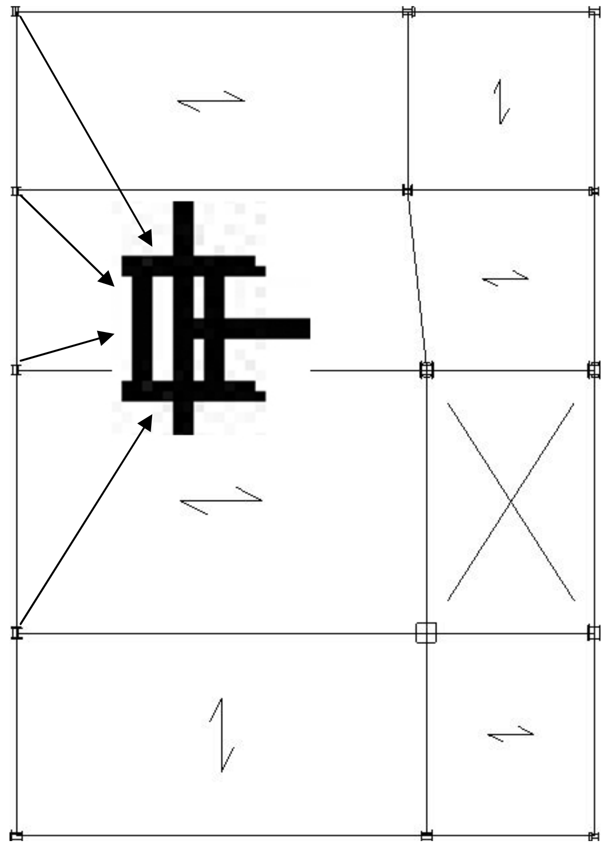


Figure 65: Exterior column direction for six-story building after rotation (90°)

Table 6 shows the maximum DCR and deflection after rotating the columns located in the exterior frame, beside the road, by 90 degrees.

Table 6 : Maximum DCR and deflection for six-story building after rotating the columns, by 90 degrees

Title	DCR_{Moment}	DCR_{Shear}	DCR_{Axial}	DEFLECTION Beam (m)	DEFLECTION Column (m)
Middle of Short Side	20.530 and $f_a > F_e$	1.080 and $f_a > F_e$	1.146 and $f_a > F_e$	0.045 at 3.471	0.1144
Middle of Long Side	1.839 and $f_a > F_e$	1.330 and $f_a > F_e$	2.045 and $f_a > F_e$	0.018 at 2.009	0.0173
Corner of Short Side	1.901	0.595	1.776	0.045 at 3.372	0.0366
Corner of Long Side	7.744 and $f_a > F_e$	1.330 and $f_a > F_e$	2.592 and $f_a > F_e$	0.017 at 1.957	0.0366

Table 6 shows that after rotating the columns by 90 degrees DCR and maximum deflection for columns or joints have been decreased in long side of building, beside the road. After removing the column in the corner of long side maximum DCR_{moment} has been 7.744. Hence, the building has a better behavior and resistance against progressive collapse.

When progressive collapse or similar subjects are considered it is better not to use gravity frames with bracing systems. In other words, if it is used in structures it is recommended not to implement diagonal bracing systems for the first floor above the grade. X-bracing system or inverted V bracing systems are more appropriate to be used in these locations and even V bracing system could be much better than diagonal bracing since it has support against the ground.

For instance:

When the V bracing system has been used in the first floor of six-story building (for exterior frames, beside the road) the resistance of structure against progressive collapse increased noticeably as can be seen from Figures 66 and 67.

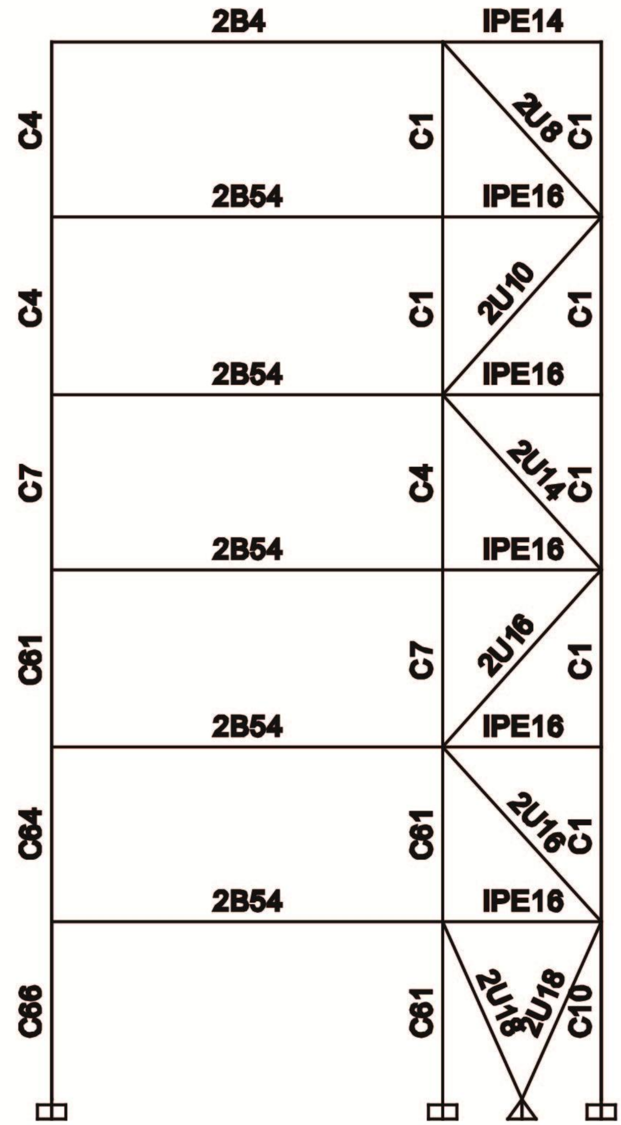


Figure 66 : V-Braced frame system of short side

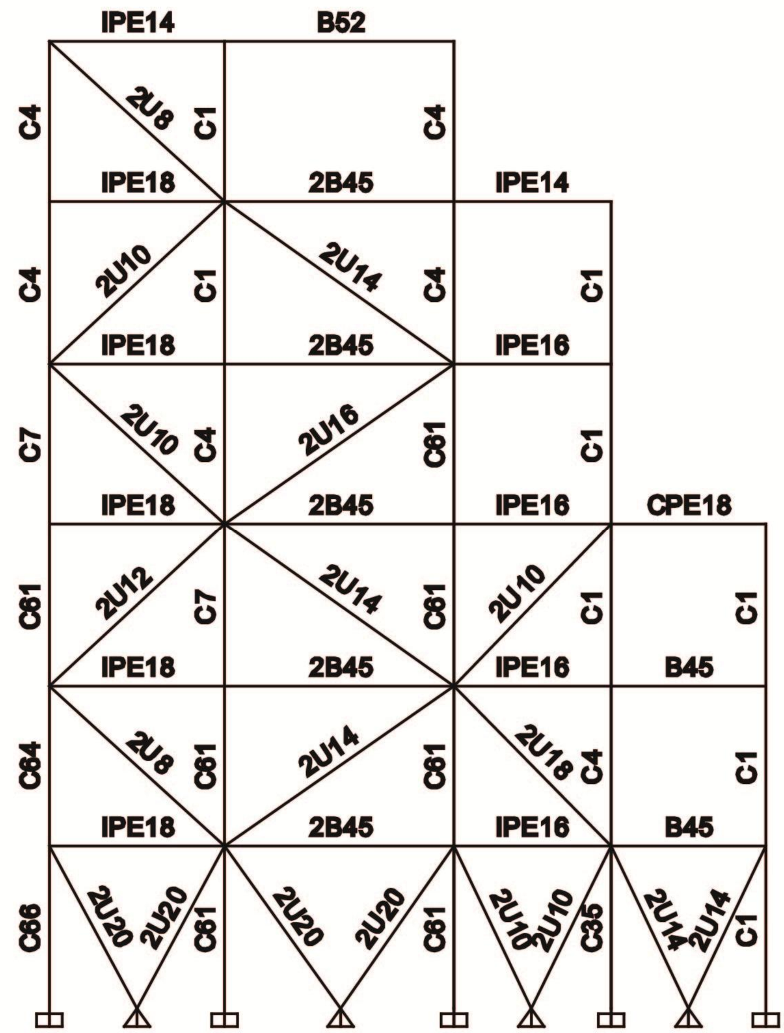


Figure 67: V-Braced frame system of long side

Table 7 shows the maximum DCR and deflection for six-story building after inserting V-bracing system in first floor, based on GSA guidelines.

Table 7: Maximum DCR and deflection for six-story building after implementing V-bracing system in first floor, based on GSA guidelines

Title	DCR_{Moment}	DCR_{Shear}	DCR_{Axial}	DEFLECTION Beam (m)	DEFLECTION Column (m)
Middle of Short Side	1.898	0.594	1.563	0.045 at 3.471	0.0216
Middle of Long Side	1.849	1.330	1.551	0.018 at 2.006	0.0168
Corner of Short Side	1.901	0.595	1.394	0. 045 at 3.372	0.0151
Corner of Long Side	1.753	1.330	1.500	0.017 at 1.431	0.0151

Table 7 shows that after inserting the V braces in first floor above the grade DCR (specially DCR_{axial}) and maximum deflection for columns or joints have been decreased even in respect to rehabilitation of six-story building by X braces. Thus, the building has a better behavior and resistance against progressive collapse.

6.4 Recommendations for Future Studies

For future study, the analysis of progressive collapse by nonlinear methods, such as nonlinear static analysis and nonlinear dynamic analysis are suggested to be used.

REFERENCES

ACI. (2002), Building Code requirements for Structural Concrete (ACI 318-02), American Concrete Institute, Farmington Hills, Michigan.

Allen, D.E,. and Schriever, W.R. (1972). Progressive Collapse, Abnormal loads, and Building Codes. Division of Building Research, National Research Council.

ASCE 7-95. American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures. Published by the American Society of Civil Engineers, New York.

ASCE. (2002). Minimum design loads for buildings and other structures. American Society of Civil Engineers publication ~note revision of 7-98, does not include building security or antiterrorism but covers.

ETABS-3D, Version 9.5.0, Computers and Structures, Inc.,Berkeley.

FEMA 274. (1997). NEHRP Commentary on the guidelines for the seismic rehabilitation of buildings. Washington (DC): Federal Emergency Management Agency.

Fu, F. (2009). Progressive collapse analysis of high-rise building with 3-D finite element modeling method. Journal of Constructional Steel Research. 10.1016/j.jcsr.2009.02.001.

General Services Administration. (GSA, 2003). Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects.

Herrle K.W., and McKay A. E. (2005). Development and application of progressive collapse design criteria for the federal government.

ISC Security Design Criteria for New Federal Office Buildings and Major Modernization Projects. (2001). Published by the Interagency Security Committee.

Kaewkulchai, G., and Williamson, E.B. (2003). Dynamic behavior of planar frames during progressive collapse. In: 16th ASCE engineering mechanics conference.

Khandelwal, K., El-Tawil, S., and Sadek, F. (2009). Progressive collapse analysis of seismically designed steel braced frames, Journal of Constructional Steel Research. 10.1016/j.jcsr.2008.02.007.

Khandelwal, K., El-Tawil, S., Kunnath, S. K,. and Lew H. S.(2008). Macromodel-Based Simulation of Progressive Collapse: Steel Frame Structures. Department of Civil and Environmental Engineering at the University of Michigan, the National Science Foundation. Journal of Structural Engineering © ASCE. 10.1061/(ASCE)0733-9445(2008)134:7(1070)

Kim, J., and Kim, T. (2009). Assessment of progressive collapse-resisting capacity of steel moment frames, Journal of Constructional Steel Research. 10.1016/j.jcsr.2008.03.020.

Kim, T., and Kim, J. (2008). Collapse analysis of steel moment frames with various seismic connections. Journal of Constructional Steel Research journal. 10.1016/j.jcsr.2008.11.006.

Lee C-H., Kim, S., Han K-H., and Lee, K. (2008). Simplified nonlinear progressive collapse analysis of welded steel moment frames, Department of Architectural Engineering, Seoul National University. Journal of Constructional Steel Research. 10.1016/j.jcsr.2008.10.008.

Marjanishvili, S. M. (2004). Progressive Analysis Procedure for Progressive Collapse. Journal of Performance of Constructed Facilities © ASCE. 10.1061/(ASCE)0887-3828(2004)18:2(79).

Marjanishvili, S. M., and Agnew, E. (2006). Comparison of various procedures for progressive collapse analysis. Journal of Performance of Constructed Facilities, 10.1061/(ASCE)0887-3828(2006)20:4(365).

National Institute of Standard and Technology (NIST). Best practices for reducing the potential for progressive collapse in buildings (Draft).

Powell G. (2005). Progressive collapse: Case study using nonlinear analysis. In: Proceedings of the 2005 structures congress and the 2005 forensic engineering symposium.

Rahman, N. A,. Booth, M,. and Bennett, G. (2007). Progressive Collapse Requirements. Cold-Framed Steel Load Bearing Construction.

Sadek, F,. Main, J.A., Lew H.S., Robert, S. D,. Chiarito, V.P , and El-Tawil, S. (2010). An Experimental and Computational Study of Steel Moment Connections under a Column Removal Scenario. U.S. Department of Commerce Gary Locke. National Institute of Standards and Technology.

Sadek, F., Main, J. A., Lew H. S., Robert, S. D,. and Chiarito, V. (2009). Testing and Analysis of Steel Beam-Column Assemblies under Column Removal Scenarios. Structures 2009: Don't Mess with Structural Engineers © 2009 ASCE.

Sezen, H., and Giriunas, K.H. (2009). Progressive Analysis of an Existing Building. The Ohio State University.

Shankar Nair, R. (2004). Progressive Collapse Basics. http://www.emergencymgt.net/sitebuildercontent/sitebuilderfiles/ProgressiveCollapseBasics.pdf.

Song,B. I,. Sezen, H., and Giriunas, K.H. (2010). Experimental and Analytical Assessment on Progressive Collapse Potential of Two Actual Steel Frame Buildings. Structures Congress © 2010 ASCE . giriunas.1@buckeyemail.osu.edu.

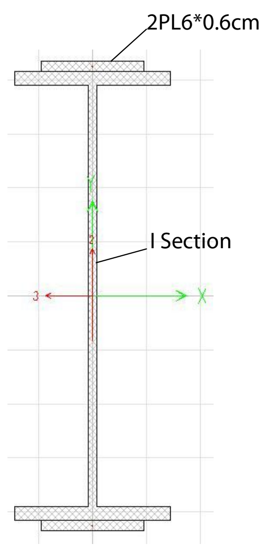
Stiemer, S.F. (2007). Case Studies Using Elastic and Plastic Analysis (steel design).

Tan, S., and Astaneh-Asl, A. (2003). Cable-Based Retrofit of Steel Building Floors to Prevent Progressive collapse. Report Number: UCB/CEE-STEEL-2003/02. Department of Civil and Environmental Engineering College of Engineering University of California at Berkeley.

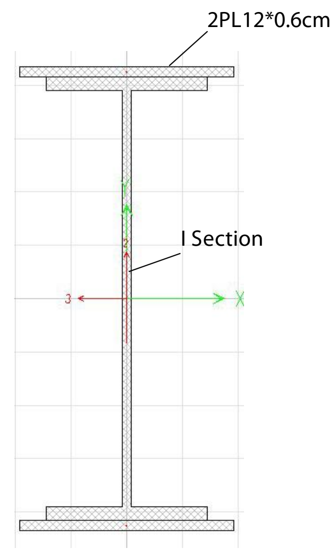
Unified Facilities Criteria (UFC). (2010). DoD. Design of Buildings to Resist Progressive Collapse, Department of Defense.

APPENDIX

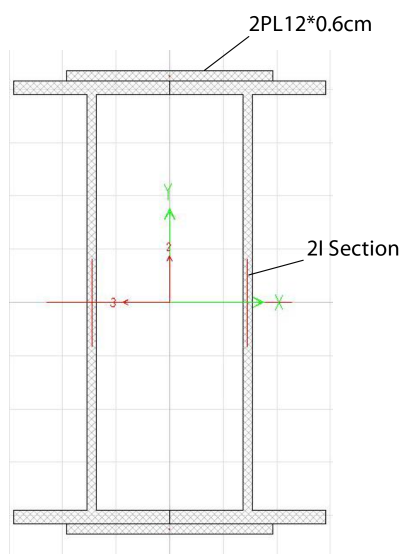
The sections shape in six-story building (exterior frames, beside the road) is shown in below Figures.



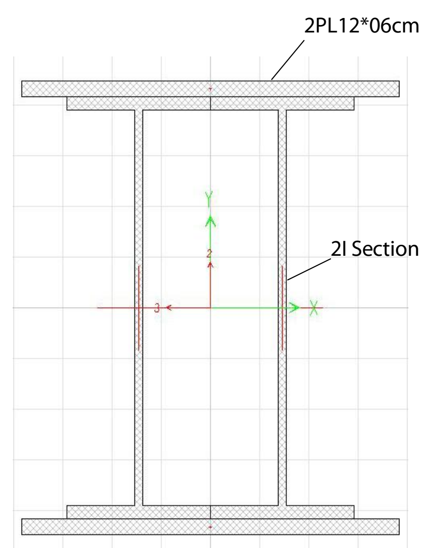
B45



B52



2B45

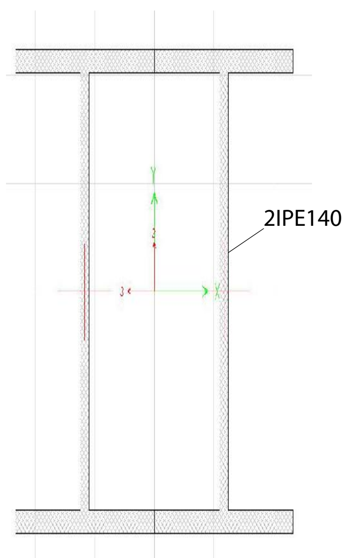


2B54

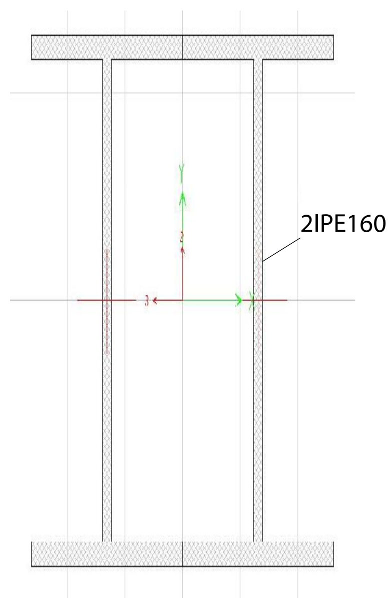
The I-section details in six-story building (exterior frames, beside the road) which have been used for beams are shown in below Table.

I section details.

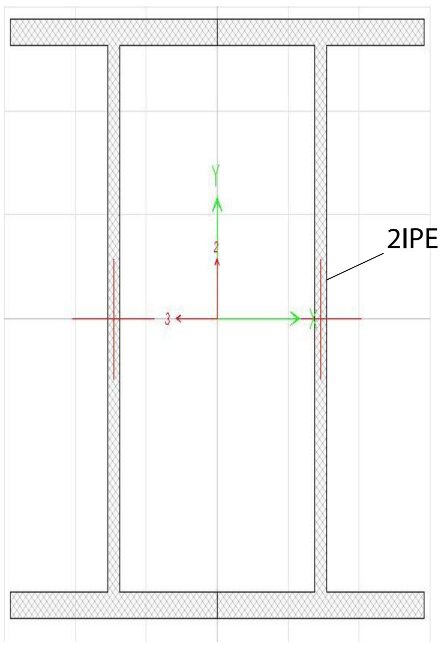
Title	No of I sections	Height (m)	Top Width (m)	Top Thick (m)	Web Thick (m)	Bot Width (m)	Bot Thick (m)
2B45	2	0.26	0.091	8e-3	5.3e-3	0.091	8e-3
2B54	2	0.26	0.091	8e-3	5.3e-3	0.091	8e-3
B45	1	0.26	0.091	8e-3	5.3e-3	0.091	8e-3
B52	1	0.26	0.091	8e-3	5.3e-3	0.091	8e-3



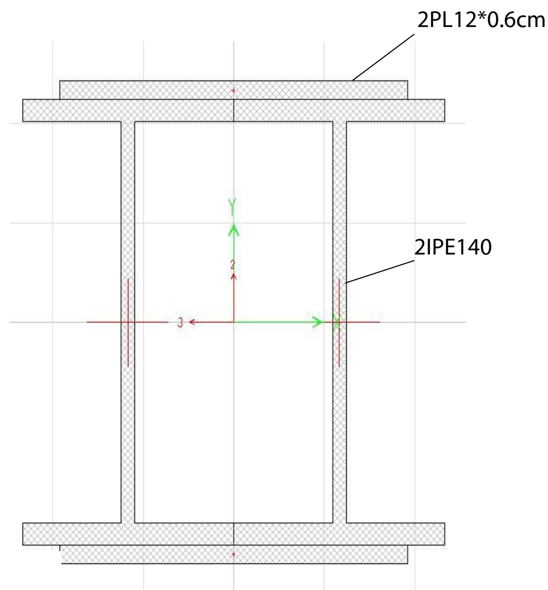
C1



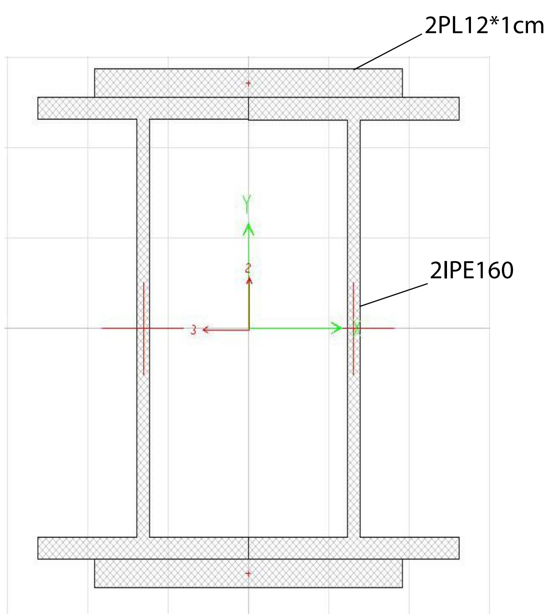
C4



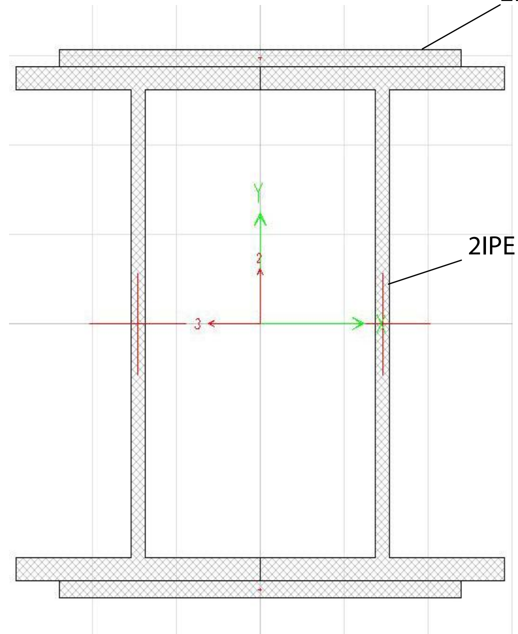
C7



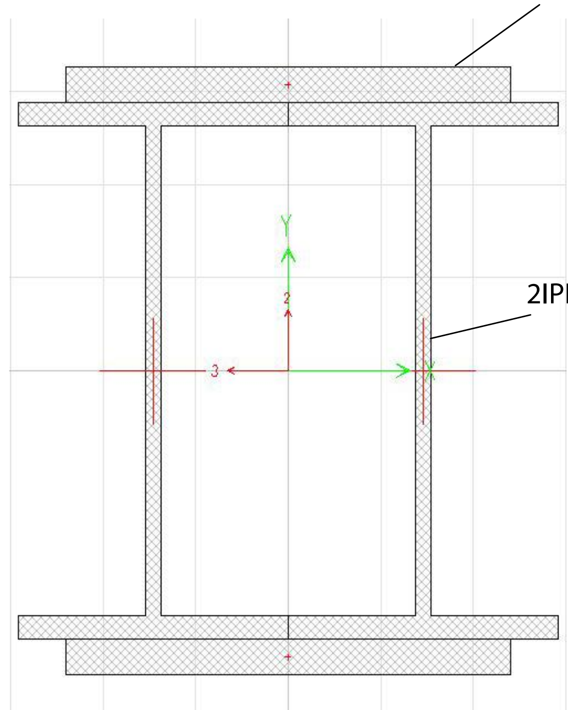
C10



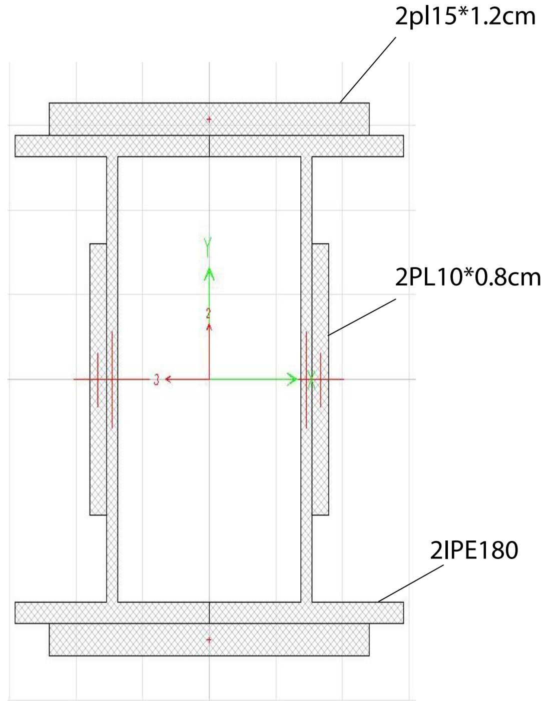
C35



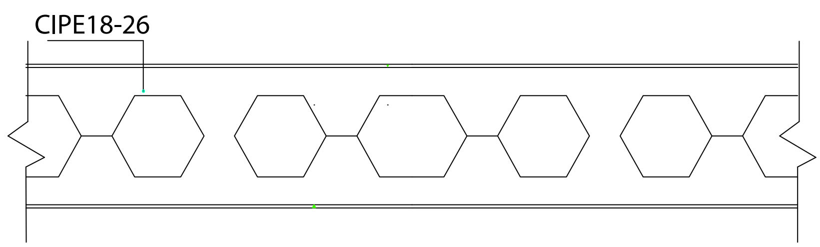
C61



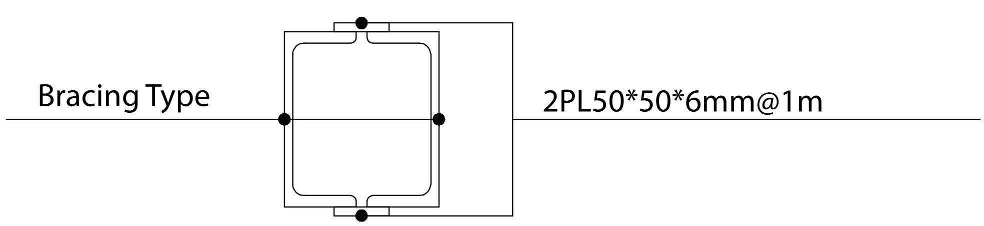
C64



C66



CPE18



Bracing Type

Nine-story section details:

The Box section details in nine-story building (exterior frames, beside the road) which have been used for columns are shown below.

Box section details

Title	Outside depth (t3) (m)	Outside width (t2) (m)	Flange thickness (tf) (m)	Web thickness (tw) (m)
BOX35-1	0.35	0.35	0.010	0.010
BOX35-1.2	0.35	0.35	0.012	0.012
BOX35-1.5	0.35	0.35	0.015	0.015
BOX35-2	0.35	0.35	0.020	0.020

The I-section details in nine-story building (exterior frames, beside the road) which have been used for beams are shown in below Table.

I-section properties

Title	Outside height (t3) (m)	Top flange width (t2) (m)	Top flange thickness (tf) (m)	Web thickness (tw) (m)	Bottom flange width (t2b) (m)	Bottom flange thickness (tfb) (m)
PG1	0.330	0.250	0.015	8e-3	0.250	0.015
PG2	0.324	0.250	0.012	8e-3	0.250	0.012
PG3	0.270	0.250	0.010	8e-3	0.250	0.010