Robustness and Retrofit Strategies for Seismically-Designed Multistory Steel Frame Buildings Prone to Progressive Collapse

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ABSTRACT

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Seyed Javad Mirvalad

Unlike seismic engineering that attracted the attention of researchers, designers, and code developers for decades, the phenomenon of progressive collapse of structures still needs considerable amount of investigation. The main motivation for this study is to investigate the vulnerability of seismic code designed multistory steel moment resisting frame buildings to progressive collapse, and to propose retrofit solutions for those buildings that show to be prone to progressive collapse.

The studied buildings had 5, 10, and 15 stories (representing low-rise, medium-rise, and high-rise buildings), where each building was designed for three seismic zones (representing low, medium, and high seismicity). All studied buildings have a 3-bays x 6-bays rectangular plan; each bay has a span of 6 meters. Alternate Path Method (APM) recommended by GSA 2003 guidelines is adopted to evaluate the robustness of the buildings against progressive collapse. Three-dimensional models of the buildings are built using the Extreme Loading for Structures (ELS) software, where nonlinear static and nonlinear dynamic time history analysis are conducted for six different column removal scenarios for each building.

The nonlinear dynamic analyses showed that buildings designed for low seismicity do not possess sufficient resistance against column removal cases, thus need to be retrofitted to safeguard against the possibility of their progressive collapse. Consequently, two retrofit methods using top beams grid system and top gravity truss system are

proposed for buildings in low seismic zones in order to enhance their robustness against progressive collapse. The nonlinear static and nonlinear dynamic analyses of the retrofitted buildings using the ELS software showed the effectiveness of the proposed retrofit systems in mitigating progressive collapse.

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To my parents and brothers

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Chapter 1
INTRODUCTION

1.1 General

Different types of loads such as dead, live, snow, wind, and seismic loads have been considered in building codes for decades. Seismic loads are one of the most uncertain types of loads that building codes have required engineers to consider in the design of buildings for many years. There have been a considerable amount of research work and study on different aspects of earthquakes and their consequent effects on the buildings in order to provide engineers with simple and practical instructions for performing a seismic design. However, there are cases that a building may face extreme unexpected loads which are not considered in the design. During the past century, there have been several reported cases of collapses in many buildings around the world which were initiated by extreme loading conditions such as explosions, impacts, or car collisions. These types of loads are associated with extreme uncertainty in both quality and quantity, and unlike seismic loads, are not usually considered in building codes. The research community has shown a growing interest in this field recently, especially after the collapse of the World Trade Center towers in 2001, yet major research works need to be conducted on extreme loading conditions and progressive collapse of structures.

Progressive collapse is defined in commentary C1.4 of ASCE 7-05 as *"the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it"*. ASCE 7-05 presents general design recommendation for improving the progressive collapse resistance of structures, but it does not provide specific rules for designers. There are a few existing guidelines which exclusively address progressive collapse of structures; among them, the guidelines from General Service Administration of United States (GSA 2003) and Unified Facilities Criteria of Department of Defense of United States (DoD 2009) are used most popularly.

The most common approach for studying the progressive collapse potential of buildings is to make use of the Alternate Path Method (APM) recommended by both GSA and DoD guidelines. In this method, a single vertical load bearing member (column or wall) of the ground floor is assumed to be instantaneously removed, and the ability of the structure to span across the missing member is evaluated. APM is a threat-independent method as it does not consider the hazard which causes the member to be lost. The building needs to transfer the loads of the removed member through new load paths in order to keep its stability after a column loss event.

Deciding whether or not a building needs to be designed to resist progressive collapse depends highly on the importance of the building. As this type of hazard rarely occurs in the buildings, considering it in the design as widely as earthquake is considered is not an economical choice. Nowadays, seismic design is an inseparable part of design and seismic loads are taken into account in the design of many buildings around the world. In steel moment resisting frame buildings, seismic design requires that the beams and columns of the buildings have higher level of strength and stiffness than what is needed to resist the gravity loads. Seismic loads are highly dependent on the seismicity of the zone where the building is located; meaning, higher seismicity requires greater seismic loads to be applied on the building and as a result, stronger sections for the beams and columns of a steel MRF building will be chosen. This excess capacity in the structural members of seismic designed buildings is reserved to be used when the building faces an earthquake; however, it can increase the building's resistance against column loss events as well.

Progressive collapse is relatively a new concern for engineers, and many buildings may need to be retrofitted in order to have sufficient resistance against progressive collapse. Due to limited financial resources and time, developing strategies for evaluating existing buildings' susceptibility to progressive collapse, with the possibility of need to retrofit them, can be invaluable. Correlating the effect of the level of seismic design of steel MRF buildings to their robustness against progressive collapse is seen to be a valuable assessment tool since seismic loads have already been considered in the design of many buildings.

Since extreme loading conditions such as progressive collapse are not normally considered in the design of the buildings, many existing buildings may be vulnerable to this catastrophic event. Therefore, one of the most challenging issues in this field is to choose a proper method for retrofitting. This is especially important for steel MRF buildings since their seismic design is extremely sensitive to any changes in the stiffness of their beams and columns.

1.2 Objectives and Scope of Study

This study has two main objectives: (1) To assess the progressive collapse resistance of seismically designed steel MRF multistory buildings; and (2) To evaluate the effectiveness of two proposed retrofit solutions on mitigating progressive collapse in steel MRF buildings.

To achieve the first objective; Typical multistory office buildings are considered. The buildings are 5, 10, and 15-story typical office buildings with a 3 bays x 6 bays rectangular plan which 6-meter spans in both directions. Each building is designed for three different seismic zones (representing low, medium, and high seismicity). Using the APM, six different instantaneous ground floor column removal scenarios are applied to each building in order to evaluate its vulnerability to progressive collapse.

To achieve the second objective; Two retrofit methods are proposed for the buildings which were susceptible to progressive collapse. These two methods are then applied to the buildings and their effectiveness is evaluated by using APM with the same six ground floor column removal scenarios used in the original buildings.

1.3 Outline of the Thesis

This thesis is presented in seven chapters. The following is a summary of the contents of each chapter:

Chapter 2 reviews some of the notable examples of progressive collapse in the past century. Existing guidelines for studying and designing buildings against progressive collapse will be mentioned with a great focus on the two most popular guidelines; the General Service Administration guidelines (GSA 2003) and the Unified Facility Criteria guidelines (DoD 2009). Also, a literature review of different types and methods of analysis of progressive collapse, tools for performing the analysis, and steel moment resisting frame as a structural system for resisting gravity and seismic loads will be presented as well as some recent related research works on the progressive collapse of buildings.

Chapter 3 describes the properties of the original buildings used in this study. It also explains the design procedures and modeling of the studied buildings in addition to the assumptions made.

Chapter 4 presents the results of nonlinear static and nonlinear dynamic analyses acquired from the ELS software program for the original buildings designed in the three defined seismic zones. Different aspect of the results will be discussed and the importance of using retrofit strategies will be noted.

Chapter 5 addresses the need to retrofit those studied buildings which do not have enough resistance against progressive collapse, and the main deficiency of these buildings will be investigated in this regard. Also, two retrofit methods will be recommended for the buildings which lack resistance; namely the top beams grid and the top gravity truss methods.

Chapter 6 evaluates the effectiveness of the two recommended retrofit methods by discussing the results of nonlinear static and nonlinear dynamic analysis of the buildings retrofitted with these methods. Moreover, Dynamic Increase Factors (DIF) and distribution of gravity loads in the original and retrofitted buildings will also be discussed.

Chapter 7 presents a summary of the study along with the major conclusions as well as recommendations for future studies.

Chapter 2
LITERATURE REVIEW

2.1 Notable Examples of Progressive Collapse around the World

The last few decades witnessed a number of reported examples of collapse around the world. According to the definition presented in Chapter 1, progressive collapse is "a type of collapse initiated from a local failure may be considered a progressive collapse if the local failure is significantly smaller than the eventual collapse." Therefore, not all types of collapses can be categorized as progressive collapse. Among these reported collapses, three major ones which are seen to be good examples of progressive collapse are mentioned in this chapter.

1- On May 16, 1968, the 22 story Ronan Point apartment tower in West Ham, London experienced a partial progressive collapse. It was around 5:45 in the morning that a gas explosion happened on one of the corners of the 18^{th} floor. The explosion was sparked by a lady trying to prepare breakfast on the stove. This explosion blew out the load-bearing flank walls which were the structural support for the 4 other above floors. As the flank walls fell away, the south-east corner of the building suffered a progressive collapse. It is believed that this collapse was a result of weakness in the joints connecting the vertical walls to the floor slabs (Pathe News).

Since the building was new, some of the floors were still unoccupied. Only four people out of the 260 residents of the building were killed and 17 others were injured. Since the extent of damage was not very huge, the collapsed section of the building was rebuilt after the explosion.

2- On April 19, 1995, the Alfred P. Murrah Federal Building in downtown Oklahoma City, Oklahoma, United States became the target of a bombing attack. At 9:02 in the morning a truck full of diesel fuel and explosives was detonated in front of the building and destroyed almost a third of the building and caused severe damage to several other buildings located nearby. As a result of this explosion 168 people were killed and 800 others were injured. The majority of the 168 fatalities were due to the partial collapse of the structure and not to the direct blast effects. Due to severe damages and safety reasons, the building could not be rebuilt or repaired; therefore, they had to demolish it only about a month after the explosion (2004).

3- On September 11, 2001, two hijacked passenger airplanes hit the World Trade Center Towers in New York, United States. The first plane hit the North Tower at 8:46 AM and the second one hit the South Tower at 9:03 AM. It took the North Tower 102 minutes to collapse while the South Tower collapsed 56 minutes after the impact of the plane. These impacts were quite deadly, strong, and destructive, but what is mostly believed to be the main cause of the collapses is the fire which followed the impacts.

The towers were designed as tube-in-a-tube structures. This structural system is able to easily resist lateral loads, and it also provides the tenants with open floor plans uninterrupted by columns or shear walls. According to the performance study of the buildings published by Federal Emergency Management Agency (FEMA) in May 2002, the collapses were caused by the weakened floor joists, which was a result of the fire. This followed by floors detaching from the main structure of the building and falling onto each other initiating a progressive collapse. This type of progressive collapse is sometimes called a progressive pancake collapse. FEMA revised its early investigation after a more detailed and precise investigation was completed by National Institute of Standards and Technology (NIST) in September 2005. While NIST was also blaming the fire as the main cause, it attributed the collapse to the fact that sagging floors pulled the perimeter columns inward. “This led to the inward bowing of the perimeter columns and failure of the south face of WTC 1 and the east face of WTC 2, initiating the collapse of each of the towers.” As a result of this attack, 2752 people were killed including the passengers of the aircrafts and the fire fighters who were performing the rescue operation.

2.2 Existing Codes and Guidelines

As progressive collapse is relatively a new concern for structures, there are still a lot of basic research and work being done in this field, and changes in the leading guidelines happen very frequently. It is also a very rare event compared to other types of hazards for structures. Therefore, building codes do not usually address this issue directly except for general approaches and recommendations to increase progressive collapse resistance. However, there are certain guidelines which are more concerned about progressive collapse and provide detailed design requirements necessary to reduce the progressive collapse potential of structures.

2.2.1 Overview of the GSA 2003 Guidelines

In 2003, the General Service Administration of the United States (GSA) published some guidelines exclusively concerning progressive collapse. These guidelines are titled “Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects”. Since the GSA guidelines are the only guidelines which are explicitly concerned with office buildings, they are adopted to be used in this study. The GSA 2003 seeks three main purposes as exactly mentioned in the guidelines:

- Assist in the reduction of the potential for progressive collapse in new Federal Office Buildings
- Assist in the assessment of the potential for progressive collapse in existing Federal Office Buildings
- Assist in the development of potential upgrades to facilities if required

These guidelines provide a threat-independent method for assessing the buildings' potential for progressive collapse. In other words, they only consider the consequences that any abnormal loading may have on the building rather than the type and characteristics of the load itself. These consequences are mainly seen as instantaneous losses of vertical load bearing members of the building. The application of these guidelines is mandatory for all professionals engaged in the planning and designing of new facilities or building modernization projects for the GSA, and this mainly includes federal office buildings.

Based on the material used in the structural members, the GSA categorizes the structures into concrete and steel structures, and from another perspective, it categorizes them into typical and atypical structures. The GSA 2003 proposes different regulations for any of these categories which will be explained in the next few paragraphs. Atypical structures may have a combination of structural systems, vertical discontinuities or transfer girders, variations in bay sizes or extreme bay sizes, plan irregularities, and/or closely spaced columns. These kinds of irregularities cannot be found in typical structures. The models that have been used in this study are all typical structures.

The GSA guidelines limit the use of simplified analysis method which is the linear procedure to low-to-medium-rise and/or typical buildings. This procedure does not require very strong computer hardware and expensive analytical tools. It is also faster and cheaper and at the same time with satisfactory accuracy. However, for atypical and/or structures that have more than 10 stories above the ground it requires a more sophisticated method which is the nonlinear procedure. A nonlinear procedure implies the use of static or dynamic analysis methods that consider both material and geometric nonlinearity.

The only method for evaluating the progressive collapse potential of the buildings which is proposed in these guidelines is the Alternate Path Method (APM). In this method, a particular column is instantaneously removed and the rest of structure is analyzed after this event. The GSA is only concerned with column losses that may occur in the first story above the ground, and only requires vertical loads to be acting on the building during the analysis. A structure's role is basically to transfer the loads from its different parts to the foundation or supports. According to the direction of the forces in structural members, some virtual load paths can be assumed which transfer the loads through beams and columns to the supports. After the column is lost, the load that was being transferred through that column should be transferred through a new path called the alternate path. Therefore, the APM mainly evaluates the ability of the structure to find and transfer the loads through an alternate path, and while not exceeding certain limitations, remain stable during this procedure. This method is a threat-independent method as it is not concerned about the reason for which the column is lost, and can be exercised by linear static, nonlinear static, linear dynamic or nonlinear dynamic analysis.

As a modeling guidance, the GSA 2003 illustrates the correct and incorrect approach for removing a column in Figure 2.2.

The APM is known to be a very useful method for evaluating the resistance against progressive collapse among researchers and it is widely used (Gudmundsson & Izzuddin, 2010). It is believed to be a better method compared to the tie force method developed earlier as it deals with ductility as well as strength.

The GSA guidelines also provide some design guidance and recommendations for concrete and steel buildings. Application of these recommendations can lead to lower potential for progressive collapse in the buildings. Although the guidance is not the requirement of the guidelines, it may be considered in initial design of the buildings. For concrete structures, the GSA recommends that the structure be designed with enough redundancy, use of detailing to provide structural continuity and ductility, and enough capacity for resisting load reversals and shear failure. It has also some recommendations for initial design of steel structures such as discrete beam-to-beam continuity, designing connections with enough resilience, redundancy, rotational capacity, and strength. It also recommends that the building be designed with global frame redundancy.

In case of conducting dynamic analysis, the load combination shall be equal to dead load plus 25% of the live load, and the time period for removing the vertical element should be less than 1/10 of the period associated with the structural response mode for the vertical element removal. If static analysis is being conducted, the aforementioned load combination should be multiplied by 2 in order to take into account the dynamic effect of the load. The GSA also allows the designers to use some recommended over-strength factors for material in case they are confident in the actual state of the structure's material.

In order to evaluate the results of the linear elastic analysis, the GSA defines the Demand Capacity Ratio (DCR):

$$DCR=\frac{Q_{UD}}{Q_{CE}}$$

Where,

Q_{UD} = Acting force (demand) determined in component or connection/joint (moment, axial force, shear, and possible combined forces)

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

If DCR for a member exceeds 1.0, it means that the force in the member has exceeded the capacity of the member. The GSA proposes certain limits for DCR factor depending on the type of the force (moment, shear) and type of the member (beam, column, connection) and also material (steel, concrete). If the structural can stay within the limits, it means that the probability of progressive collapse is low; otherwise the structure is susceptible to progressive collapse.

In order to evaluate the results of nonlinear analysis, GSA 2003 defines ductility and rotation as the main parameters. Ductility is defined as the ratio of ultimate deflection to elastic deflection (Δ_u/Δ_e), and rotation is the maximum chord rotation of the member with respect to its initial position. If the structural can stay within the presented limits, it means that the probability of progressive collapse is low; otherwise the structure is susceptible to progressive collapse.

Furthermore, the GSA requires the maximum allowable extents of collapse resulting from the instantaneous removal of a primary vertical support member one floor above grade to be limited to smaller of one of the following areas if it is an exterior vertical member:

1- The structural bays directly associated with the instantaneously removed vertical member (column or wall) in the floor directly above the removed vertical member.

2- 1,800 ft^2 at the floor directly above the removed vertical member.

And if it is an interior vertical element:

1- The structural bays directly associated with the instantaneously removed vertical member (column or wall) in the floor directly above the removed vertical member.

2- 3,600 ft^2 at the floor directly above the removed vertical member.

Figure 2.3 illustrates an example of a column lost from the ground floor level and the maximum allowable collapse area.

2.2.2 Overview of the DoD 2009 Guidelines (UFC 4-023-03)

The Department of Defense (DoD) of the United States published a set of guidelines titled "Design of Buildings to Resist Progressive Collapse". This set of guidelines generally uses the approaches mentioned in ASCE 7-05 for design which are the direct design approach and the indirect design approach.

2.2.2.1 Direct design approach

ASCE 7-05 defines the direct method as "*Explicit consideration of resistance to progressive collapse during the design process*". This can be either Alternate Path (AP) or Specific Local Resistance Method (SLR).

1- Alternate Path Method (APM): In this method, as described more in details in section 2.2.1, local failure is allowed, but the building should be capable of providing alternate path for the loads in order to prevent major collapse. This is usually done by removing a vertical load bearing member (such as column or wall), and evaluating the ability of the building to bridge across the removed member.

2- Specific Local Resistance (SLR): This method requires that the building, or parts of it, have sufficient strength to resist a specific load or threat such as blast or vehicle impact. In other words, in SLR or structural hardening method, the nature of the extreme loads which may be applied on the building are defined and specified. Since this method may only engage specific local elements of the building, it can be a cost-effective method. Progressive collapse design is typically a threat-independent procedure, but in SLR method the threat should be known. This is the main shortcoming of this method as the threat information may be considered classified, restricting its use by the general public.

However, a threat-independent SLR method is presented in this version of the DoD guidelines which is referred to as Enhanced Local Resistance (ELR) method. This mainly concerns with increasing the level of protection of the perimeter columns or walls to provide a better shield for the building against threats from outside. The shear capacity of these columns or walls and their connections is required to be greater than their flexural capacity. This provides a ductile and more controlled failure mode for the building.

2.2.2.2 Indirect Design Approach

The indirect method is defined in ASCE 7-05 as “Implicit consideration of resistance to progressive collapse during the design process through the provision of minimum levels of strength, continuity, and ductility”. This design approach improves strength, continuity, and ductility which overall, increase the integrity of the building. This is done by applying some general provisions such as designing a good plan layout, changing span directions of floor slabs, using load-bearing interior partitions, increasing the capacity of the floor slab for catenary action, using redundant structural systems, ductile detailing, and additional reinforcement for blast and load reversal.

The DoD guidelines specifies minimum tensile forces that must be used to tie the structure together in order to enhance continuity, ductility, and structural redundancy. The Tie Forces method is the only indirect method introduced in DoD guidelines.

2.2.2.3 Occupancy Category

The DoD guidelines uses the Occupancy Categories (OC) defined in UFC 3-301-01 (2010) for structural engineering. These are the same categories used in the International Building Code (IBC 2009) and they are shown in Table 2.2. The categories used in DoD guidelines are listed in Table 2.1.

According to the occupancy category of the building, DoD requires the designers to choose specific methods to design against progressive collapse. The provisions and design requirements are shown in Table 2.3.

2.3 Types of Analysis for Progressive Collapse

Choosing the type of analysis is a very important decision that has to be made in order to start a progressive collapse study. Different parameters can affect this decision such as code or guideline's requirements, structural characteristics of the building, desired accuracy for analysis, time limitations, and the availability of tools for analysis. Currently, there are four main types of analysis for studying progressive collapse cases; linear static, nonlinear static, linear dynamic, and nonlinear dynamic analysis.

2.3.1 Linear Static Analysis: Among the aforementioned types of analysis, this method is the fastest and easiest to perform. As it is a linear procedure, it does not consider the nonlinearity of material and geometry. This type of analysis is mostly allowed by the codes and guidelines in cases that the structure features simple configurations with no irregularities.

If the applied loads on the structure are small enough to create internal forces which have less magnitude than the yielding forces of the members, this type of analysis can be performed in only one step, and the results are sufficiently accurate. As in progressive collapse many structural members may reach or get close to their ultimate capacity, application of the linear static analysis must be a step by step procedure. This means that the magnitude of the loads applied to the structure should be increased in steps until the maximum internal force which is equal to the capacity of the members occurs. In many cases, the maximum internal force is considered to be the plastic moment of the members. Through these steps, the material and geometry are considered to respond in a linear manner. As soon as a section reaches the plastic moment, it can be replaced by a hinge and a concentrated moment equal to the plastic moment on the edge of it. This process continues as long as the structure remains stable. A simple form of instability can be a member having three hinges in a row which forms a mechanism.

2.3.2 Nonlinear Static Analysis: In this type of analysis the loads are considered to be static loads while the nonlinearity of the material and geometry are taken into account. The analysis is a step by step process. Although each step is a linear analysis, the nonlinearity is taken into account by changing the properties of the structure from a step to another. This means that the accuracy can be improved by increasing the number of the steps. It should be mentioned that increasing the number of the steps has notable effect on the time required for analysis. This type of analysis is very similar to the pushover analysis process except for the loads being usually applied downwards to structure as it is a case of progressive collapse. The nonlinear static analysis is an extremely useful tool to specify the behavior of the structure until failure. When this type of analysis is used for progressive collapse studies, the loads are usually multiplied by certain factors in order to take into account their dynamic nature. These factors have been the subject of research for a while (Ruth et al. (2006)), and in many cases they can lead to over-conservative results.

2.3.3 Linear Dynamic Analysis: This type of analysis is rarely used for progressive collapse. Although it does consider the dynamic nature of the loads during a progressive collapse study, the fact that it does not consider the nonlinearity is a big deficiency. If the magnitudes of the loads are small enough to keep the structure in its linear zone, the result will be reliable, but when the structure experiences large deformations (which is quite usual in progressive collapse) and enters the nonlinear zone the results lose their accuracy.

2.3.4 Nonlinear Dynamic Analysis: Progressive collapse is a phenomenon which deals with dynamic loads, nonlinear and plastic deformations, element separations, and collisions. Among the different types of analysis, nonlinear dynamic time history analysis is the most effective and accurate one to study this phenomenon as it considers the dynamic nature of the loads as well as nonlinearity in material and geometry. This analysis process is very rigorous and time consuming, but with the development of software packages and computer hardware, it is gaining its popularity in the field of progressive collapse.

If the computer software and hardware is not very limited, a combination of the nonlinear static and nonlinear dynamic analysis gives the most detailed information about the behavior of a structure during a progressive collapse. The nonlinear static analysis identifies the structure's behavior until its failure. The force-deformation diagram of the structure is one of the most valuable results of a nonlinear static analysis. It is a very useful piece of information because it shows the ultimate force and deformation of the structure right before failure. The nonlinear dynamic analysis on the other hand, gives the most accurate response of the structure to any cases of extreme loading such as progressive collapse. However, this type of analysis is highly dependent on the loads, and if the loads are changed, the analysis should be conducted again.

2.4 Methods Used for Analysis of Progressive Collapse

Finite Element Method (FEM) has been used for conducting analysis in different engineering fields for over half a century. In order to reach a higher level of efficiency, engineers of different fields have applied modifications to this method and customized it for their different purposes. FEM is widely used in many civil engineering software packages, and it is a powerful method to consider nonlinearity and dynamic effects. In this method, structural members are divided into a series of element which form a mesh. The number of elements depends on several factors, one of which is the desired level of accuracy. By increasing the number of elements, the results can be more accurate, but the analysis will be more rigorous. Each element has certain amount of stiffness, and it is connected to the other elements through some nodes. Depending on the problem, each node has a number of degrees of freedom. Since the adjacent elements share some nodes, in case of separation, different IDs should be assigned to the separated nodes which may develop difficulties for analyzing process.

Progressive collapse is a phenomenon which deals with a lot of element separations and collisions which are very difficult and at some points impossible to consider in FEM. Therefore, although FEM is a powerful and efficient method for structural analysis when there is no failure, it is not the best method for progressive collapse studies.

Applied Element Method (AEM) is relatively a new method of analysis which is believed to be more effective in progressive collapse research. With development of computer software and hardware, this method is gaining more attention in the research community as well as engineering fields. Quite similar to FEM, structural members are divided into a number of elements which form a mesh. Each element face is connected to the adjacent element's face through some contact points. Each contact point includes three springs; one to transfer the axial force and two transverse ones to transfer the shear. While elements are rigid, the springs represent the stiffness, and the degrees of freedom are located at the center of gravity of the elements. When progressive collapse study is intended, AEM has three main superiorities to FEM:

1- Connectivity of the elements: As in FEM elements are connected at the nodes, element separation may cause stress singularity. However, in AEM, when a spring reaches its ultimate capacity, it is simply removed and the stiffness matrixes are updated. This also makes the AEM capable of considering partial separation of the elements since some of the springs may fail while the other springs on the same element's face can still be working.

2- Transition elements: In FEM's mesh, when we are switching from a large element to a smaller one, we need to use some transition medium-sized elements in order to keep the connectivity of the elements at the nodes. But in AEM, there is no need for the transition elements as the elements are connected by springs located in their surface rather than nodes.

3- Meshing: For the same reason explained in the previous section, when several objects are connected at a certain point, meshing becomes very complicated yet very important to be done correctly. In AEM, however, each object can be meshed completely independent of the other objects, and the only thing controlled by meshing is the accuracy of the analysis.

2.5 Analysis Tool for Progressive Collapse

There are currently several commercial software packages available for conducting nonlinear static and dynamic analysis. In the field of progressive collapse studies, various types of computer programs have been used, most of which are based on FEM. As explained in the previous section AEM is a more effective method compared to FEM in progressive collapse studies. Therefore, a software package called "Extreme Loading for Structures (ELS)" which is developed based on AEM is used in this study.

ELS is an advanced 3D nonlinear software tool developed by "Applied Science International (ASI)". It can perform nonlinear static and nonlinear dynamic analysis. ELS allows structural engineers to study the effect of dynamic loads and such as blast, seismic, impact, progressive collapse, and wind. Since ELS is based on AEM, it is capable of automatically analyzing structural behavior and considering several structural phenomena such as yielding of the reinforcement, detection and generation of plastic hinges, buckling and post-buckling, crack propagation, membrane action and P-Delta effect, separation of the elements, and all the process of structural failure before, while, and after the collapse.

2.6 Steel Moment Resisting Frames

Moment Resisting Frame (MRF) is a structural system which has been widely used in both new and old buildings. Building codes limit the height or number of stories of buildings which use MRF as their structural system. This limit is usually around 25 stories. MRFs can be built using steel or concrete; however this study is only concerned with steel MRF.

MRFs have three main components; columns, beams, and connections. The connections are capable of transferring moment between beams and columns. This is the main feature of MRF which differentiates it from many other structural systems. Transferring the moment makes use of flexural capacity of the both beams and columns when the building is under vertical (gravity) or lateral loads. Figures 2.7, 2.8, and 2.9

show a schematic view of moment and shear distribution in MRFs under the gravity loads.

MRFs are very economical and effective systems for gravity loads. As they distribute the moment along the beams in the form of both positive and negative moments, they help to reduce the size of the beams sections. However, it is the lateral loads such as wind and earthquake which in most cases apply limitation on this system. MRFs are laterally flexible systems, and since controlling the lateral displacements and drifts is a very important requirement in the building codes, they are not the most economical systems to be designed against lateral loads. This shortcoming is one of the reasons that codes usually limit the height of moment resisting frame buildings. In order to meet the codes requirements for lateral loads, stiffness of beams and columns should be significantly increased and this generally leads to bigger and heavier sections. Therefore, in the medium or high seismic zones where the seismic forces are considerable, it is the lateral force which governs the design of a moment resisting frames.

Apart from the required lateral stiffness, the beam to column stiffness and capacity ratio is also very important. Since columns are the main members in a MRF building that transfer the vertical loads to the supports, their stability is extremely vital and more important than that of the beams. Therefore, the design must make the building capable of forming plastic hinges in the beams rather than columns when the building is under lateral loads. This guaranties the overall stability of the building. As a result, much heavier sections for columns are required compared to the case that columns are only designed based on their stress/capacity ratios due to different load combinations.

To put is briefly; considering only the gravity loads for the design, a seismically designed steel frame can be considered highly overdesigned. Since the current guidelines mostly recommend only the application of gravity loads on the structures for progressive collapse analysis, the excessive stiffness and capacity of the SMRF members due to performing seismic design can be used to provide resistance against progressive collapse.

2.6.1 Robustness of Moment Resisting Steel Frames:

Robustness is defined as the ability of a structure to withstand events like fire, explosions, impact or consequences of human error, without being damaged to an extent disproportionate to the original cause, i.e. without progressive collapse. Modern building codes treat robustness with two different strategies: 1) increasing continuity and enhancing the load distribution ability after a member loss; and 2) increasing the specific local resistance of the key elements to accommodate accidents. There can be major considerations in the design in order to increase the robustness of a structure.

- Redundancy: The main method for evaluating the progressive collapse resistance of a structure is the Alternate Path Method. As explained earlier in this chapter, in order to survive a column loss event, the structure needs to transfer the forces of the lost column through new load Paths. By increasing the redundancy in the design of the buildings, there will be more load paths available which provide a higher level of resistance against progressive collapse.

- Ductility: During a column loss event, the structural members experience extreme conditions such as large deformations, formation of plastic hinges, and force redistributions. By providing enough ductility in the design of structures, they will be capable of maintaining their strength and stability through all the extreme conditions.

Ductility of the structure may be provided by using ductile and continuous connectivity between beams and columns and incorporating the weak beam-strong column principle.

- Ties: In order for a structure to transfer the loads of a lost major load bearing member through alternate paths, affected members must be able to transfer certain amount of load from one to another. These forces are often called "Tie Forces" and by maintaining the structure's integrity, basically tie the members together. As shown in Figure 2.4, the tie forces between the members can be in different directions including vertical and horizontal.

2.6.2 Beams and Columns in a Steel Moment Resisting Frame:

Beams are capable of resisting the loads by their main two actions; flexural or moment resisting action and catenary or axial force action. In the design of steel MRFs, only the flexural capacity of the beam is usually considered, because in order for a beam to have axial forces, large relative displacements between its both ends must occur. This relative displacement can be either along the beam's direction or perpendicular to the beam's direction. Rigid diaphragms, which are usually formed by the floor slab, result in zero relative displacements between the beam's ends. However, there are rare cases like column removal scenarios that can create these relative displacements. Figure 2.5 and 2.6 show schematic views of formation of plastic hinges and catenary forces in the beams and also in the floor slabs.

In a column loss event, if the beams are pinned at both ends, they will have no flexural resistance against the loads, and it is only the axial force action that provides resistance against such events. On the other hand, if the beams are fixed at both ends as they are in MRFs, they resist against a column loss event by their flexural capacity until the plastic hinges are formed at both ends. At this point, beams have provided their full flexural capacity and start to increase their axial forces to maintain the structure's stability.

Catenary action of the beams is highly dependent on their section properties and characteristics of the connections. In order for catenary action to appear, the beams should be able to experience large deflections without failure. Many steel sections cannot provide such large deflections; therefore the amount of their exerted axial force is not considerable. Furthermore, the connections must provide enough capacity and rotation for the beams. As a result, in some progressive collapse studies, researchers neglect the catenary action of the beams. This makes the analyses easier to perform while the results will be conservative.

If the beams of a building are capable of bridging over the removed column, it means that the building is able to transfer the load of the removed column through alternate paths. This phenomenon increases the axial force in adjacent columns significantly. Moreover, the columns should be able to resist the moment and axial force coming from the beams which are connected to point of the removed column. Incorporating the weak-beam-strong-column principle in the design provides the columns with enough flexural capacity to resist the huge moments coming from the beams. It is important to mention that the forces related to the catenary action of the beams should also be absorbed by the columns.

2.6.3 Connections in a Steel Moment Resisting Frame:

Theoretically, the connections can be categorized into three groups; pin connections, rigid or fixed connections, and semi-rigid connections. Pin connections are not able to transfer moment between structural members. In this type of connection, the members can have relative rotations. Rigid or fixed connections, on the other hand, are able to transfer moments completely between the members and do not allow for relative rotation. Semi-rigid connections are able to transfer the moments while they allow for relative rotations to some extent. Studies have shown that a real steel connection is neither rigid nor pin (Kameshki & Saka, 2003). In other words, there is no fully rigid or fully pinned connection. It is also shown through experience that the relationship between the applied moment and rotation of the connections can be nonlinear (Kameshki & Saka, 2003). Figure 2.12 shows a few examples of this nonlinear relation in different types of connections.

2.7 Previous Research on Progressive Collapse

In the past decade, progressive collapse has attracted a noticeable attention by the researchers around the world. In this section a review of some of the recent related research efforts on this topic is presented.

Li and El-Tawil (2011) studied the effect of three-dimensional modeling in progressive collapse studies. They made three models of a 10-story steel MRF building; a 3-D model with slabs, a 3-D model without slabs, and a 2-D model of one of the frames of the building. Using the APM, they found that three-dimensional modeling is crucial when a progressive collapse is predicted. If the building is not close to the collapse stage in a column removal scenario, 2-D models can have reasonable results, but they can also be conservative in some cases. They also found the effect of slabs to be of very importance in reducing the progressive collapse potential of the building. They concluded that although slabs can help to enhance the integrity of the structure and redistribute the loads in a column removal event, they may help to spread the collapse in the structure if the initial collapse is not prevented.

When 3-D frames are subjected to column removals, both the longitudinal and transverse frames are affected at the same time and the forces will be distributed between the frames according to their relative stiffness. Lin et al. (2011) investigated the effect of differences between the spans of the longitudinal and the transverse frames of several seismically designed RC frame buildings on their vulnerability to progressive collapse. They concluded that when the spans of the longitudinal and the transverse frames are significantly different, the beams with shorter spans in the vicinity of the removed column attract more forces due to their higher relative stiffness. This concentration of the forces can increase the vulnerability of these buildings to progressive collapse.

Izzuddin et al. (2008) proposed a simplified framework for progressive collapse assessment of multi-story buildings based on the column removal as a design method. Their proposed assessment framework includes the determination of the nonlinear static response, dynamic assessment using a simplified approach, and ductility assessment. They stipulated "*the proposed framework offers a rational system-level approach for assessing the potential of a building structure to collapse under sudden column loss, and it could in due course replace the 'tying force' requirements and the 'notional member removal' provisions employed in current design codes.*".

J-Kim and T-Kim (2009) investigated the progressive collapse resistance of steel MRFs using the APM which is recommended in GSA and DoD guidelines. They studied 3, 6, and 15-story steel frame buildings designed for gravity loads only, and for gravity and seismic loads. They made a comparison of linear static, linear dynamic, and nonlinear dynamic analyses. They found the seismically designed steel frames to be less vulnerable to progressive collapse than the gravity frames. It was also observed that although the linear analysis resulted in smaller deflections in the buildings compared to nonlinear dynamic analysis, using the provisions of GSA and DoD guidelines, it is seen to be more conservative. They also mentioned that in their study, the nonlinear dynamic analysis results were more dependent on variables such as the applied load, location of the removed column, and number of stories.

In case of a column removal event, buildings may experience large deflections. Where the connections have sufficient strength, these large deformations can activate a new form of resistance which is the catenary action of the beams. J-Kim and An (2009) evaluated the progressive collapse resistance of steel frame buildings considering the catenary action of their beams. They carried out the nonlinear static and nonlinear dynamic analysis of 3 and 6-story steel frame buildings with or without braces based on the recommendations of GSA 2003 guidelines. They found that catenary action generally increases the resistance of the building against progressive collapse. They also found that although the increase in the number of stories reduces the progressive collapse potential of the buildings it does not affect the catenary action significantly. It was observed through their study that by increasing the constraint for lateral movement of the frames, the catenary action appears more effectively.

The GSA guidelines present two load combinations for progressive collapse studies; one for static and one for dynamic analysis. The static load combination is equal to the dynamic load combination multiplied by a factor of 2. The factor of 2 is used to represent the dynamic inertial effect. In linear elastic cases, the factor of 2 seems to be a reasonable factor, but as mentioned before, in an extreme case such as progressive collapse many members of the structure enter their nonlinear zone and experience inelastic deformations. Ruth et al. (2006) studied several analytical models and found that the factor of 2 as the dynamic amplification or the dynamic increase factor recommended by GSA is conservative as it does not consider the significant plastic deformations. Instead, they stipulated that a factor closer to 1.5 can better represent the dynamic effect especially in steel MRFs. In another more recent research effort Tsai and Lin (2008) evaluated the progressive collapse resistance of a seismically designed RC building using the GSA guidelines. They also found that the dynamic amplification factor of 2 is a conservative factor for inelastic cases and should only be used for elastic analysis. They mentioned that GSA can use a separate load combination for inelastic analysis.

Fu (2009) modeled a 20-story building using the software package ABAQUS which is capable of performing nonlinear dynamic analysis. The building featured a core shear wall and steel frames in order to resist seismic loads. In the study, several column removal scenarios were applied to the building and it was found that the dynamic response of the structure is mainly related to the affected loading area after the column removal. It was also concluded that column removals at higher levels may induce larger vertical; displacement than column removals at ground level.

Although AEM is believed to be a better method for analysis related to progressive collapse compared to FEM, among the current available research, application of AEM seems to be less than FEM. Galal and El-Sawy (2010) made a comprehensive study on different retrofit strategies that can be employed to prevent progressive collapse in steel frame buildings. A high-rise steel MRF building of 18 stories was used with different bay spans of 5, 6, 7.5, and 9 meters. They evaluated the effectiveness of three retrofit strategy aimed to improve progressive collapse resistance; increasing the beams' strength only, stiffness only, and both strength and stiffness. They performed nonlinear dynamic analysis using the software package ELS (Extreme Loading for Structures) which is based on AEM. They ended up with quantitative results for chord rotation, tie forces, and displacement ductility demand in six ground floor column removal scenarios. Based on the results, they concluded that increasing beams' strength only is a more effective retrofit strategy compared to increasing beams' stiffness only when enhancing progressive collapse resistance is intended.

As explained in section 2.6, beams can resist a column removal event by their flexural and catenary action. Unlike the flexural resistance, catenary resistance of the beams requires certain conditions to in order to be considerable in the overall resistance of the frame. One of the most important requirements is the ability of the connections to undergo large rotations. Khandelwal and El-Tawil (2007) investigated the collapse behavior of steel connections in special moment resisting frames. They mentioned that their special steel MRF was ductile enough to form catenary action when column removal scenarios were applied. They also concluded that ductility and strength of the frame were adversely influenced by an increase in beam depth and an increase in the yield to ultimate strength. This is mainly because these increases can decrease plastic deformations of the members before failure. In another study (2011), they presented a technique to investigate the robustness of building systems by computing residual capacity and establishing collapse modes of a damaged structure. This technique is based on the pushdown analysis which is the same as pushover analysis but in the downward direction. Two dimensional 10-story special and intermediate steel MRF buildings were used in their study. They made a comparison between their three suggested types of analysis; uniform pushdown, bay pushdown and incremental dynamic pushdown analysis. Although many would imagine that when a building experiences large deflection in a column removal event, tensile forces will be exerted in the beams, they observed that many beams of their model buildings experience compressive force. They attributed this observation to the global framing action of the buildings. While mentioning the dynamic amplification factors of GSA 2003 guidelines to be over conservative, they found the DoD 2009 values to be more realistic. They observed better performance from special MRFs compared to intermediate MRFs and explained it by the fact that the special MRFs featured stronger columns, RBS connections, and better seismic detailing. They suggested the use of fuses in the structures and a designing manner in which the propagation of the collapse is explicitly prohibited.

Among the current literature for progressive collapse, some research works address the effect of seismic design on progressive collapse resistance of buildings. However, they mostly investigate different types of structural systems rather than the effect of different seismicity levels on the resistance of the buildings. In addition, seismically designed pure steel moment resisting frame buildings (without other compound structural systems such as braces, trusses, shear walls) are seemed to be rarely studied for progressive collapse from the seismic perspective.

Table 2.1 Occupancy Categories (DoD 2009)

Occupancy Category	**Nature of Occupancy**
I	• Buildings in Occupancy Category I in UFC 3-301-01 • Low Occupancy Buildings
II	• Buildings in Occupancy Category II in UFC 3-301-01 • Inhabited buildings with less than 50 personnel, primary gathering buildings, billeting, and high occupancy family housing
III	• Buildings in Occupancy Category III in UFC 3-301-01
IV	• Buildings in Occupancy Category IV in UFC 3-301-01 • Buildings in Occupancy Category V in UFC 3-301-01

Table 2.2 Occupancy Categories defined in UFC 3-301-01 for structural engineering (2010)

Occupancy Category	**Nature of Occupancy**
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities
II	Buildings and other structures except those listed in Categories I, III, IV and V
III	Buildings and other structures that represent a substantial hazard to human life or

	represent significant economic loss in the event of failure, including, but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300 people • Buildings and other structures containing elementary school, secondary school, or daycare facilities with an occupant load greater than 250 • Buildings and other structures with an occupant load greater than 500 • Group I-2 occupancies with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities • Group I-3 occupancies • Power-generating stations; water treatment facilities for potable water, waste water treatment facilities, and other public utility facilities that are not included in Categories IV and V • Buildings and other structures not included in Categories IV and V containing sufficient quantities of toxic, flammable, or explosive substances to be dangerous to the public if released • Facilities having high-value equipment, as designated by the authority having jurisdiction
IV	Buildings and other structures designed as essential facilities, including, but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities • Fire, rescue, and police stations, and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers, and other facilities required for emergency response

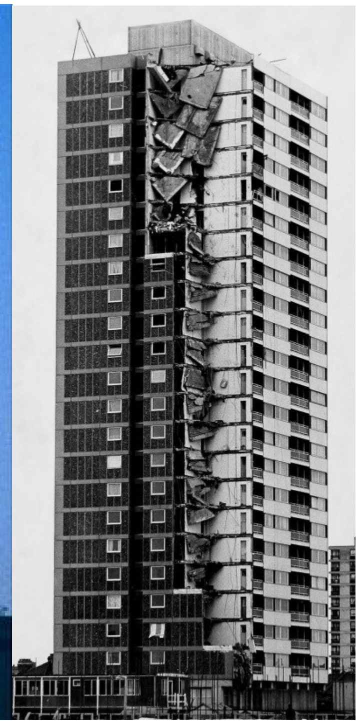
	• Emergency backup power-generating facilities required for primary power for Category IV • Power-generating stations and other utility facilities required for primary power for Category IV, if emergency backup power generating facilities are not available • Structures containing highly toxic materials as defined by Section 307, where the quantity of material exceeds the maximum allowable quantities of Table 307.7(2) • Aviation control towers and air traffic control centers required for post-earthquake operations where lack of system redundancy does not allow for immediate control of airspace and the use of alternate temporary control facilities is not feasible. Contact the authority having jurisdiction for additional guidance. • Emergency aircraft hangars that house aircraft required for post-earthquake emergency response; if no suitable back up facilities exist • Buildings and other structures not included in Category V, having DoD mission-essential command, control, primary communications, data handling, and intelligence functions that are not duplicated at geographically separate locations, as designated by the using agency • Water storage facilities and pump stations required to maintain water pressure for fire suppression
V	Facilities designed as national strategic military assets, including, but not limited to: • Key national defense assets (e.g. National Missile Defense facilities), as designated by the authority having jurisdiction.

	• Facilities involved in operational missile control, launch, tracking, or other critical defense capabilities • Emergency backup power-generating facilities required for primary power for Category V occupancy • Power-generating stations and other utility facilities required for primary power for Category V occupancy, if emergency backup power generating facilities are not available • Facilities involved in storage, handling, or processing of nuclear, chemical, biological, or radiological materials, where structural failure could have widespread catastrophic consequences, as designated by the authority having jurisdiction.

Table 2.3 Occupancy Categories and Design Requirements (DoD 2009)

Occupancy Category	**Design Requirement**
I	No specific requirements
II	Option 1: Tie Forces for the entire structure and Enhanced Local Resistance for the corner and penultimate columns or walls at the first story. **OR** Option 2: Alternate Path for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first story columns or walls.
IV	Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.

(a)



(b)

(c)

Figure 2.1 Notable progressive collapse events: a) World Trade Center towers, USA 2001, (Reuters); b) Ronan Point building, UK 1968 (Modern Structural Analysis webpage); c) Alfred P. Murrah Federal Building, USA 1995 (Associated Press)

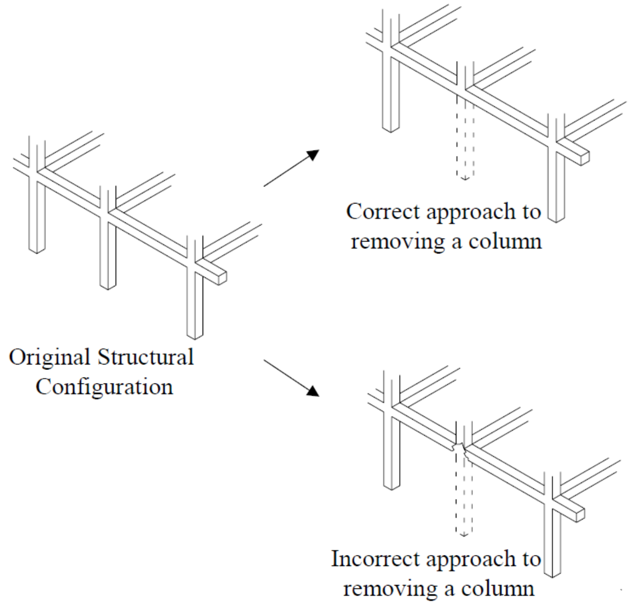


Figure 2.2 Sketch of the correct and incorrect approach for removing a column (GSA 2003)

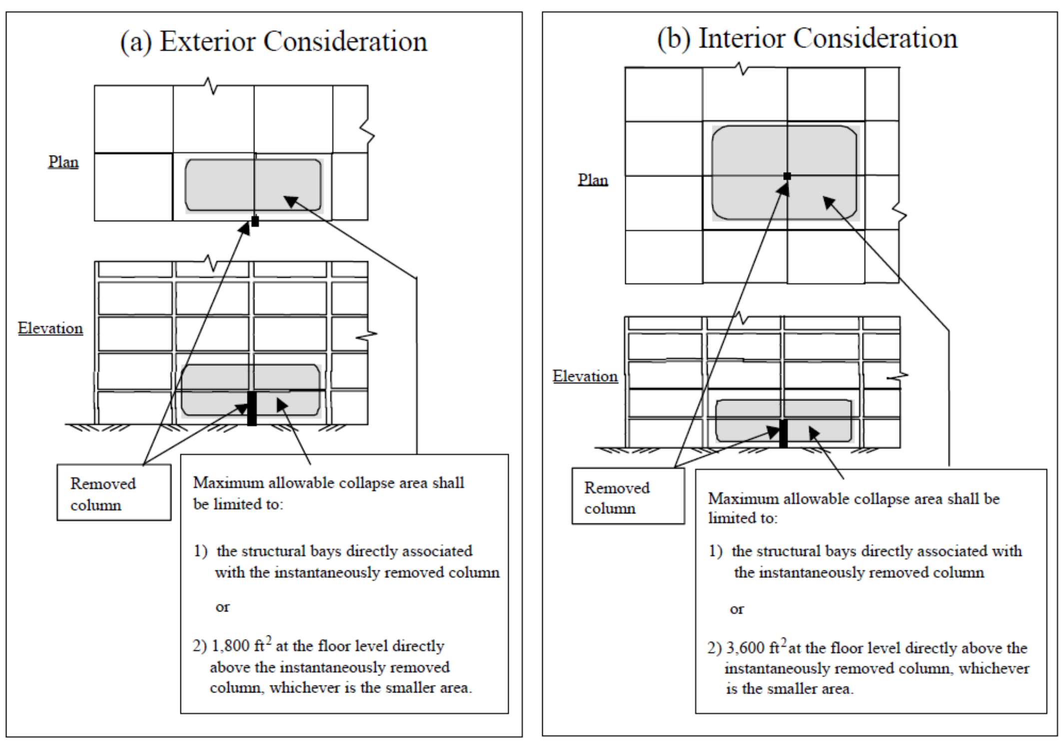


Figure 2.3 An example of maximum allowable collapse areas for a structure if a column is lost (GSA 2003)

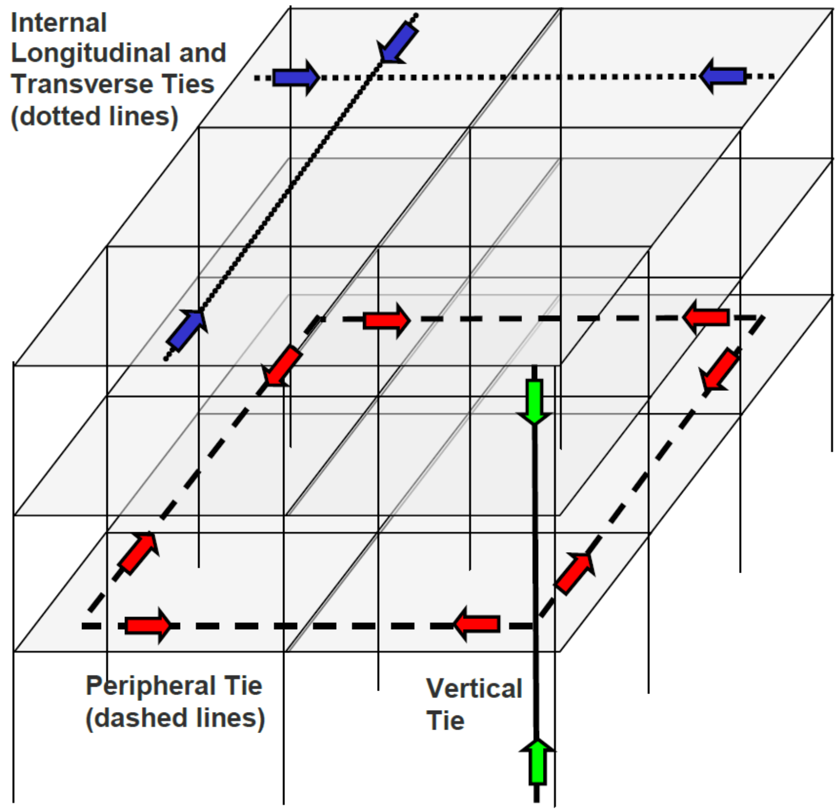


Figure 2.4 Tie Forces in a frame Structure (DoD 2009)

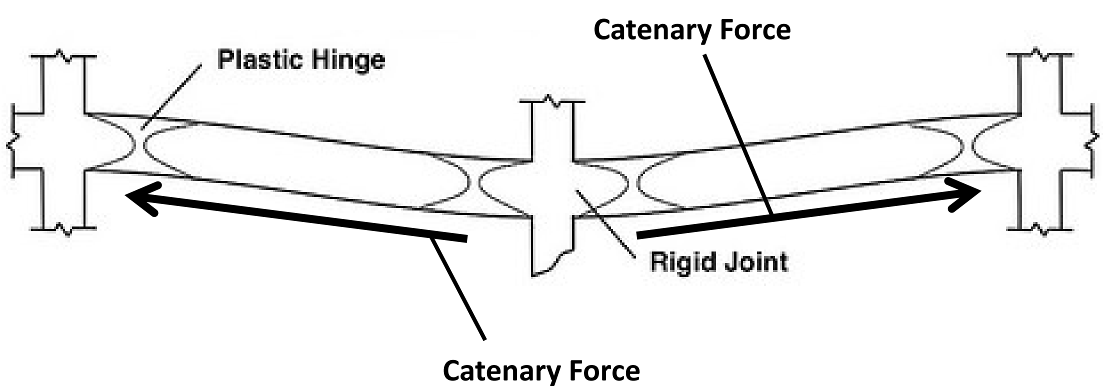


Figure 2.5 A schematic of plastic hinges and catenary forces in the beams (Will It Stand weblog)

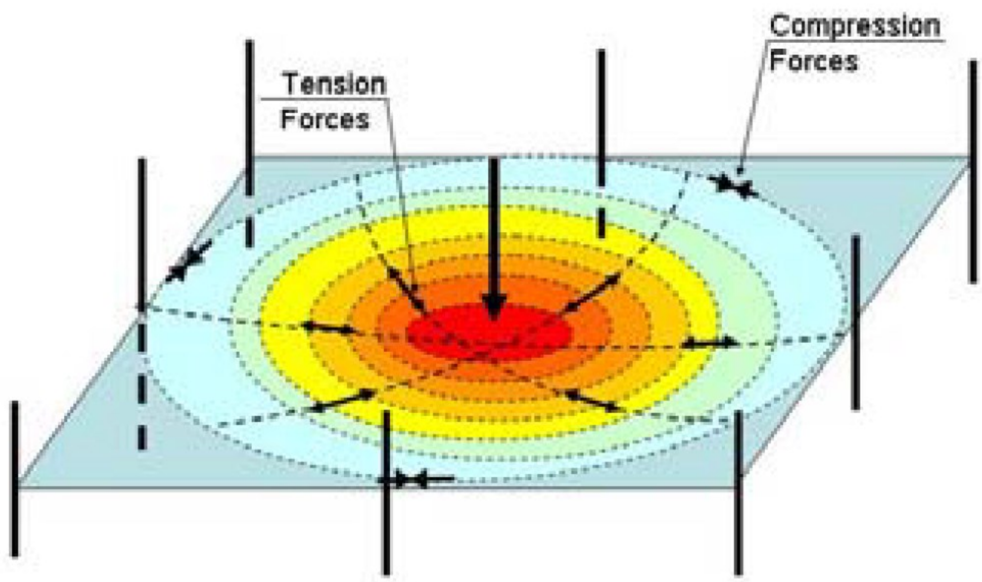


Figure 2.6 A schematic of catenary forces in the slabs (Rahimian & Moazami (2009))

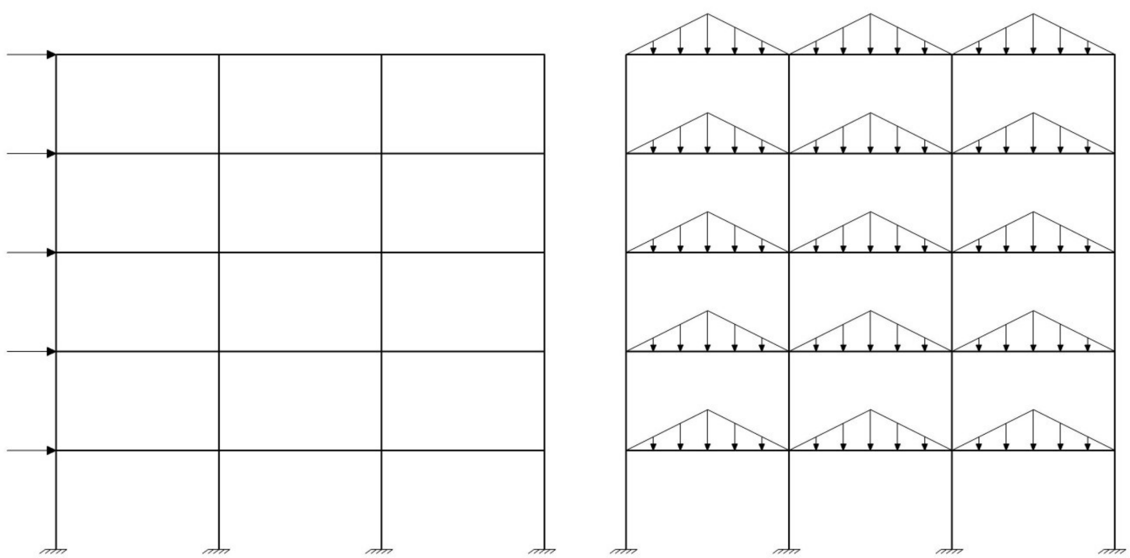


Figure 2.7 Schematic view of loads on the beams along the short span of a typical steel frame with two-way slabs on: lateral loads (left), gravity loads (right)

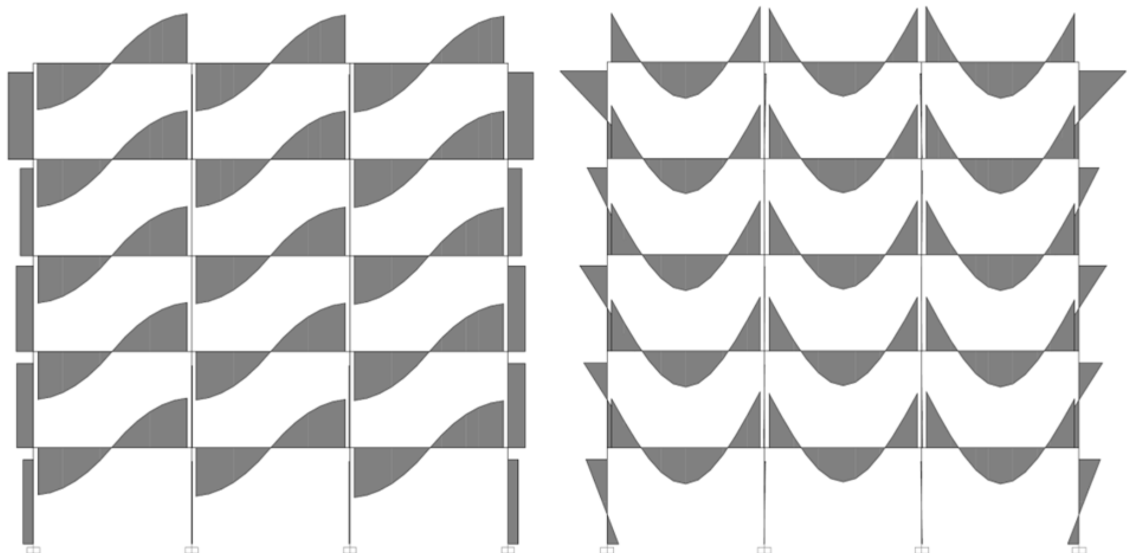


Figure 2.8 Schematic view of internal forces of a steel frame under triangular gravity loads: shear (left), moment (right)

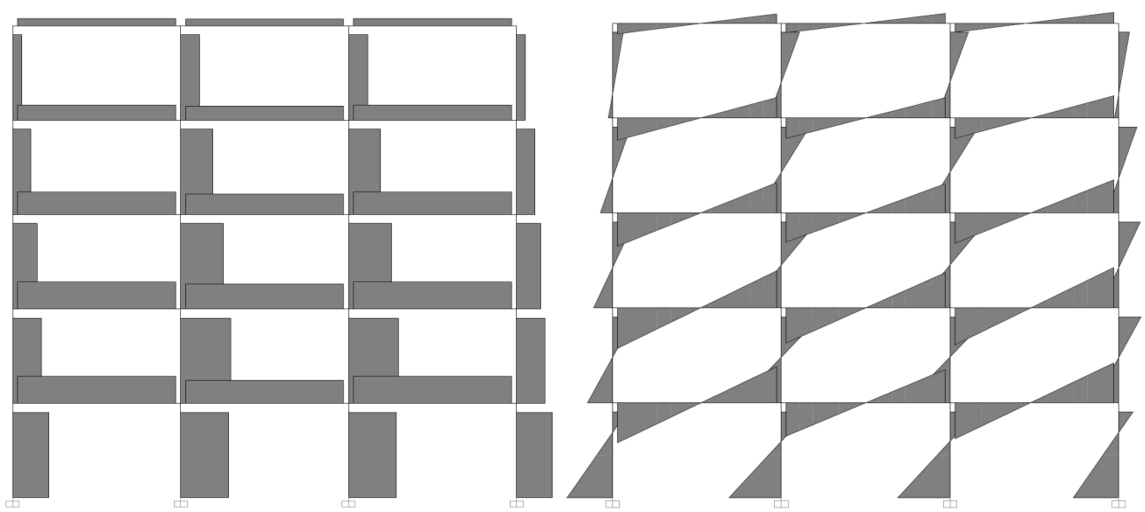


Figure 2.9 Schematic view of internal forces of a steel frame under lateral loads: shear (left), moment (right)

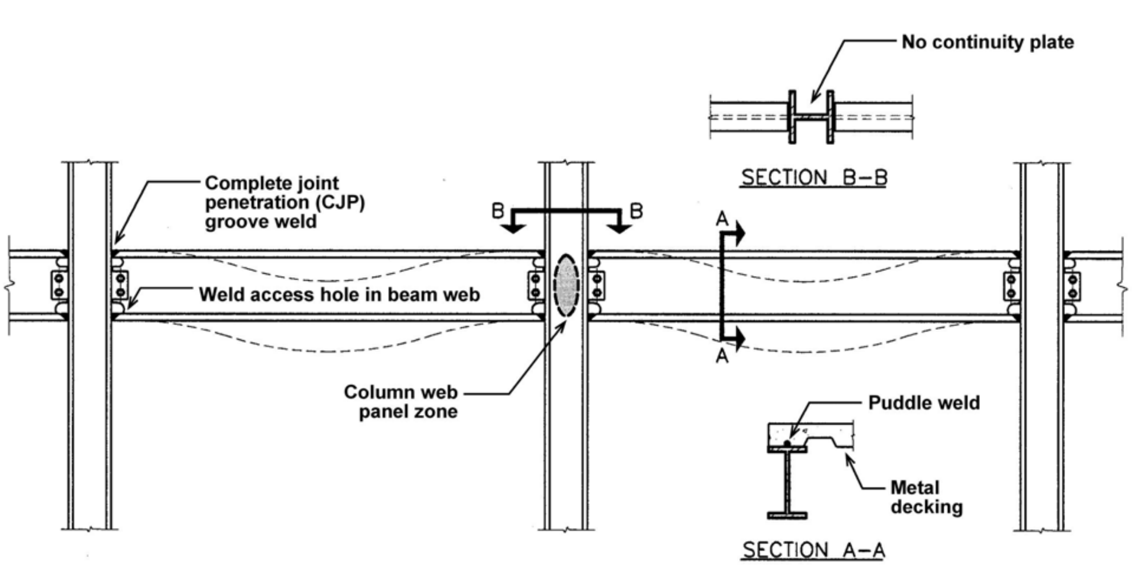


Figure 2.10 A sketch depicting a steel frame beam-to-column-to-beam 'traditional' moment connection scheme prior to removal of primary column support (GSA 2003)

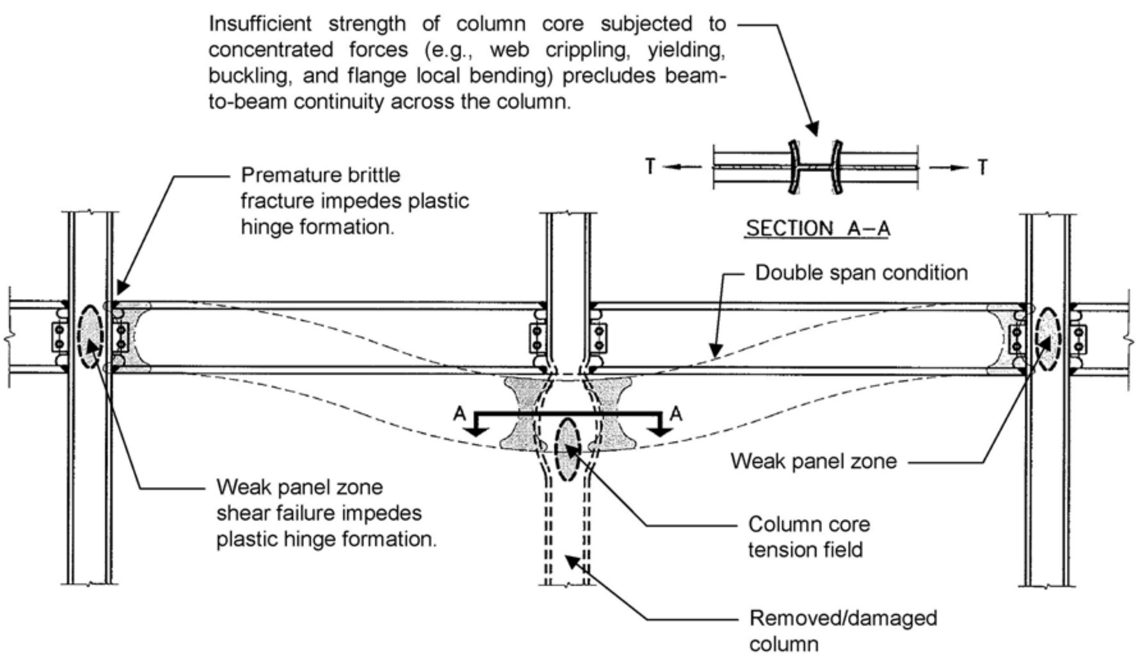


Figure 2.11 Response of the framing scheme shown in Figure 2.10, after the loss of primary column support, shows the inability to protect against progressive collapse (GSA 2003)

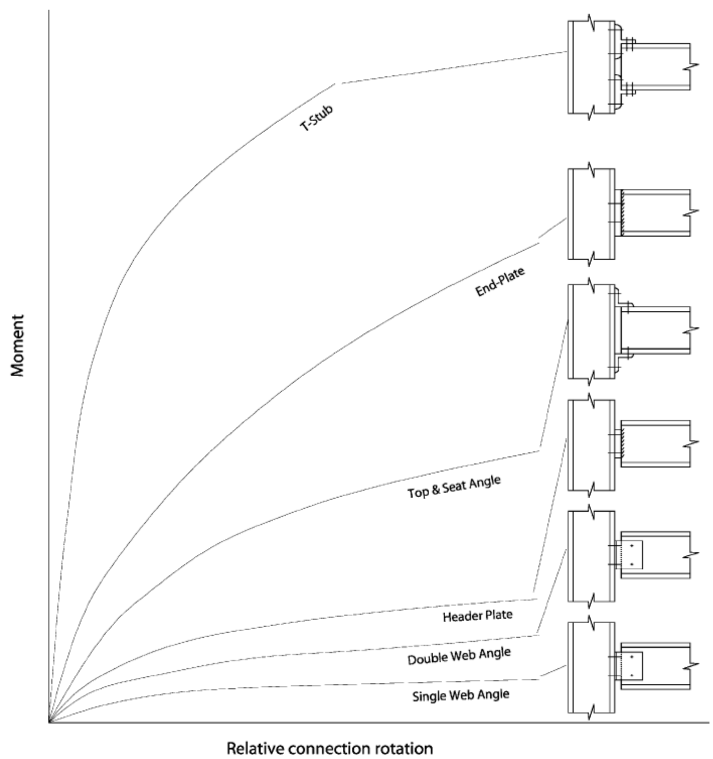


Figure 2.12 Connection Moment-Rotation Curve (Kameshki & Saka, 2003)

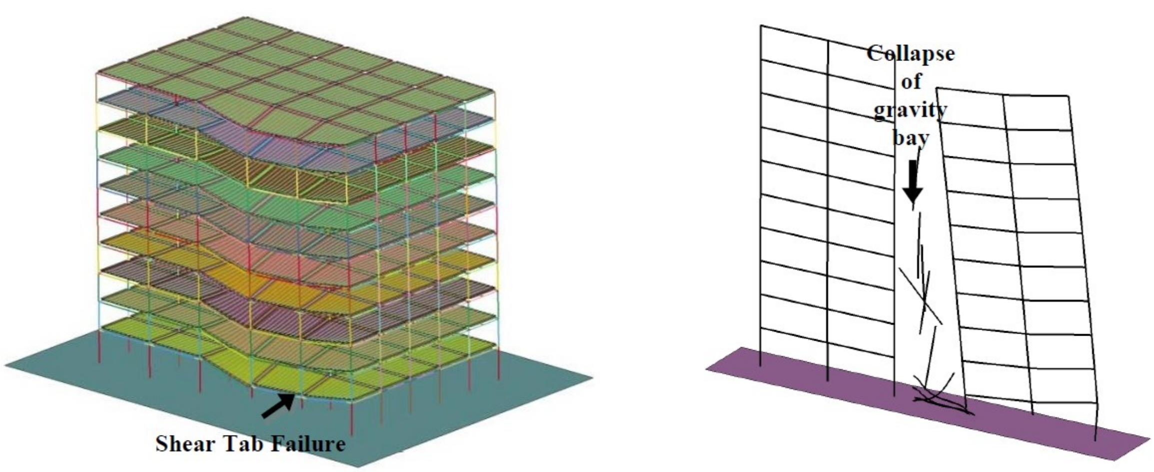


Figure 2.13 Two and three-dimensional models subjected to column loss scenario (Li & El-Tawil, 2011)

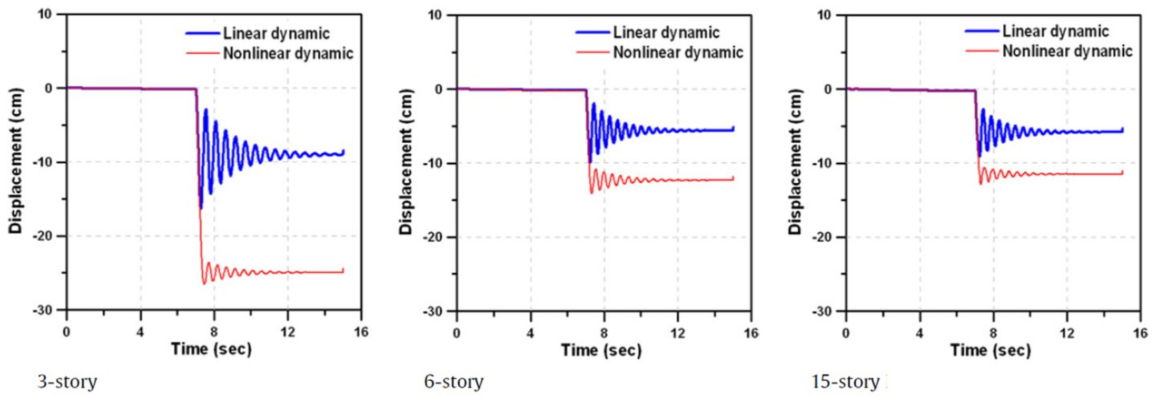


Figure 2.14 Comparison of the linear and the nonlinear dynamic analyses results after a column loss scenario (Kim & Kim, 2009)

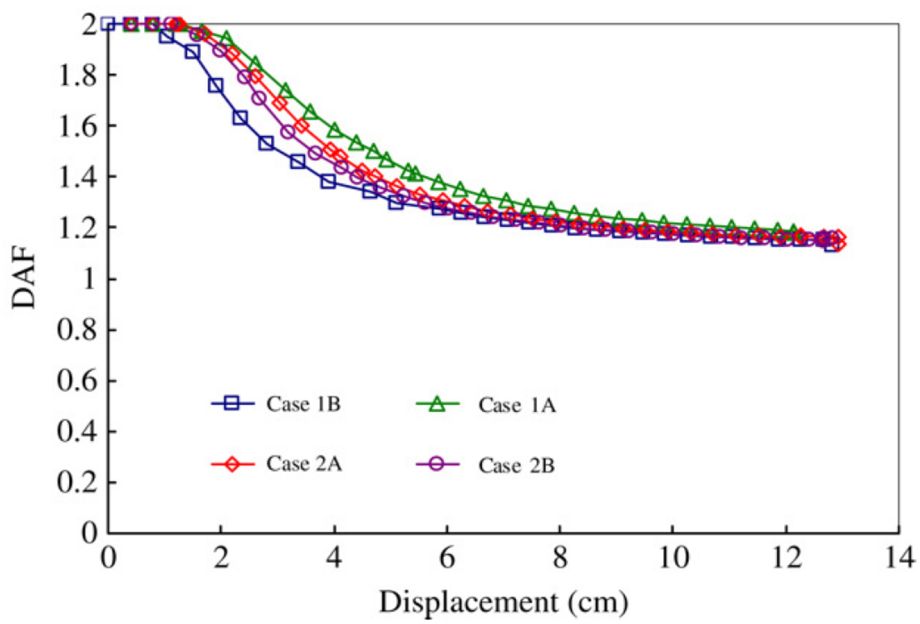


Figure 2.15 Dynamic Amplification Factor (DAF) for cases of column removal based on the capacity curves (Tsai & Lin, 2008)

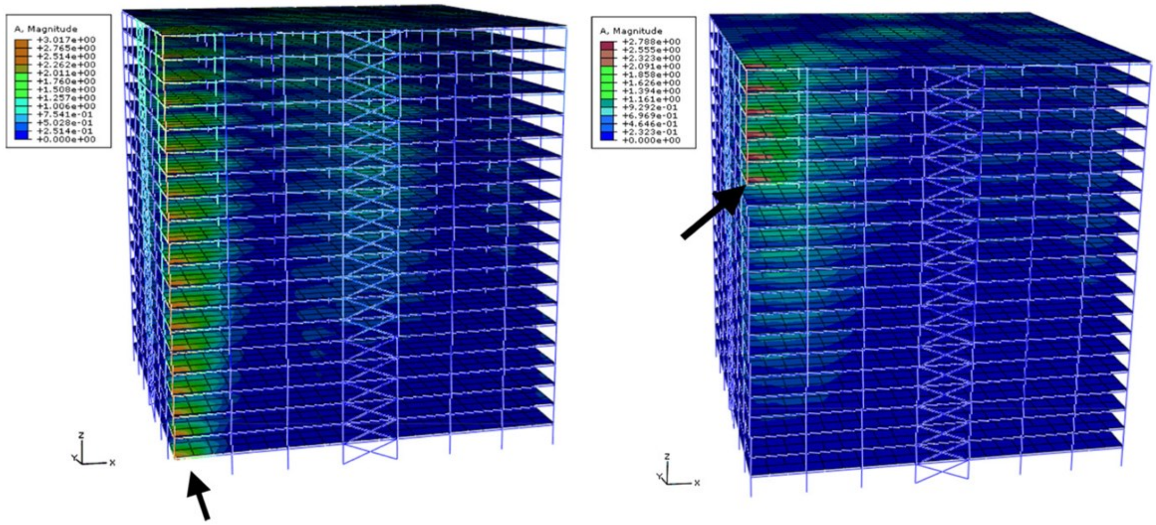


Figure 2.16 3-D models of buildings (Fu, 2009): ground floor column removal (left); above ground column removal (right)

Chapter 3
SEISMIC DESIGN AND AEM MODELS OF THE MULTISTORY STEEL MRF BUILDINGS

3.1 Introduction

The possibility of experiencing a progressive collapse in existing buildings is becoming a serious concern at both professional and public levels. Many existing buildings are either susceptible or un-intentionally resistant to this type of failure. In case of safeguarding against possible progressive collapse, the resistance of a MRF building originates from its structural members being over-designed for gravity loads. Seismic design of frame buildings, on the other hand, can result in over-designed members for gravity loads. Therefore, resistance of a frame building against progressive collapse can be highly dependent on its seismic design and consequently its seismic zone. This seems to be not well investigated in the available literature.

The aim of this research is to investigate the relationship between seismic capacity of steel frame buildings and robustness against progressive collapse. To reach the goal of the study, several steel frame buildings with different number of stories (representing low, medium, and high-rise buildings) are designed in zones of different levels of seismicity (representing low, medium, and high seismicity). The buildings are then subjected to six ground floor column removal scenarios as stated by the GSA guidelines (2003) and deflections under the removed column, internal forces in critical members, and vertical stiffness in the vicinity of the removed column are evaluated. This chapter discusses the design and modeling of the studied buildings as well as the method of analysis.

3.2 Seismic Zones

In order to evaluate the seismic design effect on progressive collapse resistance of the buildings, three different zones of low, medium, and high seismicity are considered in this study. These seismic zones can be represented by three Canadian cities, namely Toronto, Montreal, and Victoria, respectively. Table 3.1 shows the seismic parameters of these cities required in order to perform the seismic design using the National Building Code of Canada (NBCC (2005)).

3.3 Number of Stories

In order to investigate the effect of number of stories in column loss events, this study has used three types of building heights. Low, medium, and high-rise buildings are represented by 5, 10, and 15-story buildings, respectively. This can also be seen as studying the effect of redundancy and framing action when a building loses one of its columns.

3.4 Buildings' Plans and Elevations

In general, the response of a building to column removal scenarios may be highly dependent on its specific structural features and the presence of irregularities in its plan or elevation. As this study aims to focus on studying the effects of number of stories and seismic design on the response of steel frame buildings to sudden ground floor column loss events, the studied buildings are chosen not to have irregularities in the plan or elevation. Therefore, typical office buildings with 3 by 6 bay spans of 6 meters and story height of 3.65 meters are chosen, and all of the frames of the buildings in both directions are assumed to be moment resisting frames. Figure 3.1 shows the plan and elevation of the studied buildings.

3.5 Design of the Buildings

The design of the buildings of this study is performed based on the National Building Code of Canada (NBCC (2005)) and the Handbook of Steel Construction (which is based on CAN/CSA S16) and was checked using the software package ETABS (CSI, 2012). The steel sections used for beams and columns of the buildings are wide-flanged (designated W) I-Shaped steel sections listed in Tables 3.2, 3.3, and 3.4. Since moment resisting frames are used as the structural system of the buildings, all the beam-to-column connections are modeled as fixed connections which are able to fully transfer the moments. The connections between the ground floor columns and the foundation are also assumed to be fixed. The frames are designed in order to be able to resist the gravity and seismic loads while the deflections and drifts remain within the NBCC's limits. The design of the steel sections is tried to be a uniform design in order to avoid irregularity in the structural characteristics, such as mass, stiffness, and strength. Figure 3.3 shows a three-dimensional view of one of the designed 5-story buildings that were checked using the software package ETABS.

3.5.1 Gravity Loads

The steel frames of the buildings are designed to carry a concrete slab of 180 mm thickness. The beams are considered to be laterally braced in the design due to the existence of the concrete slabs. The floors are subjected to 2.4 kPa of live load for office use. The dead loads include 1.0 kPa for partitions, 0.35 kPa for mechanical services on the roof, 0.1 kPa for suspended ceiling, and the load related to the flooring system of each story. The roofs are also subjected to the snow loads corresponding to the cities where buildings are located. The slabs are considered to be two-way slabs which transfer the loads to the frames based on the tributary area method and cause the beams to have triangular distributed loads.

3.5.2 Seismic Loads

The structural features of the buildings of this study are tried to be simple, typical and regular according to the building codes in order to highlight the effect of a seismic design which is based on Equivalent Static Force Method, as described in the NBCC (2005). Table 3.1 and Figure 3.2 show the seismic parameters and the design spectrums related to the three seismic zones of this study. The steel frames of the building are considered to be moderately ductile moment resisting frames. Therefore, the values of ductility-related force modification factor, R_d, and overstrength-related force modification factor, R_o, are 3.5 and 1.5, respectively. The soil is assumed to be a Site Class C which represents "very dense soil and soft rock", the importance of the buildings is categorized as "Normal", and the importance factor for earthquake loads and effects, I_E, is taken to be 1.0 consequently.

3.6 Numerical Modeling

In this study "Extreme Loading for Structures" (ELS) software package is used which works based on Applied Element Method (AEM). This section provides an overview of this method and the analytical procedure applied in this study.

3.6.1 Applied Element Method (AEM)

As described briefly in Chapter 2, AEM has certain superiority to FEM when carrying out progressive collapse studies. This is mainly because in this phenomenon, many members experience large deformations and elements may be separated during the analysis procedure.

ELS is able to detect not only the elements but also the structural members such as beams, columns, walls, and slabs. It features a 3D graphical interface which enables the user to assemble the model using structural members rather than elements. When the model is complete, members can be virtually divided into smaller rigid elements based on the required accuracy. Any two adjacent elements are connected through several contact points. Each contact point consists of two transverse shear springs and one normal spring. Since there can be several contact points and springs on each face of an element, the stiffness of each spring depends on the area it serves as shown in Figures 3.4 and 3.5. Each rigid element has 6 degrees of freedom (3 translations and 3 rotations) which are located at the center of the element. By geometrically relating these degrees of freedom to the springs, the stiffness matrices can be assembled.

As shown in Figure 3.6, when the springs reach the separation strain, they are automatically removed, and thereafter, the elements are considered separated. However, in a case that the elements collide after this separation, some new springs with different characteristics are defined in order to make it possible to transfer the forces related to the collision.

3.6.2 Material Properties

In progressive collapse of a building, steel beams and columns may experience high stresses and strains and in many cases, pass their yielding limit. Therefore, to study the behavior of the buildings in such extreme conditions properly, the nonlinear and post-yield behavior of structural steel should be taken into account. The steel used as the structural steel in this study is 350W, and its yielding and ultimate stresses are 350 and 490 MPa, respectively. The modulus of elasticity of the steel is taken 200 GPa, and its shear modulus is 80 GPa. Figure 3.7 shows the stress-strain relationship of the steel used for the springs in AEM modeling of the frames of the buildings.

3.6.3 Column Removal Scenarios

Alternate Path Method (APM) which is adopted in this study requires a column to be removed instantaneously from the ground floor level. Determining whether or not removing a column from this level is critical to the building is very difficult as the in-plan location of the critical column and its level depend on many factors. However, the main reason for removing columns from the ground floor level is that the columns of this level are more susceptible to car collisions and explosions (GSA, 2003). Current guidelines have some recommendations on the in-plan location of the columns to be removed at the ground level. According to these recommendations, in this study six column removal scenarios are applied to each building. Figure 3.8 shows the location of these columns and their names on the plan of the buildings.

3.6.4 Steps of Loading and Column Removals

As discussed earlier in Chapter 2, it is believed that the best way to study the phenomenon of progressive collapse in the buildings is to perform a combination of nonlinear static and nonlinear dynamic analysis. These two, combined together, reveal an enormous amount of useful information about the behavior of a building which is experiencing a column loss event. Therefore, in this study, these two types of analysis are conducted and are explained in this section.

GSA Guidelines (2003) recommend two load combinations when using APM:

- Static Analysis: 2×(DL + 0.25×LL)
- Dynamic Analysis: (DL + 0.25×LL)

(DL: dead load, LL: live load)

3.6.4.1 Nonlinear Static Analysis

ELS performs this type of analysis in an incremental manner. The magnitude of the static load to be applied on the structure and the number of loading increments are determined by the user. This helps the user to easily monitor the structure's deformations and collapse mechanism during the analysis procedure by choosing the number of loading increments. In this study, the loads and loading increments are determined so that the 100th and 200th increments are associated with cases that the applying load on the structure is the dynamic load combination and static load combination respectively.

Before starting the first increment of the loading, the column related to the desired column removal scenario is removed. Although the GSA guidelines only require the analysis to be done until the loads applying on the structure is equal to the static load combination, in this study, all of the nonlinear incremental static analyses are continued until the failure in the structural members is monitored. This approach acquires the response of the buildings up to their failure stage.

3.6.4.2 Nonlinear Dynamic Analysis

The most rigorous type of analysis is the nonlinear dynamic analysis. In this study, after acquiring the results of nonlinear static analyses for the buildings models, the same models are used to perform nonlinear time history dynamic analysis. Two stages of loading are defined. In the first stage, the vertical load combination recommended by GSA guidelines for dynamic analysis (DL + 0.25LL) is statically applied to the structure. This load combination is smaller than gravity load combinations used for the design of the buildings. Therefore, no failure or even nonlinear and inelastic behavior is expected during the first loading stage. The second stage, however, is a nonlinear dynamic stage. The GSA guidelines require the time period for removing the vertical element to be less than one tenth of the period associated with the structural response mode for the vertical element removal (this response period for all of the buildings of this study is greater than 0.07 second). In the beginning of the second stage, the column is removed over a very short period of 0.001 second to simulate the instantaneous column loss, and the nonlinear dynamic analysis is conducted. Since progressive collapse of buildings may occur over a very short period of time, and the changes in the internal forces and deflections happen extremely fast, the time step within the second stage was chosen to be a very small value of 0.001 second with 10 divisions in each time step (these values are recommended by "Applied Science International" specifically for progressive collapse analysis). This creates 10000 steps for each second. One of the main evaluated parameters in this study is the deflection under the removed column; therefore, the nonlinear dynamic analysis (the second stage) is conducted until the maximum deflection under the removed column or failure is monitored. In this study, failure is considered to be a case in which some of the beams connected to the removed column, in the first floor or other floors, are detached from the adjacent columns and consequently the structure cannot keep its stability and integrity anymore. Figure 3.9 includes a 3-D view of one the 5-story buildings modeled using the ELS software package.

3.6.5 Assumptions in Modeling

In the modeling of the buildings of this study, the following assumptions have been made:

1- Composite action between steel beams and concrete slabs is neglected.

2- The flexural and shear stiffness of the concrete slabs which can contribute to the resistance of the building against column removal scenarios is neglected.

3- Beam-to-column connections have enough rotational capacity and strength that the failure will not be in the joint.

4- Increase in the materials strength due to the high rate of loading (instantaneous column removal) is neglected.

Table 3.1 Seismic parameters for the three Canadian cities used in this study (NBCC 2005)

City	Seismic Parameters				
	$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	PGA
Toronto, Ontario	0.260	0.130	0.055	0.015	0.17
Montreal, Québec	0.690	0.340	0.140	0.048	0.43
Victoria, British Columbia	1.200	0.820	0.380	0.180	0.61

Table 3.2 Designed sections for beams and columns of the buildings located in the low seismic zone (Toronto)

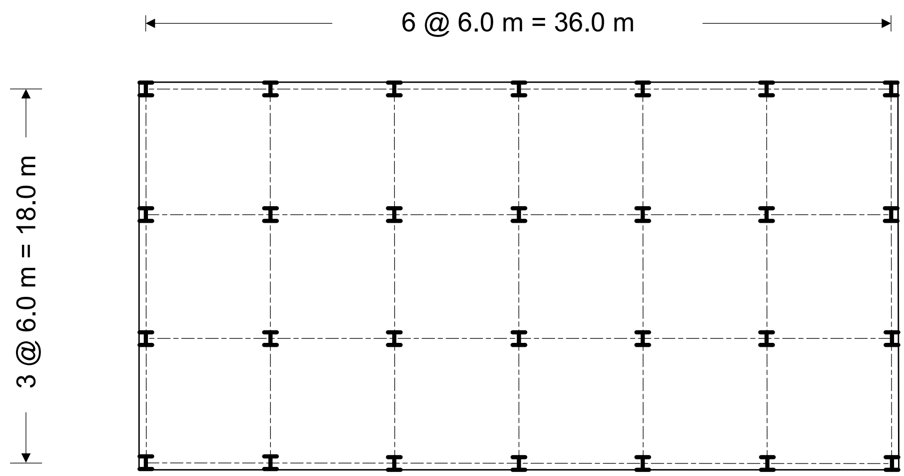
Low Seismic Zone						
Story	Beams		Columns			
	Exterior	Interior	Exterior			Interior
			Corner	Short Edge	Long Edge	
5-Story Building						
5	W310X21	W360X33	W310X107	W310X118	W310X129	W360X179
4	W200X22	W310X28	W310X107	W310X118	W310X129	W360X179
3	W200X22	W310X28	W310X107	W310X129	W310X129	W360X179
2	W200X22	W310X28	W310X107	W310X129	W310X143	W360X179
1	W200X22	W310X28	W310X107	W310X129	W310X143	W360X179
10-Story Building						
10	W310X21	W310X39	W310X107	W310X118	W310X118	W360X196
9	W200X27	W310X33	W310X107	W310X118	W310X118	W360X196
8	W200X27	W310X33	W310X107	W310X118	W310X118	W360X196
7	W200X27	W310X33	W310X118	W310X129	W310X129	W360X196
6	W200X27	W310X33	W310X118	W310X129	W310X129	W360X196
5	W200X27	W310X33	W310X118	W310X129	W310X129	W360X196
4	W200X27	W310X33	W310X158	W310X179	W310X179	W360X196
3	W200X27	W310X33	W310X158	W310X179	W310X179	W360X196
2	W200X27	W310X33	W310X158	W310X179	W310X179	W360X196
1	W200X27	W310X33	W310X158	W310X179	W310X179	W360X196
15-Story Building						
15	W200X27	W310X39	W310X107	W310X129	W310X143	W360X196
14	W310X24	W310X33	W310X107	W310X129	W310X143	W360X196
13	W310X24	W310X33	W310X107	W310X129	W310X143	W360X196
12	W310X24	W310X33	W310X107	W310X158	W310X158	W360X196
11	W310X24	W310X33	W310X107	W310X158	W310X158	W360X196
10	W310X24	W310X33	W310X107	W310X158	W310X158	W360X196
9	W310X24	W310X33	W310X129	W360X196	W360X196	W360X287
8	W310X24	W310X33	W310X129	W360X196	W360X196	W360X287
7	W310X24	W310X33	W310X129	W360X196	W360X196	W360X287
6	W310X24	W310X33	W310X143	W360X196	W360X216	W360X314
5	W310X24	W310X33	W310X143	W360X196	W360X216	W360X314
4	W310X24	W310X39	W310X143	W360X196	W360X216	W360X314
3	W310X24	W310X39	W310X158	W360X237	W360X347	W360X347
2	W310X24	W310X39	W310X158	W360X237	W360X347	W360X347
1	W310X24	W310X39	W310X158	W360X237	W360X347	W360X347

Table 3.3 Designed sections for beams and columns of the buildings located in the medium seismic zone (Montreal)

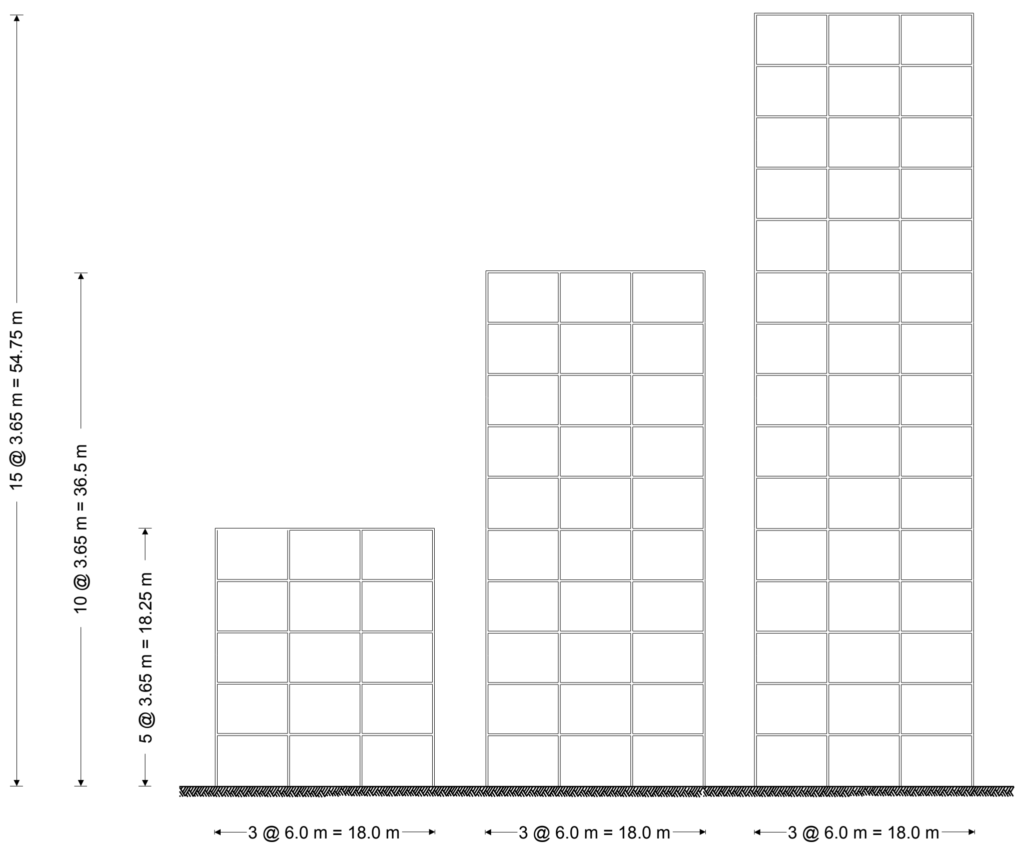
Medium Seismic Zone						
Story	Beams		Columns			
	Exterior	Interior	Exterior			Interior
			Corner	Short Edge	Long Edge	
5-Story Building						
5	W310X24	W360X39	W310X107	W310X129	W310X129	W360X216
4	W310X24	W360X39	W310X107	W310X129	W310X129	W360X216
3	W310X24	W410X39	W310X107	W310X143	W310X179	W360X237
2	W360X33	W410X46	W310X107	W310X143	W310X179	W360X237
1	W360X33	W410X46	W310X107	W310X143	W310X179	W360X237
10-Story Building						
10	W310X24	W360X33	W310X107	W310X129	W310X143	W360X196
9	W310X24	W360X33	W310X107	W310X129	W310X143	W360X196
8	W310X24	W360X33	W310X107	W310X129	W310X143	W360X196
7	W310X24	W410X39	W310X107	W310X158	W310X158	W360X216
6	W310X24	W410X39	W310X107	W310X158	W310X158	W360X216
5	W310X24	W410X39	W310X107	W310X158	W310X158	W360X216
4	W310X28	W410X46	W310X129	W310X179	W360X162	W360X262
3	W310X28	W410X46	W310X129	W310X179	W360X162	W360X262
2	W310X28	W410X46	W310X129	W310X179	W360X162	W360X262
1	W310X28	W410X46	W310X129	W310X179	W360X162	W360X262
15-Story Building						
15	W310X28	W410X39	W310X107	W310X143	W360X162	W360X237
14	W310X28	W360X39	W310X107	W310X143	W360X162	W360X237
13	W310X28	W360X39	W310X107	W310X143	W360X162	W360X237
12	W310X28	W410X39	W310X118	W310X158	W360X179	W360X262
11	W310X28	W410X39	W310X118	W310X158	W360X179	W360X262
10	W310X28	W410X39	W310X118	W310X158	W360X179	W360X262
9	W310X28	W410X39	W310X129	W310X179	W360X196	W360X287
8	W310X28	W410X46	W310X129	W310X179	W360X196	W360X287
7	W310X28	W410X46	W310X129	W310X179	W360X196	W360X287
6	W310X33	W410X46	W310X143	W360X179	W360X216	W360X314
5	W310X33	W410X46	W310X143	W360X179	W360X216	W360X314
4	W310X33	W410X46	W310X143	W360X179	W360X216	W360X314
3	W310X33	W410X46	W310X158	W360X196	W360X237	W360X347
2	W310X33	W410X46	W310X158	W360X196	W360X237	W360X347
1	W310X33	W410X46	W310X158	W360X196	W360X237	W360X347

Table 3.4 Designed sections for beams and columns of the buildings located in the high seismic zone (Victoria)

High Seismic Zone						
Story	Beams		Columns			
	Exterior	Interior	Exterior			Interior
			Corner	Short Edge	Long Edge	
5-Story Building						
5	W360X33	W460X52	W310X143	W310X202	W360X216	W360X314
4	W360X39	W460X68	W310X143	W310X202	W360X216	W360X314
3	W410X46	W460X68	W310X179	W310X226	W360X262	W360X382
2	W460X52	W530X74	W310X179	W310X226	W360X262	W360X382
1	W460X52	W530X74	W310X179	W310X226	W360X262	W360X382
10-Story Building						
10	W310X33	W460X52	W310X118	W360X179	W360X179	W360X347
9	W310X33	W460X52	W310X118	W360X179	W360X179	W360X347
8	W310X39	W460X68	W310X118	W360X179	W360X179	W360X347
7	W360X39	W530X85	W310X118	W360X237	W360X179	W360X421
6	W360X39	W530X85	W310X118	W360X237	W360X179	W360X421
5	W360X39	W530X85	W310X118	W360X237	W360X179	W360X421
4	W410X39	W610X82	W310X129	W360X237	W360X196	W360X421
3	W410X39	W610X82	W310X129	W360X237	W360X196	W360X421
2	W410X39	W610X82	W310X129	W360X237	W360X196	W360X421
1	W410X39	W610X82	W310X129	W360X237	W360X196	W360X421
15-Story Building						
15	W310X33	W460X52	W310X143	W360X179	W360X237	W360X314
14	W310X33	W460X68	W310X143	W360X179	W360X237	W360X314
13	W310X33	W460X68	W310X143	W360X179	W360X237	W360X314
12	W360X33	W530X85	W310X158	W360X237	W360X262	W360X421
11	W360X33	W530X85	W310X158	W360X237	W360X262	W360X421
10	W410X39	W530X85	W310X158	W360X237	W360X262	W360X421
9	W410X39	W530X85	W310X179	W360X237	W360X287	W360X421
8	W410X39	W530X85	W310X179	W360X237	W360X287	W360X421
7	W410X46	W530X85	W310X179	W360X237	W360X287	W360X421
6	W410X46	W610X92	W310X202	W360X287	W360X314	W360X509
5	W410X46	W610X92	W310X202	W360X287	W360X314	W360X509
4	W410X46	W610X92	W310X202	W360X287	W360X314	W360X509
3	W410X46	W610X92	W310X226	W360X287	W360X347	W360X509
2	W410X46	W610X92	W310X226	W360X287	W360X347	W360X509
1	W410X46	W610X92	W310X226	W360X287	W360X347	W360X509



(a)



(b)

Figure 3.1 Layout of the studied buildings: (a) Plan; and (b) Section elevations of the studied buildings

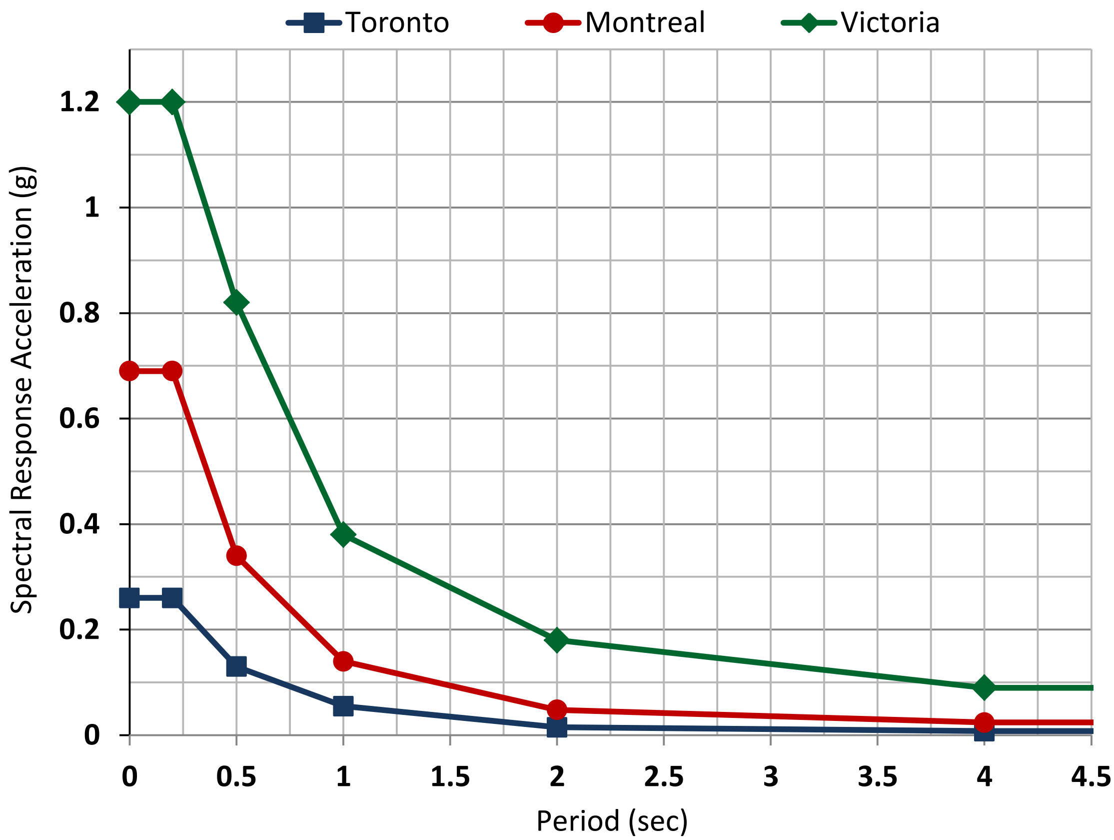


Figure 3.2 NBCC (2005) design spectra for the three seismic zones

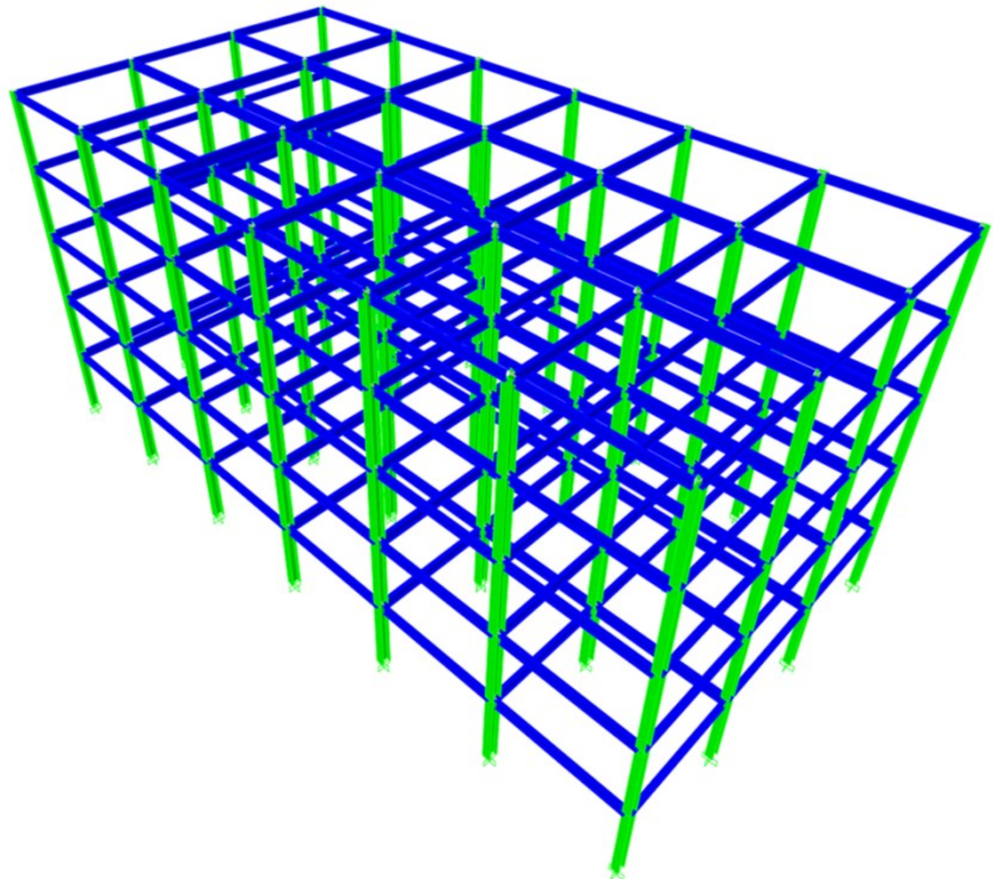
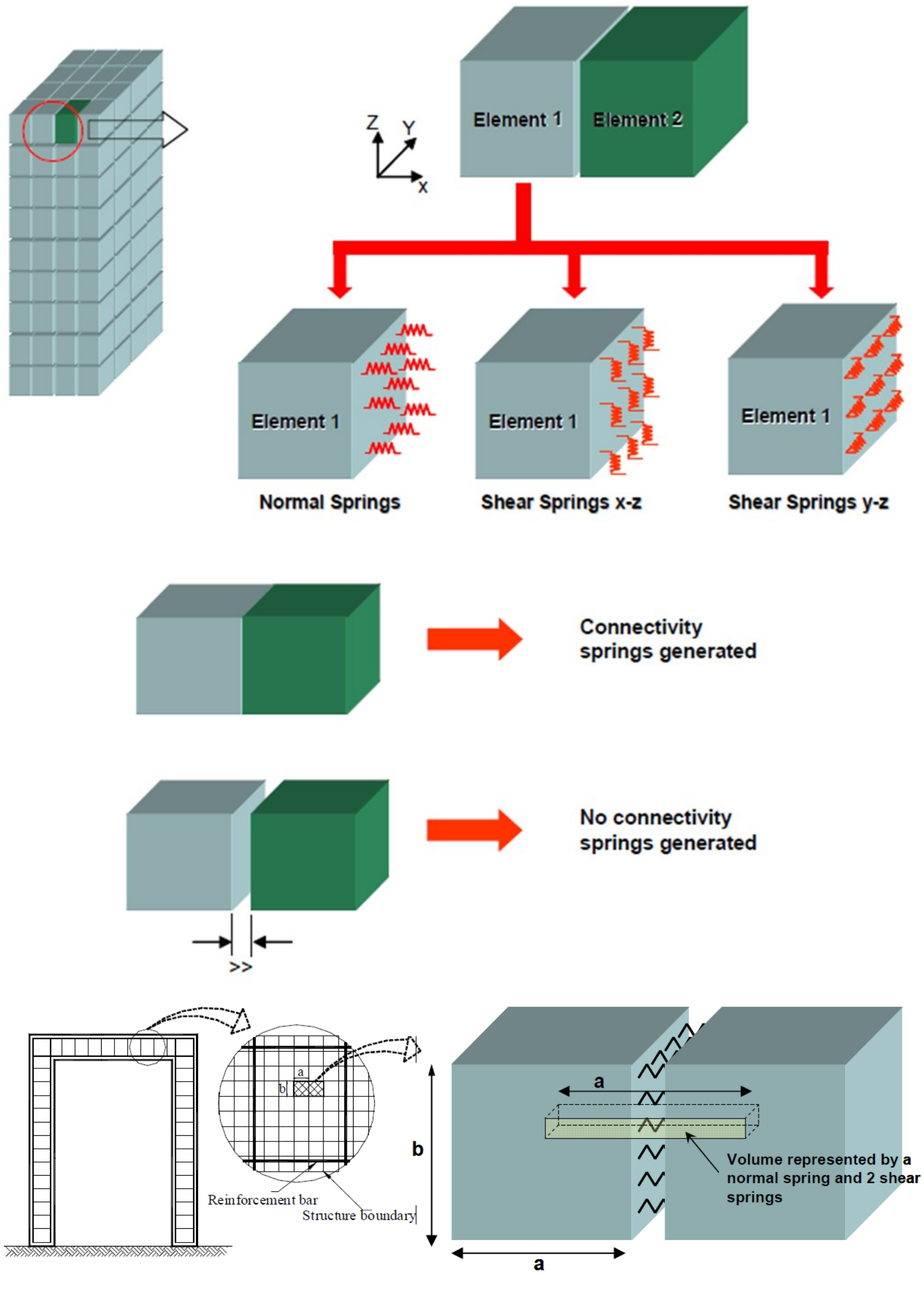


Figure 3.3 A 3-D view using ETABS for one of the designed 5-story buildings



a. Element generation for AEM **b.** Spring distribution and area of influence of each pair of springs

Figure 3.4 Elements and springs generation for AEM (ELS Theoretical Manual 2010)

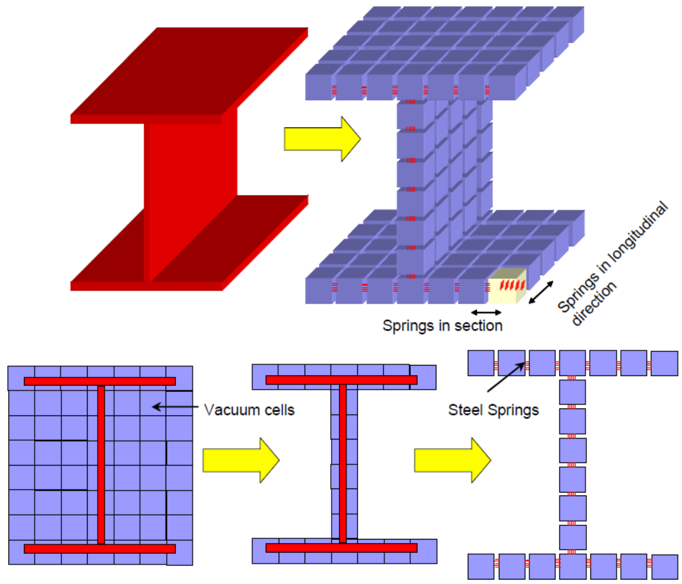


Figure 3.5 Modeling of a steel member in ELS (ELS Theoretical Manual 2010)

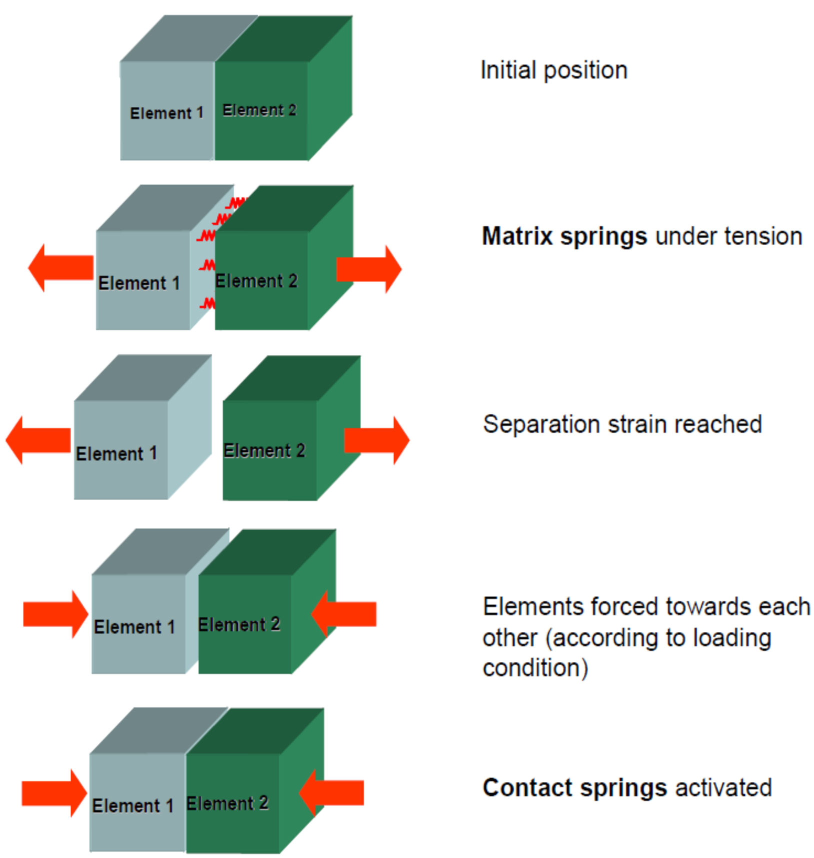


Figure 3.6 Elements and separations and collisions in AEM (ELS Theoretical Manual 2010)

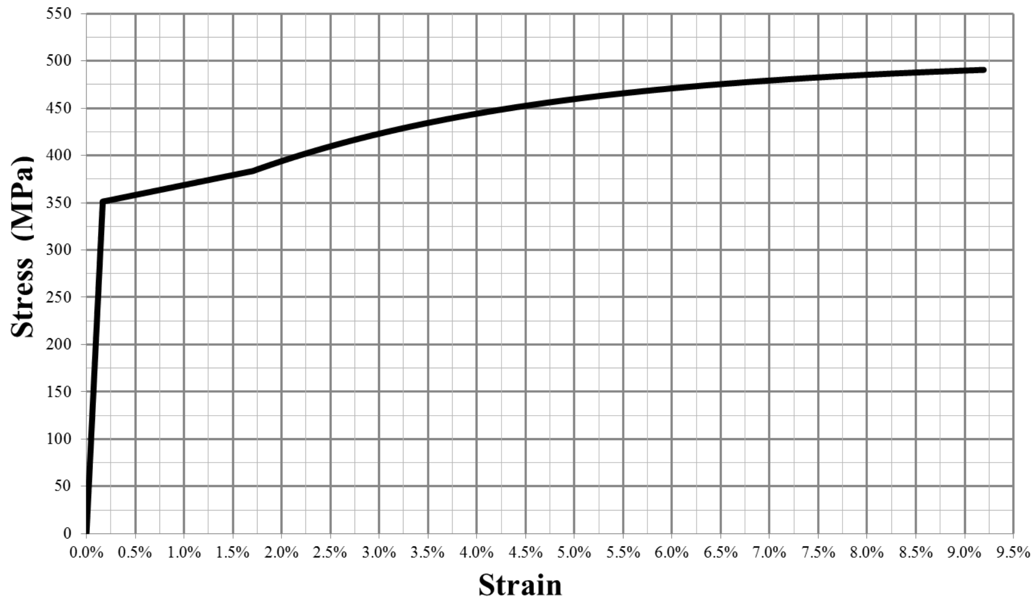


Figure 3.7 Stress-strain relationship of the structural steel used AEM modeling of the buildings (ELS software package)

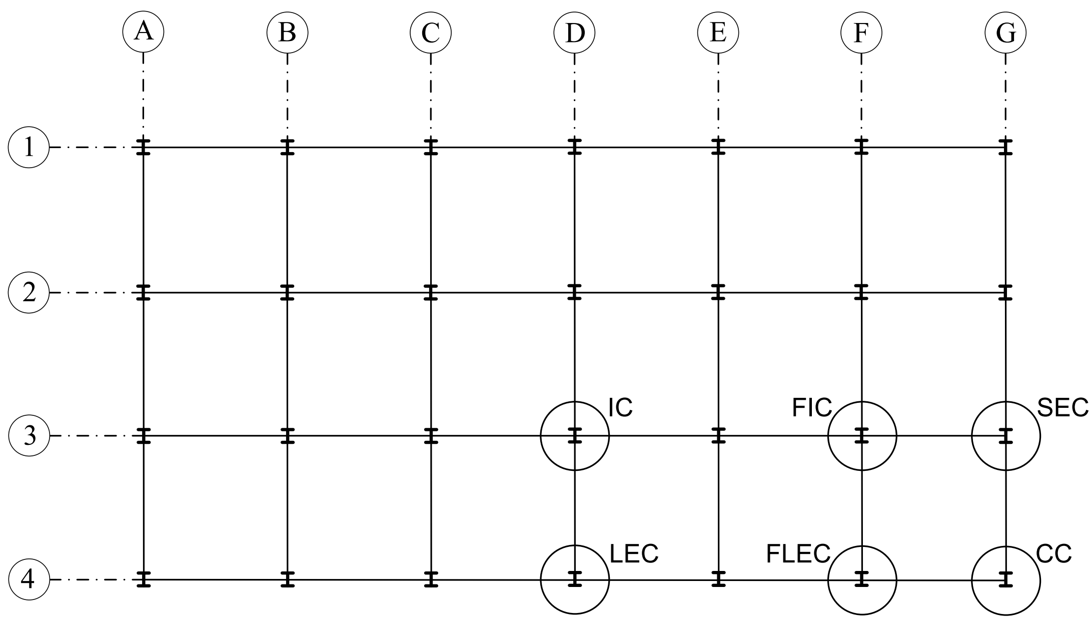


Figure 3.8 Location of the removed columns on buildings' plan (IC: Internal Column, FIC: First Internal Column, SEC: Short Edge Column, LEC: Long Edge Column, FLEC: First Long Edge Column, CC: Corner Column)

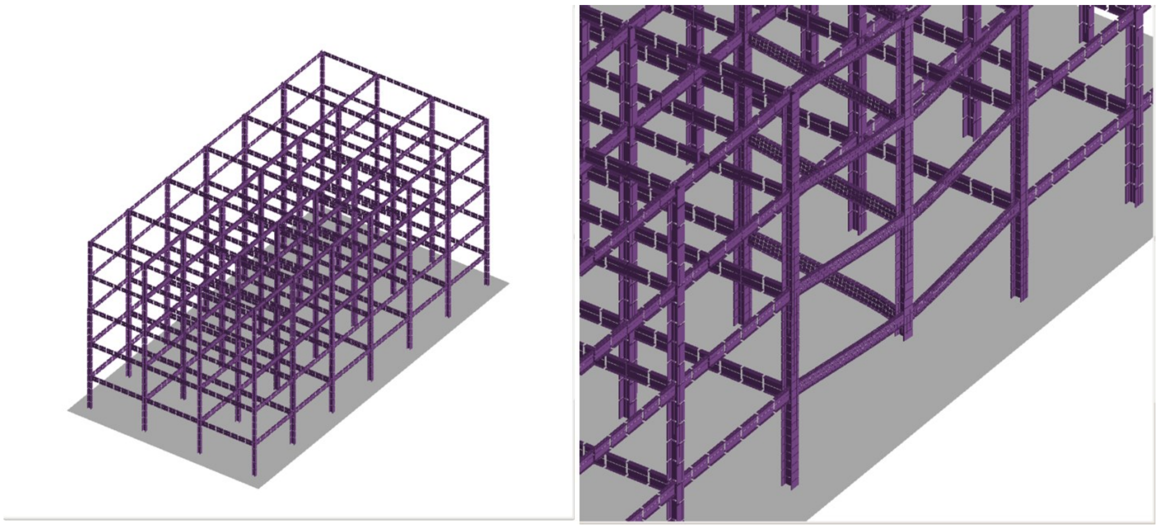


Figure 3.9 A 3-D view of a model of a 5-story building in ELS Software (left); Deflected beams after a column removal scenario (right)

Chapter 4
ROBUSTNESS OF THE DESIGNED BUILDINGS AGAINST PROGRESSIVE COLLAPSE

4.1 Introduction

In this chapter the results of nonlinear static and nonlinear dynamic analysis of all the models of the seismically-designed buildings are presented in the form of comparative charts and figures, and they are discussed through different sections.

4.2 Nonlinear Static Analysis

The main method to evaluate the resistance of the buildings of this study against progressive collapse is the Alternate Path Method (APM). APM considers instantaneous column removals for the building in order to assess its ability to transfer the loads of the removed column through other members. The nonlinear static analysis which is often called the pushdown analysis in column removal scenarios is explained in detail in section 3.6.4.1.

4.2.1 Redundancy

As mentioned in Chapter 3, six different ground floor column removal scenarios are considered for each building of this study. Figure 3.8 shows the location of these columns on the plan of the buildings. Among these columns, there are two interior, one corner, and three exterior columns. When pushdown analysis is performed, the structure starts to resist the vertical loads in a linear elastic manner. As the loads increase during the analysis, the structure gradually loses its initial resisting stiffness against vertical loads and yielding starts to propagate in the affected beams. Yielding mostly appears in the form of plastic hinges in the beams. This continues until the structure loses all of its vertical stiffness in the vicinity of the removed column and is not stable anymore. An example of the process of loss of vertical stiffness in the vicinity of the removed columns can be seen in Figure 4.2 which shows the results of incremental pushdown analysis for the six column removal scenarios of the 5-story building located in Toronto (low seismicity).

Structural redundancy is one of the important characteristics of a building that can contribute to its overall resistance against progressive collapse. Many studies as well as building codes and guidelines have emphasized the importance of increasing redundancy as means of prevention against threats such as progressive collapse.

In case of column removal scenarios, the main resistance in the MRF buildings originates from the flexural stiffness of the affected beams. Higher number of stories provides more beams that can be affected in a column removal event. According to equivalent static force procedure from NBCC adopted in this study, higher seismicity increases the lateral seismic force on the building which consequently requires the building to have stiffer sections for both beams and columns. However, in the low seismic zone addressed in this study, the lateral seismic forces are too small compared to gravity loads to the extent that they do not have significant effect on the design of the frames. As a result, such frames designed for low seismic zones can also be considered as gravity frames. Therefore, all the 5, 10, and 15-story buildings designed in this zone have very similar section for their beams although they have different heights. The only difference between the responses of these buildings against column removals is the difference between the numbers of affected beams, i.e., redundancy.

When static analysis procedures are applied, the GSA guidelines require the use of 2×(DL+0.25LL) as the vertical load combination on the affected part of the building. To conduct the incremental pushdown analysis, the vertical load combination is applied on the building through many small increments. As the buildings that were designed for the low seismic zone were the weakest buildings of this study in terms of sections of the beams and columns, none of them was able to reach 2×(DL+0.25LL) load without experiencing failure.

Figure 4.3 indicates the ultimate vertical load capacity of these buildings in terms of (DL+0.25LL) when nonlinear static (pushdown) analysis is performed for the six column loss scenarios. The figure shows that none of the buildings was able to reach the level of 200 percent, which is the load combination required by GSA. This shows the inadequate capacity of the buildings of the low seismic zone and the importance of retrofitting such structures if progressive collapse needs to be prevented. The other important observation from the figure is the effect of redundancy. According to the figure, for any column removal scenario, high-rise buildings have relatively higher ultimate capacity compared to the low-rise buildings.

4.2.2 Seismicity

In order to evaluate the building's resistance against column removal scenarios, chord rotation and deflection of the beams are known to be useful measurements. Figure 4.1 illustrates the deflection and chord rotation of the beams when a column is removed.

As mentioned in Chapter 3, the NBCC equivalent static force procedure for seismic design has been adopted in this study. In this procedure, in order account for the seismic loads in the design, the base shear is first calculated and then distributed along the height of the building. The value of the base shear depends on several parameters including seismic parameters of the field, importance of the building, weight of the building, and the fundamental lateral period of the building.

Incorporating the seismic forces in the design of steel frame buildings results in stiffer beams and columns. Therefore, beams and columns of a seismically designed steel frame building are considered to be over-designed for gravity loads. In other words, the members of such a building possess some reserved capacity that will be used if an earthquake occurs. This reserved capacity can also be effective in the resistance of the building against progressive collapse. Higher seismicity requires the design to use greater seismic forces on the building and provides the building with more reserved capacity and consequently resistance against progressive collapse.

The GSA guidelines uses 2×(DL+0.25LL) as the load combination in case of performing static analysis. The pushdown analysis of the buildings of this study reveals that none of the buildings of the low and medium seismic zones are able to resist such a load combination (Figure 4.3 and 4.4). Therefore, those buildings of this study which are designed for low and medium seismicity cannot meet the GSA requirements for static analysis.

The buildings designed for the high seismicity (Victoria, Canada), however, are strong enough to resist the 2×(DL+0.25LL) load combination of the GSA. The ultimate vertical load capacity of these buildings is shown in Figure 4.5. Since these buildings do not fail under the GSA static load combination, their chord rotation under this load can be an indication of their response. Figure 4.6 shows the chord rotation of the affected beams of these buildings under the removed columns when the buildings are undergoing the GSA static load combination.

In this study pushdown analysis is performed for six different column removal scenarios in each building. Therefore, the results of pushdown analysis of each building include 6 values of load for each for any value of deflection. Figure 4.2 is an example of these six curves for one of the studied buildings. In order to simplify the curves, the average of the 6 values of load is calculated such that only one curve represents the pushdown analysis results of each building. These curves are presented in Figure 4.7. The average of ultimate load capacity for the six column removal scenarios of each building is also calculated and shown against the Peak Ground Acceleration (PGA) of the seismic zones in Figure 4.8.

Both Figures 4.7 and 4.8 clearly show higher resistances in the buildings designed for higher seismicity levels. In other words, there is an improvement in the response of the buildings when the design level of seismicity changes from low to medium or from medium to high. However, this improvement seems to more significant when the level of seismicity changes from medium to high compared to when it changes from low to medium.

4.3 Nonlinear Dynamic Analysis

In this section the ELS software package is used to perform the nonlinear time history dynamic analysis. In this type of analysis, geometrical and material nonlinearity are considered and the dynamic nature of the loads (column removal) is taken into account. Hence, it can be seen as an excellent simulation of progressive collapse event.

4.3.1 Seismicity

The results of the dynamic time history analysis show that the buildings located in the low seismic zone (Toronto, Canada) do not have enough capacity to resist any of the column removal scenarios. In fact, all of the column removal scenarios for 5, 10, and 15-story buildings located in this zone were followed by failure, i.e. partial collapse. Figure 4.9 shows examples of the partial collapses in a 5-story building.

However, the buildings designed for the medium and high seismicity are able to survive all of the six column loss scenarios, with different levels of response. When one of these buildings faces an instantaneous column loss, the affected beams along the height of the building experiences fast increase in deflection at the point of the removed column as shown in Figure 4.10. As the beams deflect, they produce resistant internal forces in the forms of moments, shears, and axial forces. In typical steel moment resisting frame buildings (fixed beam-to-column connections and bay spans of 5 to 8 meters) the most significant resisting mechanisms against a column loss event is through flexure of the beams. The internal forces in the beams continue to increase until they can resist the vertical loads of the building in the affected area. Since instantaneous column removal is a dynamic load on the building, the deflection of the beams does not stop at this stage, and due to the existing kinetic energy in the affected beams, the deflection continues to increase until the beams reach the maximum deflection. The rest of the response is a dynamic vibration, and continues until the kinetic energy in the building is completely dissipated by the internal damping of the building. Figure 4.10 shows the deflection versus time for one of the column removal scenarios.

Figure 4.11 compares the maximum chord rotation (associated with maximum deflection) of the buildings under the removed columns for the medium and high seismic zones. As mentioned earlier, all of the buildings of the low seismic zone failed to survive any of the six column removals while no failure was captured in the other two seismic zones. The figure shows that there is a significant difference between the resistance of the buildings of the medium and high seismic zone. This highlights the effect of seismic design on the resistance of the buildings against progressive collapse.

Scrutinizing Figure 4.11 also reveals that the differences between the chord rotations of the buildings of the medium seismic zone for different column removal scenarios are more noticeable than those of the buildings of the high seismic zone. According to Figure 4.7, when pushdown analysis is being conducted, the building loses its vertical stiffness rapidly after the yielding, and as a result, the deflections start to increase faster upon continuing the incremental loading. This condition becomes more severe as the building gets closer to its failure point. Hence, it could be said that the buildings of the medium seismic zone are more susceptible to failure compared to buildings designed for the high seismic zone due to their relatively weaker cross-sections that results in less vertical stiffness. Therefore, each of the three buildings of the medium seismic zone is more likely to have higher range of chord rotations for the six cases of column removal as compared to their counterparts designed for high seismic zone.

Performing nonlinear dynamic time history analysis on 3-D models of the buildings are expected to have very accurate results which are close enough to the building's real response. This deterministic approach is case specific; hence it reduces the dependency on different conditions and limitations which are usually stated in the guidelines. Failure of the building following an extreme loading event of a column loss implies the need to retrofit the building. Moreover, although a building which is able to survive a column removal passes the test for the resistance against progressive collapse, further investigations need to be conducted in order to assess the status of the building after such an event.

4.3.2 Vertical Stiffness in the Vicinity of the Removed Column

One of the parameters which can be considered as a good indicator of the building's structural robustness against a column removal scenario is the vertical stiffness of the building at the point of the removed column. Pushdown analysis can generate curves such as the ones presented in Figures 4.2 and 4.7. They start with a linear segment in which the affected beams connected to the point of the removed column are showing linear responses. As the vertical load increases, yielding starts to happen in top and bottom chords of some of the beams sections, and the bending moments in the beams get closer to the plastic moment of their sections. This is when the building starts to lose its stiffness dramatically and is shown on Figure 4.2 as the starting point of yielding. While some of the affected beams lose their flexural stiffness, redistribution of the forces occurs among them. The force redistribution along with the excess flexural capacity of the beams related to their post yield behavior delays the failure of the building even after it passes the yielding point. However, the building continues to lose its vertical stiffness in the vicinity of the removed column until all of the affected beams reach bending moment demands that are close to their ultimate moment capacities, and finally failure happens.

In this study, a specific attention has been given to the vertical stiffness of the building at the point of the removed column as it can provide valuable information on how close the building’s status is to its failure. The initial vertical stiffness of the buildings for the six different column removal scenarios is obtained from the results of the pushdown analysis (such as the curves of the Figure 4.2). This value equals the slope at the beginning of curve. On the other hand, the final vertical stiffness of the buildings equals the slope of the curve at the point where the deflection is equal to the maximum deflection resulted from the nonlinear dynamic analysis. Using the initial and final stiffness values, the percentage of vertical stiffness loss is calculated for the six column removal scenarios of all of the buildings of the medium and high seismic zones, and the average values are presented in Figure 4.12. It is very interesting that although buildings located in the medium seismic zone are able to remain stable during the column loss cases, in average, they lose more than 80 percent of their vertical stiffness during the dynamic time history analysis. The buildings of the high seismic zone are in much better situations, and the difference between the response of the 5, 10, and 15-story buildings of this zone may be attributed to the inherent redundancy as discussed in section 4.2.1.

4.3.3 Nonlinear Dynamic Analysis and Nonlinear Static Analysis

In order to evaluate the resistance of a building against progressive collapse, writer believes that performing a combination of nonlinear static and nonlinear dynamic analysis provides the most comprehensive information about the structural capabilities of the building. As nonlinear dynamic analysis comprises a rigorous time consuming procedure and requires powerful computer tools, it is often avoided. The GSA guidelines present two different load combinations; one for static analysis and one for dynamic analysis. The static load combination is simply recommended to be twice the dynamic load combination.

The results presented in this chapter show that GSA's recommendation that the static model of the building should be able to sustain twice the load combination in a dynamic analysis is conservative. Buildings designed for medium seismic zones were able to survive a column loss when using a dynamic analysis using (DL+0.25LL), yet the static analyses of the same buildings were not able to resist twice this load. Both the static and dynamic procedures confirmed the serious susceptibility of buildings located in low seismic zones to progressive collapse. On the other hand, they show the inherent robustness of buildings designed for high seismic zones to progressive collapse. However, while the static procedure shows that the buildings of the medium seismic zone does not have enough capacity, the dynamic procedure shows that these buildings are able to survive all of the six column removal scenarios. It is worth-mentioning that the GSA guidelines state that performing nonlinear dynamic analysis exempts the building from being analyzed statically. In other words, the results of the nonlinear dynamic time history analysis can be considered as the most accurate prediction of the building's real response to column loss scenarios.

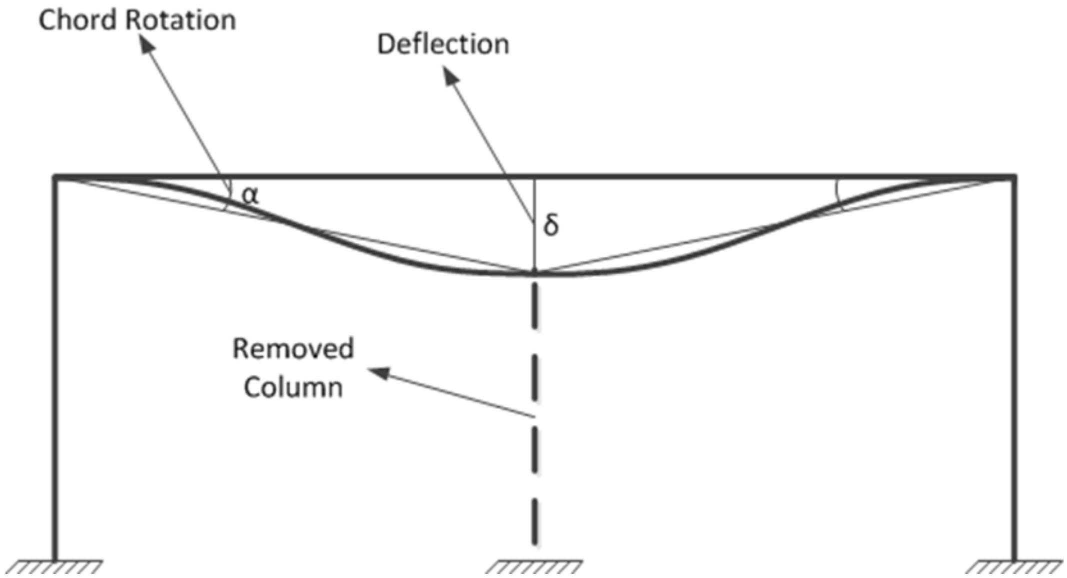


Figure 4.1 Illustration of deflection and chord rotation when a column is removed

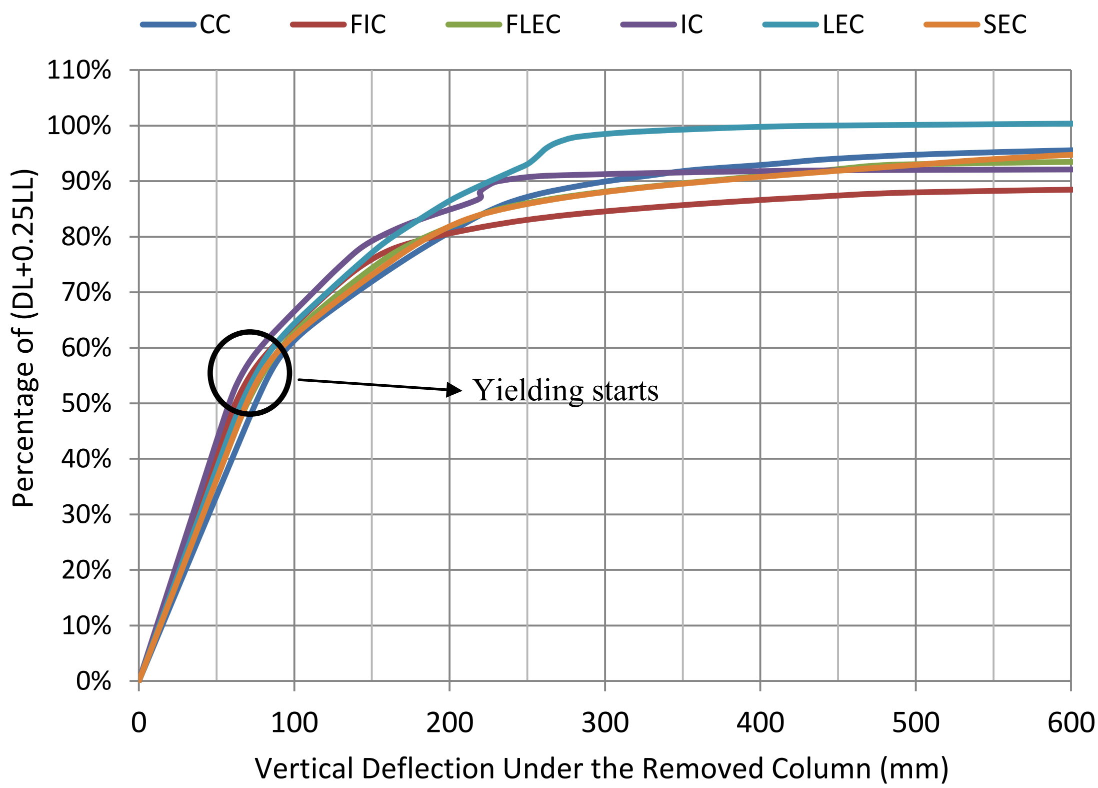


Figure 4.2 Pushdown analysis results of the six column removal scenarios for the 5-story buildings located in Toronto (Low seismicity)

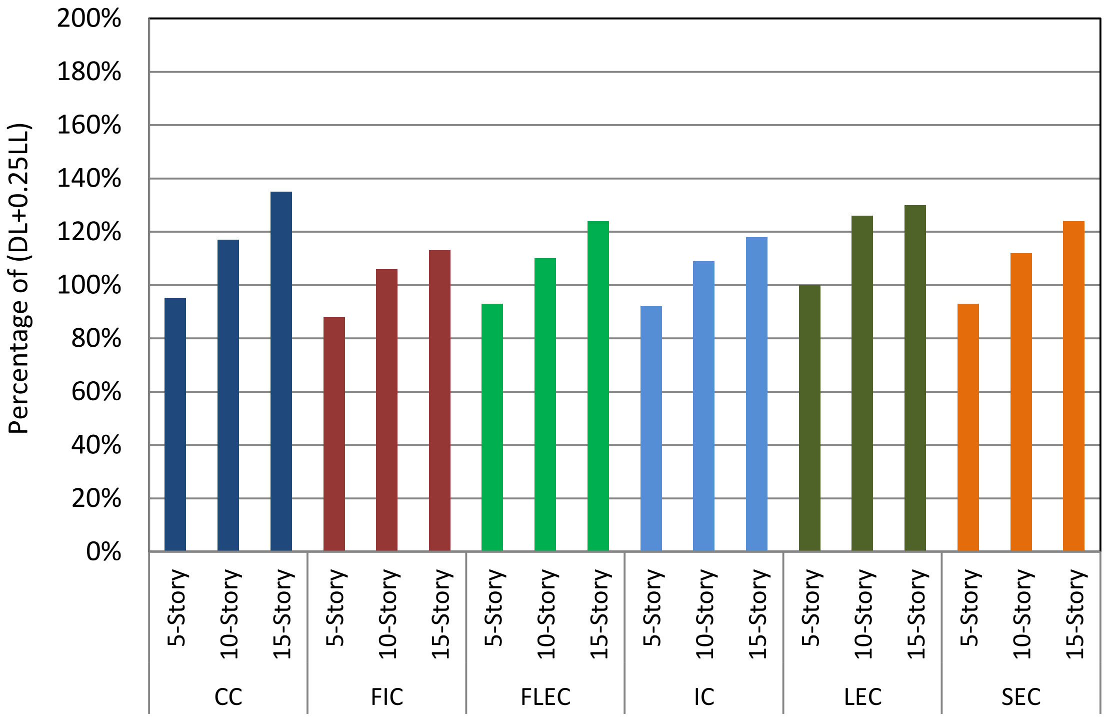


Figure 4.3 Ultimate vertical load capacity of the buildings located in Toronto (representing low seismicity) for the six column removal scenarios using incremental pushdown analysis

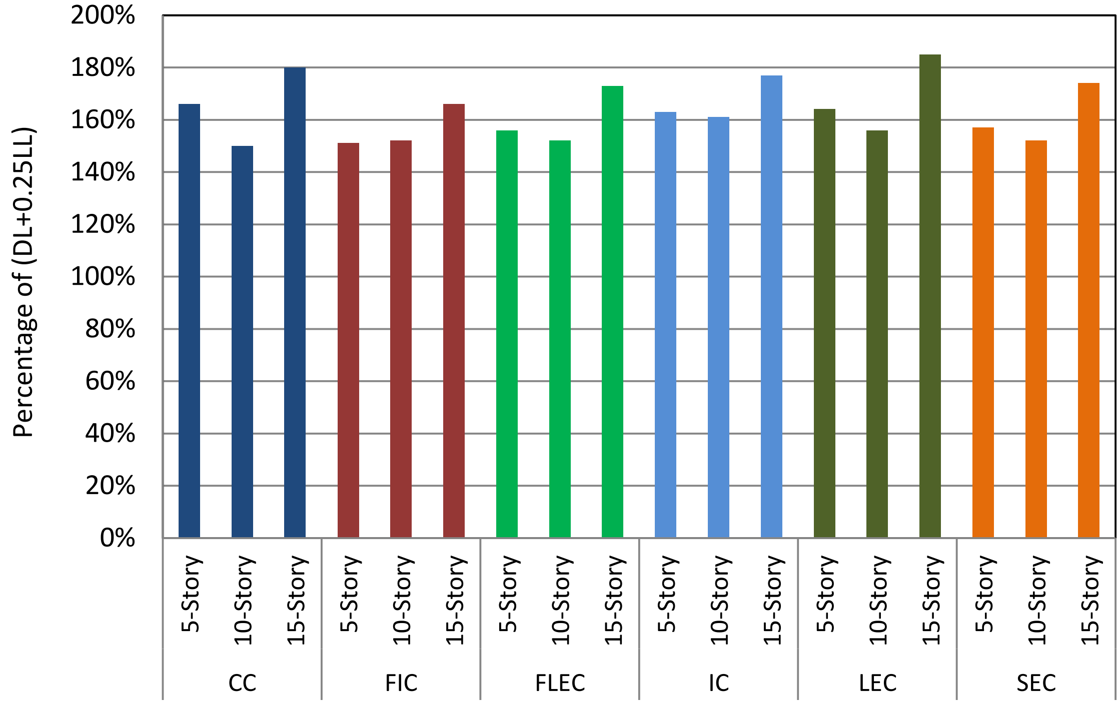


Figure 4.4 Ultimate vertical load capacity of the buildings located in Montreal (representing medium seismicity) for the six column removal scenarios using incremental pushdown analysis

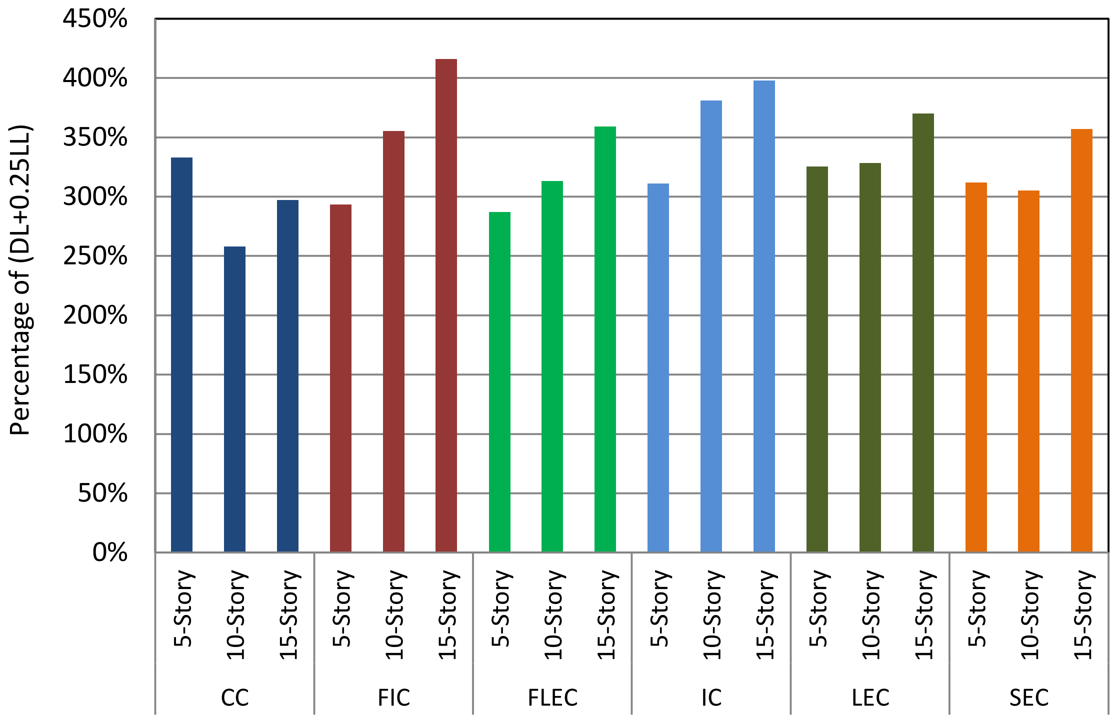


Figure 4.5 Ultimate vertical load capacity of the buildings located in Victoria (representing high seismicity) for the six column removal scenarios using incremental pushdown analysis

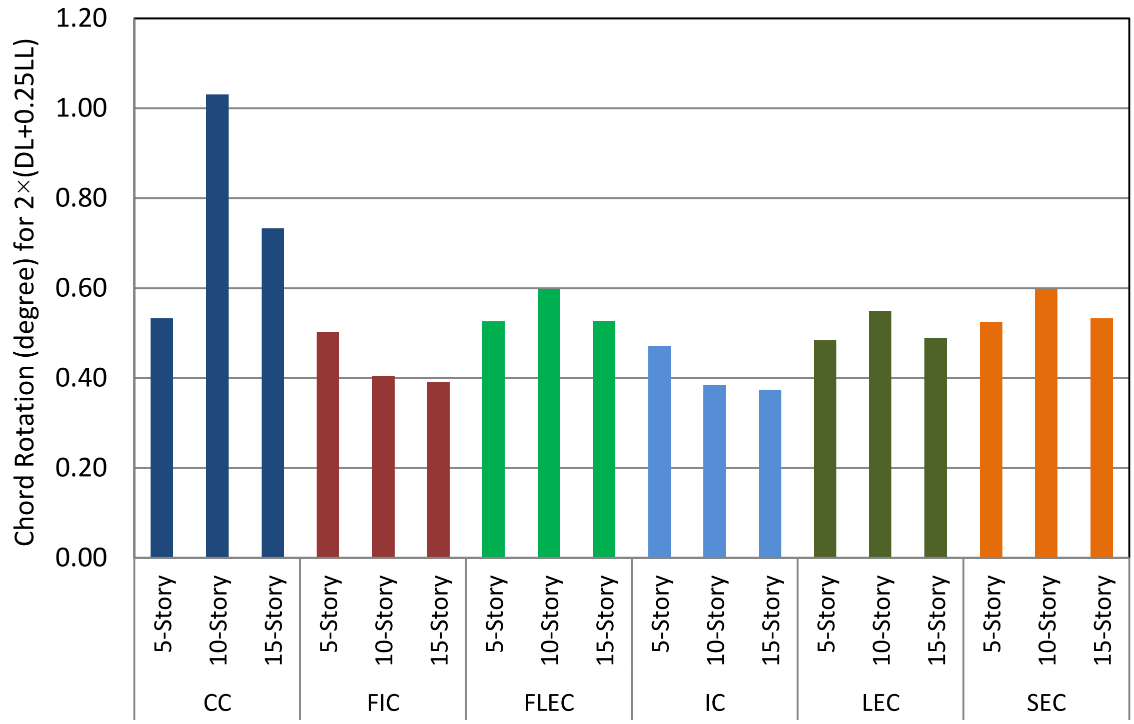


Figure 4.6 Chord rotation of the beams under the removed column for the buildings located in Victoria (representing high seismicity) for the six column removal scenarios using incremental pushdown analysis

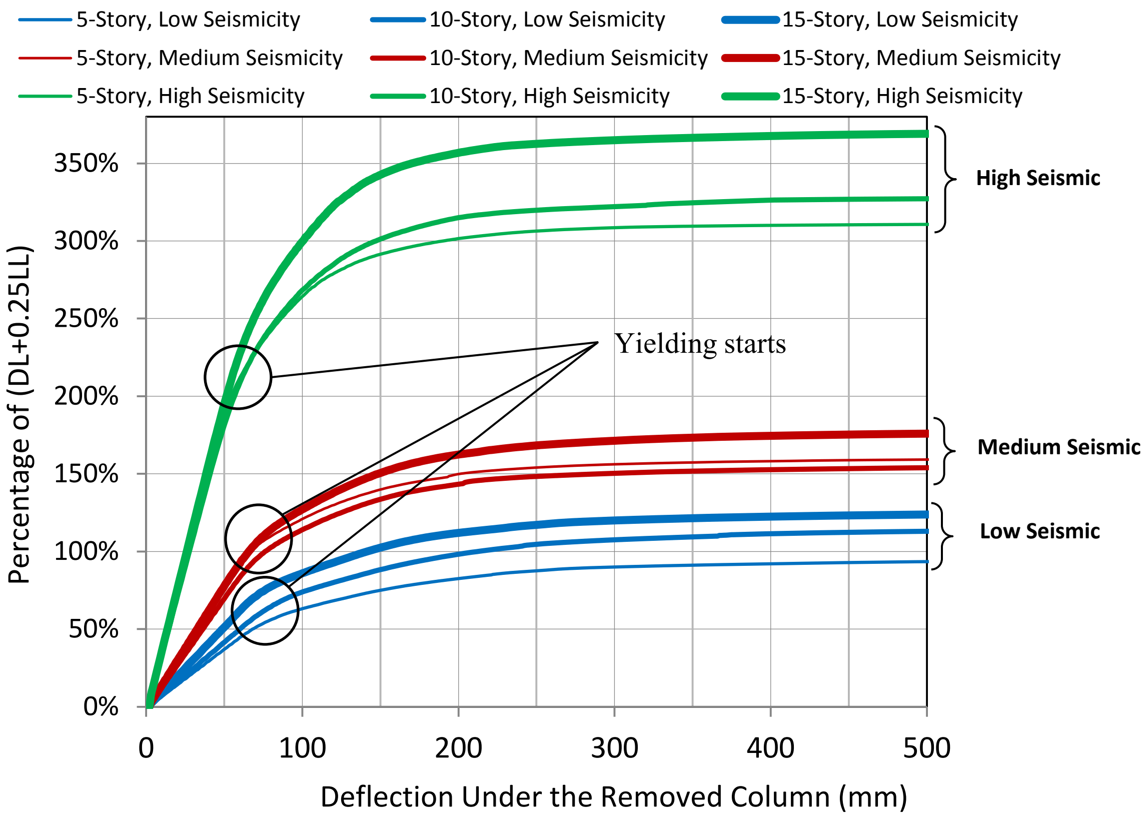


Figure 4.7 Pushdown analysis results of all buildings (average values of the six column removal (average values of the six column removal scenarios)

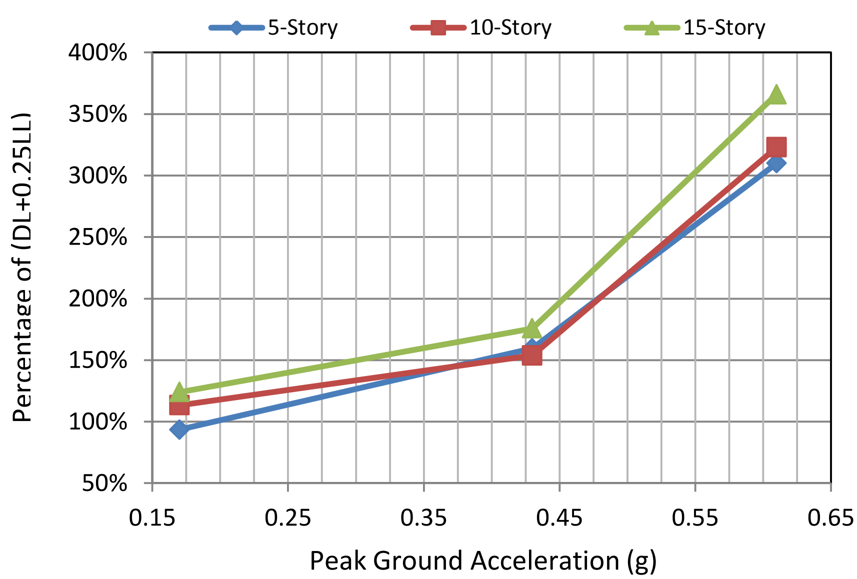


Figure 4.8 Ultimate vertical load capacity of the buildings in zones with different PGA values (average values of the six column removal scenarios)

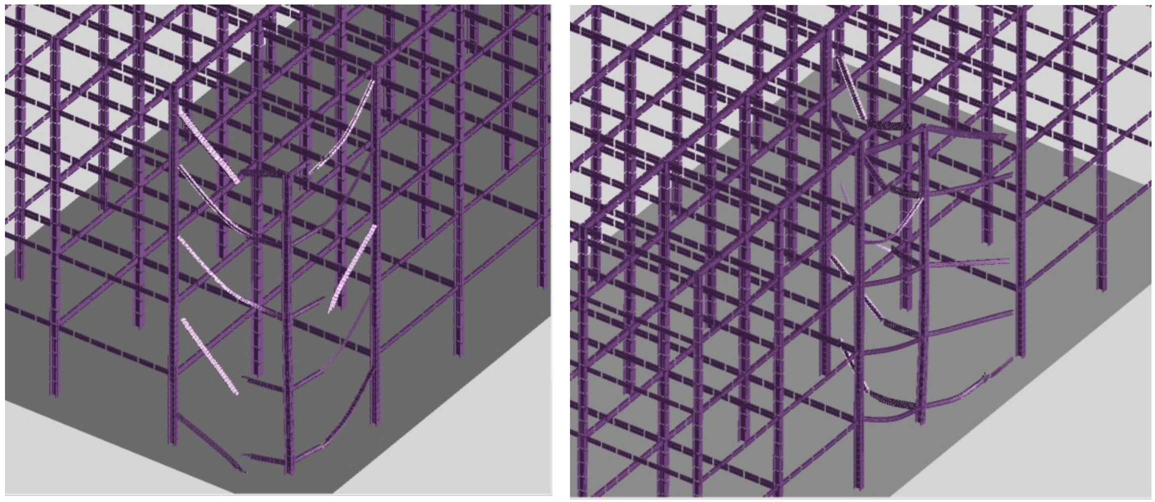


Figure 4.9 Examples of partial failure in the buildings located in the low seismic zone (Toronto, Canada)

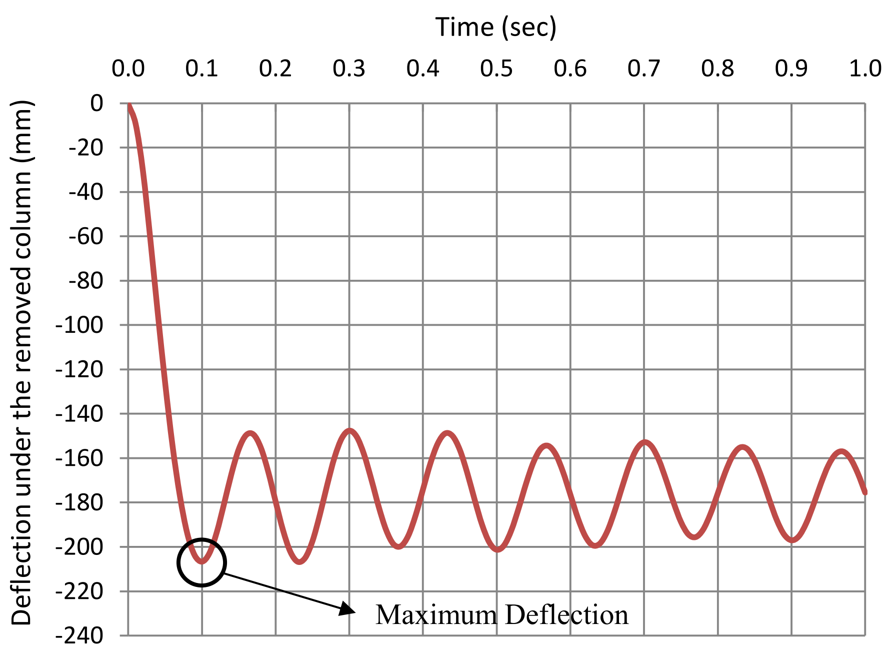


Figure 4.10 Time history of the deflection after the removal of the corner column in the 5-story building located in Montreal

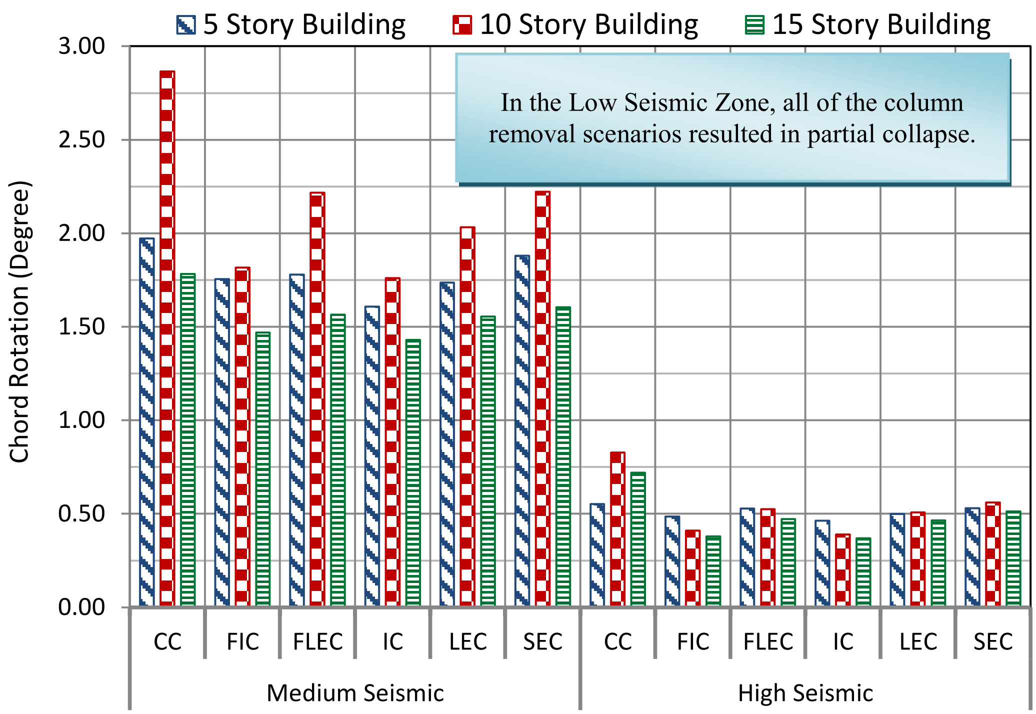


Figure 4.11 Maximum chord rotation of the buildings in the medium and high seismic zone (dynamic analysis)

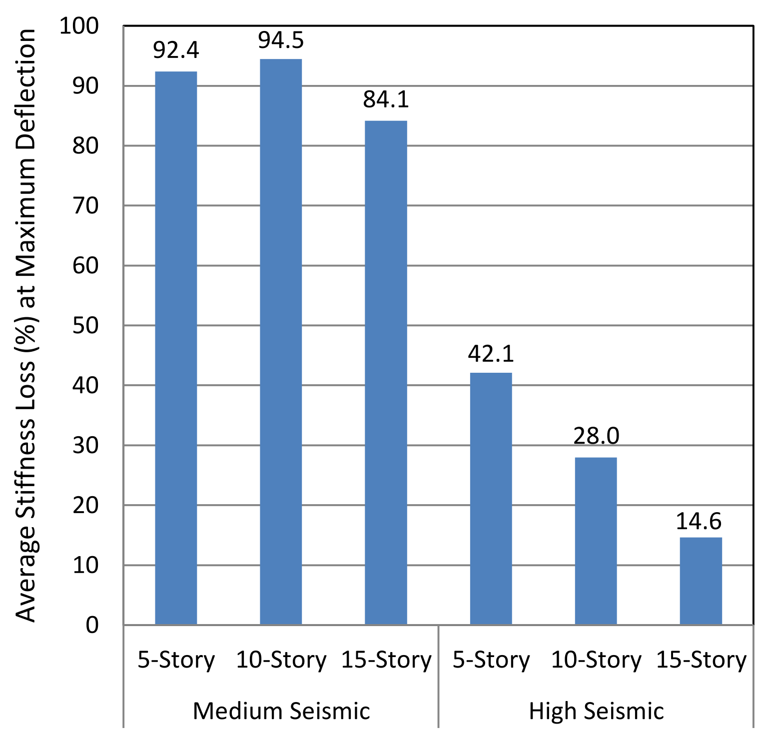


Figure 4.12 Average stiffness loss (%) at maximum deflection (dynamic analysis)

Chapter 5
RETROFIT STRATEGIES FOR MITIGATING PROGRESSIVE COLLAPSE IN STEEL MRF BUILDINGS

5.1 Introduction

The results from Chapter 4 show that retrofitting the buildings that were originally designed for low seismicity (e.g. Toronto, Canada) is inevitable if progressive collapse needs to be prevented. The nonlinear dynamic analysis results also prove that buildings designed for medium and high seismicity are expected to have satisfactory resistance against progressive collapse; however, retrofitting may also be needed for these buildings if their occupancy or loading conditions change.

In general, the seismic design of steel moment resisting frames is an iterative procedure. In this study, equivalent static force procedure from NBCC is adopted. In this method, first, the fundamental period of the building is calculated using simplified formulas based on the height of the buildings. Using the fundamental period along with other seismic parameters, base shear is calculated and distributed along the height of the building to provide storey forces. This procedure is done in two orthogonal horizontal directions for the building in order to take into account all the lateral seismic forces. In a typical steel MRF building, there can be several different MRF along each of the direction of the seismic forces. Each frame attracts a percentage of the total lateral force according to its relative stiffness. The distribution of the forces amongst the members of each frame highly depends on the relative stiffness of its structural members; i.e. beams and columns. The design of beams and columns is performed according to this force distribution. After the first round of design, many beams and columns need to be replaced with new sections; hence the building will have a new force distribution which necessitates another design cycle for the building. This iterative analysis-design procedure continues until the design no longer needs to alter any of the beams and columns. Thus, the seismic design of steel MRF buildings is highly sensitive to changes in the stiffness of the structural members, and any retrofit method should be able to avoid these changes; otherwise the entire building may need to be redesigned.

The higher level of resistance of the buildings that are designed for higher seismicity originates from the fact that a greater base shear is used in their design which reflects on the design of the beams and columns of the MRFs. Between beams and columns, it is the beams that play the major role in providing resistance against a column removal event, and any changes in the stiffness of the columns does not make a significant difference as long as they are able to resist the forces arising from the beams. According to the results presented in Chapter 4, the buildings that failed upon losing any of their columns have major shortcomings in the vertical stiffness and strength of the beams in the vicinity of the removed columns. Therefore, a retrofit strategy should be able to improve these major deficiencies. Figure 5.1 is a schematic diagram showing the needed increase in the vertical stiffness of the beams of an existing susceptible building that has static load capacity less than twice of (DL+0.25LL). The targeted vertical load capacity of a successful retrofit scheme is to reach at least 2×(DL+0.25LL).

Existing guidelines for progressive collapse are mainly concerned about the removal of the columns of the ground floor level. This could be attributed to the possibility that these columns are usually more susceptible to being damaged or lost. However, there are many cases that a column loss can happen in a floor above the ground floor level. This requires the proposed retrofit scheme be capable of improving the resistance of the building in order to safeguard against possible column losses at different levels.

The retrofit strategy's concept can also be directly incorporated in the design of new buildings in order to provide the required resistance against progressive collapse. For the existing buildings, the retrofit intervention should aim for minimum cost and least interference with the activities of their occupants.

There have been some recent research efforts to mitigate the probability of progressive collapse in steel frame buildings. Some of these works suggest the application of compound structural systems such as outrigger or belt trusses and in some cases mega trusses. These methods are mostly applicable in very high-rise buildings. Some others insist on increasing the strength and/or stiffness of the building's structural members in order to upgrade them such that they can have higher resistance against threats such as progressive collapse.

Many of the above-mentioned methods are not effective in retrofitting an already seismically design steel MRF building. Using trusses or braces in MRFs can significantly change the lateral force distribution and necessitates major changes in the original seismic design of the building. Upgrading the structural members such as beams and columns can lead to the same problem while it may also be uneconomical and interfering with the building's operations and functionality.

Therefore, in order to retrofit an existing seismically designed steel MRF building against progressive collapse, the retrofit method should be able to:

- Avoid changes in the lateral stiffness and mass of the structure in order to minimize any change to its fundamental frequency which can consequently alter the seismic design of the building.
- Aim directly for the main deficiency of the building which is the lack of vertical stiffness and strength in the vicinity of the removed column (providing gravity support only).
- Provide resistance against removal of any of the columns of building; not only the ground floor columns.
- Be cost efficient and have minimum interference to the occupants of the building.

Based on the above-mentioned recommendations, this chapter introduces two proposed retrofit methods, namely, the top beams grid method and the top gravity truss method that will be discussed in the following sections.

5.2 Top Beams Grid Method for Retrofitting

5.2.1 Introduction

As explained earlier, the main deficiency in buildings that do not survive column removals is the lack of vertical stiffness and strength in the vicinity of the removed column. The existing vertical stiffness in this area is attributed to the stiffness of the beams. The use of top beams grid as a retrofit method aims to compensate the missing vertical stiffness for the building (upon loss of any of its columns) with minimal effect on the building's seismic design. This structural system consists of a grid of steel beams that are installed above the existing steel beams of the roof floor of the building. Installing such a grid of beams would require an extension of the columns of the top floor so that the new beams can be connected to them.

This grid of top beams is added to provide extra stiffness and strength in the vertical direction that the building needs in order to withstand a column removal scenario, and as a result, it may consist of deep beams with much higher stiffness than other existing beams of a typical building in a low seismic zone. Figure 5.2 shows a schematic view of the top beams grid system, where Figures 5.3 and 5.4 show and illustration of an ETABS model of a 5-story building using this structural system.

This structural system will be able to provide support for the columns from the top floor up to the location of the removed column through tension; therefore, it can provide resistance against any column removals not only at ground floor level but at any level along the building's height.

5.2.2 Design of the Top Beams Grid System

The design of the top beams grid system proposed in this study is conducted based on the results from the nonlinear static (pushdown) analysis presented in Chapter 4. GSA guidelines require the application of 2×(DL+0.25LL) as the load combination when static analysis is performed. Figure 4.3 shows the ultimate vertical load capacity of the buildings located in Toronto (low seismicity) for the six column removal scenarios. According to this figure, none of the buildings of this zone is able to reach the 200% load which represents the GSA static load combination. The ultimate vertical load capacity of the buildings for each column removal scenario can also be expressed as a ratio multiplied by the GSA dynamic load combination. In this study, this ratio is called Capacity Ratio (CR). Therefore, the ultimate vertical load capacity is CR×(DL+0.25LL).

The values of CR for different column removal scenarios of the buildings located in Toronto are presented in Table 5.1.

The top beams grid system should be designed such that the building will be capable of resisting the GSA static load combination after the column is removed. In order to design the members of the top beams grid, a simple method is used to estimate the loads arising from every column removal scenario. In this method, the entire building with the top beams grid system (and all columns) is modeled in ETABS, and linear static analysis is performed for two different load combinations; the ultimate capacity load combination which is CR×(DL+0.25LL), and the GSA static load combination which is 2×(DL+0.25LL). As shown schematically in Figure 5.5, these load combinations are applied to the building separately, and the axial force in the column which is intended to be removed is calculated in each case ($P_{capacity,orig.}$ and P_{GSA}).

When the column is removed under the GSA load combination, the original un-retrofitted building is able to resist not more than the vertical load capacity in the vicinity of that column (measured as CR×(DL+0.25LL)). Therefore, the remaining part of the targeted GSA load combination resistance, i.e. 2×(DL+0.25LL), would be resisted by the retrofit system, which is the proposed top beams grid system in this case. In the procedure of the design, the top beams grid system will work along with the immediately affected beams in the vicinity of the removed column to resist the vertical loads that were carried by that removed column. Therefore, in order to design the top beams grid system, it will be initially assumed to be disconnected from the top floor connection along the same vertical axis of the removed column as shown in Figure 5.6., and their rotation at this point is assumed to be restrained in order to account for the flexural stiffness coming from the extended column of the top floor.

Figure 5.6 shows the downward vertical load which is used to design the members of the top beams grid system in the case of a loss of a corner column. This process should be repeated for other locations of anticipated column losses based the layout of the columns in the building. The affected beams of the grid should be able to take the design load which equals "$P_{GSA} - P_{capacity,orig.}$". All of the beams of the grid are designed from wide flanges steel I-sections made of 350W structural steel. Similar to the affected beams of the original building surrounding a lost column, the main load resisting mechanism in the beams of the top grid is the flexure. These beams are designed such that their maximum bending moment resulting from the aforementioned design load ($P_{GSA} - P_{capacity,orig}$) is less than their yielding moment (M_y). As can be seen in the 3D model perspective shown in Figure 5.4, in each of the six column removal scenarios, two, three, or four beams in the top grid will be affected and hence need to be designed. For simplicity, in the design of the proposed retrofit system in this study, only two I-sections were used in the top beams grid system of each building; one for the interior beams and one for the exterior beams. Table 5.2 includes these designed steel sections for the buildings located in Toronto (low seismicity).

As mentioned in the previous section, the top beams grid system provides extra vertical stiffness and strength by holding the columns above the removed column. This may exert tensile forces in some of these columns. In the AEM models of this study, the tensile forces were in an acceptable range and did not require any changes in the columns of the buildings. However, it is important to mention that this issue can be more severe and need to be considered in the columns of the higher floors especially in high-rise buildings which have very low original resistance against column removal events.

5.3 Top Gravity Truss Method for Retrofitting

5.3.1 Introduction

The buildings designed in the low seismic zone lack vertical stiffness and strength in the vicinity of the removed column when they are subjected to column removal scenarios. The proposed top gravity truss system for retrofitting the buildings uses the same concept as the top beams grid system in that it aims to increase the building's vertical stiffness and strength in the vicinity of the removed columns such that it will be able to survive column removal events.

The top gravity truss system consists of paneled trusses on the roof that are connected to all of the columns of the building. When an instantaneous column loss occurs, the truss works as a gravity support and holds the column from the top of the building and prevents the collapse.

The top beams grid system increases the vertical stiffness of the building through the flexural action of its beams. In high-rise buildings which have very low resistance against column loss events, this may result in using very deep and heavy steel beams. This can be uneconomical, and may also impose a significant mass to the building which requires a revision in the seismic design. In these cases, relying on the axial action of the structural members of the retrofitting system rather than their flexural action would be more effective; i.e., using a truss instead of the top beams grid. Since the truss members present their axial stiffness, trusses can provide significant vertical stiffness while having members with relatively small sections.

5.3.2 Design of the Top Gravity Truss System

Figures 5.7 and 5.8 illustrate a model of a 5-story building featuring top gravity truss system in ETABS. The top gravity truss can have different orientations and shapes depending on the structural and architectural considerations. In order to evaluate the feasibility of this retrofit method and also to provide simplicity in the models, in this study a simple form of truss is considered to be installed on the buildings. The height of the truss is 3.65m which is the same as the height of the stories of the building, and it is installed at 0.5m above the roof level. This height is considered in order to provide the space needed for installing the truss system and its connections to the columns of the building. Two diagonal members are designed in each span of the truss. It is assumed that the truss members have the same axial behavior in tension and compression. These members are made of the same steel used in the beams and columns of the building which is 350W structural steel. It is worth mentioning that depending on the building and its design requirements, a system of high-strength cables may also be used instead of the truss system.

Similar to the top beams grid system, application of top gravity truss may induce tension in some of the columns above the removed column. In the models of this study, these tensions had acceptable values and did not require any further changes in the building. However, particularly in high-rise buildings some of the columns of higher floors may need improvements to be able to transfer the tensile forces.

According to the assumed details for the top gravity truss system in this section, the only remaining parameter of this system which needs to be designed is the cross section area of its members. The design uses the same approach as the one explained in section 5.2.2 for the top beams grid system, and it is based on the results of pushdown analysis presented in Chapter 4. The original building is able to resist the ultimate capacity load combination which is the Capacity Ratio (CR) multiplied by the GSA dynamic load combination. The values of CR are listed in Table 5.1 for each column removal scenario of the buildings located in Toronto (Low Seismicity). Therefore, the ultimate load combination is CR×(DL+0.25LL). The remaining required capacity must be provided by the retrofitting system (top gravity truss) so that the structure will be capable of resisting the GSA static load combination which is 2×(DL+0.25LL).

Figures 5.9 and 5.10 illustrate the building's models and the process of calculating the concentrated design force. For each column removal scenario, a separate analysis is conducted to find the design force, and then by analyzing the building under the designed force, the cross section areas needed in the truss members are designed so that the stresses in the truss members do not exceed the yielding stress of steel (f_y). This leads to a different designed truss for each of the six column removal scenarios. Finally to present a simple and unified design for the top gravity truss system for each building, the highest cross section area resulted from the six different designs is used for all of the truss members. Table 5.3 shows the designed cross section areas for the 5, 10, and 15-story buildings of the low seismic zone.

Table 5.1 Capacity Ratio (CR) values for the buildings located in Toronto (Low Seismicity)

5-Story						10-Story						15-Story					
CC	FIC	FLEC	IC	LEC	SEC	CC	FIC	FLEC	IC	LEC	SEC	CC	FIC	FLEC	IC	LEC	SEC
0.95	0.88	0.93	0.92	1.00	0.93	1.17	1.06	1.10	1.09	1.26	1.12	1.35	1.13	1.24	1.18	1.30	1.24

Table 5.2 Designed sections for the top beams grid system used in the buildings located in Toronto (Low Seismicity)

	Designed Section for the Top Grid	
Building	**Exterior Beams**	**Interior Beams**
5-Story	W690X140	W690X240
10-Story	W690X217	W760X434
15-Story	W690X240	W920X537

Table 5.3 Designed cross sections areas for the members of the top gravity truss system used in the buildings located in Toronto (Low Seismicity)

Building	Designed Cross Section Area for the Truss Members (cm^2)
5-Story	40.00
10-Story	70.00
15-Story	85.00

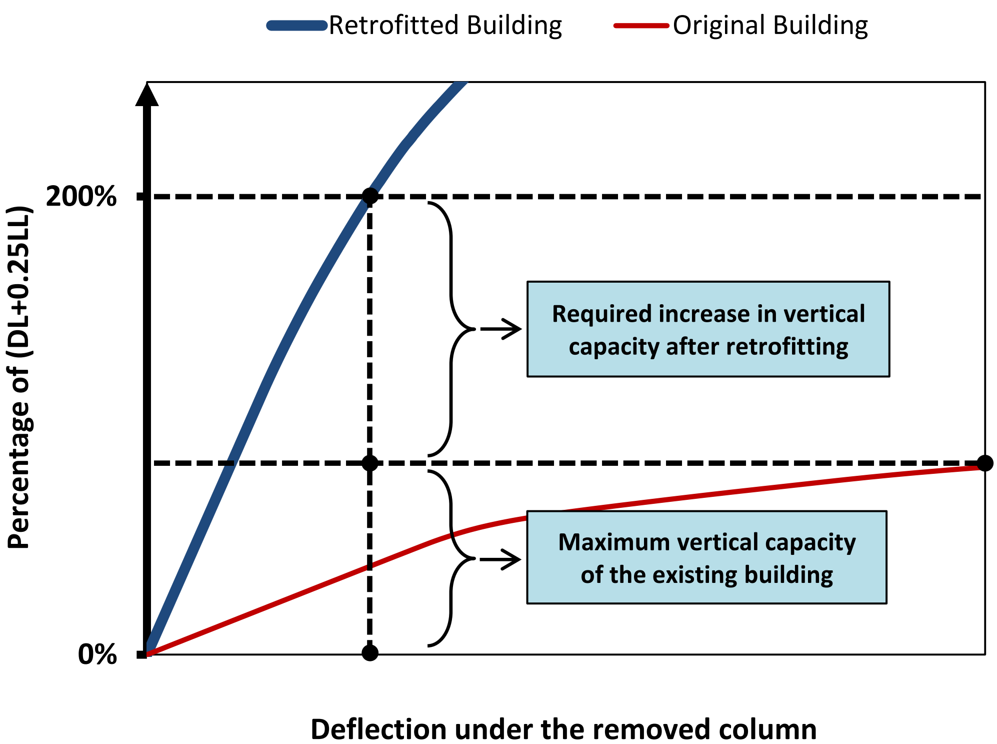


Figure 5.1 Schematic diagram of incremental pushdown analysis results before and after retrofitting a building

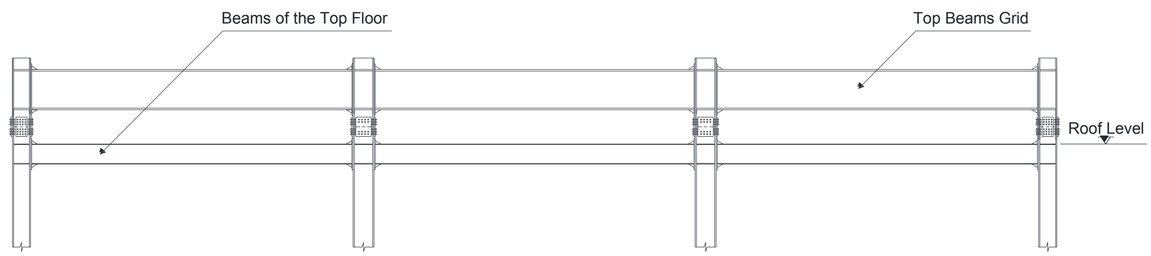


Figure 5.2 Schematic view of the top beams grid system

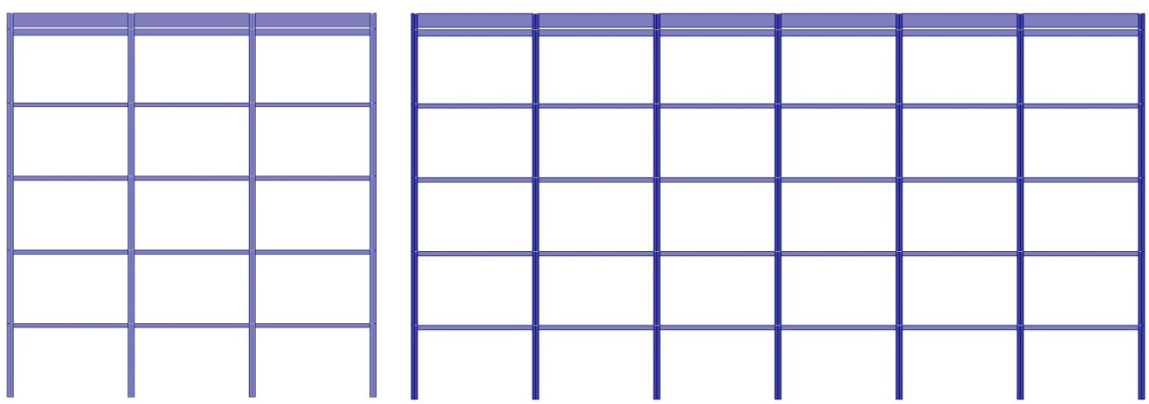


Figure 5.3 Two elevation views of model of a building with top beams grid system in ETABS

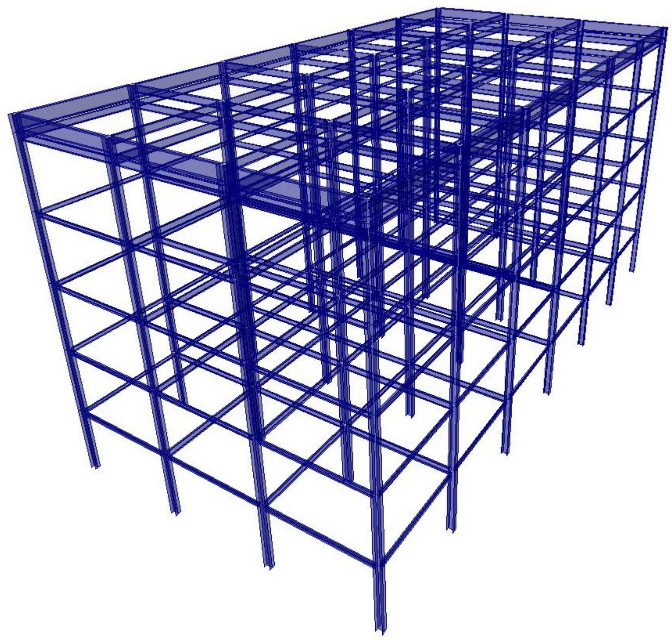


Figure 5.4 3-D views of model of a building with top beams grid system in ETABS

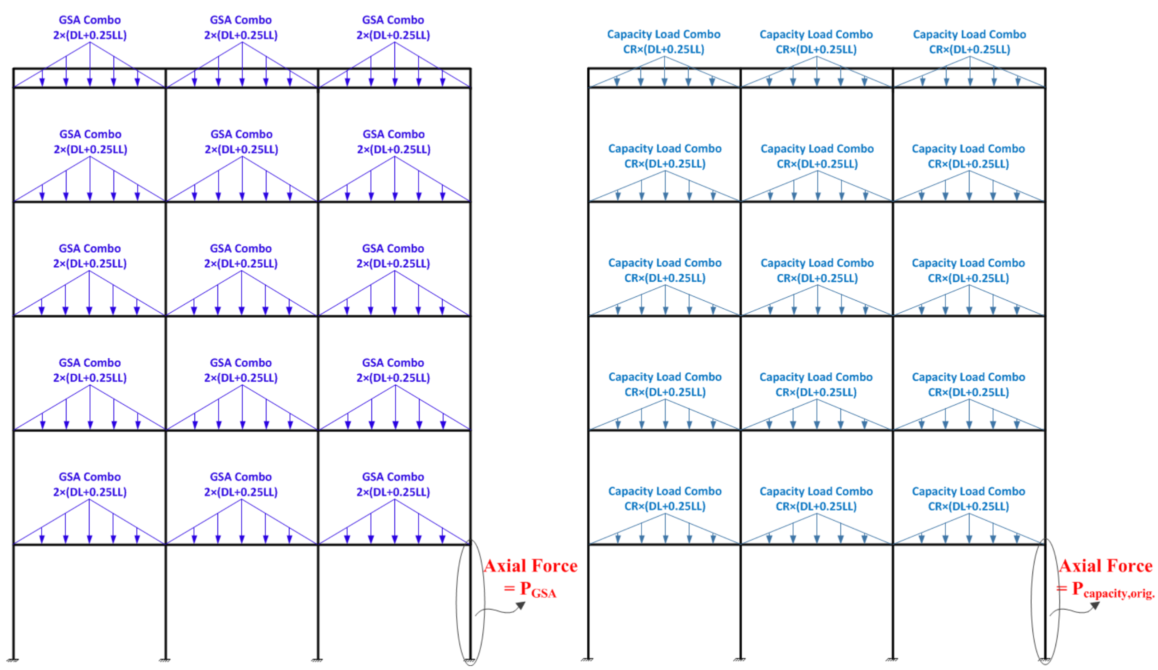


Figure 5.5 Schematic elevation view of the 5-story building with top beams grid system: (a) under the GSA static load combination; and (b) under the capacity load combination

P_{GSA}

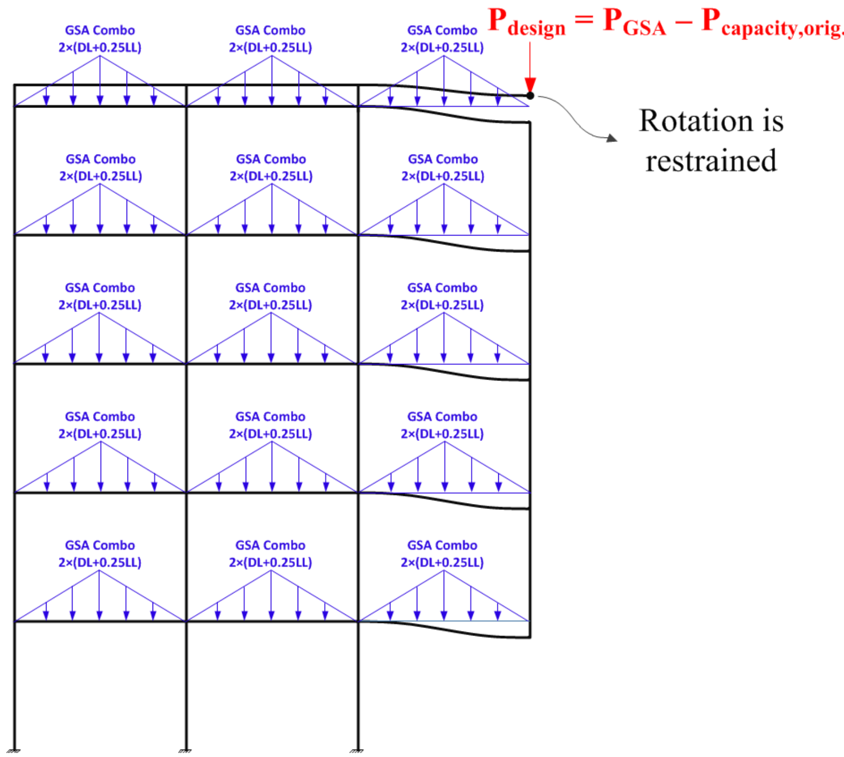


Figure 5.6 Schematic elevation view of the 5-story building with top beams grid system used to design the members of the beams grid

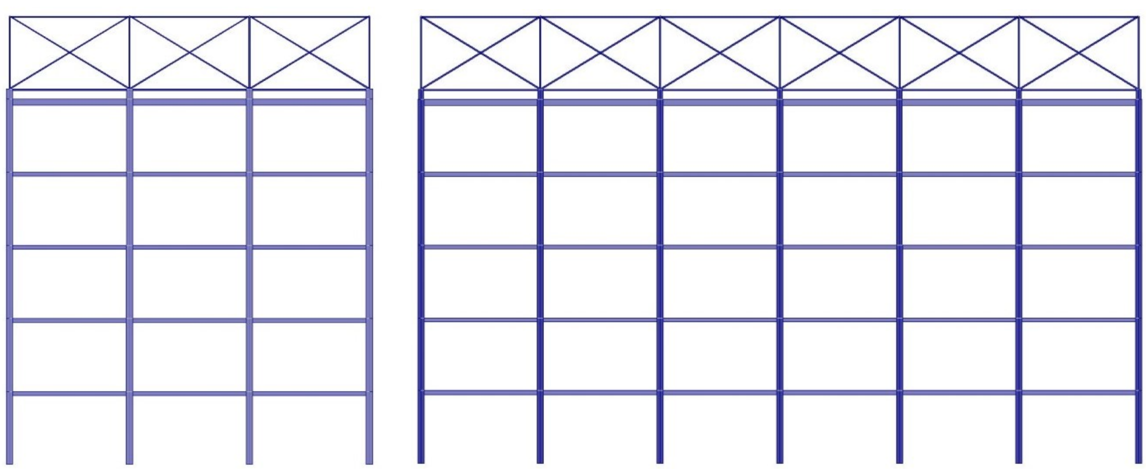


Figure 5.7 Two elevation views of model of a building with top gravity truss system in ETABS

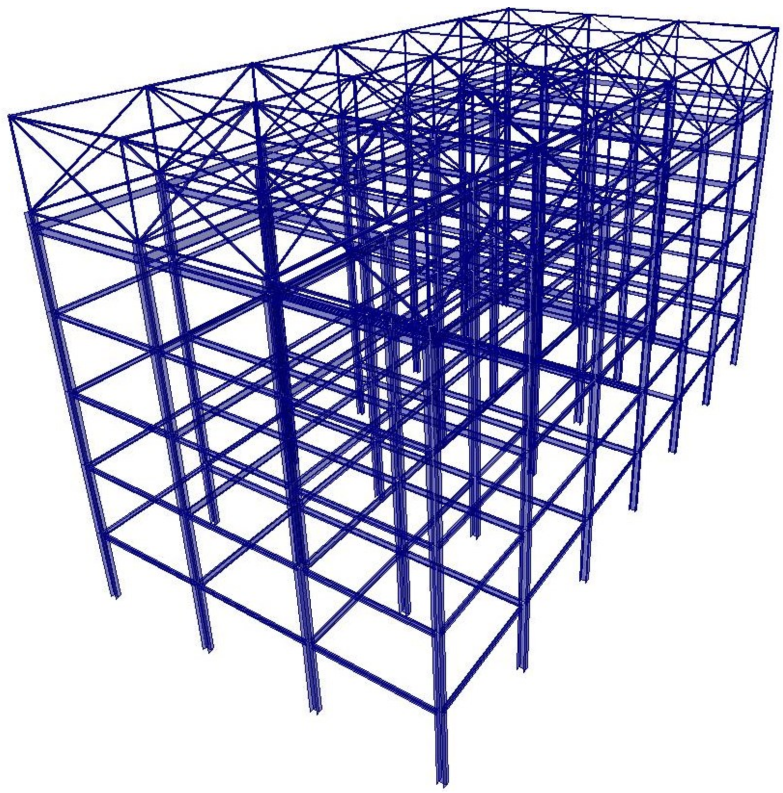


Figure 5.8 3-D views of model of a building with top gravity truss system in ETABS

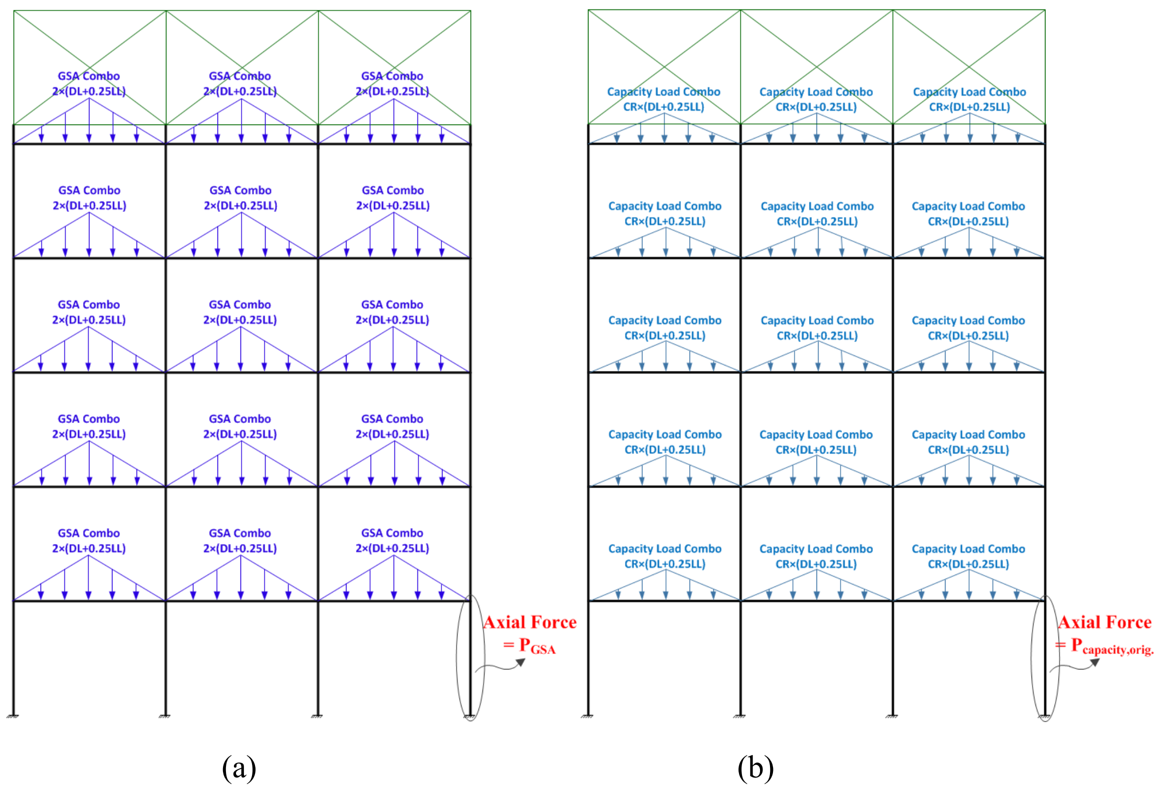


Figure 5.9 Schematic elevation view of the 5-story building with top gravity truss system: (a) under the GSA static load combination; and (b) under the capacity load combination

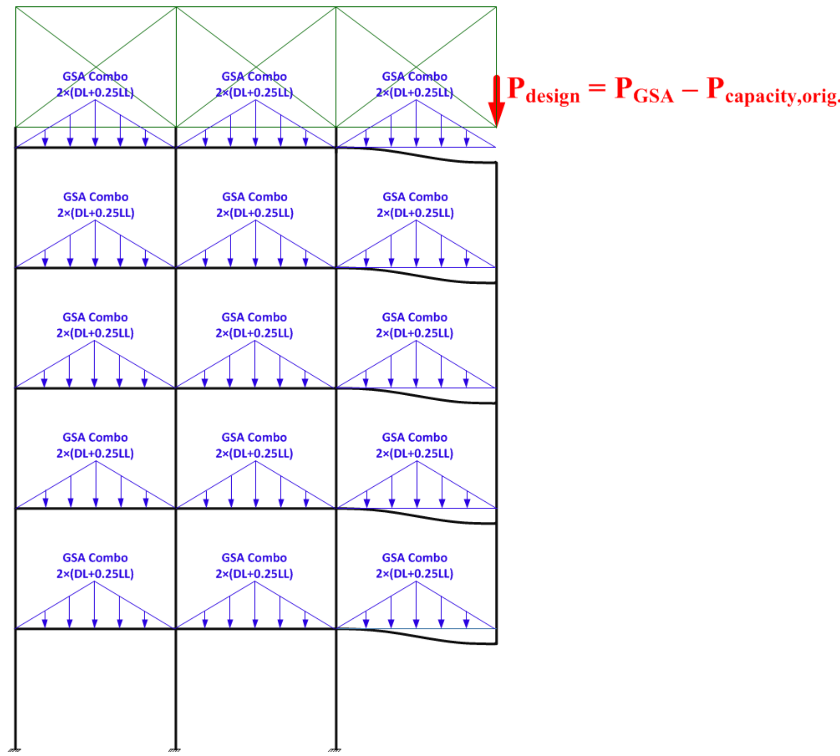


Figure 5.10 Schematic elevation view of the 5-story building with top gravity truss system used to design the members of the truss

Chapter 6
ROBUSTNESS OF THE RETROFITTED BUILDINGS AGAINST PROGRESSIVE COLLAPSE

6.1 Introduction

Chapter 5 introduced two proposed structural systems that could be constructed at the top of existing steel MRF buildings that are susceptible to column removals and progressive collapse as means of increasing their robustness against such threats. In this chapter, these retrofit methods are applied to buildings that were originally designed for a low seismicity, and models of the buildings are then analyzed using the ELS software. This chapter presents the results of nonlinear static and nonlinear dynamic analyses of all the models of the retrofitted buildings in the form of comparative charts and figures, and they are discussed in detail. Moreover, the dynamic increase factors and load distribution capabilities of the original and retrofitted buildings when they experience column removal events are presented.

6.2 Nonlinear Static and Nonlinear Dynamic Analysis

The ultimate vertical capacity of the building in the vicinity of the removed column is considered as a major parameter in the design of the top beams grid and the top gravity truss systems in this study, and it is obtained based on the results of the pushdown analysis.

Figure 4.3 presents the ultimate vertical load capacity of the buildings located in the low seismic zone for the six column removal scenarios. This figure shows a significant margin between the highest static load capacity of these buildings and the 200% level of load which is required to be applied on the building under the GSA static procedures. The top beams grid and top gravity truss systems are designed in order to compensate for this difference and the buildings' lack of resistance.

Figure 5.1 shows a schematic of pushdown analysis results for a building that lacks resistance against column removals before and after being ideally retrofitted in order to resist the GSA static load combination. Figure 6.1 includes the results of pushdown analyses of the 5-story buildings located in the low seismic zone before and after being retrofitted using the two proposed retrofit methods. The figure shows major improvements in the vertical stiffness and strength of these buildings after being retrofitted. The two retrofitted buildings were able to resist the targeted 200% of the GSA dynamic load combination while having deflections less than 100 mm in their 6-meter span bays upon loss of any columns in the ground floor.

Considering the layouts of the beams grid and gravity truss in the case of this study and the fact that the design is performed based on the required strength in their respective members, it is expected that the truss system is more capable of increasing the vertical stiffness at the point of the removed column than the beams grid system. This can be clearly seen in Figure 6.1 as the pushdown force-deformation relationships of the buildings with top gravity truss system have greater stiffness.

Figures 6.2 and 6.3 show the chord rotation of the beams connected to the removed column using static and dynamic procedures, respectively. The figures show smaller values for the chord rotation in the buildings retrofitted with the top gravity truss system than those retrofitted using top beams grid system in both static and dynamic procedures. As explained in the previous paragraph, this is mainly due to the fact that, in this particular layout and design, the vertical stiffness in the vicinity of the removed column can be increased more effectively by the top gravity truss.

Another important observation from these two figures is that the chord rotation has generally greater values in taller buildings. In fact, in all of the six column removal scenarios of the retrofitted buildings, with the chord rotation of the 10-story building being greater than that of the 5-story building, the chord rotation of the 15-story building has the greatest value. This shows that the effectiveness of such retrofit systems to increase the vertical stiffness in the vicinity of the removed column reduces with increasing the height of the buildings. This has one main reason. When the column is removed at the ground floor level, the retrofit system on top of the building will hold the columns above the removed column and exerts tensile force in them. Therefore, some of the deflection under the removed column at the ground floor level is attributed to the elongation of the columns above the removed column which are now in tension. In taller buildings, this elongation can be more significant because the total length of these columns and their tensile force are higher. As a result, both of the proposed retrofit methods of this study are less effective in reducing the deflection under the removed column in taller buildings.

6.3 Dynamic Increase Factor (DIF)

The existing guidelines for progressive collapse often present two approaches to study this phenomenon; static approach and dynamic approach. As progressive collapse is associated with nonlinearity of material and geometry and is a dynamic phenomenon, nonlinear dynamic analysis seems to be the best choice in terms of accuracy. However, nonlinear static analysis with factored loads, to account for the dynamic effects, can also result in acceptable accuracy, and since it is static, require less time and effort.

The GSA guidelines use a factor of 2 as the dynamic increase factor. This factor can be derived based on simple equations of physics for linear elastic problems, but progressive collapse is not such a simple phenomenon. The GSA guidelines use a factor of 2 for both linear static and nonlinear static analysis without considering the effect of nonlinearity. Moreover, the factor of 2 would not be an appropriate factor when the structure experiences inelastic deformations, which occurs in almost all the cases of progressive collapse. Inelastic deformations of structural members lead to some extents of energy dissipation and decrease the DIF. This issue has been mentioned in some recent research works as well as the new version of DoD guidelines (DoD 2009).

Ruth et al. (2006) suggested that a factor closer to 1.5 can better represent the dynamic effect especially in steel MRFs, and in another study Tsai and Lin (2008) found that an amplification factor of 2 for the loads in seismically designed RC buildings may be conservative. DoD (2009) also addresses this issue and stipulates that the factor of 2 which is used in GSA (2003) and DoD (2005) guidelines is not an appropriate factor in the majority of linear static and nonlinear static analysis cases for progressive collapse. However, the DoD guideline does not present a general formula or procedure for calculating the DIF.

Chapter 4 clearly shows that application of the factor of 2 in static analysis can lead to conservative results in some cases. For instance, the buildings designed for the medium seismicity show satisfactory responses to column removal cases when using the dynamic analysis procedures while none of them is able to pass when the static procedures are used.

In this study, nonlinear static and nonlinear dynamic analyses are performed for all of the column removal scenarios. The maximum deflection under the removed column is calculated using the nonlinear dynamic analysis for the buildings which remain stable after the instantaneous column removals. Then, using the pushdown analysis curves (resulted from nonlinear static analysis) the static load that can induce the same deflection as the maximum deflection that resulted from nonlinear dynamic analysis is determined. This static load combination is the dynamic load combination (DL+0.25LL) multiplied by a factor which is the DIF. The results are presented in Figures 6.4 and 6.5.

Figure 6.4 compares the DIFs of the buildings of the medium and high seismic zones. As explained in previous chapters, in the process of conducting the seismic design of steel MRF buildings, higher seismicity leads to sections with higher strength and stiffness for both beams and columns. Therefore, in the buildings of the high seismic zone, the beams in the vicinity of the removed column have greater flexural stiffness and strength compared to the case in the buildings of the medium seismic zone. This means that when a column is removed, more yielding (inelastic deformation) and consequently energy dissipation is expected in the buildings of the medium seismicity. The energy dissipation also happens in the buildings of the high seismic zone but to lower extents. As a result, the DIFs of the buildings of the high seismic zone stay close to 2 (the factor for a linear elastic case) while DIFs of the buildings of the medium seismic zone have smaller values roughly about 1.5.

Figure 6.5 compares the DIFs of the buildings of the low seismic zone which are retrofitted using two different methods; top beams grid system and top gravity truss system. In these retrofitted buildings, a significant amount of resistance against column removal events originates from the retrofit system which is installed on top of the building. The rest of the structural members of these buildings, especially their beams, are weaker and more susceptible to inelastic deformation than those of the buildings in the higher seismic zones. The DIFs of retrofitted buildings are mostly less than 2 and about 1.8. The Figures 6.4 and 6.5 clearly prove that different DIFs should be used depending on the structural characteristics of the buildings, and using a single value of 2 as the DIF for all types of buildings can lead to conservative results in many cases.

With the limited number of models of this study, it is still early to present a general formula or procedure for calculating the DIF. This topic needs a comprehensive study of the nonlinear inelastic behavior of different structural systems and is beyond the goals of this research work.

6.4 Distribution of Gravity Loads

Alternate Path Method (APM) is the most common method used for evaluating a building's resistance against progressive collapse. In this method a vertical load bearing member of the structure (such as a wall or column) is assumed to be instantly lost, and building's response to this extreme event is evaluated. This process demonstrates the building's ability to transfer the vertical loads through new load paths while it remains stable. Although an extreme effect on a structure can be caused by unpredicted events such as explosions, car collisions, or missile impacts; it is very clear that they all will be followed with very rapid loss of vertical load bearing structural members. This is the main reason which makes APM a useful method for evaluating the building's resistance against extreme cases.

If the loads of the removed member are transferred to a few other structural members only, these members may experience significant increase in their internal forces which can eventually lead to their failure. However, a building with high resistance against progressive collapse should be able to well distribute the forces of the lost vertical elements throughout the entire building rather than only to a few other structural members.

Figure 3.8 shows the location of the six column removal scenarios which are considered in this study on the buildings' plan. In this section, an interior column removal scenario (IC) is chosen to be investigated to acquire a better understanding of the building's capability to distribute the axial force of the removed column among other columns. This column removal scenario (IC) is considered in all of the buildings of the high seismic zone and the retrofitted buildings of the low seismic zone. In steel MRF buildings, when a column is removed, the most affected columns are usually the adjacent columns of the removed column. The increase in the axial load of the adjacent columns can be so significant that they may fail under the new loads. Therefore in this section, only changes in the axial forces of the adjacent columns are considered.

The buildings are first analyzed under the GSA's static load combination, i.e. 2×(DL+0.25LL), and the axial forces in the interior column (IC), which is intended to be removed, and its 4 adjacent columns are measured. Then the column is removed and nonlinear static analysis (pushdown analysis) is performed until the loads reach the GSA's static load combination. At this point, the axial forces of the adjacent columns are measured again, and the increase in their axial forces is calculated in terms of the percentage of the axial force of the interior column (IC) before it was removed.

Figures 6.6, 6.7, and 6.8 show the percentage of the axial load of the interior column (IC) which is transferred to its adjacent columns after its removal for all of the buildings of the high seismic zone and the buildings of the low seismic zone retrofitted with the two proposed retrofit methods. In addition, Figure 6.9 includes the total percentage of the axial force of the ground floor interior column (IC) which is being transferred to its adjacent columns. Since all of the buildings of the high seismic zone and the retrofitted buildings were able remain stable (i.e. did not have partial collapse) after they lost the column, it can be concluded that the remaining axial force of the removed column has been transferred to the rest of the structure apart from the adjacent columns.

According to Figure 6.9, in all the three cases of the buildings of the high seismic zone, the building retrofitted with top beams grid system, and the buildings retrofitted with top gravity truss system, the percentage of the transferred axial load to the adjacent columns is less in taller buildings. This is another proof of the positive effect of the higher redundancy in progressive collapse resistance of the buildings. The buildings with higher number of stories have more contributing elements -through redundancy- that provides them with a better vertical load distribution throughout the building in column removal events.

Moreover, the percentage of the transferred axial load to the adjacent columns of the removed column has smaller values in the buildings retrofitted with top gravity truss system compared to the building retrofitted with top beams grid system or designed for high seismicity. This highlights the effectiveness of top gravity truss system in enhancing the building's capability to distribute the loads through alternate paths. This is mainly due to the fact that members of top beams grid system provide resistance by their flexural action while the members of the top gravity truss system provide the resistance against column removals through their axial stiffness which is a more effective way. From the same figure, it could be also seen that there are not noticeable differences between the values of the chart for the buildings of the high seismic zone and the buildings retrofitted with top beams grid system as they are both resisting the extreme case of a column loss with the flexural action of their beams.

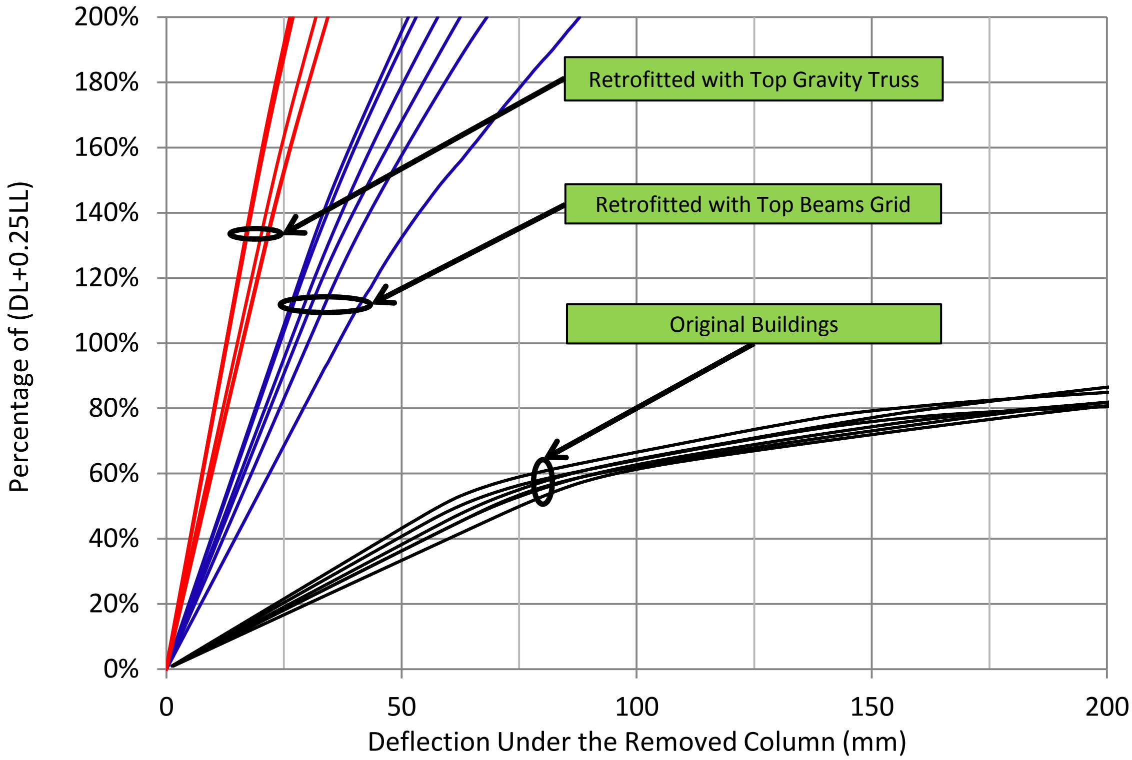


Figure 6.1 Pushdown analysis results of the 6 column removal scenarios for the 5-story buildings located in Toronto (Low seismicity) before and after being retrofitted

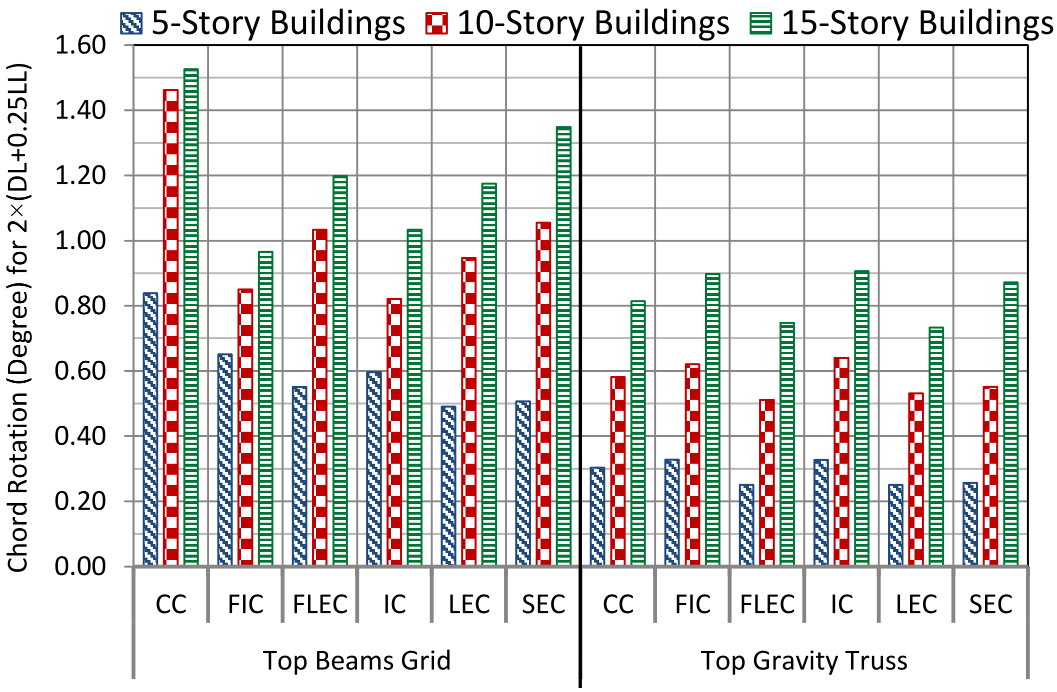


Figure 6.2 Chord Rotation of the buildings of the low seismic zone after being retrofitted with the two retrofit methods (pushdown analysis)

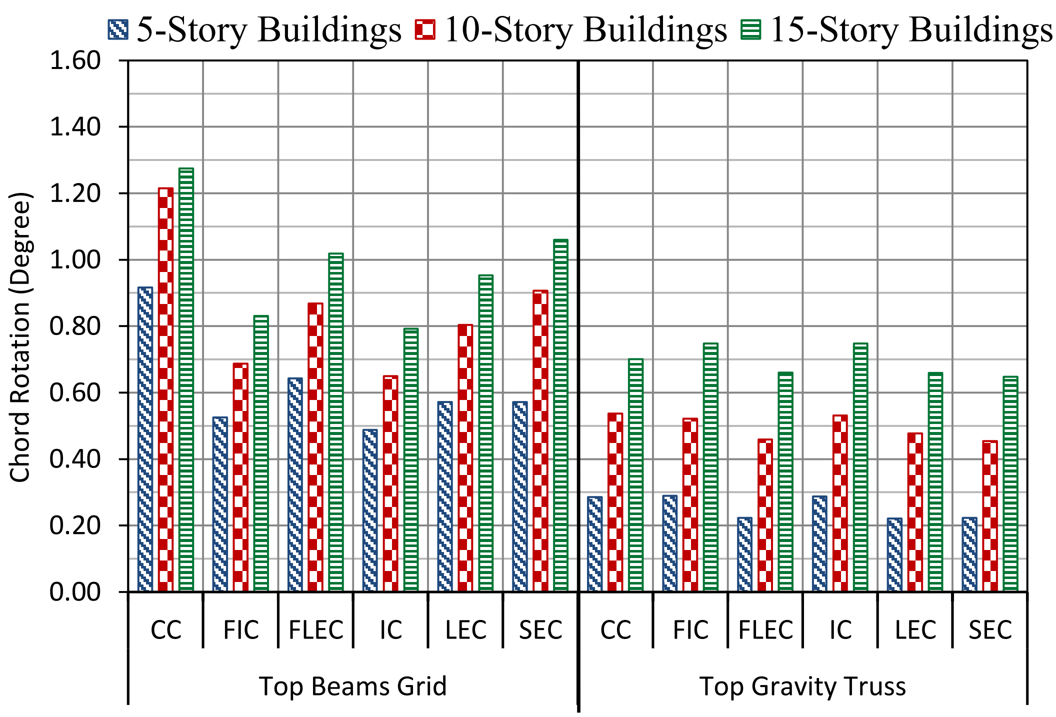


Figure 6.3 Maximum Chord Rotation of the buildings of the low seismic zone after being retrofitted with the two retrofit methods (dynamic analysis)

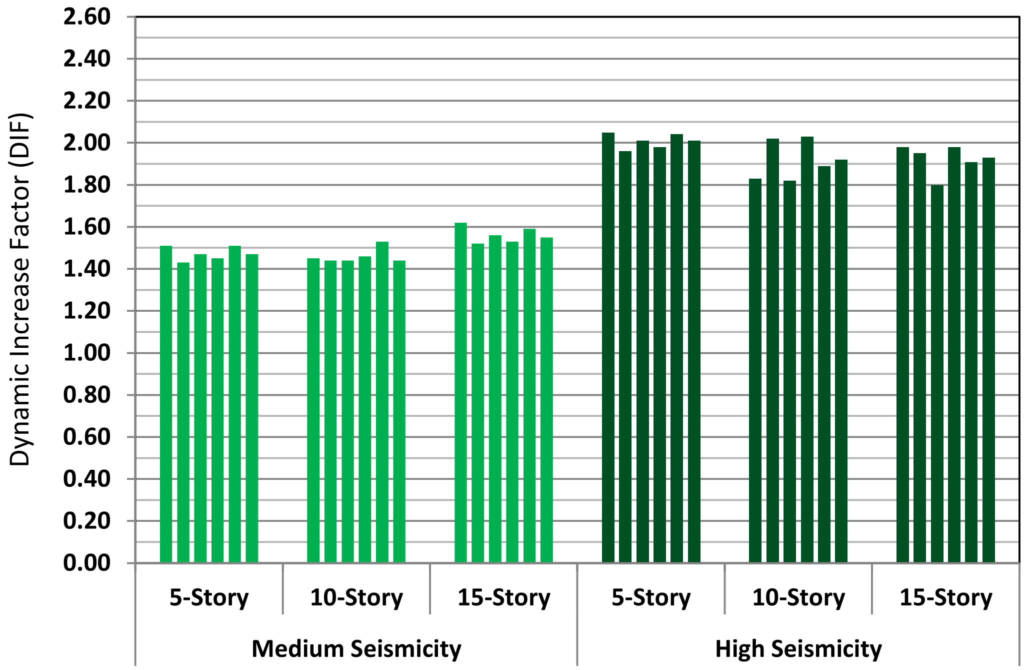


Figure 6.4 Dynamic Increase Factors (DIF) of the six column removal scenarios for the buildings designed for medium and high seismicity

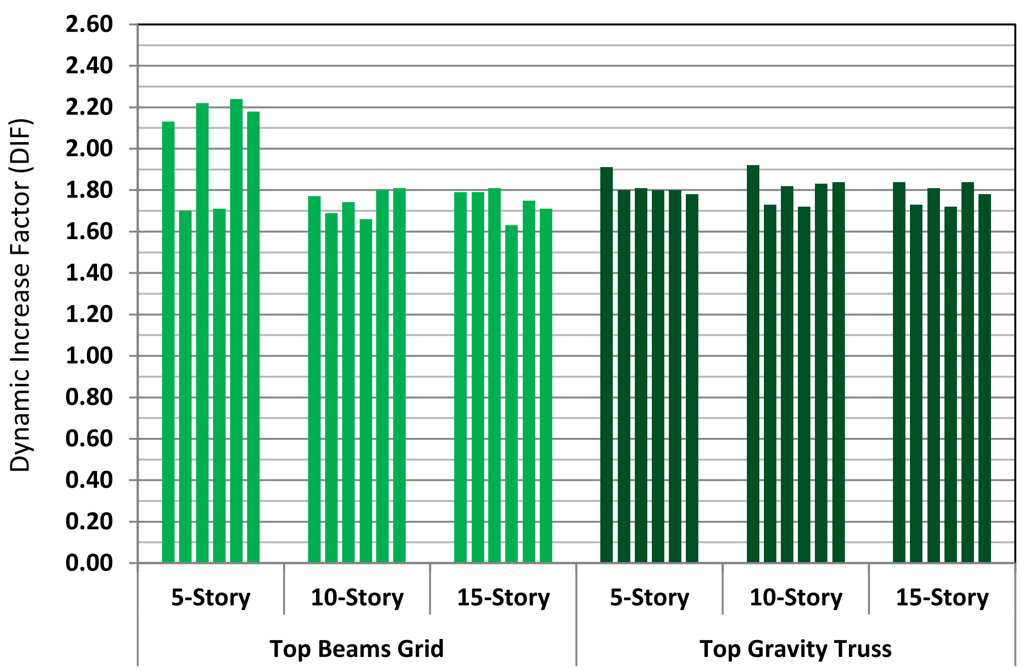


Figure 6.5 Dynamic Increase Factors (DIF) of the six column removal scenarios for the buildings retrofitted with the top beams grid and top gravity truss systems

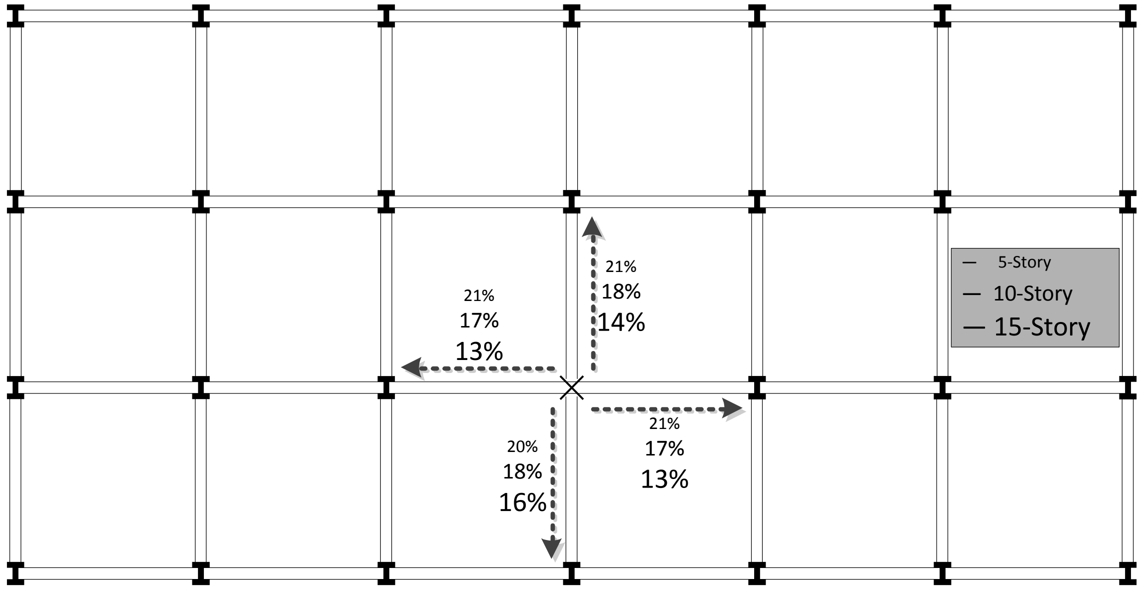


Figure 6.6 Percentage of axial load of the removed interior column (IC) transferred to its adjacent columns; buildings of the high seismic zone

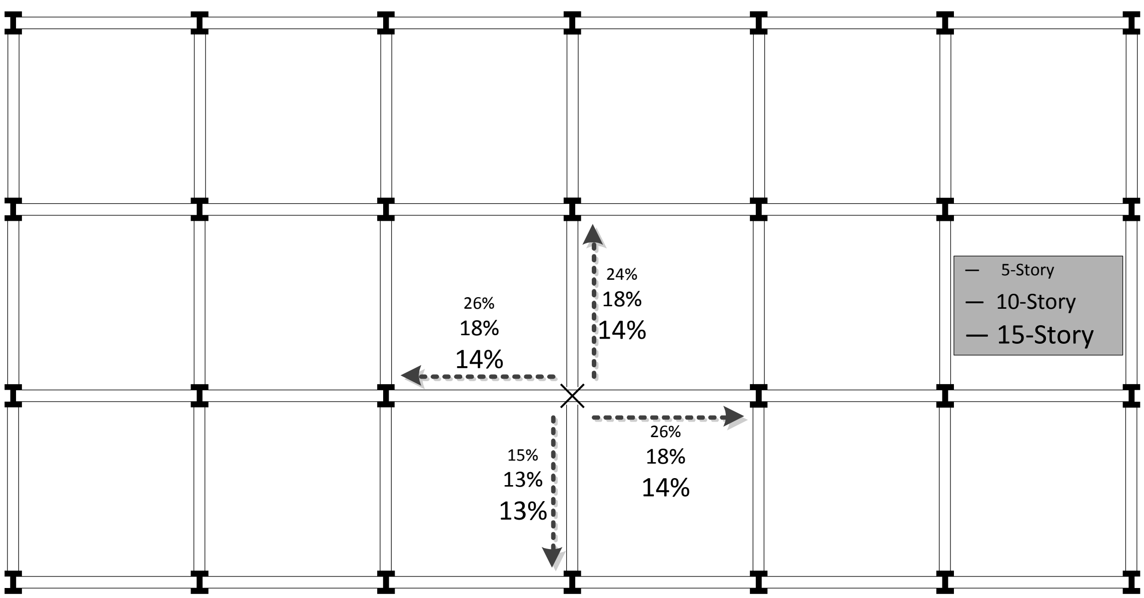


Figure 6.7 Percentage of axial load of the removed interior column (IC) transferred to its adjacent columns; buildings retrofitted with top beams grid system

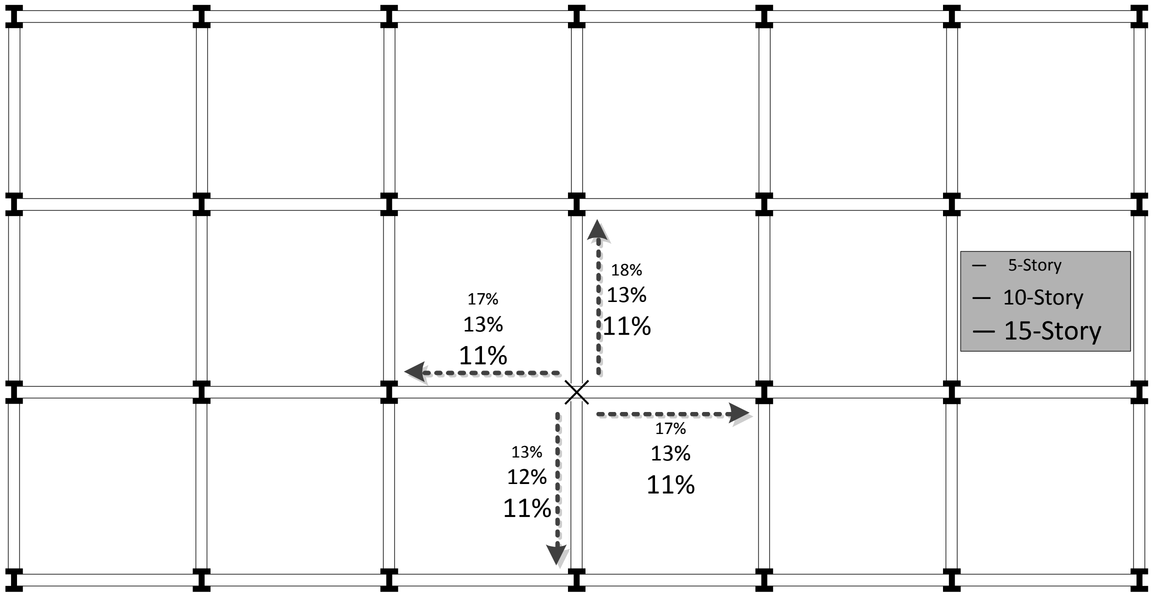


Figure 6.8 Percentage of axial load of the removed interior column (IC) transferred to its adjacent columns; buildings retrofitted with top gravity truss system

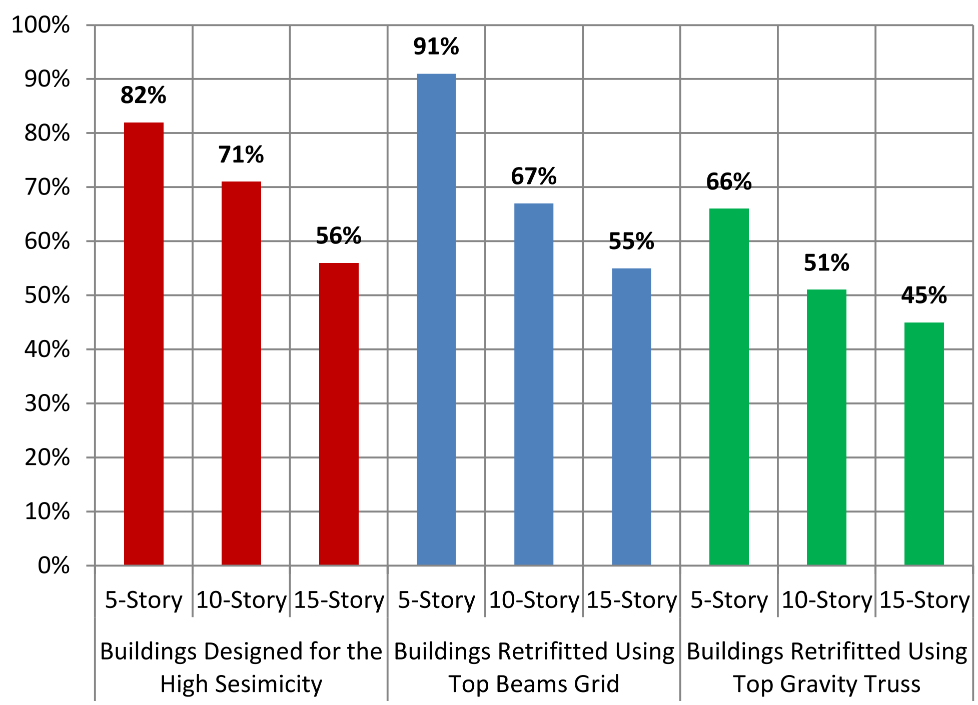


Figure 6.9 Total percentage of the transferred axial load of the removed interior column (IC) to its adjacent columns

Chapter 7
CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

This study assesses the vulnerability of seismically-designed multistory steel moment resisting frame buildings to progressive collapse, and proposes retrofit solutions for the buildings that are prone to collapse. The studied buildings had 5, 10, and 15 stories (representing low-rise, medium-rise, and high-rise buildings) and were designed for three seismic zones (representing low, medium, and high seismicity). All studied buildings had a 3-bays x 6-bays rectangular plan, each bay had a span of 6 meters. Alternate Path Method (APM) recommended by GSA (2003) guidelines is adopted to evaluate the robustness of the buildings against progressive collapse. Three-dimensional models of the buildings are built using ELS (Extreme Loading for Structures) software package, and nonlinear static and nonlinear dynamic time history analysis are conducted for six different column removal scenarios.

The buildings designed for low seismicity showed insufficient resistance against column removal cases, thus need to be retrofitted to safeguard against possibility of progressive collapse.

In this study, two retrofit methods using top beams grid system and top gravity truss system are proposed for buildings in low seismic zones in order to enhance their robustness against progressive collapse. The nonlinear static and nonlinear dynamic analyses of the retrofitted buildings using the ELS software showed the effectiveness of the proposed retrofit systems in mitigating progressive collapse.

7.2 Conclusions

The results of the nonlinear static and dynamic analyses are presented and scrutinized in Chapters 4 and 6 and can be summarized as follows:

[1] Seismic design of steel MRF buildings has a noticeable effect on their progressive collapse resistance. All the six column loss scenarios were followed by collapse for the buildings designed for low seismicity whereas all buildings of the medium and high seismic zones were able to survive these events. Furthermore, the buildings of the high seismic zone experienced smaller deflections under the removed column than the buildings of the medium seismic zone.

[2] Higher redundancy of the beams in the vicinity of the removed column can increase the ultimate vertical load capacity of the building in that area, which deems the high-rise buildings less vulnerable to progressive collapse compared to low-rise buildings.

[3] Higher redundancy of the beams in the vicinity of the removed column can result in better distribution of the forces in the building after the column loss events. This reduces the possibility of failure especially in the columns adjacent to the lost column due to the sudden increase in their axial forces.

[4] The amount of loss of the vertical stiffness in the vicinity of the removed column can be a useful indicator of the building's residual robustness after surviving a column loss event. For instance, the buildings of the medium seismic zone in this study showed satisfactory responses to column loss scenarios under the dynamic

load combination of the GSA guidelines; however, they lost up to 90% of their vertical stiffness in this area which brings them very close to their failure stage.

[5] The Dynamic Increase Factor (DIF) decreases as plastic deformations increase in the building during extreme cases such as instantaneous column removals.

[6] The factor of 2 used in GSA guidelines as DIF can be a significantly conservative factor in many cases of column removal. This may lead to poor assessment of the building's resistance against progressive collapse; especially if dynamic analysis is not performed. For instance, the buildings of the medium seismic zone in this study were found to have enough resistance based on the dynamic load combination of the GSA guidelines whereas they would be deemed to have failed under the static load combination if considering the GSA conservative DIF of 2.

[7] Both the top beams grid and top gravity truss systems were effective methods for retrofitting and increasing the progressive collapse resistance of the buildings of this study.

[8] Both the top beams grid and top gravity truss systems were seen to be less effective methods for high-rise buildings compared to low-rise buildings, as the deflection under the removed column has greater values in taller buildings.

[9] When the top beams grid and top gravity truss systems are designed using the same proposed design method presented in this study, the top gravity truss system is more effective in increasing the vertical stiffness of the buildings in the vicinity of the removed column; resulting in smaller deflections under the removed column.

[10] While the top beams grid system does not noticeably affect the ability of the buildings to transfer the loads of the removed column and distribute them among other structural members, the top gravity truss system significantly improves this ability of the buildings.

It is important to mention that the above conclusions are drawn based on the analysis results of the buildings of this study which are typical office buildings with regular structural configurations in plan and along the height. All of the frames of the buildings are assumed to be moment resisting frames in both directions. In order to generalize these statements, more buildings with different structural features and configurations need to be studied.

7.3 Recommendations for Future Research

This study investigates the response of multistory steel moment resisting frame buildings to the various scenarios of single column removal at the ground floor, as per the GSA criteria. This current work opened the door for few other research extensions that need to be investigated in order to better understand the response of steel buildings to extreme events that can lead to progressive collapse:

1- The effect of modeling of floor system (deck, secondary beams, etc.) on the response of the buildings to column removal scenarios.

2- The effect of column loss at other floor levels: All of the column removal scenarios of this study were considered to happen in the ground floor of the buildings. This is according to the recommendations of the existing guidelines for progressive collapse and also due to the fact that these columns are usually the critical columns to be removed. However, studying the buildings when they lose a column at other floors is seen to be important in order to fully understand the nature of buildings' responses to such events.

3- Simultaneous removal of two or more adjacent columns of the buildings: This could be important, because it will most probably be followed by failure of the buildings, and consequently, the focus of the research can be on the ability of the buildings to control and limit the collapse and prevent the collapse of the entire building.

4- Modeling the actual beam-column connection and proposing methods for strengthening the joint if needed.

5- Examining various design approaches for optimizing the retrofit schemes towards targeting a specific response indicator, such as a certain chord rotation or tie force.

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