Parametric Analysis of Progressive Collapse in High-Rise Buildings

By

Taras Gamaniouk

Bachelor of Science in Civil Engineering
Queen's University, 2012

Submitted to the Department of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of

MASTER OF ENGINEERING
in Civil and Environmental Engineering

at the

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

June 2014

Signature of Author: ______________________________
Department of Civil and Environmental Engineering
May 9, 2014

Certified By: ______________________________
Pierre Ghisbain
Lecturer of Civil and Environmental Engineering
Thesis Supervisor

Certified By: ______________________________
Jerome J. Connor
Professor of Civil and Environmental Engineering
Thesis Co-Supervisor

Accepted By: ______________________________
Heidi M. Nepf
Chair, Departmental Committee for Graduate Students

Parametric Analysis of Progressive Collapse in High-Rise Buildings

By

Taras Gamaniouk

Submitted to the Department of Civil and Environmental Engineering on May 9, 2014 in Partial Fulfillment of the Requirements for the Degree of Master of Engineering in Civil and Environmental Engineering

Abstract

Progressive collapse has become a topic of interest in recent years leading to a greater focus on the resilience of structures. The propagation of a local failure can become catastrophic and lead to multiple deaths, injuries and destruction of property. These types of events have been predominant in mid to high-rise buildings under both accidental and intentional circumstances. The dire consequences associated with these types of buildings have fueled research efforts into preventative measures for progressive collapse.

Three main design methods have been implemented for the design of progressive collapse: tie forces, enhanced local resistance and alternate load path. Each method features its own advantages and disadvantages; however, the alternate load path is currently the preferred procedure as it is accurate and capable of dealing with complex systems. This method is investigated in detail with a specific focus on nonlinear dynamic analysis. The technique is applied for three different structural systems which are commonly used for high-rise buildings: moment frames, braced frames and truss tube systems.

A variety of 2D structural models are analysed for their performance under progressive collapse conditions with variable building parameters. The results of the investigation infer that taller buildings are inherently better at preventing progressive collapse as the load is diminished throughout the building allowing less plastic hinges to form. This result was common in all three structural models with the braced frames exhibiting a better structural response to local failure in comparison to moment frame buildings. The study identifies the advantage of implementing hybrid structural frames for the prevention of collapse in high-rise buildings. Integration of moment frames for the lower stories of buildings is shown to be an effective mitigation method for progressive collapse.

Thesis Supervisor: Pierre Ghisbain

Title: Lecturer, Department of Civil and Environmental Engineering

Thesis Co-Supervisor: Jerome J. Connor

Title: Professor of Civil and Environmental Engineering

Acknowledgements

I would like to thank Pierre Ghisbain for his guidance with this thesis and constant availability regardless of the subject matter. His commitment to his students is extraordinary and greatly appreciated by the entire class. I would also like to thank Professor Connor for his advice and encouragement throughout the year as well as the rest of the M. Eng. faculty and administration for their commitment to their students.

Secondly, a big thank you to the rest of the graduating class for an unforgettable year. The completion of my degree would not have been possible without this awesome group of people. I hope that the friendships we formed remain for the rest of our careers.

Lastly, I would like to thank my family and friends back home for their constant support and reassurance to pursue my goals.

Table of Contents

List of Figures

List of Tables

1 Introduction

1.1 Progressive Collapse

Progressive collapse is a phenomenon which has gained a lot of attention over the last few decades. It is formally defined as “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it.” (ASCE, 2010). This propagation of a local failure has been creating concern for the structural integrity of buildings. These types of events can become catastrophic as the collapse may lead to multiple deaths, injuries and destruction of property in the immediate vicinity of the building. The consequences associated with progressive collapse are incredibly dire, which is why the topic has become of interest for government agencies, building owners and researchers.

The trigger of the phenomenon may be a result of one specific event or a combination of causes that lead to local failure. Examples of these causes include vehicular impact, earthquakes, fire, explosions as well as human error in design or construction of the structure. The majority of these causes occur under accidental circumstances, however, in light of recent events there has been an emphasis on deliberate initiation of progressive collapse. A typical example of this would be the intentional removal of a column by an explosion. The structural components of the floors above this column would experience a sudden increase in stress as well as large deflections. This amplification of the load may continue to cause failure in other primary members of the structure until the building stabilizes with noticeable deformations or until the complete collapse of the structure.

The interest in progressive collapse dates back to 1968 with the partial collapse of the Ronan Point apartment building in London, UK. The 23 story building featured a structural system with precast concrete walls and floors. The event originated from a leaking gas stove which caused an internal gas explosion on the 18th floor of the apartment building. The explosion removed an external wall which acted as the main support for the floors above. This caused the progressive collapse of the upper floors and successive failure down to the ground floor triggered by the debris of the initial explosion. The unfortunate accident, illustrated in Figure 1.1, resulted in four deaths and a number of injuries but it also led to an increased research focus on progressive collapse. Following the event, research was increased into preventative measures for this type of failure and the first progressive collapse provisions were incorporated in design standards (Kokot & Solomos, 2012).

Figure 1.1 - Partial collapse of the Ronan Point apartment building (Kokot & Solomos, 2012)

The attention towards progressive collapse continued to grow as the number of events began to increase over the years. A significant surge in research efforts occurred after two prominent terrorist attacks. The first of these being the Oklahoma City bombing of the Alfred P. Murrah Federal Building in 1995 followed by the collapse of the World Trade Center towers in 2001. These events differed from previous records of progressive collapse as they were the first to feature intentional damage which led to a large number of casualties associated with both events. The terrorist attacks served as precedence for the engineering community to intensify the amount of research in the field of structural engineering pertaining to progressive collapse (Starossek, 2009).

1.2 Design Guidelines and State of Research

With the impact of these tragic events, the engineering community began to implement standards for preventing progressive collapse. This was evident from multiple governing bodies in different countries around the world. The first standards to begin incorporating progressive collapse were the British Standards, specifically BS 6399. This was followed by the Eurocodes which included provisions for designing buildings against accidental and extreme loading conditions. The first American codes to incorporate provisions were the American Society of Civil Engineers (ASCE) Standard 7 as well as the US General Services Administration (GSA) Guidelines. The ASCE discussed general strategies for reducing the potential of progressive collapse without providing any specific requirements while the GSA guidelines provided design guidance relying on linear and nonlinear analysis techniques. The latest code to provide design provisions for progressive collapse is the Department of Defense's (DOD) Unified Facilities Criteria (UFC 4-023-03) guidelines for design of buildings to resist progressive collapse. UFC 4-023-03 offers guidelines for multiple design methods including: the tie force method, enhanced local resistance and alternate path method. The guidelines also offer a classification system for buildings which determines the type of design method to be used for progressive collapse (Kokot & Solomos, 2012). With the development of UFC 4-023-03 guidelines, the GSA replaced their original requirements with the DOD's procedures focusing on the alternate path analysis and design methods.

The emergence of the topic in recent years has also led to a number of studies looking into different aspects of progressive collapse. Early research into the topic was very case specific as it focused on the actions that triggered the progressive collapse such as fire, impact or blast loading. This continued into building specific studies such as the susceptibility of the US embassy building in Russia (Starossek, 2009). The studies examined different design techniques which have been proposed as a measure against progressive collapse. The majority of the publications implement the alternate path method and predict the behaviour of the structure after a column has been removed (Kwasniewski, 2009). Using this method, studies compare the effectiveness of the analysis techniques which are summarized in the later sections of this thesis. The techniques mainly focus on using either 2D or 3D finite element modelling to predict the structural response of the building.

As the research in the topic continues and design guidelines are further developed it is important to generate a collectively accepted method for progressive collapse design. Even with the current guidelines there is still a lack of globally recognized design procedures that agree on specific acceptance criteria.

1.3 Scope of Work

The thesis focuses on the performance of high-rise buildings under the conditions of an internal and external column removal. This local failure is used to investigate the ability to resist progressive collapse for these types of buildings. An introduction to current progressive collapse design methods is provided which identifies the advantages and limitations of each method. The nonlinear dynamic alternate path method is utilized as the primary analysis technique due to its accuracy and precision with complex systems. The goal of the analysis is to apply this technique and measure the performance of high-rise buildings with varying number of stories. These buildings will also feature three common structural systems used in practice: moment frames, braced frames with outriggers and truss tube systems.

SAP2000 is used as the primary structural analysis software to model the buildings of interest. The analysis is limited to 2D structural models due to computational and time constraints. The parameter of interest is the impact of total number of floors on the resistance of the building to progressive collapse. This parameter is investigated for multiple structural systems with the total building height ranging from 35m to 140m. The analysis utilizes the nonlinear dynamic procedure to solve for the development of plastic hinges within the structure. This formation of hinges serves as an indicator of the structure's capacity for progressive collapse. It is intended that the results are used as preliminary data for the study of high-rise buildings which can be used to develop more detailed and specific investigations into the performance of 3D models under progressive collapse conditions.

2 Progressive Collapse Design Methods

The paper focuses on DOD's UFC 4-023-03 guidelines for progressive collapse since the procedures are outlined in great detail and offer multiple design methods which have been implemented since the initial interest in the phenomenon. The three design methods include tie forces, enhanced local resistance and the alternate path method. These methods are described in more detail below.

2.1 Tie Forces

The tie forces approach utilizes mechanical ties between structural members to ensure structural integrity in the event of a local failure. The method simplifies an indeterminate structure to a locally determinate one where these tie forces can be approximated. Depending on the location and the function of the structural members, the structural ties are categorized into four types: longitudinal, transverse, peripheral and vertical. These types of ties are illustrated in Figure 2.1 below.

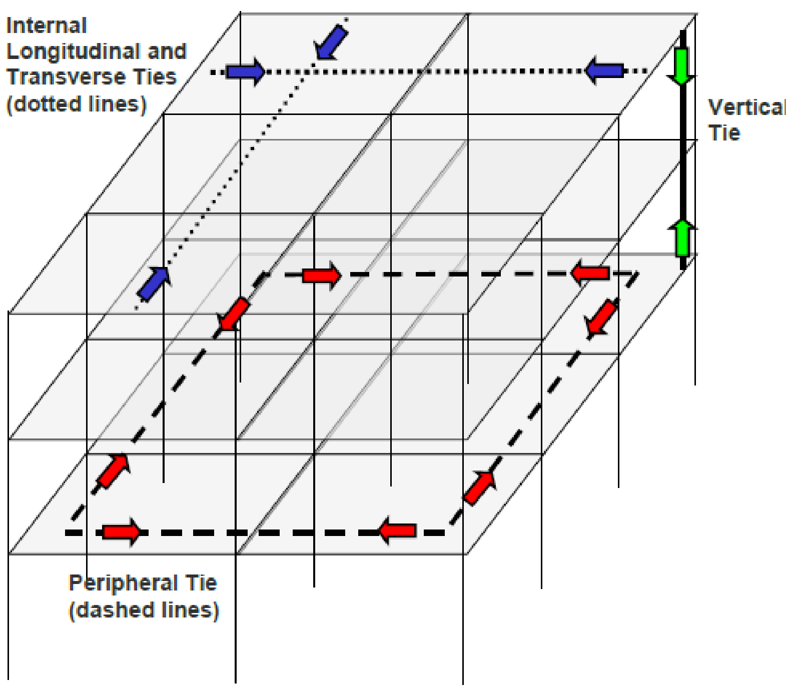

Figure 2.1 – Tie forces in a frame structure (Department of Defense, 2009)

The tie forces are developed by the existing structural elements and connections. These elements are designed using conventional methods to carry the standard loads that that are imposed on the structure. When checking the structural system it is adequate to show that the structural members (beams, girders or spandrels) and their connections can carry the tie forces while undergoing rotations of 11.3°, as specified by the UFC guidelines, otherwise these tie forces are to be carried by the floor and roof system. The only minimum structural requirement for this method is that the number of bays for the building is greater than three in each direction.

Structural members that meet the rotation requirement must also satisfy tie strength requirements which are dependent on the type of tie force in the structure. The consideration of these forces is taken separately from the typical forces that are carried by each element due to standard load combinations. The tie forces for a framed structure are defined below using the UFC 4-023-03 guidelines.

Internal tie (transverse and longitudinal):

$$F_i = 3w_F L_1 \tag{1}$$

Peripheral tie:

$$F_p = 6w_F L_1 L_p + 3W_c \tag{2}$$

Where w_F = 1.2D + 0.5L, the floor load combination given for extreme events

L_1 = Greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the direction under consideration (ft or m)

L_p = 3.3ft or 1.0m

W_c = 1.2 x Dead load of cladding over the length of L_1 (lb or kN)

The design strength for the vertical tie is to be acting in tension and equal to the largest vertical load experienced by a column in any story of the building. This loading condition is determined using the tributary area of the floor above and the w_F loading combination defined above. It is required that the vertical ties are straight in their orientation and that each column is tied continuously from the roof level down to the first story.

The tie force method utilizes these design strength and continuity requirements to redistribute any extreme loading conditions that could be caused by a local failure. The procedure outlined in UFC 4-023-03 indicates that the design of any element must be revised if the tie strength requirements are not satisfied, otherwise the designer may show that the structure is capable of bridging over this element using the alternate path method.

2.2 Enhanced Local Resistance

The enhanced local resistance (ELR) method is intended to be used in conjunction with the other progressive collapse methods such as tie forces or alternate path. It is not recommended that the procedure is used as a standalone preventative measure against progressive collapse. The principle of this design method is to ensure that a ductile failure mechanism can develop when the column is laterally loaded to failure. In order for this to occur, the column cannot fail in shear prior to the development of maximum flexural strength. The two main structural components which are emphasized by this design method is the column or vertical load bearing component and the connections between these elements and the lateral supports. An over strength factor is introduced for the areas of interest to ensure that the design strength meets the flexural and shear demands of the loading conditions.

The requirements for enhanced local resistance are established using the occupancy category of the building. This occupancy category is determined using the provisions of UFC 3-301-01. Using the occupancy classification, the guidelines summarize which columns should be targeted for ELR. For typical high-rise buildings it is recommended that the ELR method is applied to all perimeter columns of the first story above grade. Some exceptions may limit this requirement to only the perimeter corner columns of the first story. The implementation of the ELR method allows for the development of resistance for these critical elements and joints which may result in sufficient strength and continuity to provide alternate load paths (Krauthammer, 2002). This approach is recommended primarily for situations when the loss of an element cannot be tolerated by the structure.

2.3 Alternate Path Method

The alternate path method is a direct design procedure which analyses the development of forces throughout a building in the event of a local failure. In contrast to the tie force method, alternate path utilizes various analysis techniques to solve for the internal forces and deformations of an indeterminate structure. The fundamental concept of the method relies on the ductility and continuity of the building to redistribute the forces within the structure once damage is present. The method follows the general design philosophy employed by ASCE 7 for loading under extraordinary events, similar to the load combinations used for tie forces. The method is implemented using three analysis procedures: linear static, nonlinear static and nonlinear dynamic. These analysis techniques are in accordance with ASCE 41 with specific modifications to accommodate the particular issues associated with progressive collapse (Department of Defense, 2009).

Some of the requirements to implement the alternate path method for progressive collapse consist of proper designation of structural elements as well as correct removal of load bearing elements. While analysing the building of interest, all structural elements and components must be classified as primary or secondary members. The classification of these elements depends on their contribution to resist collapse. An example of a secondary element would be a steel gravity beam that is assumed to be pinned at both ends of a girder and flexural strength is ignored at the connection. Should this beam be designed as a fixed connection or partially restrained, the beam may be classified as a primary member as it provides resistance against the collapse of the structure. An important aspect of the method to consider is that the removal of any load bearing elements must maintain beam-to-beam continuity across the removal. A proper configuration of the removal process is illustrated in Figure 2.2 below. This implies that any possible removal of a column must be at a location of primary members in order to allow proper redistribution of forces.

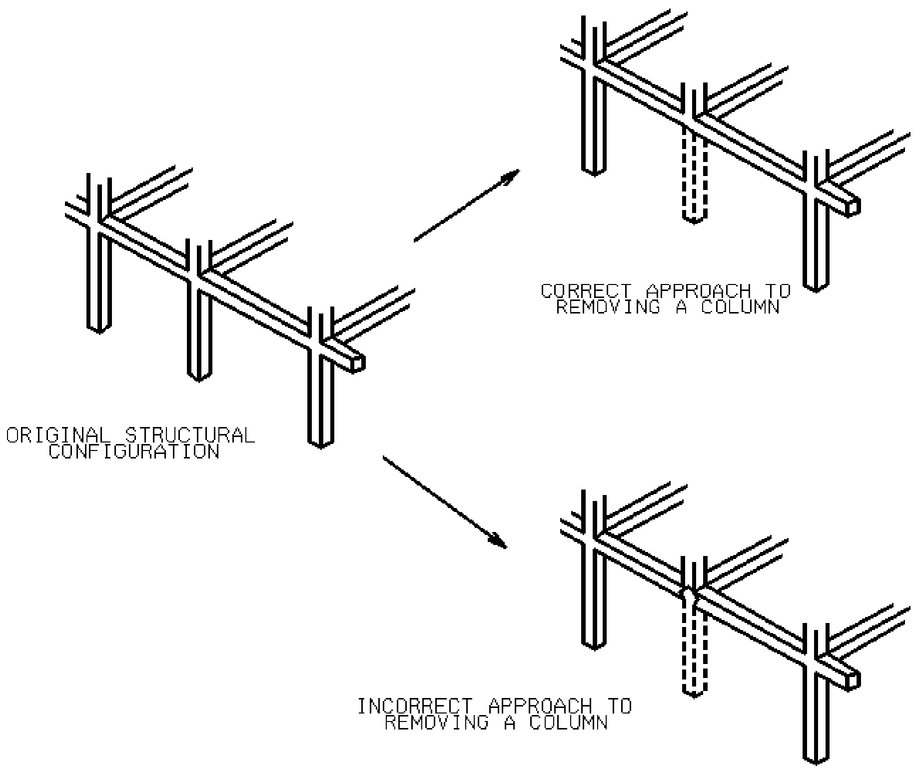

Figure 2.2 – Column removal for alternate path analysis (Department of Defense, 2009)

The configuration of these primary members within a structure serves as a good indicator of the building's ability to resist progressive collapse. The lack of primary members indicates a large number of pinned connections in the structural system. Under the removal of a column, these types of connections allow the beams to freely rotate causing large deformations and further propagation throughout the stories of the building. The rotation and deformation of the floor system is frequently accompanied by large stresses that overcome the structural capacity of the members and ultimately leads to progressive collapse. On the other hand, fixed connections allow for the dissipation of forces throughout the structure which gives the building the potential to stabilize and limit the progression of the structural failures within the building.

By using the alternate path method, this distribution of forces can be examined throughout the structure. The component capacities can either be deformation or force controlled depending on the analysis procedures; linear static, nonlinear static and nonlinear dynamic. The building is deemed structurally adequate if none of the primary and secondary elements exceed the acceptability criteria which is detailed in UFC 4-023-03. If the analysis procedure predicts that certain components or connections do not meet the acceptability criteria, the elements must be redesigned or retrofitted to meet the suitable limits for progressive collapse.

2.3.1 Linear Static Procedure

The linear static approach is regarded as the simple approach for the alternate path method. The use of the procedure is only limited to structures that meet specific irregularities and demand-capacity ratio (DCR), therefore, it is recommended for structures that have relatively simple layouts. The irregularities defined by UFC 4-023-03 are summarized below –

1) Buildings that contain significant discontinuities in the gravity-load carrying and lateral force resisting system.
2) Buildings that exhibit large variations in bay sizes.
3) Large variations in stiffness or strength for intersecting load-bearing walls in the structure.
4) Structural systems where the lateral-load resisting elements are not parallel to the major orthogonal axes of the lateral force-resisting system.

If the structure does not feature any of the irregularities identified in 1) through 4) above, then a linear static procedure may be performed without the need to calculate the DCR. Otherwise, if irregularities are present then the procedure is limited to only when the DCR ratio of the elements in the structure are less than or equal to 2.0. The DCR ratio is defined by the following equation –

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{3}$$

Where Q_{UD} = component or connection forces determined by the analysis

Q_{CE} = ultimate capacity of the component or connection

The analysis itself is conducted by modelling all primary components of the structure while the secondary members are not included since they lack the ability to redistribute the forces. The column or vertical load bearing component is then removed from the structure while the continuity is maintained at the joint, as shown in Figure 2.2. Once the removal is incorporated in the model, the following load combination is applied on the structure –

$$w_F = \Omega[1.2D + (0.5L \; or \; 0.2S)] \tag{4}$$

Where Ω = dynamic amplification factor used for linear static analysis, typically equal to 2.0 for suddenly-applied loads

The resultant stresses and forces for the components are then determined which provide the ultimate demand forces (Q_{UD}) required for the DCR computation. If the linear static procedure is deemed appropriate, these demand forces may be factored in accordance with Chapter 4 to 8 of UFC 4-023-03. The factors in these chapters account for the ductility of the material as well as governing structural conditions such as shear or flexure. The ultimate capacity of the components must meet these factored loads for the structure to be resistant against progressive collapse. Components that are determined to be sized inadequately must be redesigned to satisfy the capacity requirements.

2.3.2 Nonlinear Static Procedure

The nonlinear static procedure is similar to linear static with the main difference being the inclusion of plastic hinges within the components of the structure. The location of these hinges is up to the discretion of the designer, however, it is recommended that they are placed in areas where the highest stresses are likely to develop in the element. For a beam element this is likely to occur either at the midspan or at the ends whereas only the ends of a column would include these hinges. The plastic hinges are formed when the member transitions from elastic to plastic behaviour at a certain bending moment. Once the hinge forms it allows for rotation about the hinge with no additional bending moment. This type of behaviour is typical of an actual progressive collapse as these hinges form throughout the structure and could ultimately lead to complete collapse if the structure cannot stabilize. Typical hinge behaviour is defined by ASCE 41 with a variation in performance depending on the type of component as well as material properties and connection conditions. UFC 4-023-03 as well as the GSA guidelines utilize the defined hinge properties of ASCE 41 for their nonlinear procedures associated with the alternate path method. A generalized definition of a plastic hinge is illustrated in Figure 2.3.

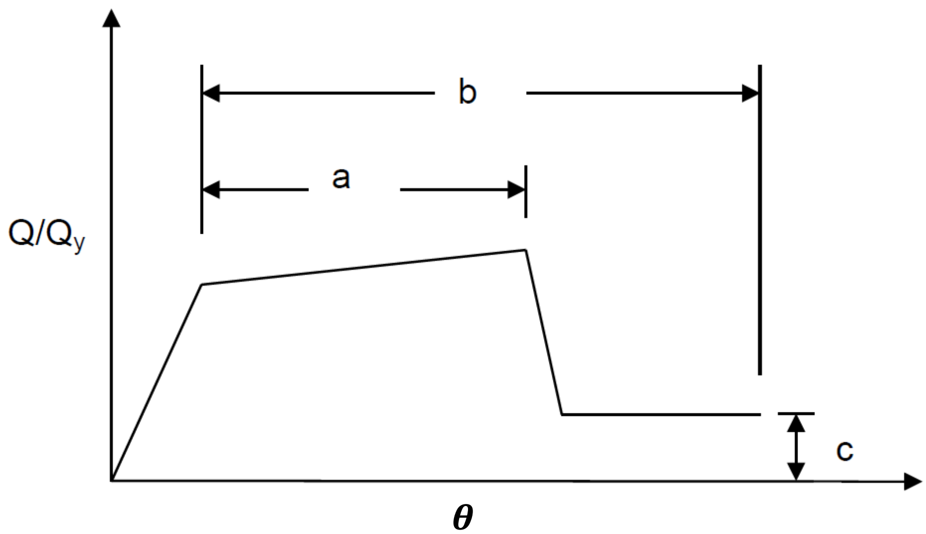

Figure 2.3 - Generalized plastic hinge definition (ASCE, 2007)

The behaviour is modeled as the ratio of moment capacity vs. the rotation of the element. The vertical axis represents the proportion of the applied load to the yield capacity of that member while θ is the rotation experienced by the member at a specific location. The a and b parameters define the formation of the plastic hinge as the elastic capacity is exceeded whereas the c parameter signifies the residual strength ratio of the hinge. These parameters are identified in ASCE 41 as well as UFC 4-023-03 for various scenarios. This behaviour of a plastic hinge is also symmetrical for the negative range of rotation, which is illustrated in Figure 4.2 for typical flexural elements.

Once these hinges are inserted into the structural model, the same load combination is applied as in the case of linear static analysis which is defined by Equation 4 above. With the hinges defined and appropriate columns removed the load must be applied using a load history that initially begins at zero and reaches the final load in at least ten load steps. The model must converge for each load increment specified.

After the analysis is conducted, the demand capacities of each member can be checked similar to the procedure outlined in Section 2.3.1. An important aspect of the nonlinear procedure is the inspection of the hinges formed throughout the structure as these are the locations of the members that developed plastic moments under the loading conditions. These hinges must be within the acceptability criteria designated as collapse prevention or life safety by ASCE 41 and the UFC guidelines. If the conditions are exceeded at any portion of the building, the elements at this location would need to be redesigned and the analysis would have to be repeated until the limits are satisfied.

2.3.3 Nonlinear Dynamic Procedure

The final analysis procedure for alternate path is the nonlinear dynamic procedure. This method is considered to be the most accurate and is recommended to be used for more complex systems. However, care must be taken to ensure that the structure is modelled accurately as small differences in the assumptions can cause large variations in the results. The main limitation of the nonlinear dynamic procedure is that the analysis is computationally intensive and could require a long amount of time for completion.

The loading procedure for this method differs from the previous two cases as the load is analysed using dynamic loading conditions. The loading combination w_F is to be applied for the nonlinear dynamic procedure, where w_F is the loading used for extraordinary events with factored loads of 1.2D and 0.5L. The analysis follows a two-step process where the model is initially brought to equilibrium under the gravity loads. This initial step is to be done prior to the removal of the column or vertical load bearing component. Once equilibrium is established, the column or wall is removed with the duration of the removal dependent on the discretion of the designer. The guidelines indicate that instantaneous column removal may be simulated by using a very small interval of time. This duration is defined as less than one tenth of the period associated with the mode shape that exhibits vertical motion of the bays located above the removed section. It is recommended by UFC 4-023-03 that the analysis is continued until maximum displacement is achieved or one cycle of vertical motion occurs at the column removal. However, the peak response may not occur during the first cycle of motion as buildings that exhibit plastic behaviour can continue to deflect. It is suggested that the analysis should continue until the model stabilizes where the displacement of the joint reaches a near constant value and exhibits very small oscillations. This may not always be possible if the building is prone to progressive collapse and geometric nonlinearities continue to develop throughout the building (Kokot & Solomos, 2012). After the model stabilizes and the analysis is complete the components and all plastic hinges can be examined and redesigned if necessary. If the model does not stabilize, it is highly likely that the building has experienced progressive collapse and the latest time steps of the analysis should be examined for hinge formations to confirm the collapse of the structure. These areas of weakness would need to be adjusted to accommodate the design requirements for progressive collapse.

3 Building Parameters

A typical building needs to be defined for the analysis of progressive collapse. Consideration must be taken into the type of structural system the building uses as well the primary material used and other parameters such as height to width ratio or the number of bays in the building. The sections below present the reasoning for the selection of these parameters and summarize the typical building that will be used for the analysis supporting this thesis.

3.1 Building Classification

Previous studies have mostly focused on relatively small structures with less than five stories. This has been the typical case examined for progressive collapse because buildings this size are fairly simple and do not require an intensive amount of computation. However, these types of buildings are not very representative of true progressive collapse events as history dictates that the majority of these events occur in mid to high-rise buildings. Examining buildings at this scale is highly important as they are more likely to be experience trigger causes such as fire, impact and accidental or intentional blast loading. This pattern is noticeable by examining the history of progressive collapse where the majority of the cases involve buildings of at least ten stories with various causes of local failure (Kokot & Solomos, 2012). Using this rationale, the paper will strictly focus on high-rise buildings for the implementation of progressive collapse analysis methods. The study for these buildings will emphasize their ability to mitigate progressive collapse.

A building is generally classified as a high-rise when the number of stories ranges from 12 to 40 or the total height of the building is at least 35m (Emporis GmbH, 2014). Using this as a guideline, the structures examined in this paper will range from 10 to 40 stories with a typical story height of 3.5m which is recommended by the Council on Tall Buildings for multi-use buildings (CTBUH, 2014). It will be assumed that the buildings are used for residential and office occupancy for the determination of the loading conditions. The building design will feature five bays with a typical width of 8m between the columns. This means that the minimum height to width ratio that will be observed is 0.875 up to a maximum value of 3.5. The variation of the total height will demonstrate how high-rise buildings perform under column removal and provide insight into the susceptibility for progressive collapse. The final parameter of interest, detailed in Section 3.2, will be the type of structural system used for the design of the building.

3.2 Structural Systems

An important factor to consider for progressive collapse is the type of structural system that is used for the building design. The selection of these structural systems vary depending on a number of factors such as architectural requirements, economics, loading conditions, material constraints as well as general preferences of the client. In order to narrow the scope of the analysis, the paper will focus on structural steel framed buildings which are relatively common for the construction of high-rise buildings. Three specific structural systems will be examined which are prominently used for high-rise design, illustrated in Figure 3.1. A major goal of the paper will be to compare the effectiveness of each system for preventing progressive collapse.

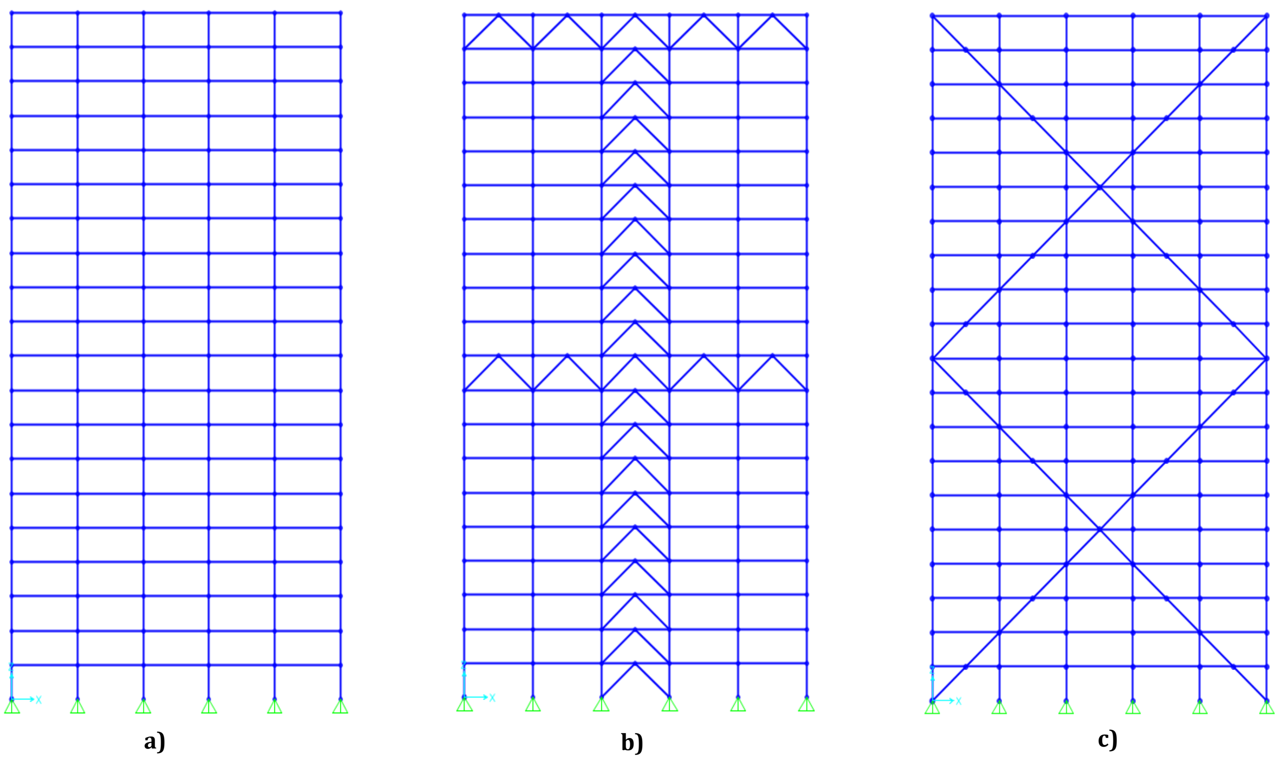

Figure 3.1 - Steel structural systems used for high-rise design; a) Moment frame b) Braced frame with outriggers c) Truss tube system

The selection of these three structural systems is based on the typical designs that have been used for high-rise buildings ranging from 10 to 40 stories (Zils & Viise, 2003). The first system to be considered is a standard moment frame which uses rigid connections throughout the building to transfer lateral loads in bending. The other two systems utilize braces to assist with the resistance to lateral loads. Since the focus of this paper is on progressive collapse these two systems will be idealized as a hybrid structural system where the braced frames still feature moment connections throughout the building. The reasoning for this is that the framed structures require continuity to deal with progressive collapse and the analysis would not be possible if the connections are assumed to be pinned. The braces in these structural systems use standard pinned connections where the moment is released at the ends of each member. These types of members would typically be classified as secondary members; however, the contribution of these components to the vertical forces of the structure provides resistance for the local failure.

It is expected that the majority of the brace contribution will occur in the first 10 stories of the braced buildings. By the time the force reaches the 10^{th} floor of the building it would have had the opportunity to transfer to a load path to the base of the building through these braces. In the braced frame system, the outriggers act as this transfer member to the core braced bay in the center of the building. For the truss tube system, it is expected that the diagonals will pick up the load from the axial reactions caused by the beams and columns that are pin connected throughout the length of the brace. This transfer of load by the braces will limit the amount of force that is redistributed in the floors above the 10^{th} story of the building.

4 Analysis Procedure

4.1 Overview

The nonlinear dynamic alternate path method was selected to conduct the parametric analysis for progressive collapse. The procedure provides the most accurate results for the behaviour of a structure experiencing progressive collapse and has the least amount of limitations in comparison to other methods.

SAP2000 was selected as the structural analysis program based on UFC recommendations and previous studies conducted on the topic. This software package is fully capable of performing all of the required steps for nonlinear dynamic analysis and also provides built in design packages that produce optimal section sizes for given loading conditions. The software is used to model 2D representations of high-rise buildings varying in the number of stories. It is important to mention that the UFC 4-023-03 guidelines recommend that the analysis procedures are conducted on 3D models, however, the research conducted for this paper is meant to provide preliminary behaviour of high-rise buildings. The structural systems examined are expected to perform very differently from one another therefore the potential error introduced by 2D modelling is not important in the context of this paper.

The purpose of the analysis is to simulate the removal of an internal and external column of a building. The building parameters used for the analysis is described in with Section 3.1. This procedure is then repeated for three structural systems: a moment frame, braced frame with outriggers and truss tube system.

4.2 Implementation of Alternate Load Path Analysis

The following procedure was formulated using the UFC guidelines as well as various standards including ASCE 41 and ASCE 7-10. The step by step process for the nonlinear dynamic analysis is detailed in the sections below.

4.2.1 Load and Mass Considerations

Three main loads were considered for sizing the structural components and for the analysis of progressive collapse. The loading conditions were determined using ASCE 7-10 for dead, live and wind loads. The summary for these loads is provided below.

1) Dead Loads – Considered weight of 6” concrete slab and finishes.

   w_{DL} = 4.55kPa   Therefore, with 8m tributary width DL = 36.4kN/m

2) Live Loads – Building was classified as Category III occupancy in accordance with UFC 4-023-03. The occupancy was assumed to be mixed use with residential/office occupancy.

   w_{LL} = 3.83kPa   Therefore, with 8m tributary width LL = 30.64kN/m

3) Wind Loads – The wind load distribution for each model was determined in accordance with ASCE 7-10 with the following factors –

   v = 51.41 m/s

   K_d = 0.85

   K_t = 1.0

   L = B = 40m

   Level II (normal) importance factor was used with a ‘B’ exposure category as defined by ASCE 7-10. The wind distributions included the combination of windward and leeward pressure acting on the building with a tributary width of 8m. The resultant distribution varied for each building model as the total height of the building changed. The load was applied to the structure in a tapered distribution to accurately represent increasing wind pressure as the building increased in height.

The nonlinear dynamic procedure utilizes the loading combination specified in Section 2.3.3 for extraordinary events. Typical loading combinations specified by ASCE 7-10 were used for the design of the structural members of each structural system.

The mass source was defined using the following load pattern –

$$m = 1.0D + 0.1L \tag{5}$$

The dead load includes self-weight of the members, slab and other imposed dead loads. The approximate 10% of live load was determined using the requirements of Chapter 12 of ASCE 7-10 for seismic weight. The inclusion of this live load is recommended for the dynamic analysis of the structure and considered in each analysis task used in SAP2000.

4.2.2 Sizing Optimal Sections

The design of the section properties for the beams, columns and braces was conducted using the integrated design features in SAP2000. The software provides optimal section sizes for specific loading conditions using a specified design code for acceptability limits. Using the software for the structural design of the frames offers a consistent method for sizing various buildings with different number of stories and structural systems. All members are designed using standard AISC structural steel sections in accordance with the AISC 360-10 design code.

The design preferences included a demand/capacity ratio limit of 0.95 to provide marginally conservative member sizes. Once static analysis was conducted, the members were sized in accordance to all the loading combinations required by ASCE 7-10. The process was then repeated with the new sections sizes until the analysis and design tasks converged to the same section properties. A typical design of the bays for two floors is shown in Figure 4.1 below.

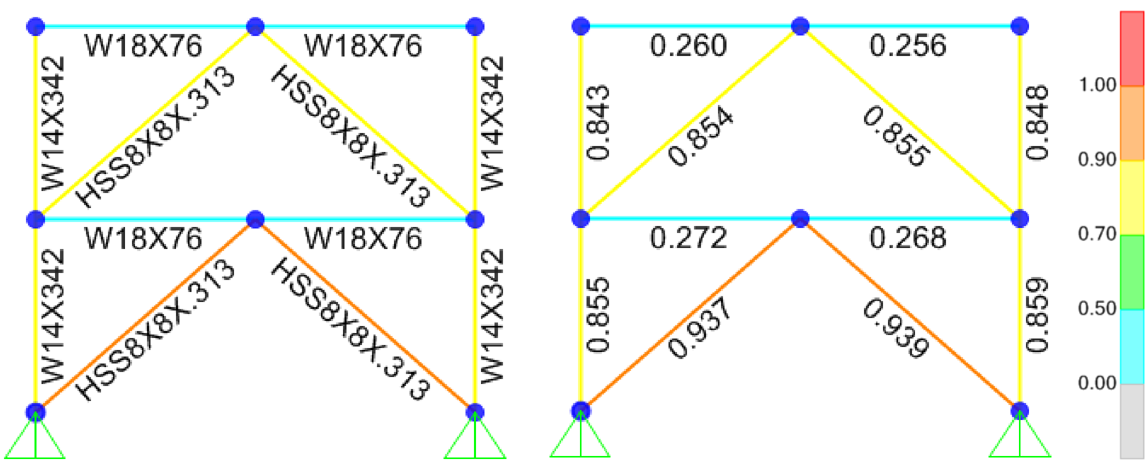

Figure 4.1 - Typical braced bays with designed sections and corresponding demand/capacity ratio

The selection criteria for the beams consisted of all AISC W shapes while the columns were restricted to all W14 shapes, a common section size used for steel W shape columns. All HSS sections were assigned for the selection of the braces. The design was generally governed by gravity loads for buildings under 20 stories while lateral load combinations began to govern for the taller models.

Models with more than 10 stories were designed in tiers with each tier incorporating 10 stories of the building. This provided a more realistic design approach as opposed to having a single beam size throughout the entire building. The result of this technique provided heavier sections towards the bottom of the building as the load is transferred from the floors above. The opposite is true for the higher tiers with the smaller sections corresponding to the lower loading conditions. The summary of the designed sections is shown in the appendix for all of the models analysed in this paper.

4.2.3 Plastic Hinge Definition

Nonlinear analysis requires plastic hinges to be defined throughout the structural model including column and brace elements. These hinges were placed at locations of high stress as recommended by UFC 4-023-03. Beam elements included plastic hinges at the midspan and ends of the members whereas hinges for the columns were added only at the ends. Structural systems which featured braces had plastic hinges included at the midspan of the member since the axial load is typically constant throughout the element.

The properties for the hinges were defined using the built-in hinge assignments for SAP2000. SAP uses the Federal Emergency Management Agency (FEMA) designations for these hinges, specifically Table 5-6 of FEMA 356 for structural steel hinge properties. ASCE 41 and the UFC guidelines recognize these as the standard properties for plastic hinges and reference this table for their own hinge definition procedures. The specific properties of the plastic hinge curve vary depending on the type of structural component. A typical flexural hinge used for beam elements is illustrated in Figure 4-2.

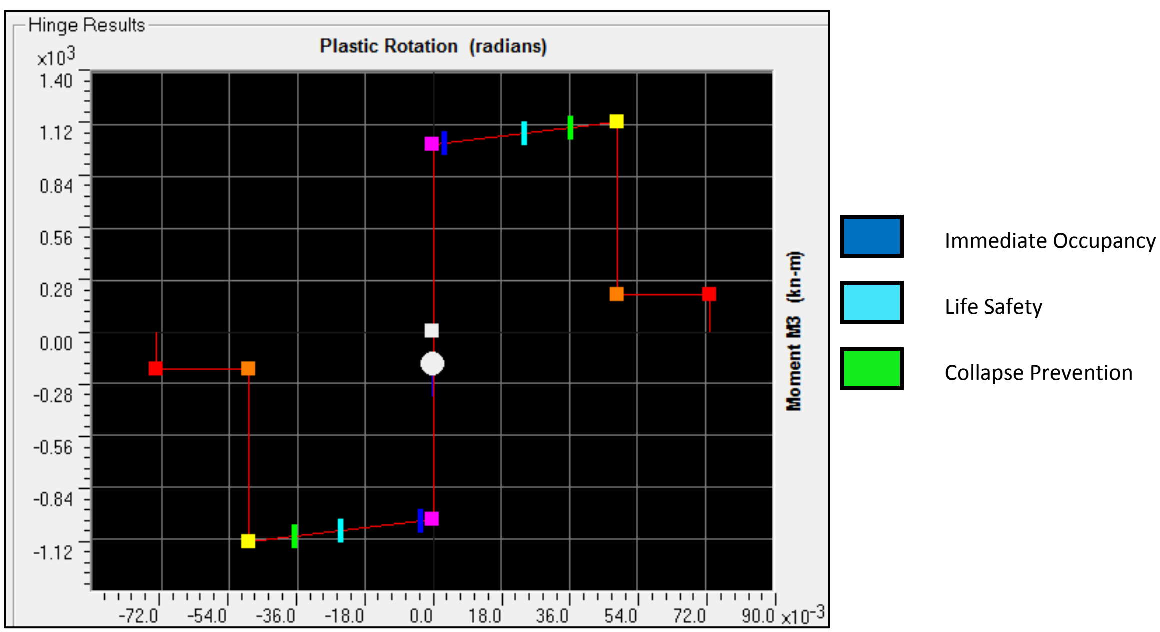

Figure 4.2 - Typical plastic hinge curve for flexural elements

Figure 4.2 shows how the elastic and plastic behaviour of a structural component is defined in SAP2000. The initial vertical portion of the curve represents the elastic properties of the element. Once the yield moment is reached, the member exhibits plastic behaviour which is demonstrated by the bend in the curve. This plastic region of the curve allows for rotation of the member about the hinge location and defines the acceptability limits for a plastic hinge. The coloured hatch marks on the curve represent the three limit states defined by ASCE and FEMA. For the purpose of progressive collapse, the hinge is determined to be inadequate if the life safety limit is exceeded. The remainder of the curve is defined by the residual strength capacity of the member, which is approximately 20% of the yield strength.

4.2.4 Analysis Parameters

Two main analysis tasks conducted are nonlinear static and dynamic. The nonlinear static case was mainly used to establish equilibrium conditions prior to the removal of the column. The dynamic analysis was performed using the Nonlinear Direct Integration Time History load case type with the default settings provided by SAP2000. The other parameters used for the analysis are summarized below –

1) Damping ratio = 2%, Typical value for inherent damping of steel structures (Paz, 2003)
2) Analysis time step = 0.01s
3) Total duration of analysis = 5s or until progressive collapse/multiple hinge failure has occurred
4) Column removal duration = $\frac{T}{15}$

The period (T) used for the column removal duration is the period of the first mode to exhibit vertical motion at the location of the removed column after the column has been removed.

4.2.5 Column Removal

The next step of the procedure focused on determining the equilibrium conditions of the structure prior to the removal of the column. This was followed by the removal of the column at the desired location. This process may be done in several different ways depending on the type of software used. The procedure detailed in this section was established with the recommendations outlined in UFC 4-023-03.

Once the structural model has been sized and the appropriate loads are applied, a linear static analysis must be conducted for the internal forces in the structure. The reaction forces and moment are then recorded for the column that is intended to be removed. These equivalent loads are used to simulate the support provided by the column. The next step is to replace the column with the equivalent forces and repeat the linear static analysis without the physical representation of the column. A comparison is made between the two cases to make sure that the equivalent forces provide an accurate replacement for the column and the structure exhibits similar structural behaviour. Figure 4.3 shows a section comparison of the two cases for a 20 story building.

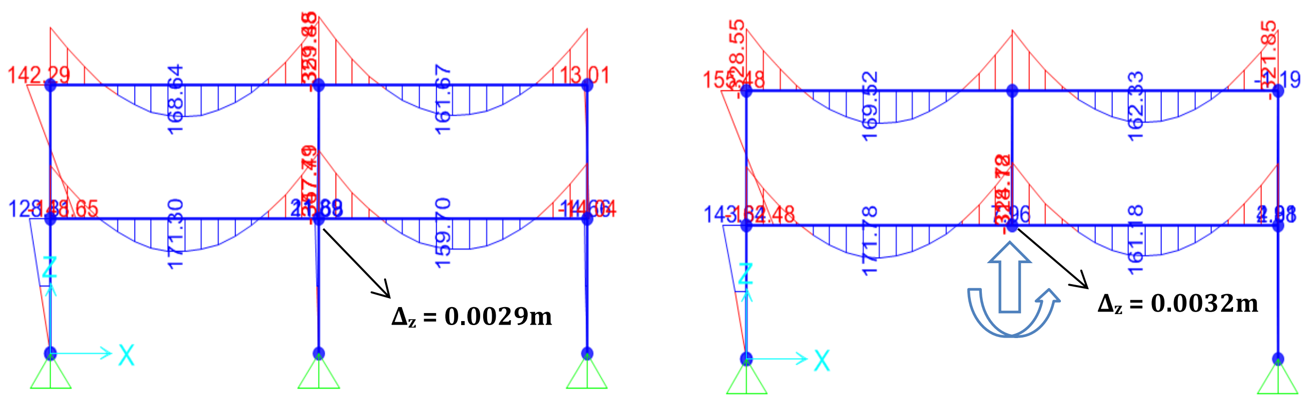

Figure 4.3 - Comparison of moment diagrams between static and equivalent static case

Following the replacement of the column a nonlinear static case is conducted to determine the initial equilibrium state of the model. This nonlinear static case includes the loading combination for extraordinary events as defined in Section 2.3.3 as well as an unfactored equivalent load that is used to replace the column. The final step is to perform a nonlinear dynamic time history analysis case with the direct integration option. The initial conditions for this analysis task are continued from the equilibrium of the nonlinear static case. The geometric nonlinearity parameters took into account P-Delta effects as well as the effect of large displacements on the structure. In order to simulate the column removal, the equivalent reaction forces are removed using a loading function for the time history analysis. This function implements a full reduction of the force over a short period of time. The full transition of the structural model is summarized in Figure 4.4.

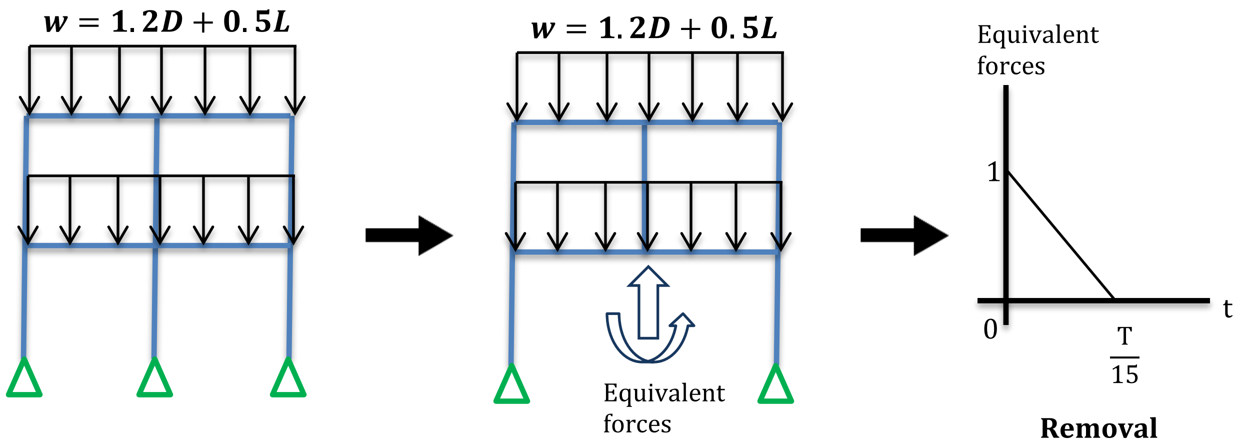

Figure 4.4 - Transition of structural model for column removal

The duration of the analysis was set to five seconds as this typically provided enough time for the structure to stabilize and reach small oscillations at the location of the column removal. However, some of the models did not display this type of performance as the hinges continued to fail and distribute the forces into other parts of the structure. For these types of occurrences, the analysis was conducted until multiple hinge failures have formed which served as an indication of progressive collapse. This process was repeated for internal and external column removal of each type of structural system at different number of stories. The number of hinges formed and their severity were used as parameters to classify a structure's susceptibility to progressive collapse. Similarly, the joint at the column removal is examined for its vertical motion over the course of the column removal.

5 Results

The sections below summarize the SAP2000 analysis results for three main structural systems: moment frames, braced frames with outriggers and truss tube systems. The nonlinear dynamic analysis task is initially evaluated for its accuracy and then conducted for a variety of building systems with heights ranging from 10 to 40 stories. A brief summary of building parameters and modal results for each structural model is provided in the Appendix.

5.1 Analysis Comparison

An initial comparison was done between the nonlinear dynamic and linear dynamic tasks to verify the accuracy of the procedure. It was expected that the nonlinear results would provide greater deflections as the nonlinearity incorporates the plastic behaviour of the structure. This is opposite to seismic loading where hinges dissipate energy and reduce the structural response. Under progressive collapse scenarios, there is no time for the dissipation of energy to occur therefore the maximum deflection is expected in the first cycle of the structure.

The analysis comparison was done using a ten story moment frame model with internal column removal. The structural model was designed using a demand/capacity ratio limit of 0.75 to provide conservative section sizes and prevent progressive collapse from occurring. This design preference resulted in a W21X93 beam and W14X233 column size.

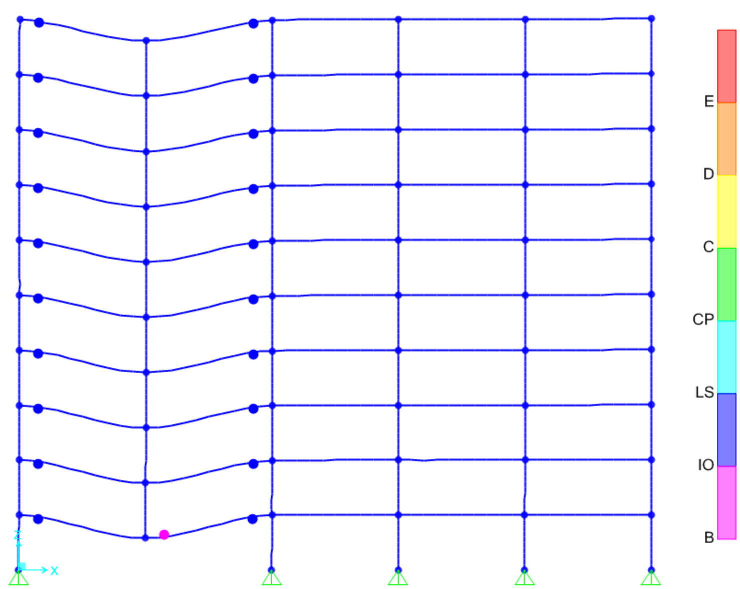

Figure 5.1 - Nonlinear dynamic results for overdesigned 10 story model

Figure 5.1 above illustrates the results from the internal column removal with the formation of plastic hinges. The severity of the hinge is described by coloured legend to the right of the model. The purple hinge formed at the joint signifies that the beam has exceeded its yield moment but has not surpassed any of the limits defined by UFC 4-023-03 for plastic hinge behaviour. These are illustrated by blue, teal and green colours with the classification of immediate occupancy, life safety and collapse prevention, respectively. The remainder of the colours are defined by the curve in Figure 5.2. The presence of a hinge that has reached the life safety limit serves as an indication that the structure is inadequate for the purpose of progressive collapse and that the members exhibiting this behaviour would need to be redesigned.

By examining the deformed shape of the ten story model, we can see that the highest level of hinge formation is in the immediate occupancy range. The column removal would require the evacuation of the building, however, the model suggests that the structure will stabilize and effectively deal with the local failure of the column. The formation of this blue hinge is evident at both ends of the beams above the column removal. As the forces are redistributed throughout the structure, the hinge formation propagates to all of the floors above this location. The load-deformation curve in Figure 5.2 shows the behaviour of a hinge located at the end of a beam above the column removal.

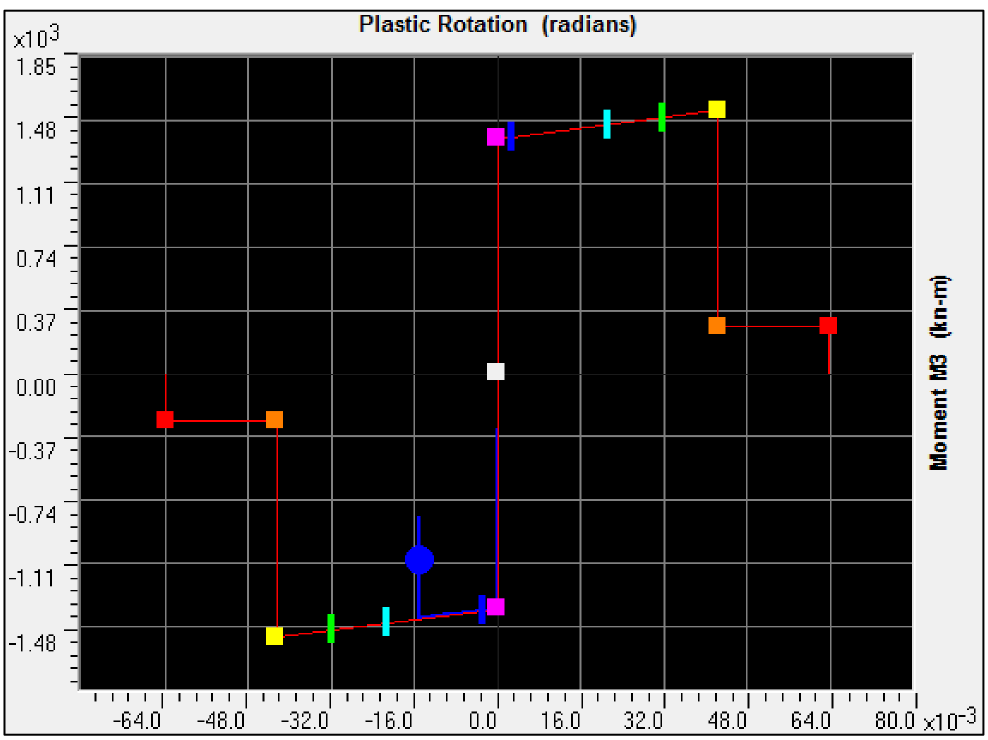

Figure 5.2 - Load-deformation curve for immediate occupancy plastic hinge

The load and rotation experienced at the hinge is detailed by the purple and blue curves in the Figure 5.2 above. The curve demonstrates the hinge reaching a maximum moment of -1430kN·m and a rotation of 0.015rad. As the analysis continued with its time step operations, the load decreased while the hinge maintained the permanent rotation caused by the plastic behaviour of the material. The model stabilized with final loading conditions of -1090kN·m at the hinge location. Even though the stabilized condition is under the yield capacity of the beam, the analysis task was able to capture the nonlinearity of the structural performance within the early steps of the time history.

The last step of the preliminary analysis task was to compare the deformation of the structure to its linear behaviour. Based on the hinge behaviour it was expected that the joint will experience greater deflections as the yield capacity of the beams were exceeded and permanent rotations formed at the ends of the members.

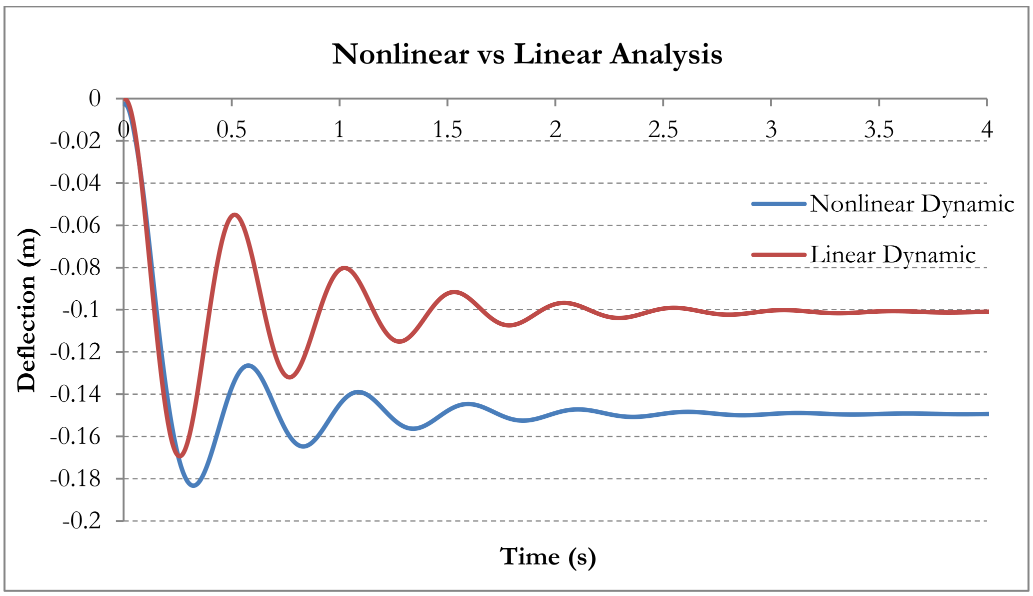

Figure 5.3 - Comparison of linear and nonlinear vertical deflection of joint at column removal

The vertical joint deflection in Figure 5.3 compares the results from the two analysis tasks. As expected the nonlinear response of the structure reaches a higher maximum deflection and stabilizes at approximately 0.15m while the linear case produces a stabilized deflection of 0.1m. The difference in deflection is within the conservative 2.0 dynamic amplification factor that is typically associated with a linear static procedure for progressive collapse (Meng-Hao & Bing-Hui, 2009). The outcome of the two analysis tasks suggests that the nonlinear dynamic procedure is being conducted appropriately with accurate results.

5.2 Moment Frames

The first structural system to be analysed using the nonlinear dynamic procedure was a moment frame model. This type of building system utilizes fixed beam to column connections to transfer the lateral loads acting on the structure. The analysis task focused on the internal and external column removal of this type of structural system with the number of stories ranging from 10 to 40. Figure 5.4 shows the results for the internal column removal.

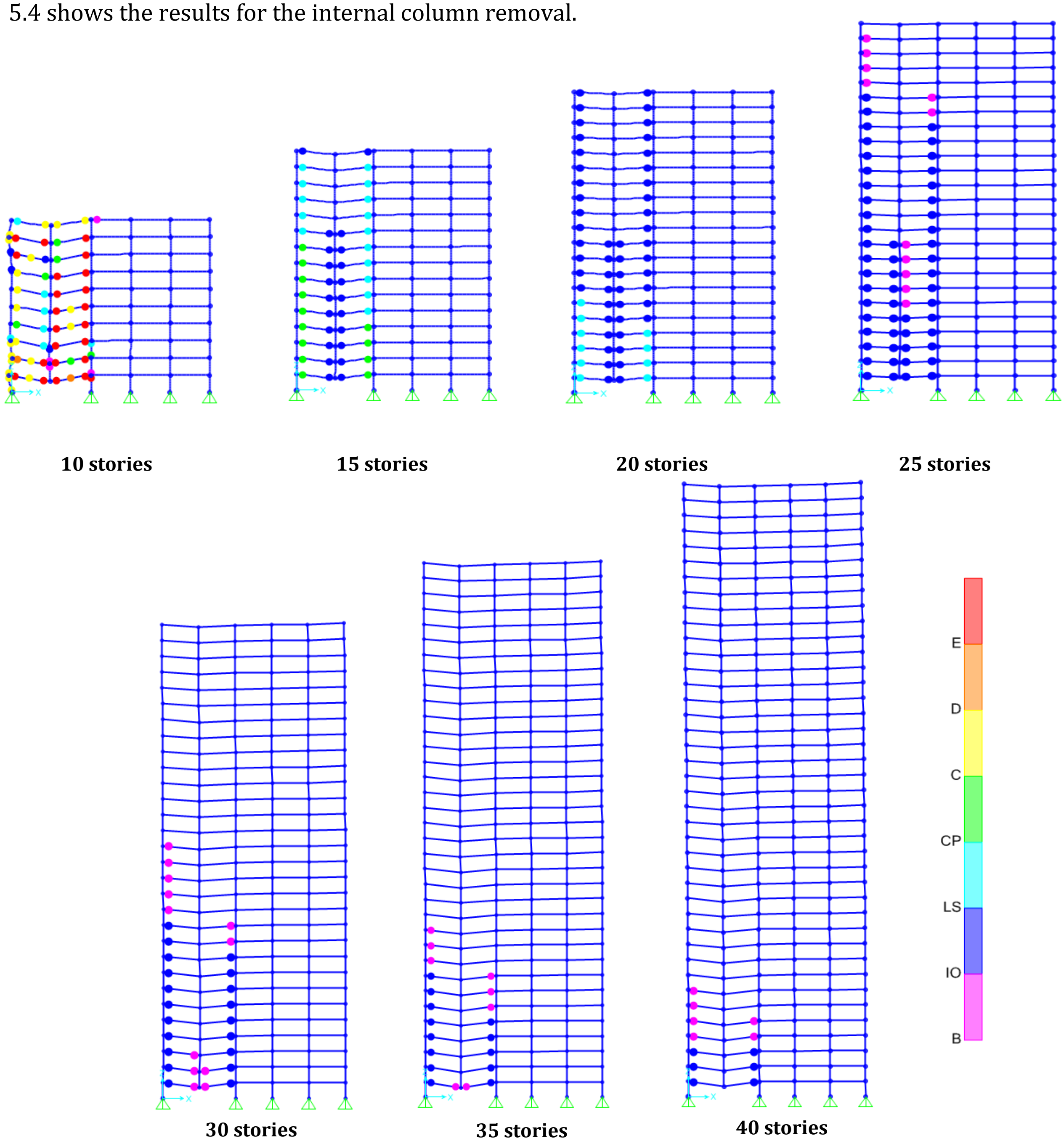

Figure 5.4 - Internal column removal for moment frames

The results for the moment frame models display a relationship between the number of stories and the amount of plastic hinges formed within the structure. The first model examined was the 10 story building which exhibited progressive collapse characteristic throughout all of the floors of the building. The majority of the hinges failed in the building and the analysis task was stopped due to lack of convergence. As each hinge failed the load was redistributed throughout the building creating a progression of the failures. As the number of stories increased, the building's ability to deal with the local failure improved. The 15 and 20 story buildings were not adequate in terms of progressive collapse as a number of the hinges exceeded the acceptability limit, however, these two models did not feature complete hinge failures as the 10 story model did. Once the buildings reached at least 25 stories, the structures proved to be relatively resistant to progressive collapse as the hinges met the acceptability criteria outlined by UFC 4-023-03. By examining the formation of the hinges, we can see that the ends of the beams are relatively critical for the design of progressive collapse as these areas are the first to develop plastic hinges. These hinges decrease in severity on the floors above due to the dissipation of forces through the fixed connections.

Looking at the joint where the column is removed we get the following deflection results.

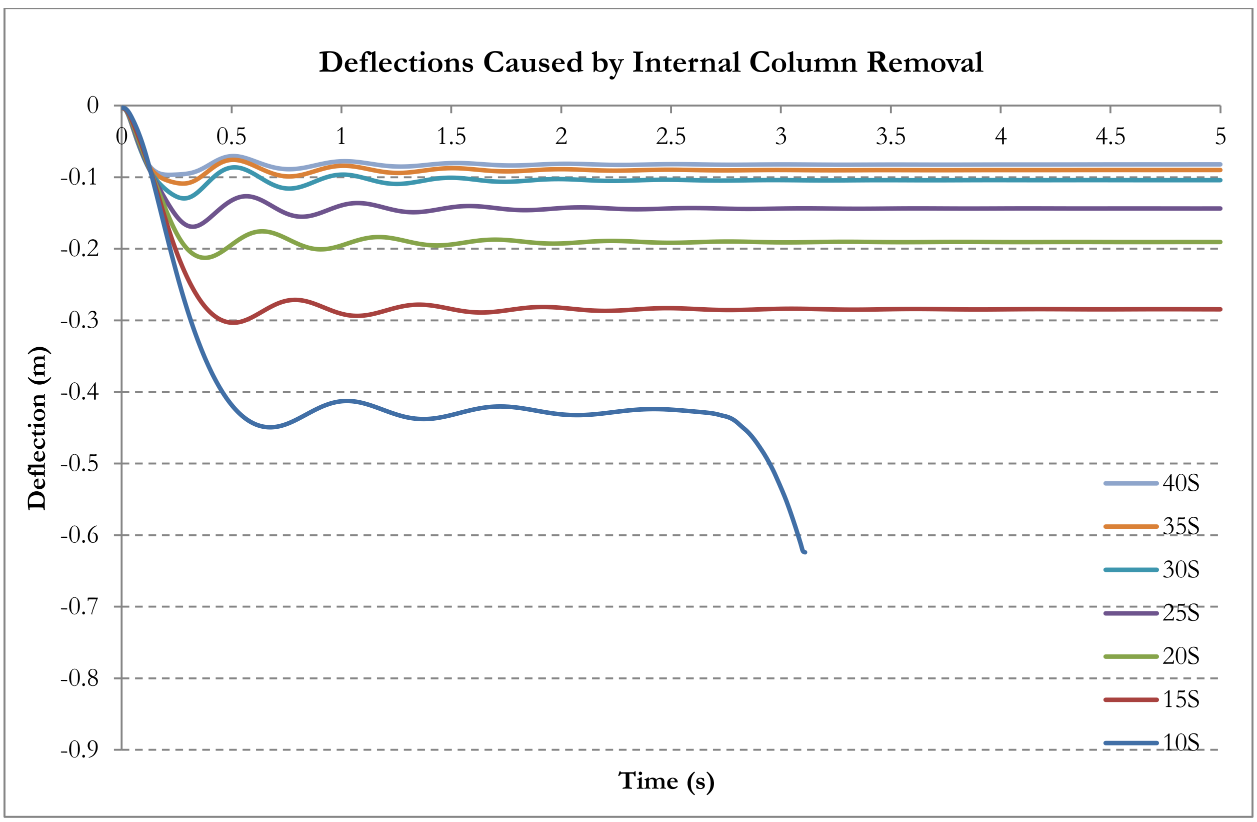

Figure 5.5 - Moment frame deflections for internal column removal

The comparison of the joint deflections, Figure 5.5 above, shows similar resistance to column removal as the number of stories is increased. The 10 story model, which failed to converge, does not stabilize its motion as the plastic hinges continue to fail throughout the building. The remainder of the models manage to reach a stable state as the response of the structures exhibit very small oscillations. The structures with more stories provide deflections that are much smaller in comparison to the short models. These taller structures undergo deflections that are very similar to the estimate provided by the linear static approach which is expected since there are a lot less hinges forming. The same process was then repeated for the external column removal of moment frames with the results shown in Figure 5.6 below.

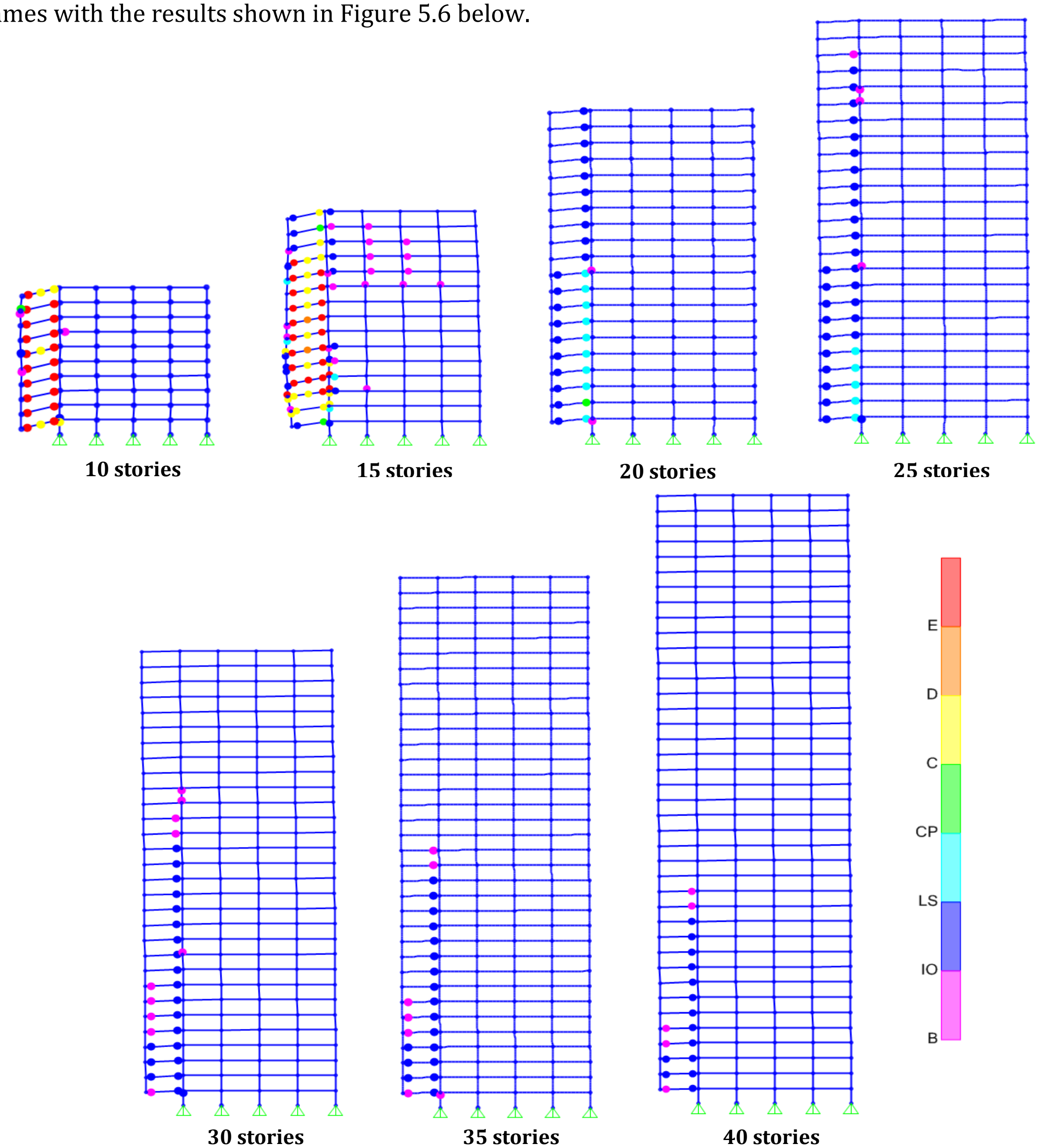

Figure 5.6 - External column removal for moment frames

The results of the external column removal are comparable to the removal of the internal column shown in Figure 5.4. The dissipation of the forces is much more effective when the building features a greater number of stories. Both of the 10 and 15 story models have a large number of hinge failures, suggesting that buildings of this size are much more susceptible to progressive collapse. In comparison to the internal removal, the severity of the hinges formed is greater as the taller models produce more hinges that exceed the immediate occupancy and life safety limits. The analysis suggests that moment frames are generally more vulnerable to progressive collapse when an external column is removed. Both of the removal conditions show that the classification of the hinges decrease to the lower limits as you the force spreads to the floors above the failure.

The vertical deflection of the joint at the external column removal is summarized in Figure 5.7 below.

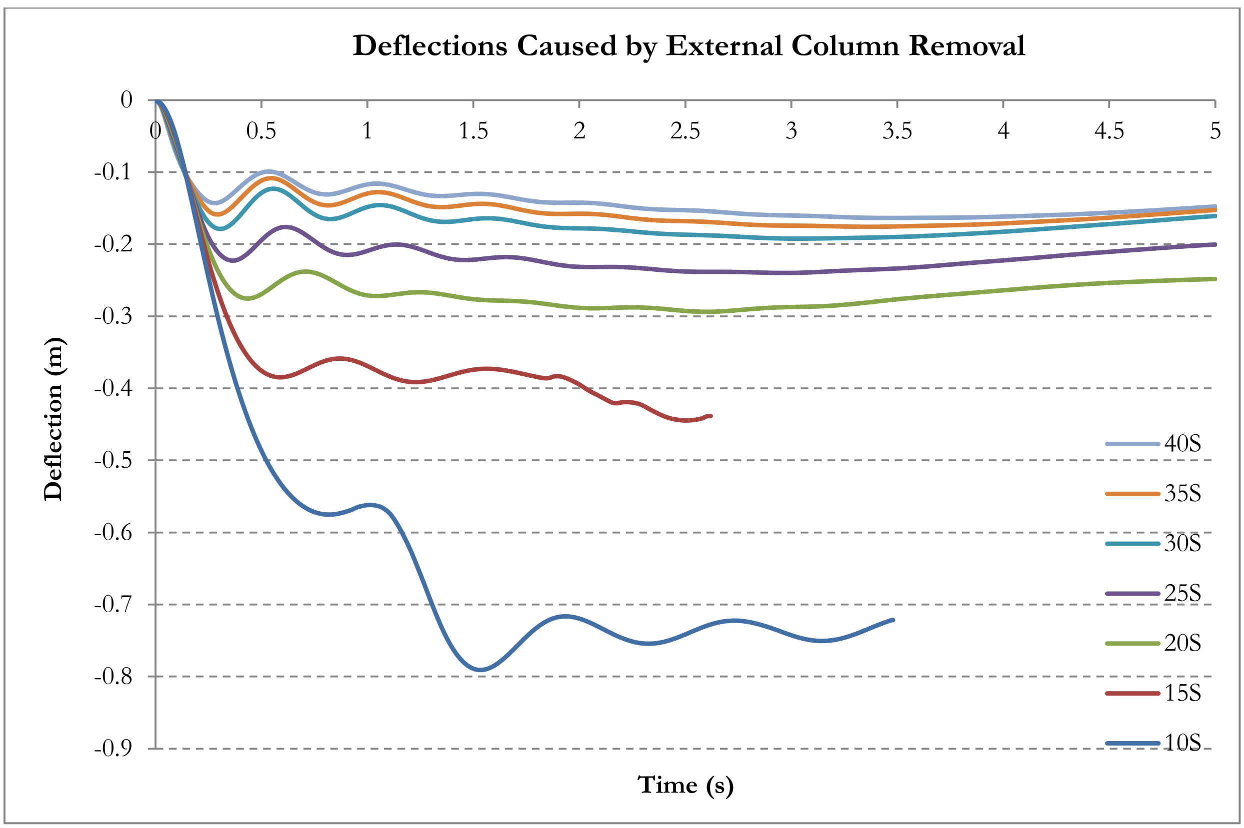

Figure 5.7 - Moment frame deflections for internal column removal

The response for the two shortest models demonstrates irregular motion as both models experience progressive collapse with the progression of hinge failures. As shown by the graph, the analysis for these models did not converge. The remainder models achieved a relatively stable deflection with the oscillations decreasing over time. Once again the structural response of the taller structures is much more manageable with an average displacement of 0.15m for buildings with 30 or more stories. There is a noticeable disparity, between the internal and external removal, for the structure's ability to stabilize the response after the first few cycles of motion. Even at higher story levels, the structure exhibits noticeable oscillations for the vertical motion. This behaviour continues for over 10 seconds until the damping of the structure finally reduces the oscillations to negligible levels.

The final comparison for the moment frames considered the percentage of hinges formed with respect to the number of stories in the structure. The percentage was determined by counting the number of plastic hinges formed and dividing by the total amount of hinges inserted in the structure for the analysis. This was done for each structural model and the results are summarized in Figure 5.8 below for the two cases of column removal.

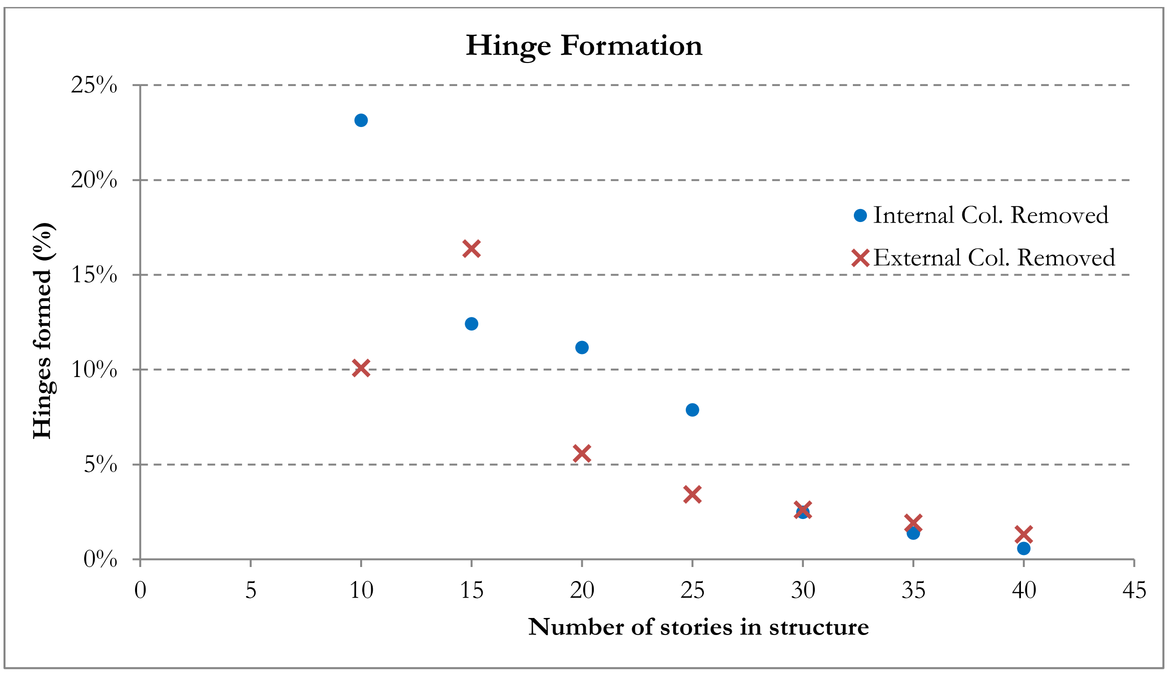

Figure 5.8 - Percentage of hinges formed for moment frame structures

The preliminary study into progressive collapse of high-rise buildings featuring moment frames suggests that there could be a correlation between a structure's height to width ratio and its resistance to progressive collapse. Based on the results shown above, there is a possibility of an exponential relationship between the two parameters. However, the results from this analysis are not enough to provide this correlation. Figure 5.8 uses the percentage of hinges formed as a measure of capacity with regards to progressive collapse but this does not take into account the severity of the hinges that have formed. The analysis for this paper also maintained a constant width and number of bays for each building. Similar research would need to be conducted on a variety of building parameters and dimensions to provide a valid data set for the correlation between the two factors.

5.3 Braced Frames with Outriggers

The second structural system which was focused on was a braced frame system with outriggers and various levels that provide lateral support for the building. The structural models used for analysis feature a hybrid structural system where the braces are pin connected in the structure while the remainder of the beams to column connection are maintained as fixed connections. This effectively made the structural system a braced moment frame for the purpose of progressive collapse analysis. Figure 5.9 illustrates the results for the internal column removal of this type of structural system.

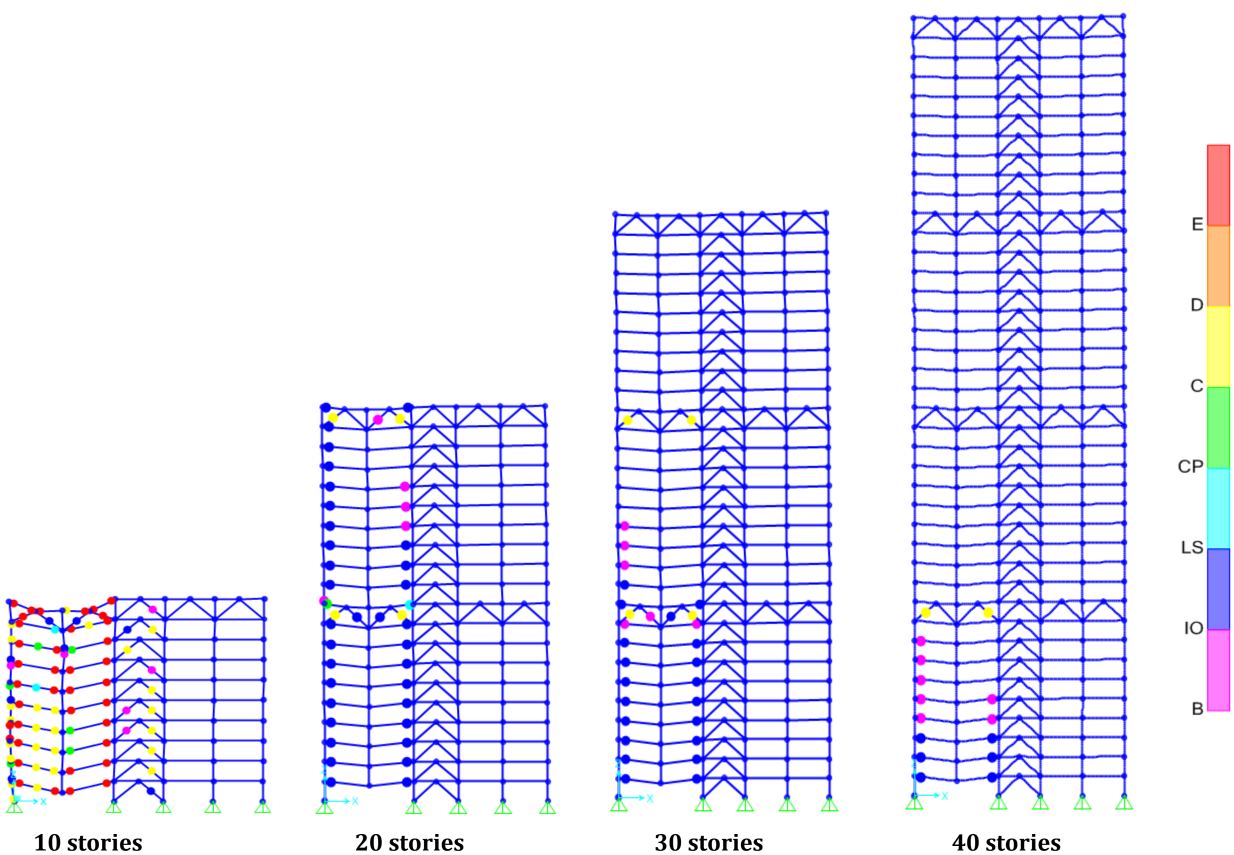

Figure 5.9 - Internal column removal for braced frames with outriggers

The structural behaviour of the braced frame models demonstrated poor performance at lower building heights while the capacity grew as the number of stories increased. The 10 story model exhibited complete failure of the hinges above the location of the removal. The remainder of the models managed to stabilize under internal column removal, however, all of them contained hinge failure in at least one brace location of the building. The reason for this is that the force that is redistributed from the local failure is much larger than what typical lateral braces are designed for.

The deflections for braced frames with outriggers are summarized in Figure 5.10 below.

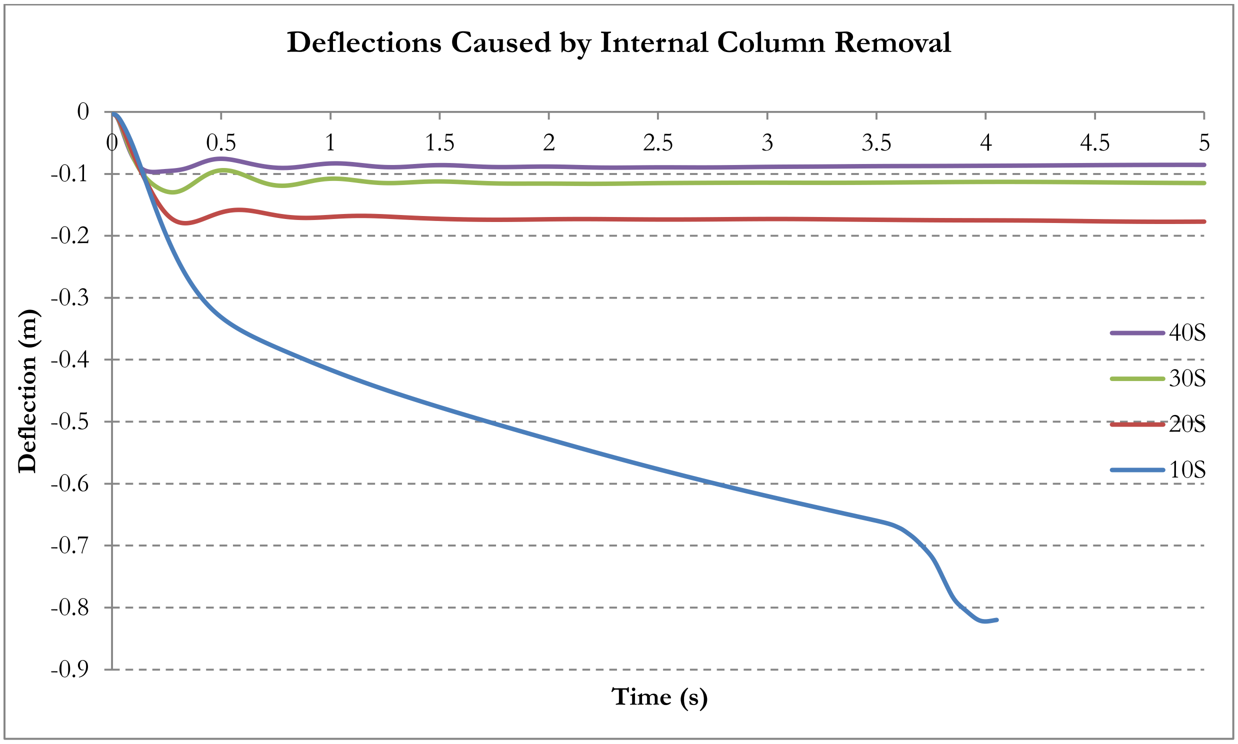

Figure 5.10 - Braced frame deflections for internal column removal

The deflections for the models are fairly stable for 20 stories and higher while the 10 story model shows signs of progressive collapse similar to the indications provided by Figure 5.9. The models that managed to stabilize demonstrate much lower deflection values in comparison to the equivalent story heights for moment frames. Based on the UFC criteria, the braced frames appear to perform slightly better in comparison to moment frames as the number of critical hinges formed in the structure is lower for the braced frame structures.

The external column removal was also conducted for the braced frame structural system with the results summarized in Figure 5.11.

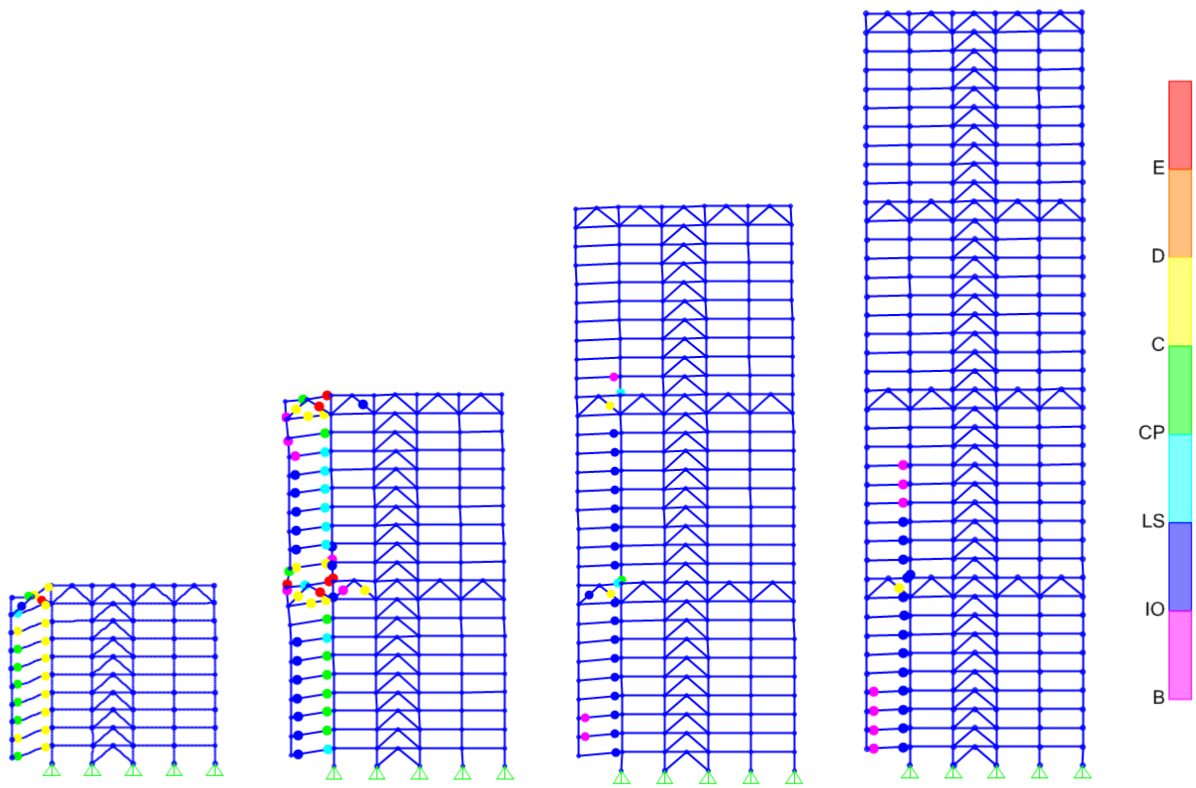

Figure 5.11 - External column removal for braced frames with outriggers

The performance for the external column removal follows a similar pattern to the internal removal. Shorter building heights provided conditions that failed to stabilize with the continuous formation of hinges throughout the structure. Both models exhibit complete hinge failures at brace, beam and column locations. The taller buildings once again demonstrated resistance to progressive collapse within the beams however the braces did develop braces that exceeded the acceptability limits for progressive collapse. The design of these systems for progressive collapse could be feasible if more consideration is given for the sizing of the braces and if continuity is maintained at the lower levels of the building since these sections of the building exhibit the largest amount of hinge formation.

Looking at the deflections of the external column removal we get similar deflections values. The graph in Figure 5.12 summarizes the structural response of the building for the removal of the external column.

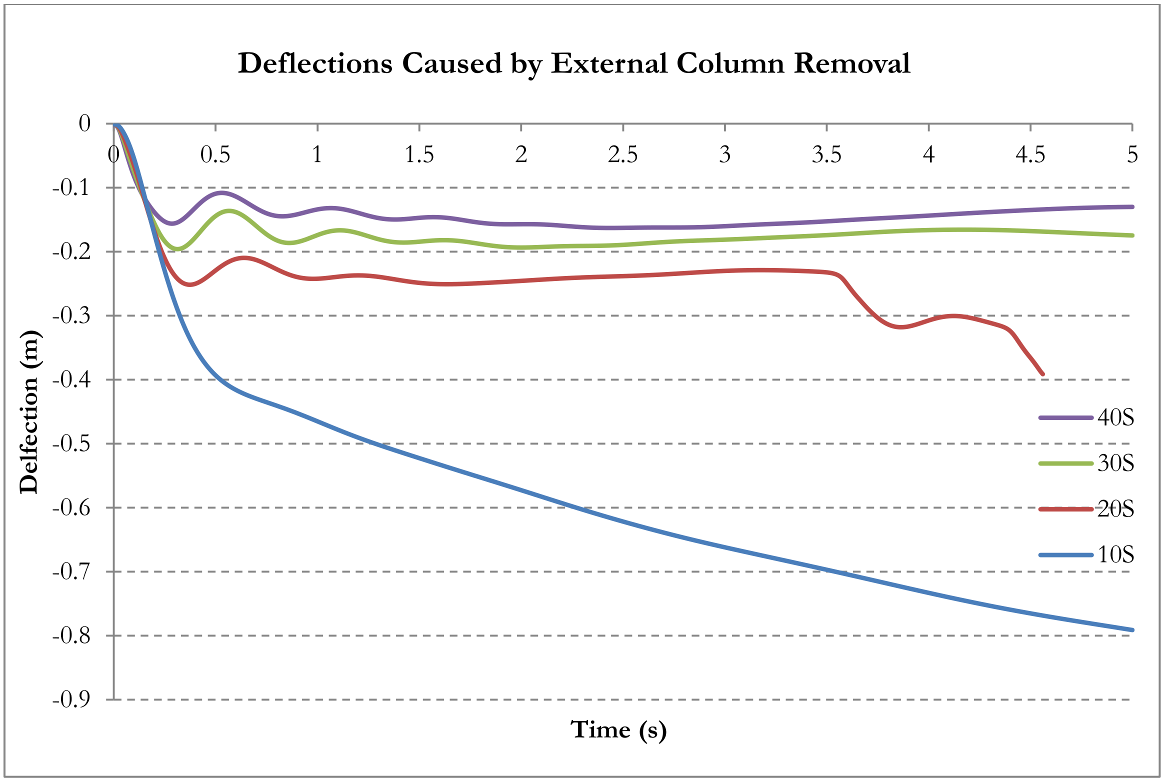

Figure 5.12 - Braced frame deflections for external column removal

The deflections of the 10 and 20 story models failed to stabilize as there is presence of hinge failure in both models. The result of the higher story levels is similar to the internal column removal with small oscillations continuing throughout the length of the analysis. Relating the deflections of the braced frame to the moment frame we see that both cases of column removal produce similar results.

An important factor to consider is that when comparing the braced frame to the moment frames, the section sizes differ at the different tiers of the building. The most important of these section sizes is the first tier, 10 floors from ground level, as this section is in the direct vicinity of the local failure. These structural components will experience the largest amount of forces before it is distributed into the rest of the structure. Comparing the 40 story model of the moment frame and brace frame, the beam sizes are W33X130 and W30X90, respectively. The inclusion of braces for lateral loads decreases the demand requirements for the beams. This means that the beam sizes for braced frame systems will generally be smaller with less capacity in comparison moment frames. This makes braced frames inherently weaker with regards to progressive collapse. Secondly, braced frames typically do not feature moment connections in the direction of the lateral bracing making the structural system lack the continuity required to resist progressive collapse. The results of this analysis signify the importance of incorporating moment connections for the first few stories of a braced frame system for the development of resistance to progressive collapse. With this type of hybrid model, with a moment frame at the lower levels and pin connected braced frame throughout the floors above, the structure can be designed to be relatively resistant to progressive collapse. This type of structural system would provide a manageable structural response to local failure that may occur at the bottom floors of the building.

5.4 Truss Tube System

The final structural system that was analysed using the nonlinear dynamic procedure was the truss tube system. This structural model once again featured a hybrid system where the beams and columns were moment connected while all the connections to the brace were pinned. The braces in this system serve as the main lateral support with each brace spanning 10 stories in height. The final analysis procedure focused on two cases of local failure; internal and external column removal. Figure 5.13 shows the results of the internal removal of the column.

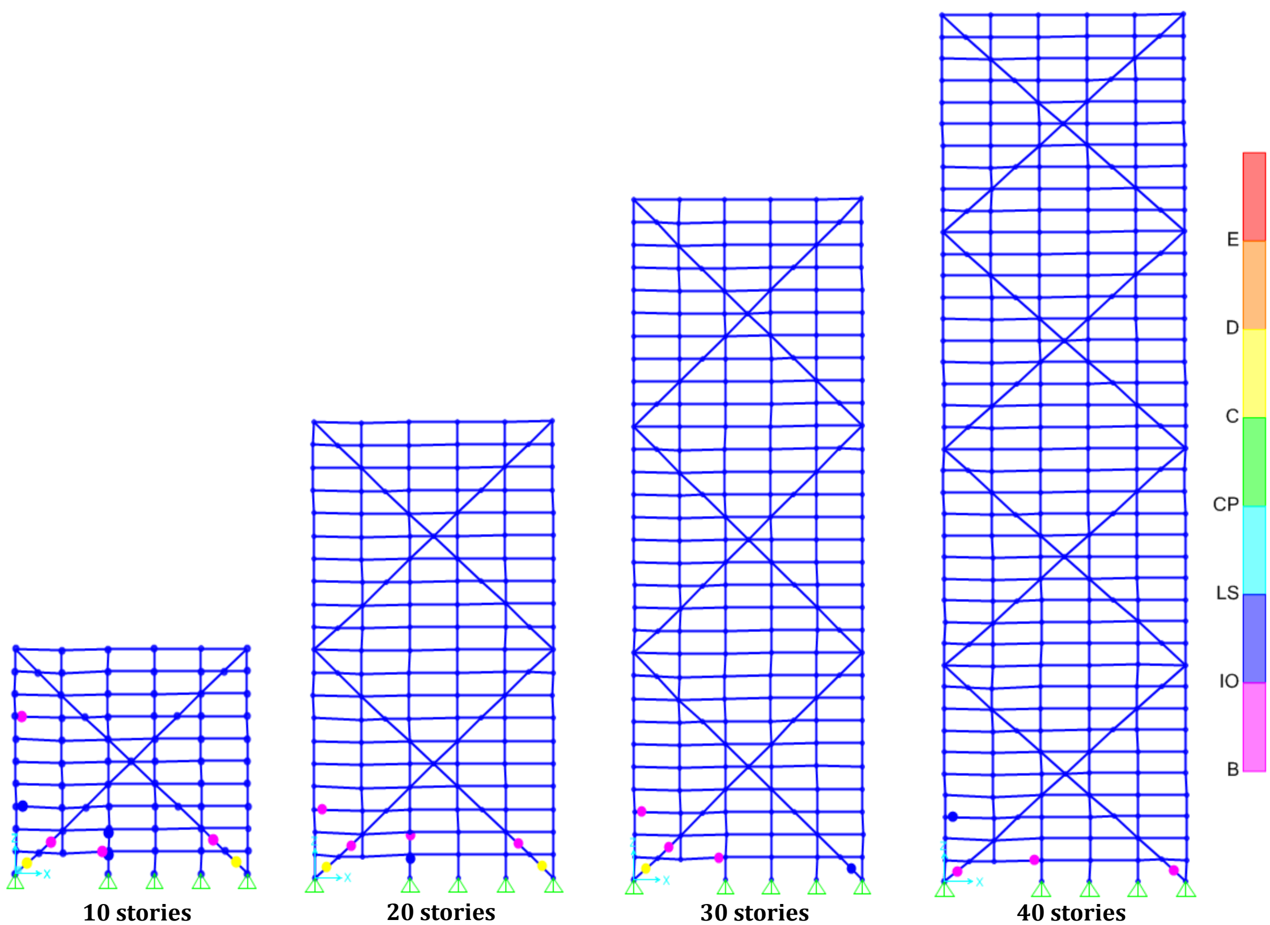

Figure 5.13 - Internal column removal for truss tube system

The truss tube structural system exhibits one of the best responses to the presence of an internal local failure. All four models develop a minimal amount of hinges with all of the flexural hinges meeting the required acceptability limits. Only the braces develop critical hinges at two primary locations of the first floor. The truss tube arrangement is the only structural system to have all four models converge and stabilize for the duration of the analysis. Even with the failure of a couple brace locations, these types of systems could easily be adjusted to account for the additional capacity requirements to deal with progressive collapse.

The deflection results, illustrated in Figure 5.14, also show promising results for the vertical motion of the joint at the column removal.

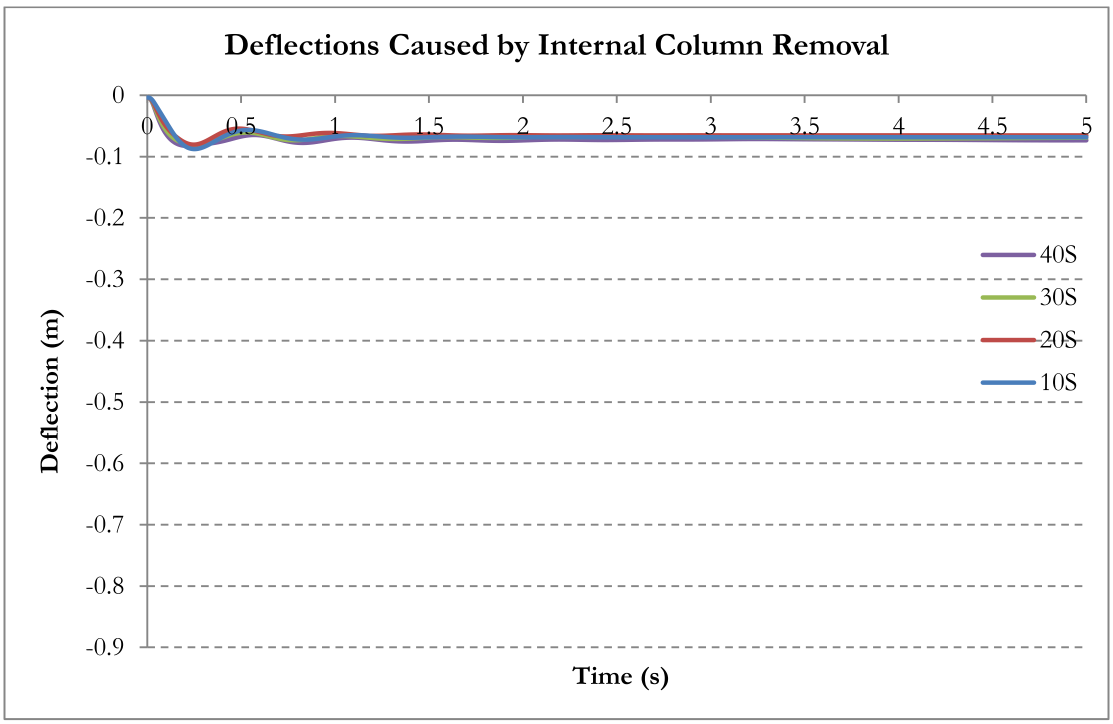

Figure 5.14 – Truss tube system deflections for internal column removal

The deformation of the joint stabilizes at a relatively low value for all four structures. In comparison to the other systems, the truss tube provides much lower deflection values with the maximum deflection reaching only about 0.09m. The results from the analysis suggest that the truss tube system with moment frames is inherently better at resisting progressive collapse. By examining the flow of forces, it is evident that the brace almost instantly picks up the distribution of the forces caused by the local failure. This load is axially transferred to the base of the building before it can be picked up by the floors above. It is expected that this type of system would perform adequately for the removal of any other internal column as there is a directed axial load path to the bracing of the building. The analysis was continued for the removal of an external column with the results demonstrated in Figure 5.15 below.

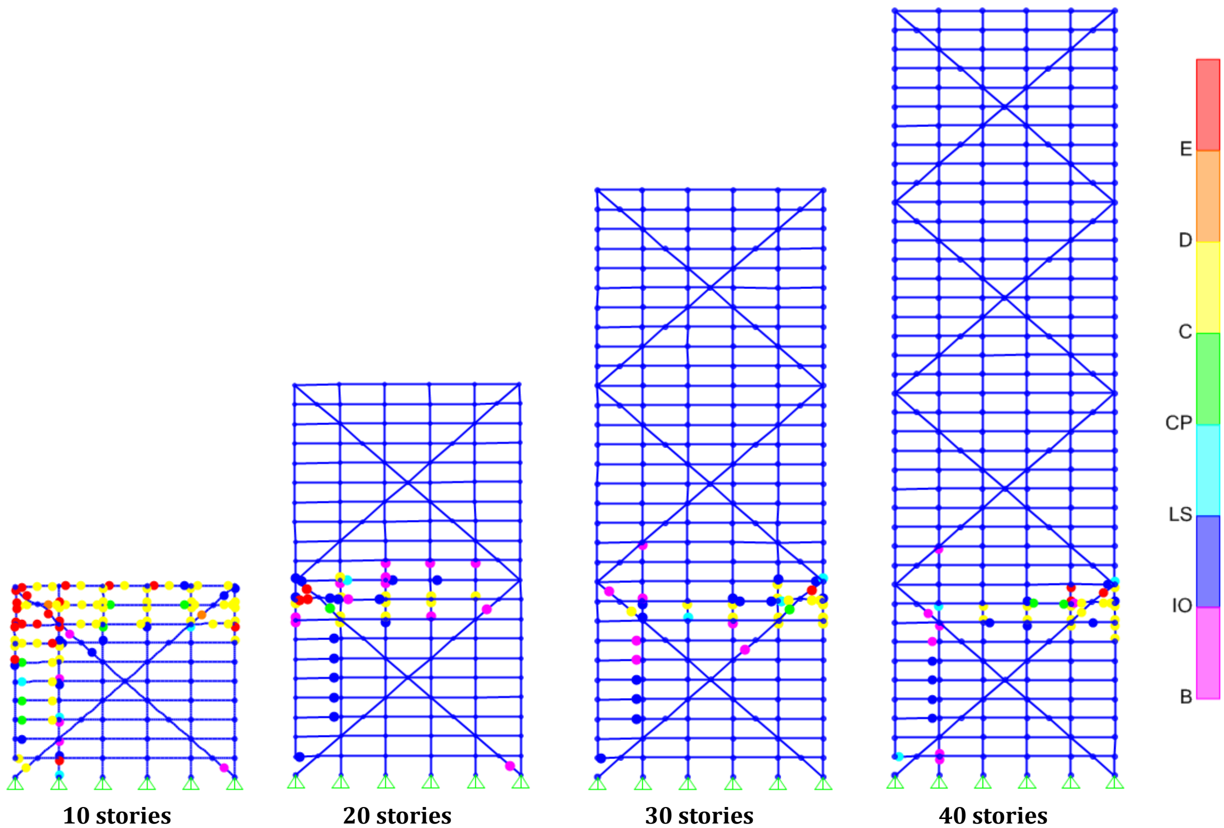

Figure 5.15 - External column removal for truss tube system

The external column removal for the truss tube system provides a much different result from the case of internal removal. All four models develop a substantial amount of hinge failures in beams, columns as well as braces. The 10 story model appears to develop hinge failures at the top of the structure as the load is transferred in tension up to the brace connection at the top of the building. The failure of the brace at this location results in the transfer of this load into axial loads on the beams and columns, ultimately resulting in the propagation of the failure. It appears that the lack of a direct load path to the braces creates this instability in the model and vulnerability for progressive collapse. The taller buildings also exhibit difficulty with adequately distributing the force without causing the progression of hinge failures.

The deflections for this scenario, shown in Figure 5.16, also show the system's inability to deal with an external column removal.

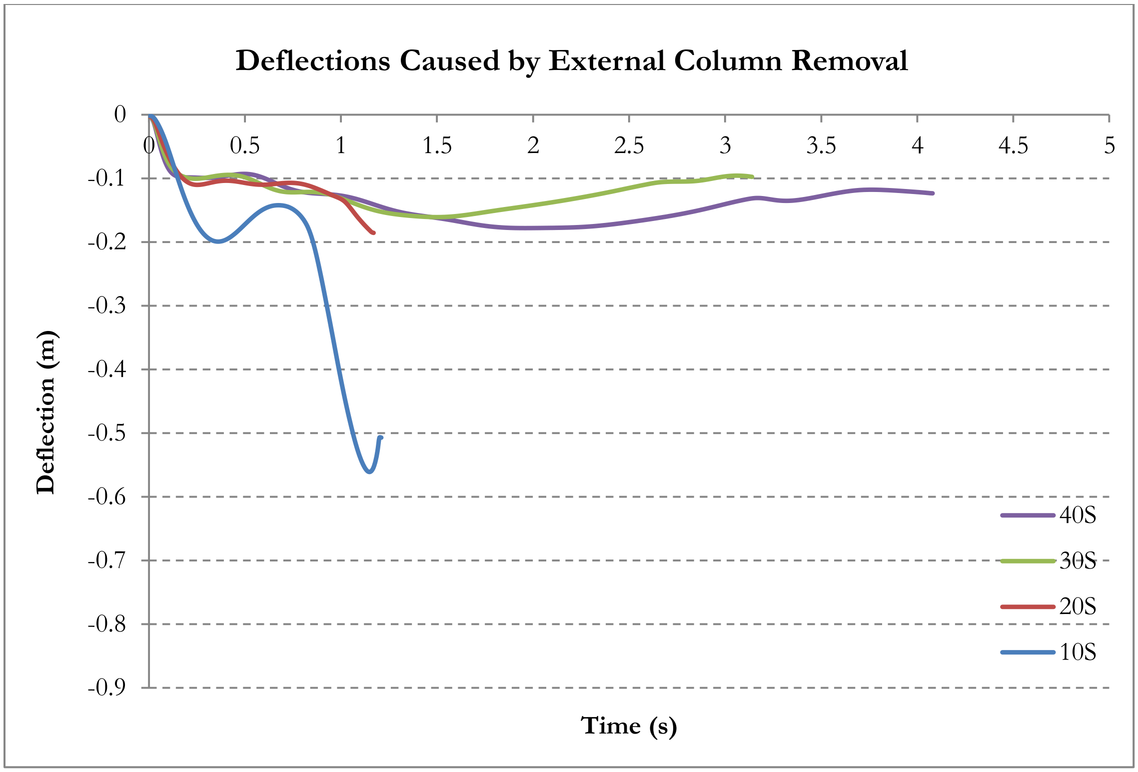

Figure 5.16 - Truss tube system frame deflections for external column removal

All four models failed to stabilize in their motion as shown in the short deflection curves of the graph above. The truss tube system proved to be less adequate at dealing with external column removal in comparison to the other structural systems. It appears that the mechanism by which the structure fails is highly dependent on the arrangement of the structure. The overall structural system for these buildings is equivalent to a truss where the external column acts as the top and bottom chords while the diagonals provide shear transfer. By removing a column at one of the chord locations, we eliminate the structural performance of this truss system resulting in a redistribution of forces that disrupts the rest of the system.

6 Conclusions

The parametric study of progressive collapse for high-rise buildings has provided multiple inferences for the performance of these buildings. Firstly, it is quite evident that the number of stories in the building has a great impact on its ability to resist progressive collapse. As the number of stories increase for a building, this creates the opportunity for the load to redistribute and diminish to a level that is within the capacity of the structural components. The taller models for all three structural systems were much more resistant to progressive collapse as fewer plastic hinges were present in these models and the hinges that did form satisfied the acceptability limits.

The relationship between the percentage of hinges formed and number of stories was briefly examined for moment frame structural systems. The results infer that there is a possibility of an exponential correlation between the two factors as the number of hinges decrease dramatically with an increase of stories in a building, however, the data set provided by this paper is not enough to confirm this finding. Additional studies should be conducted with variable number of bays as well as bay width spacing to provide more data on the relationship between the two factors.

The response of the braced frame structures provides similar results to the moment frames with the main drawback being the hinge formation in the braces. This weakness in the braces can easily be addressed and redesigned to meet the progressive collapse limits. It is important to note that both the braced frame and truss tube structural systems used in this analysis are very specific and there are many other variations of these systems used in practice. These types of systems will typically not feature moment connections throughout the structure as portrayed in this paper. This implies that both systems are inherently incapable of dealing with progressive collapse as there is a lack of continuity in the structure. The purpose of the analysis in the paper was to illustrate how these systems would function if a designer was to account for progressive collapse. Based on the results presented, the systems could be modified to a hybrid frame where the bottom floors, which are critical for the formation of plastic hinges, are moment connected and designed for local failure. The remainder of the structure may feature a typical braced frame design that is optimized for lateral loads and features pinned connections for the beams and columns. This type of efficient system could provide enough stability and continuity in the structure for the resistance of progressive collapse.

Lastly, the results of the 2D analysis show that tall buildings are fairly resilient with regards to local failure within the structure. This type of analysis needs to be compared to more sophisticated 3D models which would provide a more accurate representation of a buildings performance. It is expected that the 3D models would provide better structural behaviour in comparison to the 2D studies as the buildings may take advantage of two way action of load bearing components such as the concrete slab within the building.

References

ASCE. (2007). *Seismic Rehabilitation of Existing Buildings, ASCE/SEI 41-06.* Reston: American Society of Civil Engineers.

ASCE. (2010). *Minimum Design for Design Loads and Other Structures, ASCE 7-10.* Reston: American Society of Civil Engineers.

CTBUH. (2014). *Criteria for the Defining and Measuring of Tall Buildings.* Retrieved April 14, 2014, from Council on Tall Buildings and Urban Habitat: http://www.ctbuh.org/LinkClick.aspx?fileticket=zvoB1S4nMug%3d&tabid=446&language=en-US

Department of Defense. (2009). *Unified Facilities Criteria (UFC) Design of buildings to resist progressive collapse, UFC 4-023-03.* Washington: United States Department of Defense.

Emporis GmbH. (2014). *Building Standards*. Retrieved April 15, 2014, from Emporis: http://www.emporis.com/building/standards/high-rise-building

Kokot, S., & Solomos, G. (2012). *Progressive collapse risk analysis: literature review, construction standards and guidelines.* Luxembourg: Publications Office of the European Union.

Krauthammer, T. (2002). *Development of Progressive Collapse Analysis Procedure and Condition Assesment for Structures.* State College: Protective Technology Center, Pennsylvania State University.

Kwasniewski, L. (2009). Nonlinear dynamic simulations of progressive collapse for a multistory building. *Engineering Structures*, 1223-1235.

McKay, A., Gomez, M., & Marchand, K. (2010). *Non-Linear Dynamic Alternate Path Analysis for Progressive Collapse: Detailed Procedures Using UFC 4-023-03.* Dripping Springs: Protection Engineering Consultants.

Meng-Hao, T., & Bing-Hui, L. (2009). Dynamic Amplification Factor for Progessive Collapse of an RC Building. *The Structural Design of Tall and Special Buildings*, 539-557.

Paz, M. (2003). *Structural Dynamics, Theory and Computation.* Norwell: Kluwer Academic Publishers.

Starossek, U. (2009). *Progressive collapse of structures.* London: Thomas Telford Limited.

Zils, J., & Viise, J. (2003). An Introduction to High-Rise Design. *Structure Magazine*, 12-16.

Appendix

Table A1 - Summary of structural models used for moment frame analysis

Stories	Designed Section Sizes			Removal	Equivalent Forces		Modal Analysis		Linear Static Analysis	
	Tier	Beam	Column							
10	Tier 1	W18X76	W14X176	Int.	P (kN)=	4899	Vertical motion =	Mode 3	Max Δ_y (m) =	-0.152
					V (kN) =	1.5	Period (s) =	0.620	Joint M (kNm) =	792
					M (kNm) =	-5.3			End M (kNm) =	-1365
				Ext.	P (kN)=	2531	Vertical motion =	Mode 3	Max Δ_y (m) =	-0.204
					V (kN) =	36	Period (s) =	0.649	Joint M (kNm) =	-1386
					M (kNm) =	-125.9			End M (kNm) =	671
15	Tier 1	W24X76	W14X257	Int.	P (kN)=	7259	Vertical motion =	Mode 4	Max Δ_y (m) =	-0.126
	Tier 2	W16X67	W14X90		V (kN) =	2.94	Period (s) =	0.560	Joint M (kNm) =	1115
					M (kNm) =	-10.3			End M (kNm) =	-1664
				Ext.	P (kN)=	3925	Vertical motion =	Mode 4	Max Δ_y (m) =	0.172
					V (kN) =	38.9	Period (s) =	0.599	Joint M (kNm) =	-1716
					M (kNm) =	-136.1			End M (kNm) =	947
20	Tier 1	W27X84	W14X342	Int.	P (kN)=	9597	Vertical motion =	Mode 5	Max Δ_y (m) =	-0.111
	Tier 2	W18X76	W14X159		V (kN) =	4.2	Period (s) =	0.525	Joint M (kNm) =	1381
					M (kNm) =	-14.5			End M (kNm) =	-1924
				Ext.	P (kN)=	5484	Vertical motion =	Mode 5	Max Δ_y (m) =	-0.156
					V (kN) =	41	Period (s) =	0.543	Joint M (kNm) =	-2022
					M (kNm) =	-143.4			End M (kNm) =	1212
25	Tier 1	W30X76	W14X426	Int.	P (kN)=	11863	Vertical motion =	Mode 6	Max Δ_y (m) =	-0.101
	Tier 2	W24X76	W14X233		V (kN) =	5.4	Period (s) =	0.503	Joint M (kNm) =	1616
	Tier 3	W18X76	W14X90		M (kNm) =	-18.8			End M (kNm) =	-2149
				Ext.	P (kN)=	7127	Vertical motion =	Mode 5	Max Δ_y (m) =	-0.145
					V (kN) =	43	Period (s) =	0.573	Joint M (kNm) =	-2312
					M (kNm) =	-150.4			End M (kNm) =	1461
30	Tier 1	W30X108	W14X500	Int.	P (kN)=	14069	Vertical motion =	Mode 6	Max Δ_y (m) =	-0.092
	Tier 2	W27X84	W14X311		V (kN) =	6	Period (s) =	0.488	Joint M (kNm) =	1838
	Tier 3	W27X84	W14X176		M (kNm) =	-20.91			End M (kNm) =	-2357
				Ext.	P (kN)=	9061	Vertical motion =	Mode 6	Max Δ_y (m) =	-0.136
					V (kN) =	43.8	Period (s) =	0.519	Joint M (kNm) =	-2622
					M (kNm) =	-153.3			End M (kNm) =	1724
35	Tier 1	W33X118	W14X605	Int.	P (kN)=	16366	Vertical motion =	Mode 7	Max Δ_y (m) =	-0.084
	Tier 2	W30X90	W14X398		V (kN) =	7.3	Period (s) =	0.489	Joint M (kNm) =	2253
	Tier 3	W27X84	W14X233		M (kNm) =	-25.4			End M (kNm) =	-2751
	Tier 4	W18X76	W14X90	Ext.	P (kN)=	10893	Vertical motion =	Mode 6	Max Δ_y (m) =	-0.128
					V (kN) =	45.7	Period (s) =	0.56	Joint M (kNm) =	-3123
					M (kNm) =	-160			End M (kNm) =	2144
40	Tier 1	W33X130	W14X730	Int.	P (kN)=	18693	Vertical motion =	Mode 7	Max Δ_y (m) =	-0.080
	Tier 2	W30X108	W14X500		V (kN) =	8.05	Period (s) =	0.494	Joint M (kNm) =	2457
	Tier 3	W30X90	W14X311		M (kNm) =	-28.2			End M (kNm) =	-2967
	Tier 4	W27X84	W14X176	Ext.	P (kN)=	12951	Vertical motion =	Mode 6	Max Δ_y (m) =	-0.124
					V (kN) =	44.73	Period (s) =	0.58052	Joint M (kNm) =	-3462
					M (kNm) =	-156.56			End M (kNm) =	2462

Table A2 - Summary of structural models used for braced frame analysis

Stories	Designed Section Sizes: Tier	Designed Section Sizes: Beam	Designed Section Sizes: Column	Removal	Equivalent Forces	Modal Analysis	Linear Static Analysis
10	Tier 1	W16X67	W14X159	Int.	P (kN)= 4799	Vertical motion = Mode 3	Max Δ_y (m) = -0.109
					V (kN) = -3.3	Period (s) = 0.493	Joint M (kNm) = 239
	Brace	HSS8X8X.188			M (kNm) = -11.585		End M (kNm) = -838
				Ext.	P (kN)= 2585	Vertical motion = Mode 2	Max Δ_y (m) = -0.148
					V (kN) = 37.6	Period (s) = 0.609	Joint M (kNm) = -998
					M (kNm) = -131.4		End M (kNm) = 338
20	Tier 1	W18X76	W14X331	Int.	P (kN)= 9384	Vertical motion = Mode 4	Max Δ_y (m) = -0.102
	Tier 2	W18X76	W14X159		V (kN) = 5.8	Period (s) = 0.463	Joint M (kNm) = 414
					M (kNm) = -20.4		End M (kNm) = -1020
	Brace	HSS9X9X0.250		Ext.	P (kN)= 5665	Vertical motion = Mode 3	Max Δ_y (m) = -0.141
					V (kN) = 42.7	Period (s) = 0.546	Joint M (kNm) = -1251
					M (kNm) = -149.3		End M (kNm) = 572
30	Tier 1	W24X76	W14X500	Int.	P (kN)= 13896	Vertical motion = Mode 5	Max Δ_y (m) = -0.092
	Tier 2	W24X76	W14X311		V (kN) = 8.8	Period (s) = 0.452	Joint M (kNm) = 726
	Tier 3	W24X76	W14X176		M (kNm) = -30.7		End M (kNm) = -1327
				Ext.	P (kN)= 9246	Vertical motion = Mode 4	Max Δ_y (m) = -0.132
	Brace	HSS10X10X0.313			V (kN) = 47.5	Period (s) = 0.516	Joint M (kNm) = -1684
					M (kNm) = -166.3		End M (kNm) = 970
40	Tier 1	W30X90	W14X665	Int.	P (kN)= 18423	Vertical motion = Mode 5	Max Δ_y (m) = -0.082
	Tier 2	W30X90	W14X455		V (kN) = 10.88	Period (s) = 0.486	Joint M (kNm) = 1253
	Tier 3	W27X84	W14X342		M (kNm) = -38		End M (kNm) = -1830
	Tier 4	W21X83	W14X176	Ext.	P (kN)= 13288	Vertical motion = Mode 5	Max Δ_y (m) = -0.124
					V (kN) = -177.5	Period (s) = 0.510	Joint M (kNm) = -2427
	Brace	HSS14X10X.375			M (kNm) = 50.7		End M (kNm) = 1630

Table A3 - Summary of structural models used for truss tube analysis

Stories	Designed Section Sizes: Tier	Designed Section Sizes: Beam	Designed Section Sizes: Column	Removal	Equivalent Forces		Modal Analysis		Linear Static Analysis	
10	Tier 1	W18X76	W14x145	Int.	P (kN)=	4504	Vertical motion =	Mode 2	Max Δ_y (m) =	-0.059
					V (kN) =	11.36	Period (s) =	0.473	Joint M (kNm) =	251
	Brace	W10X49			M (kNm) =	-39.7			End M (kNm) =	-850
				Ext.	P (kN)=	2662	Vertical motion =	Mode 2	Max Δ_y (m) =	-0.115
					V (kN) =	3.38	Period (s) =	0.554	Joint M (kNm) =	780
					M (kNm) =	-11.83			End M (kNm) =	0
20	Tier 1	W18X97	W14x145	Int.	P (kN)=	8752	Vertical motion =	Mode 3	Max Δ_y (m) =	-0.060
	Tier 2	W18X97	W14x283		V (kN) =	12	Period (s) =	0.453	Joint M (kNm) =	251
					M (kNm) =	-42			End M (kNm) =	-850
	Brace	W12X65		Ext.	P (kN)=	6564	Vertical motion =	Mode 2	Max Δ_y (m) =	-0.106
					V (kN) =	2	Period (s) =	0.663	Joint M (kNm) =	1165
					M (kNm) =	-6.6			End M (kNm) =	0
30	Tier 1	W18X97	W14x426	Int.	P (kN)=	13041	Vertical motion =	Mode 3	Max Δ_y (m) =	-0.065
	Tier 2	W18X97	W14x283		V (kN) =	11.5	Period (s) =	0.506	Joint M (kNm) =	589
	Tier 3	W14X99	W14x145		M (kNm) =	-40.4			End M (kNm) =	-1198
				Ext.	P (kN)=	10767	Vertical motion =	Mode 2	Max Δ_y (m) =	-0.118
	Brace	W12X87			V (kN) =	-0.418	Period (s) =	0.857	Joint M (kNm) =	1530
					M (kNm) =	1.46			End M (kNm) =	0
40	Tier 1	W18X97	W14x550	Int.	P (kN)=	17418	Vertical motion =	Mode 3	Max Δ_y (m) =	-0.069
	Tier 2	W18X97	W14x426		V (kN) =	10.6	Period (s) =	0.575	Joint M (kNm) =	687
	Tier 3	W18X97	W14x283		M (kNm) =	-37.1			End M (kNm) =	-1295
	Tier 4	W14X99	W14x145	Ext.	P (kN)=	15092	Vertical motion =	Mode 2	Max Δ_y (m) =	-0.148
					V (kN) =	-1.7	Period (s) =	1.068	Joint M (kNm) =	1823
	Brace	W14X109			M (kNm) =	5.94			End M (kNm) =	0