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Increasing the Progressive Collapse Resistance of Steel Moment Resisting Frame Buildings

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Abstract: Studies have shown that seismic resistant buildings have higher robustness against progressive collapse. However, steel moment resisting frame (MRF) buildings designed for low seismicity may still be vulnerable to progressive collapse and therefore need to be retrofitted if such threat needs to be prevented. This paper explores the effectiveness of adding a top beams grid above the roof level on increasing the robustness of an existing building to a column loss event. This structural system would consist of a grid of steel beams added above the beams of the roof floor in order to increase the progressive collapse resistance of the buildings. Three steel MRF buildings with heights of 5, 10, and 15-story (representing low, medium, and high-rise buildings) are designed using National Building Code of Canada (NBCC 2005). The buildings are located in Toronto, Canada which is a low seismic zone and have rectangular plans of 3x6 bays with 6-meter spans. To assess the progressive collapse potential of the buildings, Alternate Path Method recommended by the U.S. General Service Administration (GSA) guidelines is adopted. Nonlinear static and nonlinear dynamic analysis of the 3D frame models are conducted for 6 ground floor column removal scenarios, and the progressive collapse resistance of the buildings is evaluated. As the results show inadequate robustness in the designed buildings, they are then retrofitted using the proposed top beams grid system, and the same analyses are conducted for the 6 column removals. The results show the effectiveness of adding the top beams as a retrofit method.

1. Introduction

Different types of loads such as dead, live, snow, wind, and seismic loads have been addressed in building codes for several decades. However, there are cases that buildings may experience extreme unexpected loads such as explosions, impacts, or car collisions. These types of loads are associated with high levels of uncertainty in both their quality and quantity, and since they are not usually considered in the design phase, they may cause failure in structural elements. The failure of the structural elements can be followed by failure of their adjacent elements and eventually partial or total collapse of the building which is called a progressive collapse. Commentary C1.4 of ASCE 7-05 defines progressive collapse 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”.

During the past century, there have been several reported cases of collapse in the buildings around the world which are believed to be progressive collapses. Ronan Point apartment tower in England, Alfred P. Murrah Federal Building in Oklahoma, and World Trade Center Towers in New York are a few famous reported cases of progressive collapse. Although these events are quite rare, their fatal and costly consequences highlight the necessity of considering them in the design of important buildings.

Due to the unknown and unpredictable nature of loads such as explosions, impacts, or car collisions, it is quite uneconomical and, in some cases, impossible to design the buildings against specific extreme load cases. Therefore, current major guidelines propose the application of a threat independent method called the Alternate Path Method (APM) in order to study the progressive collapse resistance of the buildings. APM considers an instantaneous loss of a vertical load bearing member such as a column or a wall and evaluates the building's capability of bridging across the lost member.

The research community has shown a growing interest in the field of progressive collapse of structures especially since the beginning of the third millennium. Many of the recent research works have studied the progressive collapse resistance of existing buildings which are already designed without considering this threat. Some others have explored feasibility of different retrofit methods that can upgrade the existing buildings prone to progressive collapse so that they can survive these extreme events.

Steel moment resisting frames (MRFs) are widely used in residential and office buildings in modern cities. This structural system resists both gravity and lateral loads mainly using the flexural action in its beams and columns. Several research works suggest that seismic design of MRF buildings decreases their vulnerability to progressive collapse (J-Kim and T-Kim 2009, Dinu and Dubina 2009, Mirvalad and Galal 2012). However, the level of resistance of a seismically designed MRF building depends on the level of seismicity of the zone where the building is located. Mirvalad and Galal 2012 found that many typical office buildings designed for low seismicity may be susceptible to progressive collapse and need to be retrofitted if such threat is to be prevented.

Seismic design of steel moment resisting frames is an iterative process. In this process the seismic (lateral) forces are calculated and applied on the building. Each frame attracts a percentage of the lateral forces depending on its lateral stiffness. The lateral stiffness of the frames is dependent on the stiffness of their structural members; i.e., beams and columns. Based on this distribution of the forces, the design of the beams and columns are performed and consequently the steel sections for some members need to be replaced by new sections. Changing the steel sections of the beams and columns alters the distribution of the lateral forces among the frames and their members, therefore the design process must be repeated until the section of no member needs to be changed. As a result, any retrofit plan that makes changes to the magnitude of seismic forces (by changing the mass or fundamental period of the building) or the distribution of lateral forces (by changing the stiffness of the frames and their members) will require the design of the building to be entirely revised.

The objectives of this paper are twofold; 1) to highlight the progressive collapse vulnerability of the typical multistory steel MRF office buildings designed in low seismic zones; and 2) to explore effectiveness of the top beams grid system as a retrofit method in order to upgrade the buildings' resistance against progressive collapse.

2. Guidelines for Progressive Collapse Analysis and Design

Preventing progressive collapse in existing building codes is limited to general recommendations rather than quantitative design requirements. National Building Code of Canada (NBCC) specifies general objective-based requirements for the design of major elements in order to reduce the risk for progressive collapse. Also, American Society of Civil Engineers' design code ASCE 7 provides guidance in order to ensure a minimum level of integrity in structures.

However, there are currently a few guidelines which exclusively address the phenomenon of progressive collapse. Among these guidelines, the guidelines published by General Service Administration of United States (GSA 2003) and the Department of Defense guidelines (DoD 2009) are the most updated and widely-used guidelines. These guidelines provide quantitative details for analyzing, assessing, and designing buildings against progressive collapse. Since the GSA guidelines are the only guidelines which are explicitly concerned with new federal office buildings and major modernization projects, they have been referred to in this study.

3. Method and Type of Analysis

Progressive collapse can be initiated by many different types of extreme loads which are very difficult to predict. However, the effect of these extreme loads which triggers the collapse of a building is essentially the loss of vertical load bearing elements such as columns or walls. One of the most common methods to study progressive collapse is the Alternate Path Method (APM) which is recommended by both GSA and DoD guidelines. In this method, a vertical load bearing element (column or wall) is assumed to be instantaneously lost, and the response of the building to this event is evaluated. This method is widely accepted and used by researchers and is considered as a threat-independent method since it is not concerned about the reason for which the vertical element is lost. The structure‘s role in a building is to transfer the loads from its different parts through the structural members to the supports. According to the direction of these forces, some virtual load paths can be assumed. When a column of a steel MRF building is removed, the loads which were being transferred through that column have to be transferred through alternate paths in order for the structure to sustain its stability; otherwise the column removal will be followed by a collapse. Therefore, the APM mainly evaluates the ability of the structure to find and transfer the loads of the removed element through alternate paths, and sustain stability during this process.

The GSA guidelines are only concerned about the column removals from the ground floor as these columns are usually more susceptible to destructive threats. After the column is removed, the response of the building to this event should be evaluated. Generally, four types of analysis can be performed; linear static, linear dynamic, nonlinear static, or nonlinear dynamic analysis. In a column removal, the deflections and deformations of the structural members are expected to be relatively large compared to the allowable deflections based on which the building is designed. These large deformations necessitate the consideration of nonlinearity in both materials and geometry in order to predict a realistic response for the building. On the other hand, since instantaneous column removal is a dynamic phenomenon, the most reasonable type of analysis is the dynamic analysis. However, if static analysis is to be used in such cases, the dynamic effect of the loads should be taken into account by considering some increase factors for the static loads often called Dynamic Increase Factors (DIF).

Among the four aforementioned types of analysis, nonlinear dynamic analysis is the most accurate yet rigorous and time consuming type which is able to result in the most realistic response of the building to a column removal event. This response can be a stable response or a partial or total collapse. If the response of the building to the column removal is a partial or total collapse, the nonlinear dynamic time history analysis is able to simulate the collapse and acquire valuable information about it, however it does not provide measureable details that can be used to upgrade the building’s resistance to these types of events.

Nonlinear incremental static analysis for column removal scenarios is often called pushdown analysis. In this type of analysis, a vertical load is applied to the building in the vicinity of the removed column in an incremental manner. The vertical load is increased until the failure is monitored. Nonlinear static analysis provides a deep insight to the building’s response to a column removal as well as its vertical capacity in the vicinity of the removed column. According to the type of analysis, GSA guidelines require the application of two different vertical load combinations when using APM:

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

(DL = dead load, LL = live load)

The factor of 2 in the static analysis load combination is applied in order to take into account the dynamic nature of instantaneous column loss. However, some researchers believe that this factor can lead to over-conservative results in progressive collapse studies (Ruth et al. 2006, Khandelwal and El-Tawil 2007)

4. Details of the Studied Buildings

All of the buildings of this study are designed to be located in a low seismic zone which can be represented by the city of Toronto, Canada. The seismic parameters of this zone required for performing the seismic design are presented in Table 1.

Table 1: Seismic Parameters of Toronto, Canada (NBCC 2005)

$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	PGA
0.260	0.130	0.055	0.015	0.17

Three buildings with 5, 10, and 15 stories representing low, medium, and high-rise buildings were designed in this zone based on the National Building Code of Canada (NBCC 2005) and Handbook of Steel Construction (which is based on CAN/CSA S16). The layout and loads were chosen such that the buildings can represent typical office buildings. The plan of the buildings is a 3×6 bay plan with 6-meter spans, and a typical story height of 3.65 meters was chosen for all of the buildings. Figure 1 shows the plan and elevation of the buildings.

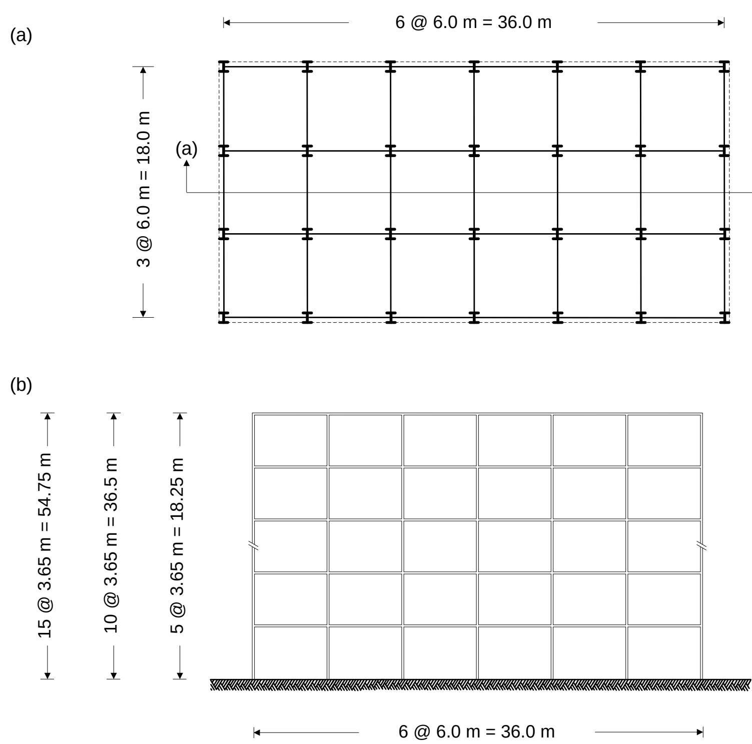

Figure 1: (a) Plan of the buildings and (b) Cross section elevation (a-a) of the three studied buildings

In order to have a uniform design, all the frames are designed to be MRFs that can resist both gravity and lateral loads. The frames carry a concrete slab of 180 mm thickness in each floor, and the beams are considered to be laterally braced in the design due to the existence of the concrete slab. 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. Snow load is also applied on the roof. The slabs are considered to be two-way slabs which transfer the loads to the frames based on the tributary area method. The steel sections used for beams and columns of the buildings are wide-flanged (designated W) I-Shaped steel sections. Modulus of elasticity and shear modulus for steel were taken as 200 GPa and 80 GPa, respectively. The yielding stress of steel was assumed to be 350 MPa, and the ultimate stress was taken as 490 MPa.

5. Modeling of the Designed Buildings

Applied Element Method (AEM) has recently attracted the attention of many researchers in the field of progressive collapse studies. Since the phenomenon of progressive collapse deals with extremely large deflections and deformations, element separations, and collisions between the elements, AEM is considered to be an efficient method in this case.

All the 3D modeling and analysis procedure of this study is performed using a software package called “Extreme Loading for Structures” or ELS (2010) produced by “Applied Science International”. Although the use of 2D models is allowed in GSA guidelines, 3D models are highly recommended by GSA and also some researchers (Li and El-Tawil 2011) for progressive collapse studies. In 2D models the effect of orthogonal structural members and out of plane displacements are not considered which decreases the accuracy.

ELS uses the AEM for modeling and analyzing. In this method, different types of structural members such as beams, columns, slabs, and walls can be defined and then divided into many smaller rigid elements; each of them has 6 degrees of freedom. Each element is connected to the other elements using several contact points, and each contact point consists of one normal spring and two shear springs in perpendicular directions. According to the desired accuracy, the number of springs and contact points can be chosen. The deformations of the springs can be geometrically related to the elements’ degrees of freedom in order to form elements’ and members’ stiffness matrixes. Nonlinear behavior of the material can be modeled by defining nonlinear force-deformation relationship for the springs. When a spring reaches its defined ultimate deformation, it will be removed from the stiffness matrixes. In case of a collision, however, new contact points and springs will be generated between the elements in order to transfer the forces between the elements so that the effect of collision or impact is considered.

The AEM 3D models of all designed buildings were initially made using beams and columns. Each of these members was then converted to a mesh of smaller AEM elements. The number of elements for each beam was determined after conducting several analysis tests in order to reach the desired accuracy with the smallest possible number of elements. Since the concrete slabs are not specifically designed to resist column removal scenarios, their structural role in these cases has been neglected in this study. In column removal events of this study, the affected beams are the first members that experience failure. Since analyzing the buildings using ELS results in a realistic visual simulation of collapse, in this study, failure is considered to be the stage in which one of the affected beams gets separated from the face of its connected column.

6. Progressive Collapse Analysis of the Designed Buildings

As mentioned in section 3, GSA is only concerned about column removals that can happen on the ground floor. In order to assess the buildings, six different column removal scenarios are considered in this study which can be seen in Figure 2.

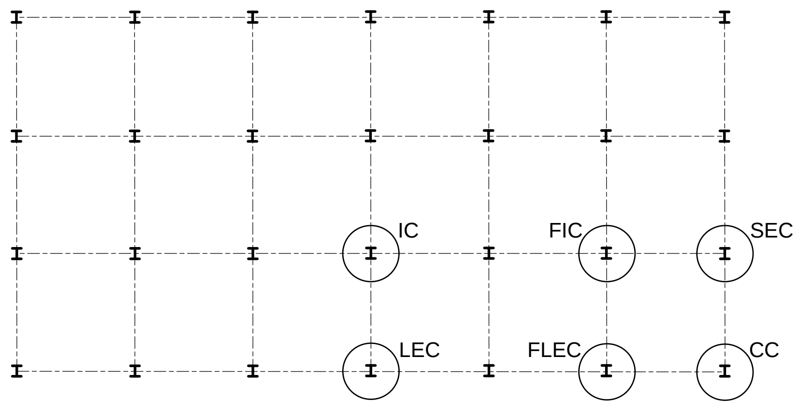

Figure 2: Location of the removed columns on buildings' plan (IC: Internal Column, FIC: First Internal Column, SEC: Short Edge Column, LEC: Long Edge Column, FLEC: First Long Edge Column, CC: Corner Column)

Nonlinear dynamic analysis is the most accurate type of analysis in order to predict the response of buildings to extreme loading effects. This type of analysis was performed for the six column removal scenarios in all the buildings of this study. The analysis procedure had two stages. In the first stage, the GSA's dynamic load combination (DL+0.25LL) was statically applied on the building without removing the column. In the second stage which was the dynamic stage, the column was removed over a very short period of 0.001 second (recommended by ELS theoretical manual) to simulate the instantaneous column loss, and after that, nonlinear dynamic analysis was conducted.

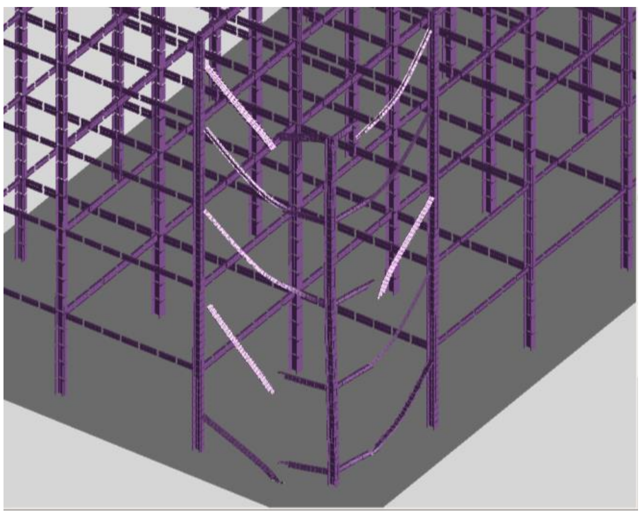

Figure 3: An example of partial collapse after a corner column removal (ELS software)

GSA guidelines require the time period for removing the vertical element to be less than 1/10 of the period associated with the structural response mode for the vertical element removal (this response period for all the buildings of this study is greater than 0.06 seconds). The results of nonlinear dynamic analysis showed that all the six column removal scenarios in the three buildings were followed by failure; i.e. partial or total collapse. In other words, all of the original buildings of this study lack resistance against progressive collapse. Figure 3 shows an example of a partial collapse in a 5-story building when the corner column is removed.

7. Retrofitting the Designed Buildings Using the Top Beams Grid System

According to the results of nonlinear dynamic analyses of the designed buildings retrofitting is inevitable for these buildings if progressive collapse needs to be prevented. There have been some recent efforts to propose methods for mitigating the probability of progressive collapse in steel MRF buildings. Some of these research works suggest the application of compound structures such as outrigger trusses, belt trusses, or mega trusses which are mostly applicable in very high-rise buildings (Vincenzo Melchiorre 2008, Kim et al. 2010). Some others try to upgrade the buildings by increasing the stiffness and/or strength of existing structural members such as beams and column (Kim and Park 2008, Galal and El-Sawy 2010). However, many of these methods are not effective for an already seismically designed steel MRF building since they can significantly change the lateral force distribution which necessitates major changes in the original seismic design of the building. These changes in the design can be very costly and in some case impossible. In order to retrofit an existing seismically designed steel MRF building against progressive collapse, the retrofit method should be able to increase the vertical load resistance of the building in the vicinity of the removed column while minimizing the changes to the mass and lateral stiffness of the building such that seismic design does not need to be revised. This study proposes the installation of a grid of beams above the roof level as the retrofit method in order to increase the vertical load resistance of the building in the affected area when a column removal happens. 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. Figure 4 shows this retrofit system on a model of a 5-story building in the software package ETABS.

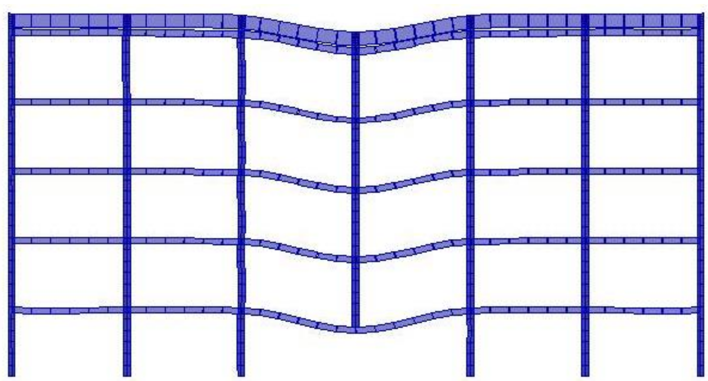

Figure 4: A model of a 5-story building featuring the top beams grid system (ETABS software)

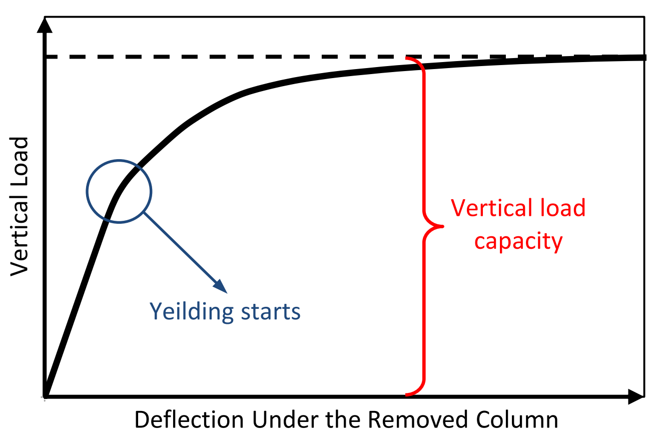

Figure 5: A schematic of incremental pushdown analysis results of a steel MRF building in the vicinity of the removed column

The design of the beams of this grid is performed based on the results of the incremental pushdown analysis of the designed buildings. Figure 5 shows a schematic of the results of incremental pushdown analysis while a column is removed from a steel MRF building. The vertical load capacity of the building in the vicinity of the removed column can be found using this curve.

On the other hand, GSA guidelines require the use of 2×(DL+0.25LL) as the load combination for static procedures. Therefore, in order for the buildings to have enough resistance against the column removals, it is assumed that their vertical load capacity in the vicinity of the removed column should be greater than 2×(DL+0.25LL). In this study, pushdown analysis has been conducted for all the designed buildings, and their vertical load capacities are presented in Table 2 in the form of percentage of the GSA dynamic load combination (DL+0.25LL). All the values of the table are less than 200% –the minimum required capacity– which means that the remaining percentage of the GSA static load combination should be taken by the top beams grid system. As a result, the design load for the top beams grid system is equal to the difference between the GSA static load combination and the vertical load capacity of the designed building. The members of the top beams grid system should have enough capacity to take the design load without failure.

Table 2: The vertical load capacity of the designed buildings in terms of percentage of GSA's dynamic load combination (DL+0.25LL)

Column loss Scenario:	CC	FIC	FLEC	IC	LEC	SEC
5-Story	95%	88%	93%	92%	100%	93%
10-Story	117%	106%	110%	109%	126%	112%
15-Story	135%	113%	124%	118%	130%	124%

Table 2 also shows that taller existing buildings have relatively higher vertical load capacity against column removal events which can be attributed to the higher level of redundancy in these buildings.

Similar to the beams and column of the MRFs of the designed buildings, the sections of the beams of the top grid were chosen from I-Shaped 350W steel sections. According to Table 2, the design of the members of the top grid may result in a different section in each column removal scenario. However, in order to simplify the design, only two different sections for the beams of the top grid in each building were chosen; one for the interior beams and one for the exterior beams listed in Table 3.

Table 3: Designed sections for the members of the top beams grid system

Building	Exterior Beams	Interior Beams
5-Story	W690X140	W690X240
10-Story	W690X217	W760X434
15-Story	W690X240	W920X537

8. Progressive Collapse Analysis of the Retrofitted Buildings

After designing the top beams grid system for each building, the retrofitted buildings were modeled in ELS software and subjected to the GSA's dynamic load combination (DL+0.25LL). Nonlinear dynamic analysis for the six column removal scenarios was then performed for all three buildings. The results show that none of the column removal scenarios was followed by failure and buildings were able to sustain their stability.

When the column is removed in these cases, the beams connected to the in-plan location of the removed column start to deflect. Since the acting load on the beams is a dynamic load, the deflection continues until reaches its maximum value. At this point, the deflection starts to decrease and forms a vibration response which goes on until the inherent damping of the structure dissipates all the kinetic energy.

The maximum deflection of all column removal cases are recorded and presented in Figure 6. This figure shows that column removal scenarios lead to greater deflections in taller buildings. Since the same method is used for designing the top beams grid in all buildings, the greater deflection in taller buildings can have two reasons.

First, in a column removal event, the columns above the removed column are held by the top beams at the roof level. This exerts tension forces and axial deformations in these columns resulting in greater deflections in affected beams of lower floors. The total length of these columns is longer in the taller buildings which results in greater elongations in the columns, and this causes more deflections under the removed columns of these buildings.

Second, the main structural action of the top beams in column removal events is their flexural action which requires relative vertical displacement between the two ends of these beams. When a column is removed, the entire building bends toward the location of the removed column. This phenomenon is more noticeable in taller buildings, and it can decrease the relative vertical displacement between the two ends of the top beams, which causes them to attract less force. As a result, the top beams grid system increases the vertical stiffness in the vicinity of the removed columns less effectively in taller buildings, resulting in greater deflections under the removed columns.

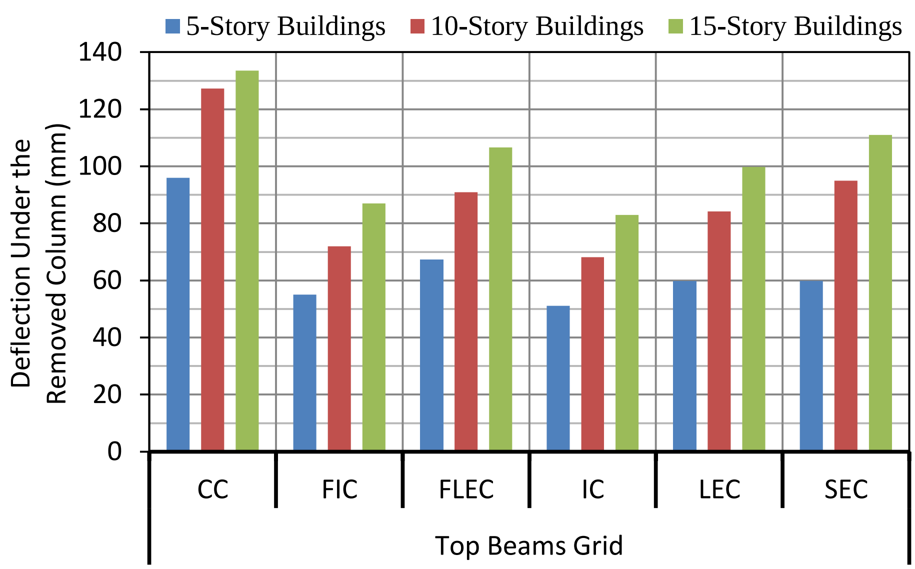

Figure 6: Maximum deflection under the removed columns of retrofitted buildings using nonlinear dynamic analysis

9. Conclusions

Although some research works suggest that seismic design of steel MRF buildings increases the progressive collapse resistance of these buildings, this resistance highly depends on the level of seismicity of the zone. In a low seismic zone such as Toronto, Canada, typical office buildings can be extremely vulnerable to progressive collapse, and retrofitting is needed if progressive collapse is to be prevented.

The main deficiency in the steel MRF buildings which do not have enough resistance against column loss events is the lack of vertical load resistance in the vicinity of the removed column. In these cases, installing a grid of beams above the beams of the top floor can be very helpful. This system increases the vertical load resistance while it barely affects the original seismic design of the building.

The top beams grid system can be very effective in preventing collapses following column removal events. However, this system is less effective in controlling the deflection under the removed columns in taller buildings.

It is important to clarify that the results of this study are only for the specific presented cases. In order to generalize the conclusions, more models of structures with different configurations and capacities should be considered along with experimental studies. Also, it should be noted that a major assumption at this stage is that the floor-to-floor column connections are rigid and can transfer the axial tension loads of the columns.

References

ASCE/SEI 7-05. 2005. *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, USA.

Canadian Institute of Steel Construction (CISC). 2011. *Handbook of Steel Construction (Tenth Edition)*, Canada.

Dinu, F. and Dubina, D. 2009. Robustness of seismic resistant multistory frame buildings in case of accidental column loss scenarios. *Behaviour of Steel Structures in Seismic Areas*, Taylor & Francis Group, London, UK.

DoD. 2009. *Design of Buildings to Resist Progressive Collapse*, Unified Facilities Criteria of Department of Defense, USA.

ELS. 2010. *Extreme Loading for Structures Theoretical Manual*, Applied Science International, USA.

Galal, K., & El-Sawy, T. (2010, April). Effect of retrofit strategies on mitigating progressive collapse of steel frame structures. *Journal of Constructional Steel Research, 66*(4), 520-531

GSA. 2003*. Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. U.S. General Services Administration*, USA.

Khandelwal, K., and El-Tawil, S. 2007. Collapse Behavior of Steel Special Moment Resisting Frame Connections. *Journal of Structural Engineering, 133*(5), 646-655.

Kim, J., Jun, Y., and Park, J. 2010. Performance of Building Structures with Outrigger Trusses Subjected to Loss of a Column. *2nd Specialty Conference on Disaster Mitigation*, CSCE, Winnipeg, Manitoba, Canada, DM-08: 1-9.

Kim, J., and Kim, T. 2009. Assessment of progressive collapse resisting capacity of steel moment frames. *Journal of Constructional Steel Research, 65*(1), 169-179.

Kim, J., and Park, J., 2008. Design of steel moment frames considering progressive collapse. *Steel and Composite Structures*, 8(1): 85-98.

Li, H., and El-Tawil, S. 2011. Three-Dimensional Effects in Progressive Collapse Modeling. *Structures Congress 2011*, ASCE, Las Vegas, Nevada, United States, 2829-2839.

Melchiorre, V. 2008. Vulnerability of Tall Buildings to Progressive Collapse. *Structure Magazine*, 58-60.

Mirvalad, S.J., and Galal, K., 2012. Effect of Seismic Design Level and Building Height on the Robustness of Steel Frame Structures Prone to Progressive Collapse. *3rd International Structural Specialty Conference*, CSCE, Edmonton, Alberta, Canada, STR-1188: 1-9.

National Research Council of Canada (NRCC). 2005. *National Building Code of Canada (NBCC 2005)*. Canadian Commission on Building and Fire Codes, Canada.

Ruth, P., Marchand, K. A., and Williamson, E. B. 2006. Static Equivalency in Progressive Collapse Alternate Path Analysis: Reducing Conservatism while Retaining Structural Integrity. *Journal of Performance of Constructed Facilities, 20*(4), 349–364.