THE INVESTIGATION OF THE EFFECT OF PLAN IRREGULARITIES ON THE PROGRESSIVE COLLAPSE RESPONSE OF LOW TO MEDIUM RISE STEEL STRUCTURES

by

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ABSTRACT

This research examines the effect of plan irregularities on the progressive collapse of steel structures. Firstly, 2, 3 and 5-storey steel structures, regular and irregular, located in regions with different seismic activity designed in accordance with AISC (2010) and ASCE7 (2010). Secondly, the effect of the four plan irregularities on the progressive collapse of braced and un-braced steel structures located in regions with different seismic activity assessed. The collapse patterns of the 14 buildings is analysed and compared under seven loading scenarios using nonlinear dynamic and static analyses. In the nonlinear dynamic analyses, node displacements above the removed columns and the additional force on the columns adjacent to them are discussed. Furthermore, the capacity of the columns is compared to determine their susceptibility to collapse. In the nonlinear static analyses, the pushdown curve and yield load factor of the structures are obtained after column removal. The results indicate that an irregular structure designed in site class C seismic zone collapses in most of the column removal scenarios. Moreover, when comparing regular and irregular structures designed in site class E seismic zone, the demand force to capacity ratio (D/C) of the columns in the irregular structures is on average between 1.5 and 2 times that of the regular ones has been discussed by Homaioon Ebrahimi et.al (2017). The lack of 2-storey building bearing capacity to withstand the removal of the column is lower than that of the 5-storey structure, which is due to the level of redundancy that characterises in the 5-storey structure.

Keywords: progressive collapse, irregularity in plans, steel building, nonlinear dynamic analysis, nonlinear static analysis, pushdown, brace and un-brace steel structures.

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NOMENCLATURE

ANSI	American National Standards Institute
APM	Alternate Path Method
ASCE	American Society of Civil Engineers
BRB	Buckling Restrained Brace
CBF	Concentric Brace Frame
CQC	Complete Quadratic Combination
C_s	Seismic response coefficient
D	Depth of section
DAF	Dynamic Amplification Factor
DCR	Demand to Capacity Ratio
DoD	Department of Defense
EBF	Eccentric Brace Frame
$f_D(t)$	Damping force
$f_I(t)$	Inertial force
$f_S(t)$	Elastic force
GSA	General Services Administration
h_n	Structural height
I_e	The importance factor determined
IDA	Incremental Dynamic Analysis
L	Length
MRF	moment resisting frame
NEES	Network for Earthquake Engineering Simulation

OpenSees	Open System for Earthquake Engineering Simulation
R	The response modification factor
RBF	Reduced Beam Flange
S_1	The mapped maximum considered earthquake spectral response acceleration parameter
SDOF	Single Degree Of Freedom
S_{DS}	The design spectral response acceleration parameter in the short period range
T	The fundamental period of the structure(s)
T_a	Approximate fundamental period
T_L	Long-period transition period(s)
UFC	Unified Facilities Criteria
V_t	Modal base shear
V	The equivalent lateral force procedure base shear
WCPF	Welded Cover Plated Flange
WUF-B	Unreinforced Flange-Bolted Web
WUF-W	Welded Unreinforced Flange-Welded Web

CHAPTER 1: INTRODUCTION

1.1 Background

Several instances on structural failure under unexpected loads, e.g. fire, accidents and explosions have occurred during the past decades. Although structures failure due to the foregoing events is a quite rare phenomenon, but if occurs, it can potentially result in extensive physical and financial damages. The above-said matter was considered for the structures in structural design guideline indirectly and through defining level of importance. However, recently certain guideline such as GSA (2013) has been drawn up concerning the discussion on progressive collapse in structures. In order to learn about any phenomenon in the nature, initially a definition of the same shall be presented and the relevant factor causing such phenomenon shall also be identified. Therefore, various definitions based on a similar generality have been suggested in different guidelines. According to ASCE7 (2010) guideline, progressive collapse phenomenon is defined as the development of failure in a structure from an element to the other, in a way that it will eventually result in total failure of the structure or a major part thereof. The potential events causing such failure include: vehicle impact, gas explosion, plane crash, fire, etc. Most of these events have short term effects, resulting in dynamic responses. Two things are needed for the occurrence of progressive collapse in a structure: (1) an abnormal loading, which may cause failure in primary structural members and (2) insufficient continuity, and degree of redundancy in the structure resulting in initial failure progressing in structural members. In order to control the progressive collapse phenomenon in structures, either of the aforementioned events shall be controlled. The structures behaviour before and after any change in the structure may be studied and a system may be set to confront the same through identifying the factors causing progressive collapse.

Irregular structures are amongst the structures representing more complex and critical behaviour against unexpected loads, as through removing the peripheral column in the structure, the members around the element are removed and not only the same is subject to intensified internal forces, but also torsion in the entire structure is intensified and may play a key role in collapse of the entire structure, or a major part thereof. Therefore, focusing on the behaviour of such structures due to column removal various scenarios may be more challenging and attractive in comparison to regular structures.

1.2 Aim and objectives

The main aim of the current project is to investigate of the effects of plan irregularities on the spread of damage in progressive collapse and of the effects of seismicity on this damage. In particular, this aim will be achieved by considering the following objectives:

1. Determining critical columns and presenting the effects of plan irregularity on progressive collapse under vertical loading when there is a sudden loss of columns.
2. Comparison of the results of the analyses of progressive collapse in areas with different seismicity.
3. Comparison of regular and irregular structures with varying number of stories.
4. Comparison of the result of four irregular steel structures with braced and un-braced system with moment resisting frames.
5. Investigation of the node displacement above the column when there is a sudden loss of column in terms of nonlinear dynamic analysis.
6. Investigation of the load factor to assess the capacity of a structure against progressive collapse in terms of nonlinear static analysis.

7. Investigation of the demand to capacity ratio that is the force taken by columns adjacent to the removed column to assess structural performance under progressive collapse.
8. Investigation of rotation of the beams when there is a sudden loss of column.
9. Determining the strain level for beams and columns according to the relevant criteria.

1.3 Layout of thesis

The present thesis has been prepared in eight chapters as follows:

Chapter 1 presents the generalities of the thesis together with clarification of study path and set aim and objectives.

Chapter 2 focuses on description of progressive collapse fundamentals, where the common definitions of progressive collapse in various standard and guideline are presented and further the regulations stipulated in guideline based on type of loading are presented. Meanwhile, a variety of examples on failure of structures due to unexpected and abnormal loads are given. Meanwhile, the experimental and numerical studies on steel moment resisting frames and braced frames were considered and collected under progressive collapse.

Chapter 3 focuses on various states of progressive collapse in building structures. Furthermore, in this chapter the failure modes and different nonlinear static and dynamic analysis methods are presented and discussed.

Chapter 4 discusses the research methodology including defining the approach used in modelling, type of analysis and design, geometry and loading of the structure and sections applied for the models. Moreover, at the end of this chapter, OpenSees software for numerical modelling is presented.

The formulations in structural dynamics are presented in Chapter 5 and then the numerical model in OpenSees software is verified and compared with the model given by Chopra (1995).

Chapter 6 discusses the results of nonlinear static analysis for all the models studied. The results are discussed and studied in terms of history of node displacement, capacity curve, failure actions and determining the load factors.

Chapter 7 focuses on studying the numerical results under nonlinear dynamic analysis. The results are presented and discussed within the framework of changes and distribution of plastic strain, connection rotation and node displacement above the removed column under different column removal scenarios. Also at the end the parametric study on 5-storey structures is conducted upon changing the number and length of spans and effect of the same on structure behaviour under column removal scenario.

Chapter 8 includes conclusions and suggestions for further relevant studies.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Over the last few decades, much attention has been paid across the globe to the issue of progressive collapse. This is due to the fact that progressive collapse is an alarming reality for huge structures and can lead to irreparable human and financial losses. This chapter will first discuss various definitions of this rare phenomenon proposed in different regulations and by different researchers. Through several examples of partial or total collapse that have occurred across the world, the chapter will then examine the factors contributing to this phenomenon and the ways of reinforcing structures according to regulations. Finally, research conducted in this field will be examined and summarized.

2.2 Definitions of progressive collapse (disproportionate collapse)

There are various definitions of "progressive collapse", and this term is still controversial for certain authors. As can be seen below, some researchers refuse to use the term "progressive", as all types of structural damage have a certain degree of "progressiveness"; these authors, therefore, prefer the term "disproportionate". The various definitions used in design standards of progressive collapse are presented below.

ASCE 7(2010) 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.*"

The National Institute of Standards and Technology (NIST) offers a definition similar to that of ASCE 7(2010): "[Progressive collapse is] *the spread of local damage from a single initiating event, from* *structural element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it.* This type of collapse is also known as disproportionate collapse."

European codes do not have specific standards for progressive collapse, but do include standards for accidental phenomena such as fire, explosion and impact.

The General Services Administration (GSA) guidelines published in 2013, which analyse progressive collapse and offer design solutions, define it as "*a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause.*"

The General Services Administration (GSA (2013)) Guideline, and the American Department of Defense (DoD (2009)) define an acceptable area of collapse by specifying the floor area that bears the initial local damage. In addition, these documents determine the initial local damage (usually one column on the ground floor), which must be noted. It can be inferred that, according to these guidelines, any type of damage beyond the allowed collapse area can be regarded as disproportionate collapse.

In order to summarize these definitions, and to specify what will be regarded as progressive or disproportionate collapse in this study, the following is proposed:

"*Progressive collapse is defined as a situation where one or more structural members suddenly fail and the structure experiences progressive destruction. In this situation, load distribution breaks other structural members one after another until a new state of equilibrium is created where parts or all of the structure is destroyed.*"

Disproportionate collapse can be regarded as disproportionate progressive collapse where the total damage is beyond that allowed according to relevant standards. Consequently, the definition of disproportionate collapse varies between cities, guidelines and standards according to the allowed risk. Different design codes tend to use the term “progressive collapse” when referring to disproportionate collapse.

2.3 The significance of progressive collapse

Although rare in most parts of the world, progressive collapse can have catastrophic consequences. In view of the rise in terrorist attacks in recent years, the study and examination of progressive collapse seems crucial.

2.4 Examples of progressive collapse

Examples of structures that have experienced partial or total progressive collapse are few and far between; thus, this phenomenon has been gradually incorporated into design standards. A number of progressive collapse cases are presented here.

A. Ronan Point building

Ronan Point was a multi-story structure that was built between 1966 and 1968 in London. In1968, gas explosion occurred in the external part of the bottom panel of the exterior wall on the 18th floor of this 22-storey building. The structural system consisted of precast concrete slabs for walls, floors and stairways. The walls and the floors had been bolted together, and the joints had been filled with compressed grout. This means that the upper floors did not have the strength to resist bending moment, especially if they were suspended (so that each floor was supported directly by the walls beneath). Therefore, when the panel of the external wall was moved towards the outside because of the explosion at the 18th floor, the upper floors (19-22) collapsed. The falling debris destroyed floors 17 down to the ground floor, which resulted in the destruction of one of the sides of the building (Fig. 2.1). The collapse of Ronan Point showed the insufficiency of the floors in resisting bending when suspended. The collapse of this building was progressive; with the collapse of one external wall, the structure was unable to bear the resulting redistributed strain (i.e., it could not provide an alternative load path).

Figure 2.1 The Collapse of Ronan Point Building

(Source:http://www.emergencymgt.net/sitebuildercontent/sitebuilderfiles/ProgressiveCollapseBasics.pdf

Source: Shankar Nair. R. Progressive collapse basics)

B. L'Ambiance Plaza

L'Ambiance Plaza was a 16-storey building in Bridgeport, Connecticut. The structure had two cantilever slabs made of pre-stressed concrete. Workers were tack-welding wedges under the 9th, 10th, and 11th floor packages to temporarily hold them in place when a loud metallic sound, followed by rumbling, was heard. An iron worker installing wedges at the time looked up to see the slab over him "cracking like ice breaking." Suddenly, this slab fell onto the one below it, which was unable to support this additional weight and in turn also fell. The entire structure then collapsed, first the west tower and then the east tower in 5 seconds, which was only 2.5 seconds longer than it would have taken for an object to free fall from that height. Ten days of frantic rescue operations revealed that 28 construction workers died in the collapse, making it the worst lift-slab construction accident ever. Fig. 2.2 shows the building at the time of the collapse (Matrin et al. 2000).

Figure 2.2 L'Ambiance Plaza at the time of collapse

(Source: https://westernctalf.org/events/31st-anniversary-lambiance-plaza)

The collapse, which occurred on April 23, 1987 when the building was still under construction, was caused by the slab collapse triggering the destruction of the flange, which was shortly followed by the destruction of the eastern flange, which led to the total collapse of the structure.

C. The Alfred P. Murrah Federal building

The Alfred P. Murrah Federal Building, in downtown Oklahoma City, Oklahoma, United States, was constructed between 1970 and 1976, as an administrative building for the US government. The building was under attack when a truck full of explosives parked in front of the Eastern side of the building exploded. The structural system consisted of reinforced concrete, accompanied by girder on the third level of the Northern side. The columns close to the explosion were immediately destroyed by the blast, which resulted in the collapse of all the upper level floors Fig. 2.3. This event is a good example of progressive collapse due to lack of sufficient capacity in the frame system and the girder to bear the increased bending moments and the shear forces caused by the collapse of three adjacent columns on the ground floor.

Figure 2.3 The Alfred P. Murrah Federal Building after collapse

(Source: https://www.cbsnews.com/pictures/oklahoma-city-bombing/)

D. The Bankers Trust building

The Bankers Trust Building is another example of structures experiencing progressive collapse. Its steel structure was built in early 1970 on Liberty Street, New York, immediately to the West of World Trade Centre. Its structural system was a steel moment frame with girders welded to the columns. The structure experienced the impact of the debris caused by the destruction of the Northern side of the World Trade Centre. The exterior parts of the external wall of the Northern Tower hit this bank on the 23rd floor. The destruction included the collapse of all floor systems and the perimeter beams between the 23rd and 29th floors, and one column was destroyed between the 18th and 9th floors. The resulting shear can be seen in the Northern side of the building in Fig. 2.4.

Figure 2.4 The shear in the Northern side of the Bankers Trust Building

(Source: http://911research.wtc7.net/mirrors/guardian2/wtc/WTC_ch6.htm)

Despite the elimination of vertical support, no additional damage directly followed from the impact of the WTC debris on this building. Evidently, the steel frames had sufficient redundancy and ductility to redistribute the stress caused by the removal of the columns and was able to absorb the kinetic energy released by the sudden removal of the column and the debris impact.

E. The Collapse the 16-Storey Building at 2000 Commonwealth Avenue

On January 25, 1971, almost two-thirds of a 16-storey building under construction collapsed in Boston, Massachusetts, resulting in the death of four people. The building, which had been under construction for over six years, collapsed in only a few minutes. Subsequent studies showed that the fall of a piece of equipment from the crane carrying it had caused punching shear. As a consequence, the roof had collapsed, which triggered the progressive collapse of the building. The studies on this building showed that the stages of the collapse were as follows:

1. Collapse of the building due to the insufficient strength of the floor against punching shear close to one of the columns.
2. Collapse of the roof slabs.
3. Initiation of progressive collapse, leading to the total collapse of the building.

The probing committee concluded that the structure suffered from several faults in terms of both design and execution, the most important of which were the inadequate props under the roof slabs and the low strength of the roof concrete. The committee also concluded that the design had not observed the required minimum roof slab thicknesses.

In this building, the force acting on the slabs around the column had overcome punching shear resistance, causing the slabs around the column to fall. Some of these slabs remained, whilst the rest fell onto the floor below. Progressive collapse is triggered if the slabs of a floor are unable to withstand the weight of a floor above it and the impact caused by the collapse of the roof. Another point to consider is that punching shear leads to the redistribution of the forces acting on the failed slab onto other columns. If these columns are unable to bear the added load, the slab will experience punching shear around those columns. In other words, the punching shear of the slabs around a column can trigger progressive collapse and the total destruction of a structure. Studies show that in order to reduce the risk of punching shear and the progressive collapse that is the result, consideration should be given to certain factors, such as concrete strength, the load-bearing-area-to-slab-thickness ratio, the shape of the load-bearing surface and the shear-force-to-bending-moment ratio at the slab-column connection.

F. The Hartford Civic Centre Coliseum collapse

In 1978, the roof of the Hartford Civic Centre Coliseum, which consisted of a space truss with spans of 110 and 95 meters, collapsed because of rain and snow with a load of approximately 0.814 KN/m^2, which was 60% of the design load. Shortly after this event, another sports stadium had a similar collapse in Missouri. The two tragic incidents culminated in ample research into the phenomenon. The point to consider is that both structures had followed the guidelines of the time and had been properly built. Therefore, the causes of this type of collapse must be examined, and the guidelines must be revised.

Figurer 2.5 The Hartford Civic Center Coliseum Collapse
(Source: http://www.engr.mun.ca/~molgaard/courses/1000/failengh.htm)

G. Kemper Arena's collapse

In 1979, and shortly after the Hartford Civic Centre Coliseum Collapse, most of the roof of Kemper Arena, Kansas City, Missouri, collapsed because of a combination of wind and rain. Ever since these two incidents, many books have been written on structural failure and collapse and a lot of research has been conducted.

The American Society of Civil Engineers (ASCE7 (2010)) tops this list with many books on case studies of several types of failure. The American Concrete Institute (ACI) has, also, published several case studies of structural failure. This helped enhance and revise structural regulations at the time, the effect of which is evident in the significant decrease in these types of failure in the following years.

H. The collapse of World Trade Center towers

One of the more recent tragic incidents of progressive collapse in tall buildings occurred on September 11, 2001, in New York. On this day, two 101-storey towers were totally destroyed as a result of the impact of two passenger planes. In this incident, 2830 people were killed and many nearby buildings were damaged. The two towers had perimeter steel frames. Interestingly, the towers had been designed to withstand potential aircraft impact, which was fully explained in the report of the investigative committee assembled after the incident.

Figure 2.6 Progressive Collapse in World Trade Centre Towers

(Source: http://www.dailymail.co.uk/news/article-2040657/Explosions-caused-jet-fuel-water-sprinklers-brought-Twin-Towers-9-11-scientists-say.html)

The reports indicated that progressive collapse in the two towers occurred as follows: first, upon the impact of the plane, around 60% of 60 columns of the perimeter steel frames on the impact side fractured, and many others experienced significant deformations. This led to the redistribution of the strain and the consequent increase in the loads of a number of columns; for some of these columns, this was near to the maximum load-bearing capacity. Then, as a result of the loss of a significant amount of the protective covering of the steel, many of the steel members reached temperatures of approximately 600°C (steel structures lose about 20% of their yield resistance at 300°C and about 85% at around 600°C). Next, different thermal expansion rates led to the deformation of the floor trusses. This deformation pulled inwards of the peripheral columns and deformed them, in turn leading to a bigger problem, i.e., the out of plane buckling of the peripheral columns. Studies have highlighted seven contributing factors to the failure.

1. Increased strain in some columns due to redistribution of the strain upon impact.

2. Overheating due to the loss of the protective coverings of the steel.

3. A significant decrease in the yield point and in the creep threshold due to heat.
4. Lateral displacement of many columns due to thermal strain.
5. Weakening of lateral supports due to a decrease in the stiffness of the planes of the deformed floors.
6. Some multi-storey columns, for which the critical load was less than that of the columns, buckling over the height of one floor.
7. Local buckling of the heated columns.

Ultimately, the columns buckled, after which the upper part of the tower collapsed after experiencing only a little resistance. This fell at least the height of one floor onto the floor beneath it, and triggered the progressive collapse. The kinetic energy caused by the fall of the upper part exceeded the allowed plastic displacement at the bottom part. This scenario was presented through computer simulations at the NIST (2007).

2.5 Direct method approach to the mitigation of progressive collapse

The direct method consists of improving the structural integrity of a building by improving the redundancy of load paths and ductile detailing. The direct method is divided into two approaches: specific local resistance and the alternate load path.

"The GSA describes cases in which one of a building's columns is removed and examines the damaged structure to check the system responses according to the alternate path method in order to reduce the catastrophic effects of progressive collapse". In this approach, which is a direct method, the structural capacity is increased by bridging over an element in the removal scenario, limiting any damage. Simplification is largely acceptable for the design process and to ensure the existence of an alternative path. The scope of this method includes nonlinear static and dynamic analyses. The alternative path approach has been adopted by many standards such as those of the General Services

Administration (GSA) and the American Department of Defense (DoD) guidelines. The GSA's guidance contains detailed calculation methods.

2.6 Guidelines and standards for the mitigation of progressive collapse

Studies show that the progressive collapse process can be triggered by numerous events, including bombing, increased loading during construction, internal blast, overloading during use, fire, earthquakes, inadequate floors and vehicle crashes. As a consequence, if any of these events is likely, the designer must consider a plan to manage the potential progressive collapse of the structure. The question is how far the structural risks can go and to what extent the structure should be strengthened against these risks. Answering these questions is quite challenging, and there have been numerous attempts to identify solutions. To this end, there are modern international regulations to, as far as possible, prevent the types of event mentioned above.

In June 2013, the GSA published its guidance for the prevention of progressive collapse, which was to be employed in all American buildings. These guidelines consider a combination of dead and live loads. Also presented are the coefficients of dynamic load strengthening for nonlinear static analyses and dynamic analyses. The event that must be considered as the initial cause of collapse is the destruction of one column on the ground floor. The nonlinear static method includes step-by-step nonlinear static calculations, which ultimately yield an estimate of the scope of the damage to the structure. This scope is then compared with what is considered to be bearing the load in other guidelines to see whether the design can be confirmed or must be rejected.

In June 2005, the American Department of Defense (DoD (2005)) published its latest guidelines with respect to progressive collapse. In this guidance, structures are classified according to their security levels. If a lower security level is required, the security of the structure is ensured based on vertical nodes, while for higher security levels, in addition to the nodes, the approach to the alternative paths is specified. The step-by-step method for static analyses is similar to that of the GSA (2013) from a general standpoint. The principal differences lie in the behaviour of the materials used in the simulations. In fact, in nonlinear static and nonlinear dynamic methods, the details of the DoD (2005) guidelines are more efficient.

2.7 Previous studies

2.7.1 Experimental investigations

Corley et al. (1998) published three papers with respect to the collapse of the Murrah Building. In their first paper, (Mlakar et al. 1998), blast loading and its effect on the building were researched. The authors claimed that the energy released by the explosion was equivalent to that from the detonation of 1,814 kg of trinitrotoluene (TNT). The blast removed one column directly and the associated air blast led to failure of two more. Slabs were also damaged because of the explosion. The damage caused by the blast, the failure mechanism for the building and engineering details of the building were described in their second paper (Sozen, et al. 1998). The loss of the three columns caused the transfer girder supporting the upper portion of the building to fail promoting further failure. It was discovered that the building would have collapsed had even a single column been lost. Another conclusion was that use of continuity reinforcement and column shear reinforcement may have reduced the potential for progressive building collapse. The authors proposed recommendations for mitigating progressive collapse in their third paper (Corley et al. 1998), including the use of compartmentalized construction, special moment frames and dual systems for new buildings, and the use of extra structural walls, supplemental supporting frames and column jacketing for existing buildings. One of the key conclusions was that seismic detailing could contribute to the collapse resistance of the structure.

In 2003, Tan and Astaneh-ASL studied the effect of using steel cables in the ceilings of steel buildings to control progressive collapse. They conducted three tests, including: (1) a sample without any mechanism to resist progressive collapse; and (2) and (3) representing steel cables situated on the beam that were web connected to the edge of the column. They discovered that the cables resulted in a significant increase in resistance to progressive collapse.

In an attempt to increase the capacity of a steel building to prevent progressive collapse, Astaneh-Asl (2007) performed a full-scale test of a steel frame with bolted seat and web angle shear beam-to-column connections. The south side of the test specimen was a 60' by 20'1-storey steel structure with a steel deck and concrete slab floor system and wide flange beams and columns. The height of the columns was equal to 6'. The north side of the specimen consisted of a similar steel frame, but this contained catenary cables running longitudinally, as seen in Figure 2.7. These catenary cables were used to develop the catenary action under the vertical load, which would, eventually, enhance the progressive collapse resistance of the frame. The centre columns on each longitudinal frame of the specimen were constructed 36 (in) above the laboratory floor. This action was taken to simulate the sudden loss of these columns in the event of a blast (columns C1 and C2 from Figure 2.7 were the two columns removed). A hydraulic actuator pushing downwards on the top of these two columns was used to simulate the gravity loads on the columns.

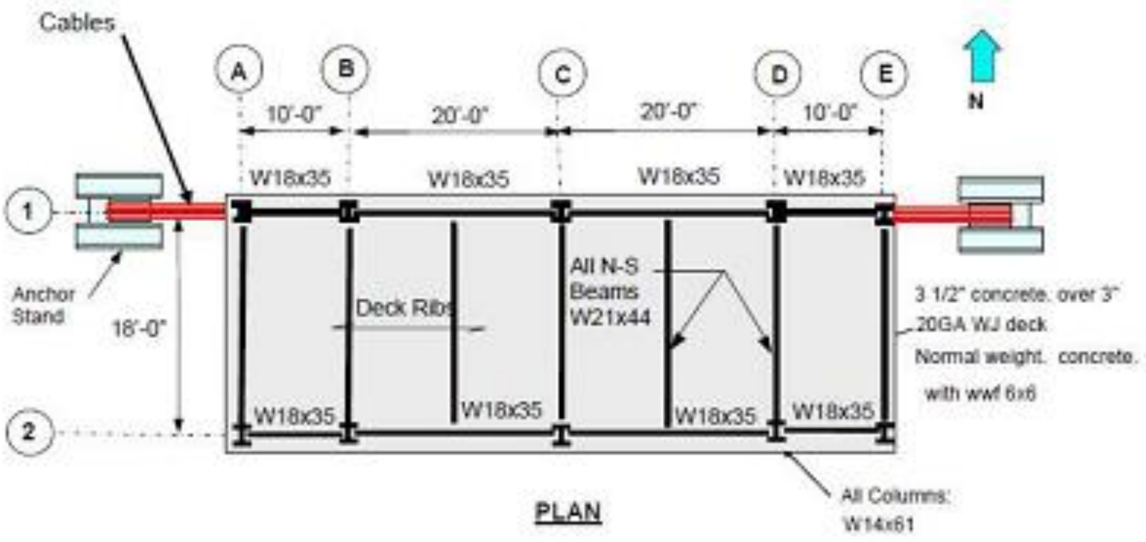


Figure 2.7 Plan view of the tested specimen with catenary cables (Astaneh-Asl 2007)

It is apparent that the frame with the catenary cables was able to resist higher loads than the frame with no catenary cables. At the column downward displacement of 27(in); the catenary cables were supporting over half of the column load. The investigator proposed to install the catenary cables as a method to retrofit existing structures to enhance progressive collapse resistance.

Despite the fact that the catenary cable provides additional vertical load resistance after a sudden column removal it would just work under particular conditions. The catenary cables would provide additional progressive collapse resistance just when an interior column is removed due to an extreme event. For the situation in which a corner column fails, the cables may lose anchorage at the end and likely provide no additional catenary action.

In 2009, Sadek et al. studied the behaviour of steel structures with two types of moment resisting frames. Their research was conducted by removing the middle column and imposing the vertical displacement on it. Two beams with different spans and three columns were considered. The main purpose of this study was to determine the behaviour of connections and their resistance to tensile force occurring in the beams.

Among the first points to become clear from a series of field experimental tests on the behaviour of WUF-B and sideplate connections (Karns et al., 2007) was the need of sufficient connection rotational capacity to arrest progressive collapse. The examination also warned that moment connections prequalified for rotational capacity due to bending alone might not perform equally under combined bending moment and axial loading. Failure for the WUF-B connection was brittle and it was observed at the compressive beam flange; after that, the flexural demand in the connection interface had to be resisted by the beam's bolted web, which in turn quickly deteriorated. The side plate connection was able to maintain stability under much higher loading and failure in the system was observed in the beam instead of the connection. Results distinguished deep rolled wide-flange steel sections as a cost-effective solution for enhancing progressive collapse resistance because of their ductility.

Comparison between experimental results and a nonlinear dynamic simulation (Kwasniewski, 2010) has shown that finite element models may not always accurately represent connection response in progressive collapse.

Limited experimental studies have been conducted concerning the analysis of structures subject to progressive collapse. One of these cases is the experimental and numerical study performed on two Ohio Union and Bankers Life and Casualty Company buildings. Figs. 2.8 and 2.9 show the state before and after the test (Song et al. 2010).

Figure 2.8 The state before and after removing the column in Ohio Union 4 storey building (Song et al. 2010).

Figure 2.9 The state before and after removing the column in Bankers Life and Casualty Company 4 storey building (Song et al. 2010).

A linear static analysis was performed on both buildings using the SAP2000 software. The relevant results indicated that due to the removal of the columns, those in the upper floors were under their own weight more than the other columns. It was concluded that the Ohio Union Building may satisfy the GSA (2003) progressive collapse criteria for all the frame members; only five columns in this structure were destroyed. On the other hand, the BLCC Building could not satisfy these criteria due to the removal of the 1st floor columns. Calculating the capacity ratio and the maximum displacement indicated that more buildings are subject to failure upon the removal of the columns. In this study, the beams were more critical compared to columns against impact loading.

In 2013, Kandil et al. studied progressive collapse in steel frames with beam-to-column connections. Their research considered a 2-storey and 2-span frame and examined the parameters of different connections, different geometry, different rigidity conditions and the damping ratio in the frames under progressive collapse. The results indicated that the maximum lateral displacement for the removed corner column was more than the calculated value, which was due to the failure of the connection to act as fully-rigid. Meanwhile, the column adjacent to the removed column was subject to greater strain compared to the other columns.

Hadjioannou et al. (2013) evaluated experimental test the effect of concrete slab on reducing the damage inflicted to steel moment frame under losing an internal column. In this regard, six samples with real scale, including three two-span, two-floor internal samples and three two-span, one-floor external samples were made. In all the tests, the middle column was removed statically while the concrete slab surface was under loading. It was understood through analysis that the concrete slab plays an effective role in total stability of the building and reducing the damage.

Guo et al. (2013) experimentally and numerically studied steel moment frame with composite slab in 1:3 scale. The main focus of the foregoing study was on catenary action upon removal of the columns and distribution of the forces in the elements. It was understood in these tests that rigid connection in concrete slab with shear stud highly helps in chain action, which in turn results in an increase in strength and ductility. The test results showed that six stages (elastic, elastic- plastic, arch action, plastic, transition and catenary action) may be considered in progressive collapse.

Ali and Falah (2014) studied the behaviour of a two-storey, two-span steel frame due to removal of perimeter column on the first floor. In this study, the effect of internal shear panels and bracings on the structure capacity was investigated. Meanwhile, this frame finite element study was also conducted in ABAQUS software. In numerical analysis, four thicknesses (2, 4, 6, 8mm) were used for shear panel and three brace types (diagonal, crossed and "V" shaped) were used. Using shear panels resulted in a further decrease in structure vertical displacement compared to brace.

Li et al. (2017) performed a pushdown analysis with a 1:3 scale 1-storey moment frame substructure. What characterized their work was their examination of the behaviour of bare steel moment frames under a column-loss scenario, and their validation of the computational models that had been developed to investigate progressive collapse in steel-frame structures. The results of the experiment show that flexural action plays a significant part in resisting progressive collapse throughout the loading process. Nevertheless, catenary action turns into the principal mechanism of resisting collapse in the last phase of the loading process. The energy-based method was employed to estimate the test specimen's dynamic response, which was shown to have elastic behaviour when subjected to the sudden removal of the centre column; as a result, there would be no progressive collapse. The dynamic increase factor was also estimated based on the test results. The results of the analysis showed that catenary action significantly affects the dynamic increase factor value in huge deformation conditions.

2.7.2 Numerical research:

2.7.2.1 Case studies on progressive collapse

One of the earliest studies on progressive collapse was conducted by Bažant and Zhou (2002), a simplified approximate analysis of the overall collapse of the WTC towers. The investigation revealed that failures of columns in a single floor due to long time exposure to fire could induce collapse of the whole building due to the impact of the upper part on lower part of the building. They proposed a technique that they thought could prevent this type of progressive collapse, i.e. by introducing collapse barriers. The collapse barriers, which consisted of foam impact absorbing material, could arrest the downward traveling stress wave which caused the structural collapse of the WTC towers.

Meanwhile, Bažant and Verdure (2007) reviewed the mechanisms of the WTC collapse and developed a dynamic one-dimensional continuum model of progressive collapse. The authors discussed that crush-down and crush-up phases of one-dimensional progressive collapse must be distinguished. After formulating and solving the differential equations for these two phases, the authors claimed that the duration of the collapse matched well with the proposed model. Another key of this investigation was that "if the total energy loss during the crushing of one storey exceeds the kinetic energy impacted to that storey, collapse will continue to the next storey." Once this criterion was satisfied, progressive collapse would progress because of gravity alone.

In addition, the investigation conducted by the National Institute of Standard and Technology (NIST) (2007) revealed that the structures had enough robustness to withstand the impact of the aircrafts and it was a different combination of impact damage and heat-weakened structural components that caused the collapse of the structures. Moreover, the report noted that the buildings would likely have survived had the thermal insulation not been widely dislodged by the aircraft impact and subsequent blast. After analyzing the failure sequence of the twin towers and the key factors that led to their collapse, it was prescribed that progressive collapse be avoided in structures through the improvement and nationwide adoption of consensus standards and code provisions, along with the tools and guidelines for their use in practice. In particular, the capability to prevent progressive collapse due to abnormal loads ought to include: (1) comprehensive design rules and practice guides; (2) evaluation criteria, methodology, and tools for assessing the vulnerability of structures to progressive collapse; (3) performance-based criteria for abnormal load and load combinations; (4) analytical tools to predict potential collapse mechanism; and (5) computer models and analysis procedures for use in routine design practice.

Following, Cherepanov (2006) compared two theories for the WTC collapse, i.e. the theory of progressive failure and the theory of fracture waves. The outcomes obtained by the former contradicted the observed free fall. The latter demonstrated that the WTC towers "disintegrated at the very beginning of each collapse", thus free fall of the building fragments was suggested.

Another simplified analysis was performed by Seffen (2008). The author examined the dynamic behaviour in the collapse of the WTC using a variable-mass collapse model. One major conclusion was that the collapse was a result of propagation of instability induced by "near free fall" of the upper part of the building. Nevertheless, Grabbe (2008) did not agree with the conclusions of this examination, demonstrating that some of the assumptions made in the analysis did not agree with physical principles, thus the results were inaccurate.

In spite of the fact that there are still controversies about the mechanism behind the collapse of WTC, it is a typical view that the twin towers collapsed because of the fire rather than the impact of the aircrafts.

In the following, Yang et al. (2016) numerically studied a steel frame under progressive collapse due to fire. In this study, the macro model of shear connections and a behavioural model able to consider the nonlinear dynamic behaviour were suggested. Using AUTODYN finite element software, compressive internal forces due to fire inside the structure were calculated. The results indicated that the columns inside the moment frame play a significant role in structure total stability. The explosion not only causes elements geometrical failure, but also results in connections yielding.

Osteraas (2006) explored the blast damage and collapse patterns of the Murrah Federal Building. He claimed that the collapse was a result of the combined effect of the blast, which destroyed one column and large portions of the slabs in the second, and third probably part of the fourth floor, and the structural configuration of the building, which lead to loss of lateral bracing of the other three buckled columns. The author speculated that ductile detailing could have improved the performance of the building subjected to abnormal loads. From these two disastrous occasions, researchers discovered that civil structures need to have enough local strength to avoid the initiation of progressive collapse. Moreover, in order to prevent occurrence of progressive collapse under extreme load events, system strength, stiffness, structural redundancy and integrity are all critical and must be adequately ensured.

In 2009, Jae Hyouk et al. theoretically examined two structures that collapsed due to progressive failure, comparing their results with what actually happened and concluding that their findings reflected reality to a great extent. The first structure studied was a railway building in South Korea, the first failure of which was due to snow. The second structure was the Ronan Point building in the UK, where a gas explosion in the vicinity to one of the building's corners was reported. After comparing the real and theoretical results, which were very similar, they suggested certain solutions to prevent progressive collapse in such buildings.

2.7.2.2 Studies of the effect of different connections against progressive collapse

One of the main parameters of uncertainty is the failure strain, especially for bolts. Finite element models do not always explicitly take this into account, as material failure leads to the component failure by deleting (eroding) a finite element from further calculations, while in actual connections failure is usually initiated by the rupture of fillet welds or by indirect bolt failure, such as shear stripping of the threads. Another case is the underestimation of the initial stiffness of the connection, which is usually attributed to the bending of the end plate due to imperfections and the realization of contact between the flush plate and the column flanges, which is not included in the model.

Sadek et al. (2009) summarized the development of 3D macro-models for steel moment-frame buildings, which were designed for the purpose of studying progressive collapse of buildings in moderate and high seismic regions. Focus was placed on the modelling of the connections and composite floor systems. However, the simplified slab model was not validated. The beams and columns were represented by Hughes-Liu beam elements. The connections were modelled with beam elements and discrete spring elements, which were validated against high-fidelity finite element simulations or full-scale test data, depending on the type of the connection. The slab was modelled with a single layer of shell element for simplicity. Some initial simulation results for one prototype building under a column removal scenario were also provided in order to illustrate the model capabilities. Additional simulations that would have illustrated the capacity of the proposed model to represent the behaviour of the structure under column-loss scenarios were not provided.

Meanwhile, Kim and Kim (2009) used a macro-scale planar model to examine the progressive collapse performance of Reduced Beam Section (RBS), Welded Cover Plated Flange (WCPF), and Welded Unreinforced Flange-Welded Web connections (WUF-W). Two types of steel moment frame buildings, designed for high seismic risk and moderate seismic risk were used in progressive collapse analysis. The buildings are 3-storey and 6-storey high with various connection types. In this study, nonlinear planar models which represented the perimeter moment frames of the buildings were used. The panel zones of all types of connections were modelled as rigid and distributed plastic hinge region was incorporated into all types of connections in order to mimic formation of plastic hinges. The beam and column members were represented by nonlinear beam-column elements provided by the OpenSees software, the interaction between axial force and bending moment reaction could also be considered by using the element. Nonlinear time-history seismic analysis, static push-down analysis and nonlinear dynamic progressive collapse analysis were conducted using the proposed models. It was concluded that although the seismic performance of the three types of connections was similar, WCPF was the most effective in resisting progressive collapse, especially in structures located in moderate-seismic regions.

In 2010, Sadek et al. conducted a study of steel frame structures that had three columns and two beams. This study was performed on two 10-storey buildings designed to mitigate progressive collapse. The first test included welded connections, while the second represented connections in which the beam was of a reduced cross-section type. The scholars increased vertical displacement on the column to define the relevant reaction. Certain points and horizontal and vertical displacements were observed, like the beams' end rotation, and their loads were calculated. The results of this study showed that the rotational capacity for both connections was double the figure found out of the values resulting from the seismic analysis.

A simplified numerical model was developed for studying the progressive collapse of three-dimensional semi-rigid jointed steel frames under nonlinear static loading conditions by Bandyopadhyay et al. (2016). Their numerical method is carried out that includes different types of semi-rigid connections and different cases of column removal at different stories. It was found that the progressive collapse resistance of a frame is influenced not only by in-plane-member end connection characteristics but also by out-of-plane-member end connection characteristics. Based on results, a new retrofitting scheme for improving progressive collapse resistance was recommended.

Karns et al. (2003) studied the behaviour of two WUF-B and Side Plate connections against explosion and column removal scenario. The results indicated that there is a significant different between the performances of two WUF-B and side plate connections when subject to column removal testing. Especially, the difference ratio in the input energy for the failure of two connection types in the test almost reaches 5%. The test results showed that side plate has a higher rotation and loading capacity in comparison to WUF-B connections against the damages caused by explosive and non-explosive loads and the analysis results also confirmed this issue.

2.7.2.3 Studies of the effect of the braced frame against progressive collapse

In 2009, Kim et al. studied progressive collapse in braced frames using nonlinear dynamic and static analyses. They studied the frames performance using 8 different braces with similarly loaded moment frames. According to the analysis results, most of the braced frames designed based on current construction guideline meet the requirements included in design guideline against progressive collapse. Most of structural models showed more brittle failure due to columns and braces buckling. In most bracings, after compressive bracing buckling, the column is buckled and brittle failure occur. Merely in “V” shaped bracing the beams are subject to unbalanced forces between buckled compressive bracing and elastic tensile bracing. Therefore, failure delays in comparison to other bracing states. Hence, among studied bracing types, “V” shaped bracings indicate a quite ductile behaviour within progressive collapse in comparison to other bracings. Additionally, based on the analyses results, bracing frames displacement is much less than that of the moment frames.

In 2009, Khandelwal et al. conducted a study to evaluate the braced steel frames using models verified based on numerical simulation. They performed the alternative path method on a 10-storey structure by removing the main load bearing column and adjacent bracings. This method was applied to determine the capability and strength the structure for element removal. Finally, they concluded that the frame braced using eccentric bracings have more resistance in comparison to concentric ones against progressive collapse.

In addition, Khandelwal et al (2009) investigated the progressive collapse resistance of seismically designed steel braced frames using the same modelling approach described above. The moment connections were modelled by rigidly attaching beams and columns to the connection region; brace-to-beam connections were not explicitly modelled because their responses were expected to be elastic; panel zones were not explicitly modelled, because it was shown that the panel zone behaved elastically under collapse conditions. The shear connections were modelled using two spring elements and a beam element which represented the bolts and shear tab interaction; the shear links were represented by 4 bars pinned together at their ends to permit the desired shear-flexural deformation, within which two bars are rigid and two are elastic, and the stiffness and strength of the link were provided by a nonlinear spring. The models were calibrated by comparing model responses to test data and more refined models that were also validated by comparison to test data. Two braced frames: a special concentrically braced frame, which addressed moderate seismic risk and an eccentrically braced frame, which addressed high seismic risk, were modelled using the proposed macro-model and nonlinear dynamic analysis was performed using alternate path method by removing critical columns and adjacent braces, if present. The analysis results revealed that although both systems benefitted from placement of the seismically designed frames on the perimeter of the building, the eccentrically braced frame had better progressive collapse resistance because of improved system and member layouts rather than use of more stringent seismic detailing.

Meanwhile, Parvari and Bahri (2016) investigated the behaviour of steel frames with knee-bracings under gravity and seismic loads due to progressive collapse. Three 4, 7 and 10-storey steel frames were subject to 2D nonlinear static and dynamic analysis in Perform-3D software. In order to compare, CBF bracings were also used. Pushdown analysis results indicated that the frames equipped with knee-bracing under the concentrate loads have capacity more than the CBF bracings.

Furthermore, Faghihmaleki et al. (2017) considered three 8-floor steel moment frames equipped with BRB, RBF and CBF bracings to study the structures behaviour under progressive collapse due to quake. In this study, unlike other studies, some of the bracings were removed and then the frames were subject to seismic load to measure system potential through dynamic analysis.

2.7.2.4 Studies of the effect of catenary action against progressive collapse

Kim and An (2009) investigated the effect of catenary action in the progressive collapse potential of steel moment frames structures using nonlinear planar macro-scale models. The program code OpenSees was used in this study. The beams and columns were modelled using "nonlinear beam column" element and both geometric and material nonlinearity were considered. Nonlinear static push-down analysis of the beam-column sub-assemblage was first conducted. It was shown that the beams could resist significantly larger load when catenary action was considered. Three-storey and six-storey steel frames with and without braces were then modelled using the proposed modelling approach and both nonlinear static and nonlinear dynamic analyses were performed. It was shown by nonlinear static pushdown analysis that the contribution of catenary action and the progressive collapse potential of structures increased as the number of stories and the number of bays increased. The effect of catenary action also increased as the constraint to lateral movement increased due to braces, for example. The nonlinear dynamic analyses showed that the peak displacement caused by the sudden removal of a column decreased when considering catenary action.

Khandelwal and El-Tawil (2007) investigated the collapse behaviour of seismic, detailed, steel special moment-resisting frame connections, focusing on the effect of catenary actions on connection performances under column loss scenarios. In order to investigate a number of key design variables that influenced the formation of catenary actions in steel special moment-resisting frame sub-assemblages, a detailed finite-element model of a two-bay steel sub-assemblage with seismic detailing was developed. The model employed a constitutive model for steel that accounted for ductile fracture, and was validated against experimental results and used to model 1st, 5th, and 7th-storey beam-column sub-assemblages of an 8-storey special moment-resisting frame building. The influence of a series of parameters on the connection catenary response was investigated, including out-of-plane pulling actions imposed by the transverse beam, a reduced beam section versus no reduction in beam flange, the yield stress to the ultimate stress ratio of steel, the beam web connection detail, and the reduction of ductility in the heat-affected zone. The results of the simulation confirmed that special moment frame connections seismically designed were, in catenary mode, capable of deforming. The out-of-plane pulling action that was induced by transverse beams was also shown to have no negative effect on the behaviour of the system. However, the rise in beam depth and that in the yield-to-ultimate-strength ratio adversely affected the ductility and strength of the connections. In addition, the beam-to-column detail was shown to have played a significant role in the connection response. Sub-assemblies with reduced beam sections were also shown to be of a higher strength and ductility compared with corresponding assemblies that had no reduced beam section (RBS). Nevertheless, the system behaviour did not markedly suffer from the heat- affected zone in beam flanges.

In 2009, Dawoon and Kim examined the catenary action in steel moment frames due to column removal. Nonlinear static and dynamic analyses were conducted on 3- and 6-storey structures with and without bracing without considering GSA regulation. By increasing number of spans, the responses are quite different with and without considering catenary action. The nonlinear dynamic analysis results indicated that the maximum displacement decreases by sudden removal of column.

In 2015, Kim et al. studied the effect of the presence of a floor slab on the behaviour of steel moment frames under progressive collapse. A simplified tensile force-displacement relationship from a frame with two spans was identified using a finite element method upon the removal of an external column. Then, by adding the slab energy absorption to balance the energy equation, a simple but precise nonlinear static analysis was suggested.

In addition, Yu et al. (2010) suggested a simplified model, based upon which they conducted a parametric study of progressive collapse. Although several studies have been conducted relating to progressive collapse in steel frames, such as the previous cases, most of these studies are based on steel frame 2D models and do not consider the ceilings' system share and other useful effects; for example, the share of ceiling slabs, compressive arch and catenary actions were not considered. Accordingly, they are irrelevant to real structural performance.

Moreover, in 2016 Pirmoz and Liu studied the progressive collapse in steel frame using post-tensioned cables and demonstrated that the beam arching action and strand catenary action are amongst the major factors of pre-stressed steel frame structural capacity against progressive collapse. They discovered that strand catenary action and beam arch action are responsible for the majority of the post-tensioned steel frame's capacity to withstand progressive collapse.

2.7.2.5 Studies of the behaviour of tall building against progressive collapse

Izzuddin et al. (2008) used a simplified model to conduct a nonlinear static analysis of tall buildings. In this study, four frame simplifying states were introduced. The deformation compatibility coefficient was used to connect simplifying model displacement and the main structure. The deficiency of this method is that such a compatibility coefficient cannot be easily calculated. The study was of a multilevel framework for the assessment of progressive collapse in structures that may experience sudden column removal. This framework benefits from practical applicability and channels the discourse on structure strength towards quantification. Both simplified and detailed models of nonlinear structural responses can be addressed with the proposed method; the two models can even be merged to develop a considerably more efficient representation of the level of structural idealization under study. There are three stages to this framework: (i) nonlinear static response determination, (ii) simplified dynamic assessment, and (iii) assessment of ductility. The conceptual clarity that the new framework provides has enabled us to see that previously popular indicators (ductility, energy absorption capacity, and redundancy) are unsuitable for measuring structural robustness. Nonetheless, the pseudo-static capacity of the system, which includes the previously mentioned indicators, has been recognized as a novel and logical measure of structural robustness in buildings subjected to sudden column removal.

Fu (2009) developed two three-dimensional models which represented two 20-storey steel frame buildings, which utilized shear walls and cross bracing to resist lateral loads, respectively, for the purpose of investigating the response of high-rise buildings under column-loss scenarios. The beams and columns were represented by beam elements and the slabs and core wall were modelled by shell elements. Both material nonlinearity and geometric nonlinearity were taken into consideration. In order to validate the proposed model, a two-storey frame model was created with the same modelling approach and the results were compared with experimental data. The author argued that the comparison was good and the proposed model was accurate enough to capture the responses of the structure under column-loss scenarios. However, the slab models used were not validated explicitly, neither was the brace model nor the shear wall model. Nonlinear analysis was performed using the two models and several column-loss cases were studied, applying APM. From the results, the author concluded that the dynamic response of the structure under column-loss scenarios was mainly related to the affected loading area, which means the larger the affected loading area was, the more the damage that could be induced. It is going to be shown in this research that this is not an accurate statement. Furthermore, the author also suggested that all the structural members, including beam to column connections, should be designed at least twice the static axial force subjected to the 1.0DL+0.25LL loading condition, although the number has been considered to be too conservative by many researchers. From the comparison of the results, the author also observed that a column removal at a higher level may induce larger vertical displacement than a column removal at ground level.

In addition, in 2010, Fu performed a numerical model for a 20-storey steel structure for progressive collapse analysis using ABAQUS finite element software. Using dynamic nonlinear analysis, he understood that the column in adjacent to the removed column shall be designed for an axial load equal to double of their designed load. Column removal in upper floors results in larger displacement, as lesser floors are involved in energy dissipation.

2.7.2.6 Studies of the effect of column removal in moment resisting frame with respect to progressive collapse

Li and El-Tawil (2014) numerically studied the behaviour of a building with steel moment frame system under progressive collapse. In this analysis the columns inside the frame and corner on first, middle and roof floors were removed. The study main focus was on composite action between concrete slab and steel beam and slab moment behaviour. The results indicated that the building is more vulnerable due to column removal on upper floors in comparison to removal of the same column on lower floors. Especially, when the frame middle column in upper floors is removed, this effect is more significant.

Szyniszewski (2009) treated survival probability of a building occupant as a measure of robustness of a 3-storey moment resisting steel framed building, which could be considered as one of the key factors to employ optimization algorithms so that the safest and most economical structural design could be obtained. It was assumed that the ratio of the collapse floor area to the total floor area was related directly to the probability of an occupant's survival. An approach to calculate the survival probability based on physics based simulations and a theorem of total probability was proposed. The calculation procedure was as follows:

1. the probability of localized damage of the structure under a truck bomb explosion scenario was assessed. The probability was calculated based on the area of "damage zones" around the building.

2. A three-dimensional model was created for the structure and a dynamic nonlinear analysis was performed under the column-loss scenario corresponding to the localized damage postulated before. The area of the

collapse floor was estimated and thus the corresponding survival probability of a building occupant could be calculated under that particular scenario.

3. Final probability of the occupant's survival was calculated by employing the total probability theorem. The research estimated the robustness of a moment resisting moment frame from the social point of view. However, the author did not provide any details about the three dimensional model and no calibration work was shown in this paper.

Hoffman and Fahnestock (2011) investigated the progressive collapse behaviour of typical multi-storey steel buildings with perimeter moment frames and composite steel-concrete floors using three-dimensional nonlinear finite element models. A three-storey and a ten-storey building were represented and a series of column loss cases were studied by performing nonlinear dynamic analysis. In the models, the beams and columns were modelled using shell elements. The steel deck and concrete slabs were modelled using individual planar layers of shell elements. Shear connections were represented with a component model consisting of nonlinear springs joining each bolt location.

Nonlinearities were taken into consideration in the modelling process. A series of conclusions were drawn from the studies: (1) after loss of the corner and perimeter columns with only shear connections, the structure could not survive progressive collapse, although the failure did not propagate; (2) composite flexural response was a significant load redistribution mechanism after column loss; (3) the concrete slab and concrete deck were subjected to inelastic demands as a results of flexural composite action; (4) demands were least severe for perimeter columns within a moment frame but the structure exhibited significant load redistribution for interior column loss scenarios that had no moment connections; (5) building height did not significantly affect progressive collapse of steel frames; and (6) the steel frames which were evaluated in this study demonstrated appreciable robustness. The main drawback of this study is lack of validation studies to validate the accuracy of the proposed models.

Rezvani et al. conducted a numerical study in (2015) of the impact of span length on steel moment frames subjected to progressive collapse. They performed multiple nonlinear static and dynamic analyses on three frames that had been designed for regions with high seismic activity. The results of the analyses showed that the beams and columns of the frames in the study were sufficiently strong upon the removal of columns on the first floor. However, to determine whether the frame would remain strong, a series of pushdown nonlinear static analyses were performed. The results revealed that halving the span lengths in these frames led to increases of 1.91 times. However, the findings of the nonlinear static analyses indicated that when the applied load increased, the structures under study had significant potential for failing upon the removal of the column on the first floor.

Meanwhile, in 2014, Elsanadedy et al. studied the explosion on the progressive collapse in a 6-storey building using moment frame system. The results indicated that through limiting the access distance to the surrounding columns, the failure potential due to explosion decreases and in order to decrease the damages the building structural system shall be reinforced using certain systems such as shear wall and diagonal bracings.

In 2008, Lee et al. conducted two nonlinear analyses to study steel moment resisting frames strength with welded connections against progressive collapse. They calculated beam rotation through simplifying load vs. displacement chart of the beam connected to the removed column. The responses directly depend on beam length (L) and depth (D). The load-displacement formula for three L/D ratios were calculated to be equal to 10, 15 and 20, and it was suggested that linear interpolation is applied for other ratios. Although the responses accuracy is proper, but no logical reason to use the linear interpolation was presented. On the other hand, the effect of adjacent beams and columns rotation resulting in structure further softening was not included in the modelling.

In 2010, Purasinghe et al. studied the progressive failure in a 3D steel building using SAP2000 software and three analysis methods: linear static, non-linear static and nonlinear dynamic. After conducting the analyses, it was concluded that in case the removed column is from moment frame side, then the progressive collapse potential is low and in case it is from structure hinged frame side then the progressive collapse potential depends on how much the beams are able to redistribute load.

The U.S.A. National Institute of Standards and Technology (NIST) has recently (November 2011) started a project on building resilience and structural robustness. The underlying stage includes testing of full-scale subsystems to approve detailed computer models (Sadek F. et al., 2010) and includes testing an individual beam under conditions that simulate column loss. The published experimental results are utilized herein for reference purposes and comparison with the Imperial College London simplified analysis results.

In 2010, Liu optimized the design of a steel moment frame in such a way that, in addition to lateral forces, it was also resistant to progressive collapse. In this study, he determined the progressive collapse potential using the alternative paths method and UFC guideline criteria, and used the nonlinear static analysis approach. Optimized designing was also conducted based on minimizing the steel weight. It was observed in this study that optimized designing based on the traditional method of steel weight minimizing, which fails to consider the progressive collapse issue, may not meet the UFC guideline criteria for progressive collapse. In order to examine the progressive collapse issue, Liu studied eight different states of column removal, while the optimized design was based on the linear static method, using the highest and heaviest designs to prevent the risk of progressive collapse, resulting in more cost-effective designs (which also represent the progressive collapse issue) using nonlinear static and dynamic analysis methods. Meanwhile, using the linear static method resulted in the most conservative design, which also meets the progressive collapse criteria in nonlinear static and dynamic methods.

Sadek and Millen (2011) studied the performance of a collection of steel beams and columns and reinforced concrete under central column uniform vertical displacement (column removal scenario simulation), both numerically and experimentally. Their studies are on certain parts of structural frames system designed as intermediate and special moment frames in C and D regions. Steel frames have been designed based on AISC 341-02 and qualified moment connections in FEMA 350 were used. Also concrete intermediate and special moment frames have been designed based on ACI 318-02 requirements. Beams and columns collection includes two spans and three columns. Vertical displacement is applied to the frame in central column until failure. Analysing using reduced models, saving the accuracy, acted quite fast and eased understanding the structural system total behaviour using such models.

Yousefi et al. (2014) studied the steel moment frames vulnerability under progressive collapse. In this study, external frame of a structure designed as per seismic regulations and in a region with quite high seismic risk was chosen. The results indicated that the columns designed in this frame do not have sufficient strength in keeping total frame system stability due to corner column removal. These structures did not meet the UFC acceptance criteria.

In 2009, Khandelwal et al. suggested and expanded numerical models to assess steel buildings strength against progressive collapse. They found out that there is a high level of strength among the frames especially for high seismic regions in comparison to the frames designed for the mid-range seismic conditions.

Using alternate path method (APM), Khandelwal and El-Tawil (2011) proposed "push-down analysis" as a way by which to measure of robustness. Push-down analysis was performed in two ways: (1) uniform pushdown, in which gravity loads on the entire damaged structure were increased proportionately until collapse was triggered; (2) bay pushdown, in which only the gravity loads in the bays which suffered damage were increased proportionately until the system collapsed.

Xu and Ellingwood (2011) proposed an energy based nonlinear static pushdown analysis to predict the dynamic peak response of the system through static analysis. The vulnerability of the structure could be assessed using this method. The studies surveyed above suggest that there are 4 general ways for measuring robustness: 1) Displacement-based method, in which robustness is describes as the overall deformation of the structure after loss of critical load-carrying members (APM); 2) Force based methods, in which robustness expressed as a ratio of the load carried by the 'damaged' structure to the nominal gravity loads; 3) energy based methods which are based on the Law of Conservation of Energy to assess the vulnerability of structures; 4) methods based on risk analysis, in which the probability of the performance of the structure reaching a certain limit state used as the identification of structural robustness.

Among the characteristics of steel construction for resisting progressive collapse is the provision of various alternative load paths after loss of a key member, the good ductility, over-strength and strain rate sensitivity properties (Kuhlmann et al., 2012), as well as satisfactory energy absorption capacity levels, especially in the case of composite construction.

Ferraioli et al. (2014) conducted pushdown nonlinear static analysis and studied the steel moment frames behaviour. Certain parameters such as number of floors, number of spans, and level of designed seismic load were considered in the analyses for the structure. Eventually, the dynamic load increase coefficient was applied for more precise and correct estimation in progressive collapse analysis.

In 2016 Gerasmidis and Sideri studied the corner column removal independent of column removal cause) in regular frames and the effects of number of floors in progressive collapse. The results showed that the column removal location in the floors shall represent different failure mechanisms. Column removal in lower floors results in yielding and failure in the upper floors columns of the member removal location and accident in the upper floors results in bending breaks of the beams.

Masjedian and Driver (2016) studied the behaviour of a one-span, one-storey steel frame with composite slab with removal of the corner column subject to the progressive collapse simulation. In the presented numerical model, various parameters effect such as composite slab thickness, slab reinforcement and loading direction on the failure mode and system instability were discussed. The results indicated that notwithstanding all the concrete cracks and crushes in composite slab, an increase in slab thickness results an increase in rotational and load bearing capacity of system, while the load bearing capacity fails to be improved by increasing the number or diameter of slab round bars.

Kim et al. (2009) investigated the progressive collapse resistance of steel moment frames by performing vertical static push-down analysis using planar macro-scale models. OpenSees software was used to carry out the analysis. 2-storey, 5-storey and 10-storey steel moment frame with 2-bays in both directions were used in this study and the effects of the number of stories and span length on progressive collapse resistance were investigated through push-down analysis. The analysis revealed that an increase in number of stories and decrease in span length lead to higher progressive collapse resistance. Furthermore, the effects of number of bays were also studies and it turned out that steel frames with larger number of bays were less vulnerable to progressive collapse. By comparing the load-displacement relationships obtained from static push-down analysis with those obtained by incremental nonlinear dynamic analyses, another conclusion was drawn that static push-down analysis might overestimate the inherent capacity of structures against progressive collapse.

Kaewkulchai and Williamson (2003) presented a beam element formulation and solution procedure for progressive collapse analysis of planar frame structures. This study shed light on the importance of dynamic load redistribution following the failure of one or more elements and both geometric and material nonlinearity were considered. The proposed nonlinear beam-column element utilized a lumped plasticity model with inelasticity concentrated at the element ends or hinges and a flexibility-based formulation within which the equilibrium of bending moment and axial force along the length of the element was satisfied by using force interpolation functions. The beam-column element also incorporated the interaction of axial force and bending moment and cyclic behaviour was captured by using multi-linear force-deformation relationships and the modified Mroz's hardening rule. Large deformation was considered through introduction of a geometric stiffness matrix. A damage index with a value varying from 0 to 1 was used to indicate member failure, through which the effects of stiffness and strength degradation at member hinges were also incorporated. When the value of the damage index of a hinge reached 1, the hinge was assumed to separate from the structure completely and failure happened. After the failure of an element, the stiffness matrix was updated using a modified member stiffness approach. To illustrate the importance of dynamic effects, the proposed element was used to model a two-bay, two-storey frame. Both static and dynamic analyses were carried out and it was concluded that a static analysis may not provide conservative estimates of the collapse potential of frame structures.

In 2012 Kheyroddin et al. studied the dynamic effect resulted from column removal on the steel frame. They suggested a simple and new method to calculate the dynamic load increase coefficient calculation resulting from sudden column removal. Their main study hypothesis was that the remained structure is able to transfer kinetic energy upon column removal. They suggested five main steps to achieve their purpose: first step, achieving nonlinear static response of remained structure using finite element method, second step, using deformability idea in maximum structure deformation to calculate increasing coefficient, third step designing a flowchart to determine dynamic increasing coefficient against a load ratio, fourth step achieving a series of equations based on static analysis to calculate dynamic load increase coefficient and fifth step including designing a simple graph using the results of previous steps.

Similarly, Meng-Hao Tsai (2012) proposes an analytical expression to replace the empirical formula used for calculating the load increase factor and dynamic increase factor for progressive collapse analysis. The analytical formula takes into account the post-yield stiffness of plastic hinges which gives more accurate and less conservative ductility requirements for weaker joints.

In 2005, Powel used a 2D model to compare the nonlinear static and dynamic analyses and concluded that the increasing coefficient load of 2 in static analyses results in conservative results and it is better to use nonlinear dynamic analysis for progressive collapse analysis.

One of the earliest examples of using 3D models is Ruth et al (2006). In this study, 3D models were used to investigate how conservative the dynamic increase factor provided by the DoD and GSA guidelines. The authors argued that the value of dynamic increase factor provided by both DoD and GSA, which was 2.0, was too conservative, and a dynamic multiplier of 1.5 would be more accurate. To determine a reasonable dynamic load factor, 11 models for steel moment frame structures were created, including 8 two-dimensional models and 3 three-dimensional models. In order to consider different factors which could affect the value of the dynamic factor, different building geometry parameters were used in these models, such as the number of stories, the number of horizontal bays, the bay dimensions, the member size, foundation constraints and the storey height. The author then proposed an approach to obtain a reasonable dynamic multiplier. Dynamic-nonlinear analysis and static-nonlinear analysis with different dynamic multipliers were performed with the proposed models, respectively. A ratio was obtained by dividing a static value by the corresponding dynamic value. Thus, when the ratio reached 1.0, the corresponding multiplier is the dynamic increase value. The results of total plastic rotation, the average plastic rotation and the maximum vertical displacement which were obtained from the two-dimensional and three-dimensional models were compared and plotted. The research revealed that a reasonable value of the dynamic multiplier was well below 2.0. The author also conducted the same analysis for a reinforced concrete frame, and it turned out the value could be even smaller. However, one of the main shortcomings of the proposed 3D models is that the slabs were not modelled thus the effects of the slabs were not accounted for.

Marjanishvili and Agnew (2006) compared two analysis approaches static nonlinear, and dynamic nonlinear by analysing a nine-storey steel moment-resistant frame building with a loss of one primary column. Their recommendations include using the nonlinear static procedure to supplement the nonlinear dynamic in determining the first yield and ultimate capacity limits, as well as in verifying and validating dynamic analysis results.

In addition, Gerasimidis and Baniotopoulos (2014) studied the impact of various strengthening techniques for reducing progressive collapse in 2D steel moment frames.

Almost all modern studies consider sudden column loss as a standard approach for evaluating the structure's resistance to progressive collapse. Although the majority of studies examine the behaviour of buildings, a few exceptions focus on isolated structural systems, like the response of cable-stayed steel roofs following sudden cable loss (Gerasimidis and Baniotopoulos, 2011).

2.7.2.7 Studies the effect of geometric irregularity in relation to progressive collapse

A parametric study considering irregular steel frames subject to vertical geometric irregularity was reported in Gerasimidis et al. (2012). They conducted a thorough parametric examination of the response of irregular steel frames in the case of initial damage in terms of the total number of columns removed, one by one. In terms of morphology, the frames had vertical geometrical irregularities, which influenced their resistance to disproportionate collapse.

Homaioon Ebrahimi et al. (2017) examine the effect of plan irregularities on the progressive collapse of four steel structures located in regions with different seismic activity. “In the non-linear static analyses, the pushdown curve and yield load factor of the structures are obtained after column removal. The results indicate that an irregular structure designed in site class C seismic zone, collapses in most of the column removal scenarios. Moreover, when comparing regular and irregular structures designed in site class E seismic zone, the demand force to capacity ratio (D/C) of the columns in the irregular structures is on average between 1.5 and 2 times that of the regular ones. In another investigation, they utilized four parameters to assess structural performance against progressive collapse, namely push-down capacity curves, yield load factors, joint displacement of the top joint of removed columns, and demand to capacity ratio (D/C). The analyses showed that those structures located in site class E seismic risk exhibit lower D/C ratio than those located in site class C and it became evident that bracing tends to increase structural resilience hence reducing the potential of progressive collapse of structures.”

In addition, Homaioon Ebrahimi et al. (2017) examines the effect of plan irregularities on the progressive collapse of braced and un-braced steel structures located in regions with different seismic activity and designed in accordance with AISC (2010) and ASCE7 (2010). “The collapse patterns of four buildings is examined and compared across seven loading scenarios using non-linear dynamic and static analyses completed as per the GSA (2013) standard. Node displacements above the removed columns and the force increase across adjacent columns, are discussed. Also, the susceptibility of columns to collapse, based on their strength and capacity, is examined and the pushdown curve and yield load factor of the structures, after column removal, is obtained and critically discussed.”

2.8 Conclusion

The literature review presented in this study shows that extensive efforts have been made to examine the behavior of structures experiencing progressive collapse. This article examined: various definitions of progressive collapse; the importance of considering progressive collapse when designing special and significant structures; instances of human and financial casualties due to progressive collapse; and the approach of various guidelines and standards to this phenomenon. As assessed in the studies’ records, the amount of experimental work conducted on the issue of progressive collapse is quite limited. On the other hand, because of experimental problems and facilities, the most suitable, reasonable and fastest method to study the behaviour of structures due to such an effect is the numerical approach. Accordingly, focusing on this method is a proper option for the better assessment of structures to determine their behaviour due to various column removal scenarios or key elements. Meanwhile, the lack of studies concerning the behaviour of structures with geometrical irregularities due to progressive collapse is obvious. Given the further vulnerability of such structures when faced with the loss of structural load-bearing elements, a comprehensive study in this regard is required.

CHAPTER 3: ANALYSIS AND DESIGN PROCEDURES IN GSA

3.1. Introduction

The preceding chapter has presented approaches to and methods of preventing the risk of progressive collapse. The findings of researchers were also mentioned. It is necessary to understand the behaviour of structures experiencing progressive collapse to be able to offer methods for their design. Therefore, in this chapter, different types of collapse are first introduced; then, methods of analysis and the design of structures will each be discussed.

3.2. Various types of progressive collapse

3.2.1. Pancake-type collapse

The most typical example of this type of collapse is what happened in the World Trade Center. The impact of the planes and the fire that was the result caused local damage to the impact site, which led to a reduction in the vertical load-bearing capacity. After this reduction, the excess load was divided between intact load-bearing sections of the tower. During the collapse, the upper floors started to fall and the kinetic energy increased cumulatively. The impact of collapsed floors, which had increasing kinetic energy, on to intact floors meant that there were more “beyond-resistance” forces increasing the collapse and so the kinetic energy. The collapse continued in this way and ultimately destroyed the entire structure (Starossek, 2009).

Figure 3.1 The Collapse of the World Trade Centre (Progressive collapse of structures, by Uwe Starossek, Thomas Telford Publishing, 2009, 168 pp.)

This type of collapse first appears as the collapse of the primary vertical load-bearing elements, or the total loss of the load-bearing capacity of vertical elements, and the partial or total detachment of the elements and their vertical fall as a rigid body motion. Then, with the conversion of the potential energy to kinetic energy and the impact of detached and collapsed members on the rest of the structure, the load-bearing capacity of other elements is lost due to the impact and collapse that progresses vertically.

3.2.2. Zipper-type collapse

According to design regulations, suspended bridges must not lose their stability upon the loss of one cable. In such cases, the failure of one cable and the resulting excess load on the adjacent cables must not lead to the tear of these cables. If the design is defective, this leads to the tear of other cables and progressive collapse. (Starossek, 2009)

Figure 3.2 Collapse due to cable failure and increased load on adjacent cables (Starossek, 2009)

This type of collapse is characterized by the initial failure of one or a small number of structural elements, redistribution of forces, impact loading due to the initial abrupt tear, combination of static and dynamic loads in the elements adjacent to the damaged element, distribution of loads and progressive collapse.

3.2.3. Domino-type collapse

The type of collapse starts with the initial damage of one element; then, the fall of the element as a solid object in angular motion around one corner of the object converts potential energy to kinetic energy. The impact of the upper corner of the element on to an adjacent element combines static and dynamic forces. This is finally accompanied by the progress of collapse in the direction of damage.

Structure with improper lateral harnessing experience this type of collapse. Collapse in scaffoldings and the columns of water pipes can have this type of collapse. (Starossek, 2009)

3.2.4. Section-type collapse

Consider a beam under bending or a rebar under tensile axial force. When part of the section is cut off, the force is divided in the rest of the section. Thus, the increased section force can lead to the rupture of the section. This type of collapse is quite similar to zipper-type collapse. According to the fracture mechanics, the stress around the crack increase infinitely with the decrease in distances. Naturally, in a structural system, the elements are far from each other and their distances are not zero. Nonetheless, there is focused force in the elements adjacent to the failed element. (Starossek, 2009)

3.2.5. Instability-type collapse

Normally, structures are designed in such a way as not to experience instability. However, the fracture of bracing elements can lead to structural instability and destruction. This can happen in the trusses or beams of structures where bracing elements, which are used to create stability in pressure elements, fail. Another example is failure in the stiffeners of beams, which leads to local instability and, ultimately, total collapse. In this case, a small event leads to extensive damage. The buckling of the column can also cause the total instability of the structure and, consequently, total collapse.

This type of collapse starts with the initial failure in the bracing members of the compressive load-bearing elements and the instability of the compressive member. Finally, with the sudden failure of this member under the slightest changes, the collapse progresses. (Starossek, 2009)

Figure 3.3 Instability-Type Collapse (Starossek, 2009)

3.2.6. Mixed-type collapse

Progressive collapses that have happened so far do not completely fall into any of the categories above. In many cases, the collapse is a combination of several types. For instance, Figure 3.4. shows a combination of pancake-type and domino-type collapses. (Starossek, 2009)

Figure 3.4 Combination of pancake-type and domino-type collapses (Starossek, 2009)

3.3 Failure modes

Steel moment-resisting frame structures have similar structural characteristics to truss structures, except trusses do not resist moments in the members. Each steel member can have different failure modes because it will be subject to different loads, cross sectional shapes, and material property combinations. Failure modes can be categorized as follows (Salmon and Johnson 1996):

- **Material failure**

a. Ductile failure

Plastic deformation failure.

b. Brittle failure

i. Fracture failure.

ii. Fatigue failure.

- **Buckling**

a. Flexural column buckling, (i.e., global buckling).

b. Torsional buckling.

c. Lateral-torsional buckling.

d. Local buckling.

Material failures are mainly expected in tension. Steel members show ductile behaviour before failure in an ideal condition such as a uniaxial tension test. Nevertheless, steel members can fail in a brittle manner in a practical case due to an initial flaw or notch. Steel members can lose stiffness in a high temperature induced by fire. Repeated loading and unloading may eventually result in a fatigue failure, even if the yield stress is never exceeded. If a steel member has relatively low torsional stiffness, it may buckle torsionally whilst the longitudinal axis remains straight. Steel beams can also buckle under bending without a proper lateral restraint, because the flange can be considered as a column when it is subject to compression by bending. Components such as flanges, webs, and angles, which are combined to form a column section, may themselves buckle locally prior to the entire section achieving its maximum capacity. The reasons for progressive collapse may be several, as follows (Christiansson 1982).

“1. Material failure due to high stresses.

2. Failure as a result of the inability of the structure to sustain the large deformations.

3. Stability failure of the entire structure.

4. Local stability failure in the form of buckling, etc.”

Christiansson (1982) stated that there is no major distinction between cases 1 and 2. Case 1 applies mainly to a brittle material, while case 2 may occur when the deformation capacity of the material has been exhausted. In the same way, there is no real distinction between cases 3 and 4. Therefore, it should be reconsidered only two categories, material and buckling failures. There is also another important failure mechanism, which is the failure of the joint connections.

3.4 Location of removed load-bearing elements

3.4.1 External columns

Remove external columns near the middle of the short side, near the middle of the long side, at the corner of the building, and adjacent to the corner of the building as shown in Figure 3.5.

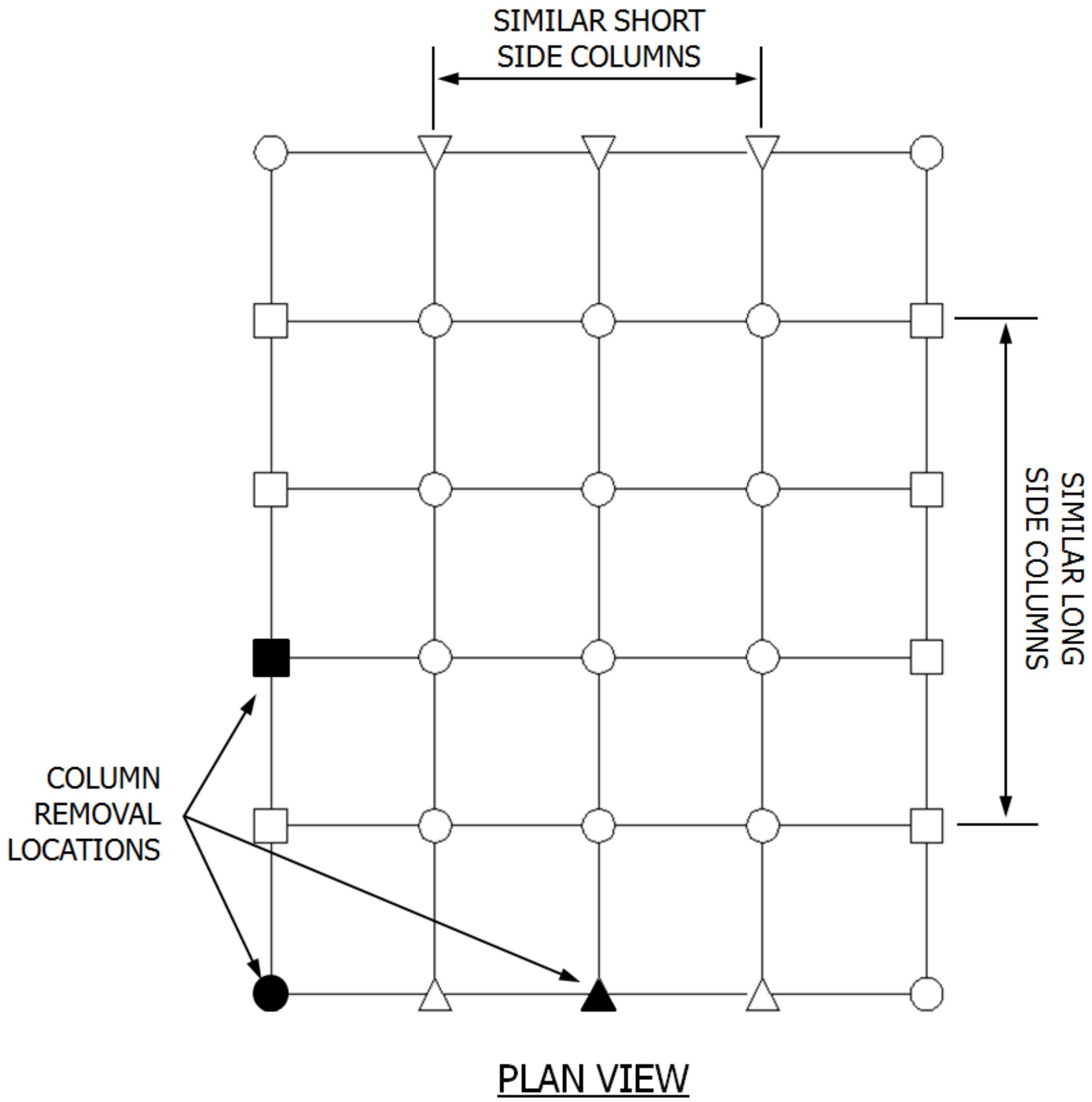


Figure 3.5. Location of External Column Removal (GSA 2013)

3.4.2 Internal columns

For structures with underground parking or areas of uncontrolled public access, remove internal columns near the middle of the short side, near the middle of the long side and at the corner of the uncontrolled space, as shown in Figure 3.6. (GSA 2013)

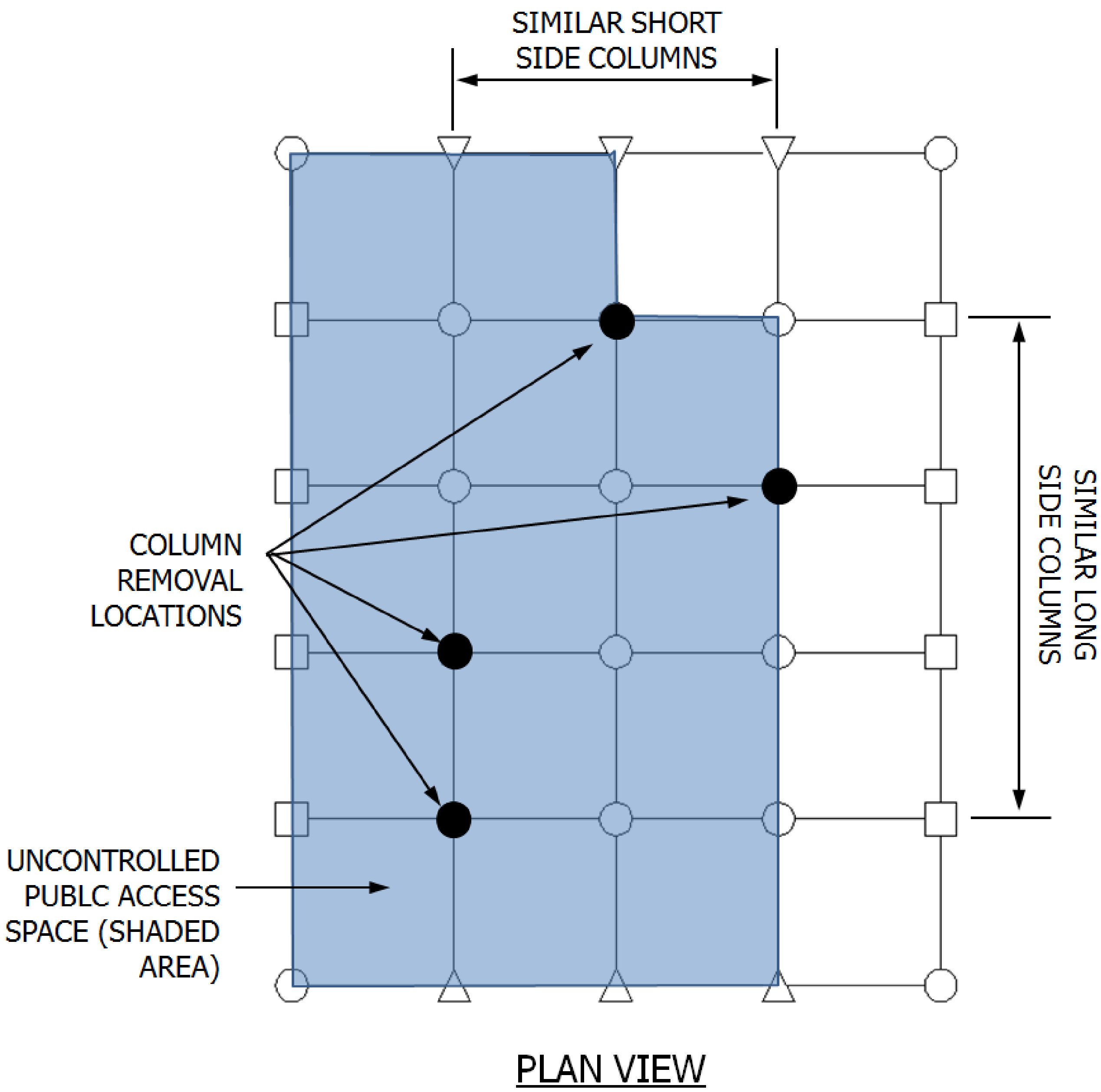


Figure 3.6. Location of Internal Column Removal (GSA 2013)

3.5. Different types of progressive collapse analysis

The alternate path method recommended in GSA guideline is employed to examine the capacity of the resistance of steel frames against progressive collapse. Among the various ways of designing structures to withstand progressive collapse, the most common way in such guides use the alternate load path. One of the advantages of this method is its simplicity and clarity. In most cases, because of the redundancy and the existence of alternate load paths, the structure can bear the load even upon the removal of one column.

To analyse progressive collapse, design guidelines have recommended following the most important analyses according to the protection needed for the structure and the accuracy required:

- Nonlinear static analysis (Pushdown)
- Nonlinear dynamic analysis

Material plasticity is included in non-linear methods, enabling them to explain geometric non-linear effects as they become more significant. Non-linear methods are also capable of enabling the development of alternative load path mechanisms (Byfield et al. (2007)).

3.5.1. Nonlinear dynamic analysis

Progressive collapse is an essentially dynamic event; instantaneous loss of a column releases significant internal energy that disturbs the initial load equilibrium of external loads and internal forces, which need to be absorbed by the ductile members of the remaining structure, mainly the connections, in order for the structure to reach a new equilibrium position, otherwise it collapses. For this reason, nonlinear dynamic analysis procedures, although being the most complex and resource demanding, are the most accurate since they inherently incorporate dynamic amplification factors, inertia, and damping forces.

Following the GSA (2013) guidelines, load combinations including 120% of dead load plus 50% of the total live load are gradually applied within a time frame of 5 s. Then, and in order to account for nonlinear dynamic effects, the load is maintained steady for the following 2 s. After the 7 s sequence, when gravity load effects are considered to be fully transferred to the structure, a pre-selected column is suddenly removed from the model and the structural response is examined.

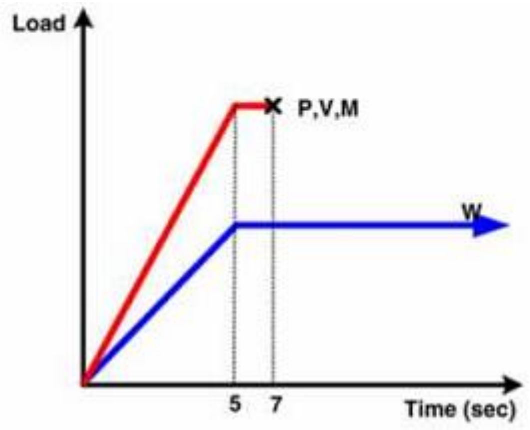


Figure3.7 Applied load in dynamic analysis (GSA 2013)

3.5.2. Nonlinear static analysis

In parallel, nonlinear static analyses performed, following the GSA (2013) recommendation for using a dynamic amplification factor of ~2. That, in order to reflect a ratio of 2 between the load that is applied to the spans that are adjacent to the removed column with respect to that applied on other spans. In this case, vertical loading is applied by following a step-wise increase until the maximum amplified loads are attained or the structure collapses. This vertical pushover analysis procedure, which is often called the 'pushdown analysis method', accounts for nonlinear effects which approximate the nonlinear dynamic response whilst providing a reliable estimation of the elastic and failure limits of the subject structure.

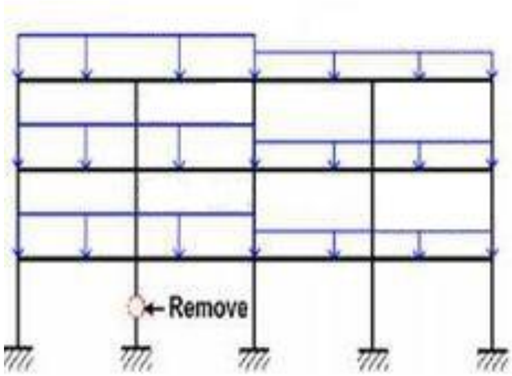


Figure 3.8 Applied load in push down analysis (GSA 2013)

Derived from the nonlinear static analyses, the effective imposed load plotted against the node displacement of the removed column indicates the capacity of a structure against progressive collapse. If the load value is divided by the standard gravity load, the vertical axes of the pushdown capacity curve are converted into dimensionless load factors, as in Equation. (3-1). This standardises the load ratio and makes it easier to establish generic observations. The load factor calculated in this way have thus been used herein as a criterion for assessing structural collapse. Namely, if the load factor corresponding to the displacement causing material yield is higher than 1, the structure can withstand the removal of a column, otherwise the structure will collapse.

$$Load\ Factor = \frac{Load}{Nominal\ gravity\ load} \tag{3-1}$$

CHAPTER 4: METHODOLOGY

4.1. Introduction

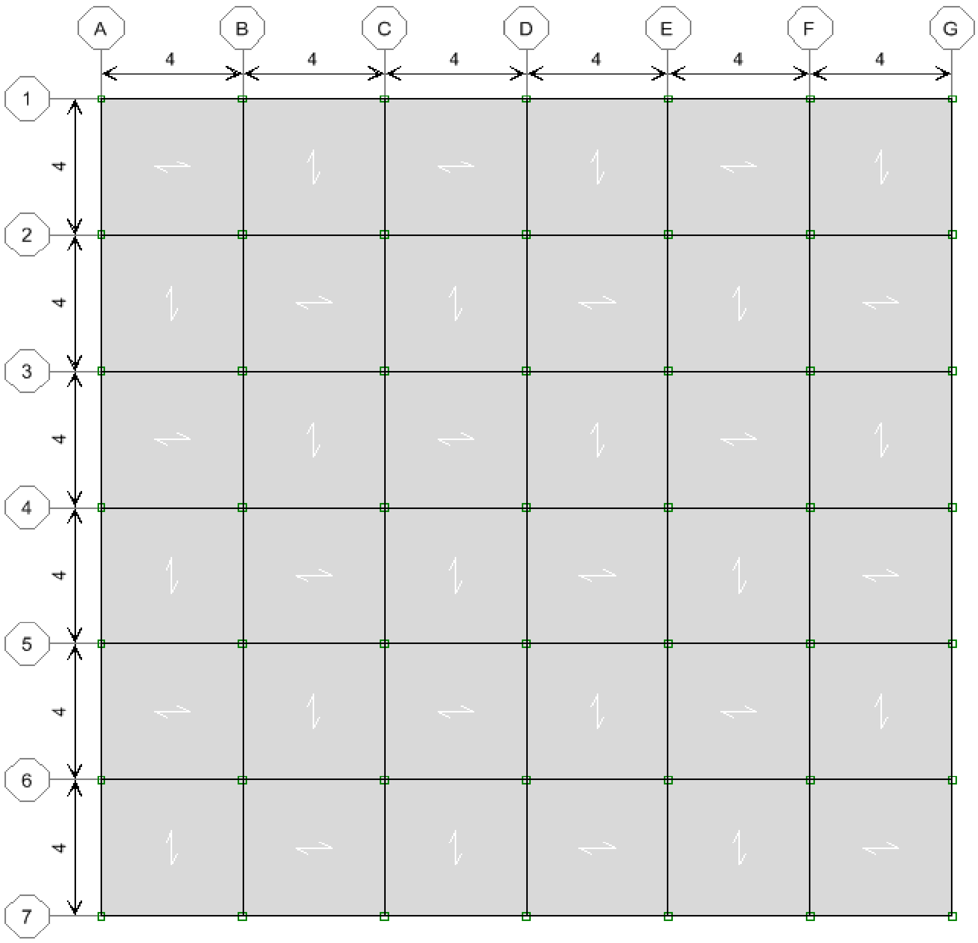
This chapter will first of all introduce the discussed structures. Then the elements and sizes of the used sections are introduced in compliance with the relevant guideline, seismic and gravity loadings. The structures are designed in three different heights and two regular and irregular states. Finally, a comparison among member sections under regular and irregular states is conducted for the structure with different heights.

4.2. Introduction of structural models

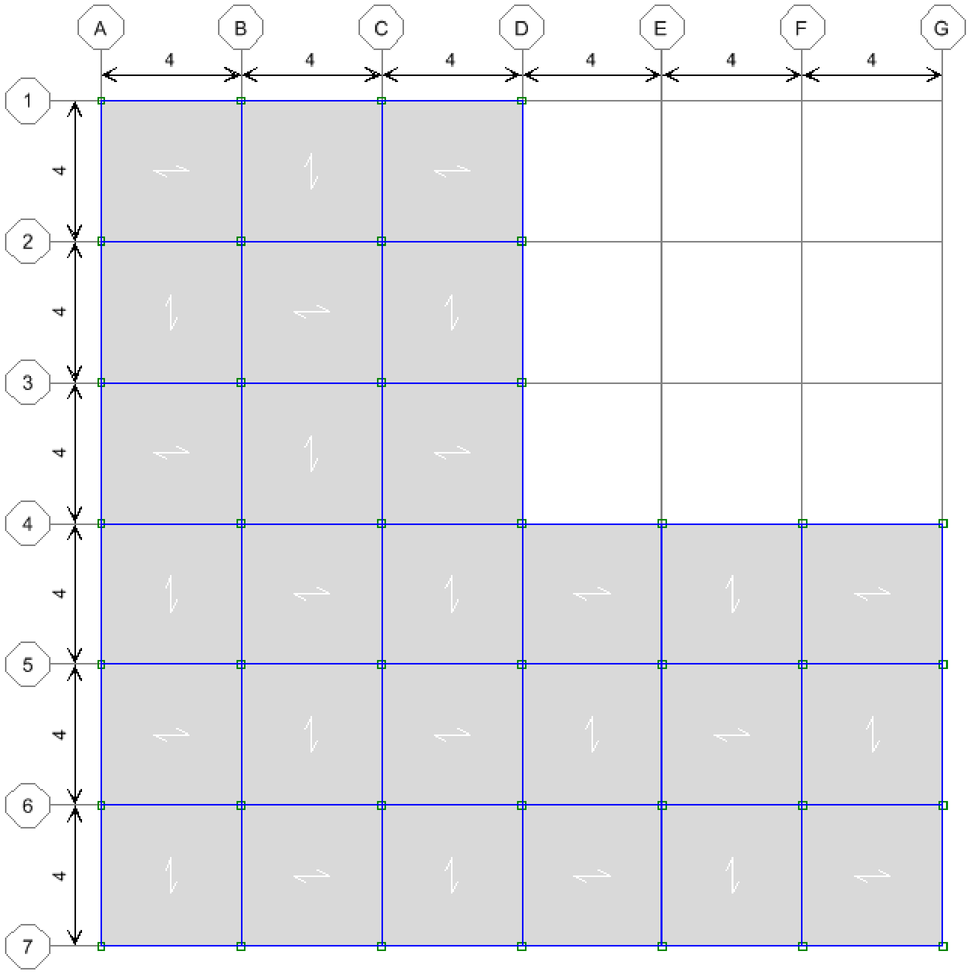
4.2.1. Introduction

One of the principal factors amplifying the damage caused during progressive collapse is structural irregularity. In fact, past instances have shown that irregularly configured buildings or those in which structural properties are distributed asymmetrically have an increased seismic demand, which amplifies the damage (Mohod, 2015). While there are many types of building configuration and they may have multiple sources, they can be broadly classified into two categories: those in plan, and those in elevation. Plan irregularities stem from plan asymmetrical mass, stiffness and strength distribution, which lead to a significant rise in torsional effects when lateral forces affect the structure. Elevation irregularities result from differences in geometrical and structural properties in the elevation of a building, which generally increase the seismic demand in certain storeys. Both plan and elevation irregularities necessitate the development of collapse mechanisms as a result of a local rise in the seismic demand in certain elements, for which sufficient strength and ductility (ASCE7, 2010) are not always provided.

Three 2, 3 and 5-storey steel structures with regular and irregular plan were selected for the present investigation. Intermediate Steel Moment Resistant frames were pre-designed with ETABS software according to the AISC (2010) and ASCE (2010) to study progressive collapse scenarios in structures. Each floor height and span width is 3 and 4 meters, respectively. In regular structures, there are six spans in both horizontal directions (total plan width: 24m). Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction (ASCE 7 (2010)). The level of indentation in these structures is equal to 50% of the plan length in the same direction. In Fig. 4.1 the plan status and sizes in both regular and irregular structures have been shown.



A: Regular plan



B: Irregular plan

Figure 4.1 Regular and irregular structures plan

4.2.2. Loading

The gravity load was applied to the floors in the form of two types dead load and live load. The dead and live loads in all floors except roof were applied as 5.1 KN/m^2 and 1.88 KN/m^2, respectively. The live load on the roof is 1.47 KN/m^2. Meanwhile, uniformly distributed linear load resulted from wall weight on the building peripheral beams was considered as 7.35 KN/m. In Fig. 4.2 the status of perimeter walls load has been shown.

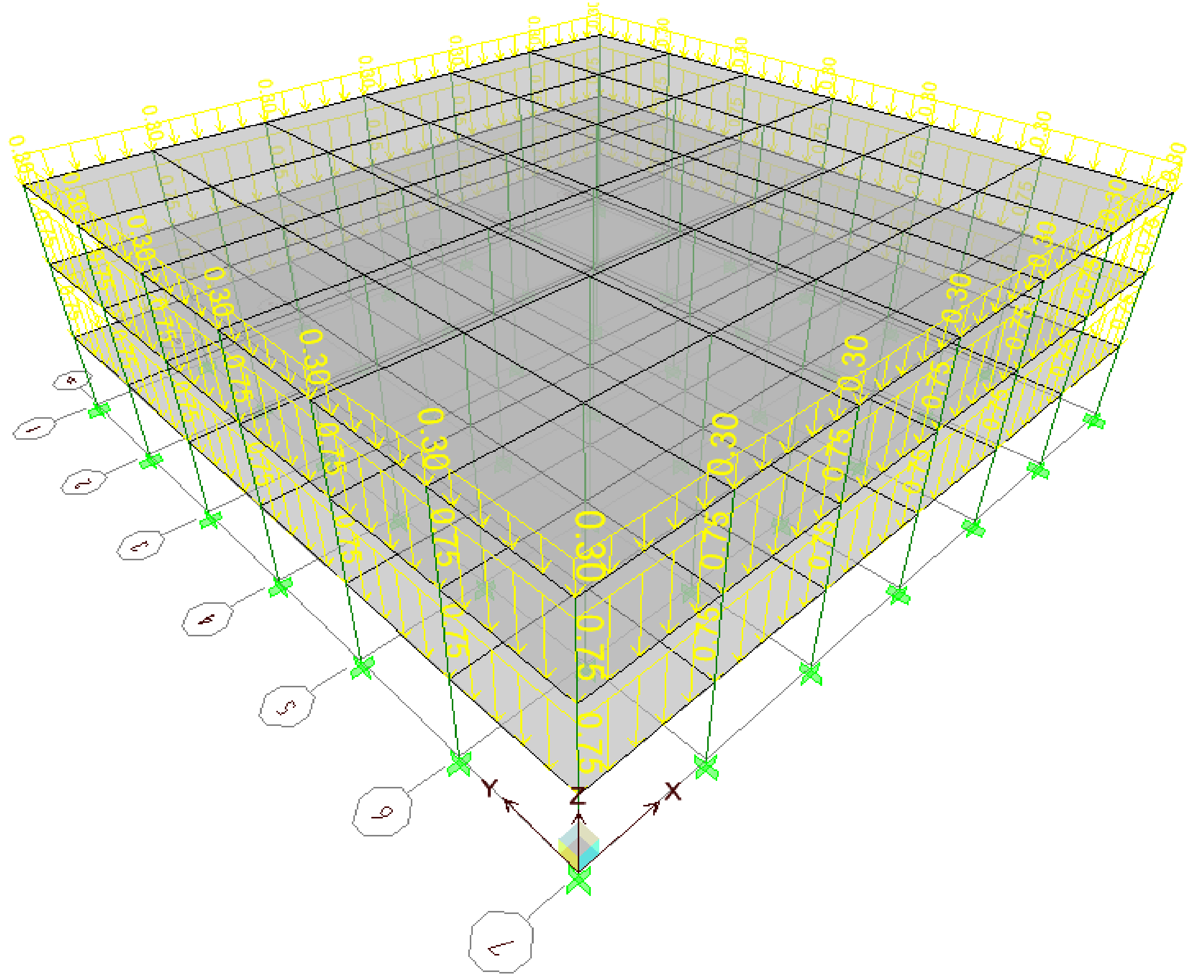


Figure 4.2 Applied uniformly distributed linear load on perimeter walls

As seen in Fig. 4.2, the load applied to the roof walls has been considered less due to lack of certain walls unlike the lower floors. 2.94 KN/m represents the parapet load. The supporting conditions of structure model are in a way that the columns base connection has been considered as fully fixed and the effects of soil-structure interaction have been ignored.

ASCE 7(2010) guideline seismic regulations have been used in seismic loading. The first step in seismic loading is to determine the seismic coefficient using the following method as per ASCE 7(2010) guideline seismic regulations.

4.2.3 Calculation of Seismic Response Coefficient in ASCE7-10

The seismic response coefficient, C_s, shall be determined in accordance with Equation (4-1):

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \tag{4-1}$$

Where:

S_{DS} = the design spectral response acceleration parameter in the short period range

R = the response modification factor

I_e = the importance factor determined

The value of C_s computed in accordance with Equation (4-1) need not exceed the following:

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \qquad T<T_L$$

$$C_S = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \qquad T>T_L \tag{4-2}$$

Cs shall not be less than:

$$C_S = 0.044 S_{DS} I_e \geq 0.01$$

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s

T = the fundamental period of the structure(s)

T_L = long-period transition period(s)

4.2.4 Period Determination

The fundamental period of the structure, T, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The approximate fundamental period (T_a), in s, shall be determined from the following equation (4-3):

$$T_a = C_t h_n^x \tag{4-3}$$

where h_n is the structural height and the coefficients C_t and x are determined from Table (4-1).

Table 4.1 Values of Approximate Period Parameters *Ct* and *x* (ASCE7-10)

	C_t	x
Steel moment-resisting frames	0.0724	0.8

The periods for 2, 3 and 5 storey structures shall be 0.304, 0.420 and 0.632 seconds, respectively. Importance factor by risk category of buildings and other structures for earthquake loads is 1.0. Meanwhile, the response modification factor using intermediate moment resistant frame system shall be equal to 4.5.

4.2.5 Modal response spectrum analysis

An analysis was conducted to determine the natural modes of vibration for the structure. The analysis includes a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model (ASCE7 (2010)).

The value for each force-related design parameter of interest, including storey drifts, support forces, and individual member forces for each mode of response shall be computed using the properties of each mode.

The value for each parameter of interest calculated for the various modes shall be combined using the complete quadratic combination (CQC) method, the complete quadratic combination method as modified by ASCE (CQC), or an approved equivalent approach. (ASCE7 (2010))

4.2.6 Scaling design values of combined response

A base shear (V) shall be calculated in each direction using the calculated fundamental period of the structure T in each direction (ASCE7 (2010)).

Where the combined response for the modal base shear (V_t) is less than 85 percent of the calculated base shear (V) using the equivalent lateral force procedure, the forces shall be multiplied by $0.85\frac{V}{V_t}$

V = the equivalent lateral force procedure base shear

V_t = the base shear from the required modal combination

The specifications of the members' steel materials and ceiling concrete are as per the following. The material properties features of steel materials have been given in Fig. 4.3 and Table (4.2). The concrete used in this thesis has a density of 2300 Kg/m^3 and an elasticity module of 23.3 GPa. The strain along with the maximum stress for regular concrete is equal to 2225 micro-strain, which is a normal value. The Poisson Ratio for concrete with regular strength has also been considered as 0.15.

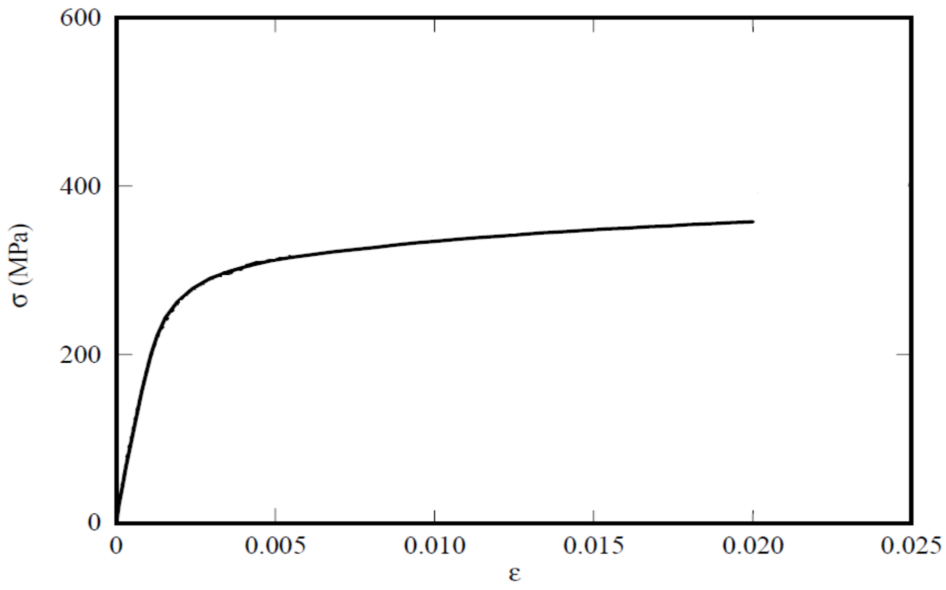


Figure 4.3 Steel stress-strain diagram with yield stress of 240 MPa, used in the studied buildings (Talja and Salmi (1995)).

Table 4.2 Specifications of the steel used in the study (Talja and Salmi (1995)).

Rupture strain (με)	Yield strain (με)	Elasticity module (MPa)	Ultimate stress (MPa)	Yield stress (MPa)	Density (kg/m^3)
2000000	1187	202100	370	240	7850

After analysis, the member sections have been selected considering tables 4.4 through 4.15. Evidently, each regular or irregular structure has been analysed and designed through being on two different seismic risks. Such type regional conditions were chosen in compliance to the regulations given in ASCE (2010) guideline of C and E regions. The column section is rectangular and beam section is as “H” or IPE type. Also a concrete ceiling slab of thickness: 100 mm has been considered.

In this thesis, two seismic design zones (C and E) were used for designing the structures and also they situated on the stiff and soft soils.

Table 4.3 Geometrical characteristics and seismic zones of structures.

Structure	Seismic zone	Type of Soil	Regularity	Number of storeys
Structure 1	C	very dense soil and soft rock	Irregular	2,3,5
Structure 2	E	soft clay soil	Irregular	2,3,5
Structure 3	C	very dense soil and soft rock	Regular	2,3,5
Structure 4	E	soft clay soil	Regular	2,3,5

Table 4.4 Specifications of sections sizes in irregular and regular 2-storey structure situated in different seismic region

		Regular-Seismic region C		Regular-Seismic region E		Irregular-Seismic region C		Irregular-Seismic region E	
		1^{st}	2^{nd}	1^{st}	2^{nd}	1^{st}	2^{nd}	1^{st}	2^{nd}
Column	b	200 180	180	200 180	180	200 180	180	200 200	180
	t	10 8	8	10 10	8	12 10	8	12 10	10
Beam	b_f	150	140	150	140	150	140	150	150
	t_f	8	8	8	8	8	8	8	8
	b_w	250	250	250	250	250	250	250	250
	t_w	8	6	8	6	8	6	8	8

Table 4.5 Specifications of sections sizes in regular and irregular 3-storey structure situated in different seismic region

		Regular-Seismic region C			Regular-Seismic region E			Irregular-Seismic region C			Irregular-Seismic region E		
		1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
Column	b	200 200	180	180	200 200	180	180	200	200	180	200 200	200	200
	t	12 10	8	8	12 10	10	8	12	10	8	15 12	12	10
Beam	b_f	150	140	140	150 150	140	140	150	150	140	150	150	150
	t_f	8	8	8	10 8	8	8	8	8	8	12 10 8	8	8
	b_w	250	250	250	250	250	250	250	250	250	250	250	250
	t_w	8	6	6	8	6	6	8	8	6	8	8	8

Table 4.6 Specifications of sections sizes in regular 5-storey structure situated in C region

Floor	Column		Beam			
	b	t	b_f	t_f	b_w	t_w
1st	200	12	150	8	250	8
	200	10				
2nd, 3rd, 4th, 5th	200	10	150	8	250	8

Table 4.7 Specifications of sections sizes in regular 5-storey structure situated in E region

Floor	Column		Beam			
	b	t	b_f	t_f	b_w	t_w
1st	200	15	150	12	250	8
	200	12	150	10	250	8
	200	10	150	10	250	8
2nd	200	15	150	12	250	8
	200	12	150	10	250	8
3rd	200	12	150	10	250	8
	200	10	150	8	250	8
4th	200	10	150	8	250	8
5th	200	10	150	8	250	8

Table 4.8 Specifications of sections sizes in irregular 5-storey structure situated in C region

Floor	Column		Beam			
	b	t	b_f	t_f	b_w	t_w
1st	200	12	150	8	250	8
2nd	200	10	150	8	250	8
3rd	200	10	150	8	250	8
4th	200	10	150	8	250	8
5th	200	10	150	8	250	8

Table 4.9 Specifications of sections sizes in irregular 5-storey structure situated in E region

Floor	Column		Beam			
	b	t	b_f	t_f	b_w	t_w
1st	200	20	150	15	250	8
	200	15	150	12	250	8
	200	15	150	10	250	8
2nd	200	15	150	15	250	8
	200	15	150	12	250	8
	200	15	150	10	250	8
3rd	200	12	150	12	250	8
	200	12	150	10	250	8
	200	12	150	8	250	8
4th	200	10	150	8	250	8
5th	200	10	150	8	250	8

Also in Tables (4.10) and (4.11), different details of irregular 5-storey equipped with concentric braces presented.

Table 4.10 Irregular structure 3 with braced frame

Floor	Column		Beam				Brace section
	b	t	b_f	t_f	b_w	t_w	$b \times t$
1st	200	12	150	10	240	8	160×6
	200	12					
2nd	200	12	150	10	240	8	140×6
3rd	200	12	150	10	200	8	120×6
4th	200	12	150	10	180	6	100×6
5th	200	12	150	10	160	6	80×6

Table 4.11 Irregular Structure 4 with brace frame

Floor	Column		Beam				Brace section
	b	t	$b \times t$	t_f	b_w	t_w	$b \times t$
1^{st}	200	15	150	10	240	8	140×8
	200	12					
2^{nd}	200	15 12	150	10	240	8	120×8
3^{rd}	200	12	150	10	240	8	100×6
4^{th}	200	12	150	10	240	8	100×6
5^{th}	200	12	150	10	240	8	80×6

In Figure 4.4, the arrangement of bracing in irregularity plan illustrated.

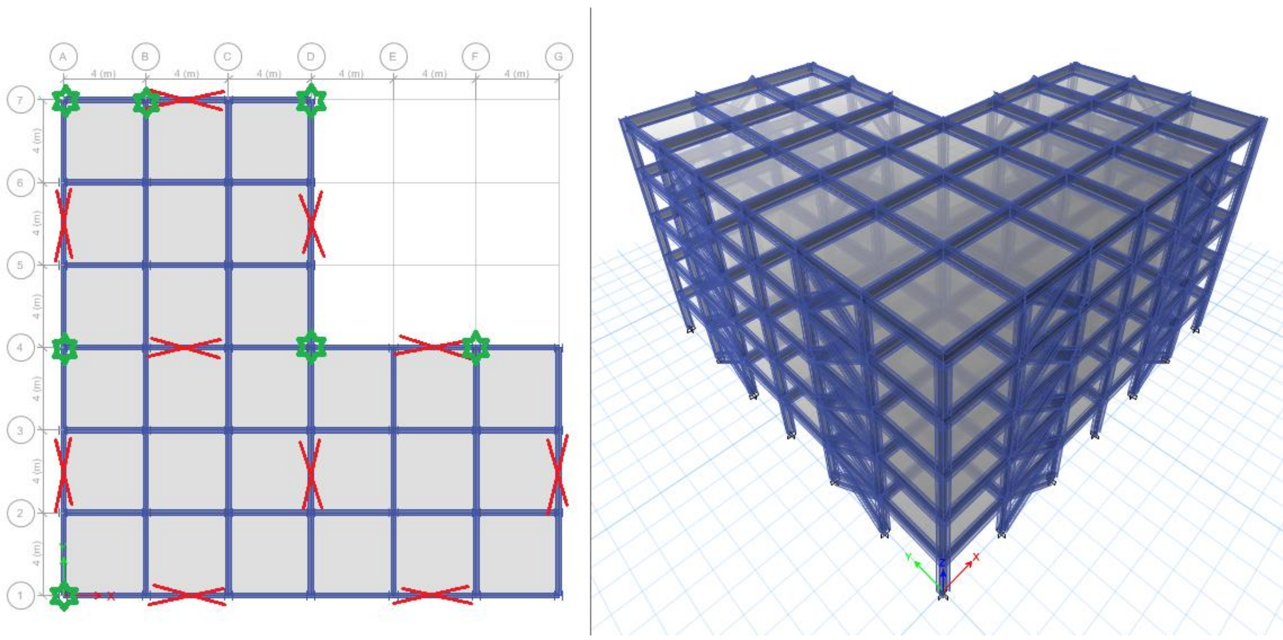


Figure 4.4 Arrangement of bracing system in irregular 5-storey building

4.3 OpenSees Software

4.3.1 Introduction

The FE analyses presented in this dissertation were performed using the Open System for Earthquake Engineering Simulations (OpenSees), which is an open-source, object oriented software framework developed by the Pacific Earthquake Engineering Research (PEER) Center. OpenSees allows users to create finite element applications to simulate the response of structural and geotechnical systems subjected to variety of loading conditions (http://opensees.berkeley.edu). It consists of a set of modules to perform the creation of the FE model, specification of an analysis procedure, selection of responses to be monitored during the analysis, and the output of the results. OpenSees has the additional benefit of containing a large library of linear and nonlinear geotechnical and structural materials that facilitate the realistic simulation of engineering problems.

Various structures have infinite degrees of freedom upon applying a dynamic load. The finite element methods convert such system with infinite degrees of freedom into a model with finite degrees of freedom representing similar physical behaviour. In order to solve the equilibrium and compatibility equations, apart from classical equations methods, the finite elements methods may also be applied, in which, for instance, in solving a frame, instead of solving the entire element points, the element is divided into separated points and the results are interpolated through form functions for the points among the same. OpenSees Software is able to conduct a variety of analyses within the short period of time and using certain tools to achieve a better convergence.

OpenSees Software uses finite elements method to analyze various types of structures. TCL programming languages is applied by the users for the said software, which is able to develop and generate various types of structural and non-structural sections regarding analysis and the relevant user may use such method to enter into the core of the software system and may also define model geometry and loading used materials and analysis method through texting. This software has been developed and distributed by NEES research unit members, affiliated to Berkeley University, USA, in which regard no company is involved. TCL programming language is composed of a series of fields, for which the following particulars may be defined:

. Variables and replacing their values

. Conducting calculations and reporting the same

. Using conditional relationships (for each, if, while, for)

. Analysis methods

. Making the file manually through typing the same

Finally, in order to use TCL productively, it should be made convergent with the finite elements method, while the following categorization shall be achieved after such convergence:

. Modelling: defining nodes, elements, loading

. Analysis: identifying analysis method

. Output specification: choosing outputs to be displayed after analysis operations.

There is no certain unit in OpenSees Software to define the numbers. It is determined the unit as a certain value from the very beginning and enter the figures in terms of the optional unit and all the results will then be based on the output given units.

4.3.2 OpenSees software advantages

1. This software is free and may easily be downloaded from the relevant website.
2. The software is periodically updated and the relevant facilities and seed are improved may also be downloaded.
3. A special website and forum supports the users.
4. There are various and strong library objects in the same for nonlinear systems numerical simulation.
5. The software enables the user to add new elements of materials, analytic tools, etc based on the study requirements.
6. Various linear and nonlinear structures and geotechnical models may be modelled using this software.
7. Various nonlinear analyses may be simulated using various methods and algorithms.
8. It is excellent software to train finite elements method.
9. The main and superior feature of OpenSees Compiler is changeability and changing the elements as well as the ability to integrate library objects in the software sing new elements made by the user.
10. Simultaneous implementation with MATLAB.
11. Ability to extract abundant outputs in text files which enables the user for post processing.

4.3.3 Introducing finite element method in OpenSees software

1. Model builder object

This command is used to construct the Basic Builder object. These additional commands allow for the construction of Nodes, Masses, Materials, Sections, Elements, Load Patterns, Time-Series, Transformations, Blocks and Constraints.

2. Domain Object

The Domain object is responsible for storing the objects created by the Model Builder object and for providing the Analysis and Recorder objects access to these objects.

3. Recorder Object

The recorder object monitors user-defined parameters in the model during the analysis. This, for example, could be the displacement historey at a node in a transient analysis, or the entire state of the model at each step of the solution procedure. Several Recorder objects are created by the analyst to monitor the analysis.

4. Analysis object

The Analysis objects are responsible for performing the analysis. The analysis moves the model along from state at time t to state at time t + dt. This may vary from a simple static linear analysis to a nonlinear analysis. In OpenSees each Analysis object is composed of several component objects, which define the type of analysis how the analysis is performed.

5. Steel material

Each material was subjected to a series of uniaxial tension and compression strain histories. The following is the response of this material to such strain excursions. The data shown are the normalized stresses versus strain. In the normalization, the steel stress was divided by the yield stress Fy and the concrete stress was divided by the absolute value of compressive strength fc to maintain positive tension and negative compression. This command is used to construct a uniaxial Giuffre-Menegotto-Pinto steel material object with isotropic strain hardening.

6. Section command

This command is used to construct a Section Force Deformation object, hereto referred to as Section, which represents force-deformation (or resultant stress- strain) relationships at beam-column and plate sample points. The valid queries to any section when creating an Element Recorder are 'force' and 'deformation.'

7. Fiber section

A fiber section has a general geometric configuration formed by subregions of simpler, regular shapes (e.g. quadrilateral, circular and triangular regions) called patches. In addition, layers of reinforcement bars can be specified. The subcommands patch and layer are used to define the discretization of the section into fibers.

8. Nonlinear beam-column elements

There are typically two types of Nonlinear Beam-Column Elements:

- Force based elements

Distributed plasticity (nonlinearBeamColumn)

- Concentrated plasticity with elastic interior (beamWithHinges)

Displacement based element

Distributed plasticity with curvature distribution

This command is used to construct a nonlinearBeamColumn element object, which is based on the non-iterative (or iterative) force formulation, and considers the spread of plasticity along the element.

9. Pattern Command

The pattern command is used to construct a Load Pattern object, its associated with the Time Series object and the Load and Constraint objects for the pattern.

10. Recorder

The recorder commands are used to construct a Recorder object, which is used to monitor items of interest to the analyst at each commit.

11. Constrain object

This command is used to construct the ConstraintHandler object. The ConstraintHandler object determines how the constraint equations are enforced in the analysis. Constraint equations enforce a specified value for a DOF, or a relationship between DOFs.

Chapter 5: NUMERICAL MODEL VALIDATION

5.1. Introduction

In this chapter, first of all a numerical model developed in OpenSees software is compared with the analytical model and formula suggested by Chopra (1995) for numerical model verification, reliance and extending the same for the relevant structures, to ensure the software accurate performance and results thereof. The results are compared through force vs. time changes curve. The details are as followed.

5.2. Components of fundamental dynamic system

The principal physical characteristics of each linear elastic system under dynamic loads include mass, elastic features (softness and stiffness), energy dissipation mechanism or damping and finally external source of stimulation or load applied to the same. In the simplest model, i.e. a system with single degree of freedom, it is presumed that each of those characteristics is focused in a separate physical element. Considering Fig. 5.1, the total mass of this system (m) is focused in a hard block, constrained using rolling supports. These constraints are in a certain way that the relevant mass may have a simple displacement movement. Therefore, v(t) (the displacement coordinate) fully defines the mass position. Elastic strength against displacement is provided through a mass-free spring with “k” as stiffness, while energy dissipation mechanism is defined by “c” as damper. The external loading mechanism generating the dynamic response of this system is the p(t) load, which changes with the time.

The simple system motion equation of Fig. 5.1 may easily be formulated through direct expressing the balance of all the forces acting on the mass by taking benefit from D’Alembert Principle (The concept that a mass causes emergence of a force of inertia in proportion to its acceleration and counter to the same is referred to as D'Alembert Principle. This principle is a quite suitable means to solve structures dynamic problems, as enables expressing motion equations as dynamic equilibrium equations). The forces acting along the freedom degree displacement include external force p(t) and three forces resulted from motion, i.e. inertia $f_I(t)$, damping $f_D(t)$, and spring elastic force $f_S(t)$. Therefore, the motion equation is merely an expression of such forces balance as per Equation. 5.1.

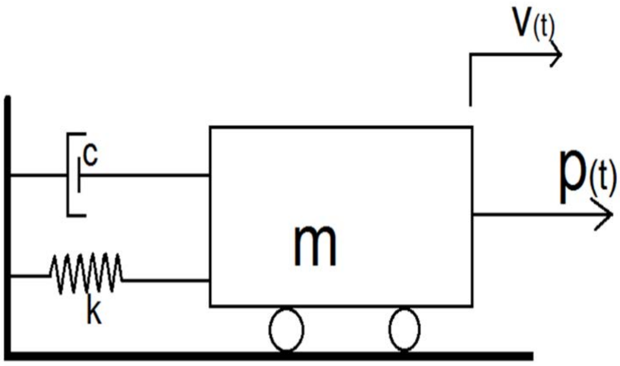


Figure 5.1 SDOF Idealized System (Chopra 1995)

$$f_I(t) + f_D(t) + f_S(t) = p(t) \tag{5.1}$$

According to D'Alembert Principle, inertia force is equal to multiplication of mass by acceleration.

$$f_I(t) = -m\ddot{v}(t) \tag{5.2.a}$$

Presuming viscose damping mechanism, the damping force is equal to multiplication of damping coefficient by velocity.

$$f_D(t) = c\dot{v}(t) \tag{5.2.b}$$

Elastic force is determined of multiplying of spring stiffness by displacement.

$$f_S(t) = kv(t) \tag{5.2.c}$$

Final equations shall be as per Eq. 5.3.

$$m\ddot{v}(t)+c\dot{v}(t)+kv(t)=p(t) \tag{5.3}$$

A variety of analytical methods and practices have been suggested for dynamic equilibrium equation. Evidently, response analysis approaches within time and frequency framework requires assessing the contribution shares of several independent responses to the combined in order to achieve the general response.

In time-domain method (Duhamel's Integral), p(t) loading is considered as a series of short term consecutive strikes and the free vibration response against each strike has a separate share in the total response in each further time. In frequency-domain method, it is presumed that p(t) loading is frequent and decomposed into its harmonic discrete components, Pn using Discrete-time Fourier transform (DTFT). Then the relevant components of structure harmonic response are resulted through multiplication of these loading components by Hn structure frequency response and eventually the total structure response is found by combining the harmonic response components (reverse DTFT). Due to the application of superposition in both methods to achieve the final result, none of these methods are applied for the nonlinear response analysis. Hence, regarding these matters application, although it is expected that severe quakes or severe progressive collapses induce inelastic deformation in a structure designed based on the applicable guideline, due judgment shall be mandatory.

Step-by-step method is the second general solution for dynamic response analysis and well suitable for nonlinear response analysis, as superposition principle is avoided in the same. There are a variety of step-by-step methods; however, in all of them the history of loading and response are distributed to several time intervals or steps. Therefore, the relevant response is calculated within each step out of the initial conditions (displacement and velocity) available in the step commencement and from the loading history within the step. Thus, each step response is an independent analysis problem and there is no need to combine the response shares within the step. Nonlinear behaviour may easily be studied using this method. This is conducted only presuming that the structural characteristics remain constant within each step and may be changed based on each behavioural form from a step to another. Therefore, achieving nonlinear solution, including change in the mass and damping features as well as nonlinear items due to changed stiffness may be applied through choosing short term steps. Step-by-step methods merely represent fully general nonlinear response analysis, while these methods are also similarly valuable regarding linear response.

5.3 Verification of dynamic analysis

The inelastic performance of a single degree of freedom system can be obtained by numerically integrating its equation of dynamic equilibrium whose general form is given by Equation. (5.4).

$$p(t) = \mathrm{m}\ddot{u} + \mathrm{c}\dot{u} + \ f_s\,(u, \dot{u}) \tag{5.4}$$

with $\mathrm{u} = \mathrm{u}(0), \dot{u} = \dot{u}(0)$

In Equation. (5.4) m is the mass, c is damping coefficient, and $f_s\,(u, \dot{u})$ is the restoring force of the system. If the effective force *p*(*t*) is an arbitrary and/or complex function of time (*t*), then solving single degree of freedom system motion equation analytically is not impossible. In order to solve the differential equation of dynamic equilibrium, time step-by-step numerical methods, such as those discussed by Chopra (1995), may be used. In this case the Newton-Raphson algorithm was selected. In this method, the external force *p(t)* is divided into separate and consecutive forces p_i= $p(t_i)$, *i*= 0~N. Single degree of freedom system response, including motion, velocity and acceleration, u_i, $\dot{u}_i$, $\ddot{u}_i$, were thus determined in separate points in time t (called the *i-th* time). The discreet response values satisfy Equation. (5-5), accordingly.

$$p_i = m\ddot{u}_i + c\dot{u}_i + (f_s)_i \qquad (5-5)$$

For a linear elastic system, the restoring force satisfies $(f_s)_i = ku_i$. However, for a nonlinear and inelastic system, the value of the restoring force depends on the rate of motion. Using the numerical method quoted above, the values of $u_{i+1}, \dot{u}_{i+1}, \ \ddot{u}_{i+1}$ in the $(i+1)^{th}$ time were determined. The dynamic response obtained in this way thus satisfies Equation. (5.6), being the dynamic motion of the system controlled by Equations. (5.7).

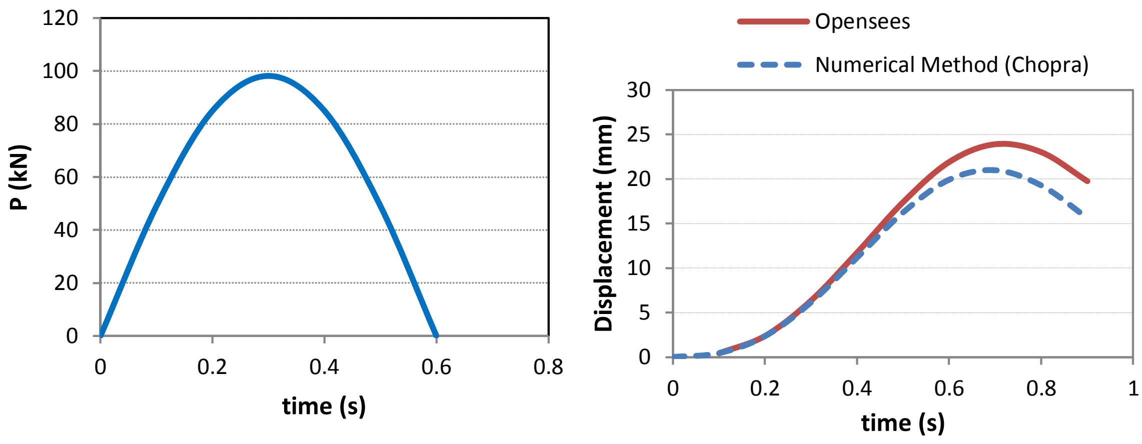
$$p_{i+1} = m\ddot{u}_{i+1} + c\dot{u}_{i+1} + (f_s)_{i+1} \qquad (5.6)$$

$$\ddot{u}_{i+1} = \dot{u}_i + [(1-\gamma)\Delta t]\ddot{u}_i + (\gamma\Delta t)\ddot{u}_{i+1} \qquad (5.7)$$

$$u_{i+1} = u_i + (\Delta t)\,\dot{u}_i + [(0.5-\beta)(\Delta t)^2]\ddot{u}_i + [\beta(\Delta t)^2]\ddot{u}_{i+1} \qquad (5.8)$$

In Equation (5.7) β and γ define the rate of acceleration in a time step. Common values for these constants are $\gamma = \frac{1}{2}$ and $\frac{1}{6} \leq \beta \leq \frac{1}{4}$. Eqs. (5.6) and (5.7) were combine to determine u_{i+1}, $\dot{u}_{i+1}$, and $\ddot{u}_{i+1}$ in (i+1) following the known values of u_i, $\dot{u}_i$, and $\ddot{u}_i$ estimated for time *i*.

In order to calibrate the quasi-static analyses used here, a case study drawn from Chopra (1995) consisting of one degree freedom elasto-plastic system with a mass of 253.3 kg, 5% damping and yield deformation of $u_y = 7.5$ mm, was analysed with OpenSees as well as with the algorithm outlined above. Figure 2a shows the load applied to the model whilst Figure 5.2.b shows the discrepancy between the two procedures. The differences encountered seem to be caused by a numerical error by Chopra (1995) when estimating the tangent stiffness. This explains why the curves in Figure 5.2.b coincide in the first half of the analysis but diverge in the second half.



a) b)

Figure 5.2 A case study analysis a) Load applied to the model b) Displacement in dynamic analysis (Homaioon Ebrahimi et al.2017)

In fact, error is associated with the use of tangent stiffness instead of unknown secant stiffness, and is illustrated by the force deformation relation of Fig 5.3 b. The displacement at time i, the beginning of a time step, is known as point a. Using the tangent stiffness at a, numerical integration from time i to time i+1 leads to the displacement u_{i+1}, identified as a point b. (Chopra 1995).

$$m\,\Delta\ddot{u}_i + c\,\Delta\dot{u}_i + (\Delta f_s)_i = \Delta P_i$$

$$(\Delta f_s)_i = (k_i)_{sec}\,\Delta u_i \tag{5.8}$$

$$(\Delta f_s)_i \approx (k_i)_T\,\Delta u_i$$

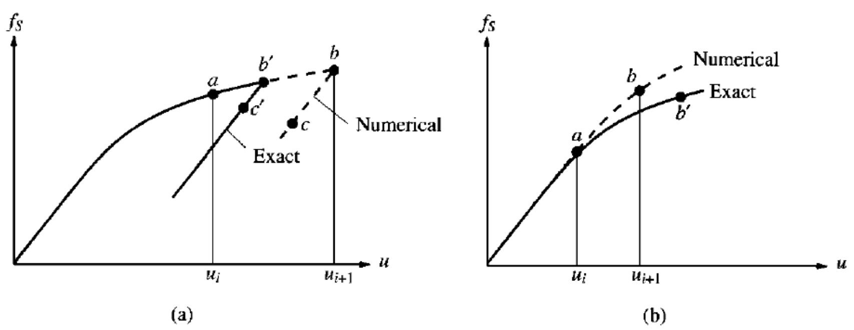
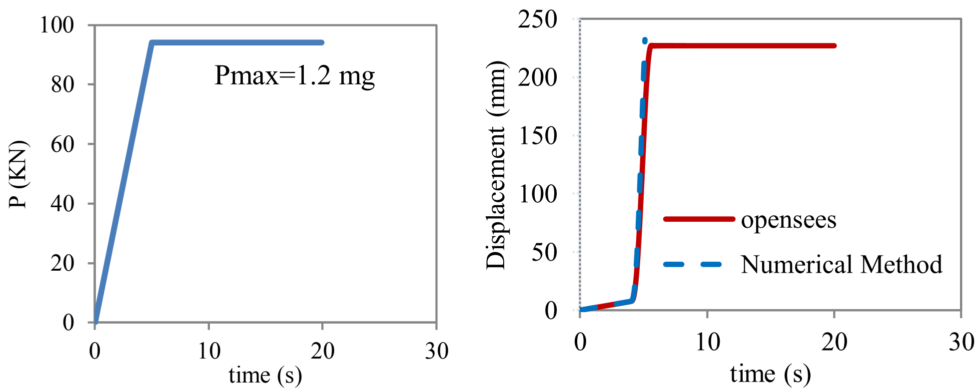


Figure 5.3 Tangent stiffness and secant stiffness definition (Chopra 1995)

Following, the two models were loaded with the time-varying load shown in Figure 5.4.a. That load represents the load transfer process recommended by GSA (2003, 2013), hence it is the one used to run the full analyses of the multi-storey buildings described in sections 2-4. The dynamic performance obtained for the single degree of freedom system for the initial 5 s is illustrated in Figure 5.4.b.



a) b)

Figure5.4 Validation of the progressive collapse analysis a) Load b) Displacement (Homaioon Ebrahimi et al.2017)

In order to verify the non-linear dynamic analyses in terms of the members' rotation estimation and displacement of nodes in the case of progressive collapse, Kim et al. (2009) research on a 3-storey and a 3-bay frame was studied. The columns and girders were model using the Nonlinear-Beam-Column element. The bi-linear material model with the post-yield stiffness of the initial stiffness was used, and the damping ratio was assumed to be 5% of the critical damping. The general scheme of this frame is set out in Fig. 5.5.

The rotation was calculated in a progressive collapse analysis of the sudden removal of the corner column from the first storey of the frame using the non-linear dynamic analysis method. The locations of the nodes where the rotation has been calculated in this thesis are shown in Fig 5.6. In Table 5.1, the values of rotation in the hinges have been identified and the final value of the A node displacement provided. As can be seen in table, it is concluding that the accuracy of the non-linear dynamic method applied in this structure can be used to analyse progressive collapse.

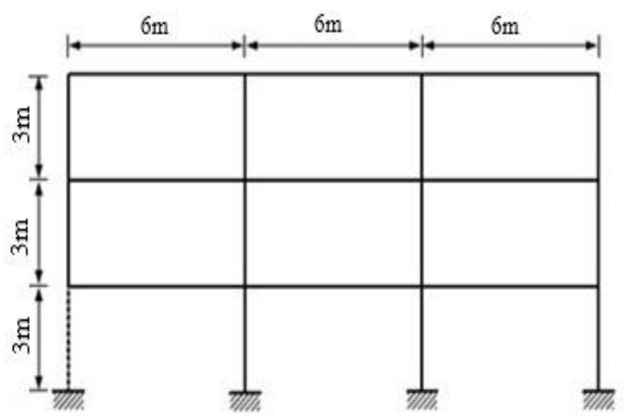


Figure 5.5 The studied frame by Kim et al. (2009)

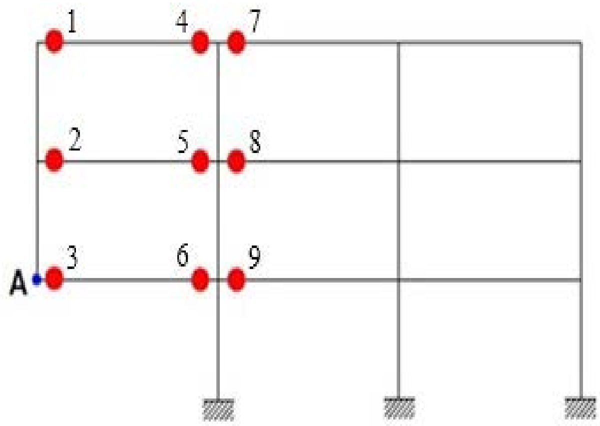


Figure 5.6 Locations of the investigated nodes (Kim et al. 2009)

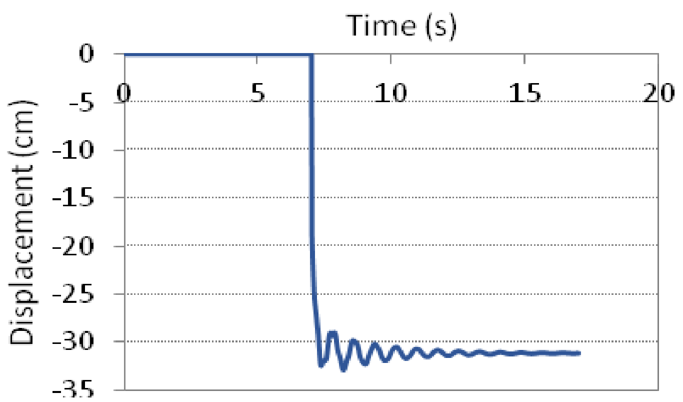


Figure 5.7 Displacement of node A in this study

Table 5.1 The Plastic rotation

Responses	Kim et al.	H.E et al.
Plastic rotation (point 1)	0.020	0.0205
Plastic rotation (point 2)	0.032	0.0327
Plastic rotation (point 3)	0.018	0.0184
Plastic rotation (point 4)	0.032	0.0328
Plastic rotation (point 5)	0.032	0.0327
Plastic rotation (point 6)	0.050	0.0513
Plastic rotation (point 7)	0.003	0.0031
Plastic rotation (point 8)	0.001	0.00102
Plastic rotation (point 9)	0.003	0.0031
Vertical disp. at point A	30.85	31.2

Chapter 6: NONLINEAR STATIC ANALYSIS RESULTS

6.1. Introduction

The 3D model structures were numerically analysed with OpenSees. Nonlinear analyses were run considering a simple bi-linear material model with post-yield stiffness. Nonlinear beam-column elements were used for modelling the cross-sectional areas as precisely as possible. The plastification over element length and cross-sections were also considered, whereas large displacements effects were also accounted for by the employment of the co-rotational transformation of the geometric stiffness matrix. The dynamic behaviour caused by sudden column removal was not a factor in the load reversal because, in structures subjected to earthquake loads, using a complicated hysteretic model is unnecessary. The fraction of damping was assumed to be 5% which is usually the case for structures with large deformations.

In this part of the thesis, the Pushdown Analysis results on 14 regular and irregular 2, 3 and 5-storey steel structures using intermediate moment resisting frame through removal of selective columns. These structures were introduced in Chapter 4. Meanwhile, the loading model imposed to these structures under nonlinear static analysis was suggested in Chapter 3. The study results are given in the following.

6.2. Introduction to pushdown analysis method

The positions of the removed columns have been indicated in Fig. 6.1. The position of seven columns in all the regular and irregular structures is the same. in all cases, the removed column has been chosen from the first floor, as such column removal represents more critical conditions for structure stability and deformations.

Additionally, a wide range of different column removal scenarios have been adopted to determine and identify structure yield capability as per Fig. 6.1. Such column removal scenarios have been given in Table 6.1. In each of these scenarios, the structure response at the spot of removed column is assessed under nonlinear static Analysis.

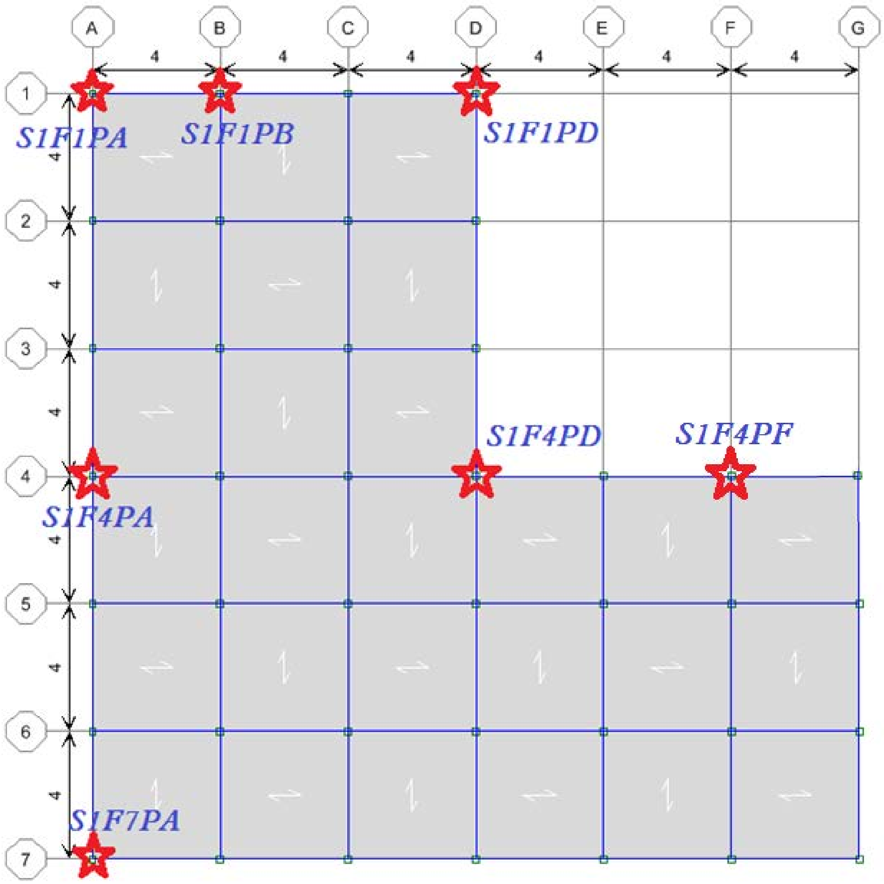


Figure 6.1 Position of removed columns in all the studied structures

Table 6.1 Column-removal scenarios for each of the four structures.

	Location of removal column			
Number	Storey	Frame	Pier	Scenario notation
1	1	1	A	S1F1PA
2	1	1	B	S1F1PB
3	1	1	D	S1F1PD
4	1	4	A	S1F4PA
5	1	4	D	S1F4PD
6	1	4	F	S1F4PF
7	1	7	A	S1F7PA

A brief explanation is given here on nonlinear static analysis prior to present the results. This includes a certain analysis of structure at the removal of one or more its member(s) position through increased gravity loads. The gravity loads increase linearly until structure collapse, which is seen as structure inability to robustness the applied loading.

The force relating to such conditions is referred to as the yield load. The structure capacity at this point is expressed as a form of load factor, introduced as the Eq.6.1.

$$Load\ Factor = \frac{Load}{Nominal\ gravity\ load} \tag{6.1}$$

Such load ratio normalization causes a better understanding on structure capacity. Such factor is seen as a standard to assess the damaged structure. For instance, in case the load factor is higher than 1 according to the displacement causing materials yielding, then the structure may resist under column removal; otherwise, the structure shall suffer collapse.

In the nonlinear static analysis method is merely focused on damaged spans, where the gravity loads increase merely in those spans representing damages and column removal until system collapse level is reached. The rest of structure parts are subject to normal gravity loads. Therefore, such analysis offers a certain level of collapse as per the yield in the damaged spans. The system remained capacity is calculated in load factor model, which is as a ratio between the force caused yielding in the damaged spans and normal gravity load. In nonlinear static analyses were performed, following the GSA recommendation for using a dynamic amplification factor of~2. That, in order to reflect a ratio of 2 between the load that is applied to the spans that are adjacent to the removed column with respect to that applied on other spans.

6.3. Two-storey moment-resisting frame steel structure results

Considering the foregoing, fourteen regular and irregular structures are going to be analysed in this thesis using nonlinear static analysis. First of all, the 2-storey structure results are given. In Fig. 6.2, a comparison has been conducted between load factors in different column removal scenarios for 2-storey structure.

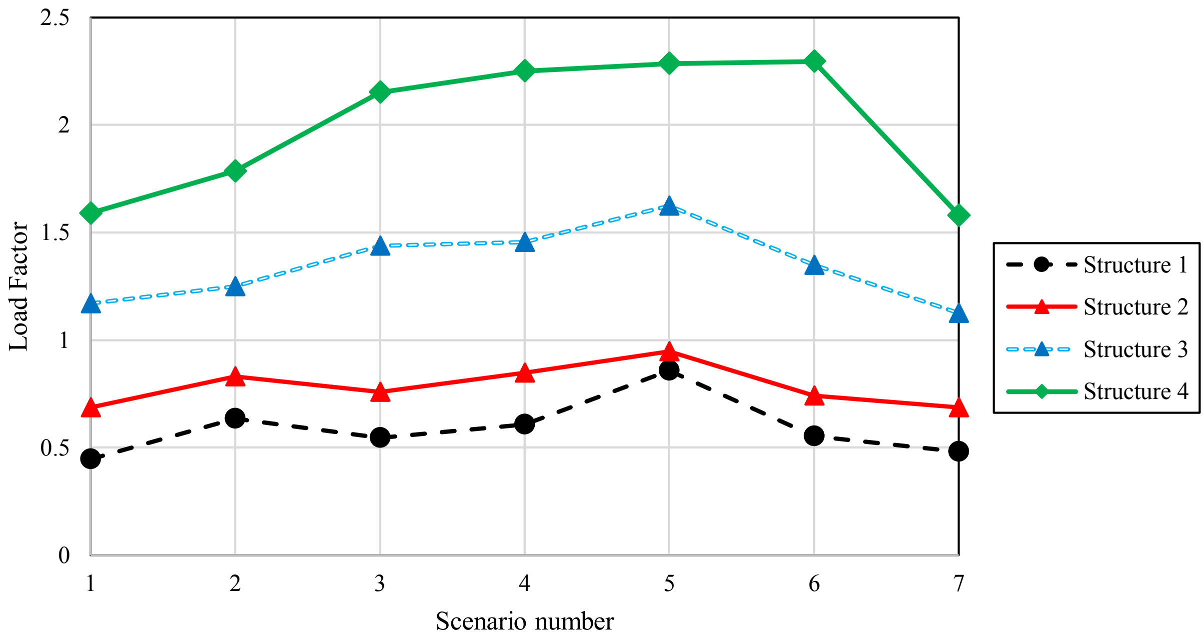


Figure 6.2 Load factors for all the structures and scenarios in 2-storey

Considering Fig. 6.2, it may be understood that under all column removal states, the two irregular structures (1 and 2) are not able to bear the force imposed by column removal in two C and E seismic regions, respectively. The most critical state (minimum structure ability against column removal) is related to a state when the corner columns on “A” axis are removed. Under various column removal scenarios, both regular structures (3 and 4) managed to bear the load imposed to adjacent columns. Also in these two structures the most critical state is related to a state when the corner columns on “A” axis are removed. Meanwhile, under all regular and irregular structural states, removal of the column on “D and 4” axes impose the minimum damages and load to the structural system, which is due to higher stiffness at the column removal location caused by beams connected to the relevant node. In other words, corner and internal columns removal imposes maximum and minimum damages to the structures, respectively.

6.4. Three-storey moment-resisting frame steel structure results

In Fig. 6.3 the load factor for regular and irregular structures in 3-storey structure has been shown. As it may be seen, like 2-storey, also here removal of corner column represents maximum damages and minimum structural endurance against progressive collapse are resulted. Both regular structures (3 & 4) managed to resist against progressive collapse under various column removal scenarios. Also in these structures, corner (S1F1PA) and internal (S1F4PD) columns removal imposes maximum and minimum damages to the structures, respectively.

On the other hand, it is seen that upon increasing structure height from 2 to 3 stories, the structure capacity against progressive collapse also increases. Comparing the load factor in two 2- and 3-storey structures with similar status the same issue may be instated. For instance, the load factor in structure 1 with scenario 3 is equal to 0.545, while for 3-storey the same is equal to 0.581. In other words, a rough increase of 7% has been observed in the structure capacity. However, as seen in Table 6.2, such increase rate varies among scenarios.

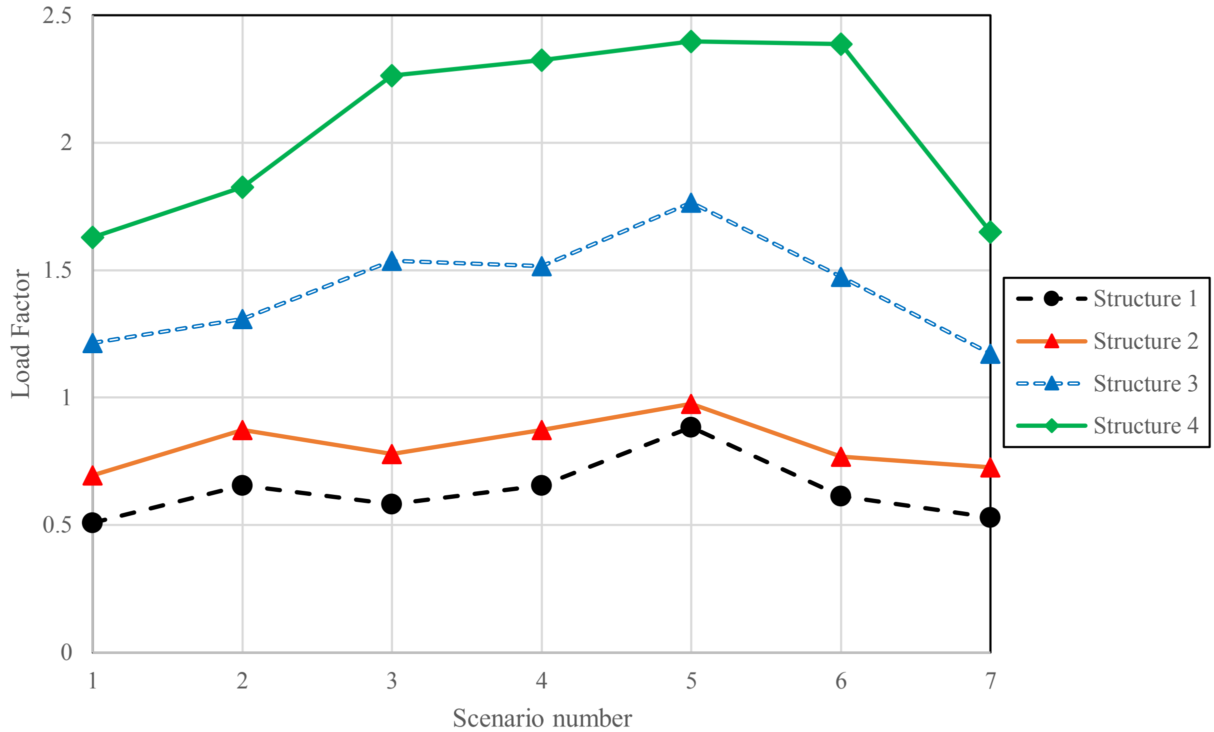


Figure 6.3 Load factors for all the structures and scenarios in 3-storey

Table 6.2 increasing rate of load factor ratio between 3-storey and 2-storey for each of the four structures

Scenario number	Structure 1	Structure 2	Structure 3	Structure 4
1	1.14	1.01	1.04	1.03
2	1.03	1.05	1.05	1.02
3	1.07	1.03	1.07	1.05
4	1.08	1.03	1.04	1.03
5	1.03	1.03	1.09	1.05
6	1.11	1.04	1.09	1.04
7	1.14	1.01	1.04	1.03

Considering Table 6.2, it is observed that on average, those structures situated in E seismic regions (regular and irregular) are roughly subject to an increased structural capacity of 3%. Irregular Structure 1 situated in C seismic region managed to increase its capacity for 14% under most critical column scenario (S1P7FA) upon increasing the height for 3m and generating of an additional stiffness grid on the 3rd floor to the total system stiffness. However, still the structure has a load factor less than 1 (0.53).

The minimum column removal risk amongst the scenarios mentioned in the above section is related to S1F4PD internal column removal, reaching from 0.95 to 0.98. It is expected that by further height increase the structure capacity may reach 1, while still other scenarios cause structure collapse (It refers to irregular structures).

6.5. Five-storey moment-resisting frame steel structure results

Considering Fig. 6.4, as expected, by increasing the height, and in case of removing S1F4PD Column, the structure may bear the load caused by vertical loading at the removed column. This issue has been expressed using a 1.18 load factor for Structure 2 situated in E seismic region. However, still irregular Structure 1 may not bear the force caused by vertical loading at the removed column and force distribution amongst its adjacent members.

A comparison between the 5-storey structure with those 2- and 3-storey ones as per tables 6.3 and 6.4 demonstrates that under all states the capacity and load factor have increased upon increasing the structure height and developing more stiffness along the structure height.

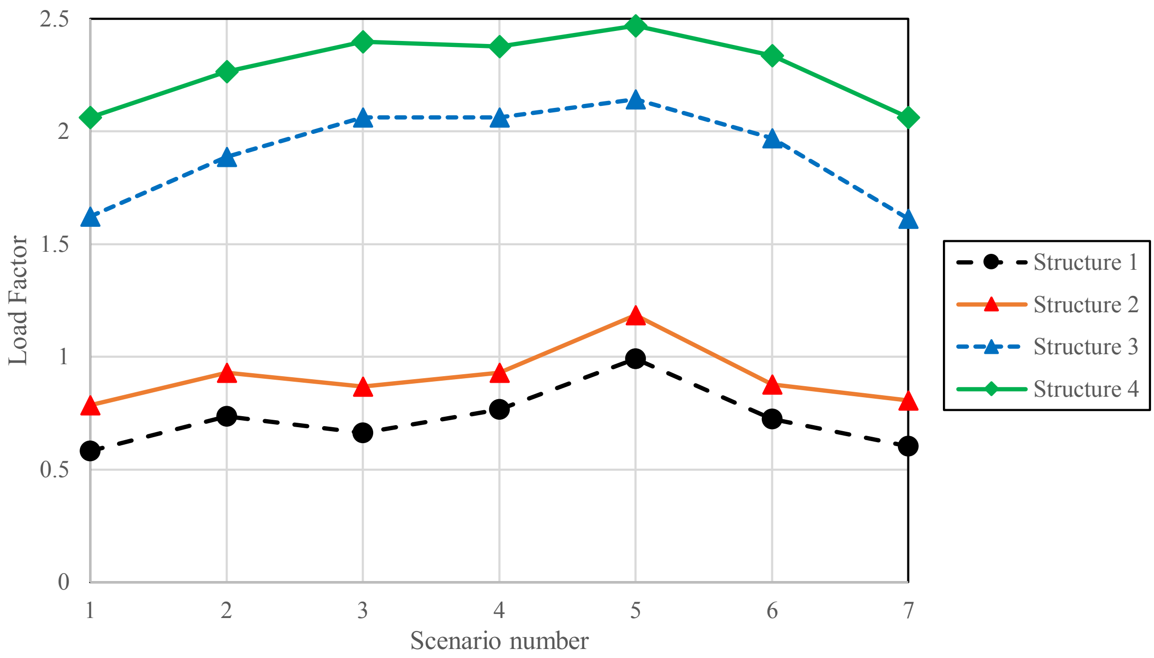


Figure 6.4 Load factors for all the structures and scenarios in 5-storey

Table 6.3 Increasing rate of Load Factor ratio between 5-storey and 2-storey for each of the four structures

Scenario number	Structure 1	Structure 2	Structure 3	Structure 4
1	1.3	1.14	1.39	1.3
2	1.16	1.12	1.51	1.27
3	1.22	1.14	1.43	1.11
4	1.26	1.09	1.42	1.06
5	1.15	1.25	1.32	1.08
6	1.31	1.18	1.46	1.02
7	1.25	1.17	1.43	1.3

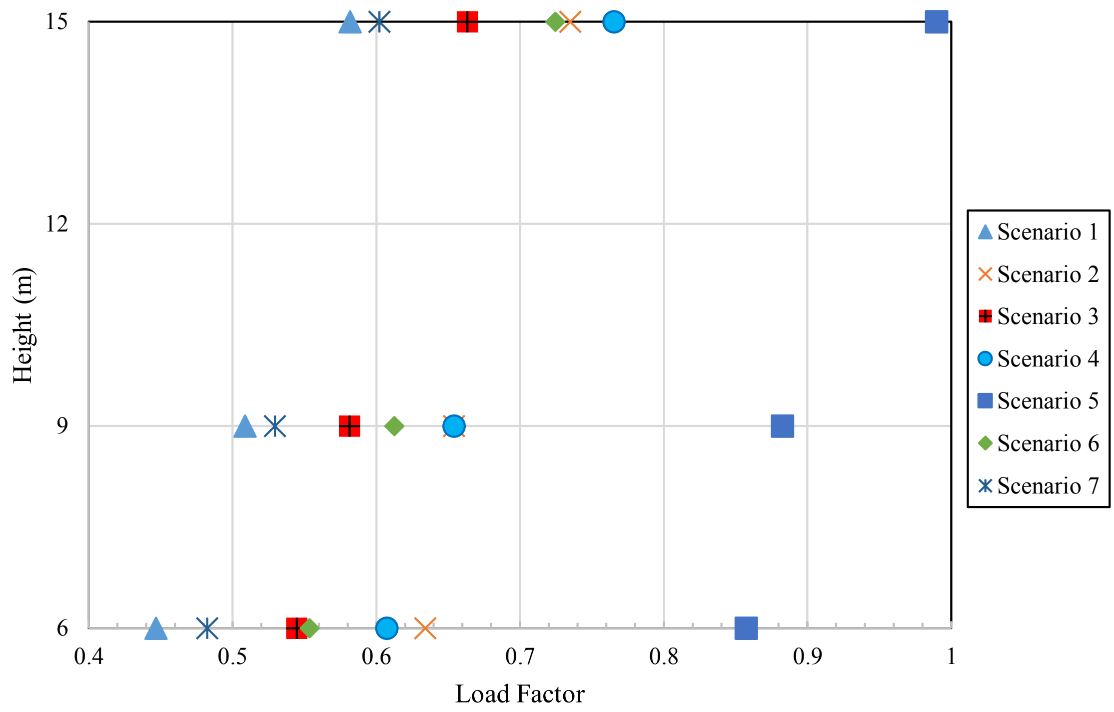
Minimum height effect from 6m to 15m on load factor is relating to removal of columns situated on axis 4 (scenarios 4, 5 and 6) for regular Structure 4. Considering Table 6.4, it is even seen that under scenario 6 (S1F4PF), increasing the number of floors from 3 to 5 failed to increase load factor.

In other words, it may be expressed that increasing height and further stiffness affect corner columns removal scenarios, which may be understood through comparing numbers in scenarios 1 and 7 for all four structural states.

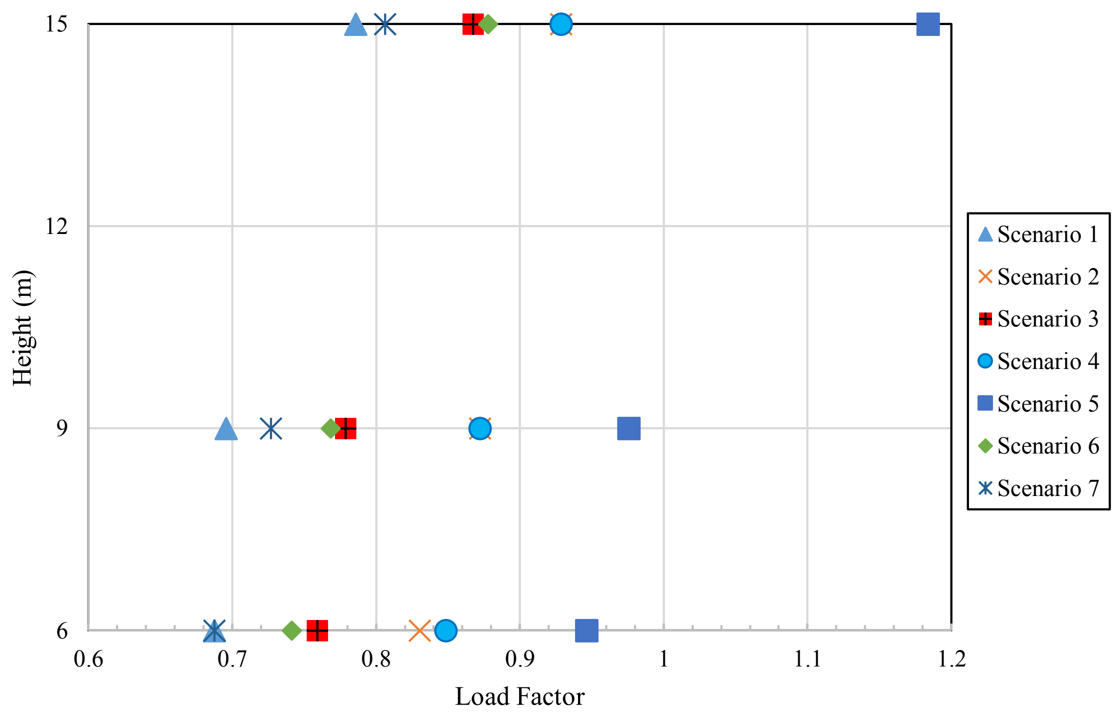
Table 6.4 Increasing rate of Load Factor ratio between 5-storey and 3-storey for each of the four structures

Scenario number	Structure 1	Structure 2	Structure 3	Structure 4
1	1.14	1.13	1.34	1.26
2	1.12	1.06	1.44	1.24
3	1.14	1.11	1.34	1.06
4	1.17	1.06	1.36	1.02
5	1.12	1.21	1.21	1.03
6	1.18	1.14	1.34	0.98
7	1.14	1.11	1.37	1.25

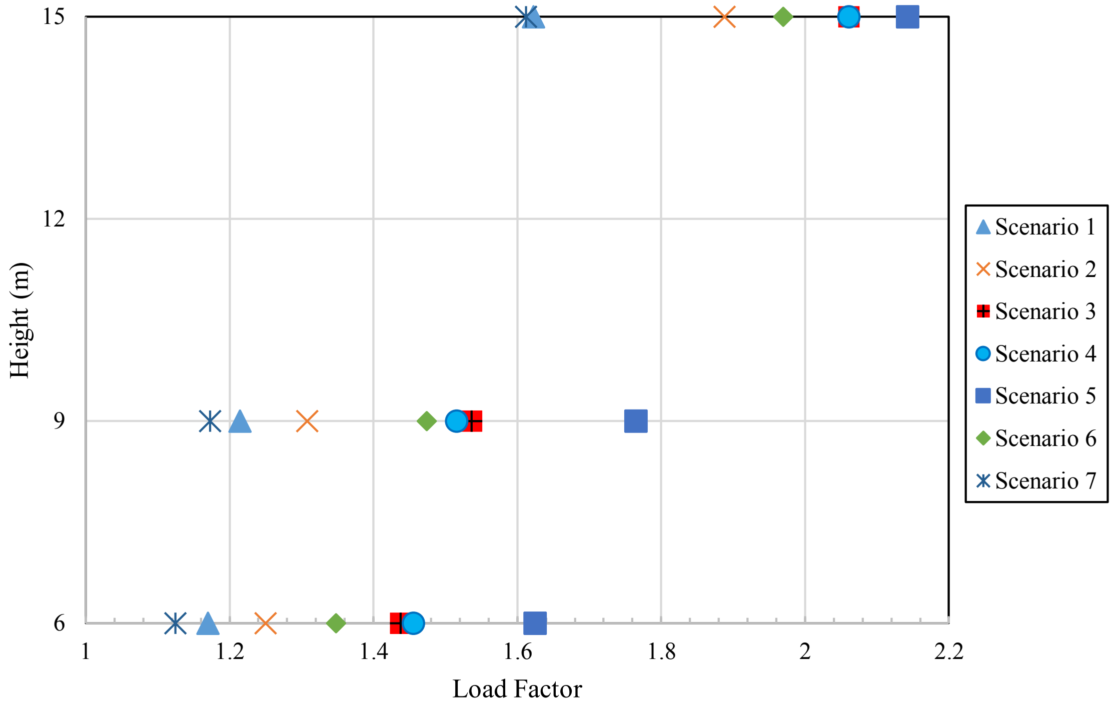
In Fig. 6.5, the load factor changes in terms of height have been given for all four structures (1 through 4). Regarding forces in height ratio changes it may have expressed that the relation between force and height for regular structures is almost linear, while the same is nonlinear for irregular ones.



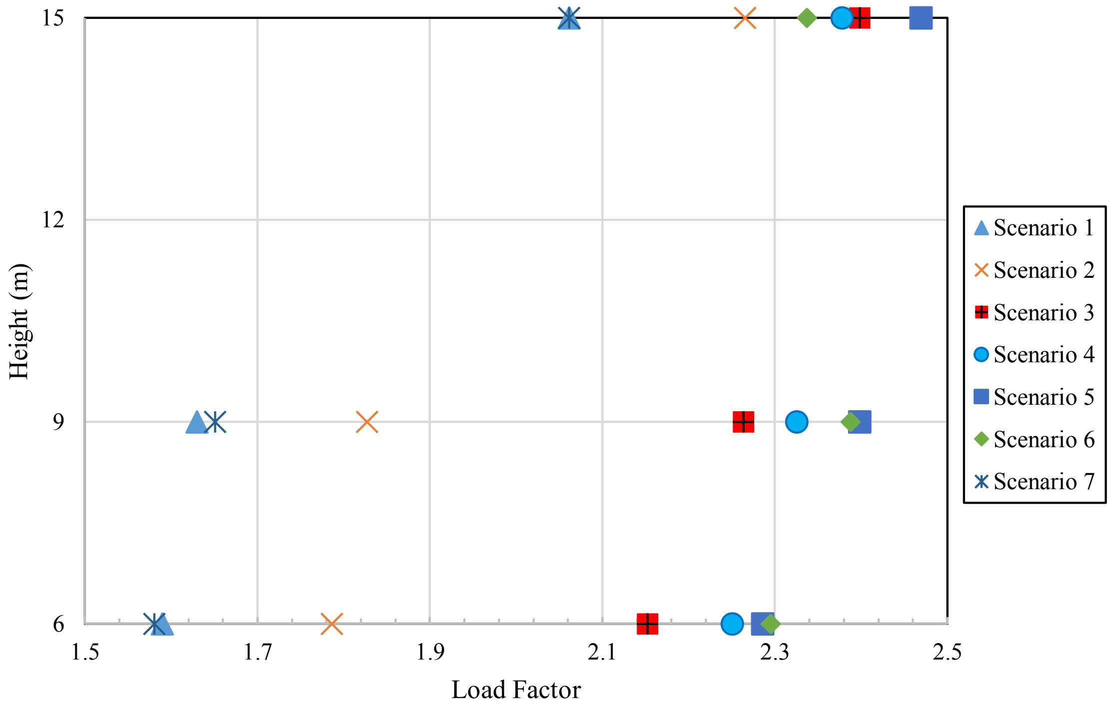
(a) Structure 1



(b) Structure 2



(c) Structure 3



(d) Structure 4

Figure 6.5 Yield load factors vs. Height for all the structures and scenarios

6.6. Force-vertical displacement curve at the column removal position

As mentioned by (Mazzoni et al. 2006), the push-down analysis generally involves several runs and highly depends on load step or tolerance. In this study the push-down analyses were carried out using the program code OpenSees. In the analysis, gravity load was imposed on structures with its original loading pattern unchanged. The amount of the load is referred to as the "load factor," which is the ratio of the equivalent load to the gravity load. The original loading pattern remained unchanged at every step. In this way the results of the pushdown analysis were maintained from load controlled push-down analysis until the ultimate loads were reached.

Capacity curves have been given in Figs. 6.6 through 6.8. Generally, force-vertical displacement changes may be classified into four stages: elastic stage, elastic–plastic stage, arch stage, and catenary stage. According to Fig. 6.6, the distance between OA, AB, BC and CD are known as linear region, elastic-plastic region, arch stage and catenary stage, respectively. In OA region the force-displacement relation may be taken as linear, where force and deformation are small. For instance, in Figure 6-8 scenario S1F1PA, when the load factor increase from 1/3, then the curve enters into the second region (elastic-plastic), where the force starts increasing in a relatively nonlinear manner with respect to the displacement changes, while the stiffness commences reducing. The third stage, i.e. from B to C, is known as the Plastic Stage. In this stage, the force also increases upon increased displacement.

Upon applying force and increasing at the position of removed column, the connection at the removed column position is exposed to positive moment, and the farther connection is exposed to negative moments.

This action causes higher strength in comparison to plastic strength. In the final stage, known as catenary stage, slab and steel beam resist against tensile forces like a chain. In this stage, the force still increases linearly along with the increased vertical displacement. These two stages have been shown in Fig. 6.7.

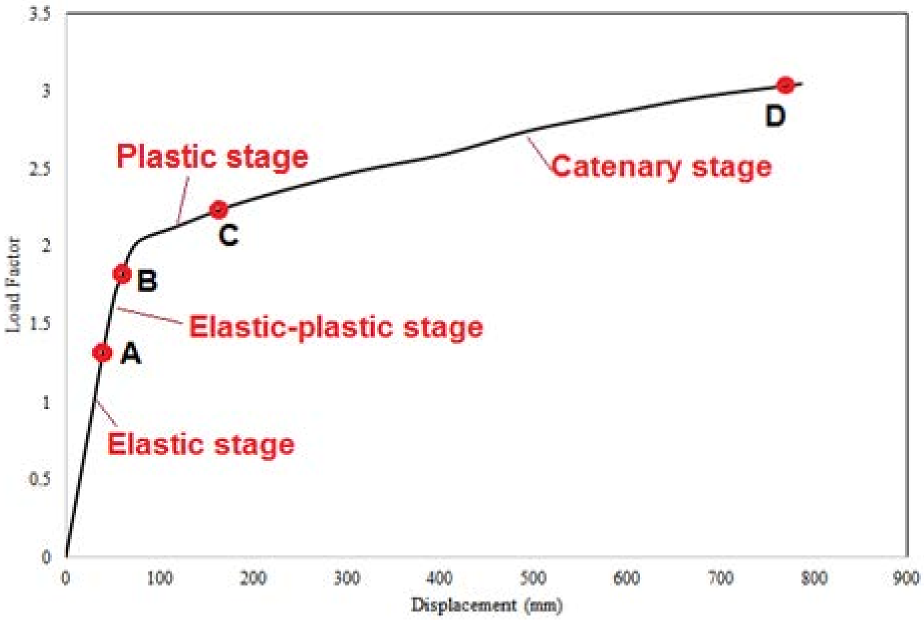
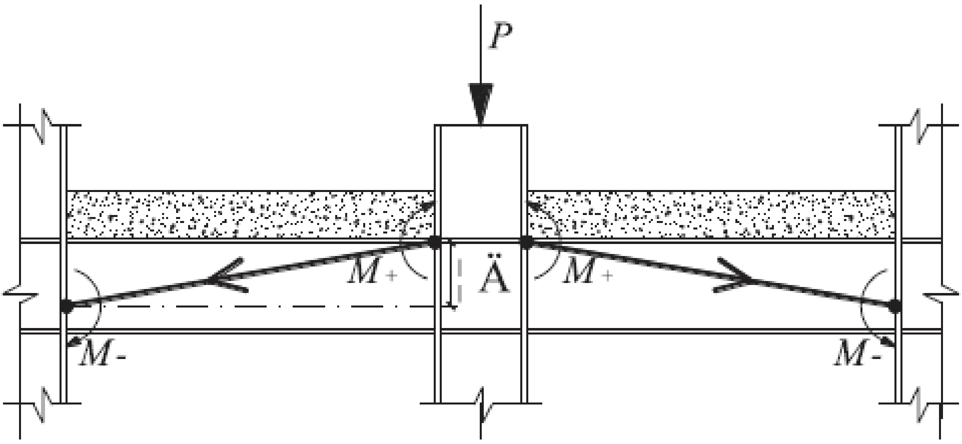
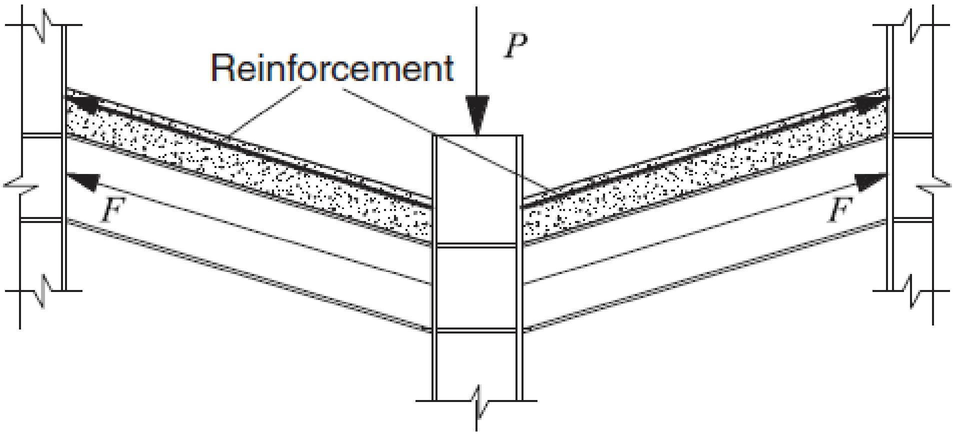


Figure 6.6 Vertical load vs. displacement of removed column relationship curve

Under all states, Structure 4 has maximum elastic stiffness and plastic strength. For instance, regarding 2-storey structure under S1F1PD column removal scenario, the elastic stiffness for regular structure 4 is almost four times of that of irregular structure 1. Meanwhile, stiffness in catenary stage is also highest in structure 4 in comparison to other structures, which is also due to symmetry and being located in a region with E seismic risk.

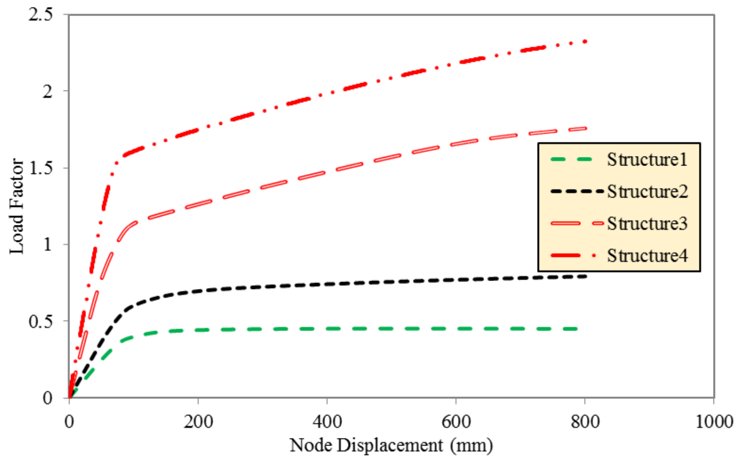


(a) Plastic stage

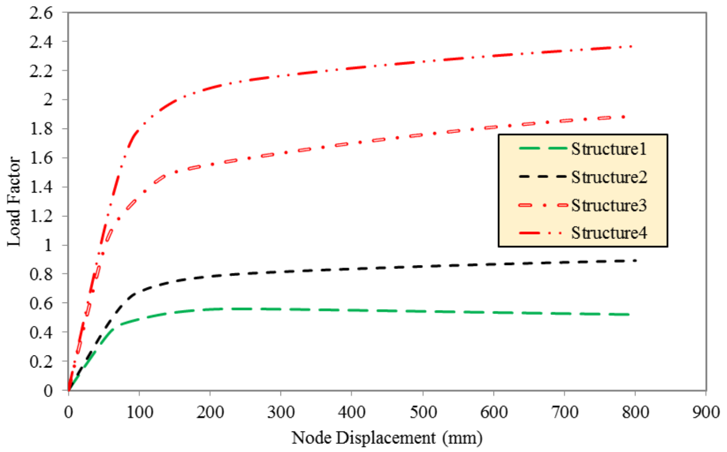


(b) Catenary action

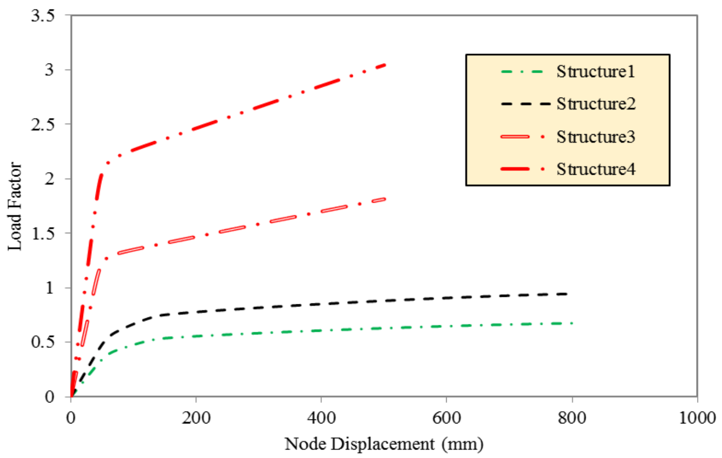
Figure 6.7 Two stages of load transmitting system in frame



(a) Scenario S1F1PA



(b) Scenario S1F1PD



(c) Scenario S1F4PF

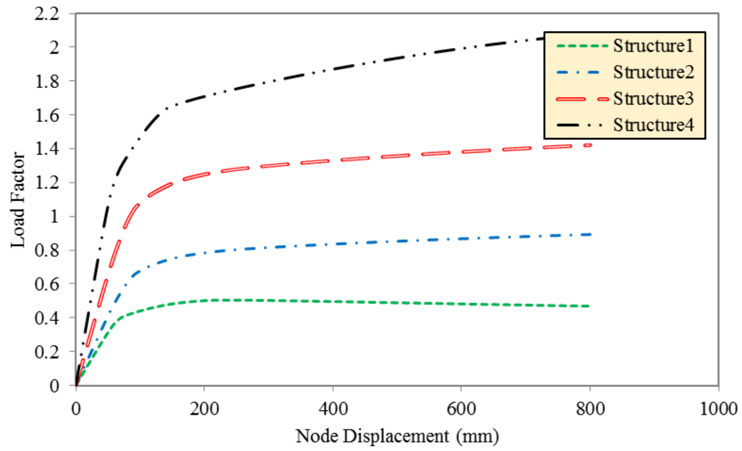
Figure 6.8 Pushdown capacity curve for 2-storey structures

In Table 6.5 a comparison has been given amongst values of elastic stiffness, yield strength, plastic strength and stiffness of catenary stage for 2-storey structure under three different scenarios.

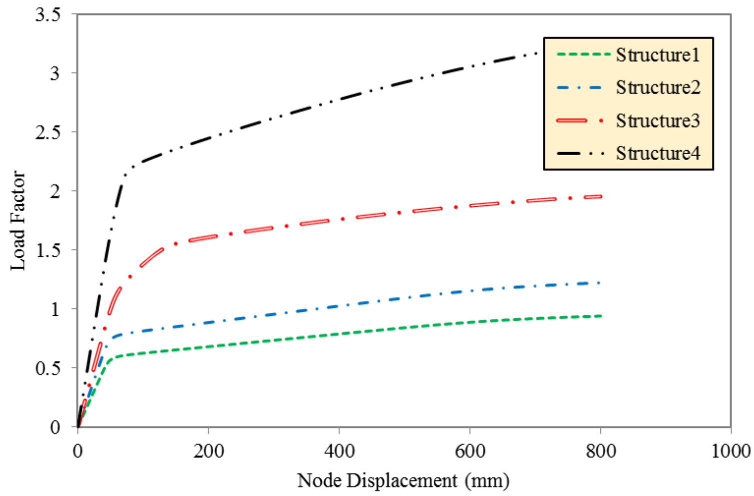
Table 6.5 Detailed results of simulations with different scenarios and structures

Scenario	Structure	$K_{elastic}$ (kN/mm)	Plastic resistance (kN)	Plastic displacement (mm)	$K_{catenary}$ (kN/mm)
S1F1PA	1	29.20	2164.14	83.92	0.52
	2	43.55	3293.25	92.57	0.87
	3	118.78	8112.86	81.07	7.48
	4	204.59	12587.45	85.34	7.28
S1F1PD	1	40.37	2634.60	69.17	1.43
	2	50.82	3747.00	85.14	2.78
	3	148.76	9679.66	72.59	6.37
	4	187.67	14519.53	97.17	28.70
S1F4PF	1	41.41	2665.33	88.67	4.13
	2	61.20	4036.05	101.15	3.30
	3	205.36	10167.10	57.47	16.33
	4	353.92	17389.70	59.77	19.57

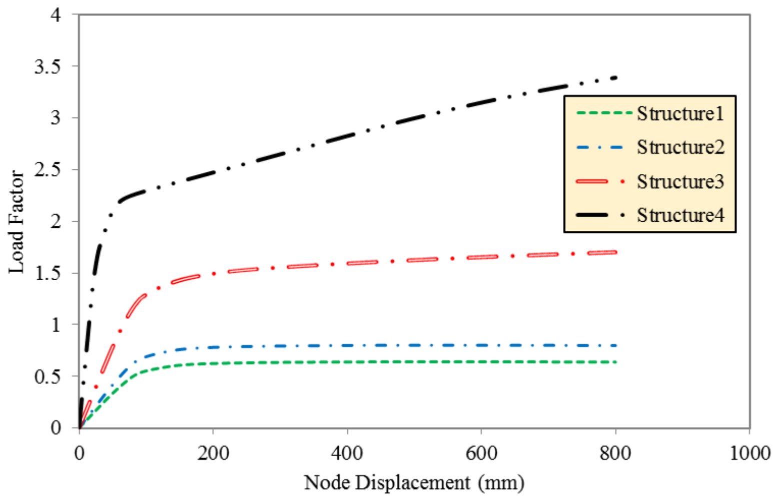
Considering Table 6.5, in all cases elastic stiffness, plastic strength and stiffness in catenary stage are higher for regular structures than those of the irregular structures. In other words, regular structures have higher energy absorption capability due to column removal in comparison to irregular ones. It may be noted that due to column removal in peripheral areas or where there are coincidence spots of opening or indentation in the plan, the difference between stiffness in catenary stage in regular and irregular structures increases, which is due to higher indeterminacy degrees or higher stiffness of floor and beam grid system at the spot of column removal. In other words, the structural system under S1F1PA scenario (corner column) has less plastic strength and stiffness due to connection of two beams perpendicular onto the column. Although plastic resistance of structures increases from irregular towards regular structure, displacement represented by such point does not significantly change and may not be found a real increase/decreased trend for the same.



(a) Scenario S1F1PA



(b) Scenario S1F1PD



(c)Scenario S1F4PF

Figure 6.9 Push down capacity curve for 3-storey structures

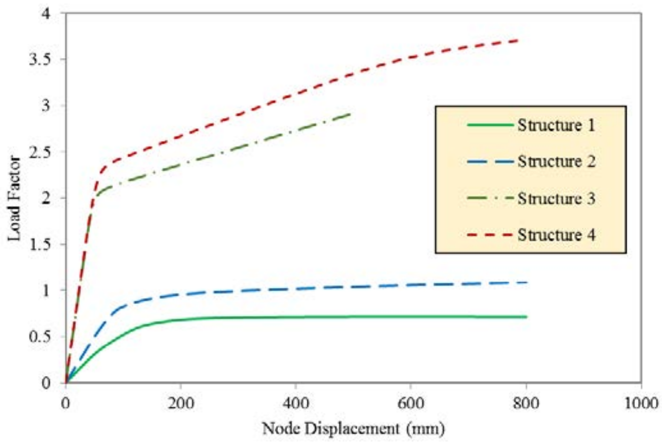
In Table 6.6 a comparison has been given amongst values of elastic stiffness, yield strength, plastic strength and stiffness of catenary stage for 3-storey structure under three different scenarios.

Table 6.6 Detailed results of simulations with different scenarios and structures

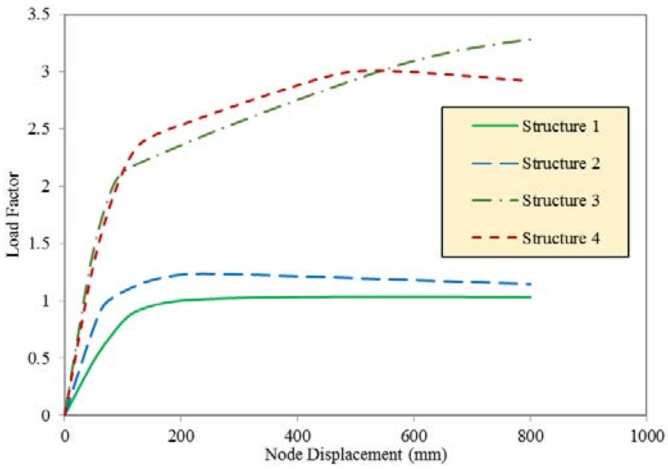
Scenario	Structure	$K_{elastic}$ (kN/mm)	Plastic resistance (kN)	Plastic displacement (mm)	$K_{catenary}$ (kN/mm)
S1F1PA	1	57.01	3903.00	72.23	1.58
	2	77.30	5967.10	89.17	3.67
	3	162.32	12396.00	85.93	3.20
	4	283.06	16871.24	84.12	6.40
S1F1PD	1	112.60	5393.97	52.00	2.14
	2	147.24	6972.70	55.70	4.12
	3	226.34	13507.00	58.23	5.44
	4	392.64	26400.00	72.15	17.00
S1F4PF	1	59.20	4858.34	87.00	2.53
	2	70.02	6036.11	99.56	5.81
	3	158.43	15114.83	97.75	8.75
	4	568.55	21396.37	36.37	22.11

It is understood according to the details taken from 3-storey structure vertical curves under three column removal scenarios that in all scenarios, irregulars structures 1 and 2 have the least plastic strength and stiffness in catenary stage in comparison to regular structures 3 and 4. Meanwhile the relevant structure demonstrates minimum stiffness and plastic strength upon corner column removal amongst different column removal scenarios. Regular structure 4 has the highest capacity amongst the studied structures.

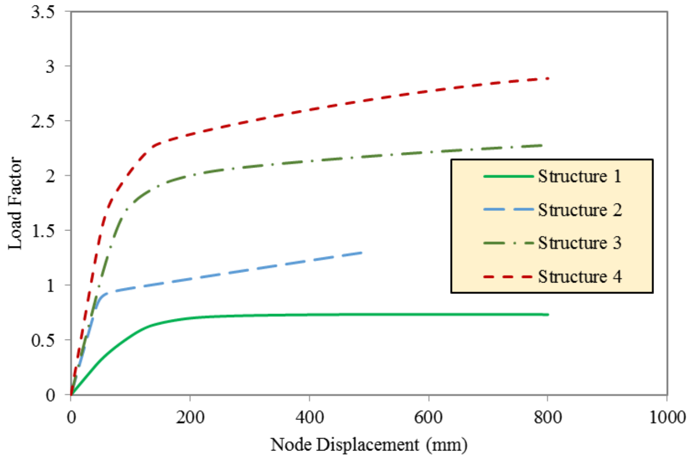
Also in these samples, like 2-storey structures, upon changing from irregular towards regular plan, stiffness in catenary stage increases, which results in an increase in capacity of load-bearing and transferring tensile forces developed in the beams.



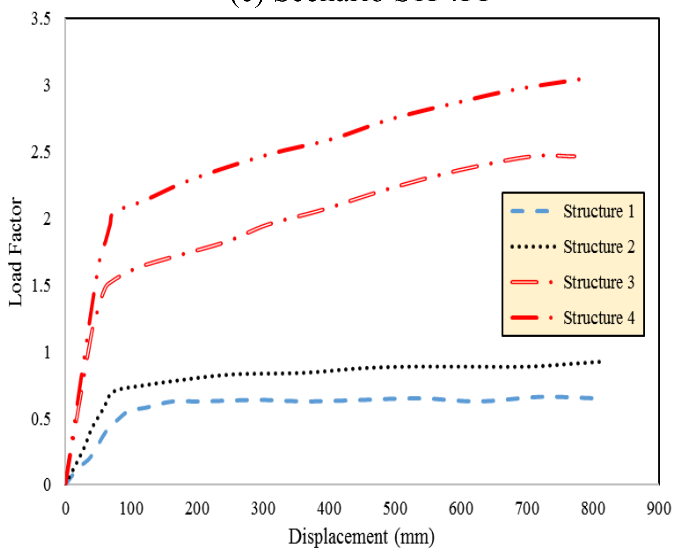
(a) Scenario S1F4PA



(b) Scenario S1F4PD



(c) Scenario S1F4PF



(d) Scenario S1F7PA

Figure 6.10 Push down capacity curve for 5-storey structures

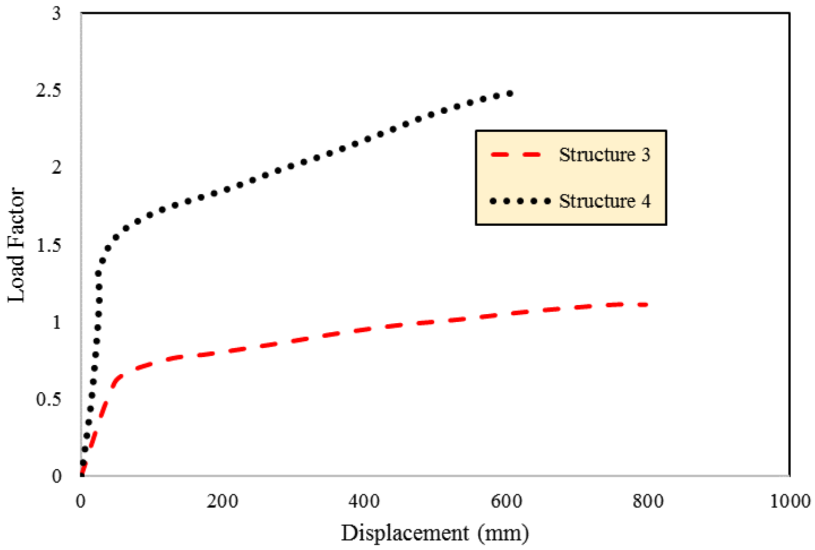
In Table 6.7 a comparison has been given amongst values of elastic stiffness, failure strength, plastic strength and stiffness of catenary stage for 5-storey structure under three different scenarios.

Table 6.7 Detailed results of simulations with different scenarios and structures

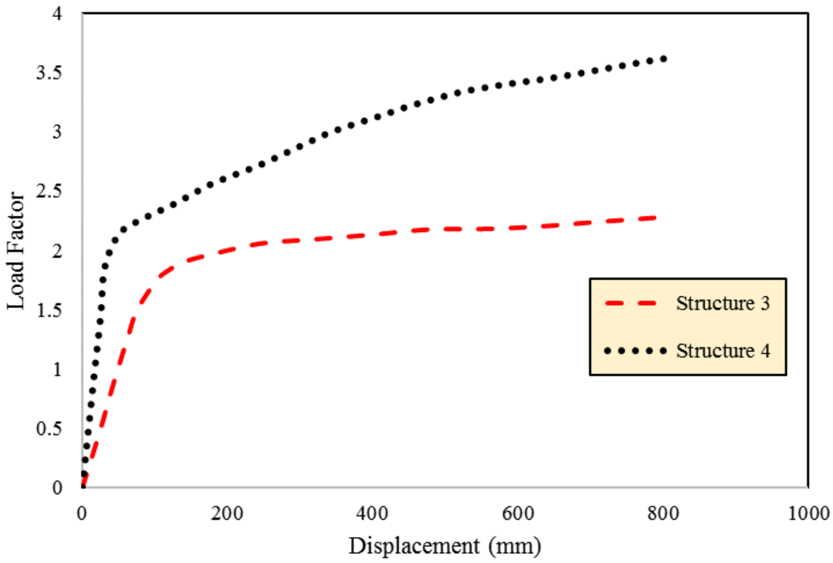
Scenario	Structure	$K_{elastic}$ (kN/mm)	Plastic resistance (kN)	Plastic displacement (mm)	$K_{catenary}$ (kN/mm)
S1F4PA	1	90.51	9529.21	95.12	10.34
	2	156.86	12032.63	90.00	10.38
	3	915.02	40817.83	51.15	43.10
	4	947.88	47775.65	57.28	25.01
S1F4PD	1	150.56	11798.29	89.39	11.39
	2	245.00	15891.64	92.95	10.08
	3	529.00	43501.27	99.89	42.61
	4	612.67	40932.75	92.88	37.27
S1F4PF	1	94.85	9109.76	83.2	15.50
	2	274.41	13641.50	61.22	11.95
	3	425.34	35804.00	93.54	12.73
	4	591.40	36054.34	33.17	32.97
S1F7PA	1	84.29	8668.14	71.40	5.84
	2	162.88	10835.16	73.50	5.46
	3	579.49	30097.71	58.36	12.29
	4	668.72	37802.65	61.00	19.26

6.7. Investigation of 5-storey steel structure with concentric brace

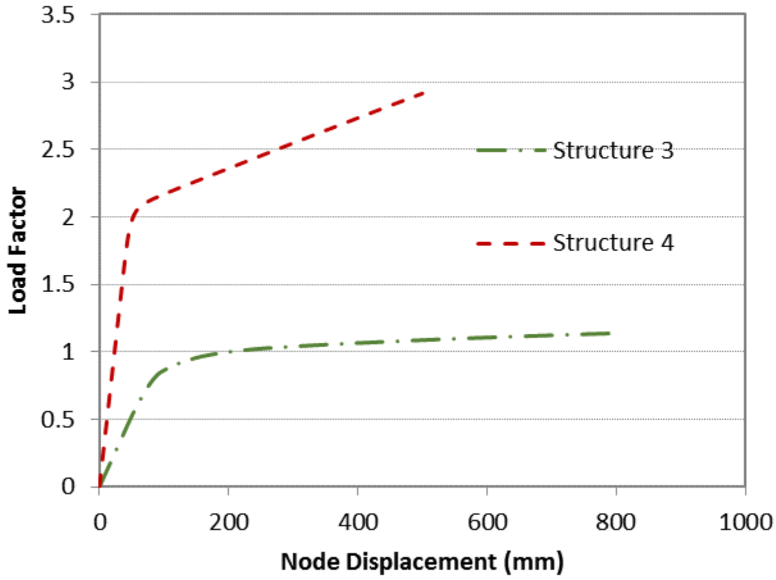
In Fig. 6.11 the capacity curves of two irregular 5-storey structures have been shown for C and E seismic regions, respectively. In Fig. 6.12, a comparison between two 5-storey irregular moment-resisting frame and braced structures (ASCE 7-10) has been given.



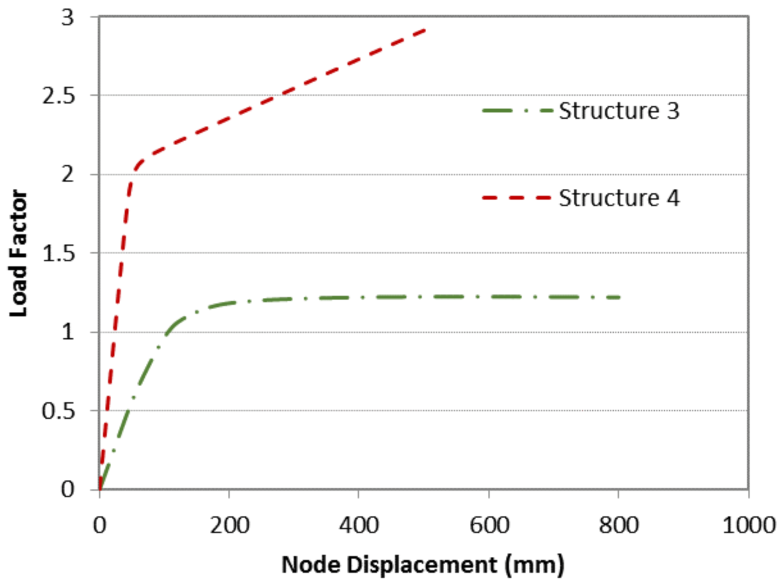
Pushdown capacity curve in Scenario S1F1PA



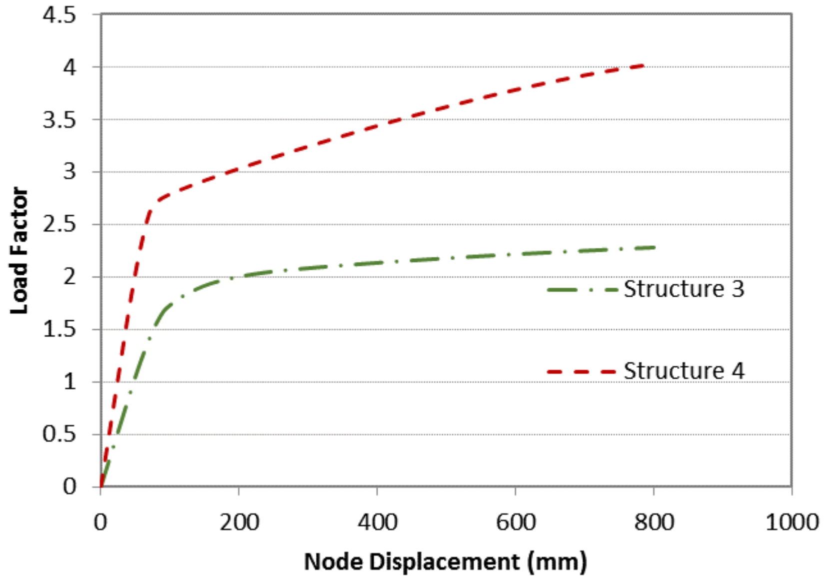
Pushdown capacity curve in Scenario S1F1PB



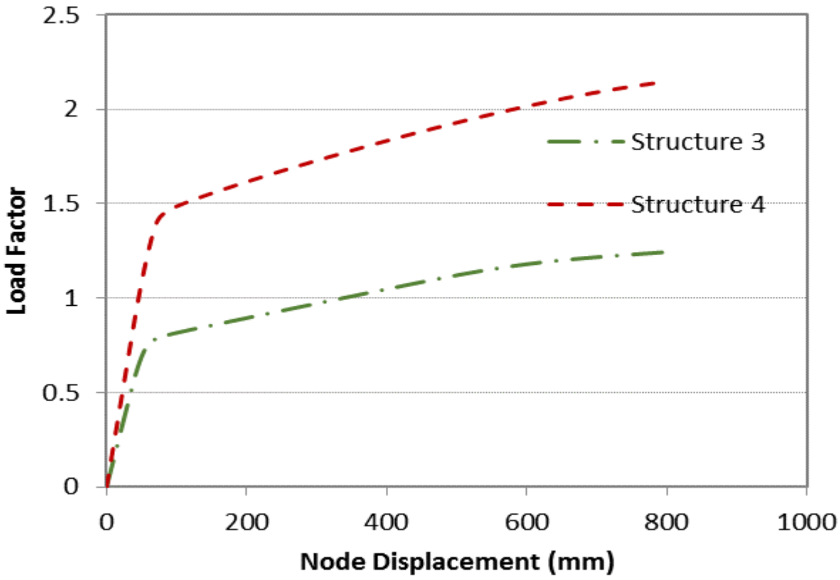
Pushdown capacity curve in Scenario S1F4PA



Pushdown capacity curve in Scenario S1F4PD



Pushdown capacity curve in Scenario S1F4PF



Pushdown capacity curve in Scenario S1F7PA

Figure 6.11 Pushdown capacity curve for six removal column scenario

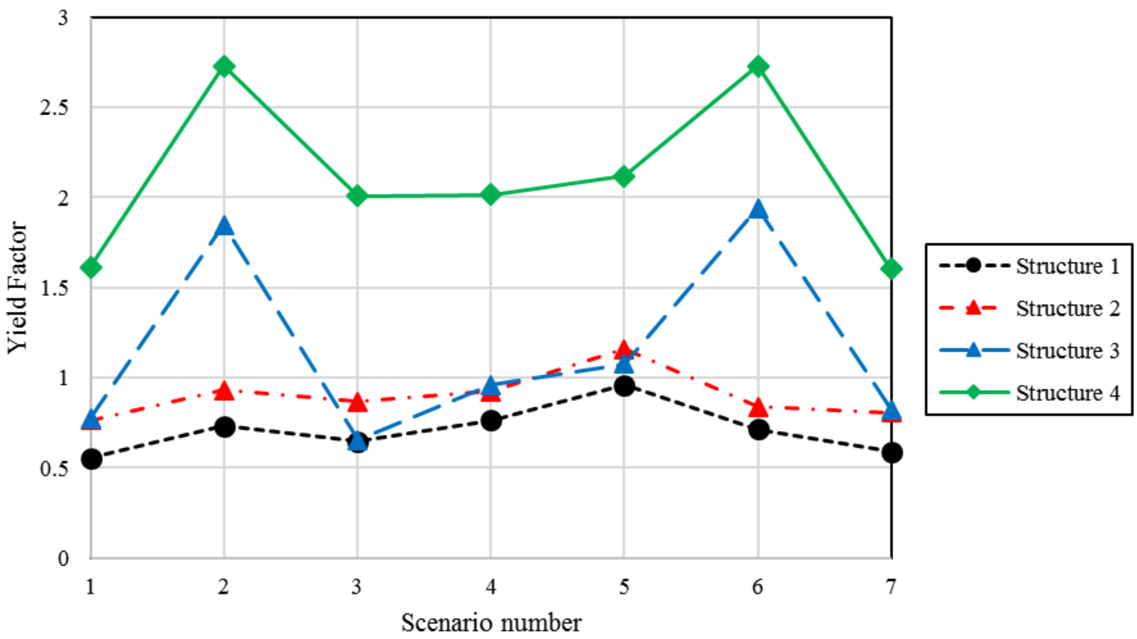


Figure 6.12 Load factors for all the structures and scenarios

In Fig. 6.12 a comparison between two 5-storey braced and unbraced structures has been given. As discussed earlier, by increasing the height and formation the floor and beam gird along the height, the structure managed to prevent collapse and yield progress to some extent. However, still in irregular structures this technique of stiffness increase along the height failed to meet the column removal load bearing. The irregular structure managed to achieve a force ratio higher than 1 in most scenarios through distributing braces in the plan and height. In Structure 4, under all column removal scenarios, a load factor higher than 1.5 was achieved. Merely in 4 scenarios representing removal of corner and side columns in Structure 3 the relevant structure failed to have the ability to confront the force caused by column removal.

Chapter 7: NONLINEAR DYNAMIC ANALYSIS RESULTS

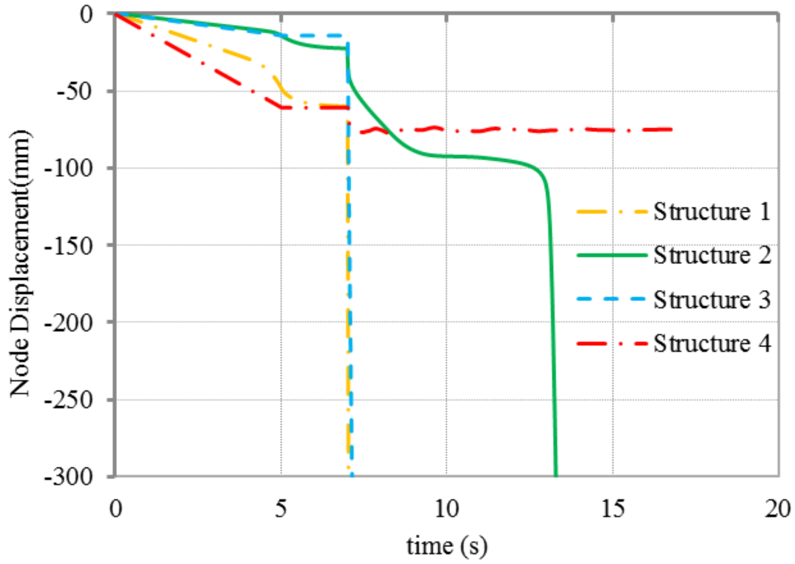
7.1. Introduction

As mentioned earlier, the most precise method in assessing the structure capacity and ability against sudden load and examining progressive collapse phenomenon is taking benefit from nonlinear dynamic approach. Although sudden column removal is a quite proper scenario to examine the structures behaviour against progressive collapse, it is not a scenario similar to dynamic effect of column removal due to strike or explosion. However, this scenario may give the effect of column yield and destruction within a relatively short period of time with respect to the structure response. Therefore, sudden losing column is used as the main design scenario in this guideline (GSA (2013)). Meanwhile, according to the most of designers, the fact that they may make the structure resistant against critical column removal does matter. Thus, the structure ability under sudden column removal by taking benefit from nonlinear dynamic analysis approach may be studied through 3D finite element method (FEM), while in this part the 3D behaviour of fourteen 2-, 3- and 5-storey regular and irregular steel structures with moment-resisting frame system is focused.

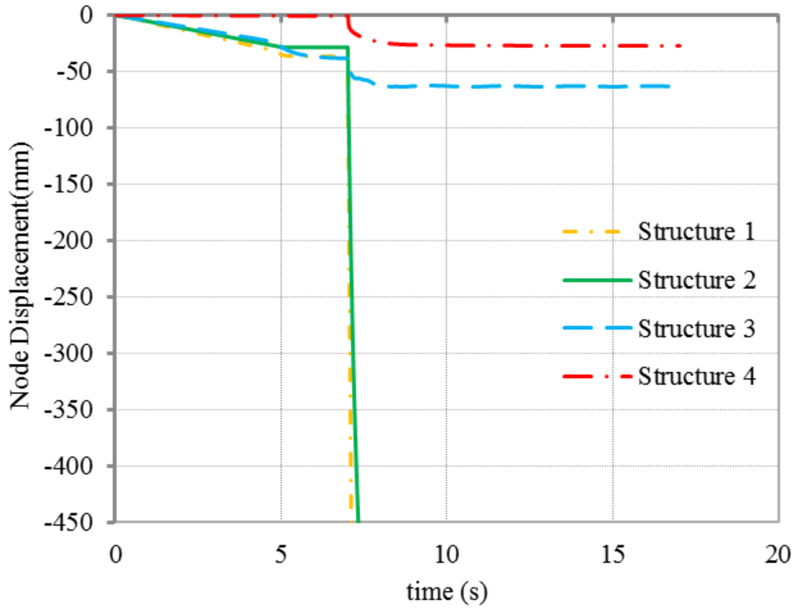
7.2. Results of 2-Storey Steel Structure with Moment-resisting Frame System

7.2.1. Node Displacement at Column Removal Position

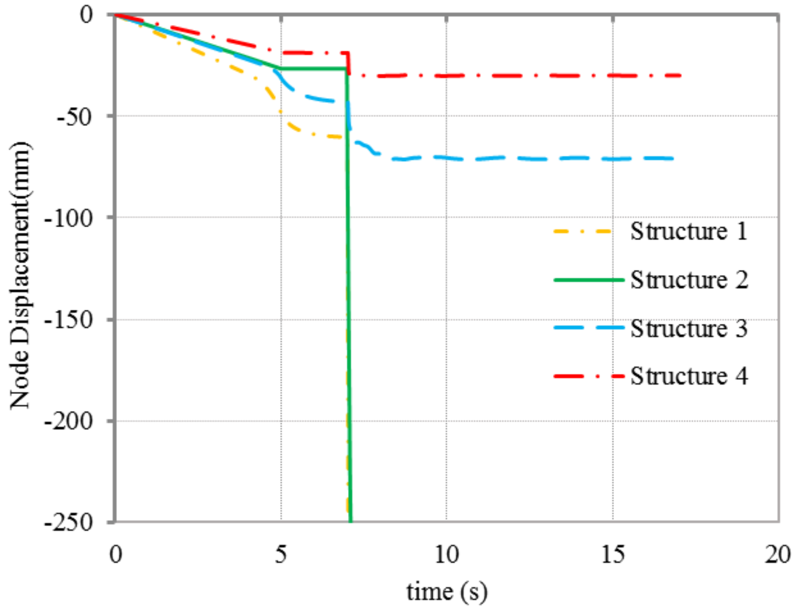
The results have been given for each structure within the framework of node vertical displacement changes at column removal position, D/C ratio, connection rotation and column and beam plastic strain. Initially, in Fig. 7.1 the node displacement at column removal position has been given for different column removal scenarios.



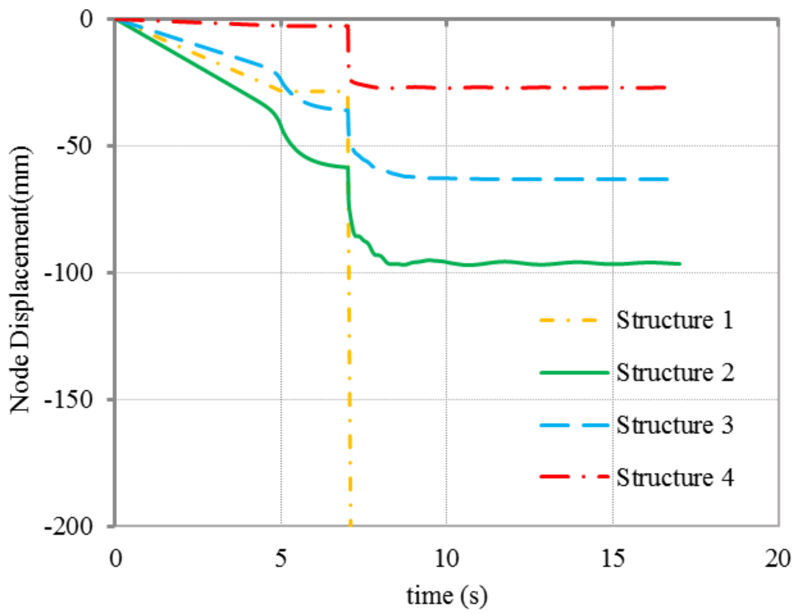
(a)S1F1PA



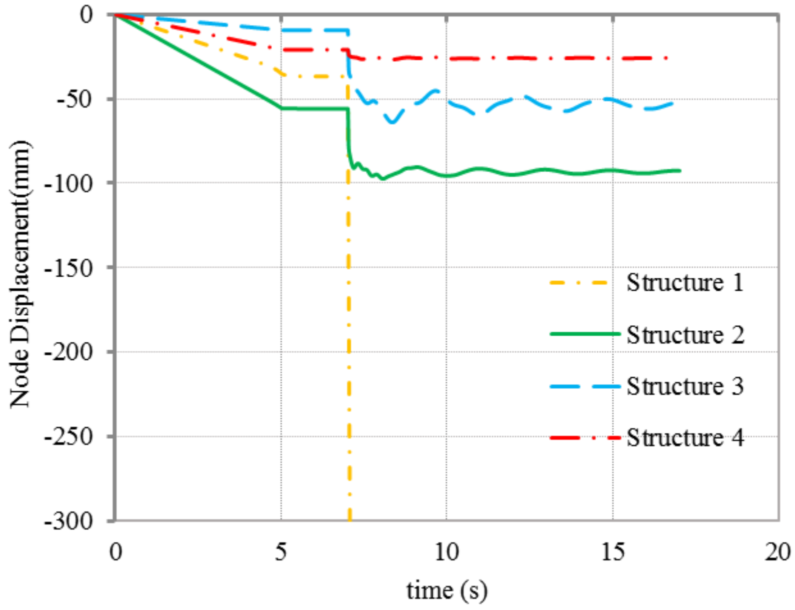
b) S1F1PB



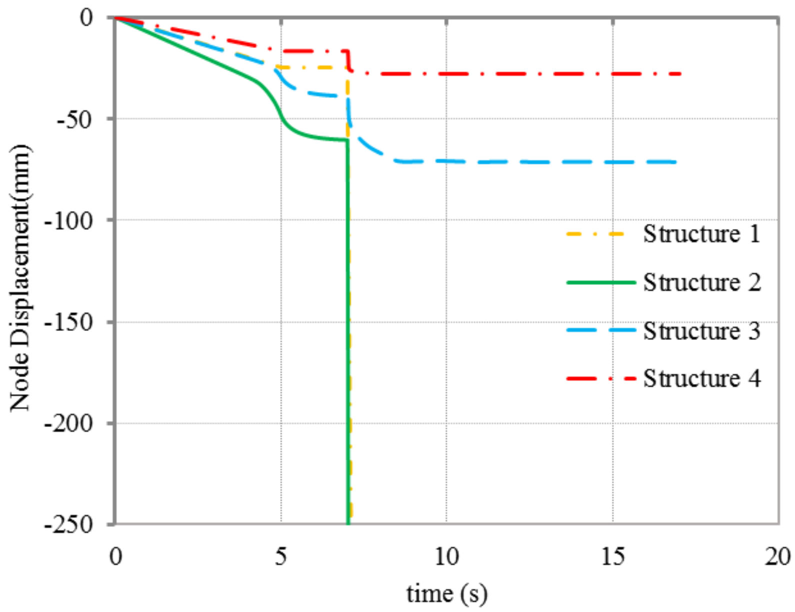
c) S1F1PD



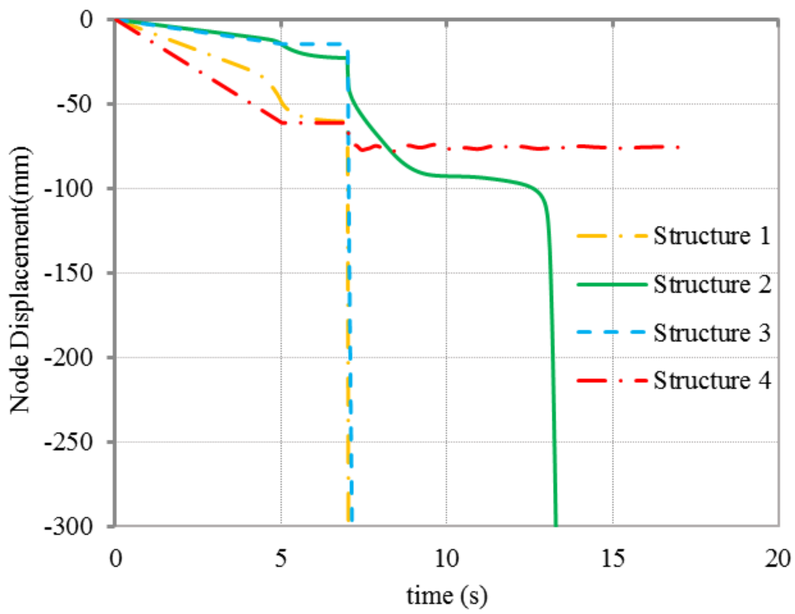
d) S1F4PA



e) S1F4PD



f) S1F4PF



g) S1F7PA

Figure 7.1 Vertical displacement of removal point in several scenarios.

It is seen in most of the displacement diagrams in Fig. 7.1 that upon column removal at 7th second in 1 and 2 irregular structures, the relevant structure cannot bear the deformation caused by column removal and is exposed to severe deformation resulting in building collapse. Also the column removal spot affect node final deformation. Generally, removal of S1F1PA, S1F1PD and S1F7PA corner columns has resulted in collapse and failed bearing by the structure under deformation caused by column removal for irregular structures. Upon middle columns removal, merely irregular structure 2 situated in E seismic region failed to withstand sudden deformation. Additionally, the corner column removal importance in A and 1 two axes junction as well A and 7 axes junction is further emphasized when regular structure 3 situated in C seismic region also failed to withstand sudden load caused by columns removal.

7.2.2. Calculating values for connection rotation and strain level in beam and column members

Generally building structures are designed to meet a certain performance level such as life safety. In case a given structure elements exceed Collapse Prevention level structure yields and collapses. In the thesis, the structural performance of all the structural elements was controlled in each analysis step based on the acceptance criterion. Here, the connection rotation values have been adopted as per GSA (2013) Standard.

First of all, the plastic rotation and nonlinear acceptance criterion are presented as per calculated beam cross section for steel structure connections in Table 7.1. Beam web is the only main parameter in Eq. 7.1. Evidently, beam members in all frames are considered as primary members. The addressed materials are related to two-storey buildings.

$$0.0284-0.0004d \quad \text{(Unit: inch)} \tag{7.1}$$

Table 7.1 Calculating plastic rotation of beams

Depth of beam (mm)	Plastic rotation angle (Radians)
266	0.024211
270	0.024148
274	0.024085
280	0.023991

The acceptable values of connections rotation for each floor have been given in Table 7.2.

Table 7.2 Plastic rotation angle acceptance of connections in GSA-2013

	1st	**2nd**
Structure 1	0.024211	0.024211
Structure 2	0.024211	0.024211
Structure 3	0.024211	0.024211
Structure 4	0.024211	0.024211

Plastic rotation of adjacent connection at removed column spot has been determined for different floors, which have been given in Table 7.3. The numbers indicate that the connection rotation has exceeded the acceptable value stipulated in Table 7.2.

Table 7.3 Plastic rotation angle of connections

Scenario	Type of structure	Storey	
		1st	2nd
S1F7PA	Structure 1	0.04039	0.03787
	Structure 2	0.03338	0.03144
	Structure 3	0.03124	0.03057
	Structure 4	0.02389	0.02265
S1F4PA	Structure 1	0.03596	0.03433
	Structure 2	0.02569	0.024359
	Structure 3	0.02365	0.02266
	Structure 4	0.02166	0.02027
S1F1PA	Structure 1	0.04091	0.03723
	Structure 2	0.03525	0.03122
	Structure 3	0.03181	0.03036
	Structure 4	0.02398	0.02256
S1F1PB	Structure 1	0.03846	0.03715
	Structure 2	0.03488	0.03226
	Structure 3	0.02387	0.02227
	Structure 4	0.02157	0.02101
S1F1PD	Structure 1	0.04017	0.03722
	Structure 2	0.03563	0.03308
	Structure 3	0.02397	0.02256
	Structure 4	0.02216	0.02106
S1F4PD	Structure 1	0.04459	0.03844
	Structure 2	0.02517	0.02426
	Structure 3	0.02322	0.02157
	Structure 4	0.02201	0.02137
S1F4PF	Structure 1	0.03902	0.03783
	Structure 2	0.04023	0.03882
	Structure 3	0.02391	0.02232
	Structure 4	0.02266	0.02197

Maximum strain level in adjacent column to the removed column has been given in the following tables. It should be note that strain yielding, strain hardening and ultimate strain are 0.0012, 0.015 and 0.2, respectively.

In Table 7.4 the maximum generated plastic strain in beams at adjacent to removed column has been given. It is seen considering the results that in all cases of column removal scenarios, the beams exceeded the strain representing yield stress. However, none has reached final strain of 0.2, i.e. under column removal conditions, due to severe load applying caused by such removal, a severe stress is imposed to beam and column connection, causing earlier yield of connection in some cases comparison to that of beam, which in turn results in preventing the beam to achieve its maximum capacity and final strain.

Table 7.4 Strain level in column element adjacent to removed column

Scenario	Type of structure	Storey	
		1st	2^{nd}
S1F7PA	Structure 1	0.04288	0.04154
	Structure 2	0.041659	0.039865
	Structure 3	0.0396854	0.038659
	Structure 4	0.0386514	0.0376902
S1F4PA	Structure 1	0.03886	0.03752
	Structure 2	0.038602	0.036598
	Structure 3	0.038659	0.036854
	Structure 4	0.037986	0.036932
S1F1PA	Structure 1	0.043569	0.0413265
	Structure 2	0.0421569	0.0402365
	Structure 3	0.0398654	0.038111
	Structure 4	0.0379865	0.0369887
S1F1PB	Structure 1	0.0369865	0.03125688
	Structure 2	0.03752	0.03484
	Structure 3	0.03484	0.03216
	Structure 4	0.03386	0.03152

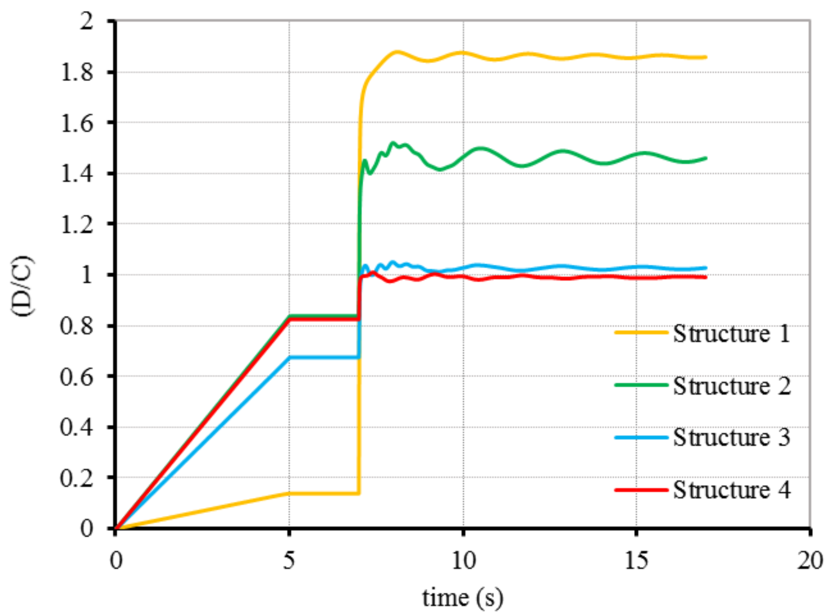
S1F1PD	Structure 1	0.04493	0.04154
	Structure 2	0.041678	0.040159
	Structure 3	0.022462	0.02005
	Structure 4	0.02546	0.02144
S1F4PD	Structure 1	0.038154	0.03684
	Structure 2	0.03684	0.03448
	Structure 3	0.02946	0.0287
	Structure 4	0.028484	0.026814
S1F4PF	Structure 1	0.03658	0.03454
	Structure 2	0.03546	0.033742
	Structure 3	0.012596	0.011988
	Structure 4	0.012546	0.012278

Table 7.5 Strain in beam element adjacent to removed column

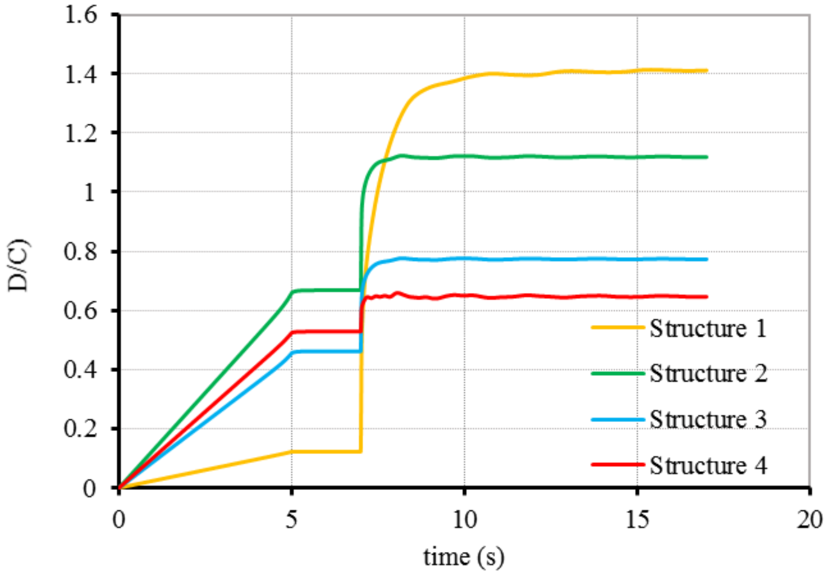
Scenario	Type of structure	Storey	
		1st	2^{nd}
S1F7PA	Structure 1	0.040086238	0.038935398
	Structure 2	0.0384498	0.037202457
	Structure 3	0.03647319	0.03242034
	Structure 4	0.03588648	0.03221001
S1F4PA	Structure 1	0.036636365	0.035488173
	Structure 2	0.034577462	0.033429268
	Structure 3	0.03519399	0.03399105
	Structure 4	0.032422554	0.02971188
S1F1PA	Structure 1	0.04139811	0.03897747
	Structure 2	0.04032678	0.03747564
	Structure 3	0.04016442	0.03905619
	Structure 4	0.03816198	0.03536373
S1F1PB	Structure 1	0.0364449	0.03499719
	Structure 2	0.03521613	0.03263313
	Structure 3	0.033186495	0.030887462
	Structure 4	0.03319155	0.03152613

S1F1PD	Structure 1	0.04014843	0.03821487
	Structure 2	0.03994794	0.037025829
	Structure 3	0.02909934	0.02779554
	Structure 4	0.0266295	0.02505387
S1F4PD	Structure 1	0.03319155	0.03154581
	Structure 2	0.03254457	0.028588429
	Structure 3	0.02663934	0.02417442
	Structure 4	0.02538105	0.0226689
S1F4PF	Structure 1	0.03645105	0.02872788
	Structure 2	0.03521982	0.03094557
	Structure 3	0.014103731	0.012722724
	Structure 4	0.0155349	0.013821387

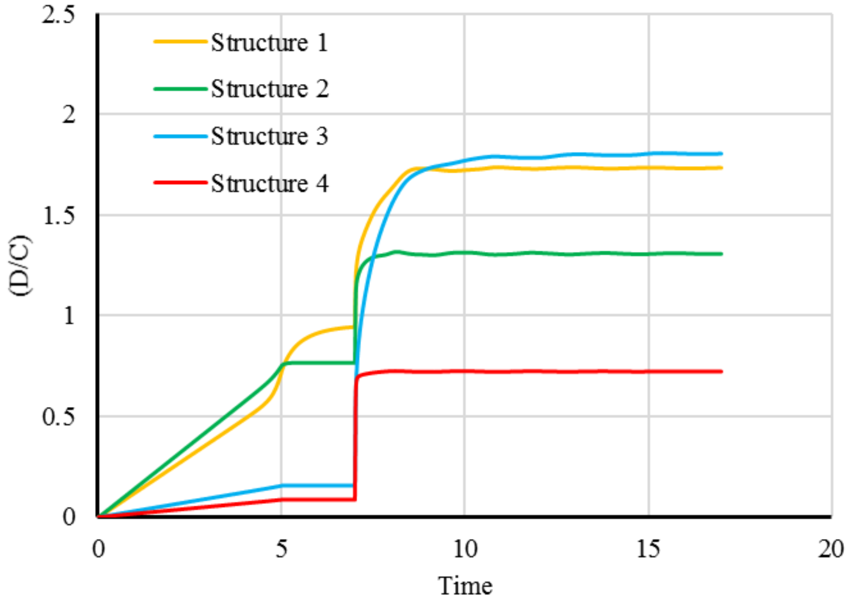
7.2.3. D/C ratio for adjacent columns to the removed column



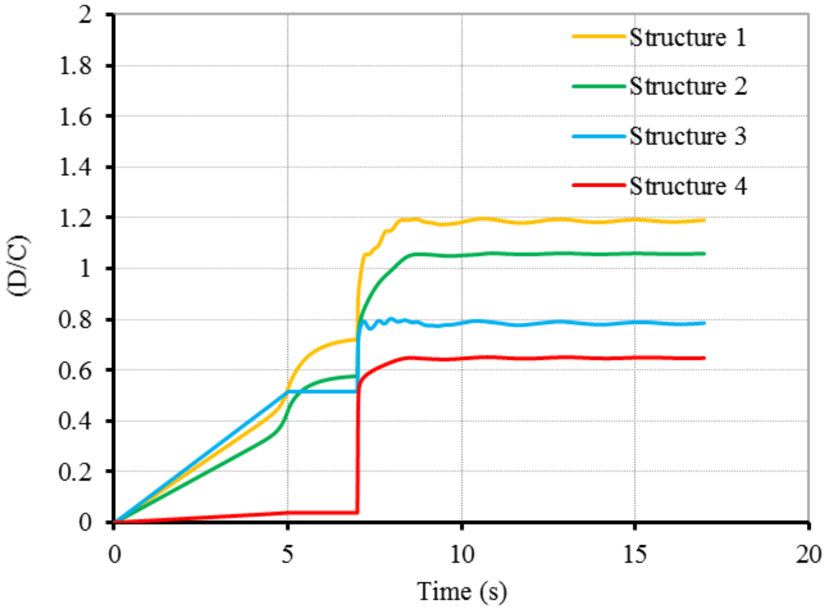
a) Scenario S1F1PA



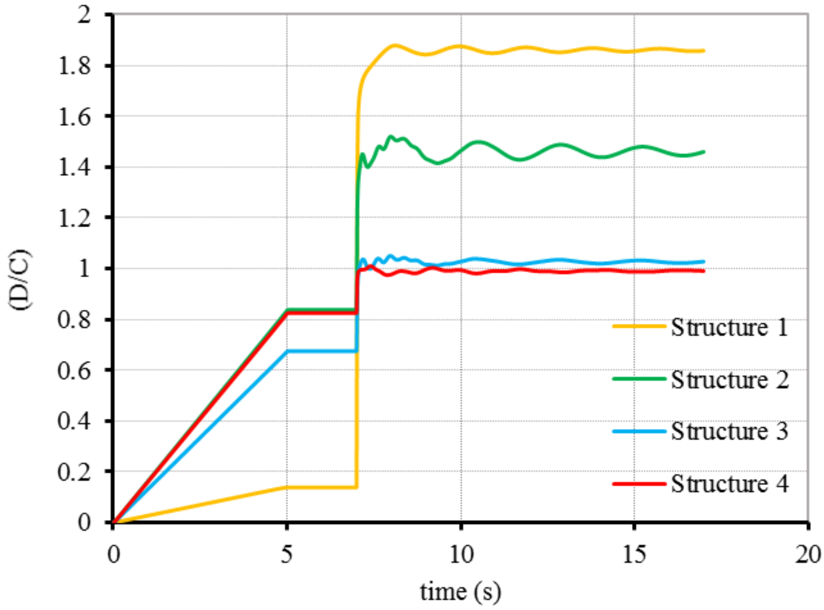
b) Scenario S1F1PB



c) Scenario S1F1PD



d) Scenario S1F4PD



e) Scenario S1F7PA

Figure 7.2 The demand force to capacity ratio (D/C) of the adjacent columns in different scenario

According to results given in Fig. 7.2, upon column removal in both corners in A axis junction with two 1 and 7 axes in two irregular structures 1 and 2, D/C ratio exceeds 1, while in case of peripheral or internal column removal in some cases the D/C ratio for adjacent column the ratio approaches 1. Upon two corner columns removal, even regular structure 3 experiences the ratio to be a higher than 1, on the other hand regular structure 4 has a less ratio than 1.

It may be pointed out that in irregular structure 1, the highest demand or force due to sudden removal adjacent column is generated which may be well seen in sudden positive jump in the curves. In Table 7.6 a brief on results of 2-storey structure had been given.

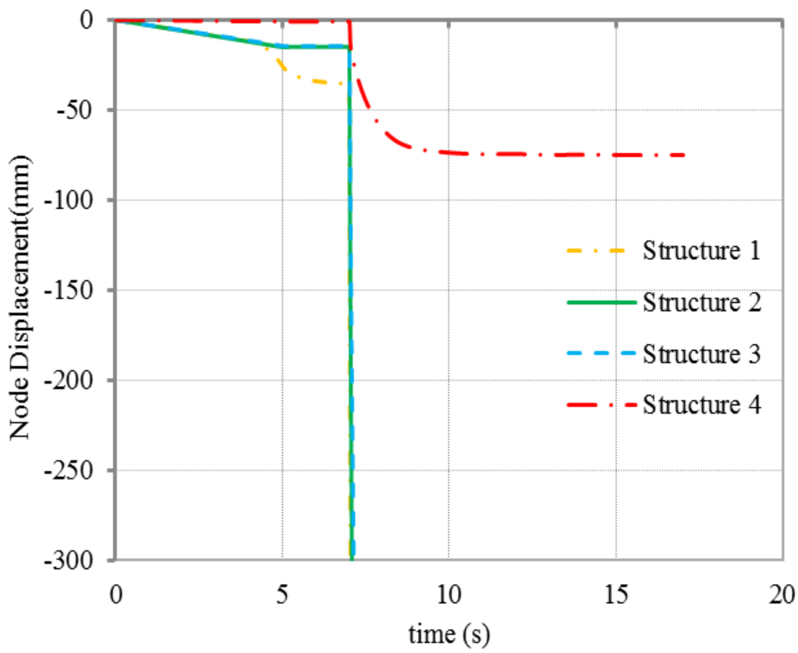
Table 7.6 Node displacement and maximum D/C ratio of adjacent columns for all the scenarios and structures

Scenario	Structure 1		Structure 2		Structure 3		Structure 4	
	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C
S1F1PA	Fail	1.89	Fail	1.55	Fail	1.16	78	1.01
S1F1PB	Fail	1.42	Fail	1.12	64	0.79	27	0.66
S1F1PD	Fail	1.75	Fail	1.32	71.1	1.82	30	0.72
S1F4PA	Fail	1.33	97	1.09	63.4	0.78	27.1	0.67
S1F4PD	Fail	1.2	95.5	1.07	64.2	0.79	27	0.66
S1F4PF	Fail	1.47	Fail	1.15	71	0.82	28	0.67
S1F7PA	Fail	1.87	Fail	1.52	Fail	1.16	78	1.01

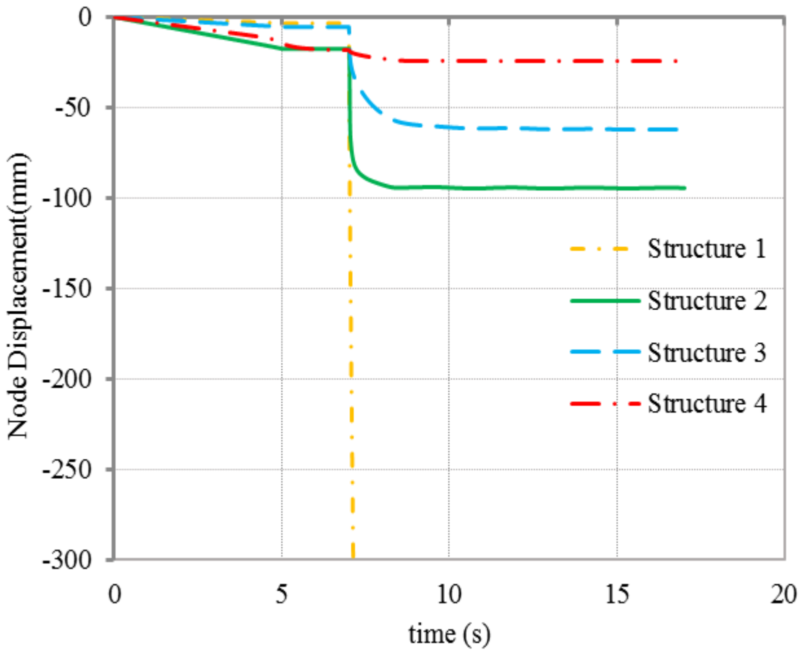
7.3. Results of 3-storey steel structure with moment-resisting frame system

7.3.1. Node displacement at column removal position

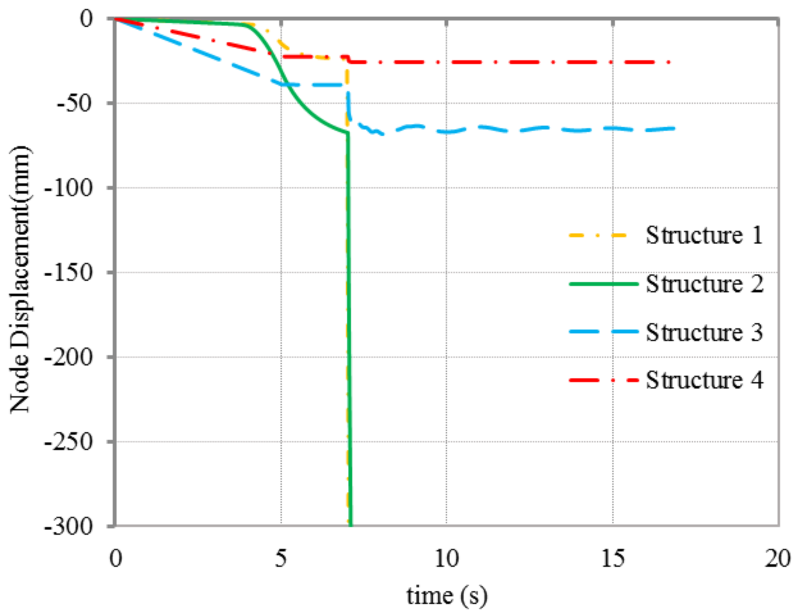
The results have been given for each structure within the framework of node vertical displacement changes at column removal position, D/C ratio, connection rotation and column plastic strain. Initially, in Fig. 7.4 the node displacement at column removal position has been given for different column removal scenarios.



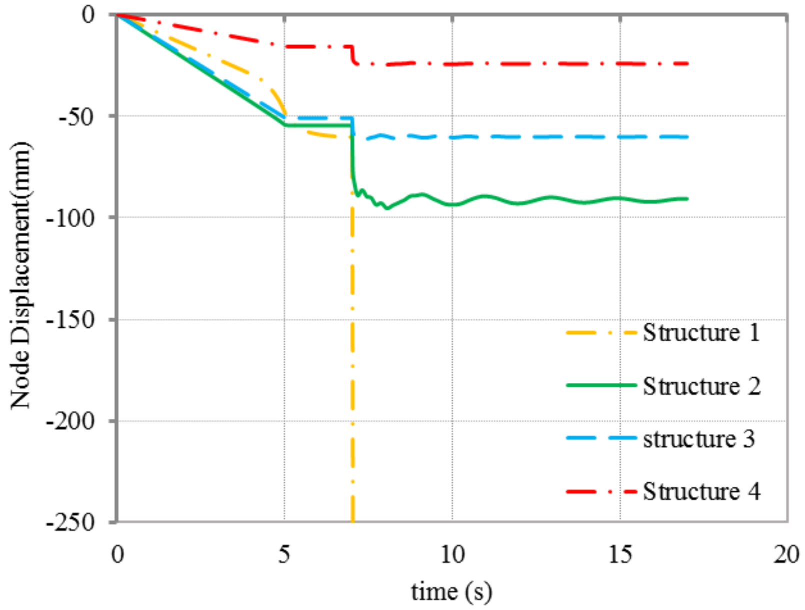
a) S1F1PA



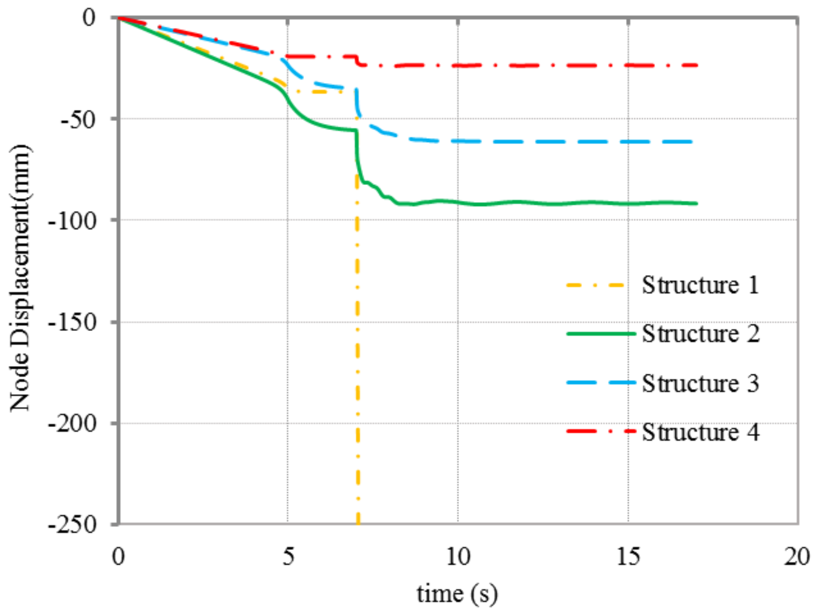
b) S1F1PB



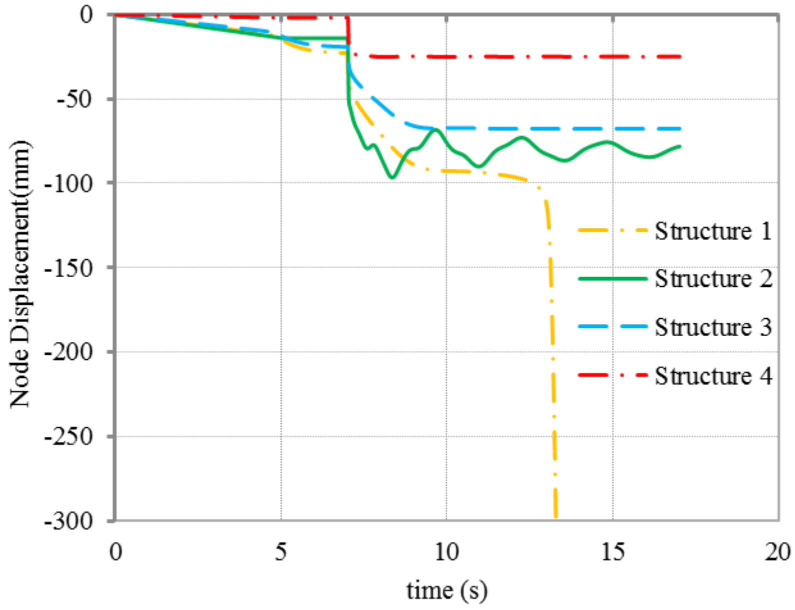
c) S1F1PD



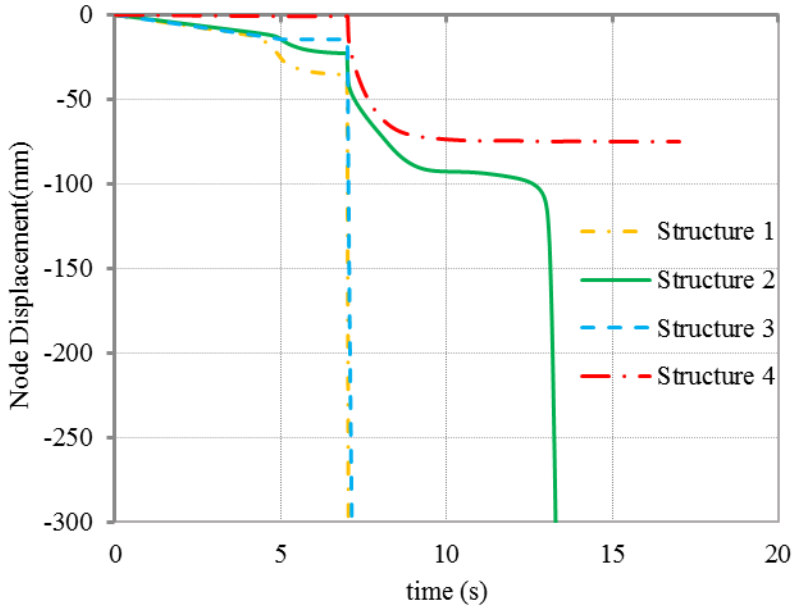
d) S1F4PA



e) S1F4PD



f) S1F4PF



g) S1F7PA

Figure 7.3 Vertical displacement of removal point in several scenarios.

Like 2-storey structure, a similar trend been observed in these regular and irregular 3-storey structures. In case of corner columns removal, the maximum displacement is seen in irregular structures. For instance, in case of column removal under S1F7PA scenario, merely regular structure 4 managed to withstand sudden deformation caused by the removal of column. At 7^{th} second the displacement at removed column location node reaches from roughly 1mm to 73mm. However, such a thing has been observed in regular structure 3 and two irregular structures 1 and 2. At 7^{th} second and before column removal, structures 1, 2 and 3 have had displacements equal to 12.24, 22.2 and 35mm, respectively.

In S1F4PA scenario, two regular structures 3 and 4 representing 17 and 52% increased displacement, respectively due to column removal. Meanwhile, irregular structure 2 managed to reach from 53mm to 93mm against node displacement at column removal and withstand against sudden failure. However, irregular structure 1 failed to have such withstanding.

7.3.2. Calculating values for connection rotation and strain level in beam and columns

In the following the results on rotation of connections adjacent to removed column in 3-storey structure are given, the acceptable values for connections rotation per floor has been given in Tables 7.7 and 7.8. According to Table 7-8, the joint adjacent to the removed column in both irregular structures 1 and 2 did not satisfy the acceptance plastic rotation of connections (according to Table 7-7) under column-removal scenarios. Moreover, under the two corner-column-removal scenarios, it was seen that the connections of the two regular structures located in C Seismic region could not satisfy the acceptance plastic rotation.

Table 7.7 Plastic rotation angle acceptance of connections in GSA

	1st	2nd	3rd
Structure 1	0.024211	0.024211	0.024211
Structure 2	0.024148	0.024211	0.024211
Structure 3	0.024211	0.024211	0.024211
Structure 4	0.024148	0.024211	0.024211

Table 7.8 Plastic rotation angle of connections

Scenario	Type of structure	Storey		
		1st	2nd	3rd
S1F7PA	Structure 1	0.03897	0.03746	0.03261
	Structure 2	0.03099	0.02966	0.02728
	Structure 3	0.02822	0.02786	0.02607
	Structure 4	0.02397	0.02376	0.02316
S1F4PA	Structure 1	0.03329	0.03213	0.02849
	Structure 2	0.02426	0.02401	0.02312
	Structure 3	0.02387	0.02316	0.02237
	Structure 4	0.02104	0.02031	0.02011
S1F1PA	Structure 1	0.03821	0.03772	0.03194
	Structure 2	0.03079	0.02898	0.02712
	Structure 3	0.02836	0.02711	0.02598
	Structure 4	0.02398	0.02302	0.02296
S1F1PB	Structure 1	0.03479	0.03213	0.02938
	Structure 2	0.02469	0.02441	0.02391
	Structure 3	0.02363	0.02307	0.02257
	Structure 4	0.02174	0.02111	0.02043
S1F1PD	Structure 1	0.03813	0.03713	0.03839
	Structure 2	0.03329	0.03293	0.02994
	Structure 3	0.02361	0.02269	0.02114
	Structure 4	0.02186	0.02111	0.02107
S1F4PD	Structure 1	0.32877	0.02939	0.02849
	Structure 2	0.02469	0.02412	0.02389
	Structure 3	0.02313	0.02301	0.02257
	Structure 4	0.02122	0.02017	0.02001
S1F4PF	Structure 1	0.03111	0.0302	0.02813
	Structure 2	0.02451	0.02432	0.02323
	Structure 3	0.02223	0.02116	0.02101
	Structure 4	0.02104	0.02087	0.02012

In the following the beam and column strains adjacent to the removed column are given for 3-storey structure.

Table 7.9 Strain level in column element adjacent to removed column

Scenario	Type of structure	Storey		
		1st	2nd	3rd
S1F7PA	Structure 1	0.03760259	0.036365009	0.030151889
	Structure 2	0.036474914	0.034818033	0.02886922
	Structure 3	0.03465216	0.03370421	0.027945699
	Structure 4	0.033697191	0.032809458	0.027203819
S1F4PA	Structure 1	0.033889847	0.032652266	0.027073485
	Structure 2	0.033651567	0.031800737	0.026367442
	Structure 3	0.03370421	0.03203717	0.02656348
	Structure 4	0.033082649	0.032109208	0.02662321
S1F1PA	Structure 1	0.038238928	0.036167827	0.029988397
	Structure 2	0.036934758	0.035161138	0.029153705
	Structure 3	0.034818402	0.033198095	0.027526056
	Structure 4	0.033083111	0.032161575	0.02666663
S1F1PB	Structure 1	0.032159543	0.02686785	0.022277361
	Structure 2	0.032652266	0.030177104	0.025021215
	Structure 3	0.030177104	0.027701943	0.022968945
	Structure 4	0.029272008	0.027110859	0.022478851
S1F1PD	Structure 1	0.039495904	0.036365009	0.030151889
	Structure 2	0.036492462	0.035089562	0.029094358
	Structure 3	0.018745181	0.016517536	0.013695443
	Structure 4	0.021514038	0.017801295	0.014759866
S1F4PD	Structure 1	0.033237808	0.03202424	0.026552759
	Structure 2	0.03202424	0.02984462	0.024745537
	Structure 3	0.025208309	0.024506398	0.020319373
	Structure 4	0.024306907	0.022764549	0.018875127

S1F4PF	Structure 1	0.031784113	0.029900034	0.024791483
	Structure 2	0.030749717	0.029163027	0.024180397
	Structure 3	0.011633261	0.011071732	0.009180078
	Structure 4	0.011587082	0.011339566	0.009402152

Table 7.10 Strain level in beam element adjacent to removed column

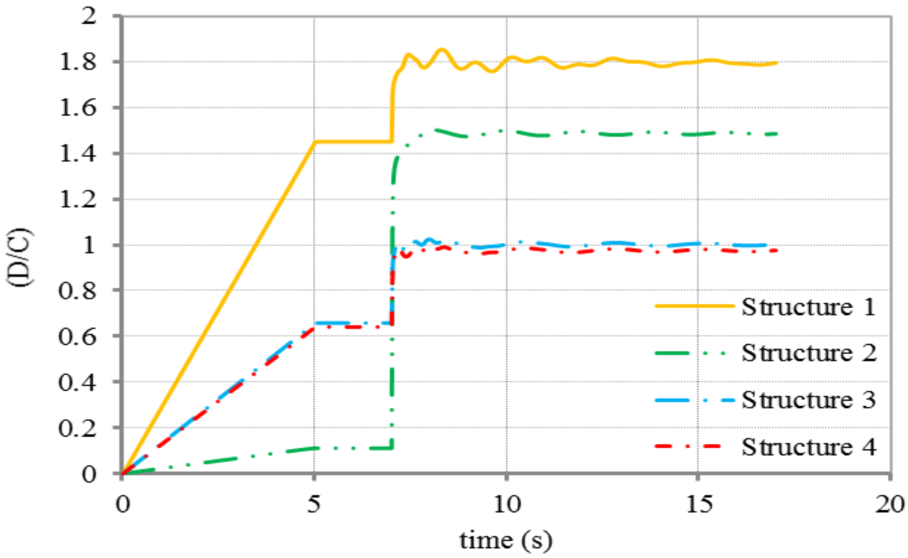
Scenario	Type of structure	Storey		
		1st	2nd	3rd
S1F7PA	Structure 1	0.034692185	0.033696204	0.029677694
	Structure 2	0.033275948	0.032196449	0.028466164
	Structure 3	0.031565313	0.02805782	0.027002788
	Structure 4	0.031057552	0.027875792	0.02656842
S1F4PA	Structure 1	0.031706532	0.030712841	0.027123594
	Structure 2	0.029924677	0.028930986	0.025599293
	Structure 3	0.030458244	0.029417173	0.026055737
	Structure 4	0.028059736	0.025713813	0.024003915
S1F1PA	Structure 1	0.03582753	0.033732614	0.030648934
	Structure 2	0.03490036	0.032432872	0.029855779
	Structure 3	0.034759847	0.033800742	0.029735576
	Structure 4	0.033026858	0.030605144	0.028253077
S1F1PB	Structure 1	0.03154083	0.030287926	0.026981844
	Structure 2	0.030477405	0.028241976	0.026072129
	Structure 3	0.02872088	0.026731208	0.024569496
	Structure 4	0.028725255	0.027283936	0.024573238
S1F1PD	Structure 1	0.034746009	0.033072631	0.029723738
	Structure 2	0.034572497	0.032043589	0.029575306
	Structure 3	0.025183698	0.024055338	0.021543586
	Structure 4	0.023046202	0.021682591	0.01971505
S1F4PD	Structure 1	0.028725255	0.027300968	0.024573238
	Structure 2	0.028165333	0.024741535	0.024094249
	Structure 3	0.023054718	0.020921481	0.019722335
	Structure 4	0.021965745	0.019618545	0.018790764

S1F4PF	Structure 1	0.031546153	0.024862222	0.026986397
	Structure 2	0.030480598	0.026781497	0.02607486
	Structure 3	0.012205916	0.011010739	0.010441644
	Structure 4	0.013444505	0.011961564	0.011501204

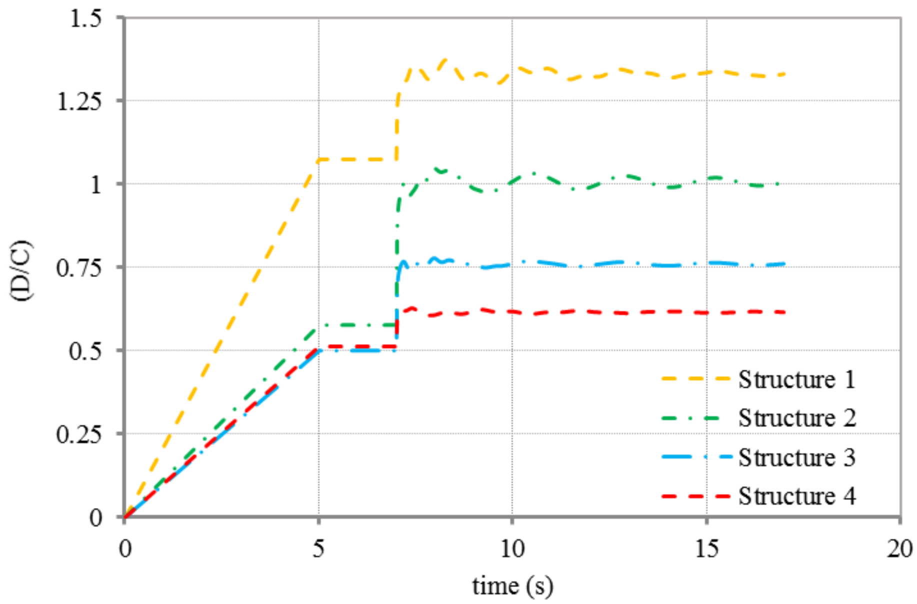
7.3.3. D/C ratio for adjacent columns to the removed column

When the structural elements are loaded beyond their ultimate capacities, the structure has its loading pattern or boundary conditions changed and hence the progressive collapse occurs and the structure fails. When any element fails, the remaining elements of the structure seek alternative load paths to redistribute the load applied to it. As a result, other elements may fail due to insufficient resistance capacity causing partial or total failure mechanism. It is a dynamic process, usually accompanied by large deformations, in which the collapsing system continually seeks alternative load paths in order to survive. One of the important characteristics of progressive collapse is that the final damage is not proportional to the initial damage.

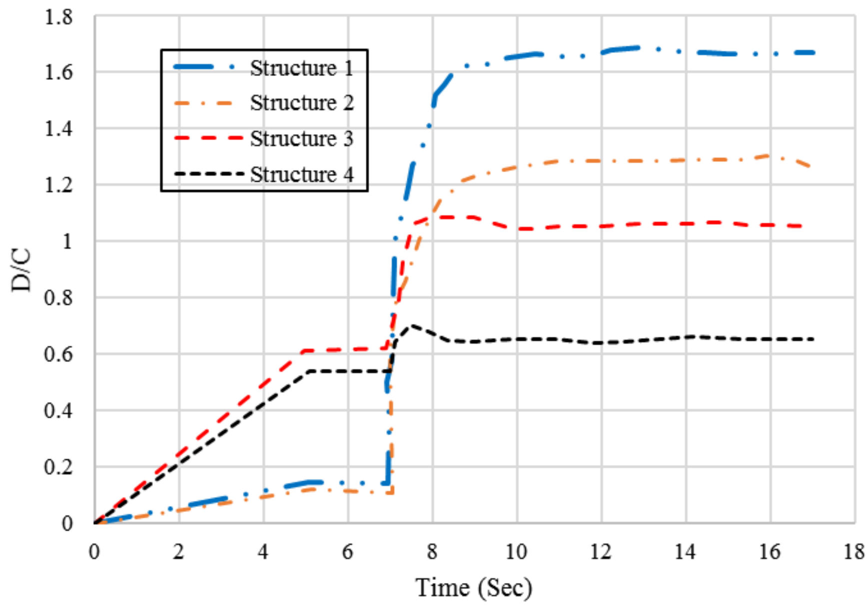
The results of analysis and also the values of DCR for columns are presented in this section. The susceptibility of different configuration (Irregular and regular building) with the same frame systems against progressive collapse has been assessed. DCR of primary elements (columns) are given with their specific details in all frames. According to nonlinear dynamic analysis procedure, the plastic hinge rotation exceeds the permissible values and hence the structure is said to be collapsed.



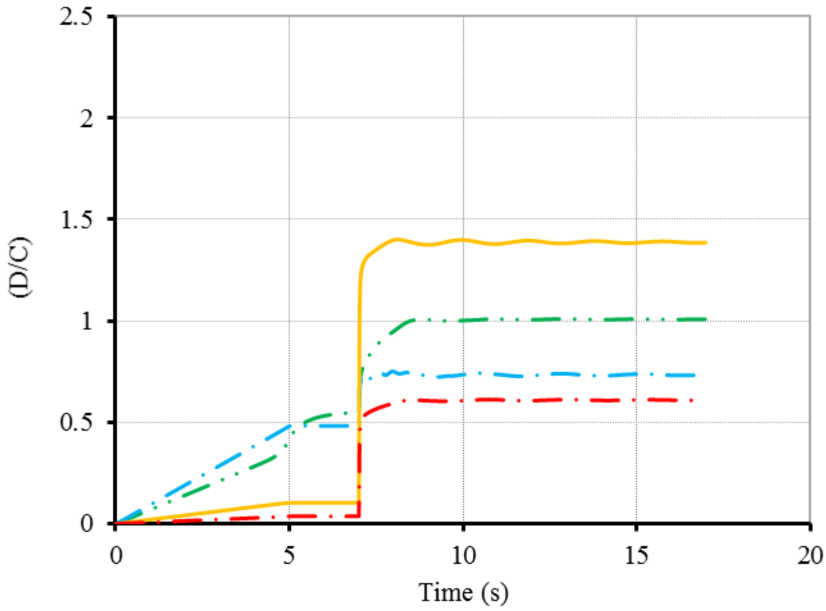
a) Scenario S1F1PA



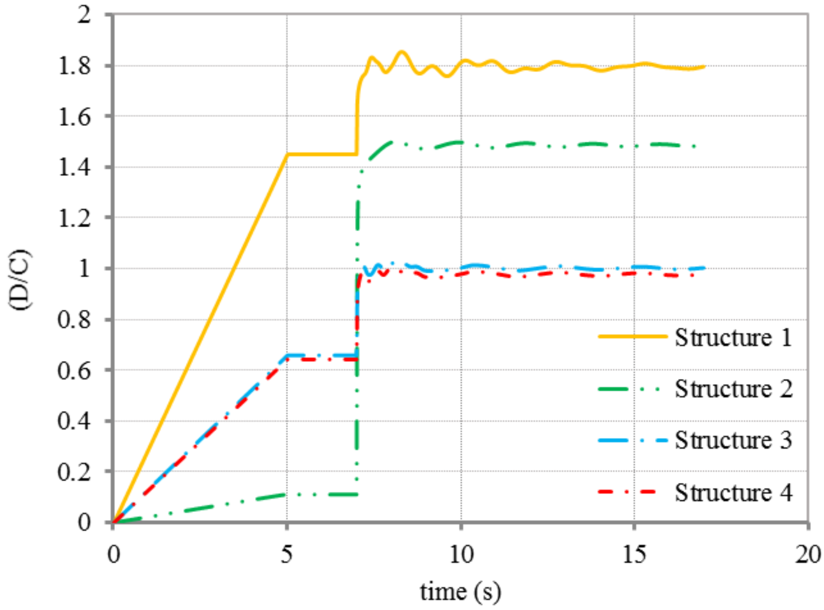
b) Scenario S1F1PB



c) Scenario S1F1PD



d) Scenario S1F4PD



e) Scenario S1F7PA

Figure 7.4 The demand force to capacity ratio (D/C) of the adjacent columns in different scenario

Like the results expressed for 2-storey structures, it is observed regarding 3-storey regular and irregular structures that upon corner columns removal at the structure plan situated on the first floor the maximum D/C ratio and sudden change in the load imposed to the column are related to irregular structures 1 and 2. In two cases of corner column removal at A axis junctions with 1 and 7 axes, irregular structure 1 has a D/C ratio higher than 1.

Table 7.11 Node displacement and maximum D/C ratio of adjacent columns for all the scenarios and structures

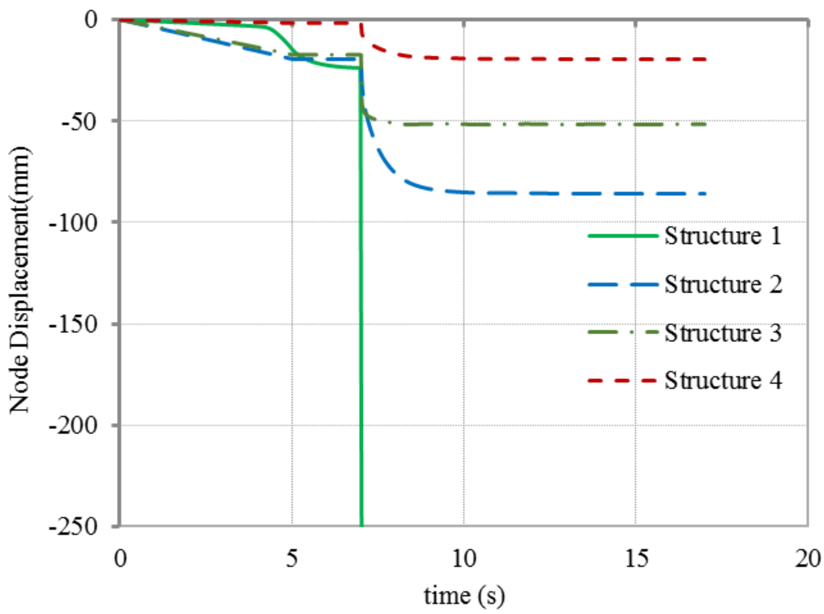
Scenario	Structure 1		Structure 2		Structure 3		Structure 4	
	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C
S1F1PA	Fail	1.85	Fail	1.51	Fail	1.13	75	0.99
S1F1PB	Fail	1.36	95	1.07	62	0.77	24.1	0.62
S1F1PD	Fail	1.68	Fail	1.29	68.2	1.79	26	0.68
S1F4PA	Fail	1.27	93	1.04	61.5	0.77	24.2	0.61
S1F4PD	Fail	1.16	92	1.03	61	0.75	24	0.57
S1F4PF	Fail	1.41	96.5	1.09	68	0.77	25	0.63
S1F7PA	Fail	1.8	Fail	1.45	Fail	1.14	75	0.99

As per Table 7.11 it is seen that irregular structure 1 has exposed to collapse under all column removal states, while irregular structure 2 has merely been subject to damages and severe deformation due to removal of corner columns. On the other hand, under all scenarios regular structure 4 managed to withstand the load caused by column removal in the adjacent column and the D/C ratio remained less than 1.

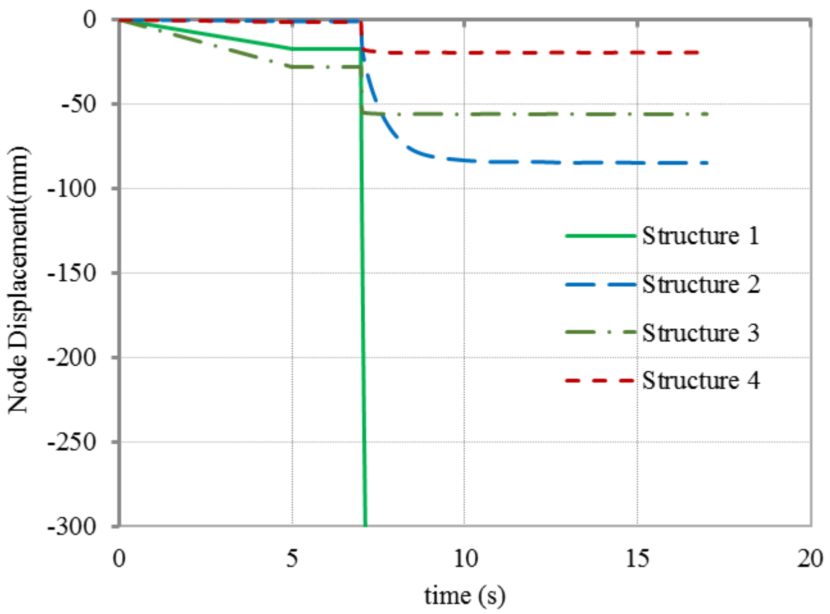
7.4. Results of 5-storey steel structure with moment-resisting frame system

7.4.1. Node displacement at column removal position

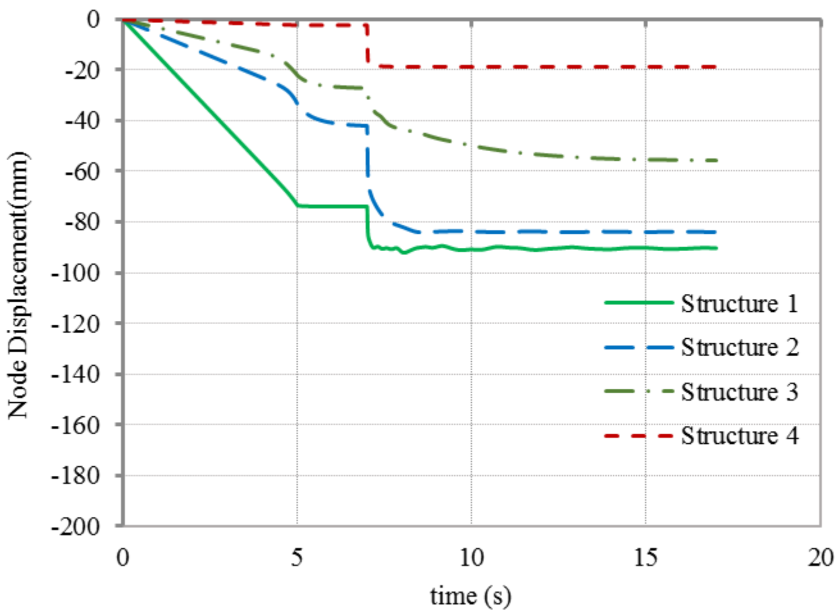
Fig. 7.5 demonstrates vertical node displacement at column removal spot. Due to S1F1PB adjacent column removal, merely irregular structure 1 failed to withstand the high deformation caused by such removal. Three structures 2, 3 and 4 had displacements as 25, 19.7 and 1.7mm before column removal, respectively, which changed by 228%, 1665% and roughly 11 times upon column removal. Also similar behaviour was observed upon S1F4PA column removal. However, by S1F4PD column removal all the structures managed to withstand severe deformation. Under such scenario, the final deformation of structures 1 through 4 reached 89, 84, 55 and 19mm, respectively. Meanwhile, upon S1F7PA corner column removal two irregular structures suffered from severe displacement and failed to withstand the deformation caused by column removal.



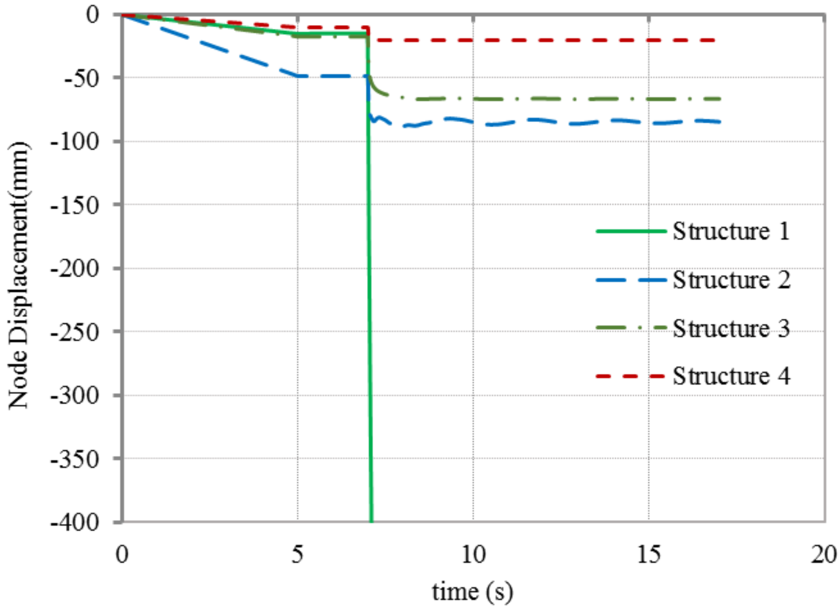
a) S1F1PB



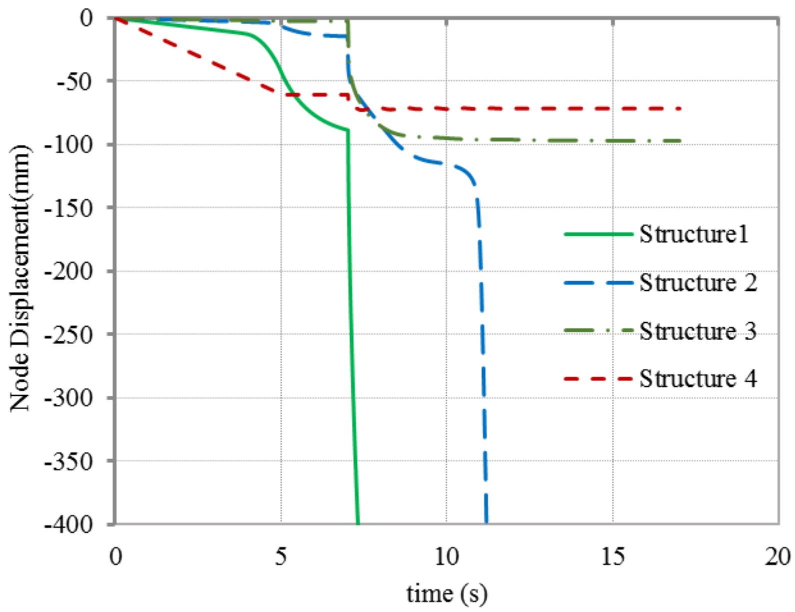
b) S1F4PA



c) S1F4PD



d) S1F4PF



e) S1F7PA

Figure 7.5 Vertical displacement of removal point in several scenarios.

7.4.2. Calculating values for connection rotation and strain level in beam and column members

In the following the results on rotation of connections adjacent to removed column in 5-storey structure are given, the acceptable values for connections rotation per floor has been given in Tables 7.12 and 7.13.

Table 7.12 Plastic rotation angle acceptance of connections in GSA (2013)

	1st	**2nd**	**3rd**	**4th**	**5th**
Structure 1	0.024211	0.024211	0.024211	0.024211	0.024211
Structure 2	0.023991	0.024085	0.024148	0.024211	0.024211
Structure 3	0.024211	0.024211	0.024211	0.024211	0.024211
Structure 4	0.024085	0.024085	0.024148	0.024211	0.024211

Plastic rotation of the beam adjacent to the removed column has been found in different floors given in Table 7.13.

Table 7.13 Plastic rotation angle of connections in 5-storey

Scenario	Type of structure	Storey				
		1st	2nd	3rd	4th	5th
S1F7PA	Structure 1	0.03565	0.03351	0.03106	0.03074	0.02813
	Structure 2	0.03061	0.03006	0.02993	0.02911	0.02807
	Structure 3	0.02429	0.02402	0.02334	0.02284	0.02201
	Structure 4	0.02365	0.02349	0.02377	0.02311	0.02264
S1F4PA	Structure 1	0.03206	0.03197	0.03126	0.03044	0.03006
	Structure 2	0.02304	0.02353	0.02323	0.02233	0.02216
	Structure 3	0.02397	0.02364	0.02381	0.02366	0.02346
	Structure 4	0.02181	0.02166	0.02112	0.02091	0.02016
S1F1PA	Structure 1	0.03579	0.03344	0.03228	0.03019	0.02809
	Structure 2	0.03044	0.03123	0.02947	0.02901	0.02893
	Structure 3	0.02638	0.02687	0.02544	0.02523	0.02512
	Structure 4	0.02392	0.02387	0.02376	0.02288	0.02202

S1F1PB	Structure 1	0.03379	0.03344	0.03328	0.03219	0.03209
	Structure 2	0.02344	0.02323	0.02304	0.02231	0.02211
	Structure 3	0.02318	0.02267	0.02214	0.02189	0.02123
	Structure 4	0.02198	0.02147	0.02076	0.02011	0.02004
S1F1PD	Structure 1	0.03462	0.03452	0.03419	0.03337	0.03283
	Structure 2	0.02921	0.02902	0.02853	0.02823	0.02761
	Structure 3	0.02398	0.02316	0.02294	0.02245	0.02191
	Structure 4	0.02191	0.02159	0.02215	0.02201	0.02161
S1F4PD	Structure 1	0.02519	0.024258	0.024145	0.023623	0.023121
	Structure 2	0.02346	0.023125	0.037562	0.037126	0.037036
	Structure 3	0.02362	0.023211	0.023163	0.023012	0.02223
	Structure 4	0.02269	0.022171	0.021411	0.02119	0.02106
S1F4PF	Structure 1	0.036982	0.036226	0.035268	0.034659	0.033865
	Structure 2	0.02365	0.02314	0.022323	0.02211	0.02201
	Structure 3	0.023169	0.022539	0.022096	0.02192	0.021753
	Structure 4	0.021836	0.021293	0.020484	0.020156	0.02011

Table 7.14 Plastic strain level in column element

Scenario	Type of structure	Storey				
		1st	2nd	3rd	4th	5th
S1F7PA	Structure 1	0.033260781	0.032166098	0.02667038	0.024949198	0.023339092
	Structure 2	0.032263312	0.030797744	0.025535816	0.023887853	0.022346242
	Structure 3	0.030651024	0.02981253	0.024718929	0.023123685	0.021631389
	Structure 4	0.029806321	0.029021091	0.024062711	0.022509815	0.021057137
S1F4PA	Structure 1	0.029976732	0.02888205	0.023947426	0.02240197	0.020956251
	Structure 2	0.029765965	0.028128842	0.023322907	0.021817755	0.020409739
	Structure 3	0.02981253	0.028337976	0.023496309	0.021979967	0.020561482
	Structure 4	0.029262738	0.028401696	0.023549143	0.022029391	0.020607716
S1F1PA	Structure 1	0.033823644	0.031991684	0.026525765	0.024813916	0.023212541
	Structure 2	0.032670061	0.031101233	0.025787452	0.02412325	0.022566447
	Structure 3	0.030798071	0.029364854	0.024347741	0.022776451	0.021306564

	Structure 4	0.029263146	0.028448016	0.023587549	0.022065318	0.020641325
S1F1PB	Structure 1	0.028446219	0.023765535	0.01970509	0.018433415	0.017243808
	Structure 2	0.02888205	0.026692684	0.022132123	0.020703818	0.01936769
	Structure 3	0.026692684	0.024503318	0.02031682	0.019005667	0.017779129
	Structure 4	0.025892095	0.023980485	0.019883315	0.018600138	0.017399771
S1F1PD	Structure 1	0.034935482	0.032166098	0.02667038	0.024949198	0.023339092
	Structure 2	0.032278834	0.031037921	0.025734957	0.024074143	0.022520509
	Structure 3	0.016580756	0.014610327	0.012114089	0.011332302	0.010600968
	Structure 4	0.019029904	0.015745856	0.013055608	0.01221306	0.011424886
S1F4PD	Structure 1	0.029399982	0.028326539	0.023486826	0.021971096	0.020553183
	Structure 2	0.028326539	0.02639859	0.021888276	0.020475708	0.019154301
	Structure 3	0.022297614	0.021676749	0.017973182	0.016813277	0.015728226
	Structure 4	0.021500293	0.020136024	0.016695697	0.015618234	0.014610306
S1F4PF	Structure 1	0.028114138	0.026447606	0.021928917	0.020513727	0.019189866
	Structure 2	0.027199179	0.025795698	0.02138839	0.020008083	0.018716854
	Structure 3	0.010290018	0.009793326	0.008120094	0.007596061	0.007105846
	Structure 4	0.010249172	0.010030235	0.008316526	0.007779816	0.007277743

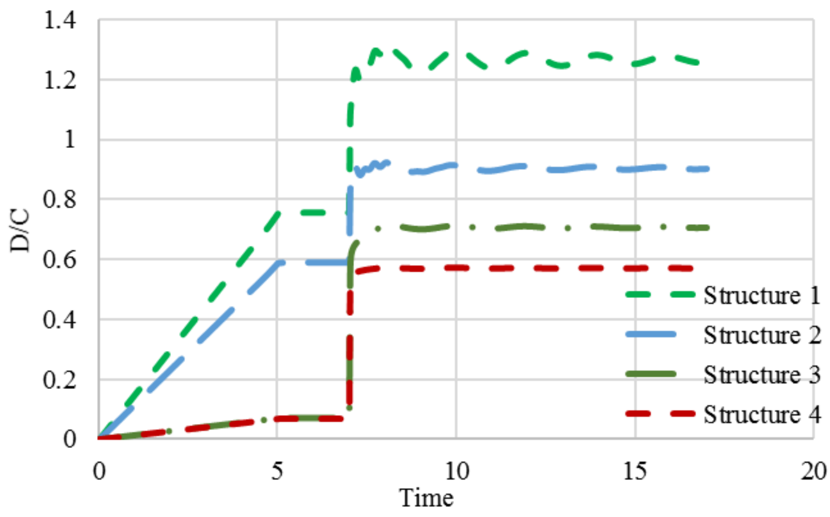
Table 7.15 Plastic strain level in beam element

Scenario	Type of structure	Storey				
		1st	2nd	3rd	4th	5th
S1F7PA	Structure 1	0.030202835	0.029309901	0.025707158	0.022307886	0.020007723
	Structure 2	0.028933127	0.027965315	0.024620976	0.021280826	0.018994483
	Structure 3	0.027399479	0.02425488	0.023309005	0.020040268	0.017770617
	Structure 4	0.026944251	0.024091685	0.022919577	0.019672037	0.017407341
S1F4PA	Structure 1	0.027526086	0.026635206	0.023417312	0.02014268	0.017871651
	Structure 2	0.025928587	0.025037706	0.02205072	0.018850472	0.016596831
	Structure 3	0.02640695	0.025473591	0.022459939	0.019237417	0.016978569
	Structure 4	0.024256598	0.022153391	0.020620404	0.01749801	0.015262566
S1F1PA	Structure 1	0.031220715	0.029342545	0.02657791	0.023131243	0.020820001
	Structure 2	0.030389472	0.028177278	0.025866817	0.022458855	0.020156661

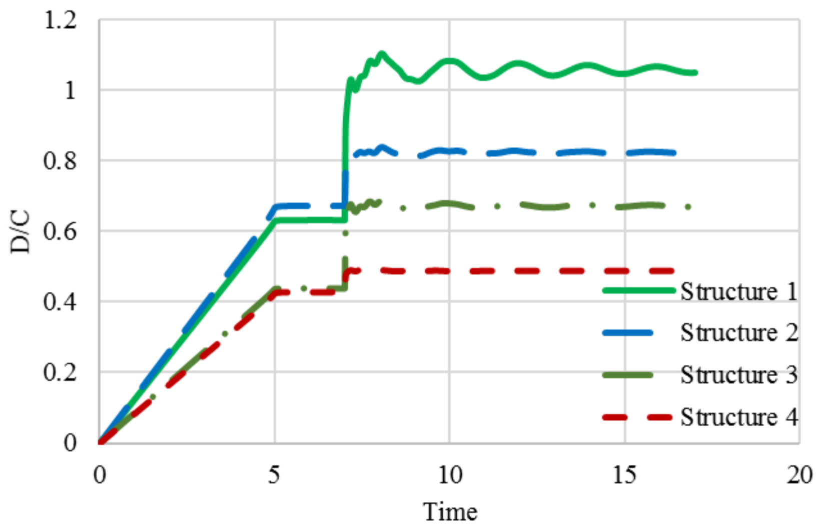
	Structure 3	0.030263497	0.029403623	0.025759051	0.022356955	0.020056132
	Structure 4	0.028709808	0.026538651	0.024429936	0.021100185	0.018816272
S1F1PB	Structure 1	0.027377528	0.026254253	0.023290227	0.020022512	0.0177531
	Structure 2	0.026424128	0.024419983	0.022474634	0.019251313	0.016992277
	Structure 3	0.024849338	0.023065523	0.021127468	0.017977474	0.015735579
	Structure 4	0.02485326	0.023561064	0.021130823	0.017980647	0.015738709
S1F1PD	Structure 1	0.03025109	0.028750845	0.025748438	0.022346919	0.020046231
	Structure 2	0.03009553	0.02782827	0.025615363	0.022221088	0.019922093
	Structure 3	0.021678122	0.020666506	0.018414627	0.015412297	0.013204917
	Structure 4	0.019761778	0.01853925	0.016775276	0.013862177	0.011675656
S1F4PD	Structure 1	0.02485326	0.023576334	0.021130823	0.017980647	0.015738709
	Structure 2	0.02435127	0.021281707	0.020701391	0.01757459	0.015338116
	Structure 3	0.019769413	0.017856886	0.016781807	0.013868353	0.011681748
	Structure 4	0.018793108	0.016688756	0.01594662	0.013078626	0.010902647
S1F4PF	Structure 1	0.0273823	0.021389907	0.023294309	0.020026372	0.017756908
	Structure 2	0.026426991	0.023110609	0.022477083	0.019253628	0.016994562
	Structure 3	0.010043058	0.008971537	0.008461322	0.006000763	0.00392002
	Structure 4	0.011153499	0.009823988	0.009411258	0.006898992	0.004806163

7.4.3. D/C ratio for adjacent columns to the removed column

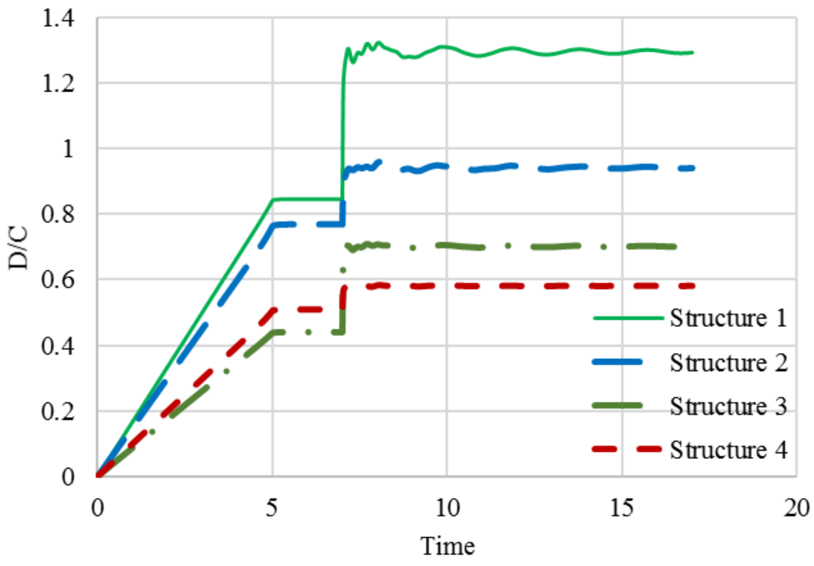
In Fig. 7.6, D/C ratio has been given for four different scenarios. Upon S1F1PB and S1F4PA corner columns removal merely irregular structure 1 has had a demand to capacity ratio exceeding 1, while the least ratio is related to regular structure 4. However, upon S1F7PA corner column removal scenario two irregular structures 1 and 2 experienced ratios higher than 1.



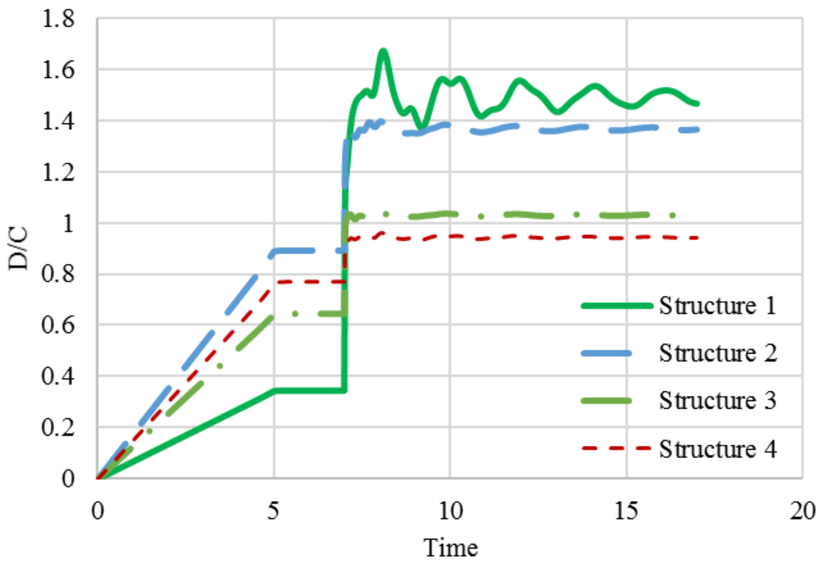
a) Scenario S1F1PB



b) Scenario S1F4PA



c) Scenario S1F4PF



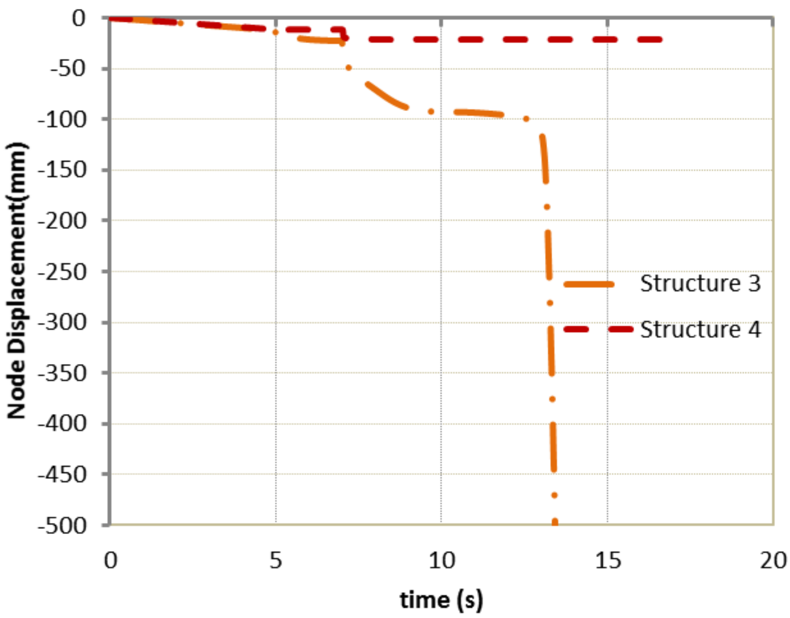
d) Scenario S1F7PA

Figure 7.6 The demand force to capacity ratio (D/C) of the adjacent columns in different scenarios

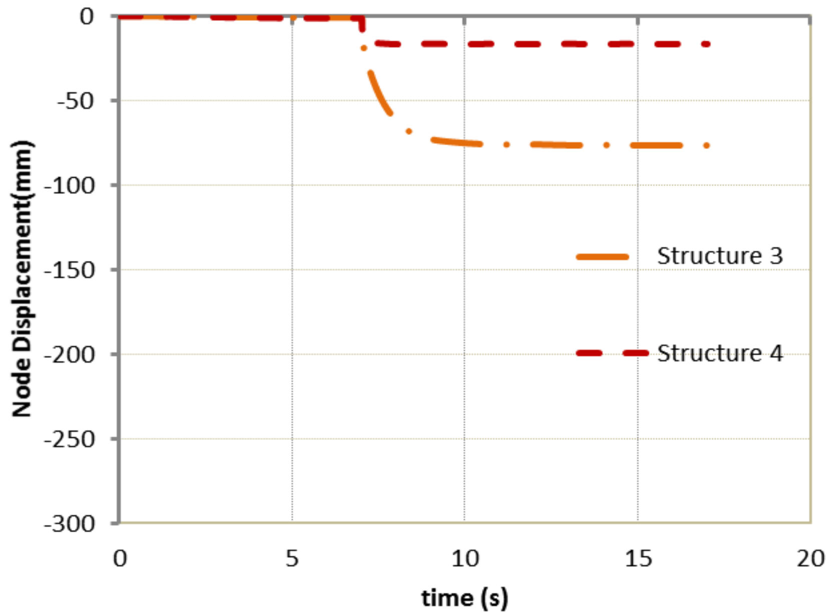
Table 7.16. Node displacment and maximum D/C ratio of adjacent columns for all the scenarios and structures

Scenario	Structure 1		Structure 2		Structure 3		Structure 4	
	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C
S1F1PA	Fail	1.77	Fail	1.44	97	1.03	72.6	0.96
S1F1PB	Fail	1.29	86	0.92	56.5	0.71	19.4	0.57
S1F1PD	Fail	1.5	Fail	1.23	64.4	0.73	21	0.6
S1F4PA	Fail	1.15	84.9	0.87	56.3	0.71	19.3	0.56
S1F4PD	91	1.05	84	0.83	55.7	0.68	19.1	0.49
S1F4PF	Fail	1.31	87.8	0.94	62.6	0.71	20	0.58
S1F7PA	Fail	1.67	Fail	1.38	97	1.03	72.6	0.96

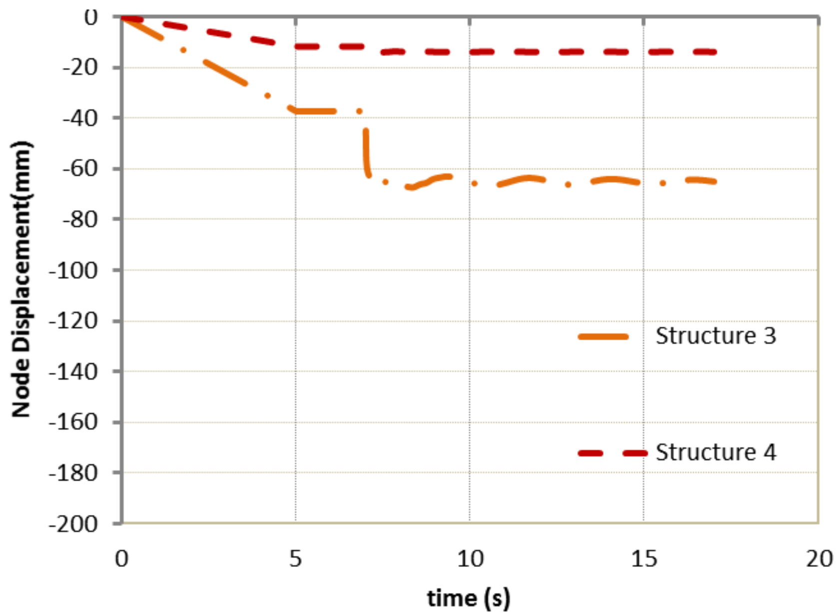
In the following, a summary on node vertical displacement at the location of removed columns in terms of time is given in Fig. 7.7. Such curves are related to two structures 3 and 4 mentioned in 4th Chapter for 5-storey structure equipped with bracing with irregularity in the plan.



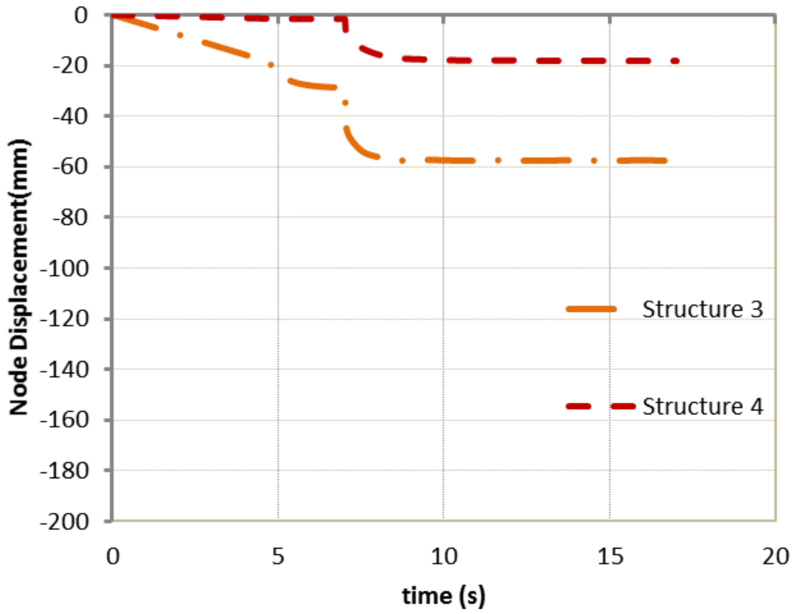
S1F1PA



S1F4PA

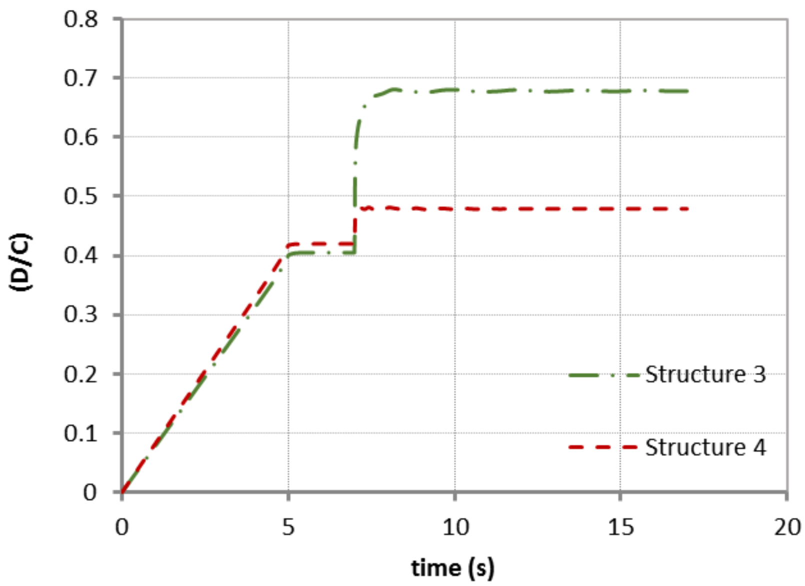


S1F4PD

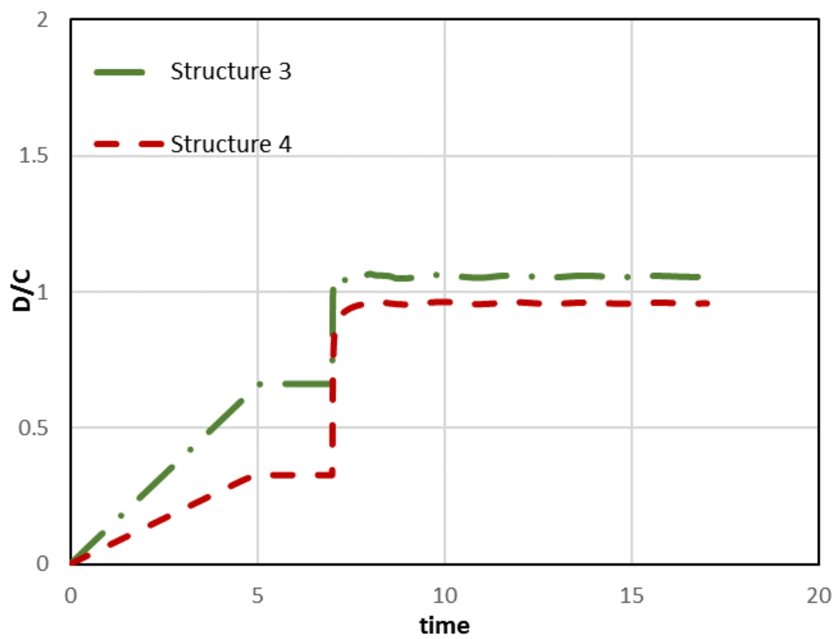


S1F4PF

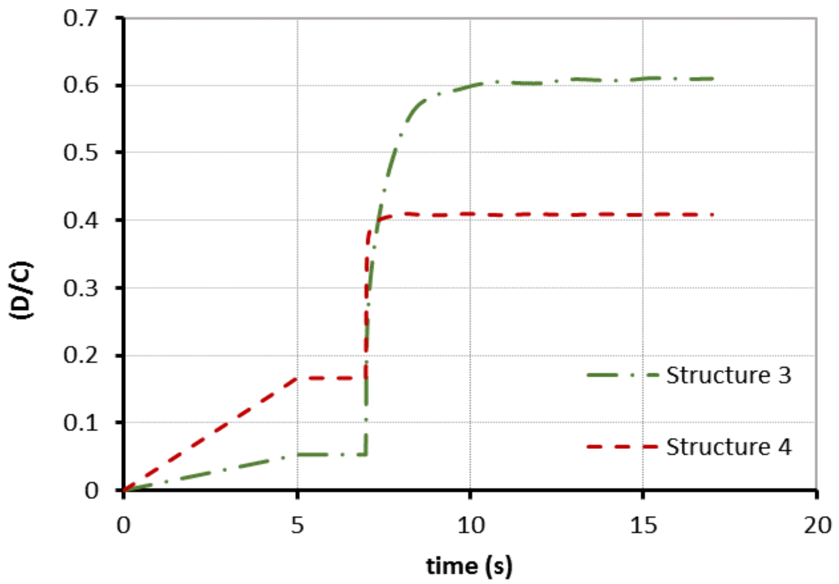
Figure 7.7 Node displacement history for irregular 5- storey building equipped with concentric bracing system.



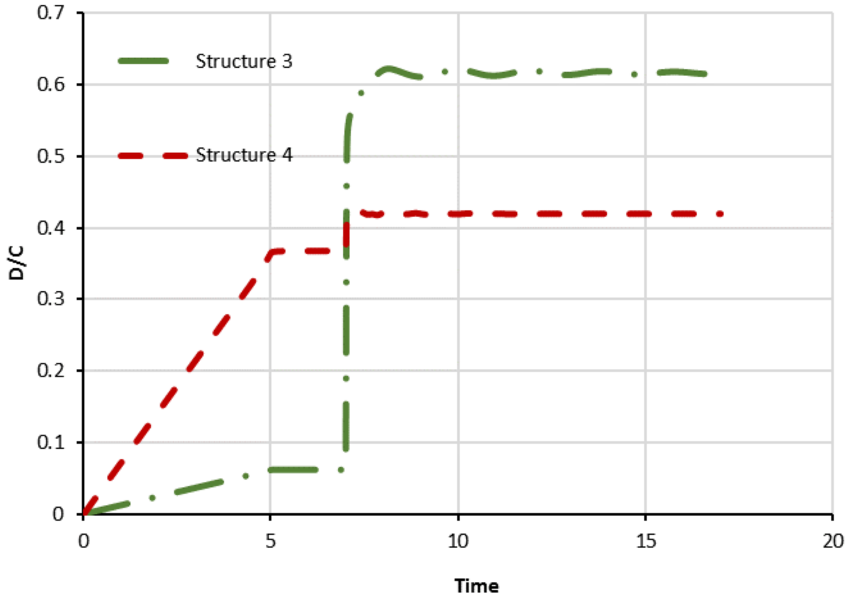
a) The demand force to capacity ratio (D/C) of the adjacent columns in Scenario S1F1PB



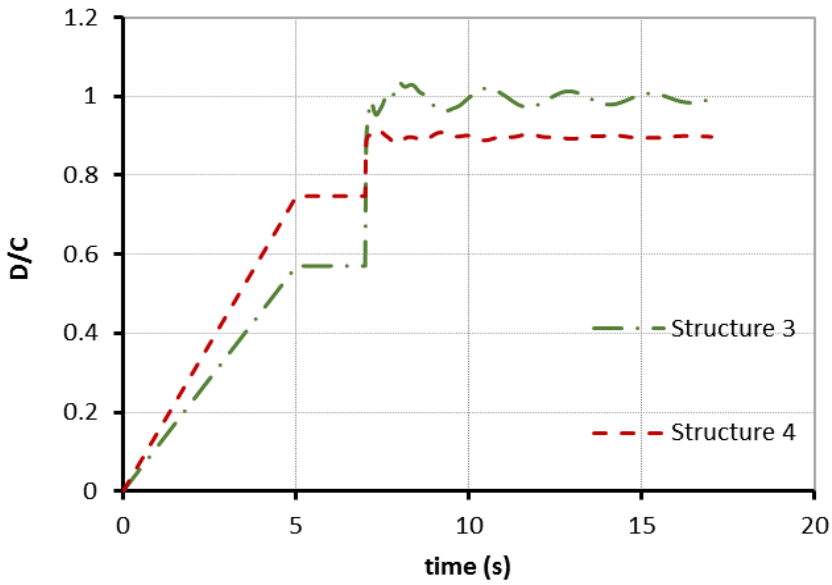
b) The demand force to capacity ratio (D/C) of the adjacent columns in Scenario S1F1PD



c) The demand force to capacity ratio (D/C) of the adjacent columns in Scenario S1F4PA



d) The demand force to capacity ratio (D/C) of the adjacent columns in Scenario S1F4PF



e) The demand force to capacity ratio (D/C) of the adjacent columns in Scenario S1F7PA

Figure 7.8 D/C ratio for irregular 5-storey bracing frame

A comparison has been conducted in Table 7.17 between capacity ratio values for two irregular 5-storey structures located in C and E seismic regions and under two states of with/without bracing. Considering the presented values, it is evident that inclusion of bracing in moment-resisting frame system and changing load-bearing system from moment-resisting frame into moment-resisting and bracing system (ASCE7-10) lead to an increase in vertical load bearing capacity, stiffness, and D/C ratio in comparison to moment-resisting system (structures 1 and 2).

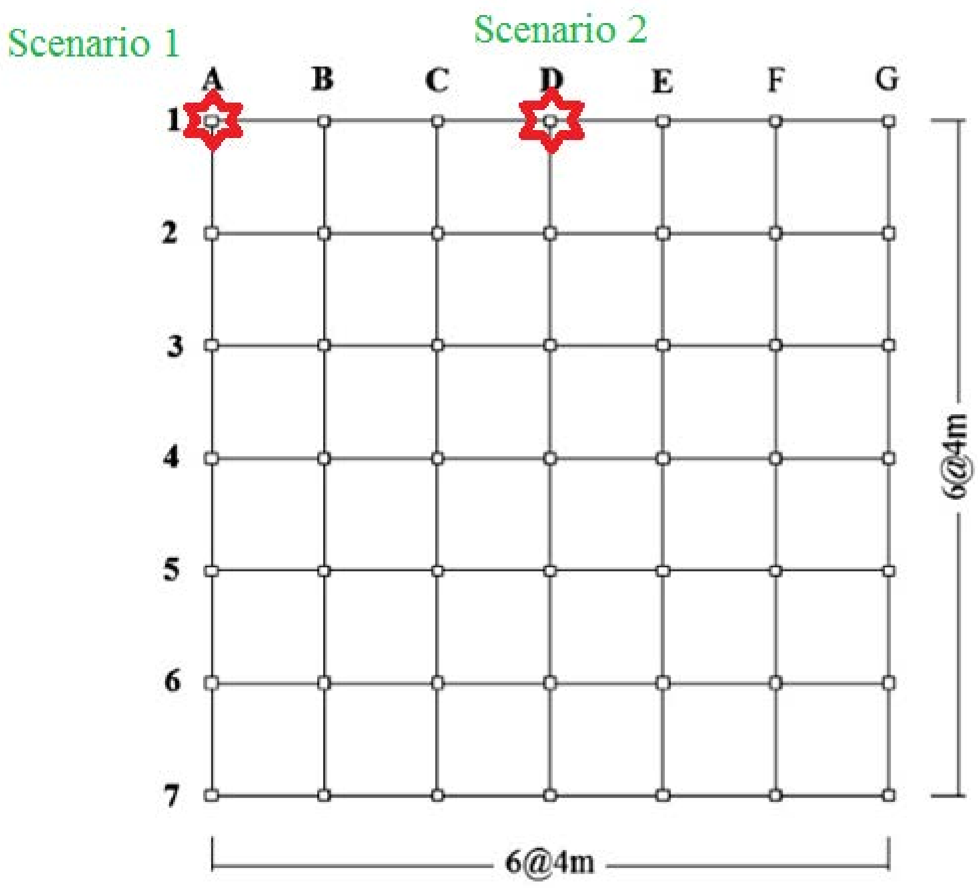
Table 7.17 Node displacement and maximum D/C ratio of adjacent columns for all the scenarios and structures

Scenario	Structure 1		Structure 2		Structure 3		Structure 4	
	Node Displacement	D/C	Node Displacement	D/C	Node Displacement	D/C	Node Displacement	D/C
S1F1PA	Fail	1.77	Fail	1.44	Fail	1.06	42.4	0.96
S1F1PB	Fail	1.29	86	0.92	64.61	0.68	21.4	0.48
S1F1PD	Fail	1.5	Fail	1.23	Fail	1.06	42.4	0.96
S1F4PA	Fail	1.15	84.9	0.87	76.3	0.61	16.3	0.41
S1F4PD	91	1.05	84	0.83	65.7	0.52	14.1	0.39
S1F4PF	Fail	1.31	87.8	0.94	57.6	0.62	18.2	0.42
S1F7PA	Fail	1.67	Fail	1.38	Fail	1.03	40.1	0.91

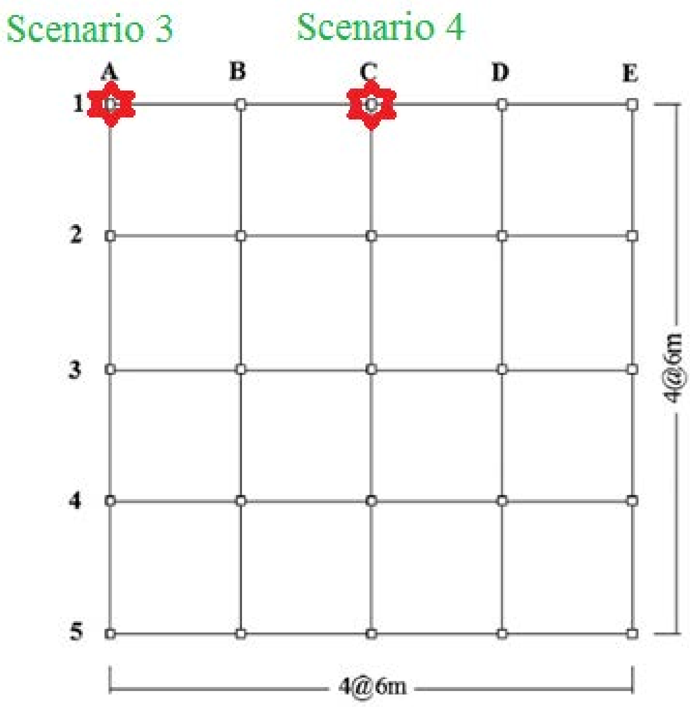
7.5 Investigation on geometrical characteristic of building

7.5.1 Parametric study

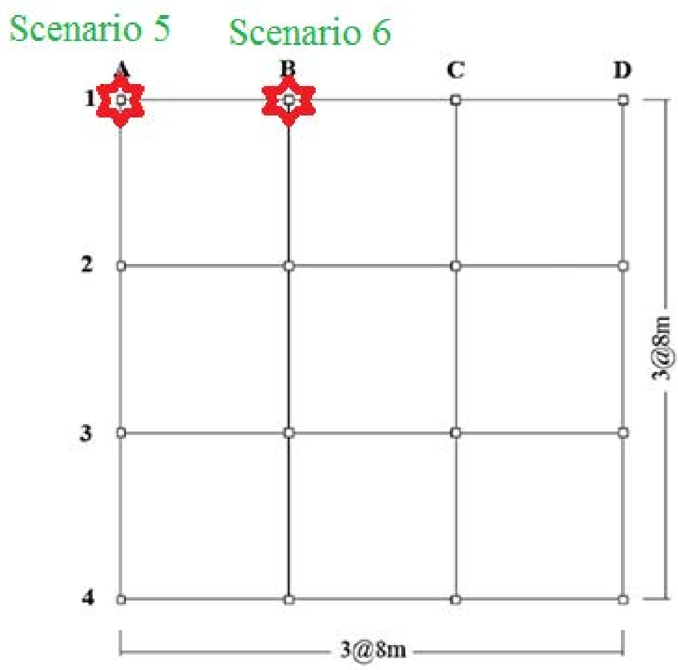
Parametric study is performed by varying the span length (4, 6 and 8 meters) on the impact on progressive collapse in a 5-storey building. Figure 7-9 shows the plan features and the views of the three structures. In all the analyses, the frame has been selected at Axis 1. For column removal scenarios, in the 4-meter span building columns A and D have been chosen, in the 6-meter span structure, columns A and C, and in the 8-meter span structure, columns A and B.



a) plan views of building with the span length of 4 m (Frame 1)



b) plan views of building with the span length of 6 m (Frame 2)



c) plan views of building with the span length of 8 m (Frame 3)

Figure 7.9 Plan views of 5-storey building with different span length

In the first assessment, for each removal scenario, a structure's response is investigated by through nonlinear dynamic analysis, after which analysis results are compared with the performance to decide whether progressive collapse happens. If a member fails, it can be said that the building is susceptible to collapse progression and the analysis stops in that scenario. Otherwise, the structure seems likely to be able to achieve a static balance after the loss of an element but only when it is subjected to the imposed loads.

DAF is responsible for the dynamic nature of sudden column loss and the method of the DAF calculation can be seen in Fig. 7-10 and Eq. 7.2.

$$DAF = \frac{peak\ value\ of\ internal\ force}{steady\ value\ of\ internal\ forces\ before\ removal} \tag{7.2}$$

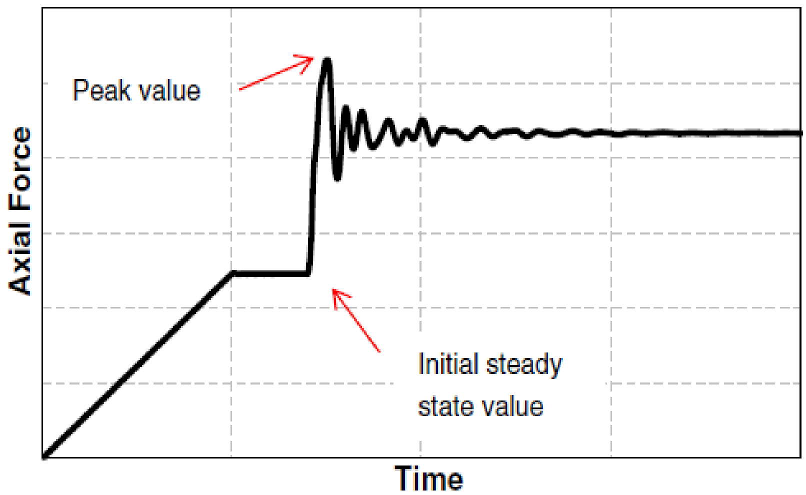


Figure 7.10 Parameters needed for calculating Dynamic Amplification Factor (DAF).

7.5.2 The results of nonlinear dynamic analysis

Figs. 7-11 and 7-12 depict vertical displacements of the top nodes of the removed columns. According to these diagrams, the longer the span, the larger the removal point's vertical displacement;

thus, doubling this length the maximum vertical displacement increases more than five times. At the same time, the figures show that vertical displacements of corner removal nodes are roughly 27% larger than vertical displacements of internal removal nodes. Nonetheless, this does not necessarily indicate that structures are more vulnerable to progressive collapse upon the loss of a corner column.

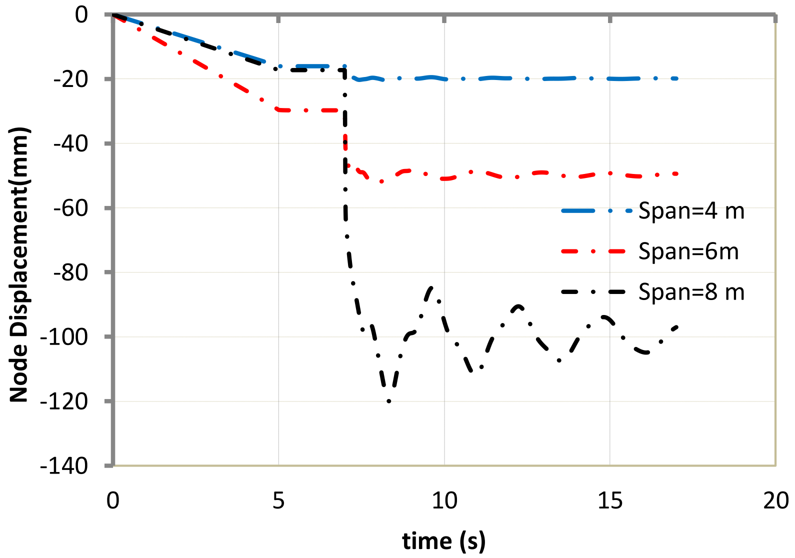


Figure 7.11 Vertical displacement of column positioning removed A.

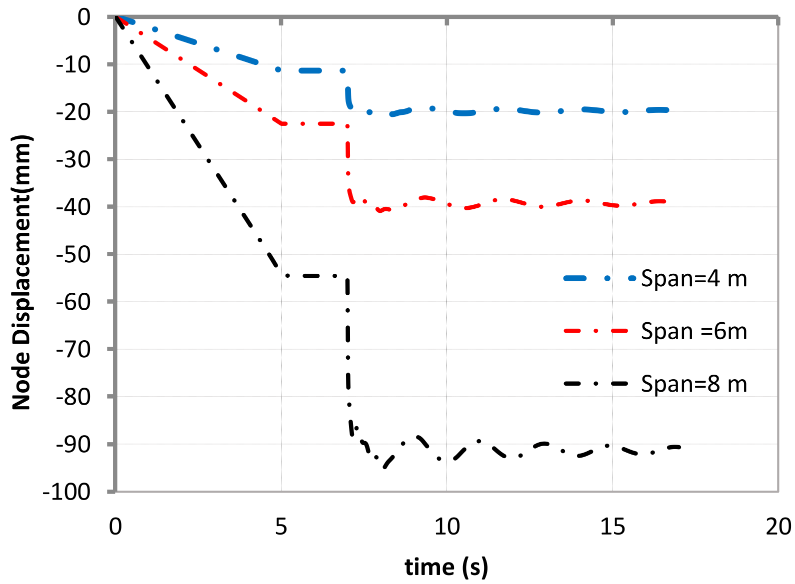


Figure 7.12 Vertical displacement of column positioning removed D, C, B.

The total rotation to yield rotation ratios of some critical beams and columns are shown in Figs. 7-13 and 7-14 for the scenarios studied. As per these figures, it can be seen that the investigated frames survive loss of one column and do not go beyond life safety performance level.

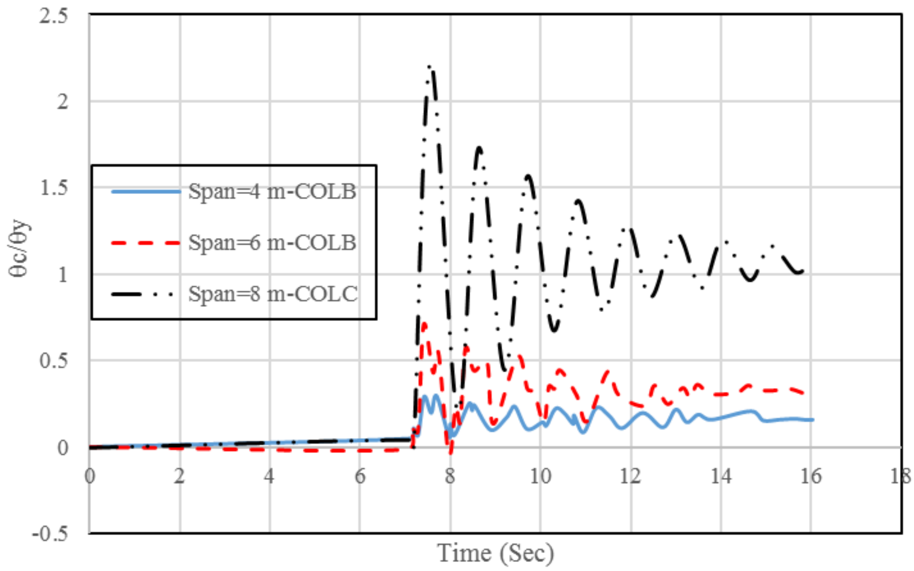


Figure 7.13 Total rotation to yield rotation of columns

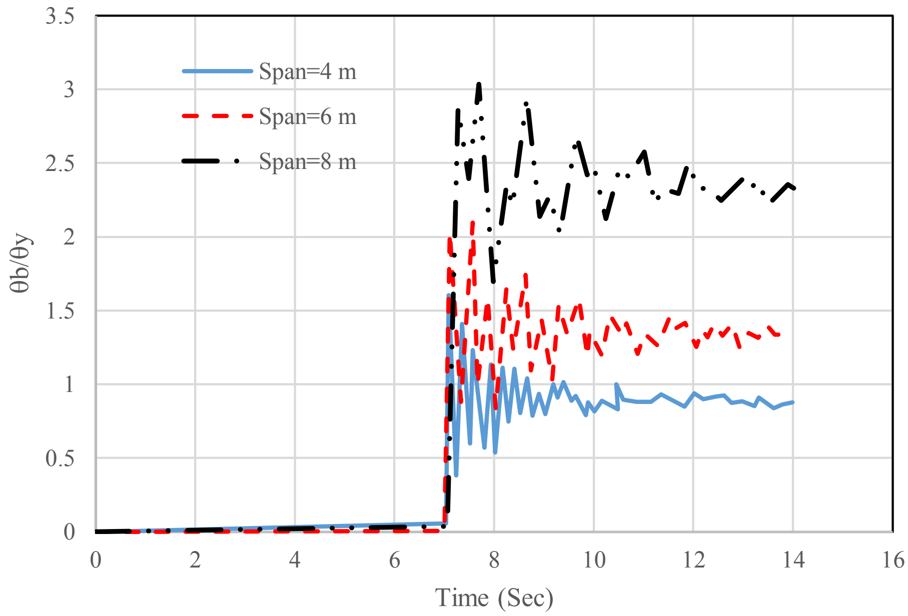


Figure 7.14 Total rotation to yield rotation of beam AB-1.

Axial forces and bending moments of critical columns are depicted in Figs. 7-15 and 7-16. The results of the simulation showed that the system was capable of successfully absorbing structural member losses. Yet, a significant redistribution of forces was seen. For example, when the corner column was suddenly removed in Scenario 1, the axial force of Col-B-1 rocketed from 504 kN to a maximum value of 1024 kN before it settled down at a stable value of 817 kN, lower than its maximum load (1024 kN), and the bending moment of this column was 79 kN·m. In Scenario 3, the axial force of the column in Frame 2 soared from 1013 kN to a maximum value of 2114 kN before it settled down at a stable value of 1695.32 kN. In addition, the bending moment of this column was 255 kN·m.

Nevertheless, the columns show that in spite of these increases in axial forces and bending moments, the structural systems were capable of successfully absorbing the loss of predefined members and redistributing excess forces to alternate columns. Thus, no progressive collapse was predicted. This situation occurred because the studied frames were designed to support the seismically induced forces. Therefore, the columns had sufficiently large dimensions to permit the frames to continue to successfully carry all the gravity loads. In frames like these, spans that were affected by element removal obtained their stability from undamaged bays, and therefore, they did not experience collapse.

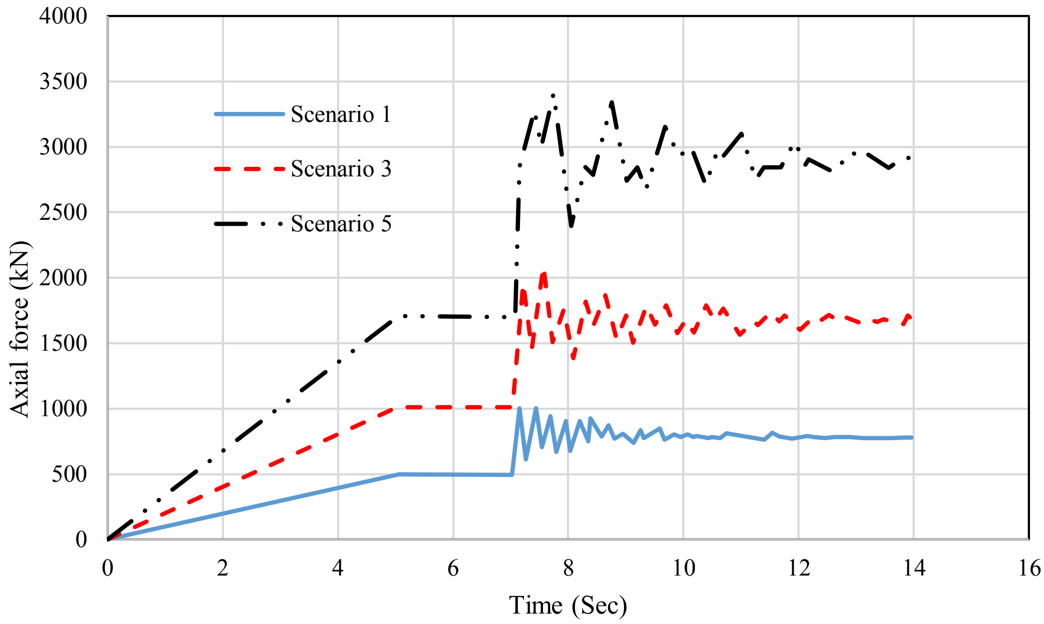


Figure 7.15 Axial forces of columns positioning B.

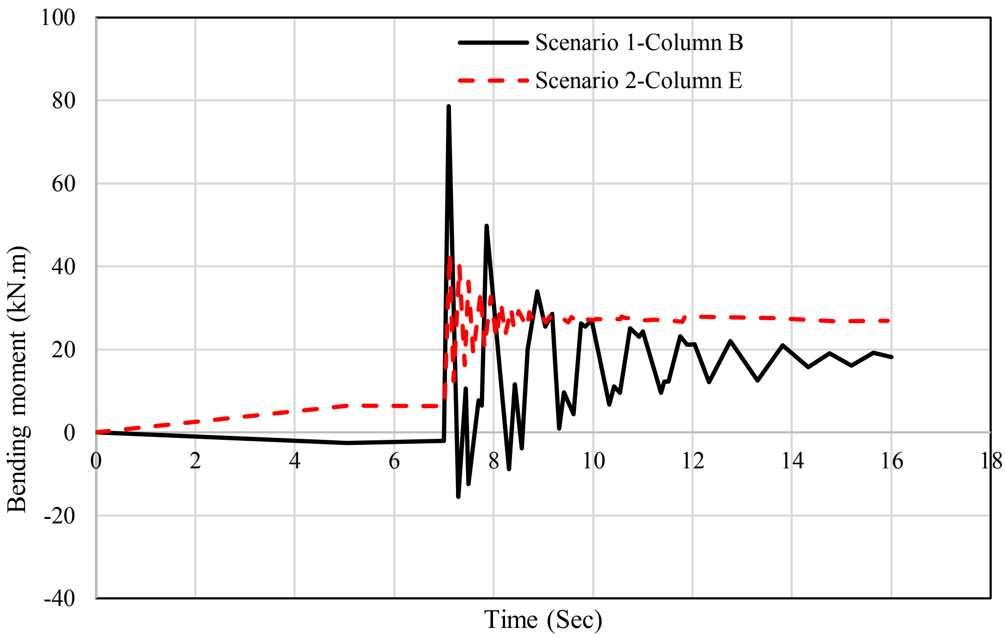


Figure 7.16 Bending moments of columns in two scenario.

The critical Dynamic Amplification Factors (DAFs), which were calculated according to nonlinear dynamic analysis results are shown in Table 7.18.

The values calculated for DAF are related to steel moment frame structures based on nonlinear dynamic analysis close to the coefficient recommended by GSA (2013). Thus, the removal of any corner or inside column of the structure, the force exerted upon the column adjacent to the removed column suddenly rises by 73%-109%.

Table 7.18 Critical DAF for columns.

Scenario	Removal column	DAF	
		Value	Element
1	Column A-Span 4m	2.03	Column B
2	Column D-Span 4m	1.99	Column E
3	Column A-Span 6m	2.09	Column B
4	Column C-Span 6m	1.95	Column B
5	Column A-Span 8m	2.00	Column B
6	Column B-Span 8m	1.73	Column C

CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Progressive collapse is a disastrous phenomenon in which failure of one key structural member leads to failure of other members, and this in return, leads to partial or even entire collapse of the structure. Plane impact, car collision and gas explosions are a few examples of the hazards which can produce such an event. As structures are not generally designed for such unusual events that may result in structural element removal, they might fail catastrophically. GSA describes cases in which one of a building's columns is removed and the damaged structure is examined to check the system response in order to reduce the catastrophic effects of progressive collapse in structures according to the APM. In this thesis, two nonlinear static (pushdown) and dynamic analysis were used, the capacity of fourteen, 2, 3 and 5-storey irregular and regular steel structures with moment-resisting system under various column removal scenarios were examined. The most important parameters studied in these structures was the effect of plan irregularities and type of seismic regionalisation on progressive collapse have been analysed under various column removal scenarios. The structures were designed in site class C and E seismic regions by ETABS software according to the AISC (2010) and ASCE7 (2010). Then using OpenSees software, their behaviour under different column removal scenarios was examined. In the following, a brief on the most important results of these studies are presented.

1- The numerical model developed in OpenSess software was compared with the analytical model and formula suggested by Chopra (1995) for numerical model verification and extending the same for the relevant structures, to ensure the software accurate performance and results thereof, and in order to verify the non-linear dynamic analysis in terms of the members' rotation estimation and displacement of nodes in case of the progressive collapse, Kim et al.(2009) research on a 3-storey and a 3-bay frame was studied.

2- Two irregular 2-storey structures were not able to bear the force imposed by column removal in C and E seismic regions. The most critical state was related to a state when the corner columns were removed. Under various column removal scenarios, both 2-storey regular structures managed to bear the load imposed to adjacent columns and connecting beams by sudden column removal. Also in these two structures the most critical state was related to a state when the corner columns were removed. Meanwhile, under all regular and irregular structural states, removal of the internal column imposes the minimum damages and load to the structure system, which was due to higher stiffness at the column removal location caused by beams connected to the relevant node. In other words, corner and internal columns removal leads to maximum and minimum damages to the structures, respectively.

3- The results showed that upon increasing structure height from 2 to 3 stories, the structure capacity against progressive collapse also increases. Comparing the yield load factor in two 2- and 3-storey structures with similar status the same issue may be instated. The yield load factor in structure 1 with scenario 3 was equal to 0.545, while for 3-storey the same was equal to 0.581. In other words, a rough increase of 7% has been observed in the structure capacity.

4- A comparison between the 5-storey structure with those 2- and 3-storey ones demonstrated that under all states the capacity and yield load factor have increased upon increasing the structure height.

5- Regarding forces in height ratio changes it may have expressed that the relation between force and height for regular structures was almost linear, while the same was nonlinear for irregular ones.

6- In all cases elastic stiffness, plastic strength and stiffness in catenary stage were higher for regular structures than those of the irregular structures. In other words, regular structures had higher energy absorption capability due to column removal in comparison to irregular ones.

7- In order to remove columns in peripheral areas or indentation in the plan, the difference between stiffness in catenary stage in regular and irregular structures increases, which was due to higher redundancy degrees or higher stiffness of floor and beam grid at the spot of column removal. In other words, the structural system under corner column removal scenario has less plastic strength and stiffness due to connection of two beams perpendicular onto the column. Although structures plastic resistance of structures increases from irregular towards regular structure, displacement represented by such point does not really change and may not be found a real increase/decreased trend for the same.

8- The irregular structure managed to achieve a force ratio higher than 1 in most scenarios through distributing braces in the plan and height.

9-In order to remove columns at 7th second in two irregular 2-storey structures, the relevant structure cannot bear the deformation caused by column removal and is exposed to severe deformation resulting in building collapse. Also the column removal spot affect node final deformation. Generally, removal of corner columns has resulted in collapse and failed bearing by the structure under deformation caused by column removal for irregular structures.

10- It is seen considering the results that in all cases of column removal scenarios, the beams exceeded the strain representing yield stress However, none has reached final strain of 0.2, i.e. under column removal conditions, due to severe load applying caused by such removal, a severe stress is imposed to beam and column connection, causing earlier yield of connection in comparison to that of beam, which in turn results in preventing the beam to achieve its maximum capacity and final strain.

11- It is observed regarding 2 and 3-storey structures that upon corner columns removal at the structure plan situated on the first floor the maximum D/C ratio and sudden change in the load imposed to the column are related to irregular structures. In two cases of corner column removal irregular structure 1 has a D/C ratio higher than 1.

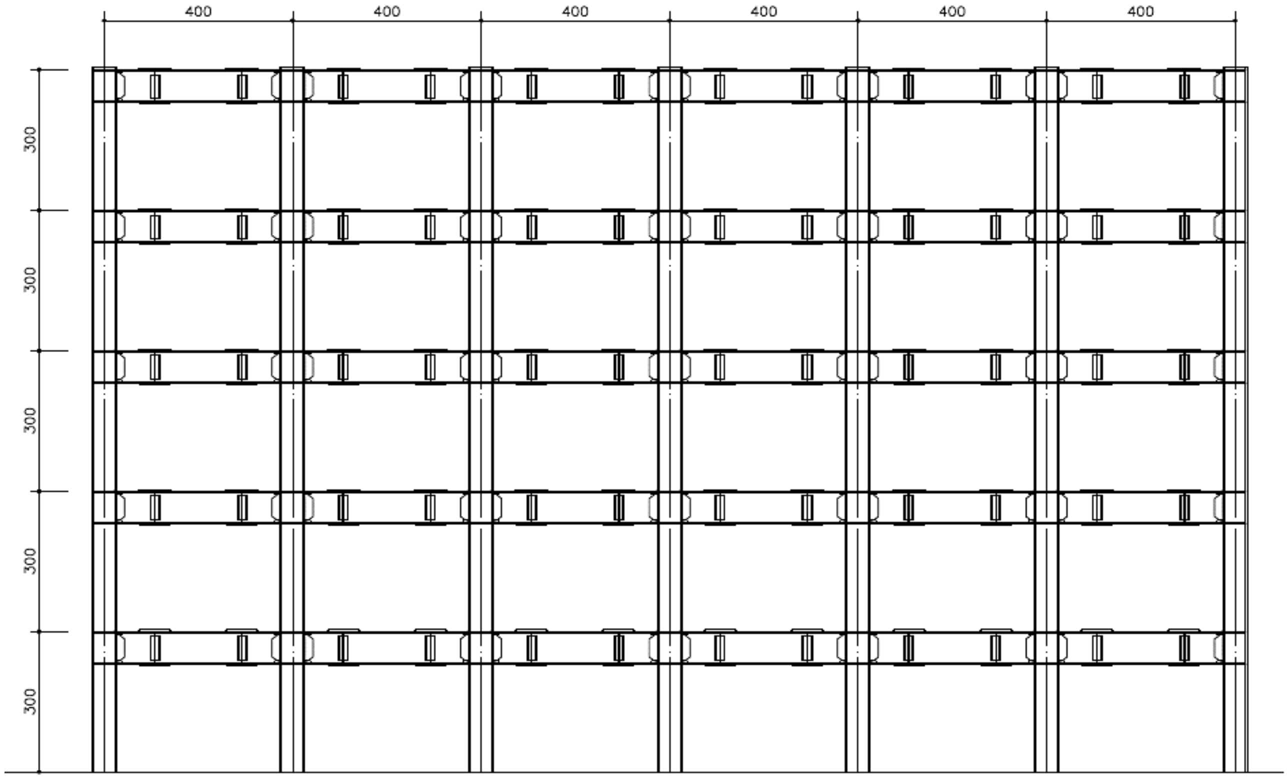
8.2. Recommendations for future work

Considering the completed studies, the following are recommended for future studies:

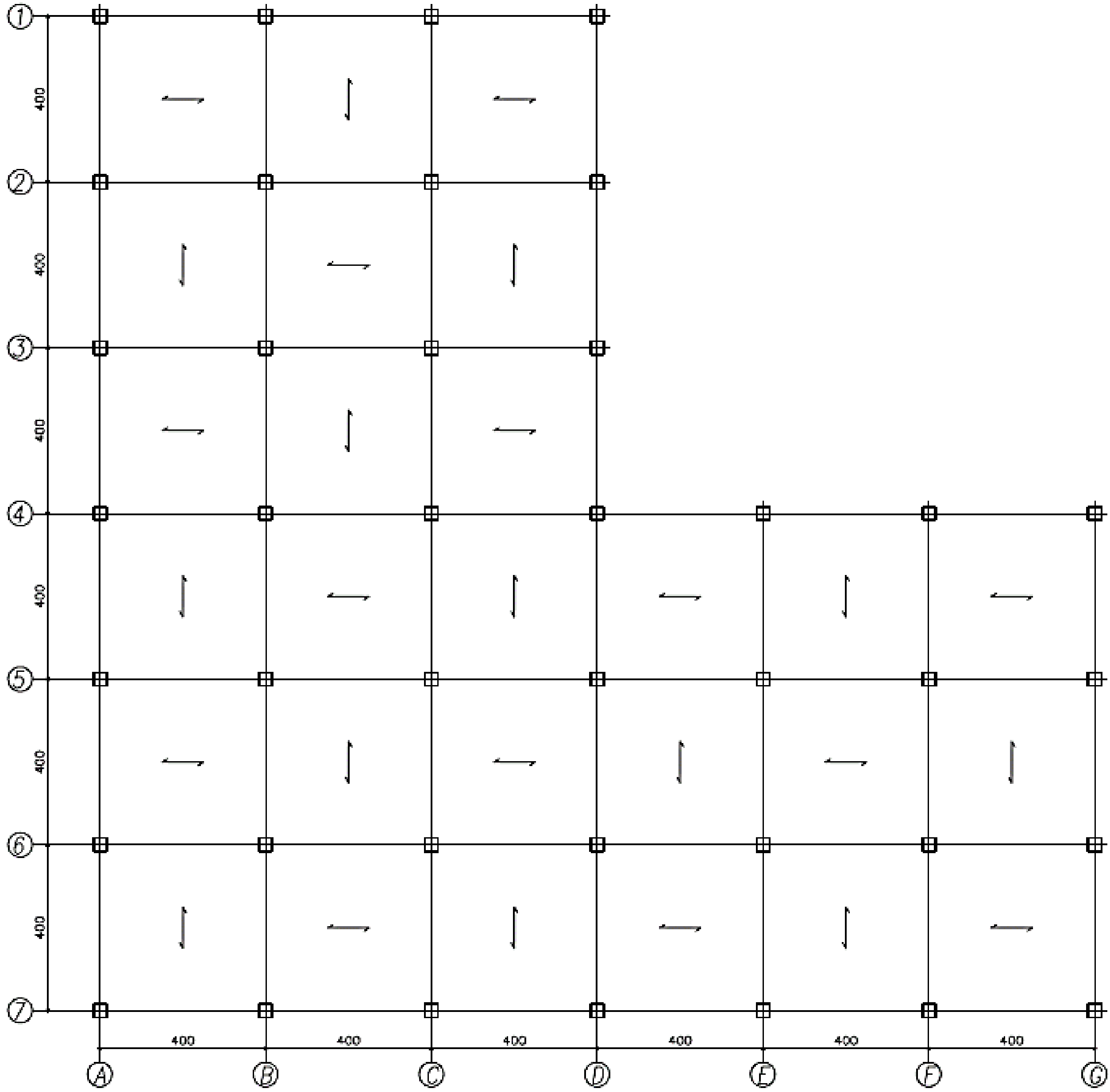
- It is recommended that the structures studied in this thesis are externally strengthened and reinforced using a variety of methods such as applying pre-stressed cables with normal strength and their behaviour and strength are studied under nonlinear dynamic analysis.
- Considering increasing application of steel and concrete materials and conducting several studies on progressive collapse relating to such type of structures, it is recommended to use composite column and beam members as concrete filled steel tube or partially encased composite to enable simultaneous steel and concrete characteristics utilization.
- It is recommended to use other different systems such as tube in tube, central shearing core in the structures and their behaviours are assessed and compared with those of the moment-resisting frame systems under various column removal scenarios.
- Due to the column removal under different situations, the force increase ratio, plastic hinges deformation and distribution in adjacent spans and members in adjacent to the removed columns are recommended to be calculated and the damages are discussed and examined upon introduction of a new parameter.

APPENDIX

DESIGN DETAIL



TYPICAL VIEW



TYPICAL PLAN

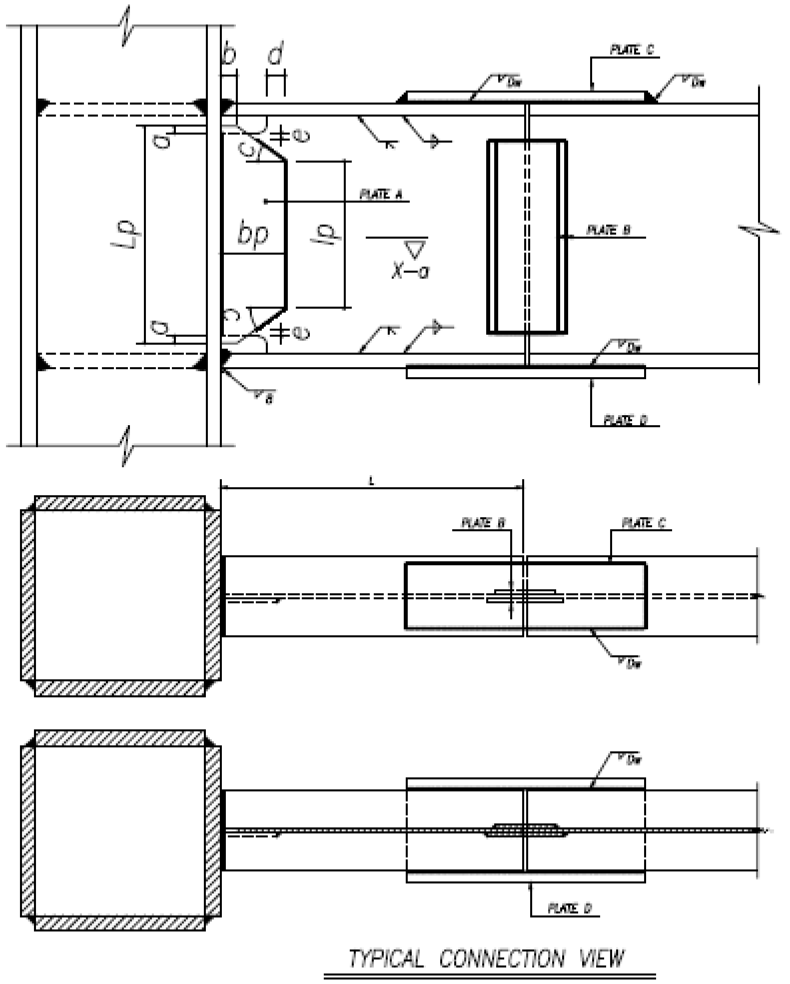
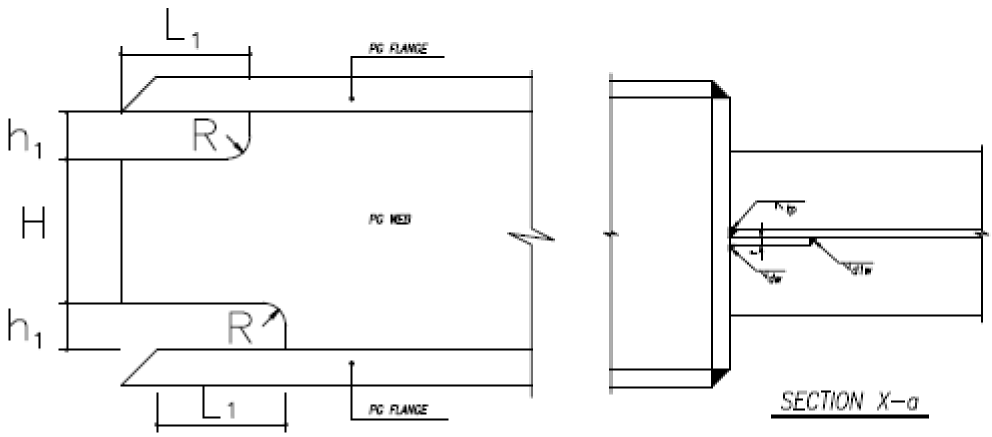
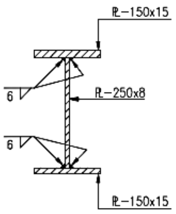
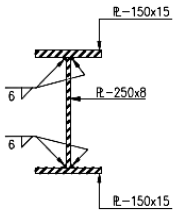


PLATE C
PLATE A
PLATE B
PLATE D
Lp
bp
lp
X–a
PLATE B
PLATE C
PLATE D
TYPICAL CONNECTION VIEW

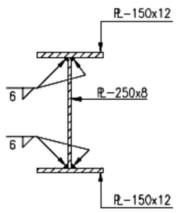

L1
PG FLANGE
h1
R
H
PG WEB
h1
R
L1
PG FLANGE
SECTION X–a



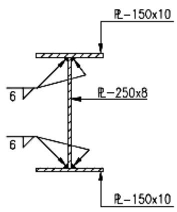
PG-21



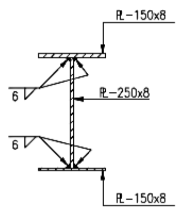
PG-22



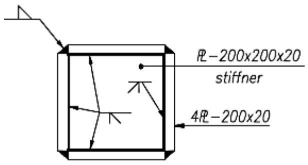
PG-23



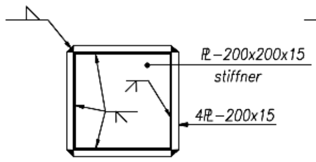
PG-24



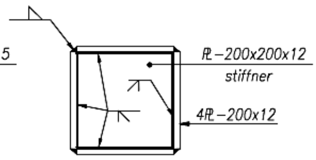
PG-25



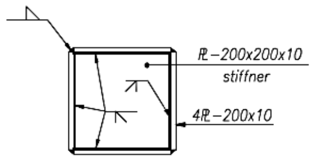
SECTION C-2



SECTION C-3



SECTION C-4



SECTION C-5

CONNECTIONS TABLE

PLATE GIRDER	PLATE GIRDER DIMENSION		L (mm.)	PG WEB (mm.)				DIMENSION OF PLATE A (mm.)									
	FLANGE (mm.)	WEB (mm.)		L_1	R	h_1	H	a	b	e	c Degrees	L_p	l_p	b_p	tp	dw	d1w
PG-21	150x20	250x8	750	40	10	30	190	10	30	20	30	230	100	120	10 mm.	10 mm.	8 mm.
PG-22	150x15	250x8	750	40	10	30	190	10	30	20	30	230	100	120	8 mm.	8 mm.	6 mm.
PG-23	150x12	250x8	750	40	10	30	190	10	30	20	30	230	100	120	8 mm.	8 mm.	6 mm.
PG-24	150x10	250x8	750	40	10	30	190	10	30	20	30	230	100	120	8 mm.	8 mm.	6 mm.
PG-25	150x8	250x8	750	40	10	30	190	10	30	20	30	230	100	120	8 mm.	8 mm.	6 mm.

PLATE GIRDER	PLATE C		PLATE D		PLATE B	
	PL-axbxt	Dw	PL-axbxt	Dw	PL-axbxt	Dw
PG-21	PL-600x130x25	15 mm.	PL-600x200x20	15 mm.	PL-200x175x8 PL-200x225x8	6 mm.
PG-22	PL-600x130x20	12 mm.	PL-600x200x15	12 mm.	PL-200x175x8 PL-200x225x8	6 mm.
PG-23	PL-600x130x15	10 mm.	PL-600x200x15	10 mm.	PL-200x175x8 PL-200x225x8	6 mm.
PG-24	PL-600x130x12	8 mm.	PL-600x200x12	8 mm.	PL-200x175x8 PL-200x225x8	6 mm.
PG-25	PL-600x130x10	6 mm.	PL-600x200x12	6 mm.	PL-200x175x8 PL-200x225x8	6 mm.

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