Enhancing Progressive Collapse Resistance of Building using Energy Dissipation Devices

By

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14MCLC04

DEPARTMENT OF CIVIL ENGINEERING

INSTITUTE OF TECHNOLOGY

NIRMA UNIVERSITY

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Enhancing Progressive Collapse Resistance of Building using Energy Dissipation Devices

Major Project

Submitted in partial fulfillment of the requirements

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Declaration

This is to certify that

a) The major project comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis And Design) at Nirma University and has not been submitted elsewhere for a degree.

b) Due acknowledgement has been made in text to all other material used.

Parth D. Javia

Certificate

This is to certify that the Major Project Report entitled **"Enhancing Progressive Collapse Resistance of Building using Energy Dissipation Devices"** submitted by **Mr. Javia Parth D. (Roll No: 14MCLC04)**, towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis And Design) of Nirma University is the record of work carried out by him under our supervision and guidance. The work submitted has in our opinion reached a level required for being accepted for examination. The results embodied in this major project work to the best of our knowledge have not been submitted to any other University or Institution for award of any degree or diploma.

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Abstract

Progressive collapse denotes a failure of substantial part of the structure, causing greater damage to the structure than the initial damage. It is initiated by failure of a relatively small part of the structure such as failure of any vertical load carrying elements (typically columns). Failure of large part of any structure will results into substantial loss of human lives and natural resources. Therefore, it is important to prevent progressive collapse which is also known as disproportionate collapse.

Generally, viscoelastic dampers are used for improving performance of building during earthquakes. In the present study, effect of viscoelastic dampers on progressive collapse resistance of 4-storey reinforced concrete frame structure, 4-storey reinforced concrete symmetric building and 12-storey resendential building is evaluated. Three different damping i.e. 10%, 15% and 20% is considered for viscoelastic dampers. Linear static, Linear dynamic, Nonlinear static and Nonlinear dynamic analysis are performed by following U. S. General Service Administration (GSA) and Department of Defense (DoD) guidelines for evaluating progressive collapse potential. Modeling and analysis is performed using SAP2000 for different threat independent column removal scenarios. Demand Capacity Ratio (DCR) is calculated using alternate load path method for linear static analysis. Linear dynamic analysis is performed to obtain displacement at location of removed columns.

Nonlinear Static (Push Down) analysis is performed for evaluating the progressive collapse load resistance capacity. Nonlinear dynamic analysis is carried out to obtain the vertical deflection at the location of column removal. From the analysis results, it is observed that viscoelastic dampers contributes in load resistance and enhances the performance of building during progressive collapse scenario. Also the vertical deflection at column removal location is decreased to significant level.

It is also observed that DCR in beams and columns of buildings without dampers are exceeding the allowable limit i.e. 2 for flexure and 1 for shear and column, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly, which indicates enhanced progressive collapse resistance of building. It is also evident that, displacement at the location of column removal is maxi-

mum in case of building without dampers. After incorporating viscoelsatic damper with suitable damping, considerably reduces the displacement upto 50%-70% at the location of removed column. Viscoelastic dampers significantly increases load resistance capacity of structure with significant reduction in vertical deflection at the location of removed column. Formation of 1^{st} hinge in the frames with viscoelastic dampers has 35%-70% more load resistance capacity as compared to frames without dampers. Similarly, load resistance capacity corresponding to collapse load increases by 40%-70% for frames with viscoelastic dampers as compared to frames without dampers.

Abbreviation, Notation and Nomenclature

AP .... Alternate Load Path Method
DCR .... Demand Capacity Ratio
DoD .... Department of Defence
FEMA .... Continuous Wavelet Transform
GSA .... U.S. General Service Administration
UFC .... Unified Facilities Criteria
SAP .... Structural Analysis Program
WTC .... World Trade Center
PVC .... Polymethyl Chloride
PMMA .... Polymethyl Methacrylate
DL .... Dead Load
LL .... Live Load
A_{sv} .... Cross sectional area of vertical legs of stirrups
τ_c .... Design Shear Strength of Concrete
p_t .... Percentage of tensile reinforcement
b .... Width of Beam
f_{ck} .... Characteristic compressive strength of concrete
f_y .... Characteristic yeild strength of steel
M_u .... Moment capacity of beam
V_c .... Shear resisted of concrete
V_{us} .... Shear resisted by shear reinforcement
M_{ux} .... Moment about x axis due to design load
M_{uy} .... Moment about y axis due to design load
S_v .... Spacing of the stirrups
V_s .... Total shear resisted by section
K_d .... Stiffness of a Viscoelastic Damper
G' .... Storage Modulus of Viscoelastic Material
G" .... Loss Modulus of Viscoelastic Material
t_d .... Thickness of Viscoelastic material
A_d .... Area of Viscoelastic Pad

C_d .... Additional damping provided by Viscoelastic Damper

γ .... Shear strain of Viscoelastic material

ζ .... Desired damping ratio for Dampers

η_d .... Loss Factor for Viscoelastic material

ω_n .... Natural Frequency of Structure

T .... Operating Temperature

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Chapter 1

Introduction

1.1 General

The term progressive collapse has been used to explain the spread of an initial local failure in a manner similar to a chain reaction that leads to partial or total collapse of a building [17]. The concept of progressive collapse can be demonstrated by the famous 1968 collapse of the Ronan Point apartment building . The structure was a 22-story precast concrete, bearing wall building. A gas explosion in a corner kitchen on the 18^{th} floor blew out the exterior wall panel and failure of the corner bay of the building propagated upward to the roof and downward almost to ground level as shown in Fig.1.1. Progressive collapse is the expansion of local damage, from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it and is also known as disproportionate collapse.

Progressive collapse occurs when a structure has its loading pattern or boundary conditions changed such that structural elements are loaded beyond their capacity and fail. The remaining structure is forced to pursue alternate load paths to redistribute the unbalanced force. As a result other elements may fail causing further load redistribution. The process will continue until this supplementary forces are balanced.

In order to prevent the progressive collapse, structure should be capable for providing alternate load path to redistribute additional forces, when one or more column is removed. Prevention or mitigation of progressive collapse appears to be an important issue

Figure 1.1: Ronan point appartment collapse

in the development of several structural design codes. US General Service Administration (GSA) [14] and Department of Defense guidelines (DoD) [15] have issued design and analysis guidelines for progressive collapse evaluation of building structures. Linear static, Linear dynamic, Nonlinear static and Nonlinear dynamic analysis have been recommended to estimate the alternate paths to transfer loads under sudden column removal scenario from critical location.

Khobar Towers was a complex of numerous apartment buildings in Al-Khobar near Dhahran, Saudi Arabia. On June 25, 1996, one of the apartment buildings was extensively damaged and others were seriously damaged when a massive bomb was detonated in the road way that passed in front of the building as shown in Fig.1.2

1.2 Mechanism of Progressive Collapse

Progressive collapse is activated by localized damage that can not be restricted and leads to a chain reaction of failures resulting in a partial or total structural collapse, where the final damage is out of proportion compared to the local damage from the initiating

Figure 1.2: Collapse of Khobar Towers

event. Once a column is failed the buildings weight (gravity load) transfers to neighboring members in the structure. If these members are not properly designed to resist and redistribute the additional load that part of the structure fails. The vertical load carrying elements of the structure continue to fail until the additional loading is stabilized.

1.3 Causes of Progressive Collapse

The initial local damage of structural elements of the building may occur under emergency situations (gas explosions, terrorist attacks, aircraft, fires, seismic impacts and failures of footings, assaults transport, defects of design, construction or reconstruction, etc.) which are not considered by the terms of the normal operation of the building. Accidents and damages of load bearing structures, caused by design, manufacture or installation errors, inadequate quality of materials, and improper use of buildings can also be reasons of collapse.

A number of potential abnormal load hazards, which could trigger progressive collapse are as follows:

1. Gas Explosions
2. Bomb explosion (Blast load)
3. Design/Construction error
4. Fire
5. Overload due to occupant misuse
6. Vehicular collision
7. Aircraft Impact
8. Transportation and storage of hazardous materials

1.4 Viscoelastic Dampers

In passive energy dissipation system the motion of structure is controlled by installing devices to structure which can suitably modify stiffness, mass and damping properties of structure. Passive energy dissipation devices can be effective against winds and earthquake induced motion [18].

1.4.1 Features of Viscoelastic Damper

Viscoelastic dampers are widely used passive energy dissipation system. Some of the features of viscoelastic dampers are as :

- Viscoelastic dampers are lateral load carrying elements and are designed such that part of the mechanical energy of the building motion is transferred into heat, which results in reduction of amplitude of the vibratory motion. The medium in which this transfer of energy takes place is a viscoelastic material.
- The damping achieved is mostly due to shear deformation of viscoelastic material.

- The most common type of Viscoelastic damper is formed of two layers of viscoelastic material bonded between a central driving plate and two outer plates as shown in Fig.1.3. These devices significantly increase the capacity of the structure to dissipate energy, but have the little influence on the natural periods, which are shortened by about 10% to 20%. Energy is dissipated by relative motion between the outer steel flanges and the center plate of the device.

- Viscoelastic dampers show significant potential for providing economic structures, which can behave elastically and develop small drifts even when subjected to a major earthquake thereby protecting both structural and non structural components.

- Viscoelastic dampers provide velocity dependent damping force which increases the damping in structure and results in reduction of vibration. The viscoelastic damper has another benefit of adding stiffness to the structure. Thus, the addition of viscoelastic dampers consistently reduces the displacement demands and thus decreases or eliminates the nonlinear response in the primary structure.

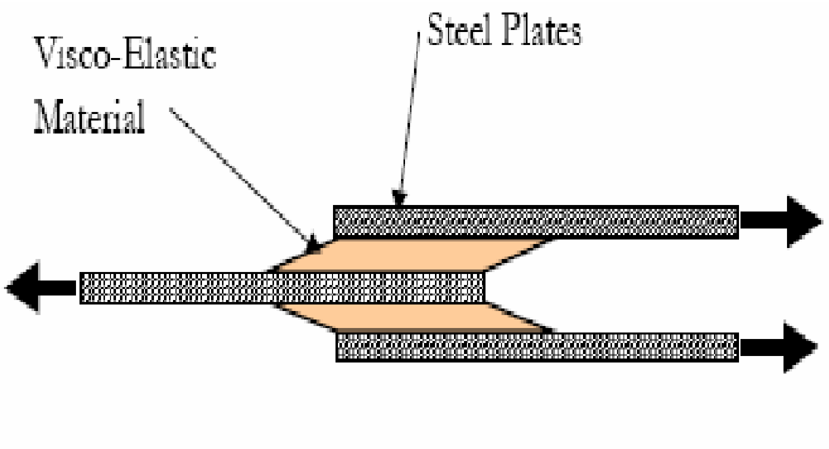


Figure 1.3: Viscoelastic Damper

1.4.2 Various types of Viscoelastic material

Due to the effectiveness of the viscous fluid and viscoelastic dampers in reducing the response due to the seismic excitations and the wind loads, many buildings were constructed with these dampers. One of the most famous buildings in the world, the World

Trade Center, New-York 1969, had about 20,000 viscoelastic dampers in the two towers. The viscoelastic dampers were used to increase the resistance of the tubular steel frame against the wind induced building oscillations. The various viscoelastic material used are as follows [18]:

Table 1.1: Types of Viscoelastic Materials

Sr No	List of some polymer types
1	Acrylic rubber
2	Butadiene rubber(BR)
3	Butyl rubber
4	Chloroprene
5	Chlorinated Polyethylenes
6	Ethylene Propylene
7	Fluorosilicone rubber
8	Fluorocarbon rubber
9	Nitrile rubber
10	Natural rubber
11	Polyethylene
12	Polystyrene
13	Polymethyl Chloride(PVC)
14	Polymethyl Methacrylate(PMMA)
15	Polybutadiene

1.5 Objective of Study

The objectives of present study are as :

- To study the basics of Progressive Collapse.
- To study the various analysis approaches for evaluation of the progressive collapse potential of building.
- To study the effectiveness of viscoelastic dampers during progressive collapse.

- To study the mitigation measures of progressive collapse to improve the capacity of building to resist progressive collapse.

1.6 Scope of Work

In order to achieve the above outlined objective of work, following scope of work is identified.

- Study the effectiveness of dampers during progressive collapse.
- Design of viscoelastic dampers
- Performing Linear static, Linear dynamic, Nonlinear static and Nonlinear dynamic analysis on 4-storey 2-D frame structure, 4-storey Symmetric building and 12 Storey Residential Building.
- Mitigation of Progressive Collapse prone building by introducing Viscoelastic damper and performing Linear static, Linear dynamic, Nonlinear static and Nonlinear dynamic analysis using software SAP2000.

1.7 Organisation of Major Project

The contents of major project report is divided into various chapters as below.

Chapter 1 presents the introduction and overview of progressive collapse. The mechanism of progressive collapse and causes are discussed. Introduction of viscoelastic dampers is included in this chapter. It also includes objectives of study and scope of work.

Chapter 2 includes brief literature review pertaining to progressive collapse of structures, various analysis procedures to evaluate progressive collapse and mitigation of progressive collapse and effect of viscoelastic dampers.

Chapter 3 discusses progressive collapse analysis of 4-storey 2-D Frame reinforced concrete frame. Evaluation of progressive collapse potential of seismically designed building is carried out by following U.S. General Service Administration (GSA) and Department of Defense (DoD) guidelines.

Chapter 4 presents progressive collapse analysis of 4-storey Symmetric reinforced concrete building. Analysis is performed using structural analysis program SAP2000 by following alternate load path method. The demand capacity ratios found using linear static analysis for frame without damper and with damper is compared. The displacement at the column failure point is compared for linear dynamic analysis and nonlinear dynamic analysis. Also load resisting capacity is compared for nonlinear static analysis.

Chapter 5 includes progressive collapse analysis of 12-storey Residential reinforced concrete building.

Chapter 6 summarizes the work carried out in the major project. It also includes conclusions derived from the study and future scope of work.

Chapter 2

Literature Review

2.1 General

Literature in form of research papers regarding various aspects of progressive collapse analysis are referred and review is presented in this chapter.

2.1.1 Progressive Collapse Analysis

Marjanishvili and Agnew [1] studied four different analysis procedures e.g. linear static, nonlinear static, linear dynamic, and nonlinear dynamic and explained step by step procedure using software SAP 2000. Nine-storey steel moment frame structure was considered with composite slab. For linear static analysis load increase factor 2 was multiplied to suffice dynamic and nonlinear behaviour and DCR was compared as per GSA guidelines and were found within safe limit. Nonlinear static analysis was performed assigning non linear hinges to members and found that first plastic hinge was formed at 48 % of progressive load and collapse load was at 66 % of progressive load and deflection at failure was 190 mm. Linear dynamic analysis was performed with zero initial condition in time history analysis. Maximum deflection for linear dynamic analysis was found to be 153 mm slightly less then linear static analysis. Likewise nonlinear dynamic analysis was performed and result were evaluated based on maximum rotation and maximum ductility which were 2.17 ° and 3.5 respectively. Maximum deflection was 281 mm. For further studies the load was increase by a factor which results DCR value near to 3 and all the four procedure were carried out for various cases deflection and rotation were measured.

McKay et al. [2] formulated new Load Increase Factor(LIF) and Dynamic Increase Factor(DIF). As Progressive collapse is a dynamic and nonlinear event, the load cases for the static procedure requires the use of factors to account for the dynamic and nonlinear effect. The LIF and DIF used by GSA and DoD was 2 and yielding over conservative results. Based on the nonlinear dynamic analysis for the extreme load condition of (1.2DL+0.5LL) value of plastic rotation and displacement were noted at the column removal location. Linear static analysis was performed with trial LIF value and was re-run until it matches the value of displacement as obtained from nonlinear dynamic analysis. Like wise nonlinear static analysis was performed with trial DIF value and re-run until rotation value matched for nonlinear dynamic analysis. As LIF and DIF value changes with section properties and geometries, graph of normalised rotation against various LIF value was plotted and the linear fit of the data was performed. The equation obtained for RCC structure was LIF= (1.2m+0.80) for Steel structure LIF=(0.9m+1.1), where m was direct multiplier on the expected component strengths given in the revised UFC 4-023-03. Similarly graph of normalised rotation against DIF was plotted and the best fit of the data was performed yielding the equation for RCC structure as

$$DIF = 1.04 + \frac{0.45}{\frac{\Theta_{all}}{\Theta_{yeild}} + 0.48}$$

and for Steel structure as

$$DIF = 1.08 + \frac{0.76}{\frac{\Theta_{all}}{\Theta_{yeild}} + 0.83}$$

where Θ_{all} was allowable rotation and Θ_{yeild} was yeild rotation.

Kima and Kimb [3] presented two types of analysis model structure to assess potential for progressive collapse. Gravity load resisting system (GLRS) in which gravity load was resisted by steel moment resisting frames while lateral load was resisted by shear walls. Lateral load resisting system(LLRS) in which steel moment frames were design to resist both gravity and lateral loads. Linear static analysis was performed on the structure assigning the hinge to the member and was rerun until DCR value was exceeded the limiting value for three different column removal(corner, second left, centre) scenario for GLRS and LLRS structure using both guidelines. Linear dynamic analysis was performed and as compared to linear analysis less hinge were formed and less DCR value was obtained.

Also the vertical displacement obtained was much lesser than linear static analysis. Comparison of nonlinear dynamic and linear dynamic analysis was done in terms of vertical displacement having significant changes. Rotation of members and ductility was also found out for both GLRS and LLRS structure. Removal of corner column showed the higher possibility of progressive collapse.

Tavakoli and Alashti [4] considered 3D and 2D models of multi-storey MRF steel structure. Pushover analysis was performed on these models, with different locations of column elimination, the hinge rotation in beams and columns was checked and compared with progressive collapse acceptance criteria. Two lateral loading pattern were selected 1) Triangular distribution and 2) Uniform distribution. 5-story and 15-story MRF buildings with 4 and 6 bays were prepared to assess progressive collapse. 2D Push over analysis was performed and showed that uniform distribution has higher base shear capacity than a triangular pattern. 3D Pushover analysis was carried out which suggested that base shear capacity increased, with increase in number of bays. Robustness of a member was found out by comparing the base shear capacities before removal and after removal of column by performing lateral nonlinear pushover analysis. Ductility of members were found out by dividing maximum deflection to yield deflection for 5-storey and 15-storey structure and were within safe limit of GSA 2003 guidelines. Target displacement was applied to all the four structures and it was found that all the column remain in elastic region and did not exceeded collapse prevention (CP) level.

Rahai et al. [5] evaluated the performance of the RC load bearing wall 10-story structure under progressive collapse. The RC load bearing wall system was modelled with PERFORM 3D software. Nonlinear material behaviour of RC load bearing walls were defined using fiber sections and assigning nonlinear material stress-strain curve to these fibers. The geometric nonlinear behaviour of the elements in all analyses was considered by including P-Δ effects. The stress-strain curve of confined concrete was assigned to the concrete fibers of walls boundary and the stress-strain curve of unconfined concrete was assigned to the concrete fibers of walls web. RC load bearing wall sections were removed as recommended by GSA guidelines. Pushdown analysis was performed for three different wall removal scenario wall W1 and W2, wall W4 and wall W5 as shown in Fig. 2.1. The maximum displacement for all three wall removal were obtained as 5.07 mm for W1 and W2, 2.08 mm for W4 and 1.66 mm for W5. Also the load bearing walls were removed at various locations in different stories and vertical displacement were found at various joints. It was found that potential of collapse increases where the cross-section of member is changed.

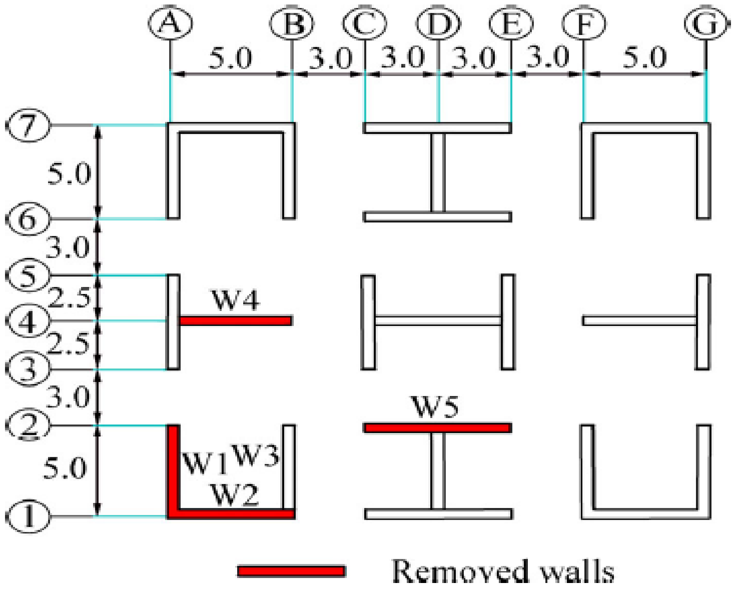


Figure 2.1: Structural plan of RC load bearing wall system

Ren et al. [6] considered two typical 15-storeyed building models designed with equivalent overall lateral resistance to seismic actions. Building A was a weak wall-strong frame structure while building B was a strong wall-weak frame system. The progressive collapse resistances of the frames and the shear walls in both structures was evaluated under various column (shear wall) removal scenarios. The height of the first story was 4.5 m and that of each remaining stories was 3.6 m. The total height of each building was 54.9 m. The dead load on each story was 7.0 kN/m^2, whereas the live load on each story was 2.0 kN/m^2. Building A was having higher reinforcement ratio in its frame beams and columns than that of building B, which in turn leads to higher redundancy of the frames in Building A in resisting progressive collapse.

The shear walls were singly arranged in building A but in building B, they were arranged in a common C shape as shown in Fig. 2.2. The finite element models of buildings A and

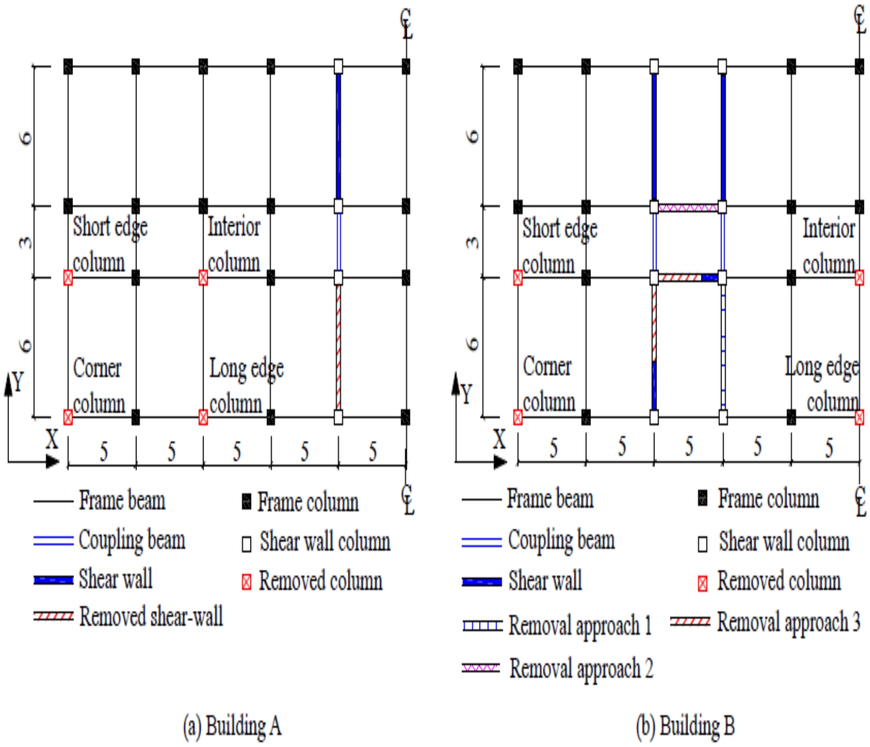


Figure 2.2: Floor plan of the building models

B were established based on the general finite-element program MSC.MARC (MSC 2007). The beams, columns and coupling beams were simulated using the fiber beam model developed by the authors, whereas the shear walls were simulated using the multilayer shell model of MSC.MARC. Nonlinear dynamic alternate load path analysis was carried out for four different column removal scenarios (corner column, long edge column, short edge column and an interior column) on each story. For shear wall removals in Buildings A and B. 2H length was removed (where H is storey height) for length of shear wall greater than 2H; whereas if the length of the wall is less than 2H, the entire length of the wall was removed. No progressive collapse occurs in building A for removal of any column from any story. For building B, progressive collapse does not occur when the short edge column on any story is removed. However, collapse was triggered when the corner, the long edge or the interior column was removed from any story. For building A, the analysis results indicate that progressive collapse does not occur when the shear wall is removed from any representative story. For the prevention collapse of building B the linear static and nonlinear dynamic method was rerun by increasing the reinforcement in the critical section until it reached collapse resistance.

Kokot et al [7] experimentally tested the 3 storied and 2-bay reinforced concrete frame building with 0.24 m thick slab, height of each storey 2.7 m and width of bay 6 m and 4 m. The structure was first tested for the designed earthquake and suffered minor damage. After that middle column were cut one after another and the building survived in the absent of load bearing member. For collapse of the structure it was decided to progressively destroy two external columns. The experiment took only the static behaviour of the structure. Finite element model was created in SAP2000 containing 186 frame elements and 171 nodes. Linear static analysis was performed on 1) CASE-1 removal of central column, 2) CASE-2 left corner column removal and 3) CASE-3 right corner column removal and was found critical for CASE-1 and CASE-2, by evaluating DCR values it was found that structure was not susceptible to progressive collapse. Linear-dynamic analysis was performed to evaluate the actual behaviour of structure under column removal scenario. It was found that the for CASE-1 and CASE-2 the DCR value exceeded the limiting value and was prone to progressive collapse. Nonlinear dynamic analysis procedure was performed by assigning hinge properties to beams and columns. The structure was found to be safe for all the three cases. Also as the structure was examined experimentally by removal of two columns and was found safe with more vibrations due to low rigidity as shown in Fig. 2.3.

Figure 2.3: Removal of columns in experimental setup

2.1.2 Progressive Collapse Mitigation Techniques

Patel and Joshi [8] studied progressive collapse potential of 4-storey and 10-storey asymmetrical concrete frame building by linear static and linear dynamic analysis and modelling was performed in SAP 2000 for 5 different threat-independent column removal conditions. In alternate load path method original structure was designed for gravity and seismic loading. Column was removed at ground floor depending on various cases and loading to the critical sections were given as per GSA 2003 and UFC 2009 guidelines. Demand in terms of shear force and bending moment was evaluated from the analysis and DCR of each member was calculated and compared to limiting DCR values for flexure, shear and axial. Result showed that members were exceeding the limit of failure for all 5 column removal locations and were critical in case-4. Three new techniques were proposed to mitigate progressive collapse. Alternative-1 By providing bracing at top storey level. Alternative-2 moderate increase in the size of frame member for all storey level. Alternative-3 Significant increase in size of frame member at bottom two storey level. DCR for flexure was calculated at three points left, centre and right side of the column removal position and was found within the limiting value for the flexure for all the three alternatives. Likewise DCR for critical column were noted at various storey level before mitigation and for three alternatives. Deflection was also observed at column removal point and was significantly reduced for all three alternatives. Also the comparison of cost was done in terms of additional concrete for three alternatives and Alternative-1 was found cost effective.

Kim et al. [9] evaluated progressive collapse potential of braced frames with eight different configuration using nonlinear static and nonlinear dynamic analysis. For bracing system special concentric braced frame(SCBF) were used. For static analysis GSA(2003) and UFC-DoD(2005) amplification factor 2.0 in load combination was used. The nonlinear static pushdown analysis method was applied to investigate vertical displacement in the location of removed column. Configuration for concentric bracing include diagonal braces, X-type braces, V-type and inverted V(chevron type). The analysed structure consist of four storey with four bay of 6.1 m bay length and 3.1 m storey height subjected to loss of first storey centre column. For all eight configuration graph of vertical displacement versus load factor was plotted and a graph of axial force versus yield strength for tension members and axial force versus buckling load for compression member and the buckling of all critical members are plotted. For the moment resisting frame with Knee bracing the load factor obtained was highest which was 3. Nonlinear dynamic analysis was performed on all eight structure and time history is plotted against vertical displacement and was found to approach the static result. The nonlinear static analysis result showed that the model structure was having strength twice as required by the GSA guidelines.

Alrudaini and Hadi [10] took a ten storey typical building structure having four bays of 6.5 m in both directions. Height of first storey was 5.0 m and other storey were 3.0 m. Size of column was 0.6 m × 0.6 m and that of beam was 0.3 m × 0.6 m. To prevent progressive collapse vertical cables were embedded in column and hanged to the hat braced frame placed on the top of the building. The bearing capacity of the cable was 5560kN, having diameter 82 mm and modules of elasticity 158 GPa. The whole structure was modelled in ANSYS 11.0. Nonlinear dynamic analysis was performed with sudden column removal scenario. The first floor corner column was removed suddenly and result were matched for structure without mitigation technique and with. The rotation above the removed column was exceeding the criteria limit(0.024 rad) for structure without mitigation technique and was found to be 0.0069 rad for the structure with mitigation technique. Also the cables were in elastic region with maximum tension force of 2930 kN. So the structure was not prone to progressive collapse after introducing the mitigation technique.

2.1.3 Viscoelastic Dampers

Jinkoo Kim and Sunghyuk Bang [11] studied a strategy developed for an appropriate plan-wise distribution of viscoelastic dampers to minimize the torsional responses of an asymmetric structure, with one axis of symmetry subjected to an earthquake-induced dynamic motion. The modal characteristic equations of a single-storey asymmetric structure with four corner columns and added viscoelastic dampers were derived, and a parametric study was performed to identify the design variables that influence the torsional responses. Based on the results of parametric study, a simple and straight forward methodology to find out the optimum eccentricity of added viscoelastic dampers to compensate for the torsional effect of a plan-wise asymmetric structure was developed using modal coefficients. The results indicates that the torsional response of asymmetric structures can be reduced significantly following the proposed method, and that the viscoelastic dampers turn out to be more effective than viscous dampers in controlling torsional response of a plan-wise asymmetric building structure.

2.2 Summary

All these papers gives an idea about the various research works carried out on progressive collapse analysis and its mitigation techniques. These research paper gives an idea about the various methods for progressive collapse analysis and various techniques to mitigate its effect.

Chapter 3

Progressive Collapse Analysis of Framed Structure with Viscoelastic Dampers

3.1 General

In progressive collapse, the failure of few primary structural components leads to the redistribution of forces in adjoining members and it further causes the failure of these adjoining members. As a result, a substantial part of the structure may collapse, causing greater damage to the structure than the initial impact. To study the effect of failure of primary structural component on the entire structure, one 4-storey reinforced concrete (RC) frame is analyzed for progressive collapse using the structural analysis and design software SAP2000 [22].

Performance of building designed for seismic loading is evaluated under progressive collapse. Linear static, linear dynamic, nonlinear static and nonlinear dynamic analyses are performed to evaluate the potential for progressive collapse of building designed for seismic loading. Alternate load path method is used to determine the capacity of structure to link over the removed element by following the U. S. General Service Administration (GSA) [14] and Department of Defense (DoD) guidelines [15]. The Demand Capacity Ratio (DCR) is calculated at each storey for linear static analysis. DCR is calculated at three locations left, center and right side of removed column for the two different column removal as highlighted in Fig. 3.1. Comparison of DCR for both the analysis is carried out for each column removal case.

Study of the vertical displacement at the column removal point is carried out for linear dynamic and nonlinear dynamic analysis and is compared with frame, with and without damper. The displacement obtained from linear static analysis is compared with the maximum displacement obtained from linear dynamic analysis. Nonlinear static analysis of frame structure is performed to determine the hinge formation pattern. Comparison of linear static and nonlinear static analysis is also carried out. Nonlinear dynamic analysis is performed to understand the behavior of building considering both material and geometrical nonlinearities.

3.2 Problem Formulation

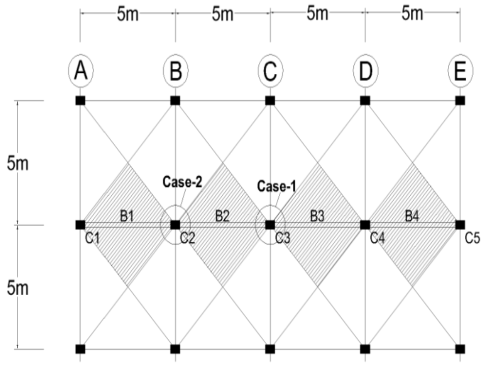
In this study, progressive collapse potential of 4-storey building frame, is evaluated. 2-D frame, considered for the study, is extracted from building having overall plan dimensions 10 m $\times$ 20 m as shown in Fig. 3.1, by transferring forces of slabs on beams. The frame is having 4 bays at 5 m c/c spacing as shown in Fig. 3.1. Total height of the frame is 12.7 m having the first storey height as 3.4 m and height at all other storey is 3.1 m. Typical elevation of the 4-storey frame considered for the study is shown in the Fig. 3.1. Walls of 115 mm thickness are considered on all the beams. Frame is analysed and designed by considering seismic forces. Modelling, analysis and design is carried out using SAP2000. Progressive collapse potential for frame is carried out for two different column lost scenarios as highlighted by a circle in Fig. 3.1.

3.3 Loading Data

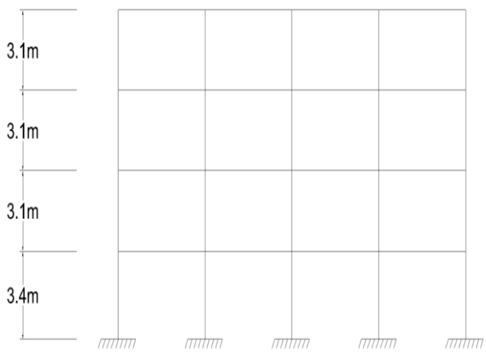
4-Storey 2-D Frame is analyzed and designed by considering following loading parameters and material properties.

Gravity Loading Parameters:

- Dead Load : Self weight of the structural elements



(a) Plan of frame



(b) Elevation of frame

Figure 3.1: Frame configuration

- Live Load on Roof : 1.5 kN/m^2
- Live Load on Floors : 3.0 kN/m^2
- Floor Finish : 1.5 kN/m^2
- Wall Load : 6.325 kN/m^2

Seismic Loading Parameters:

- Seismic Zone : 5
- Soil type : Medium (II)
- Importance Factor : 1

Material Properties:

- Grade of Concrete : M25
- Grade of Steel : Fe415

Building Configuration:

- Slab Thickness : 150 mm
- Beam Size : 300 mm × 350 mm
- Column Size : 350 mm × 500 mm

- Wall Thickness : 115 mm
- Bay Span : 5 m
- Bottom Storey height : 3.4 m
- Typical Storey height : 3.1 m

Seismic design of the building is carried out for the governing load case, out of following load combinations as suggested by IS 1893 (part 1) : 2002 [21].

- 1.5 (DL + LL)
- 1.2 (DL + LL ± EQx)
- 1.5 (DL ± EQx)
- (0.9DL ± 1.5EQx)

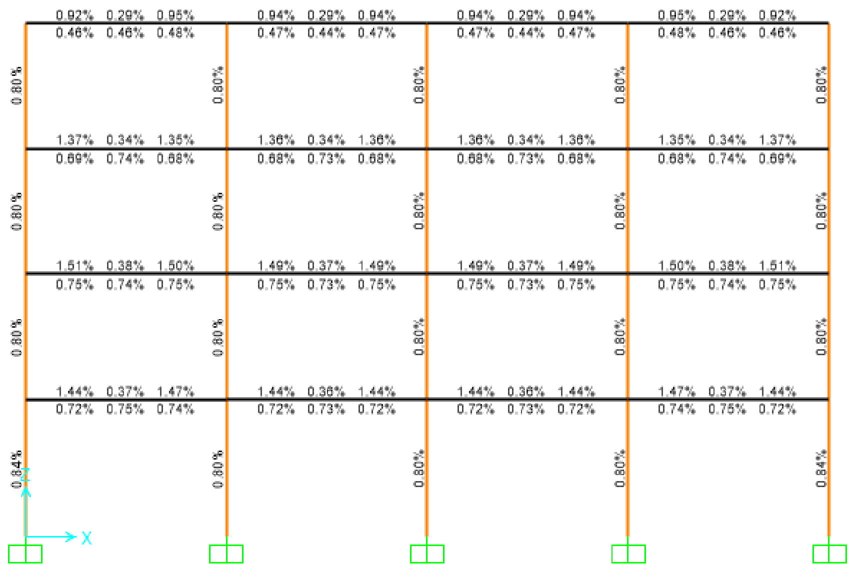


Figure 3.2: Percentage reinforcement in elevation

Fig. 3.2 shows the percentage reinforcement required for 2-D frame. The typical reinforcement detailing of beam at first floor level is shown in the Fig. 3.3. Reinforcement details for beams at all the floors are given in Table 3.1. Typical reinforcement detailing of column section is shown in Fig. 3.4. Reinforcement details for all the columns at different floor levels are given in Table 3.2

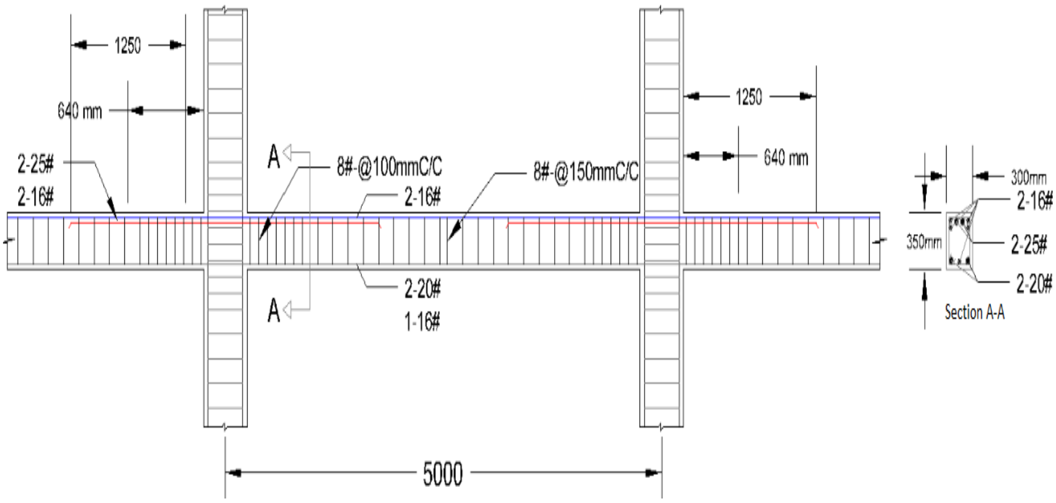


Figure 3.3: Typical reinforcement detailing of beam at first floor level

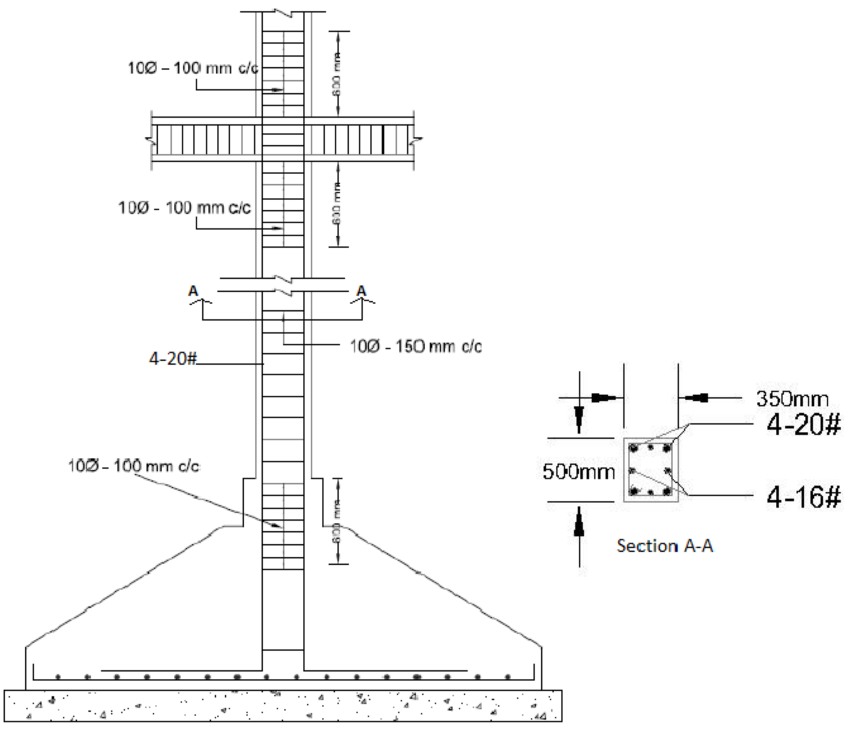


Figure 3.4: Typical reinforcement detailing of column

Table 3.1: Reinforcement detailing of all the beams

Storey	Beam size (mm)	Top Steel	Extra Top Steel	Bottom Steel	Stirrups (up to 640mm) from support	Stirrups in remaining portion
Srorey-1	300 × 350	2-16#	2-25#, 2-16#	2-20#, 1-16#	10#-100c/c	10#-150c/c
Srorey-2	300 × 350	2-16#	2-25#, 2-16#	2-20#, 1-16#	10#-100c/c	10#-150c/c
Srorey-3	300 × 350	2-16#	2-20#, 2-20#	2-20#, 1-16#	10#-100c/c	10#-150c/c
Srorey-4	300 × 350	2-16#	2-20#, 2-12#	2-16#, 1-12#	10#-100c/c	10#-150c/c

Table 3.2: Reinforcement detailing for of all the columns

Column No	Column size	Vertical bar	Stirrups
STOREY-1			
C1, C2, C3, C4, C5	350 × 500	4-20#, 4-12#	8#-150c/c
STOREY-2			
C1, C2, C3, C4, C5	350 × 500	4-20#, 4-12#	8#-150c/c
STOREY-3			
C1, C2, C3, C4, C5	350 × 500	4-20#, 4-12#	8#-150c/c
STOREY-4			
C1, C2, C3, C4, C5	350 × 500	4-20#, 4-12#	8#-150c/c

3.4 Progressive Collapse Analysis

After designing of 2-D frame, the vertical member as shown in Fig 3.1 is removed separately from bottom storey level. These two cases have been considered based on exterior and interior condition for column removal given by guidelines [14].

3.4.1 Linear Static Analysis

In linear static analysis column is removed from the location being considered and analysis is carried out for following vertical load which shall be applied downward on the structure.

As per GSA guideline, Load = 2(DL + 0.25LL)

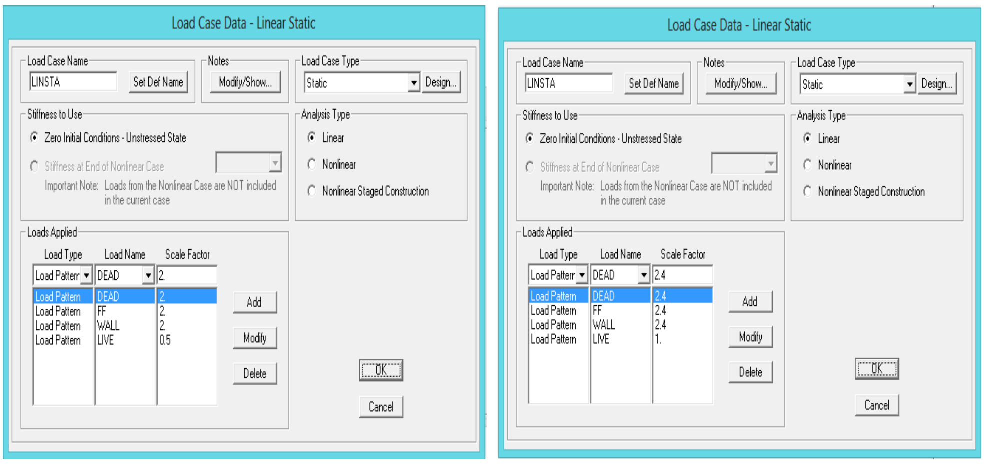
As per UFC guideline, Load = 2(1.2DL + 0.5LL) + 0.2WL

Where,

- DL = Dead load, LL = Live load and WL = Wind load

Steps to perform linear static analysis :

- Build a finite-element computer model in SAP2000 with loadings specified above;
- Apply the amplified static load combination as shown in Fig 3.5;
- Perform static linear analysis, a standard analysis procedure in SAP2000 [22];
- Evaluate the results based on demand to capacity ratio DCR.



(a) GSA load case (b) UFC load case

Figure 3.5: Linear static analysis case definition in SAP2000

3.4.2 Linear Dynamic Analysis

The failure of vertical members under extreme events, such as blast and impact, is a highly dynamic phenomenon. So it is necessary to study the response of building structure by performing dynamic analysis. Linear dynamic analysis method involves real-time removal of load carrying structural elements. Thus it is more appropriate to refer to this method of analysis as a time history analysis, Here in time history analysis, the frame is assumed to be at rest in its original configuration, and then subjected to a sudden column removal. To perform linear dynamic analysis the dynamic amplification factor of 2.0, used in the static analysis, is not considered because dynamic effect is already considered in analysis.

In the linear dynamic procedure the load applied is half of that applied in the static procedure. This difference in load application is for the reason that the dynamic effects are already considered in the time history analysis. Linear dynamic analysis is carried out for following vertical load which shall be applied downward on the structure.

As per GSA guideline, Load = (DL + 0.25LL)
As per UFC guideline, Load = (1.2DL + 0.5LL) + 0.2WL

Where,

- DL = Dead load, LL = Live load and WL = Wind load

Steps to perform linear dynamic analysis :

- Build a finite-element computer model in SAP2000 with loadings specified above;
- Apply the dynamic load combination as shown in Fig 3.8;
- Perform time history analysis as given in SAP2000 with zero initial conditions;
- Evaluate the results based on vertical deflection at column removal location.

Time history analysis is a step-by-step analysis of the dynamic response of a structure to a specified loading that may vary with time. A reaction is applied at the location of column removal as shown in Fig 3.6 and is make to zero after some elapsed time to incorporate dynamic effect using RAMPDOWN function.

To simulate the dynamic effect of column removal, reaction is applied at the column removal location and time history function is defined for this analysis as RAMPDOWN, at t = 0, f (t) = 1, and at t = 0.002, f (t) = 0, also at t = 1, f(t) = 0. The time history function is defined in SAP2000 is as shown in Fig 3.7 for gravity loading which remains constant thought analysis.

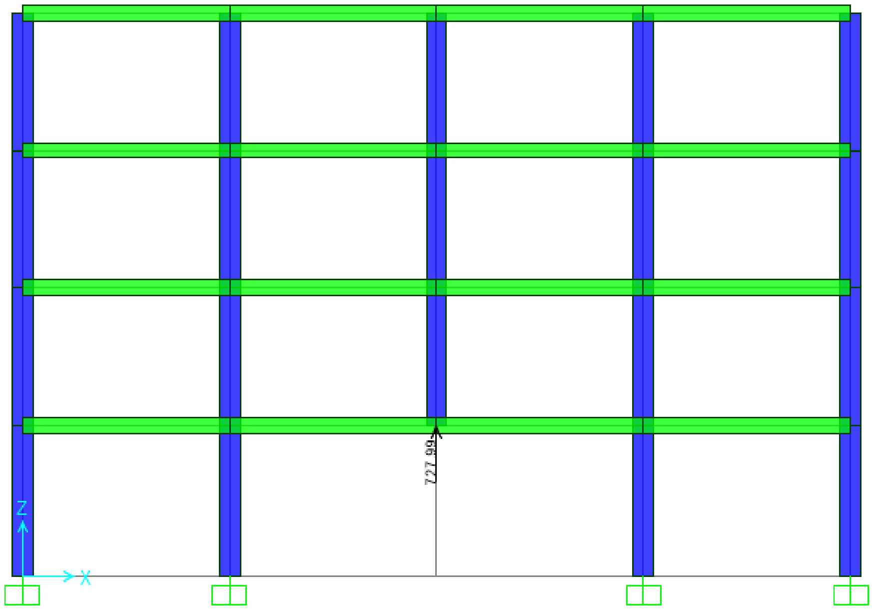
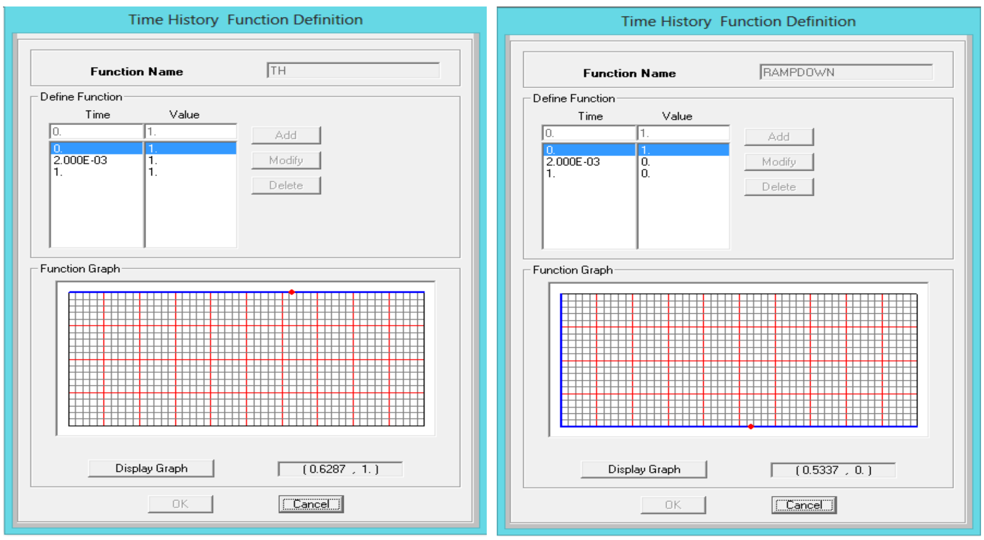
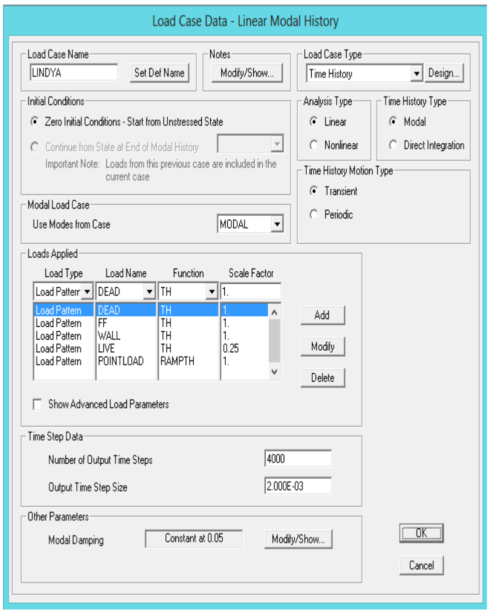


Figure 3.6: Reaction at column removal location

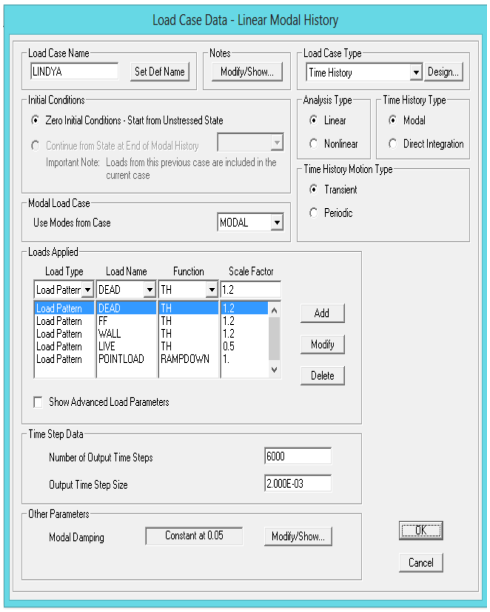


(a) Gravity load function (b) Point load function

Figure 3.7: Time history function definition in SAP2000 for linear dynamic analysis



(a) GSA load case



(b) UFC load case

Figure 3.8: Linear dynamic analysis case definition in SAP2000

3.4.3 Nonlinear Static Analysis

Nonlinear static analysis is widely used to analyze a building for a lateral load and is known as pushover analysis". In this study, vertical pushover analysis procedure is adopted to understand the behaviour of building structure. In this method load is applied step by step until maximum load is attained or until the structure collapses. In the nonlinear static analysis for progressive collapse, structural elements are allowed to deform beyond elastic limit, hence it undergoes in to the inelastic behavior. In progressive collapse analysis, vertical pushover is applied, using normal service loading, until the maximum load or the maximum displacement is attained.

Steps to perform nonlinear static analysis :

- Build a finite element computer model;
- Define and assign nonlinear plastic hinge properties, to beams and columns;
- Apply static load combination as shown in Fig. 3.9;
- Perform nonlinear static analysis;

- Verify and validate the results based on hinge formation.

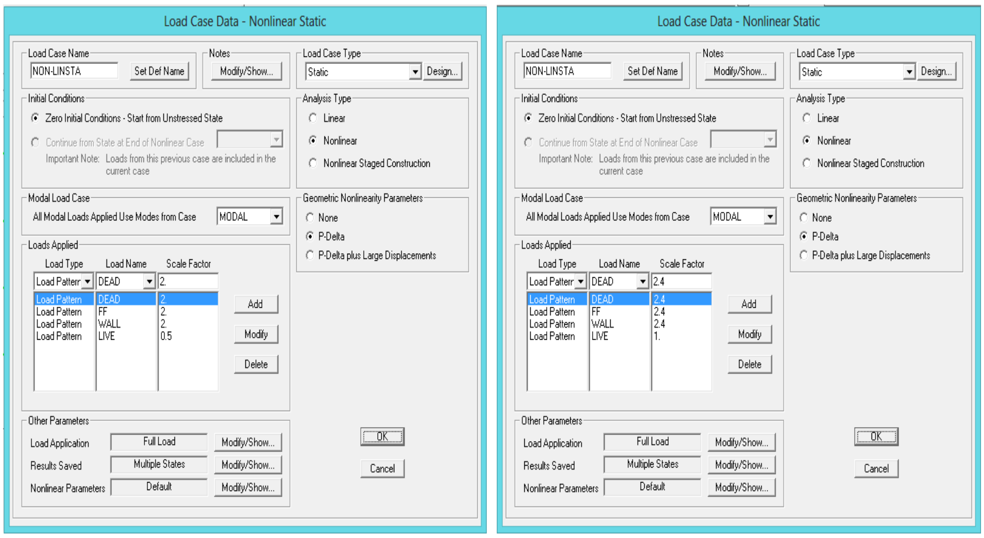
A new analysis case for the static nonlinear analysis is defined. Load case for static nonlinear analysis is taken same as given for static linear analysis as specified in guidelines. Nonlinear static analysis is carried out for following vertical load which shall be applied downward on the structure.

As per GSA guideline, Load = 2(DL + 0.25LL)

As per UFC guideline, Load = 2(1.2DL + 0.5LL) + 0.2WL

Where,

- DL = Dead load, LL = Live load and WL = Wind load

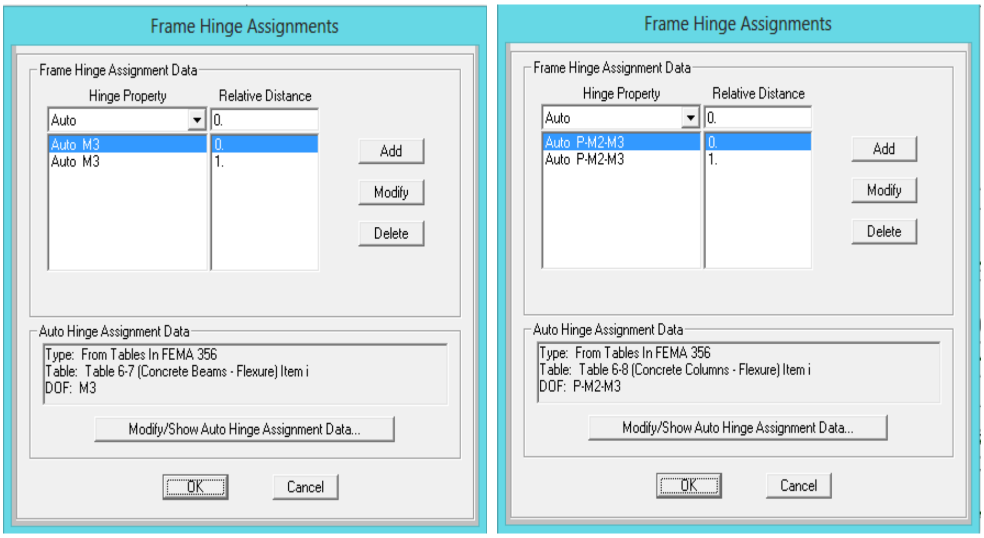


(a) GSA load case (b) UFC load case

Figure 3.9: Nonlinear static analysis case definition in SAP20

Load case data and parameters considered for nonlinear static analysis are presented in Fig. 3.9.

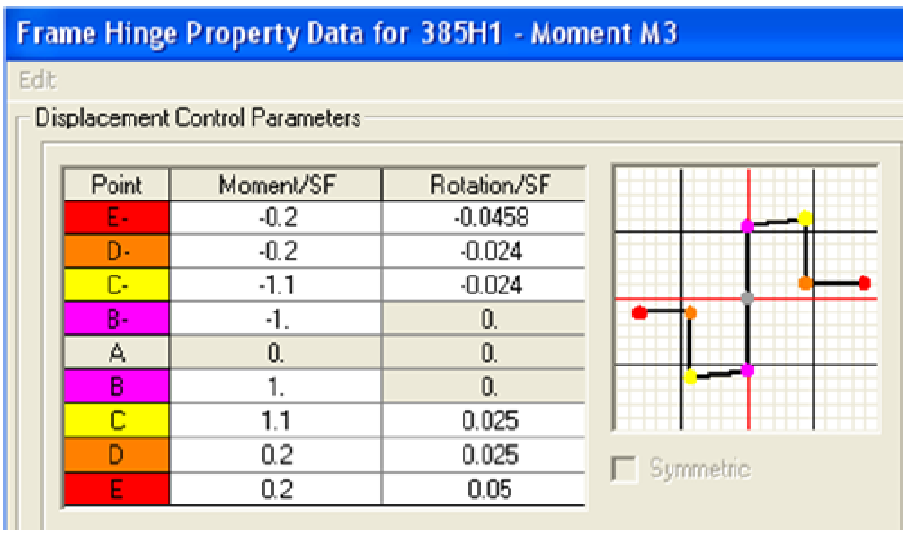
For nonlinear analysis automatic hinge properties and user-defined hinge properties are assigned to frame elements. When automatic or user-defined hinge properties are assigned to a frame element, the program automatically creates property for each and every generated hinge. For beam default hinge property Moment (M3) is assigned and for column coupled (P-M2-M3) hinge property is assigned as shown in Fig. 3.10. The hinge properties are calculated by the program for the cross section and reinforcement details provided as shown in Fig. 3.11.



(a) Hinge properties for beam (b) Hinge properties for column

Figure 3.10: Assigning default hinge property for beam and column

After assigning hinges to frame members nonlinear static analysis has been performed and results obtained at various displacement levels in terms of hinge formation due to column failure.


Frame Hinge Property Data for 385H1 - Moment M3

Edit

Displacement Control Parameters

Point	Moment/SF	Rotation/SF
E-	-0.2	-0.0458
D-	-0.2	-0.024
C-	-1.1	-0.024
B-	-1.	0.
A	0.	0.
B	1.	0.
C	1.1	0.025
D	0.2	0.025
E	0.2	0.05

Figure 3.11: Moment (M3) hinge property

3.4.4 Nonlinear Dynamic Analysis

The nonlinear dynamic analysis method is the most detailed method for the progressive collapse analysis in which a primary load-bearing structural element is removed dynamically and the structural material is allowed to undergo nonlinear behavior. This allows larger deformations and energy dissipation through material yielding, cracking, and fracture. But this analysis is usually avoided due to the complexity of the analysis. Evaluation and validation of the results obtained from nonlinear dynamic analysis can be very time-consuming which makes this analysis procedure even less attractive. In this analysis both material and geometrical nonlinearities are considered.

This dynamic effect of column removal is simulated by time history function. Time history function defined in SAP2000 for nonlinear dynamic analysis is shown in Figure 3.11.

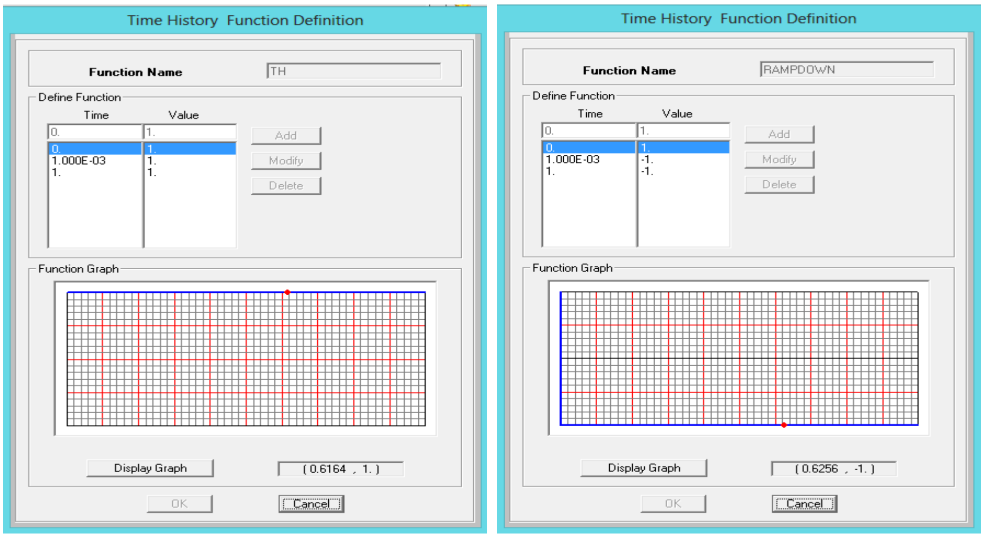
Nonlinear dynamic analysis is carried out by following vertical load which shall be applied downward on the structure.

As per GSA guideline, Load = (DL + 0.25LL)
As per UFC guideline, Load = (1.2DL + 0.5LL) + 0.2WL

Where,

- DL = Dead load, LL = Live load and WL = Wind load

To simulate the dynamic effect of column removal, reaction is applied at the column removal location and time history function is defined for this analysis as TH, at t = 0, f (t) = 1, and at t = 0.001, f (t) = 1, also at t = 1, f(t) = 1 and RAMPDOWN, at t = 0, f (t) = 1, and at t = 0.001, f (t) = -1, also at t = 1, f(t) = -1. The time history function definition in SAP2000 is shown in Fig. 3.12. The load ccase data for nonlinear dynamic analysis is as shown in Fig. 3.13



(a) Gravity load function (b) Rampdown function

Figure 3.12: Time history function definition in SAP2000 for nonlinear dynamic analysis

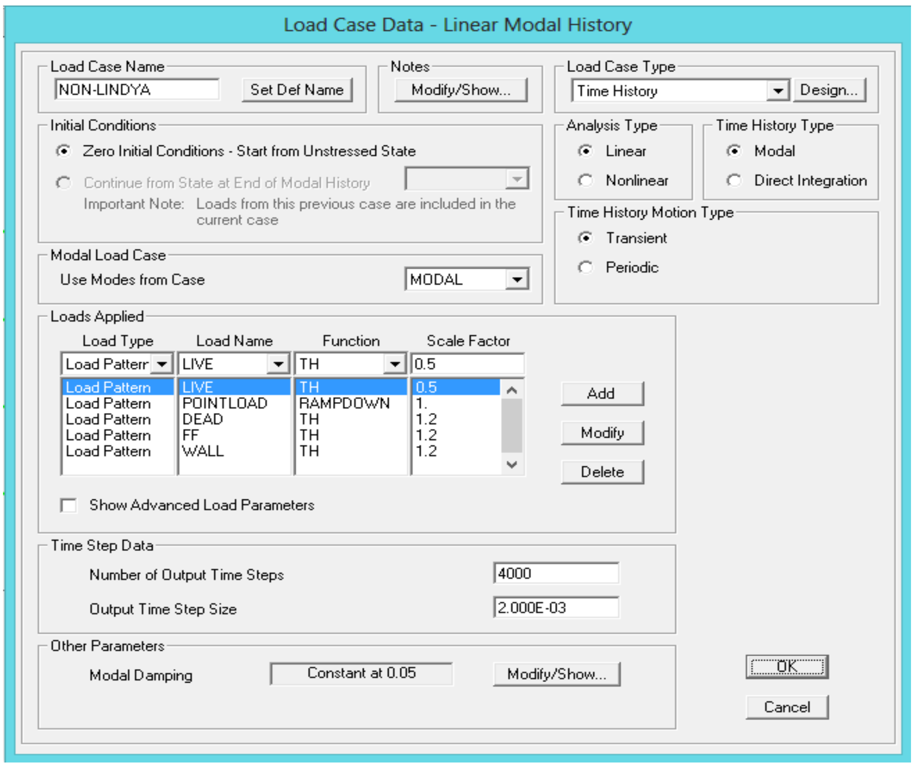


Figure 3.13: Nonlinear dynamic analysis case definition by UFC guideline in SAP2000

3.5 Design of Viscoelastic Dampers

Viscoelastic dampers has been used successfully in several high rise building for the effective reduction in earthquake and wind induced response. Following design procedure illustrates the parameters like number, size and required properties of damper for any structure to achieve target structural response. The design is carried out according to standard available literature [12], which recommends Kelvin Model for analysis. Step wise procedure for design of viscoelastic damper is discussed below :

Steps for Design of Viscoelastic Damper

- **Decide the required damping ratio :**

  In this study, the required structural damping ratio ζ is assumed as 10% for the initial stage.

- **Calculate required stiffness for viscoelastic damper :**

  The required stiffness of viscoelastic damper is calculated from the following expression, which give the damper of particular stiffness, for required damping ratio.

$$K_d = \frac{2\zeta}{\eta_d - 2\zeta} \times K_s$$

  Where K_d is stiffness provided by the damper at each storey level, ζ is the desired damping ratio, η_d is the loss factor, K_s is the storey stiffness of the structure without added dampers.

- **Determine thickness of damper :**

  The thickness of viscoelastic material can be determined based on the maximum allowable damper deformation. This is controlled by maximum allowable storey drift ratio. Damper thickness is also controlled by maximum allowable strain in material (γ). Final thickness can be given as,

$$t_d = \frac{0.005 \times h_s \times \cos\theta}{\gamma_d}$$

  t_d is thickness of one layer of viscoelastic material in a damper and h_s is typical storey height, θ angle of inclination of damper and γ_d is the allowable shear strain in the material which is assumed to remain constant at 100%.

- **Calculation of Area of damper :**

  The area of damper can be determined using formulae,

$$A = \frac{K_d \times t_d}{G'}$$

  Where, K_d is the damper stiffness, t_d is the thickness of one layer of viscoelastic material, G' is the damper storage modulus which is given as

$$G' = 16.0 \times \omega^{0.51} \times \gamma^{-0.23} \times e^{(72.46/T)}$$

  Thus, damper size can be decided by initially assuming width and then finding required length of damper.

- **Calculation of damping co-efficient :**

  The damping co-efficient C_d of viscoelastic damper can be determine from following equation,

$$C_d = \frac{A \times G"}{\omega_n \times t_d}$$

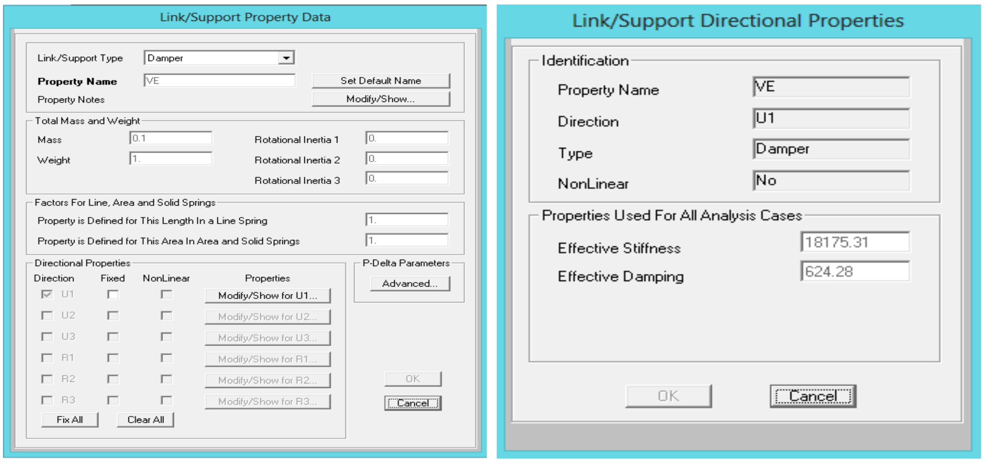
  Where A is area of damper, t_d is thickness of damper, ω_n is natural frequency of frame, G" is shear loss modulas and is given by,

$$G" = 18.5 \times \omega^{0.51} \times \gamma^{-0.20} \times e^{(73.89/T)}$$

3.5.1 Modeling of Viscoelastic Dampers

The modeling procedure of viscoelastic damper using SAP2000 is discussed here. Viscoelastic damper increases stiffness and damping of the structure. For getting stiffness and damping of viscoelastic damper, the damper needs to be designed and properties calculated in section 3.6 are to be feeded to the model prepared in SAP-2000 as shown in Fig. 3.14.

SAP2000 software provides tools to model various energy dissipation devices within a building. Initially design of viscoelastic damper is carried out for specified amount of damping and the calculated properties are used to model viscoelastic damper in SAP2000 through nonlinear link Properties.

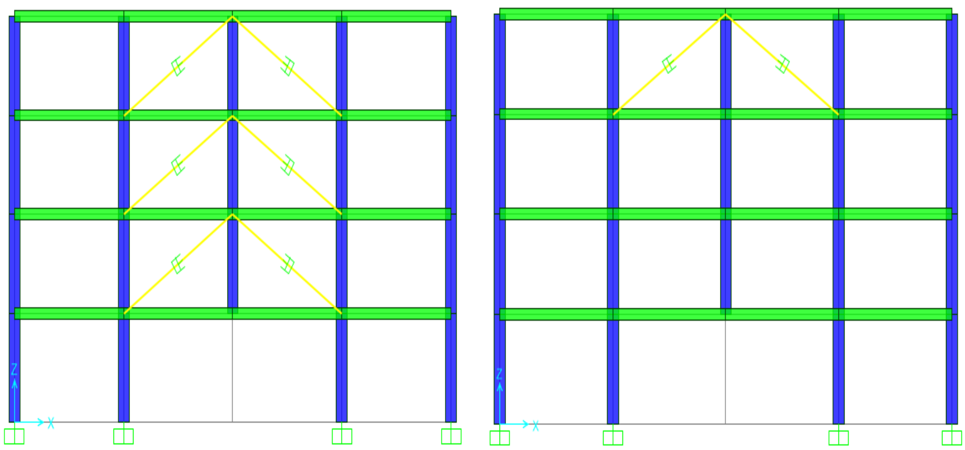


(a) Viscoelastic damper property (b) Stiffness and damping coefficient

Figure 3.14: Definition of viscoelastic damper in SAP2000

3.5.2 Placement of Viscoelastic Dampers

Two types of configuration are suggested for frame. Configuration-1 consists of placing dampers at all storey level with target damping ratio 15%, while configuration-2 consists of placing dampers at top storey only with increased in target damping ratio to 30% as shown in Fig. 3.15. DCR for flexure, shear and bending are calculated considering both GSA and UFC guidelines for both types of configuration and result are presented. DCR for beams as well columns reduces significantly after placing visoelastic dampers which indicates progressive collapse resistance of frame.



(a) Configuration-1 (b) Configuration-2

Figure 3.15: Configuration of viscoelastic dampers

3.6 Calculation of Demand Capacity Ratio (DCR)

The Demand Capacity Ratios (DCR) are calculated at each storey for linear static analysis. DCR is calculated at three points left, center and right side of the column removal position as shown in Fig. 3.16. L, C and R indicates the value of DCR at left, center and right side from the position of removed column respectively for static analysis. According to the guidelines structural elements having Demand Capacity Ratio (DCR) values exceeding 2.0 for flexure and 1.0 for shear are considered as severely damaged or collapsed.

3.6.1 DCR for Flexure

Moment capacity of section above the removed column is calculated with reference to IS 456:2000 [20]. Demand capacity Ratio (DCR) at critical locations L, C and R is found as shown in Fig. 3.16 at all storeys to study the potential for progressive collapse. Moment of resistance of beam at the section above column removed is obtained by

$$M_u = 0.87 f_y A_{st}\Big(1 - (\frac{A_{st} f_y}{b d f_{ck}})\Big)$$

Area of steel in beam located above failed column,

A_{st} = 1785.09 mm^2

f_{ck} = 1.25 × 25 = 31.25 N/mm^2

f_y = 1.25 × 415 = 518.75 N/mm^2

(1.25 is the strength increase factor to take in account behaviour of material at high strain rate.)

Here for b = 300 mm and d = 310 mm,

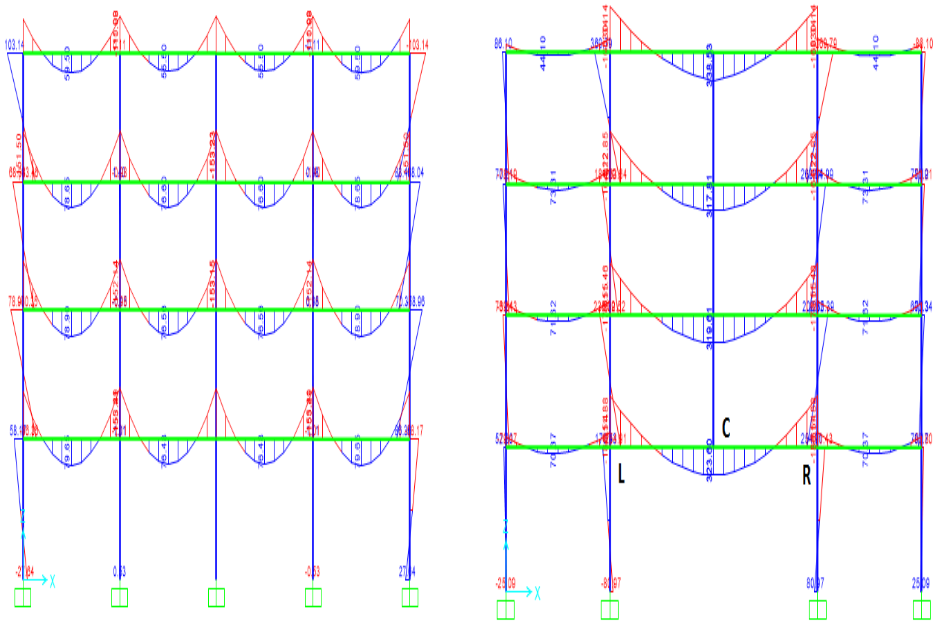
Putting in above formulae

M_u = 178.23 kN m

DCR for flexure $= \frac{611.88}{178.23} = 3.43$

3.6.2 DCR for Shear

Shear capacity of section above the removed column is found out with reference to IS 456:2000 [20]. Fig. 3.17 shows the shear force in the beam members before and after removal of the column for case 1. Procedure for calculation of DCR for shear is illustrated



(a) (b)

Figure 3.16: Bending moment diagram before and after column removal

below for column removal case 1.

Diameter of stirrups = 8 mm

No. of leg = 2

A_{sv} = 100.48 mm^2, Cross sectional area of vertical legs of stirrups

S_v = 100 mm, Spacinf of stirrups

f_y = 415×1.25 = 518.75 N/mm^2

Shear resisted by shear reinforcement = V_{us}

V_{us} = 226.74 kN

Shear resisted by concrete $V_c = \tau_c$bd

τ_c is taken from SP 16 [23] for different fck and Pt values.

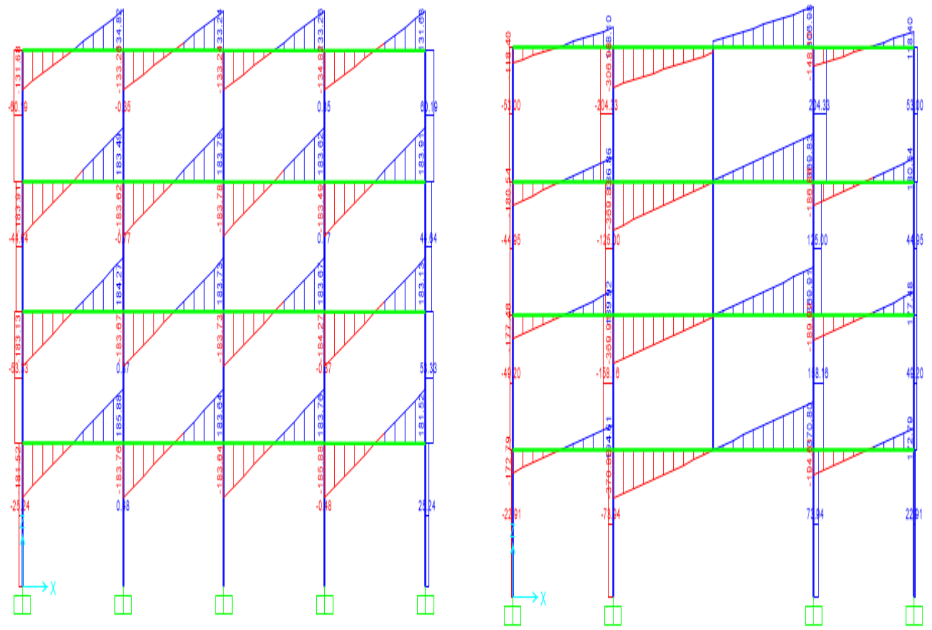
Pt = 0.54

τ_c = 0.50, Hence V_c = 48.27 kN

Total shear resisted by section $V_s = V_{us} + V_c$

V_s = 275.01 kN.

DCR for shear $=\frac{360.26}{275.01} = 1.31$



(a) (b)

Figure 3.17: Shear force diagram before and after column removal

3.6.3 DCR for Column

Due to removal of one column redistribution of forces takes place in the structure, so forces in the column i.e. axial force, moment about major axis and moment about minor axis, changes and can affects the adequacy of the columns. Demand capacity ratios for columns designed for seismic loading using static and dynamic analysis are calculated as per following equation. If it exceeds unity column can be considered as failed.

$$\Big[(\frac{M_{ux}}{M_{ux1}})^{\alpha n}\Big] + \Big[(\frac{M_{uy}}{M_{uy1}})^{\alpha n}\Big] \leq 1$$

Where,

M_{ux}, M_{uy} = Moments about X and Y axis due to design loads

M_{ux1}, M_{uy1} = Maximum uniaxial moment capacity for an axial load, bending moment about x and y axis.

αn= constant which depends on P_u/P_{uz}.

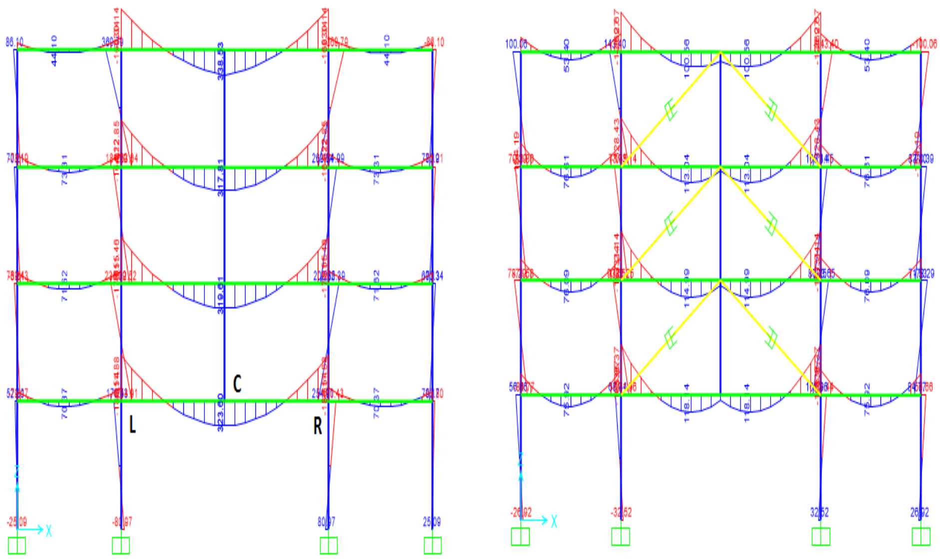
$$\left[\left(\frac{36}{133}\right)^{0.2}\right] + \left[\left(\frac{204}{228}\right)^{0.2}\right] = 1.16$$

DCR for column can be found out with reference to SP-16 [23] by considering the column under axial force and biaxial bending.

3.7 Results and Discussions

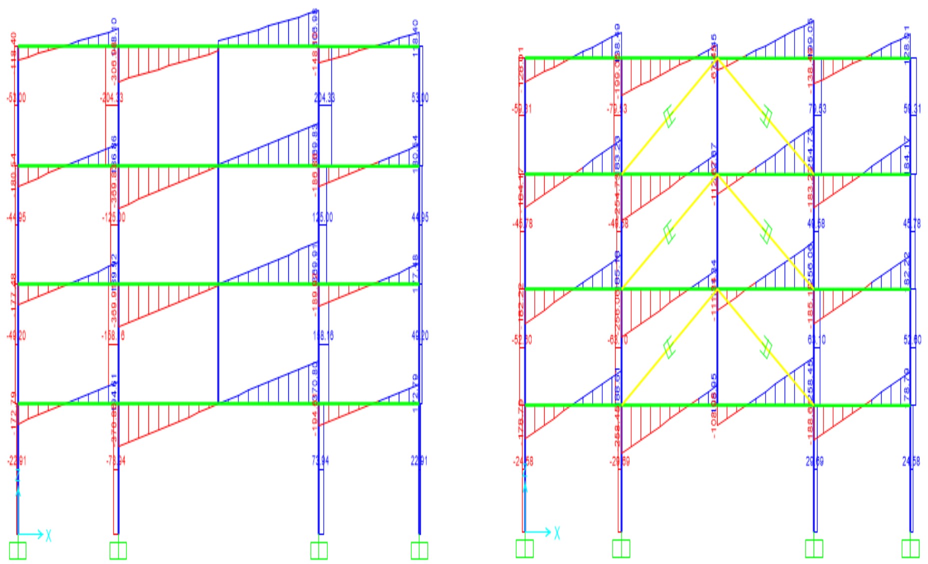
The demand capacity ratios are calculated at each storey for static analysis. DCR is calculated at three points left, center and right side of the column removal position. L, C and R indicates the value of DCR at left, center and right side from the position of removed column respectively. For static analysis, frame with damper and without damper for different configuration is compared.

Here the Fig. 3.18 shows that the bending moment at the center and both sides of column removal location is considerably decreased after placing of Dampers. Also Fig. 3.19 shows that the shear force at the center and both sides of column removal location is considerably decreased for frame with damper.



(a) (b)

Figure 3.18: Bending moment diagram before and after placing of damper

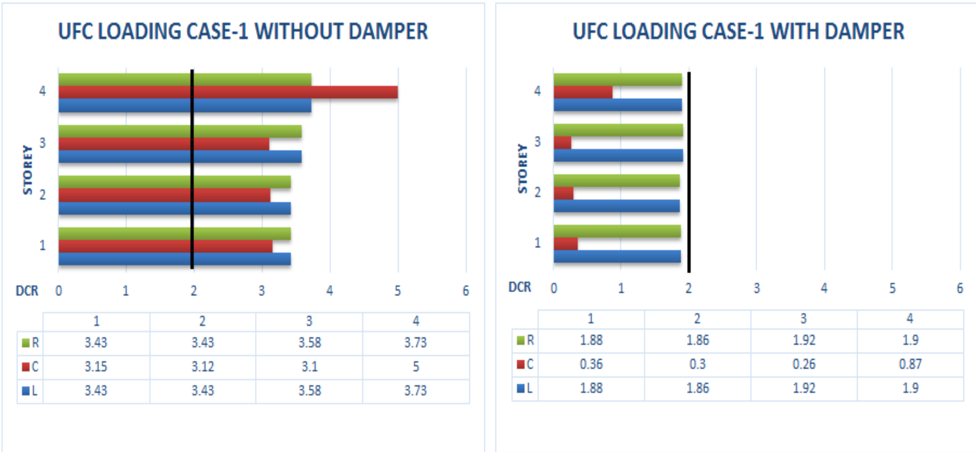


(a) (b)

Figure 3.19: Shear force diagram before and after placing of damper

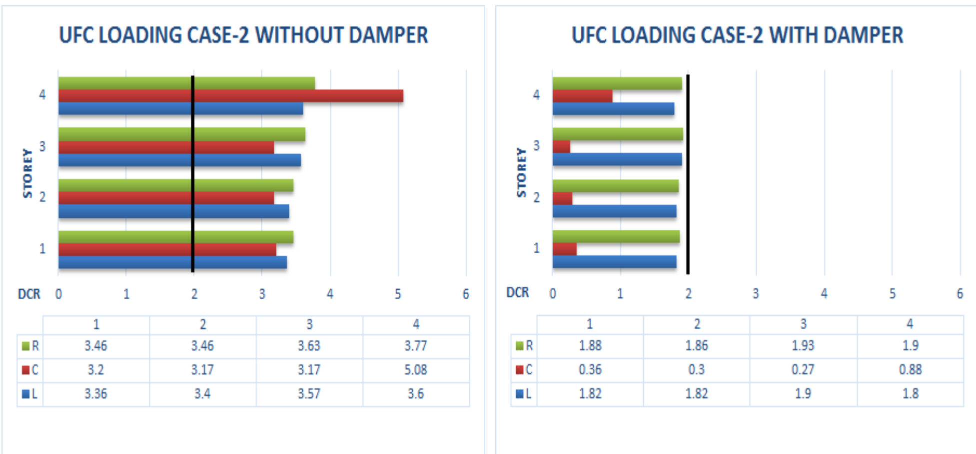
Comparison of Demand Capacity Ratio (DCR) for Flexure

DCR for flexure is calculated at all the four stories for frame with damper and without damper and is compared as shown in Fig. 3.20 to Fig. 3.27 for both type of configurations. From result it is observed that DCR in beams of buildings without dampers are exceeding the allowable limit i.e. 2 for both column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 2, which indicates enhanced progressive collapse resistance of building.



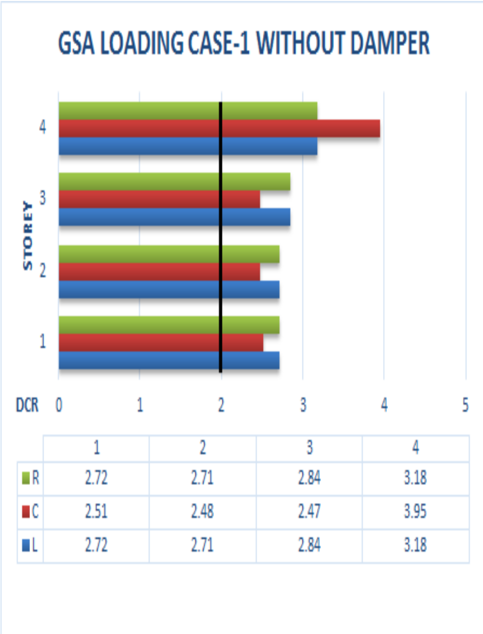
(a) DCR without damper (b) DCR with damper

Figure 3.20: DCR for flexure for case 1 (UFC Loading) configuration-1



(a) DCR without damper (b) DCR with damper

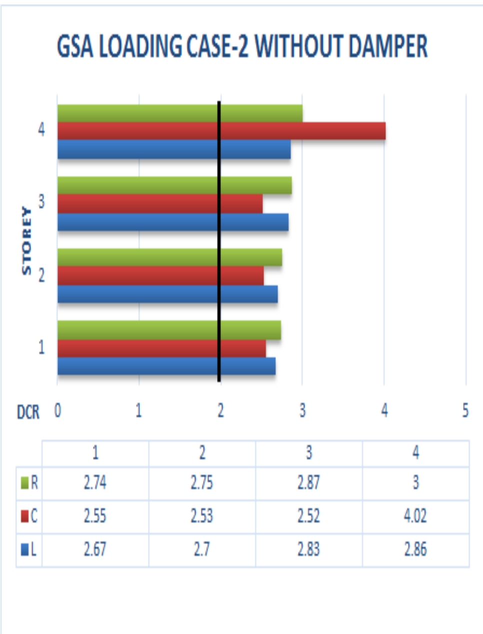
Figure 3.21: DCR for flexure for case 2 (UFC Loading) configuration-1



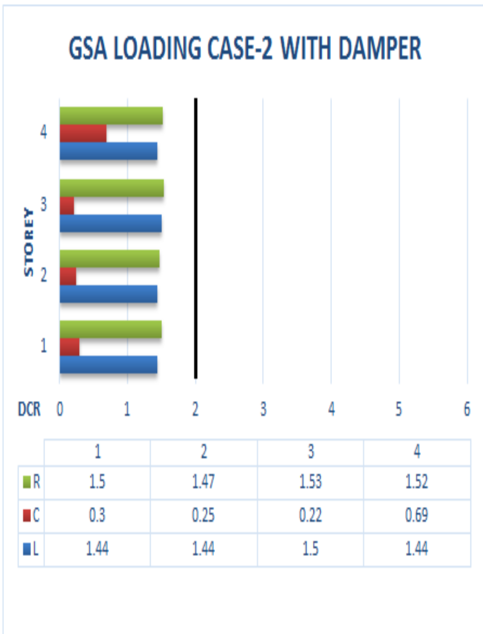
(a) DCR without damper

(b) DCR with damper

Figure 3.22: DCR for flexure for case 1 (GSA Loading) configuration-1



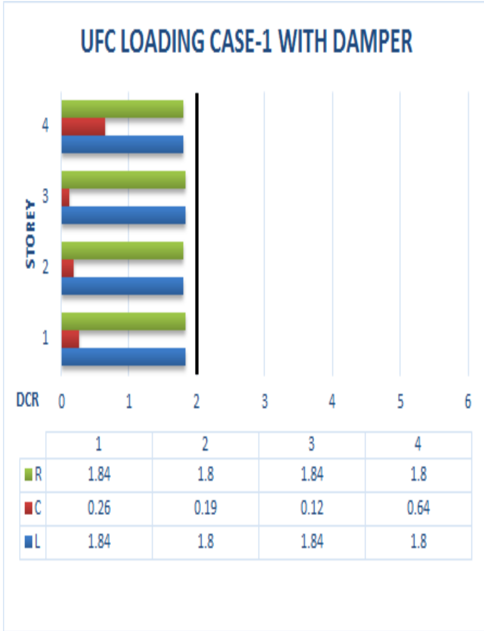
(a) DCR without damper



(b) DCR with damper

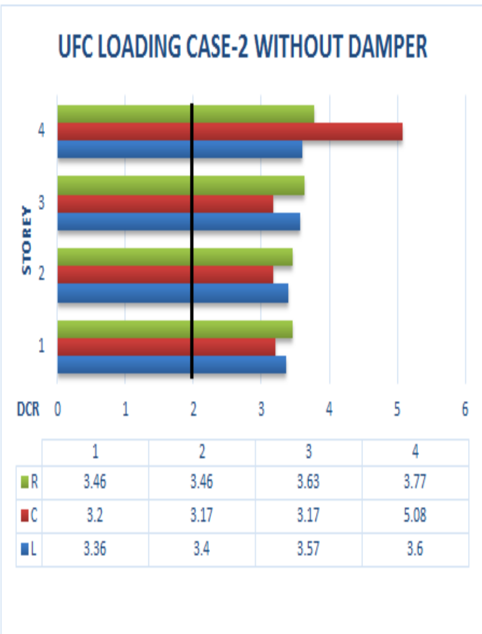
Figure 3.23: DCR for flexure for case 2 (GSA Loading) configuration-1

(a) DCR without damper



(b) DCR with damper

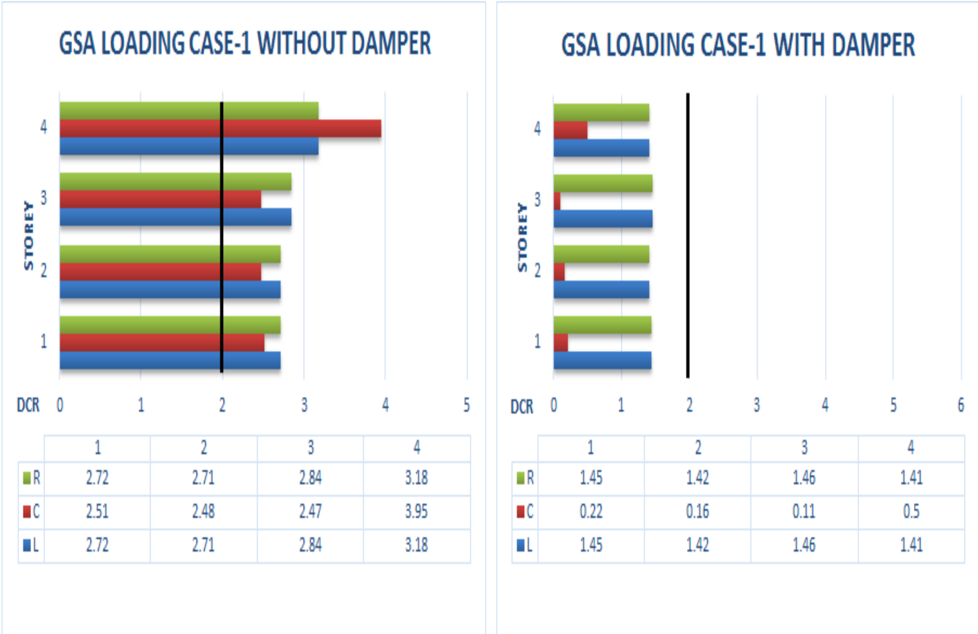
Figure 3.24: DCR for flexure for case 1 (UFC Loading) configuration-2



(a) DCR without damper

(b) DCR with damper

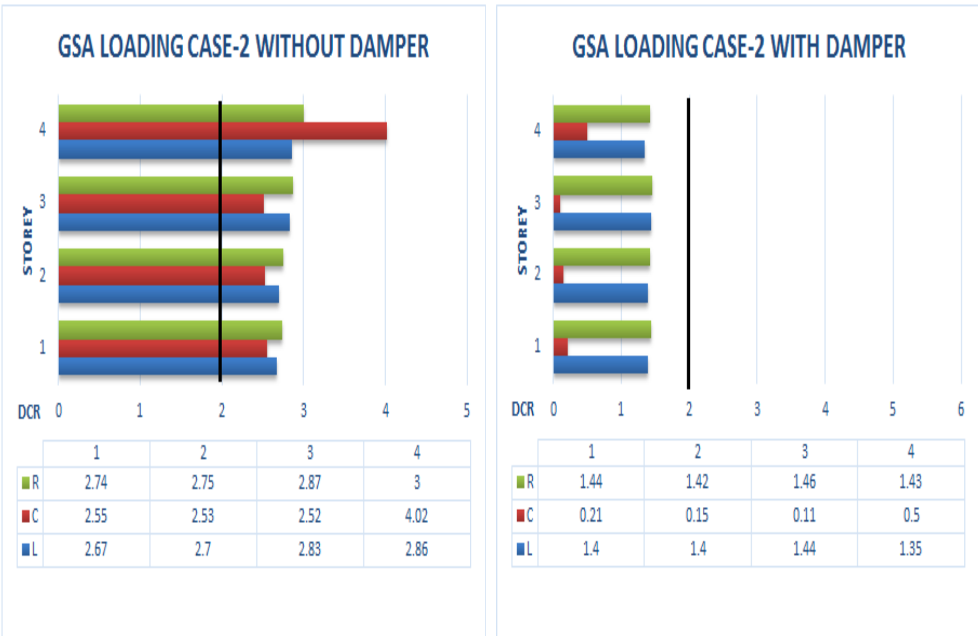
Figure 3.25: DCR for flexure for case 2 (UFC Loading) configuration-2



(a) DCR without damper

(b) DCR with damper

Figure 3.26: DCR for flexure for case 1 (GSA Loading) configuration-2



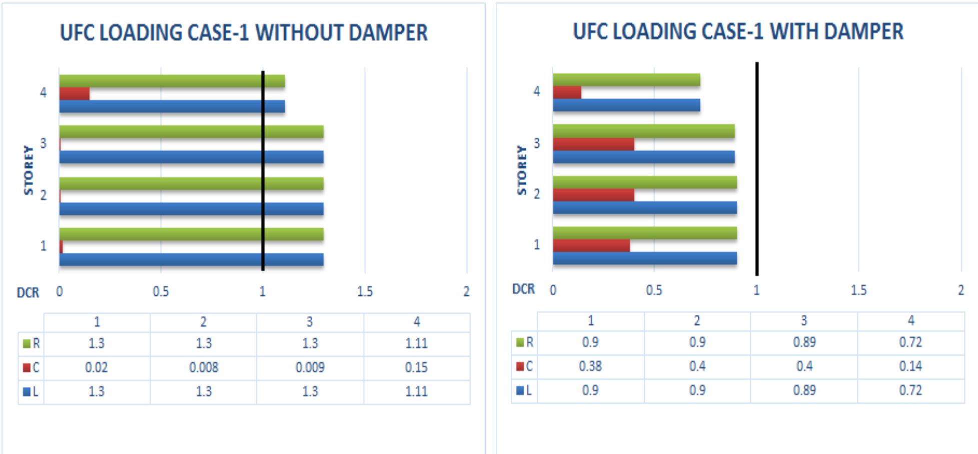
(a) DCR without damper

(b) DCR with damper

Figure 3.27: DCR for flexure for case 2 (GSA Loading) configuration-2

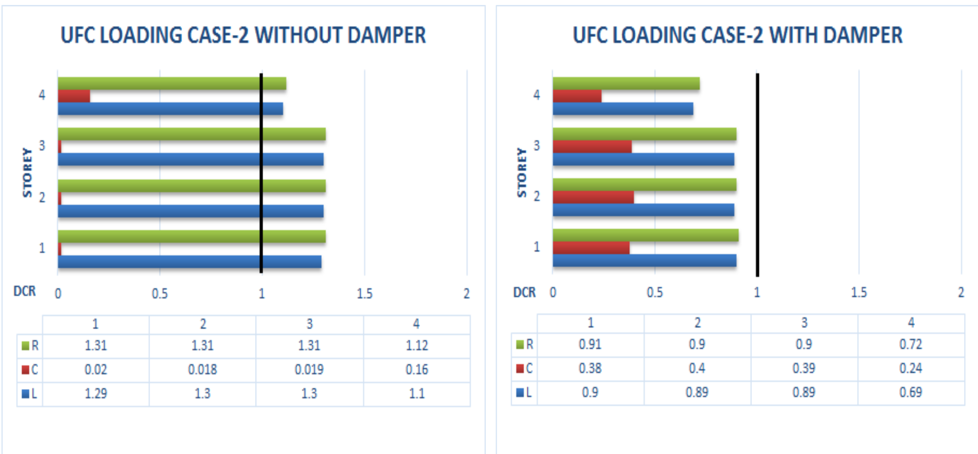
Comparison of Demand Capacity Ratio (DCR) for Shear

DCR for shear is calculated at all the four stories for frame with damper and without damper and is compared as shown in Fig. 3.28 to Fig. 3.35 for both configurations. From result it is observed that DCR in beams of buildings without dampers are exceeding the allowable limit i.e. 1 for both column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 1, which indicates enhanced progressive collapse resistance of building.



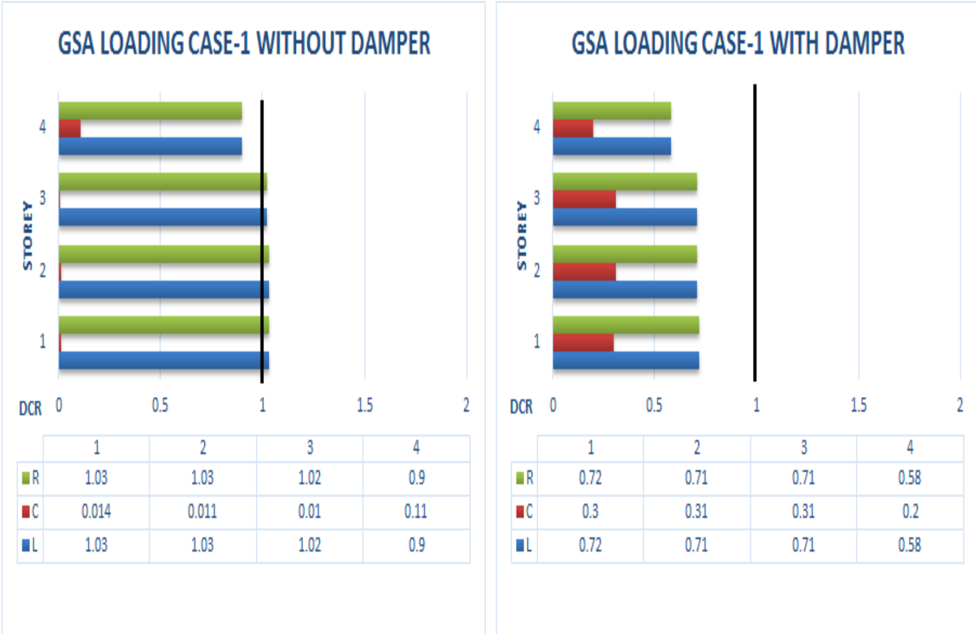
(a) DCR without damper (b) DCR with damper

Figure 3.28: DCR for shear for case 1 (UFC Loading) configuration-1



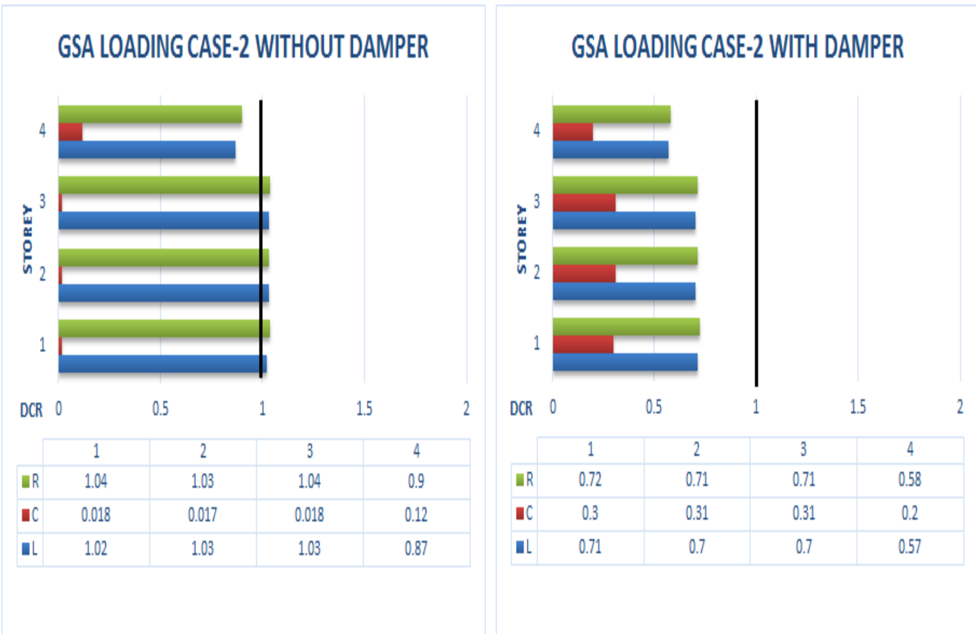
(a) DCR without damper (b) DCR with damper

Figure 3.29: DCR for shear for case 2 (UFC Loading) configuration-1



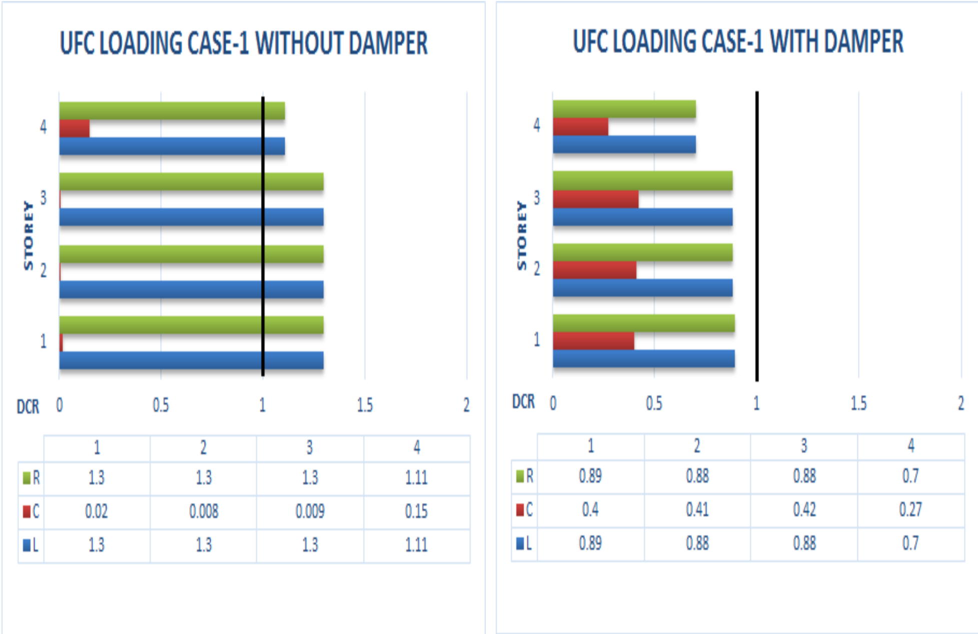
(a) DCR without damper (b) DCR with damper

Figure 3.30: DCR for shear for case 1 (GSA Loading) configuration-1



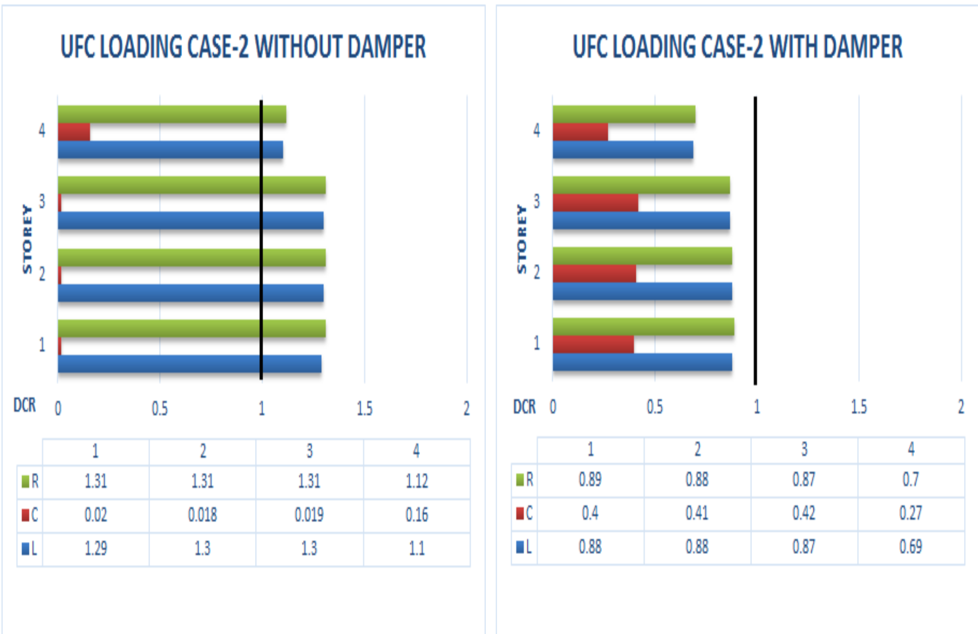
(a) DCR without damper (b) DCR with damper

Figure 3.31: DCR for shear for case 1 (GSA Loading) configuration-1



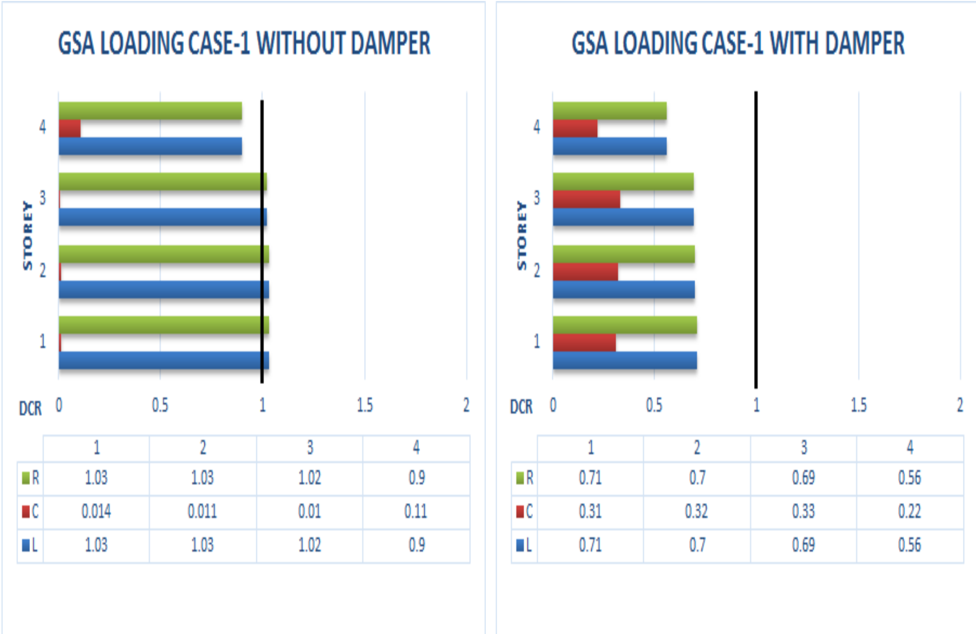
(a) DCR without damper (b) DCR with damper

Figure 3.32: DCR for shear for case 1 (UFC Loading) configuration-2



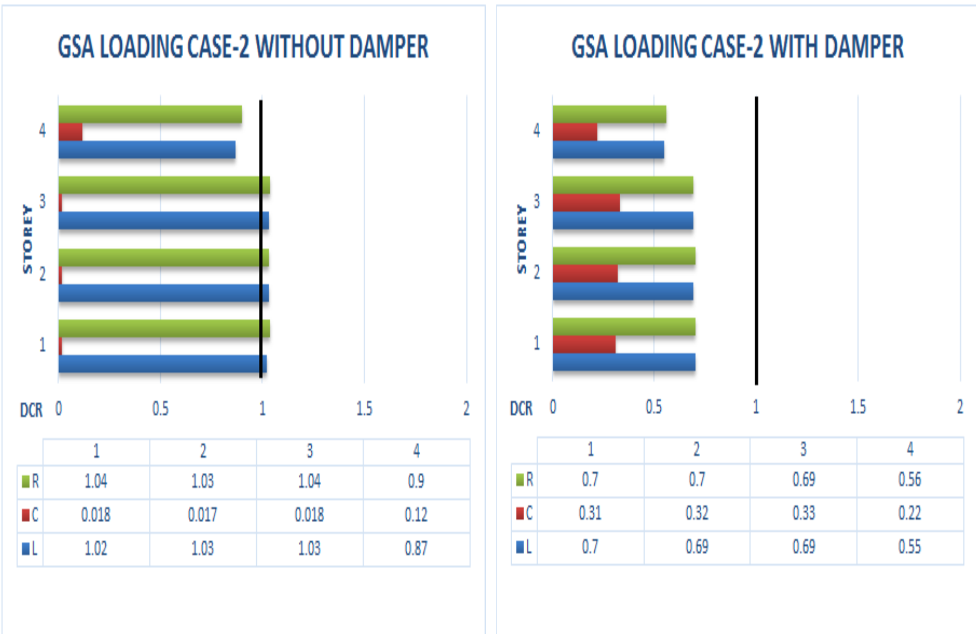
(a) DCR without damper (b) DCR with damper

Figure 3.33: DCR for shear for case 2 (UFC Loading) configuration-2



(a) DCR without damper (b) DCR with damper

Figure 3.34: DCR for shear for case 1 (GSA Loading) configuration-2

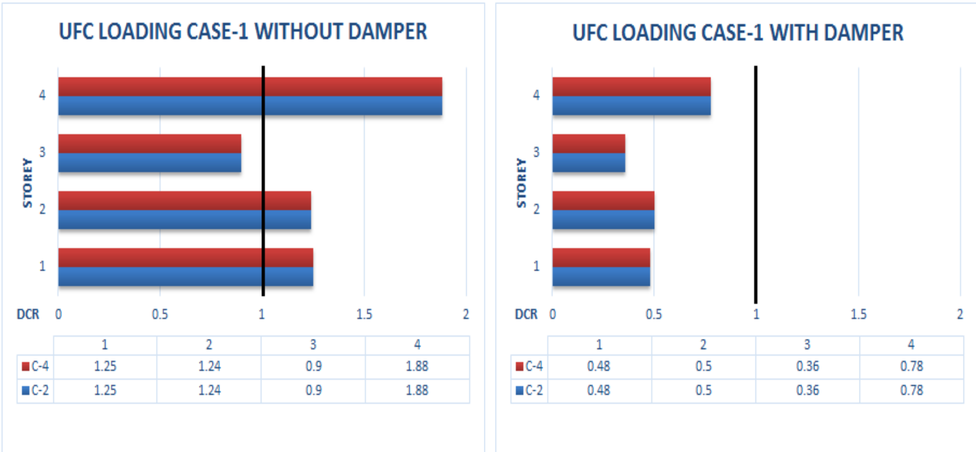


(a) DCR without damper (b) DCR with damper

Figure 3.35: DCR for shear for case 1 (GSA Loading) configuration-2

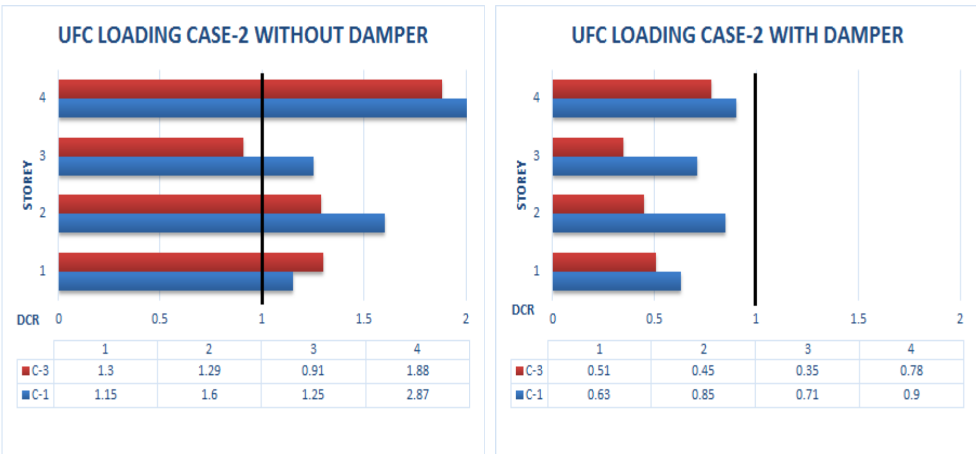
Comparison of Demand Capacity Ratio (DCR) for Column

DCR for column is calculated at all the four stories for frame with damper and without damper and is compared as shown in Fig. 3.36 to Fig. 3.43 for both configurations. From result it is observed that DCR in column of buildings without dampers are exceeding the allowable limit i.e. 1 for both column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 1, which indicates enhanced progressive collapse resistance of building.



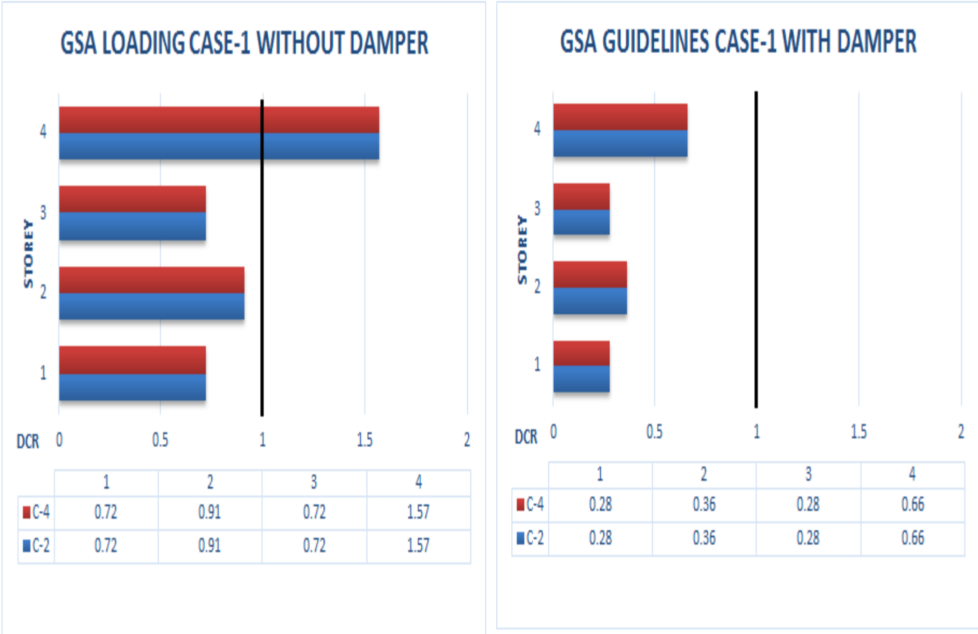
(a) DCR without damper (b) DCR with damper

Figure 3.36: DCR for column for case 1 (UFC Loading) configuration-1



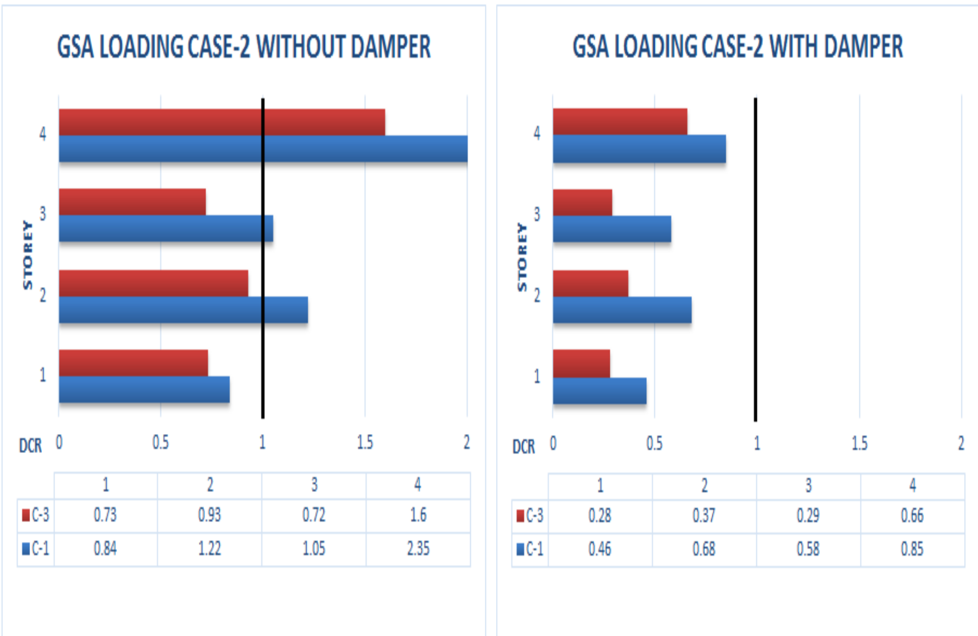
(a) DCR without damper (b) DCR with damper

Figure 3.37: DCR for column for case 2 (UFC Loading) configuration-1



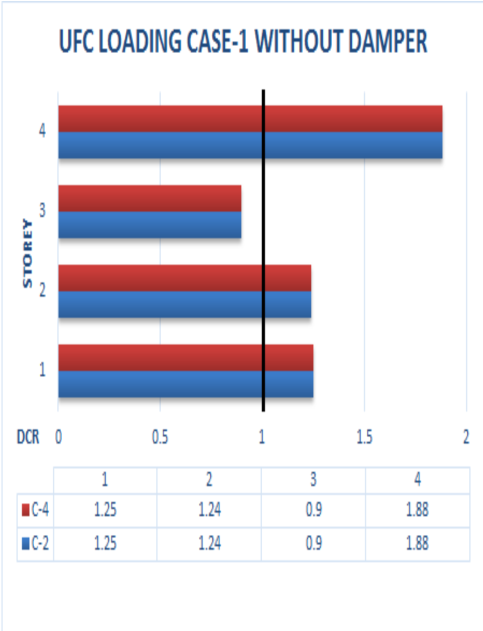
(a) DCR without damper (b) DCR with damper

Figure 3.38: DCR for column for case 1 (GSA Loading) configuration-1

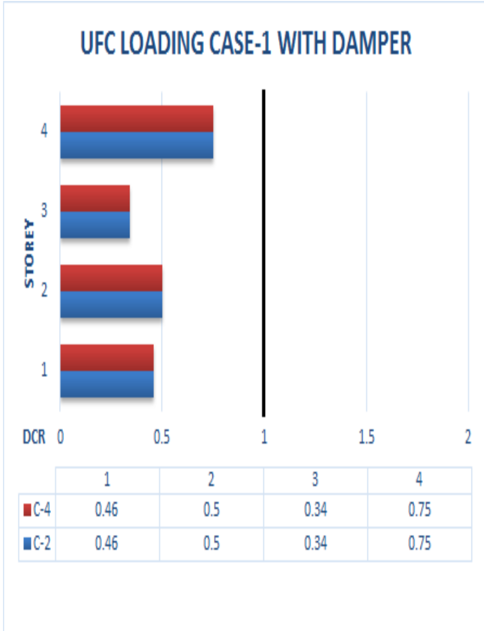


(a) DCR without damper (b) DCR with damper

Figure 3.39: DCR for column for case 2 (GSA Loading) configuration-1

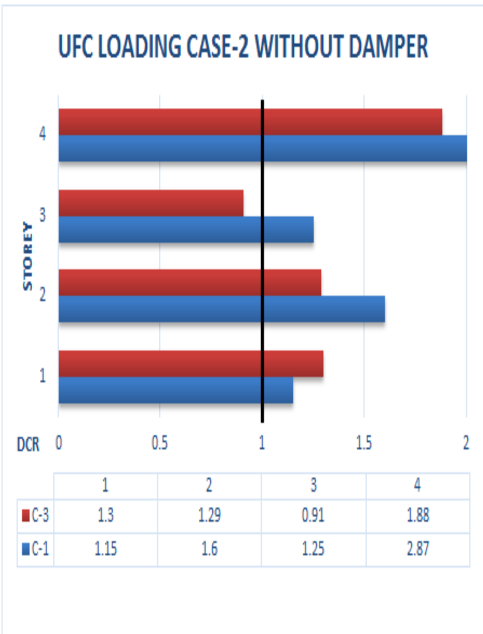


(a) DCR without damper

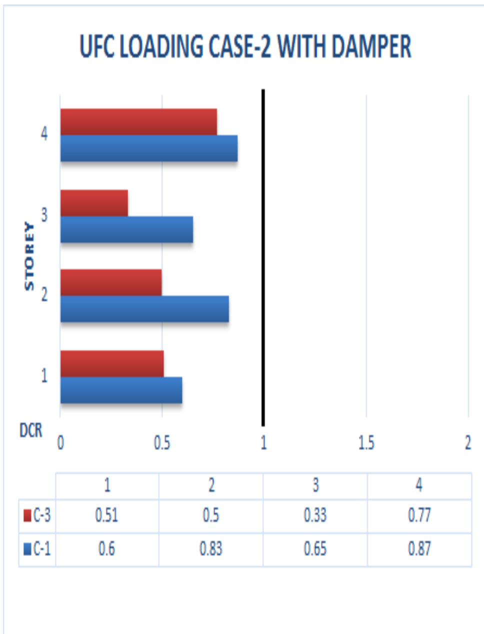


(b) DCR with damper

Figure 3.40: DCR for column for case 1 (UFC Loading) configuration-2

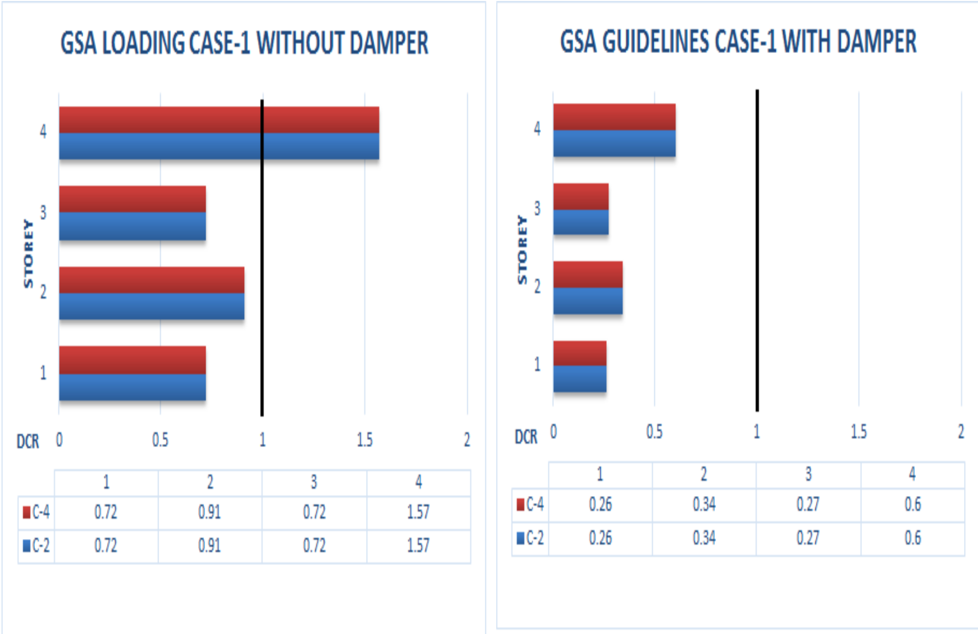


(a) DCR without damper



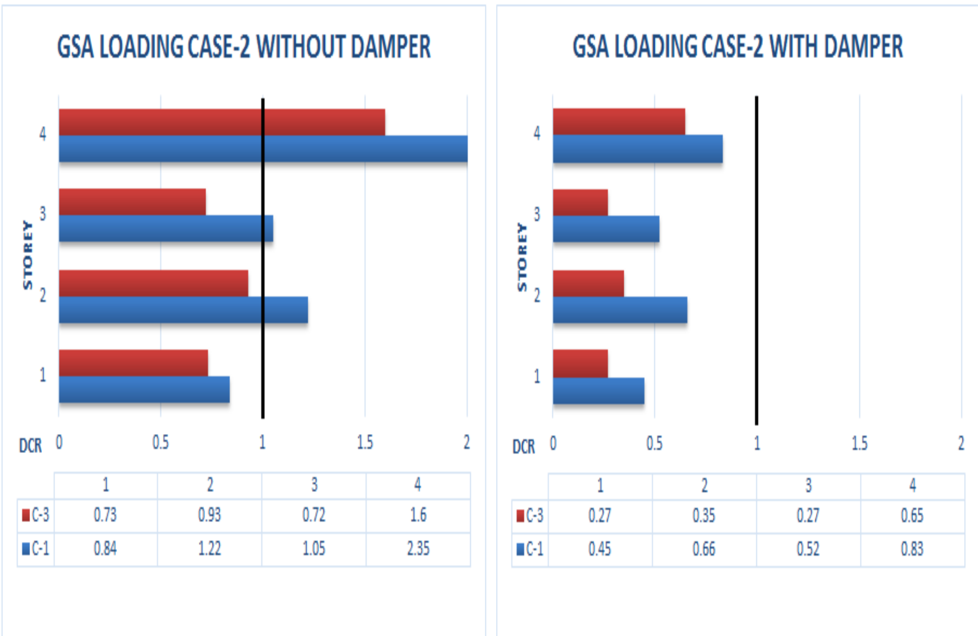
(b) DCR with damper

Figure 3.41: DCR for column for case 2 (UFC Loading) configuration-2



(a) DCR without damper (b) DCR with damper

Figure 3.42: DCR for column for case 1 (GSA Loading) configuration-2

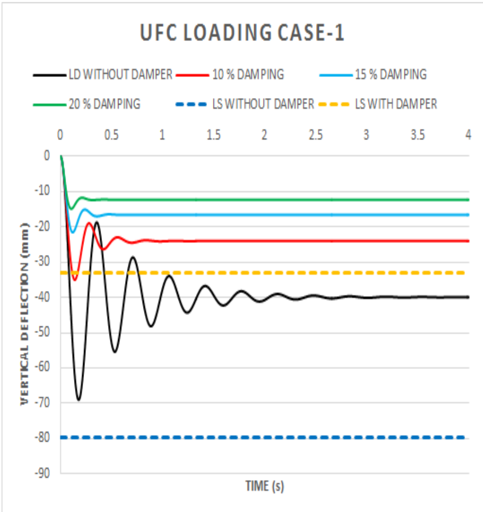


(a) DCR without damper (b) DCR with damper

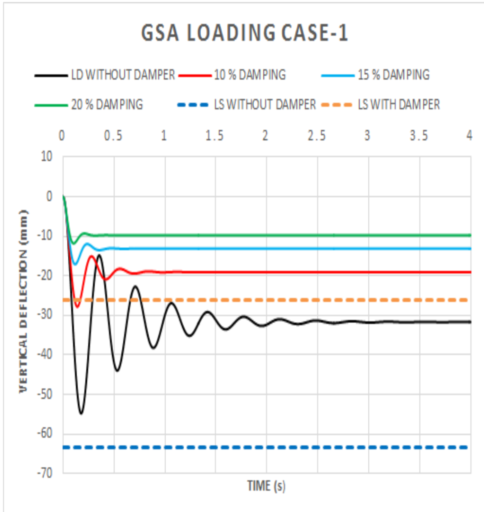
Figure 3.43: DCR for column for case 2 (GSA Loading) configuration-2

Comparison of Linear Dynamic Analysis Results

Linear dynamic analysis is performed to obtain displacement at location of removed columns. Displacement at the location of column removal is maximum in case of frame without dampers and it is observed that after incorporating viscoelastic damper with suitable damping there is considerable reduction in the displacement at the location of removed column as shown in Fig. 3.44 to Fig. 3.47 for both the configuration.

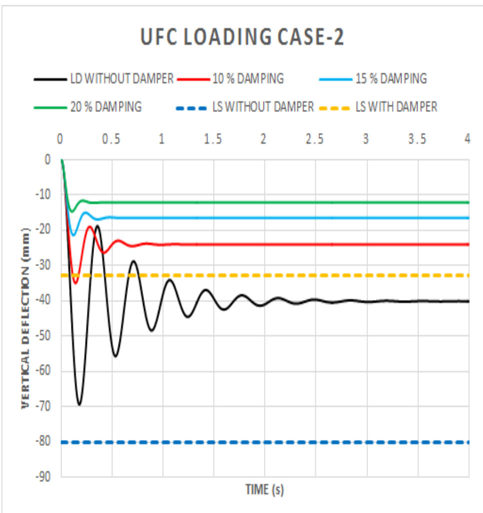


(a) Deflection for UFC loading

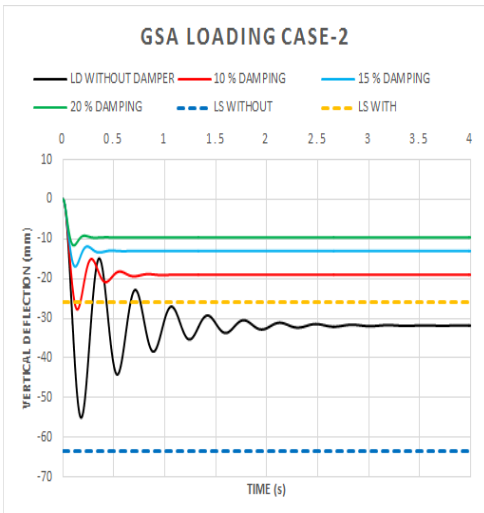


(b) Deflection for GSA loading

Figure 3.44: Deflection under column removal point for Case 1 configuration-1



(a) Deflection for UFC loading



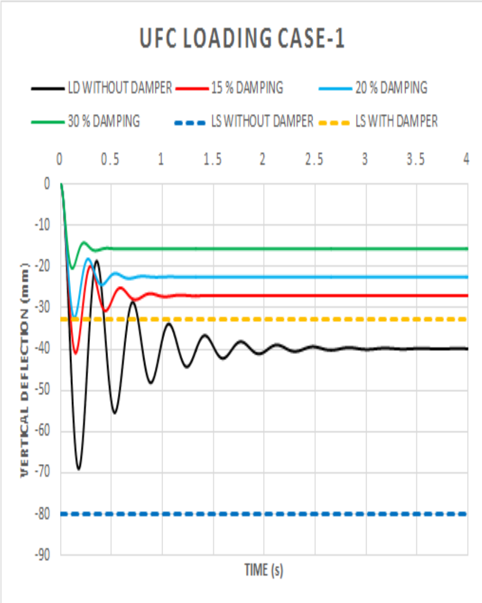
(b) Deflection for GSA loading

Figure 3.45: Deflection under column removal point for Case 2 configuration-1

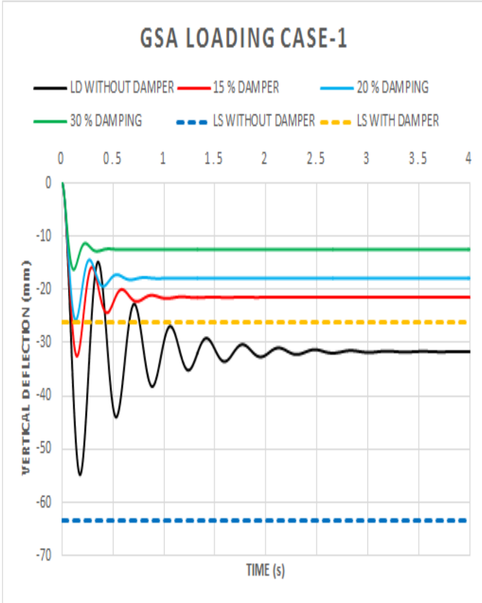
Here the Table 3.3 represents the vertival deflection at column removal location for both GSA and UFC guidelines for configuration-1. Vertical deflection is maximum for linear static analysis as compared to linear dynamic analysis. Incorporating viscoelastic damper with suitable damping, considerably reduces the displacement at the location of removed column.

Table 3.3: Comparison of Vertical deflection of frame configuration-1

	Maximum Vertical Deflection (mm)					
Loading & Column Removal Case	Linear Static Without Damper	Linear Dynamic Without Damper	Linear Static With Damper	10 % Damping	15 % Damping	20 % Damping
GSA Guidelines Case-1	63.30	54.77	26.13	27.77	17.06	11.77
GSA Guidelines Case-2	63.36	55.01	26.00	27.74	16.95	11.61
UFC Guidelines Case-1	79.80	69.01	32.90	34.98	21.50	14.83
UFC Guidelines Case-2	80.15	69.32	32.76	34.94	21.35	14.63

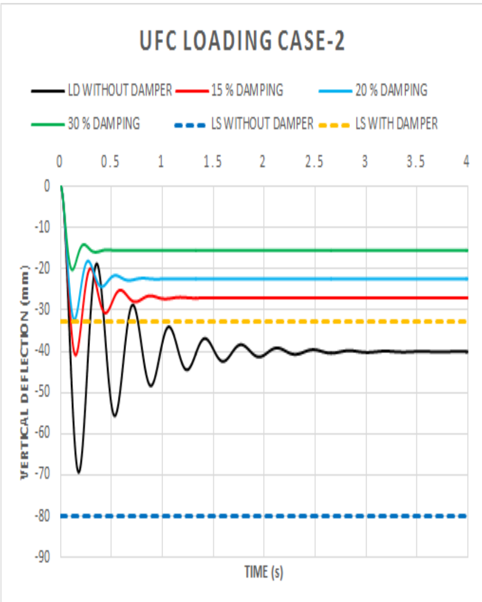


(a) Deflection for UFC loading

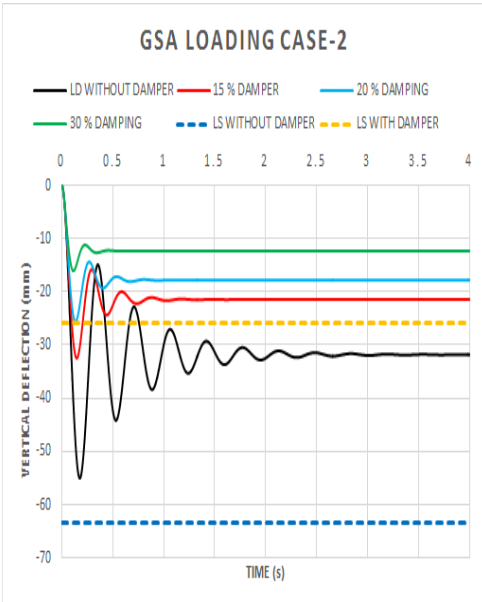


(b) Deflection for GSA loading

Figure 3.46: Deflection under column removal point for Case 1 configuration-2



(a) Deflection for UFC loading



(b) Deflection for GSA loading

Figure 3.47: Deflection under column removal point for Case 1 configuration-2

Here the Table 3.4 represents the vertival deflection at column removal location for both GSA and UFC guidelines for configuration-2. Vertical deflection is maximum for linear static analysis as compared to linear dynamic analysis. Incorporating viscoelastic damper with suitable damping, considerably reduces the displacement at the location of removed column.

Table 3.4: Comparison of Vertical deflection of frame configuration-2

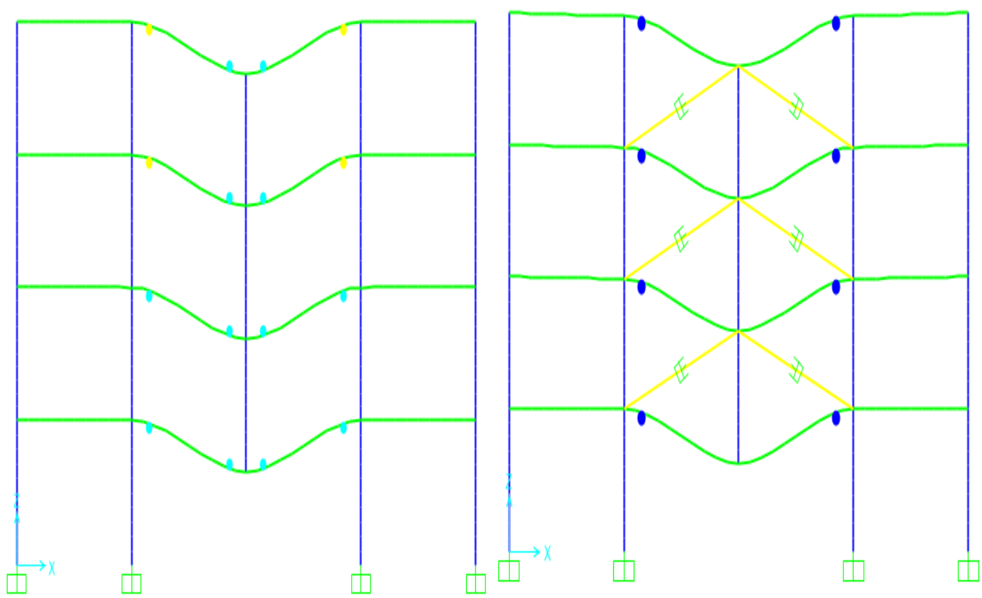
	Maximum Vertical Deflection (mm)					
Loading & Column Removal Case	Linear Static Without Damper	Linear Dynamic Without Damper	Linear Static With Damper	15 % Damping	20 % Damping	30 % Damping
GSA Guidelines Case-1	63.30	54.77	26.13	32.50	25.72	16.24
GSA Guidelines Case-2	63.36	55.01	26.00	32.47	25.62	16.06
UFC Guidelines Case-1	79.80	69.01	32.90	40.95	32.40	20.46
UFC Guidelines Case-2	80.15	69.32	32.76	40.92	32.28	20.23

Comparison of Nonlinear Static Analysis Result

In progressive collapse analysis, vertical pushover is applied, using normal service loading, until the maximum load or the maximum displacement is attained. In this method load is applied step by step until maximum load is attained or until the structure collapses. The failure of one load bearing element may likely to cause failure of other elements resulting in progressive collapse of entire structure. Therefore, nonlinear static analysis is useful for evaluation and to observe the hinge formation pattern in the building during column removal scenario.

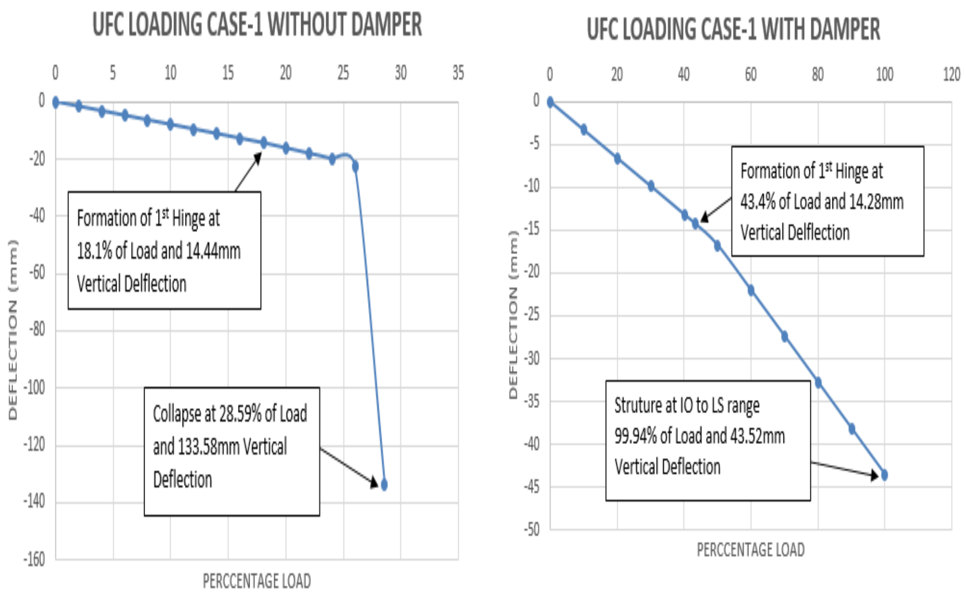
Results are observed in terms of vertical deflection under column removal scenario, formation of hinge pattern and load carrying capacity. Vertical deflection is measured at each step at an interval of 2.5% increase in load. A graph of vertical deflection at location of removed column corresponding to % load resisted by frame is plotted as shown in Fig. 3.49. From the results, it is observed that, viscoelastic dampers significantly increases load carrying capacity of the frame as well as drastically reduces the vertical deflection.

Pattern of hinge formation at the collapse load for frames with and without dampers are presented. From the hinge formation pattern, it is observed that less number of hinges are formed when viscoelastic dampers are provided, as compared to frame without dampers as shown in Fig. 3.48. Load resistance corresponding to formation of 1^{st} hinge in the frames with viscoelastic dampers in 35% to 45% more as compared to frames without dampers. Similarly, load resistance capacity corresponding to collapse load increases by 50% to 70% for frames with viscoelastic dampers as compared to frames without dampers.



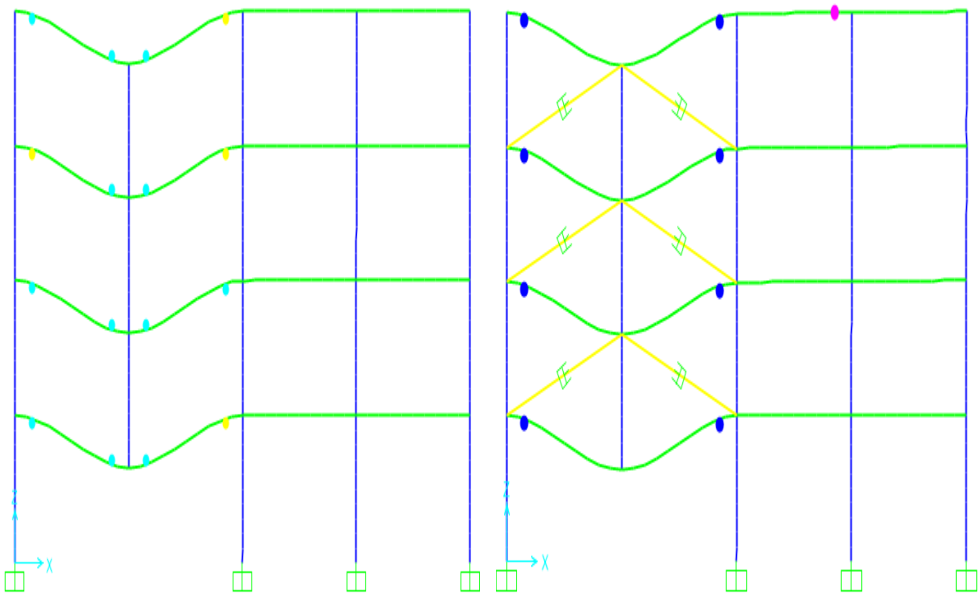
(a) without dampers (b) with dampers

Figure 3.48: Hinges at collapse for UFC loading case-1 configuration-1



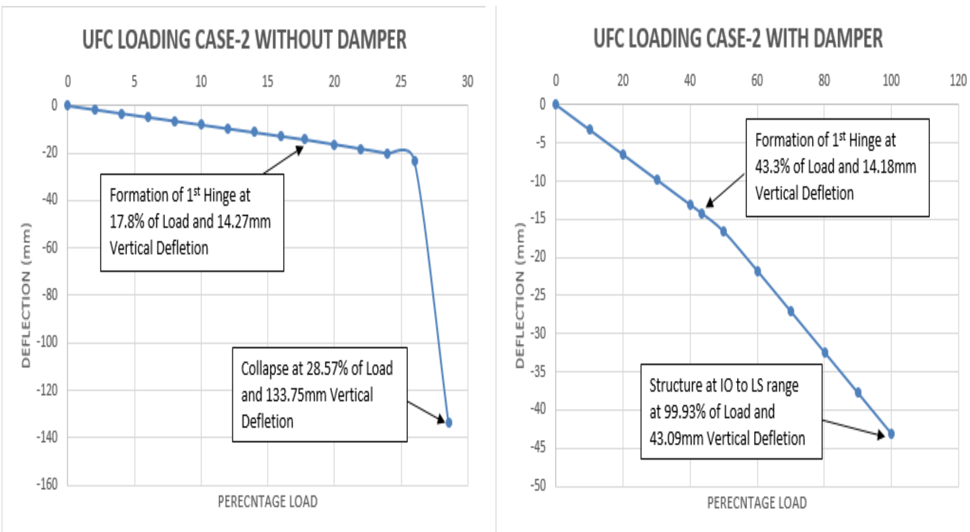
(a) without dampers (b) with dampers

Figure 3.49: Percentage Load v/s Vertical Deflection for UFC loading case-1 config-1



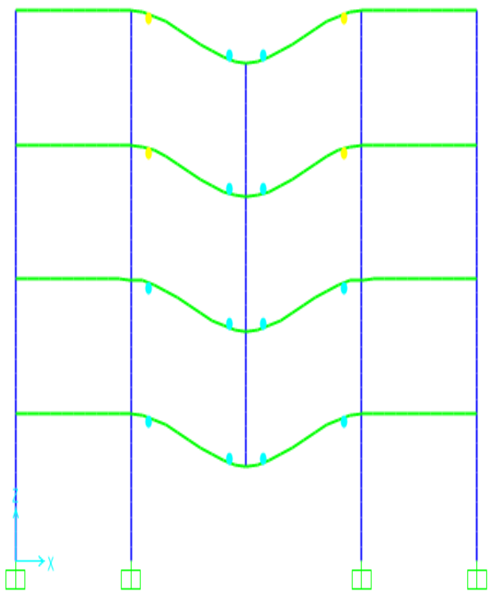
(a) without dampers (b) with dampers

Figure 3.50: Hinges at collapse for UFC loading case-2 configuration-1

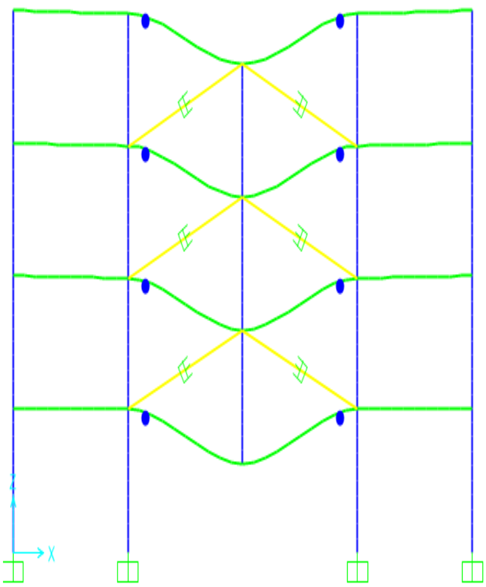


(a) without dampers (b) with dampers

Figure 3.51: Percentage Load v/s Vertical Deflection for UFC loading case-2 config-1

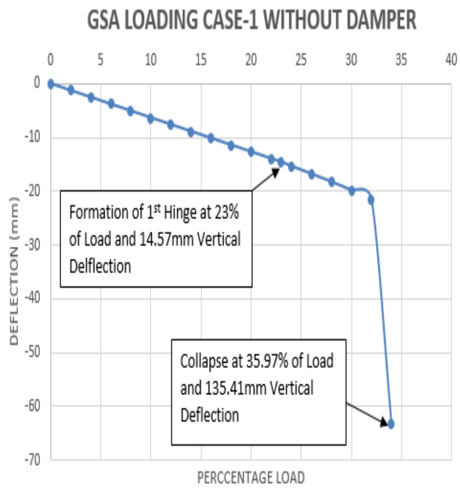


(a) without dampers

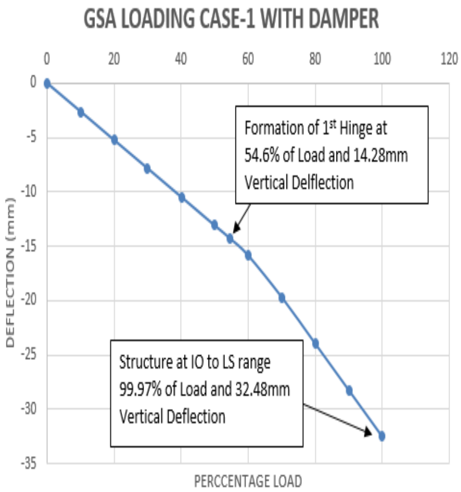


(b) with dampers

Figure 3.52: Hinges at collapse for GSA loading case-2 configuration-1

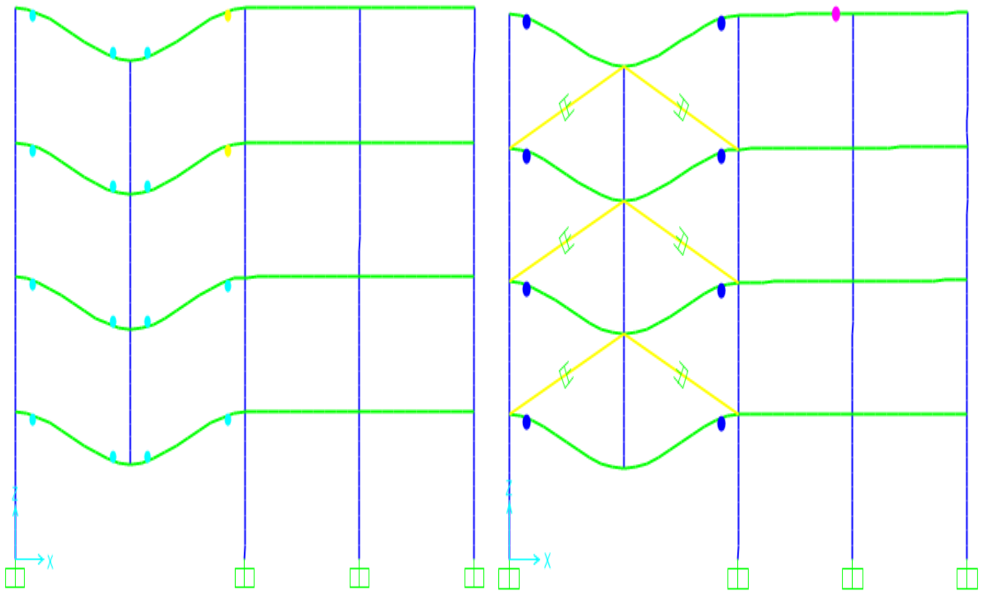


(a) without dampers



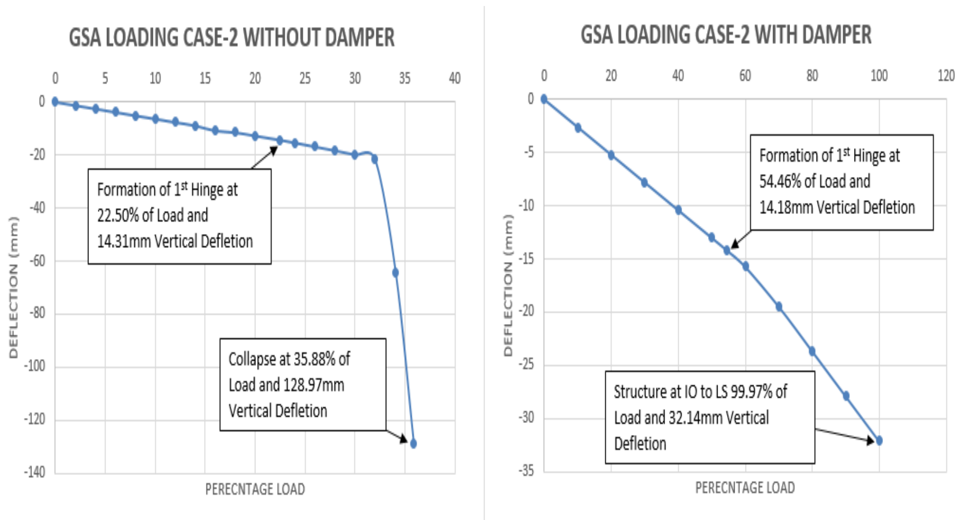
(b) with dampers

Figure 3.53: Percentage Load v/s Vertical Deflection for GSA loading case-1 config-1



(a) without dampers (b) with dampers

Figure 3.54: Hinges at collapse for GSA loading case-2 configuration-1



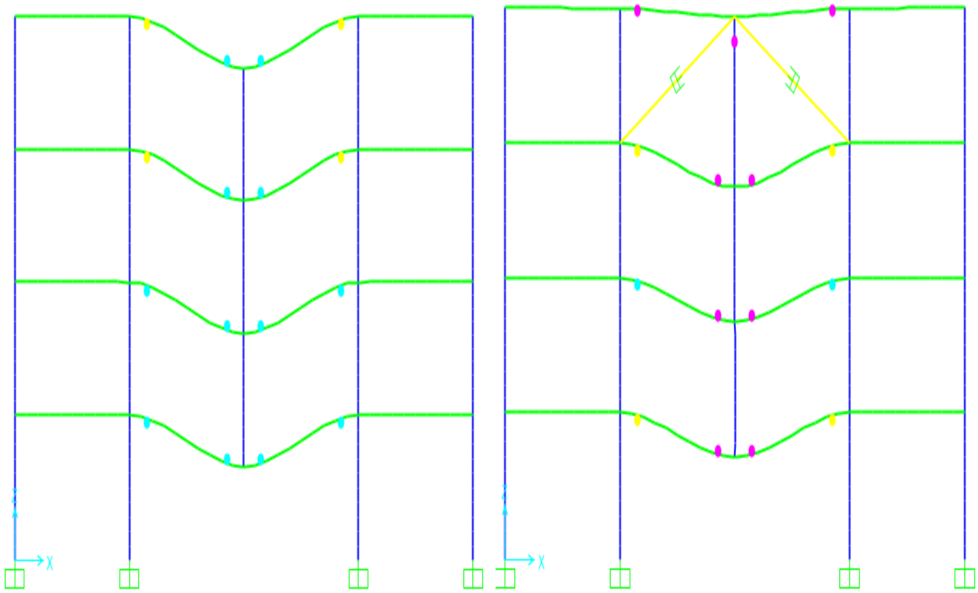
(a) without dampers (b) with dampers

Figure 3.55: Percentage Load v/s Vertical Deflection for GSA loading case-2 config-1

Table 3.5 shows comparison of load carrying capacity of frame with and without dampers for both the guidelines under two different column removal cases for configuration-1. Here case - 1 and case - 2 represents removal of middle column and removal of internal column, respectively.

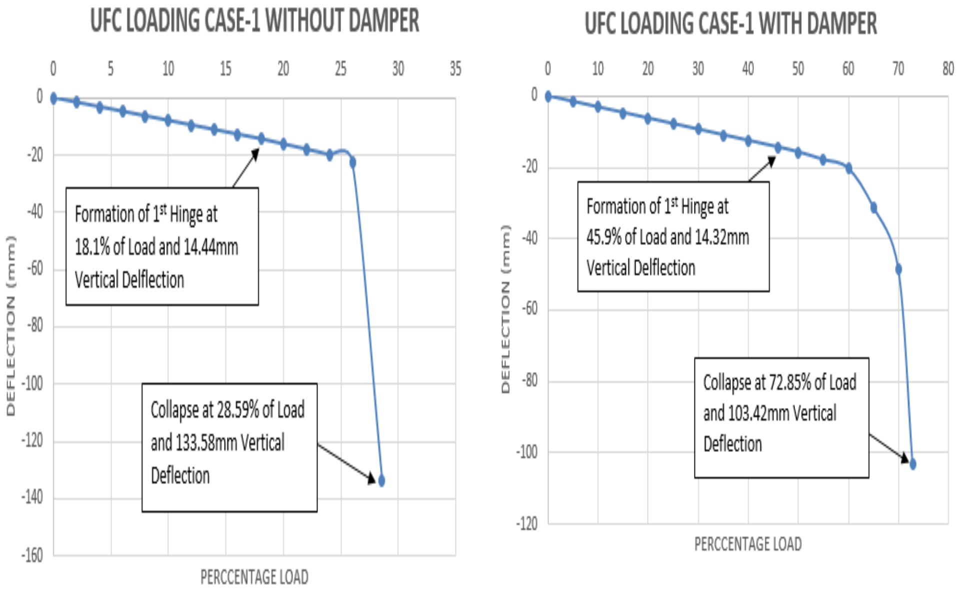
Table 3.5: Comparison of Load carrying capacity of frame for configuration-1

Loading & Column Removal Case	**Percentage Load corresponding to formation of 1st Hinge**			**Percentage Collapse Load**		
	Without Damper	**With Damper**	**% Difference**	**Without Damper**	**With Damper**	**% Difference**
GSA guidelines Case-1	23.00 %	54.60 %	**+31.60 %**	35.97 %	99.97 %	**64.00 %**
GSA guidelines Case-2	22.50 %	54.46 %	**+31.96 %**	35.88 %	99.97 %	**64.09 %**
UFC guidelines Case-1	18.10 %	43.40 %	**+25.30 %**	28.59 %	99.94 %	**71.35 %**
UFC guidelines Case-2	17.80 %	47.17 %	**+29.37 %**	28.57 %	99.93 %	**71.36 %**



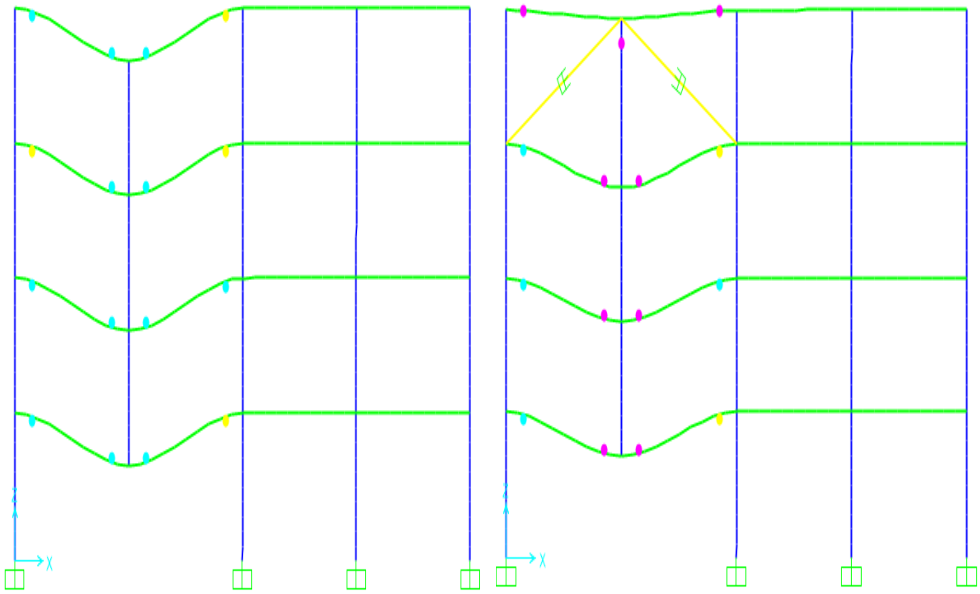
(a) without dampers (b) with dampers

Figure 3.56: Hinges at collapse for UFC loading case-1 configuration-2



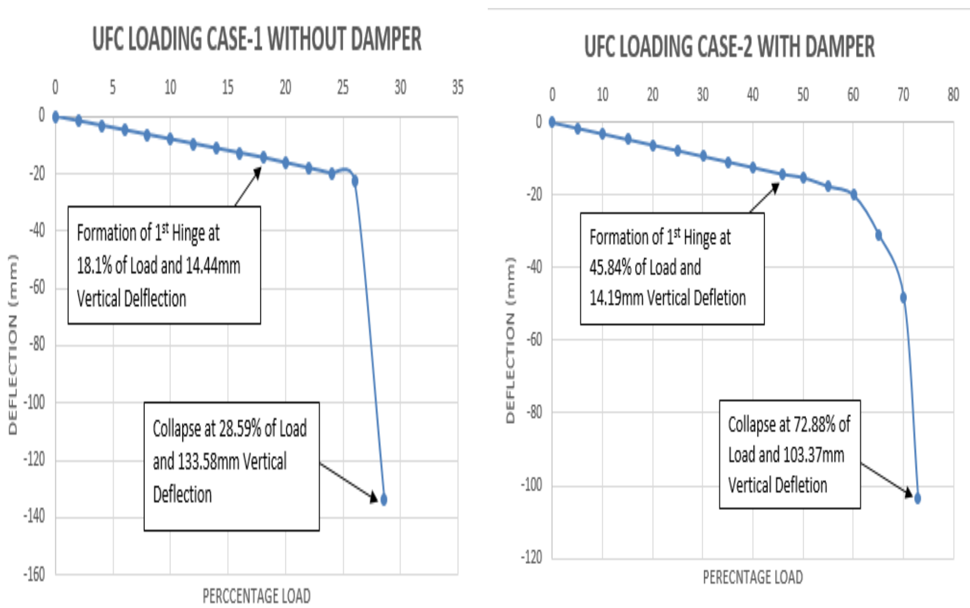
(a) without dampers (b) with dampers

Figure 3.57: Percentage Load v/s Vertical Deflection for UFC loading case-1 config-2



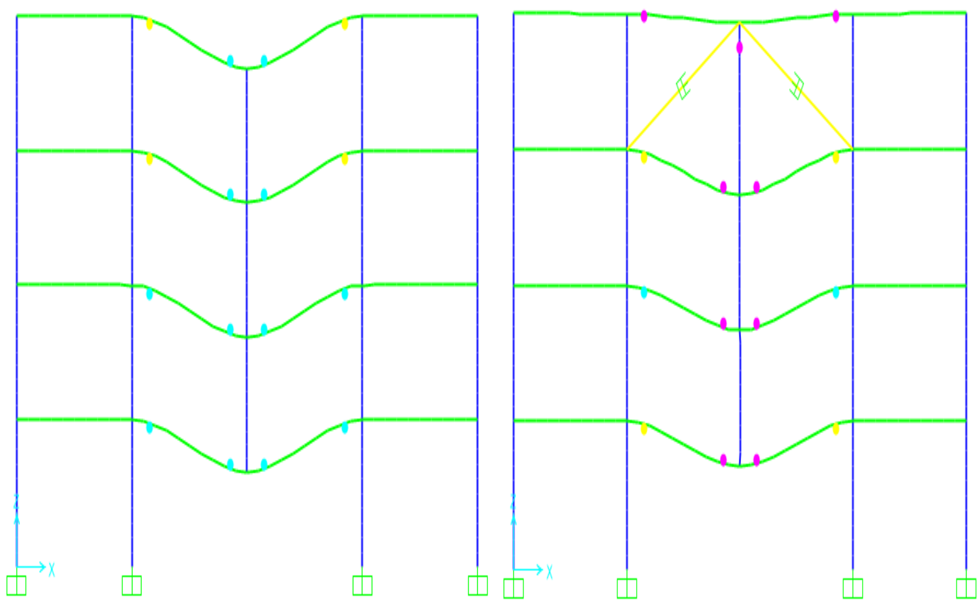
(a) without dampers (b) with dampers

Figure 3.58: Hinges at collapse for UFC loading case-2 configuration-2



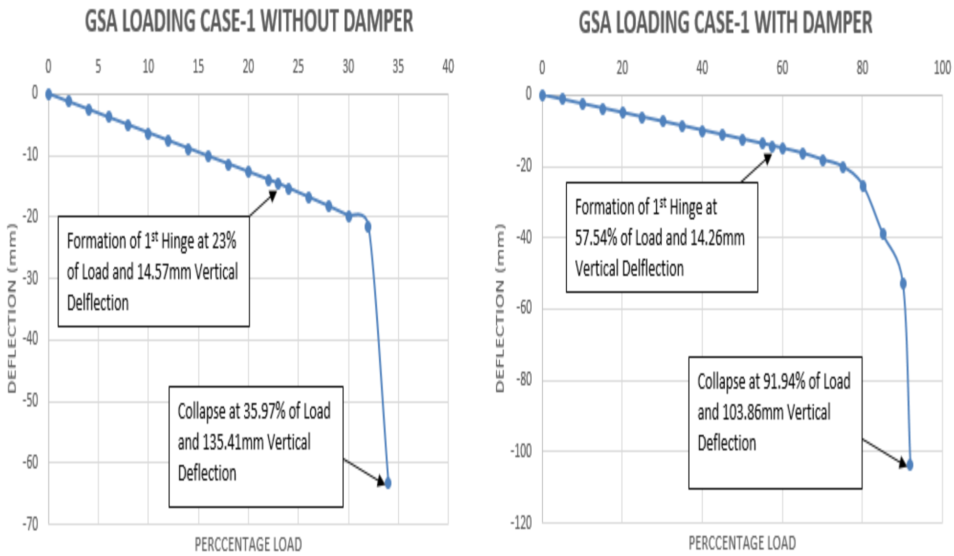
(a) without dampers (b) with dampers

Figure 3.59: Percentage Load v/s Vertical Deflection for UFC loading case-2 config-2



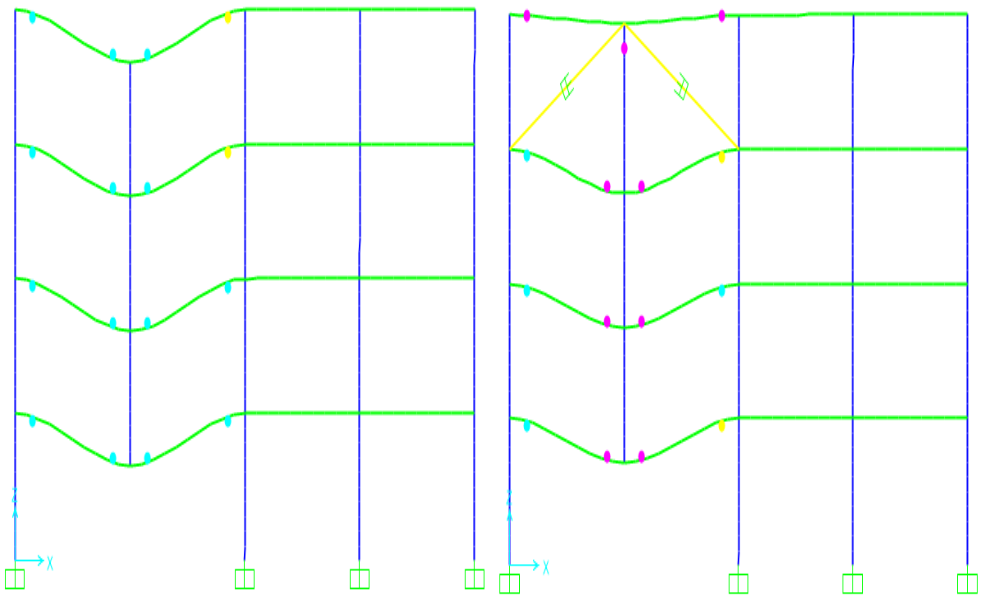
(a) without dampers (b) with dampers

Figure 3.60: Hinges at collapse for GSA loading case-1 configuration-2



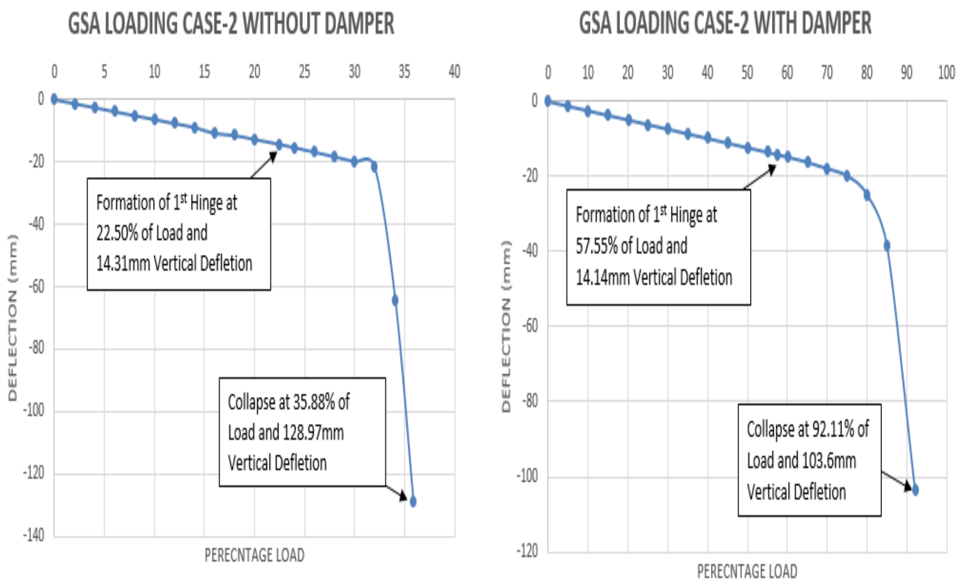
(a) without dampers (b) with dampers

Figure 3.61: Percentage Load v/s Vertical Deflection for GSA loading case-1 config-2



(a) without dampers (b) with dampers

Figure 3.62: Hinges at collapse for GSA loading case-2 configuration-2



(a) without dampers (b) with dampers

Figure 3.63: Percentage Load v/s Vertical Deflection for GSA loading case-2 config-2

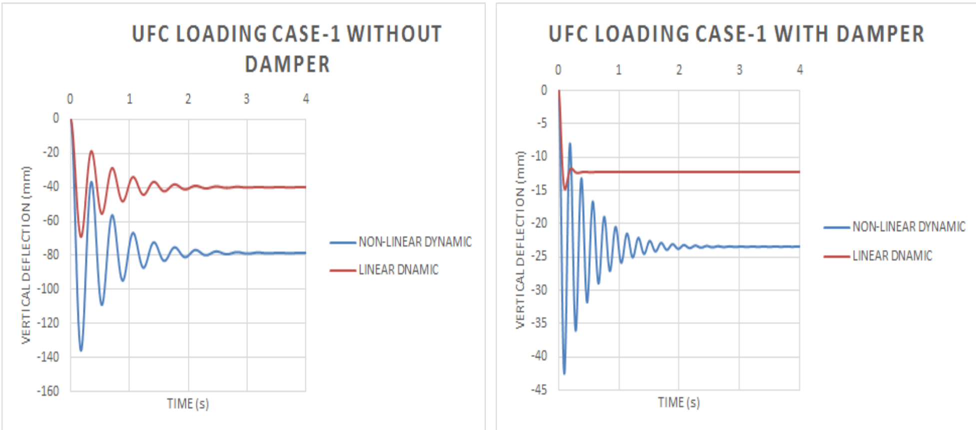
Table 3.6 shows comparison of load carrying capacity of frame with and without dampers for both the guidelines under two different column removal cases for configuration-2. Here case - 1 and case - 2 represents removal of middle column and removal of internal column, respectively.

Table 3.6: Comparison of Load carrying capacity of frame for configuration-2

Loading & Column Removal Case	**Percentage Load corresponding to formation of 1st Hinge**			**Percentage Collapse Load**		
	Without Damper	**With Damper**	**% Difference**	**Without Damper**	**With Damper**	**% Difference**
GSA guidelines Case-1	23.00 %	57.54 %	**+34.54 %**	35.97 %	91.94 %	**+55.97 %**
GSA guidelines Case-2	22.50 %	57.55 %	**+35.05 %**	35.88 %	92.11 %	**+56.23 %**
UFC guidelines Case-1	18.10 %	45.90 %	**+27.80 %**	28.59 %	72.85 %	**+44.26 %**
UFC guidelines Case-2	17.80 %	45.84 %	**+28.04 %**	28.57 %	72.88 %	**+44.31 %**

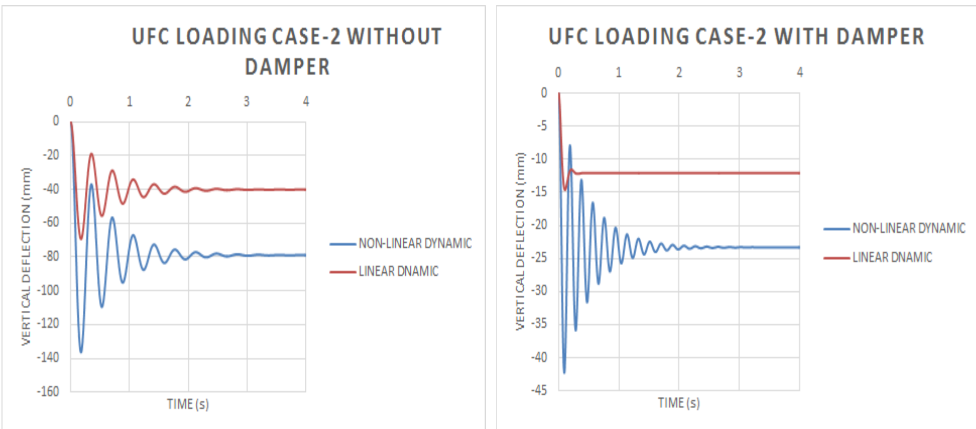
Comparison of Nonlinear Dynamic Analysis Result

Nonlinear dynamic analysis is performed to obtain displacement at location of removed columns. Displacement at the location of column removal is maximum in case of frame without dampers and it is observed that after incorporating viscoelastic damper with suitable damping there is considerable reduction in the displacement at the location of removed column. Nonlinear dynamic and linear dynamic analysis result are plotted for the vertical deflection under column removal and is compared to that of frame with dampers as shown in Fig. 3.64.



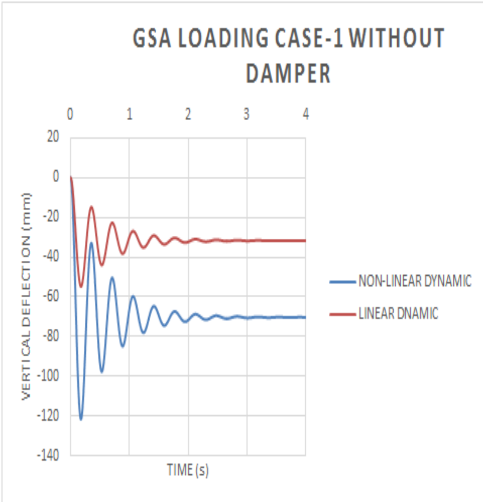
(a) (b)

Figure 3.64: Deflection under column removal point for Case 1 configuration-1

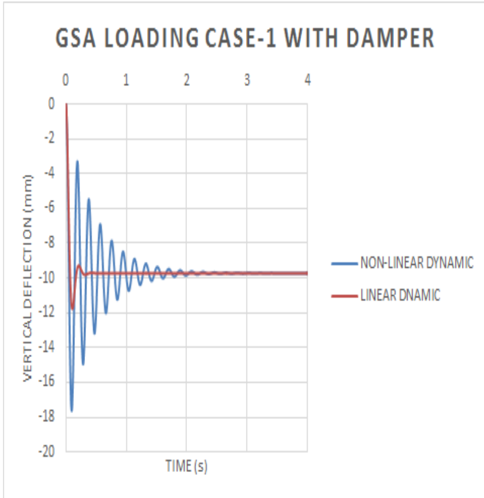


(a) (b)

Figure 3.65: Deflection under column removal point for Case 2 configuration-1

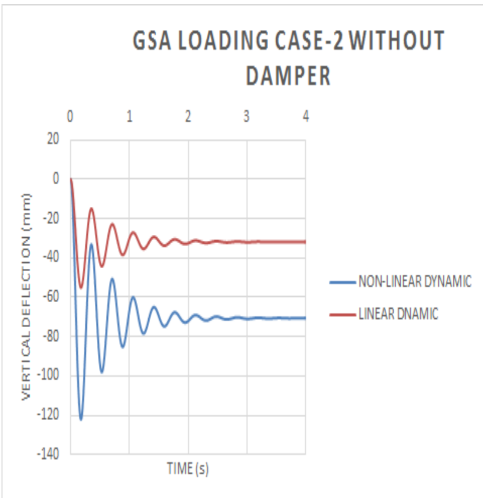


(a)

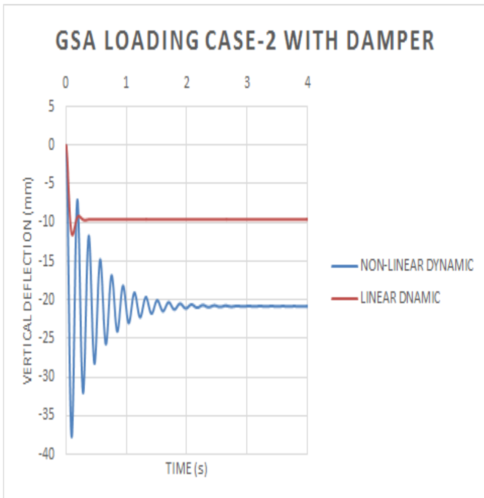


(b)

Figure 3.66: Deflection under column removal point for Case 1 configuration-1



(a)



(b)

Figure 3.67: Deflection under column removal point for Case 2 configuration-1

3.8 Summary

In this chapter, column is removed at two different locations and remaining structure is analyzed to evaluate the potential for progressive collapse of 4-storey reinforced concrete frame using structural analysis and design software SAP2000. Linear static, linear dynamic, Nonlinear static and Nonlinear dynamic analyses are performed by following the U.S. General Service Administration (GSA) and Department of Defense (DoD) guidelines using the alternate load path method. Demand Capacity Ratios (DCR) is calculated for flexure and shear in beams and axial force,biaxial moment in column and comparisons of results of frame without viscoelastic dampers and with damper. The DCR obtained by linear static analysis is compared for both with and without viscoelastic dampers. Vertical deflection for linear dynamic analysis and Non-linear dynamic analysis is obtained at the column removal location. For Non-linear static analysis load resistance capicity and deflection is plotted for both type of system.

Chapter 4

Progressive Collapse Analysis of 4 Storey Symmetric Building

4.1 General

Progressive collapse analysis is necessary to evaluate the capability of a structure to resist abnormal loadings. The proposed progressive collapse analysis method is threat independent, in the sense that it is initially assumed that some type of short duration abnormal loading has caused local damage represented by the removal of one or more critical members. Progressive collapse occurs when a structure has its loading pattern changed such that primary structural elements are loaded beyond their capacity and fail. The failure of few or more primary structural components leads to the redistribution of forces in adjoining members and further causes the failure of those members. As a result, a substantial part of the structure may collapse, causing greater damage to the structure than the initial impact.

To study the effect of failure of primary structural component on the entire structure, one 4-storey symmetrical reinforced concrete (RC) seismically designed building is analyzed for progressive collapse using the structural analysis and design software SAP2000 in this chapter.

4.2 Building Configuration

Progressive collapse analysis of 4-storey symmetric building is discussed here. The building is having bay width of 5 meter as shown in Fig. 4.1. Building is modeled in SAP2000 with the first storey height of 3.4 meter and all other storey with 3.1 meter height. Walls of 115 mm thickness are considered on all the beams. Building is analyzed and designed for seismic loading using structural analysis and design software SAP2000. Progressive collapse potential for building is carried out for five different column lost scenarios as highlighted by a circle in Fig. 4.1.

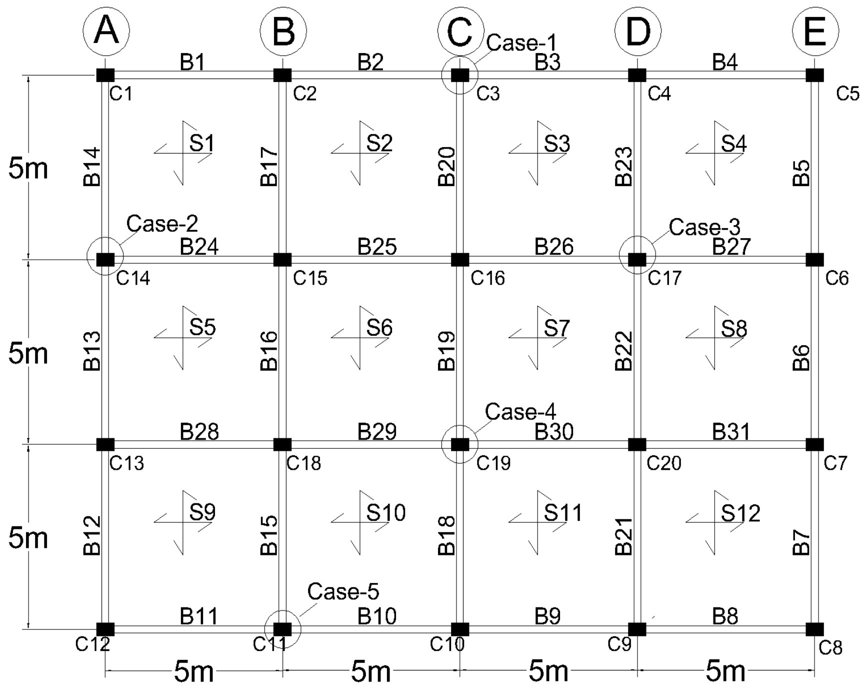


Figure 4.1: Plan of 4-store symmetric building

4.3 Loading Data

4-Storey symmetric building is analyzed and designed by considering following loading parameters and material properties. Building is analyzed and designed for seismic loading. Progressive collapse potential for building is carried out for five different column removal scenarios. Various loading data and size of the elements are as follows :

Gravity Loading Parameters:

- Dead Load - Self weight of the structural elements
- Live Load on roof = 1.5 kN/m^2
- Live Load on floors = 3.0 kN/m^2
- Floor Finish = 1.5 kN/m^2
- Wall Load = 6.325 kN/m^2

Seismic Loading Parameters:

- Seismic Zone - 5
- Soil type - Medium (II)
- Importance Factor - 1

Material Properties:

- Grade of Concrete - M25
- Grade of Steel - Fe415

Building Configuration:

- Slab Thickness = 150 mm
- Beam Size 300 mm × 550 mm
- Column Size 350 mm × 600 mm
- Wall Thickness = 115 mm
- Bay Span = 5 m
- Bottom Storey height = 3.4 m
- Typical Storey height = 3.1 m

Analysis and design of building is carried out by considering plan as shown in Fig. 4.1. Modeling of the building is carried out in SAP2000 with beam size 300 × 550 mm and column size 350 × 600 mm. Seismic design of the building is carried out for the maximum of following load combinations as suggested by IS 1893 (part 1) : 2002 [21].

- 1.5 (DL + LL)
- 1.2 (DL + LL ± EQx) and 1.2 (DL + LL ± EQy)
- 1.5 (DL ± EQx) and 1.5 (DL ± EQy)
- (0.9DL ± 1.5EQx) and (0.9DL ± 1.5EQy)

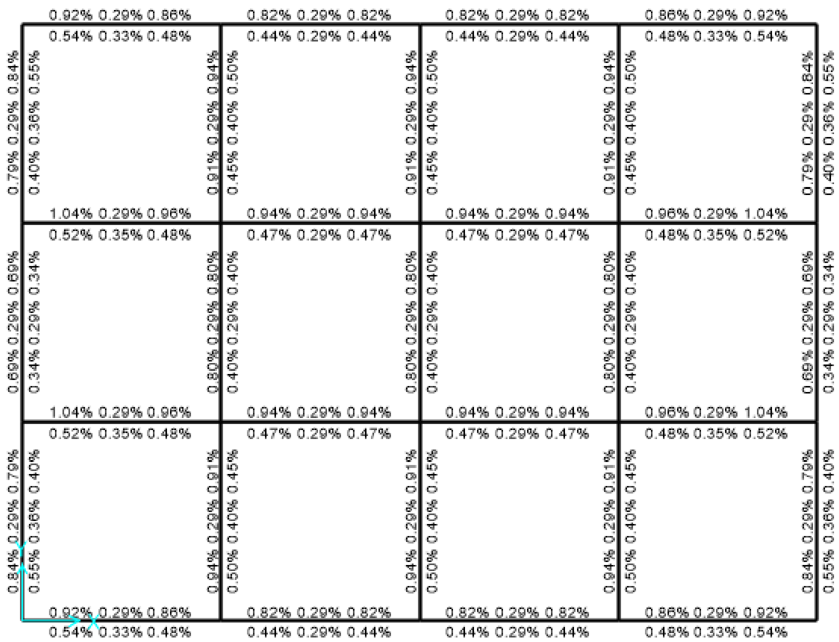


Figure 4.2: Percentage reinforcement in plan

Fig. 4.2 shows the percentage reinforcement required for beam at first floor level. The typical reinforcement detailing of beam at first floor level is shown in the Fig. 4.3. Typical reinforcement detailing of column section is shown in Fig. 4.4.

4.4 Progressive Collapse Analysis

After designing of 4-storey symmetric building, the vertical member as shown in Fig. 4.1 is removed separately from the bottom storey level. SAP2000 is used to understand the

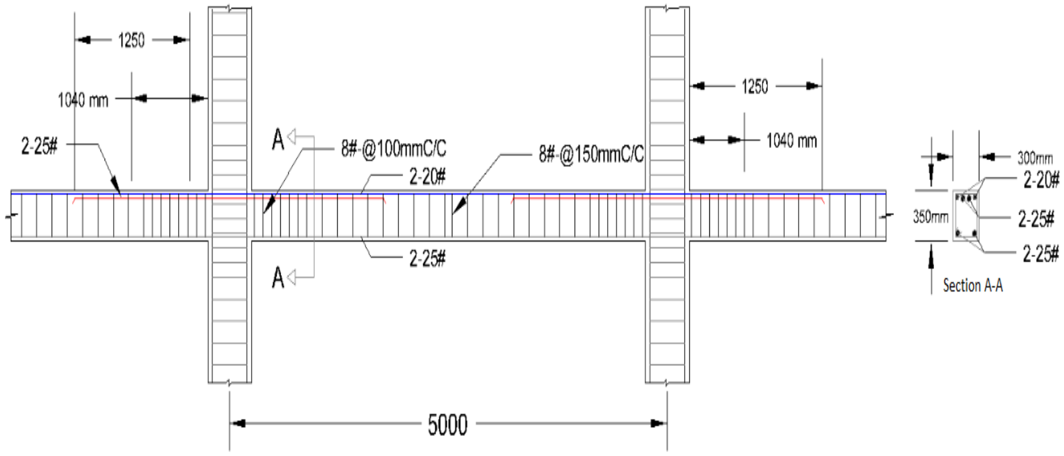


Figure 4.3: Typical reinforcement detailing of beam at first floor level

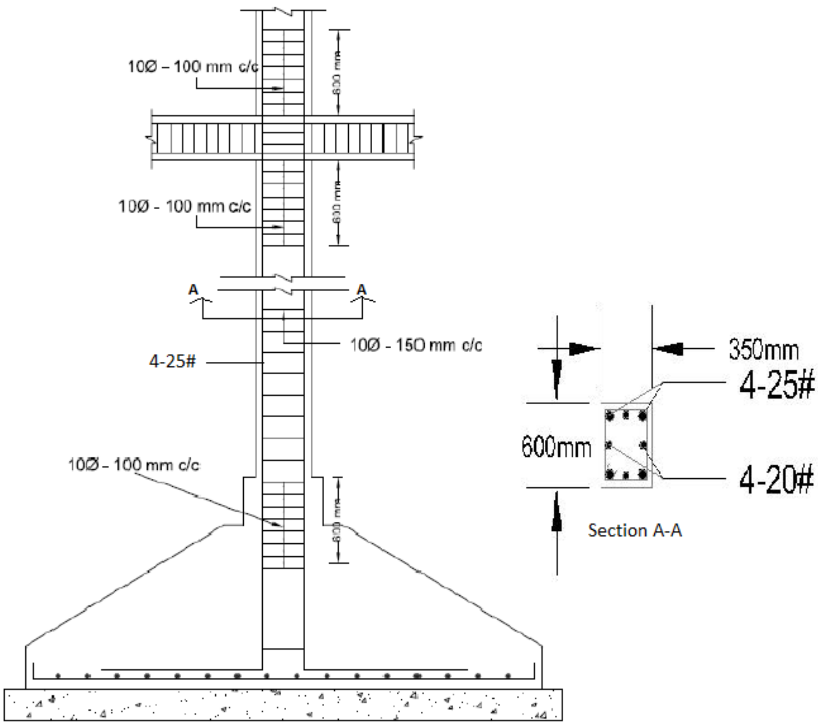


Figure 4.4: Typical reinforcement detailing of column

behavior of structure under different “failed column” scenarios. These five cases have been considered based on exterior and interior condition for column removal given by guidelines.

4.4.1 Linear Static Analysis

Linear static analysis is most simple method of analysis for progressive collapse. Linear static analysis is performed as illustrated in chapter 3 for 4-storey frame. Check for the DCR in each structural member is carried out above removed column location. Procedure for calculating DCR is illustrated in chapter 3. Various considerations regarding calculation of DCR and acceptance criteria of DCR suggested by guidelines are presented in previous chapter.

4.4.2 Linear Dynamic Analysis

The failure of vertical members under extreme events, such as blast and impact, is a highly dynamic phenomenon. So it is necessary to carry out the dynamic analysis of building to find out its response during abnormal loading. Linear dynamic analysis method involves real-time removal of load carrying structural elements. Thus it is more appropriate to refer to this method of analysis as a time history analysis. The detailed procedure to perform linear dynamic analysis is discussed in 3.4.2.

4.4.3 Nonlinear Static Analysis

Nonlinear static analysis is widely used to analyze a building for a lateral load and is known as “pushover analysis”. In this study vertical pushover analysis procedure is adopted to understand the behaviour of building structure. In this method loads is applied step by step until maximum load is attained or until the structure collapses. In the nonlinear static analysis for progressive collapse, structural elements are allowed to deform beyond elastic limit, hence it undergoes in to the inelastic behavior. In progressive collapse analysis, vertical pushover is applied, using normal service loading, until the maximum load or the maximum displacement is attained. Procedure to perform nonlinear static analysis is presented in 3.4.3.

4.4.4 Nonlinear Dynamic Analysis

The nonlinear dynamic analysis method is the most detailed method for the progressive collapse analysis in which a primary load-bearing structural element is removed dynamically and the structural material is allowed to undergo nonlinear behavior. This allows larger deformations and energy dissipation through material yielding, cracking, and fracture. Nonlinear dynamic analysis includes both material and geometrical nonlinearities. Procedure to perform nonlinear dynamic analysis is explained in the chapter 3.

4.5 Calculation of Demand Capacity Ratio (DCR)

From the analysis results, demand at critical points are obtained and from the designed section the capacity of the member is determined. With the help of calculated demand and capacity, check for the DCR in each structural member is carried out above the column removal location. Procedure to calculate DCR is illustrated in chapter 3. Various considerations regarding calculation of DCR and acceptance criteria of DCR suggested by guidelines are presented in previous chapter.

4.5.1 DCR for Beam

DCR is calculated for both flexure and shear for beam at critical locations. DCR is calculated at three locations left, center and right side of column removal position. Moment and shear capacity of section above the removed column can be found out with reference to IS 456:2000 [20]. Procedure for calculation of DCR for flexure and shear is presented in section 3.6.

4.5.2 DCR for Column

when column is removed from its position, as suggested by guidelines, redistribution of forces takes place in the structure, so forces in the column i.e. axial force, moment about major axis and moment about minor axis, changes and affects the adequacy of the columns. Therefore DCR is calculated for proximity column which is subjected to maximum redistributed forces for each column removal case. Procedure to calculate DCR for column is explained in previous chapter of analysis of 4-storey frame.

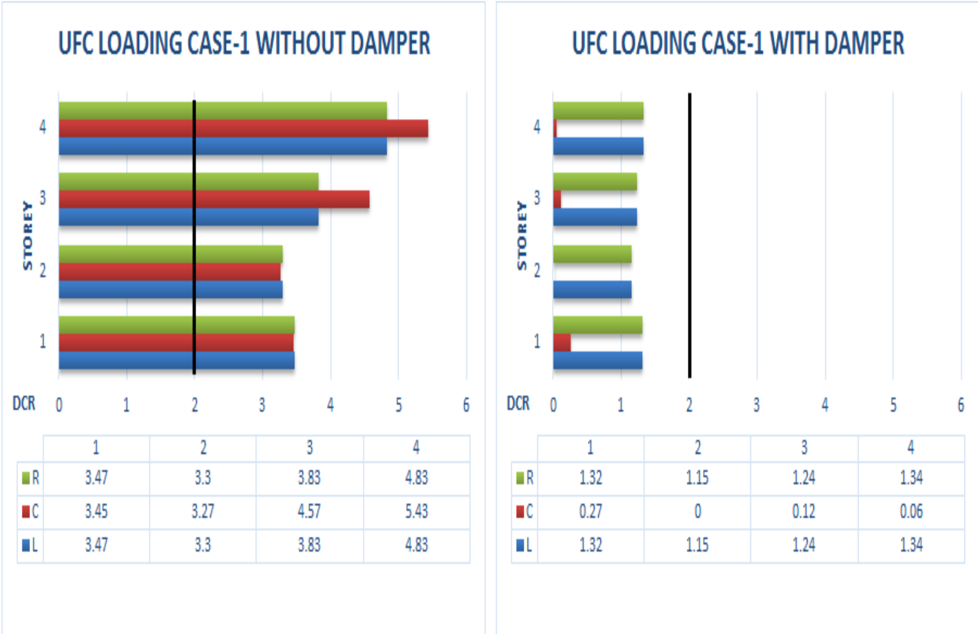
4.6 Results and Discussion

In this chapter, progressive collapse potential of 4-storey symmetrical Reinforced Concrete building is examined in terms of Demand Capacity Ratio (DCR). The linear static analysis of 4-storey symmetrical Reinforced concrete building has been performed by following the U.S. General Service Administration (GSA) and Department of Defense (DoD) guidelines. Demand Capacity Ratio (DCR) is found at critical locations by creating the member lost scenario for five different columns. Nonlinear static analysis is performed using SAP2000 to determine hinge formation pattern of building considered for the study. Linear dynamic and nonlinear dynamic analysis is also performed to obtain vertical deflections at column removal locations considering material and geometric nonlinearities .

Comparison Demand Capacity Ratio (DCR) for Flexure

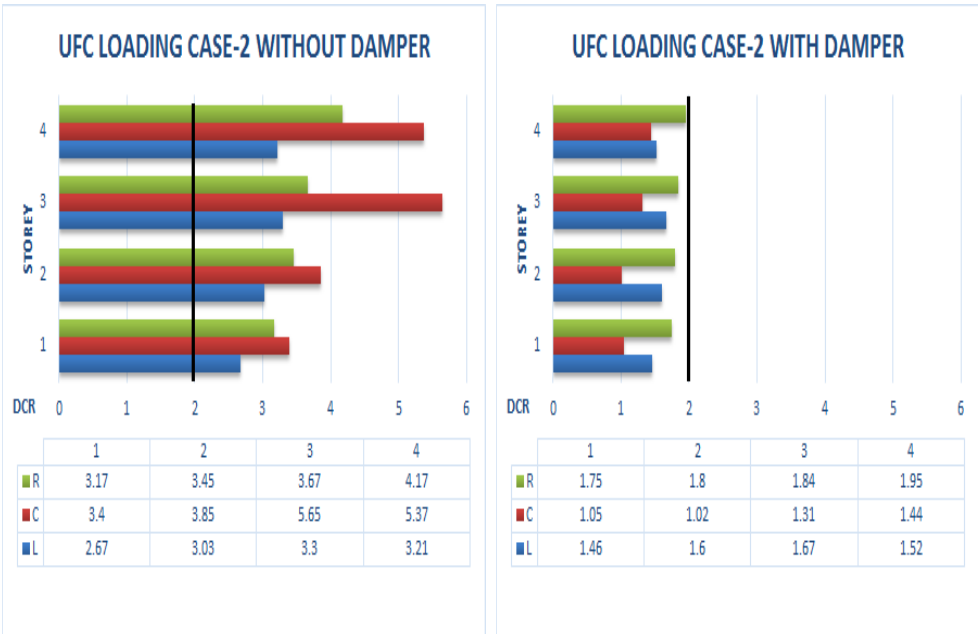
The Demand Capacity Ratios (DCR) are calculated at each storey for static analysis by removing the column from ground storey. DCR is calculated at three points left, center and right side of the column removal position. L, C and R indicates the value of DCR at left, center and right side from the position of removed column respectively for static analysis.

DCR for flexure is calculated at all the four stories for frame with damper and without damper and is compared as shown in Fig. 4.5 to Fig. 4.14 for both guidelines. From result it is observed that DCR in beams of buildings without dampers are exceeding the allowable limit i.e. 2 for all five column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 2, which indicates enhanced progressive collapse resistance of building.



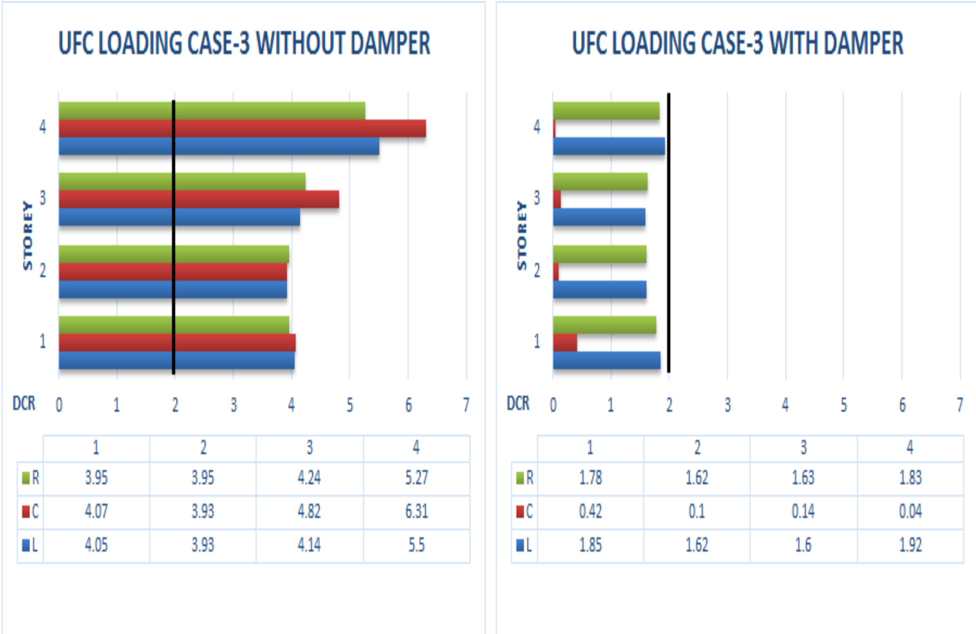
(a) DCR without damper (b) DCR with damper

Figure 4.5: DCR for flexure for case 1 (UFC Loading)



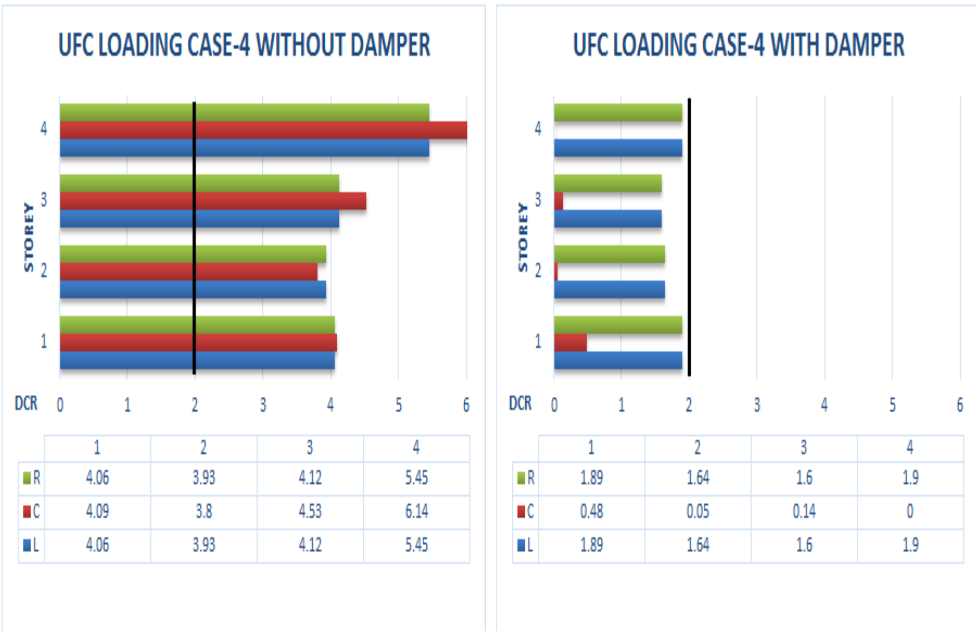
(a) DCR without damper (b) DCR with damper

Figure 4.6: DCR for flexure for case 2 (UFC Loading)



(a) DCR without damper (b) DCR with damper

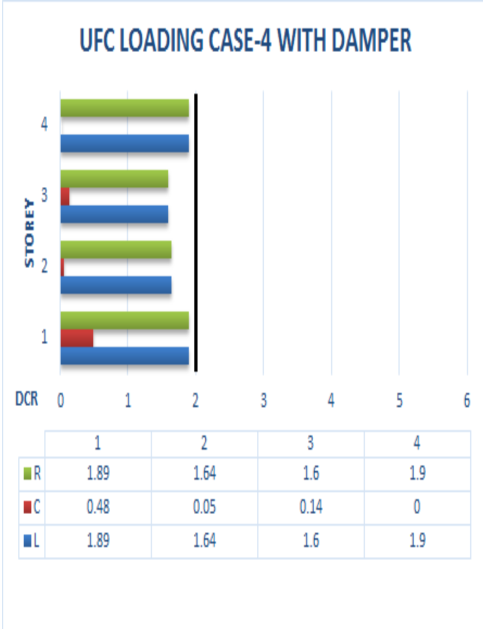
Figure 4.7: DCR for flexure for case 3 (UFC Loading)



(a) DCR without damper (b) DCR with damper

Figure 4.8: DCR for flexure for case 4 (UFC Loading)

(a) DCR without damper



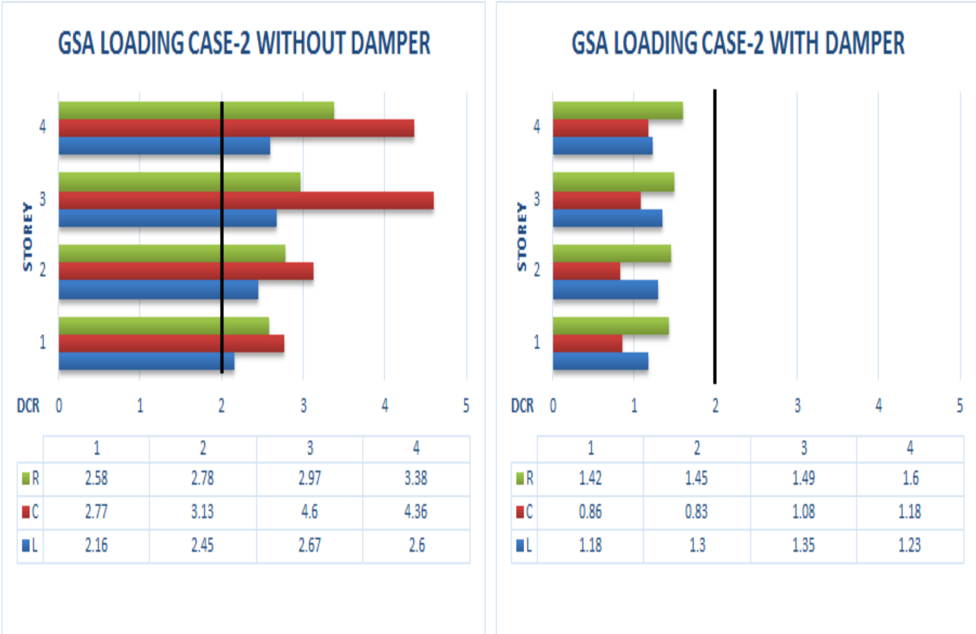
(b) DCR with damper

Figure 4.9: DCR for flexure for case 4 (UFC Loading)

(a) DCR without damper

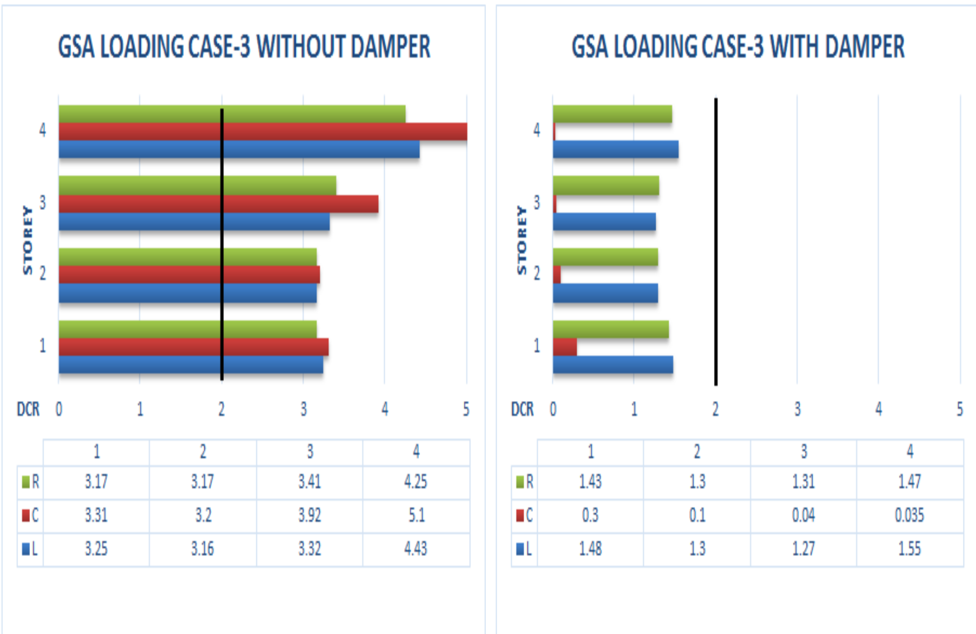
(b) DCR with damper

Figure 4.10: DCR for flexure for case 1 (GSA Loading)



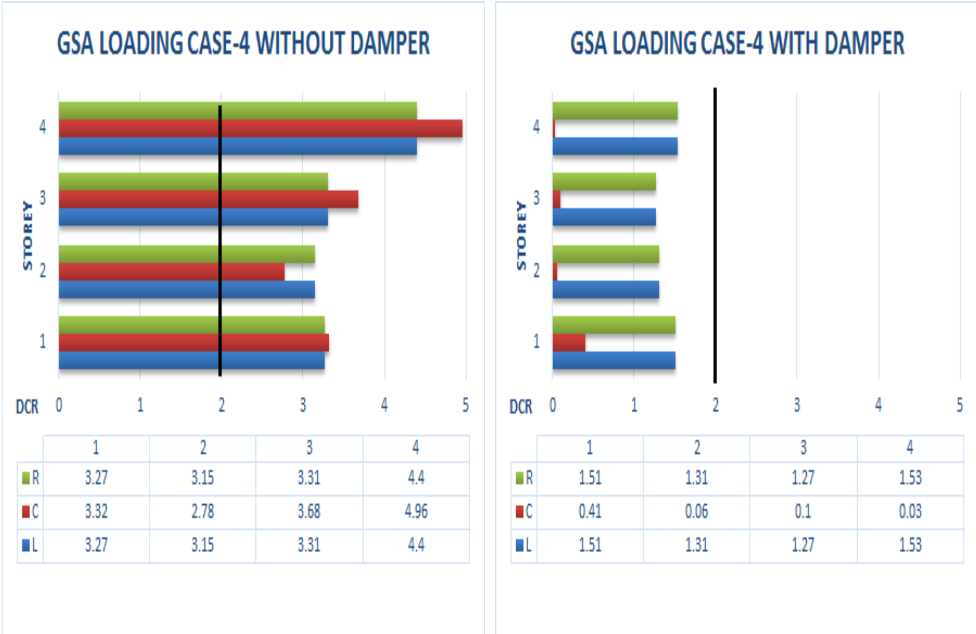
(a) DCR without damper (b) DCR with damper

Figure 4.11: DCR for flexure for case 2 (GSA Loading)



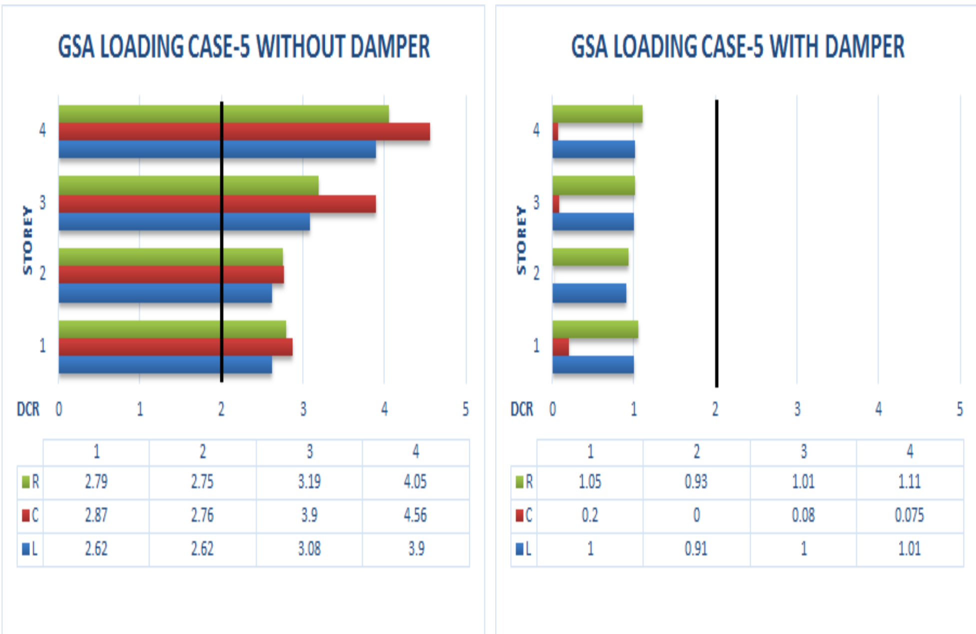
(a) DCR without damper (b) DCR with damper

Figure 4.12: DCR for flexure for case 3 (GSA Loading)



(a) DCR without damper (b) DCR with damper

Figure 4.13: DCR for flexure for case 4 (GSA Loading)

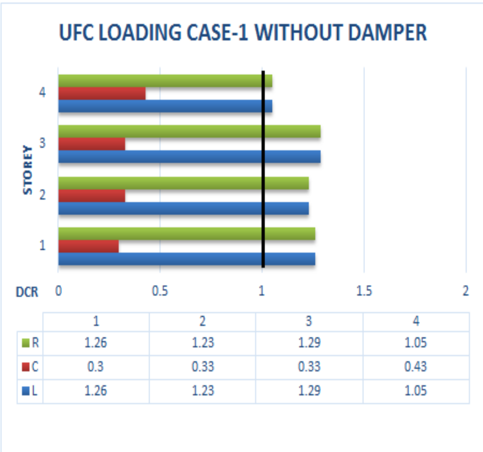


(a) DCR without damper (b) DCR with damper

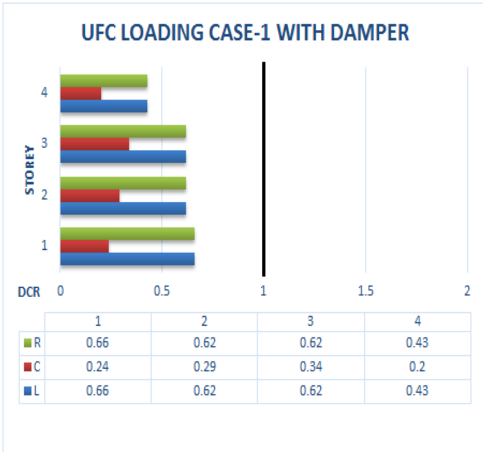
Figure 4.14: DCR for flexure for case 5 (GSA Loading)

Comparison of Demand Capacity Ratio (DCR) for Shear

DCR for shear is calculated at all the four stories for frame with damper and without damper and is compared as shown in Fig. 4.15 to Fig. 4.24 for both guidelines. From result it is observed that DCR in beams of buildings without dampers are exceeding the allowable limit i.e. 1 for all five column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 1, which indicates enhanced progressive collapse resistance of building.

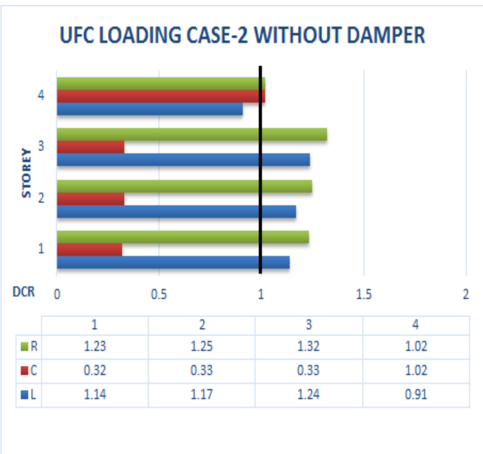


(a) DCR without damper

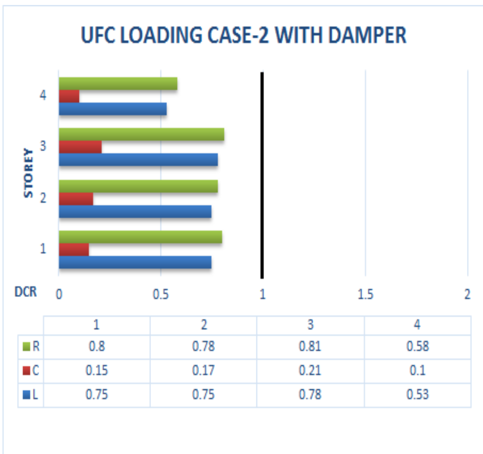


(b) DCR with damper

Figure 4.15: DCR for shear for case 1 (UFC Loading)

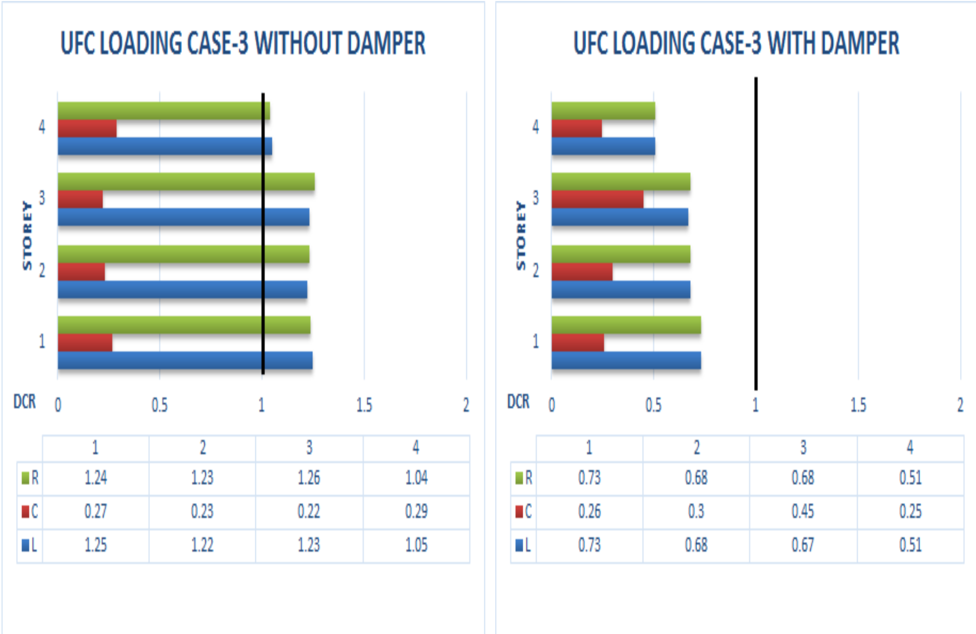


(a) DCR without damper



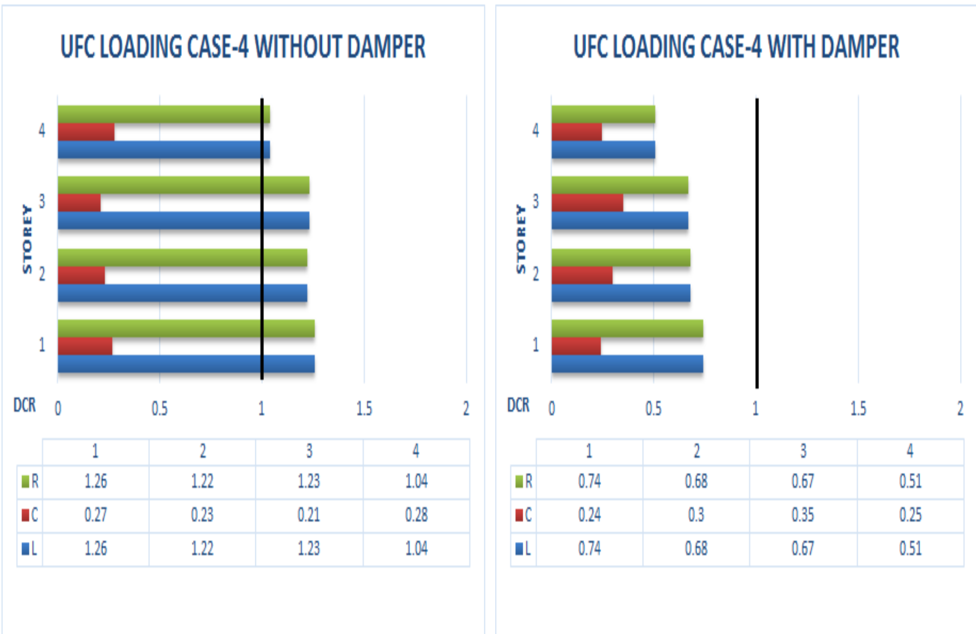
(b) DCR with damper

Figure 4.16: DCR for shear for case 2 (UFC Loading)



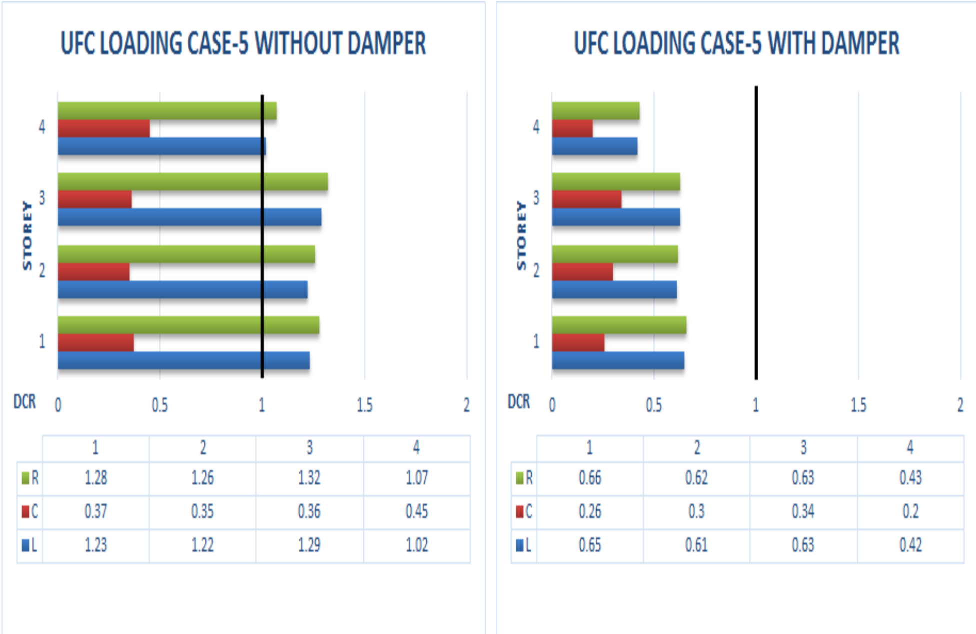
(a) DCR without damper (b) DCR with damper

Figure 4.17: DCR for shear for case 3 (UFC Loading)



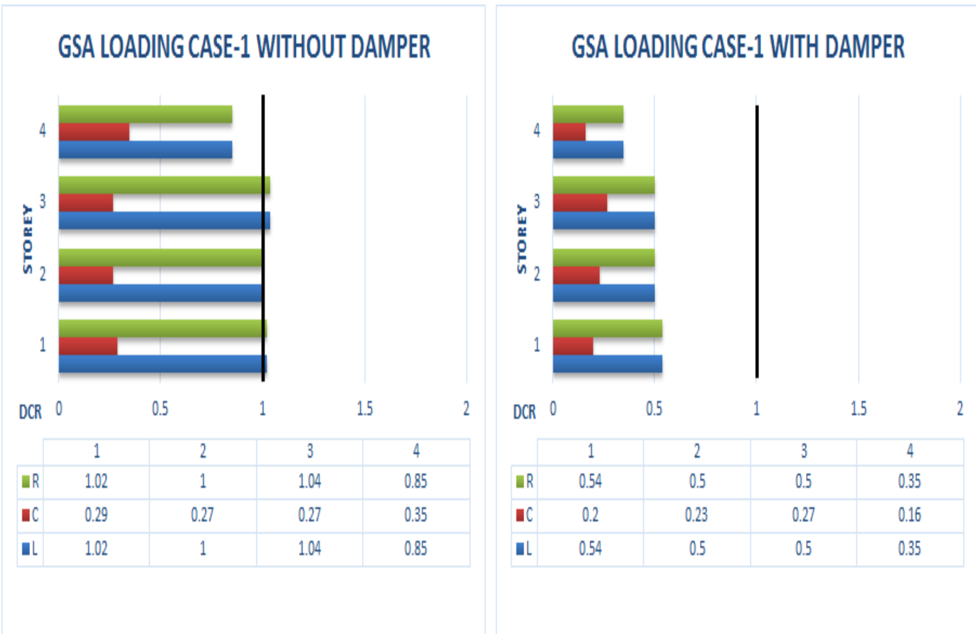
(a) DCR without damper (b) DCR with damper

Figure 4.18: DCR for shear for case 4 (UFC Loading)



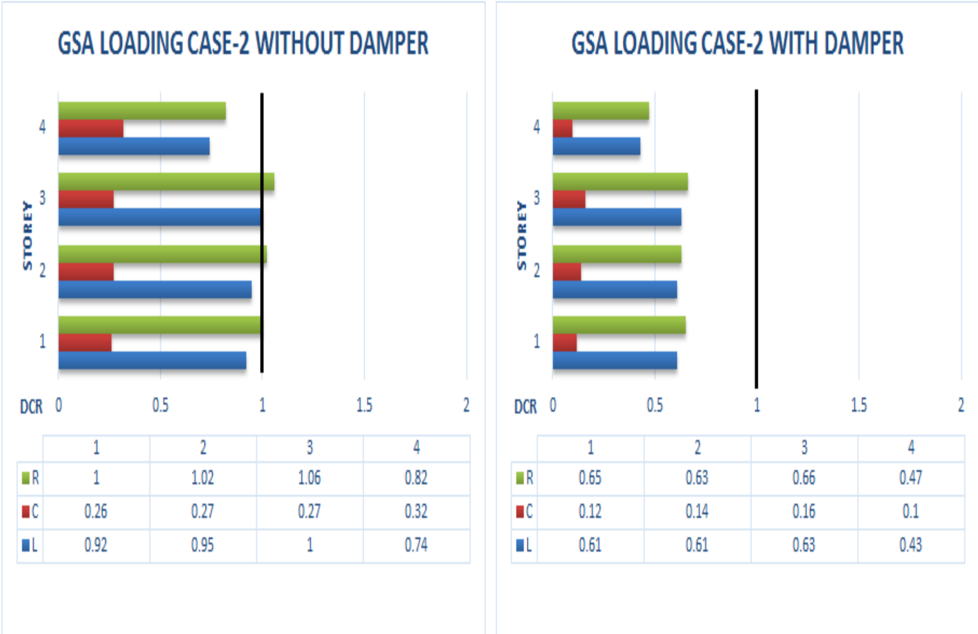
(a) DCR without damper (b) DCR with damper

Figure 4.19: DCR for shear for case 5 (UFC Loading)



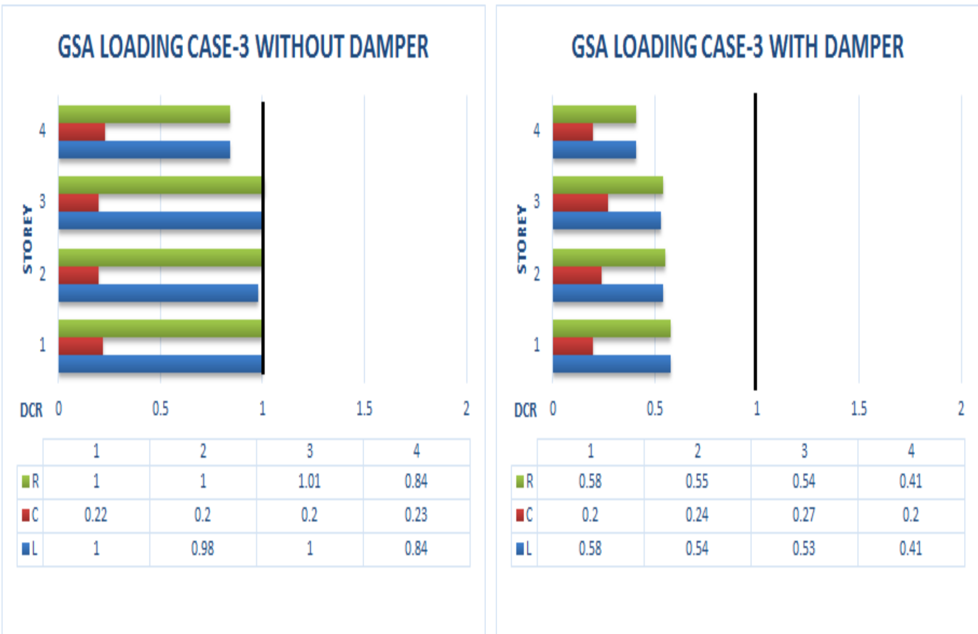
(a) DCR without damper (b) DCR with damper

Figure 4.20: DCR for shear for case 1 (GSA Loading)



(a) DCR without damper (b) DCR with damper

Figure 4.21: DCR for shear for case 2 (GSA Loading)



(a) DCR without damper (b) DCR with damper

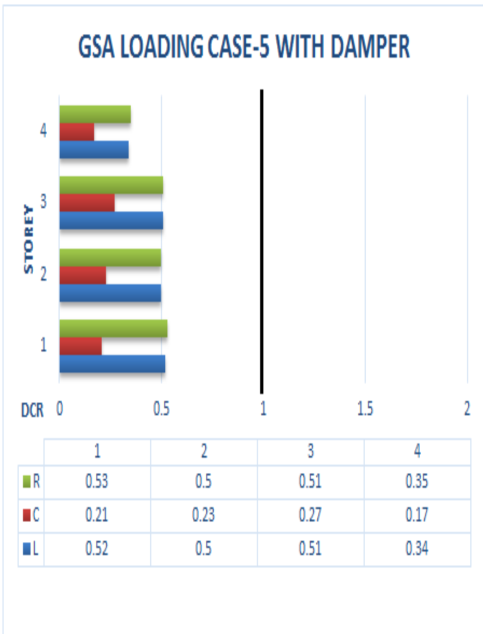
Figure 4.22: DCR for shear for case 3 (GSA Loading)

(a) DCR without damper

(b) DCR with damper

Figure 4.23: DCR for shear for case 4 (GSA Loading)

(a) DCR without damper

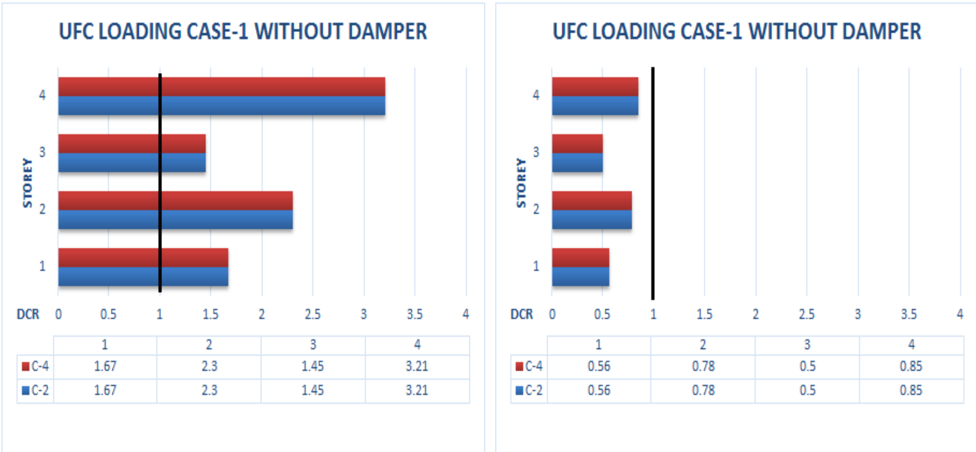


(b) DCR with damper

Figure 4.24: DCR for shear for case 5 (GSA Loading)

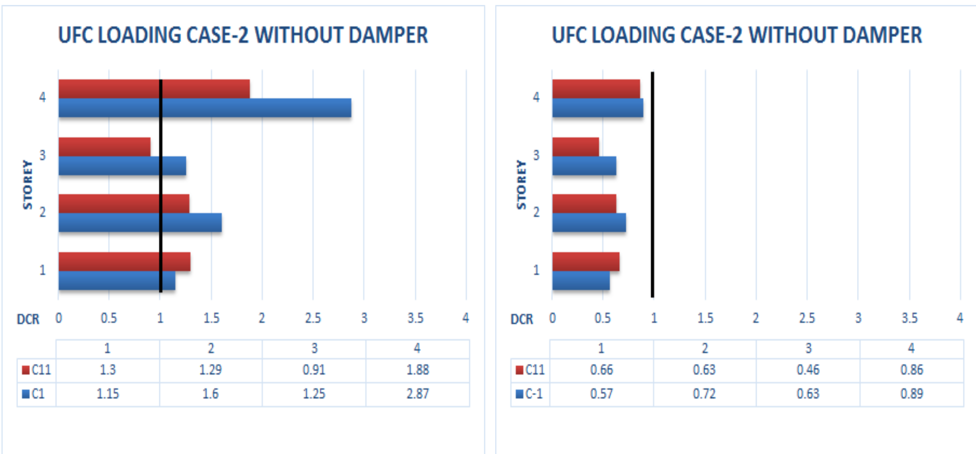
Comparison of Demand Capacity Ratio (DCR) for Column

DCR for column is calculated at all the four stories for frame with damper and without damper and is compared as shown in Fig. 4.25 to Fig. 4.34 for both guidelines. From result it is observed that DCR in columns of buildings without dampers are exceeding the allowable limit i.e. 1 for all five column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 1, which indicates enhanced progressive collapse resistance of building.



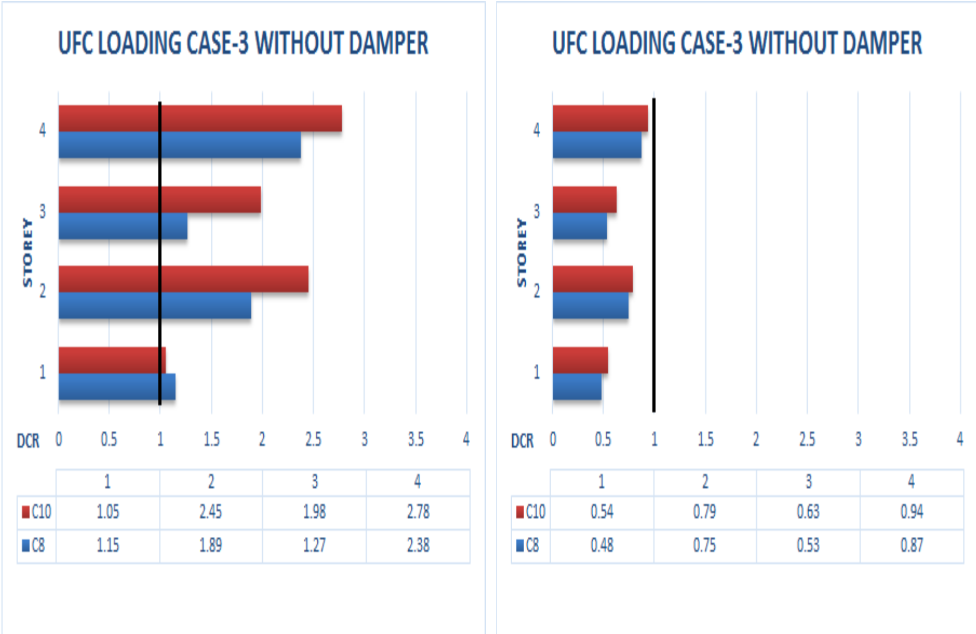
(a) DCR without damper (b) DCR with damper

Figure 4.25: DCR for column for case 1 (UFC Loading)



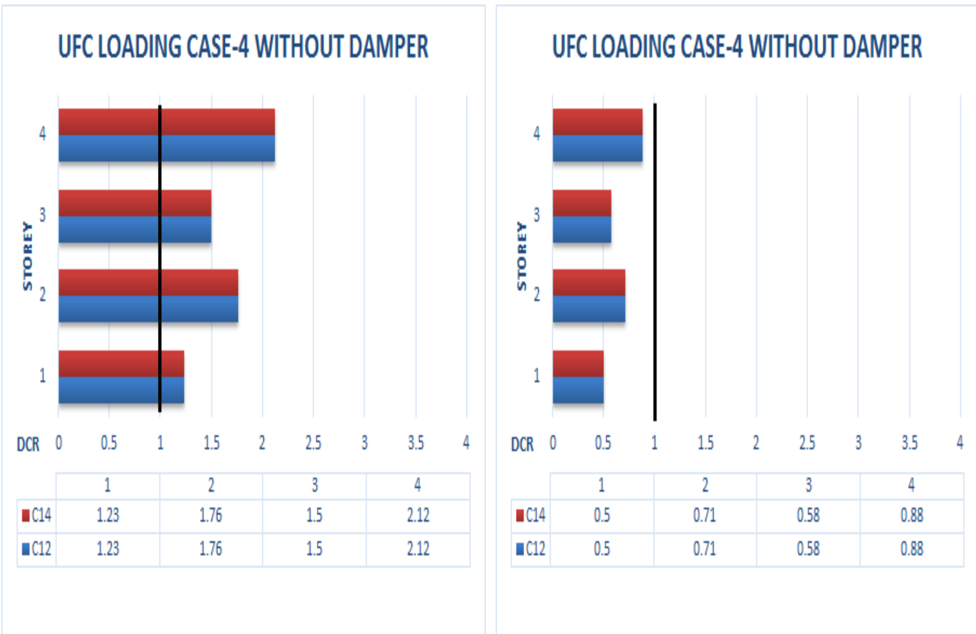
(a) DCR without damper (b) DCR with damper

Figure 4.26: DCR for column for case 2 (UFC Loading)



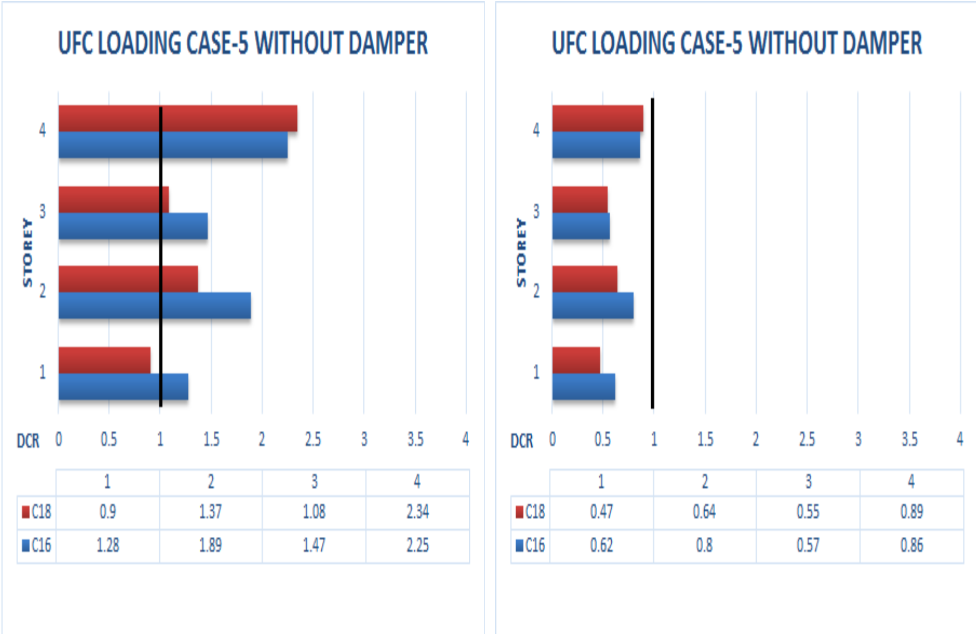
(a) DCR without damper (b) DCR with damper

Figure 4.27: DCR for column for case 3 (UFC Loading)



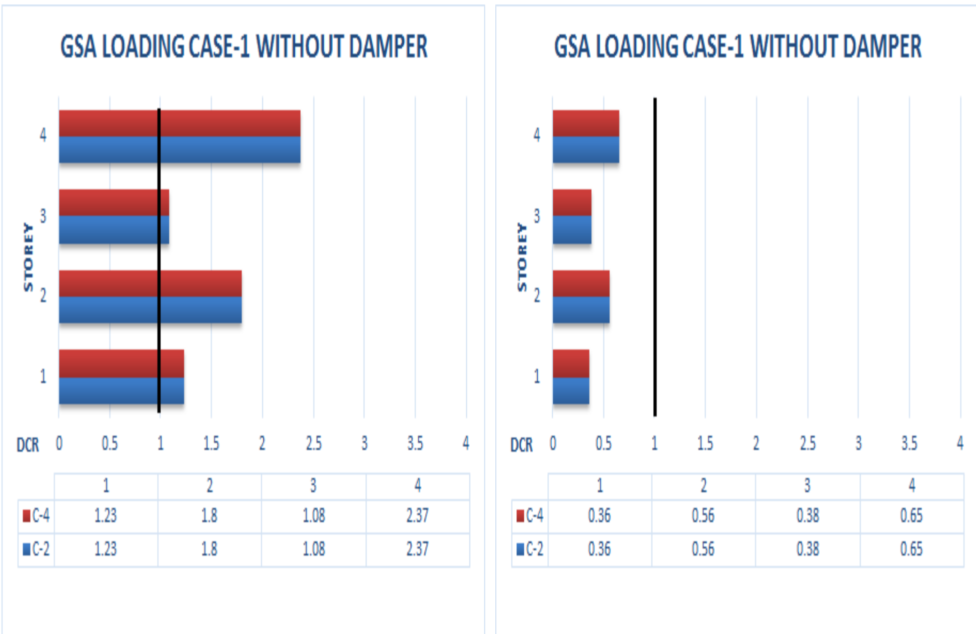
(a) DCR without damper (b) DCR with damper

Figure 4.28: DCR for column for case 4 (UFC Loading)



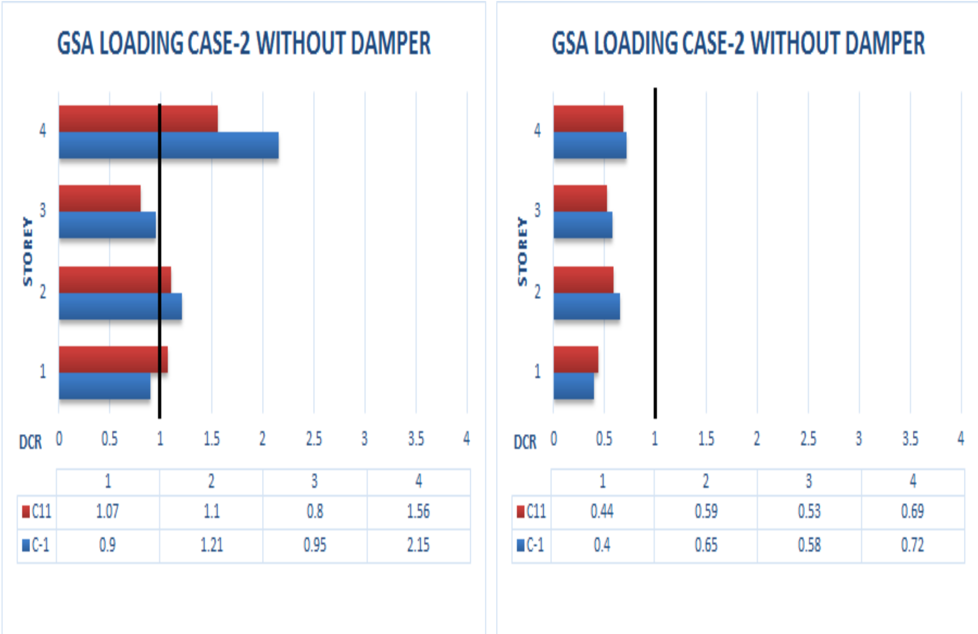
(a) DCR without damper (b) DCR with damper

Figure 4.29: DCR for column for case 5 (UFC Loading)



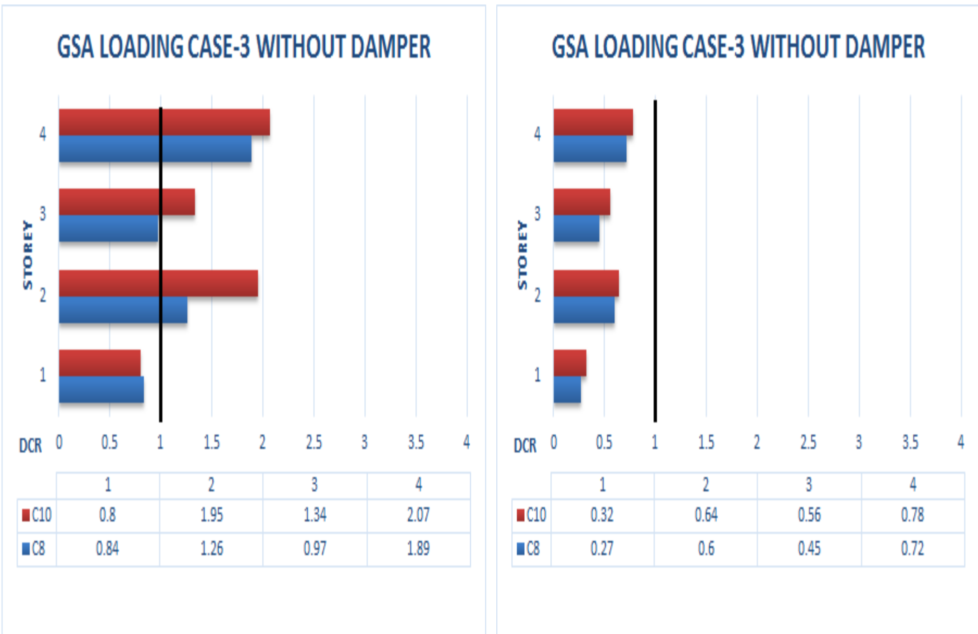
(a) DCR without damper (b) DCR with damper

Figure 4.30: DCR for column for case 1 (GSA Loading)



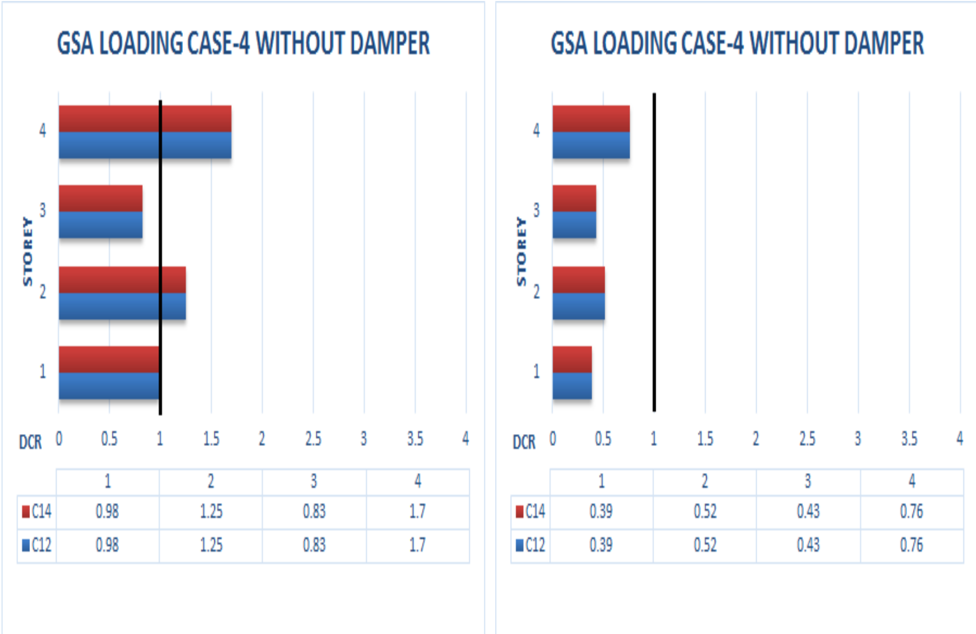
(a) DCR without damper (b) DCR with damper

Figure 4.31: DCR for column for case 2 (GSA Loading)



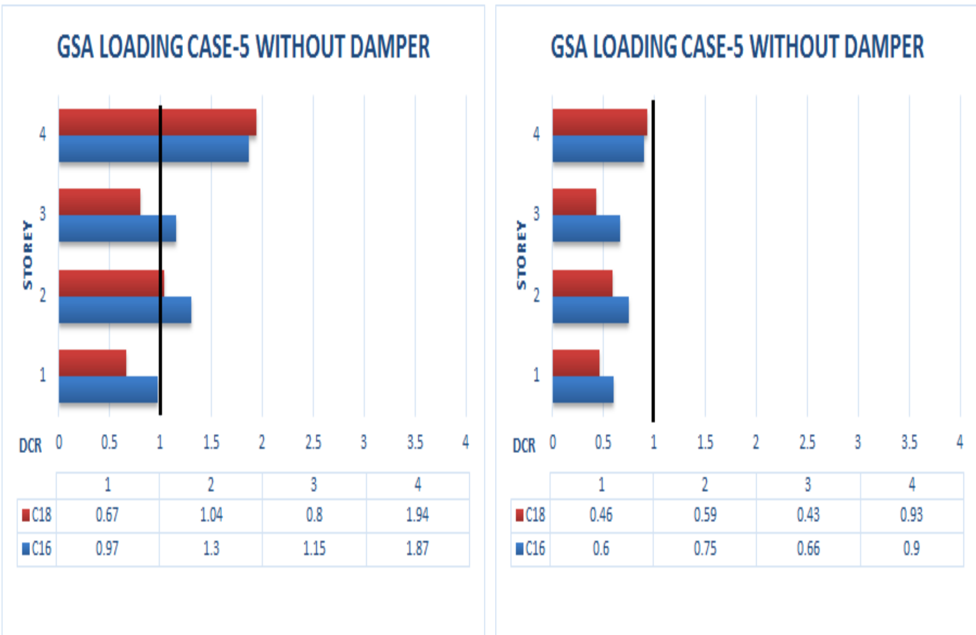
(a) DCR without damper (b) DCR with damper

Figure 4.32: DCR for column for case 3 (GSA Loading)



(a) DCR without damper (b) DCR with damper

Figure 4.33: DCR for column for case 4 (GSA Loading)

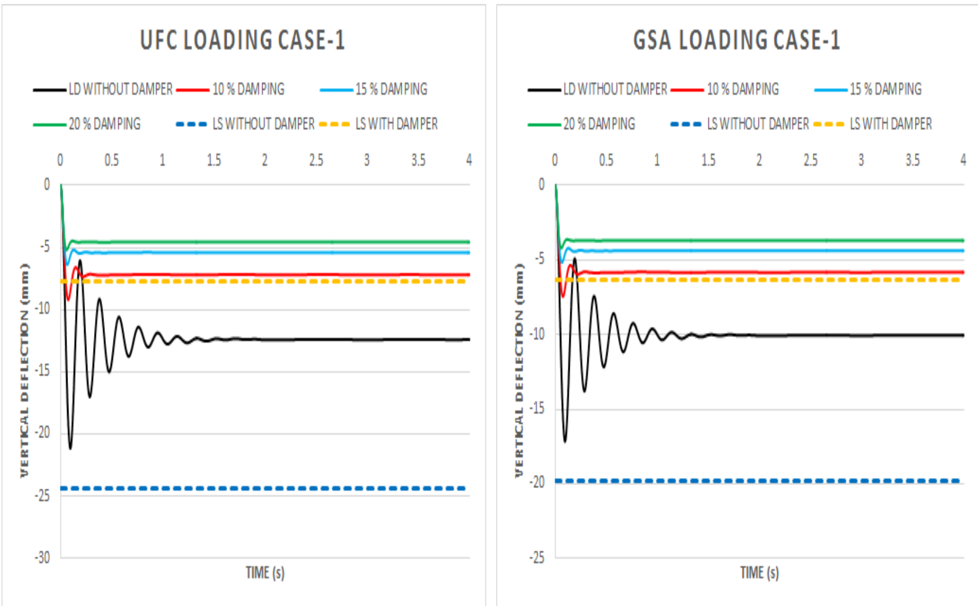


(a) DCR without damper (b) DCR with damper

Figure 4.34: DCR for column for case 5 (GSA Loading)

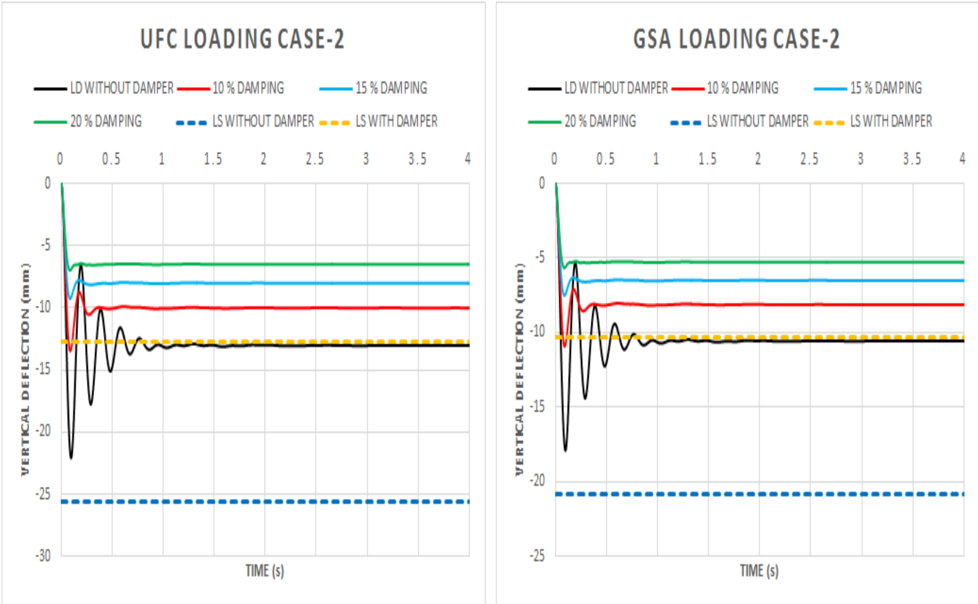
Comparison of Linear Dynamic Analysis Results

In linear dynamic analysis reaction is applied at the location of column removal and is make to zero after some elapsed time to incorporate dynamic effect. Linear dynamic analysis is performed to obtain displacement at location of removed columns. Displacement at the location of column removal is maximum in case of frame without dampers and it is observed that after incorporating viscoelastic damper with suitable damping there is considerable reduction in the displacement at the location of removed column as shown in Fig. 4.35 to Fig. 4.39 for all the five column removal cases.



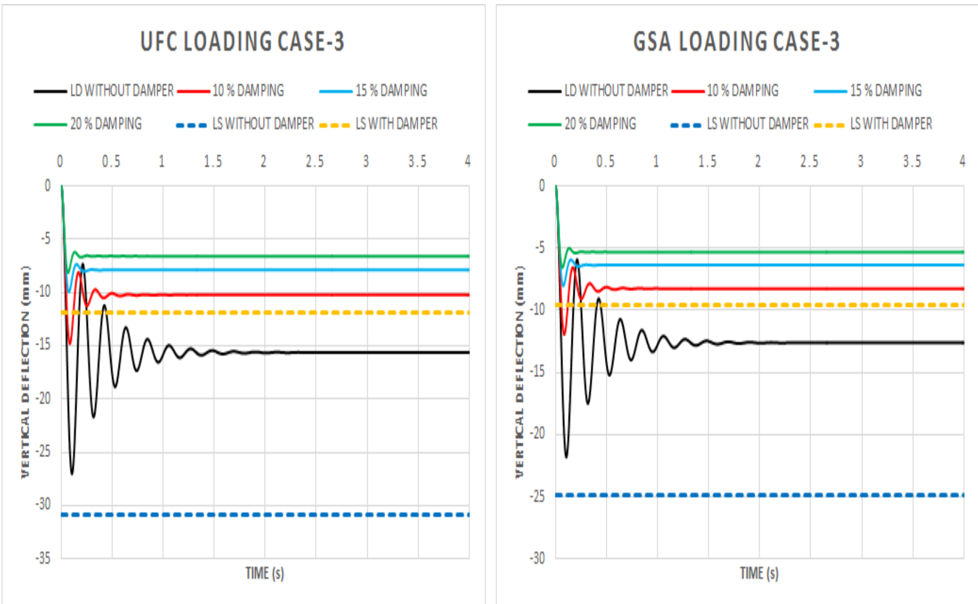
(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.35: Deflection under column removal point for Case 1



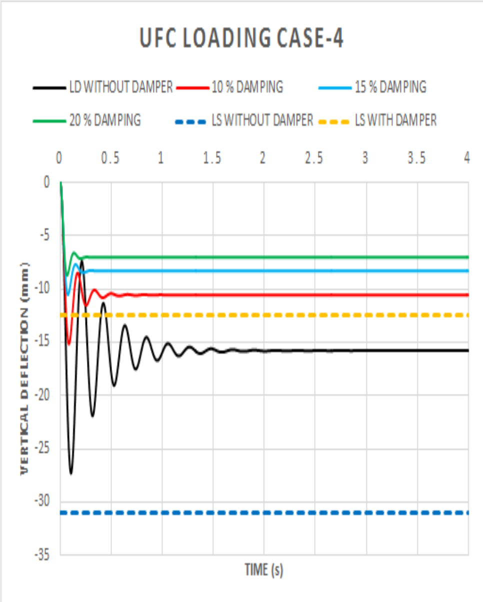
(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.36: Deflection under column removal point for Case 2

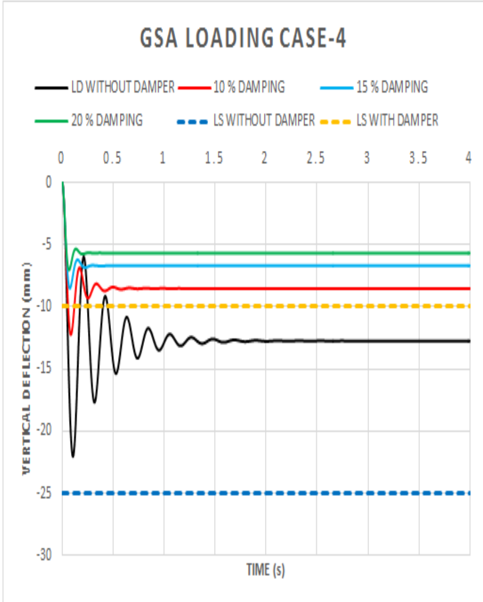


(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.37: Deflection under column removal point for Case 3

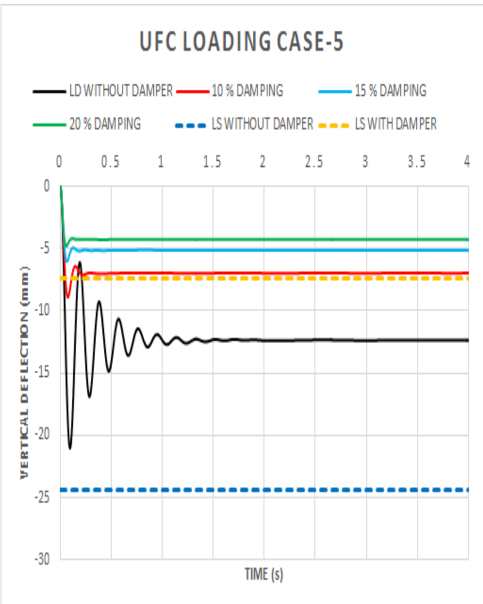


(a) Deflection for UFC loading

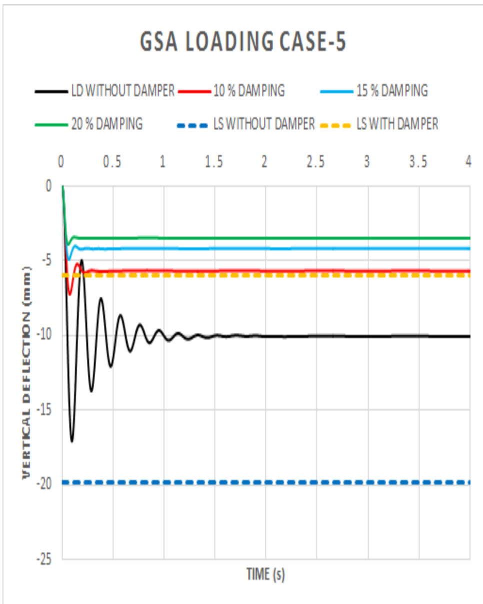


(b) Deflection for GSA loading

Figure 4.38: Deflection under column removal point for Case 4



(a) Deflection for UFC loading



(b) Deflection for GSA loading

Figure 4.39: Deflection under column removal point for Case 5

Here the Table 4.1 represents the vertival deflection at column removal location for both GSA and UFC guidelines. Vertical deflection is maximum for linear static analysis as compared to linear dynamic analysis. Incorporating viscoelastic damper with suitable damping, considerably reduces the displacement at the location of removed column.

Table 4.1: Comparison of Vertical deflection at column removal location for UFC loading

	Maximum Vertical Deflection (mm)					
Loading & Column Removal Case	Linear Static Without Damper	Linear Dynamic Without Damper	Linear Static With Damper	10 % Damping	15 % Damping	20 % Damping
UFC Guidelines Case-1	24.40	21.16	7.7	9.19	6.36	5.18
UFC Guidelines Case-2	25.66	22.08	12.70	13.47	9.27	6.99
UFC Guidelines Case-3	30.81	27.02	11.88	14.80	9.95	8.12
UFC Guidelines Case-4	31.00	27.30	12.40	15.16	10.54	8.72
UFC Guidelines Case-5	24.40	21.03	7.40	8.92	6.04	4.79

Here the Table 4.2 represents the vertival deflection at column removal location for both GSA and UFC guidelines. Vertical deflection is maximum for linear static analysis as compared to linear dynamic analysis. Incorporating viscoelastic damper with suitable damping, considerably reduces the displacement at the location of removed column.

Table 4.2: Comparison of Vertical deflection at column removal location for GSA loading

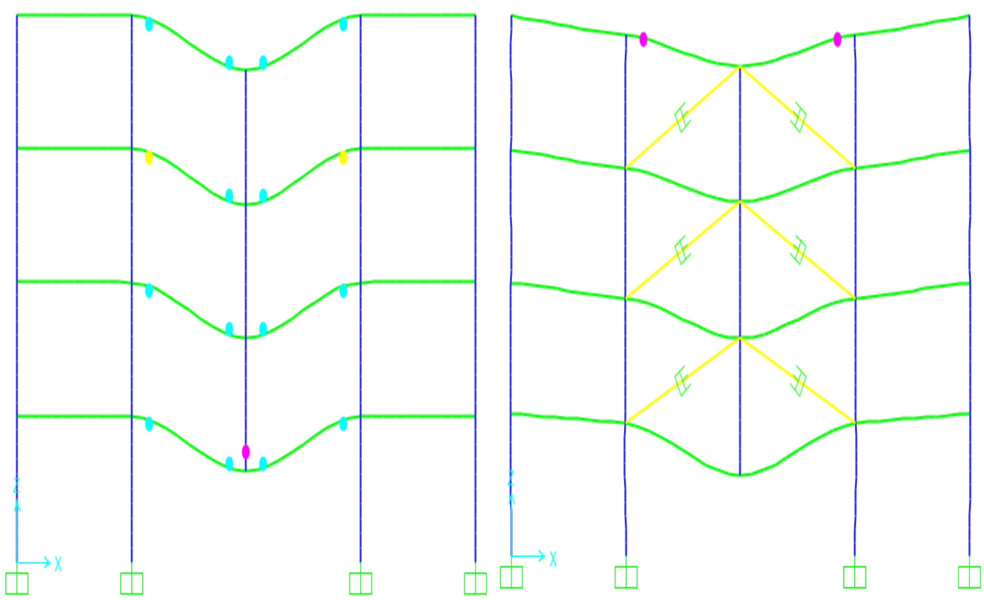
	Maximum Vertical Deflection (mm)					
Loading & Column Removal Case	Linear Static Without Damper	Linear Dynamic Without Damper	Linear Static With Damper	10 % Damping	15 % Damping	20 % Damping
GSA Guidelines Case-1	19.80	17.16	6.30	7.45	5.16	4.20
GSA Guidelines Case-2	20.80	17.90	10.30	10.92	7.51	5.66
GSA Guidelines Case-3	24.90	21.81	9.6	11.95	8.03	6.56
GSA Guidelines Case-4	25.00	22.04	10.00	12.24	8.51	7.04
GSA Guidelines Case-5	19.80	17.06	6.00	7.23	4.90	3.89

Comparison of Nonlinear Static Analysis Result

In progressive collapse analysis, vertical pushover is applied, using normal service loading, until the maximum load or the maximum displacement is attained. In this method load is applied step by step until maximum load is attained or until the structure collapses. The failure of one load bearing element may likely to cause failure of other elements resulting in progressive collapse of entire structure. Therefore, nonlinear static analysis is useful for evaluation and to observe the hinge formation pattern in the building during column removal scenario.

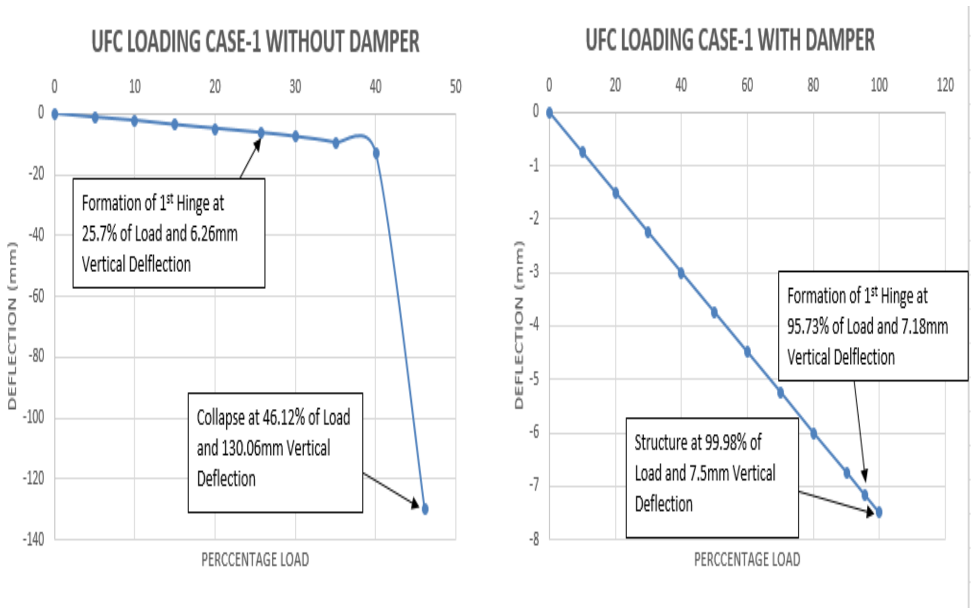
Results are observed in terms of vertical deflection under column removal scenario, formation of hinge pattern and load carrying capacity. Vertical deflection is measured at each step at an interval of 5% increase in load. A graph of vertical deflection at location of removed column corresponding to % load resisted by frame is plotted as shown in Fig. 4.41. From the results, it is observed that, viscoelastic dampers significantly increases load carrying capacity of the frame as well as drastically reduces the vertical deflection.From the hinge formation pattern, it is observed that less number of hinges are formed when viscoelastic dampers are provided, as compared to frame without dampers

Pattern of hinge formation at the collapse load for frames with and without dampers are presented. From the hinge formation pattern, it is observed that less number of hinges are formed when viscoelastic dampers are provided, as compared to frame without dampers as shown in Fig. 4.40.



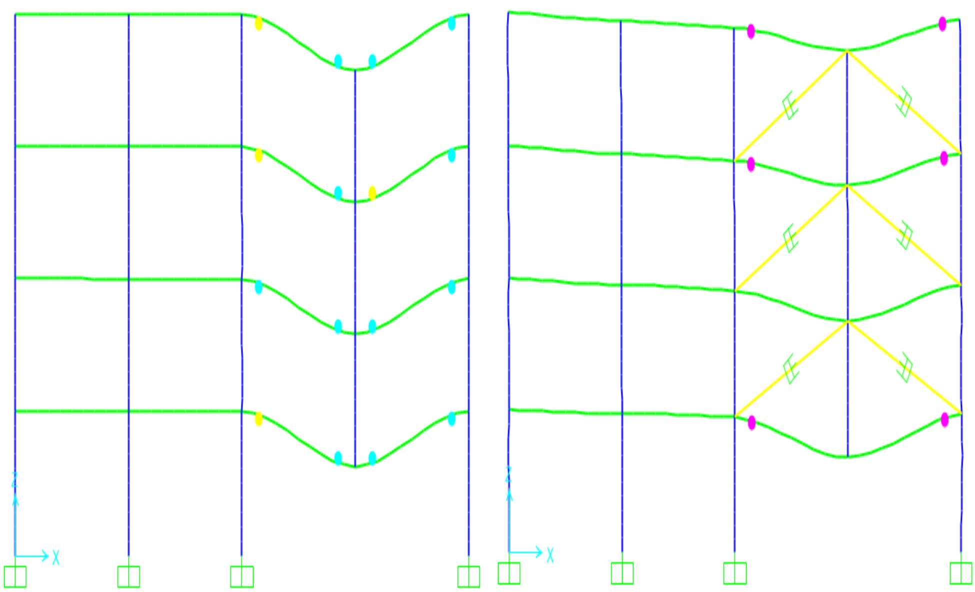
(a) without dampers (b) with dampers

Figure 4.40: Hinges at collapse for UFC loading case-1



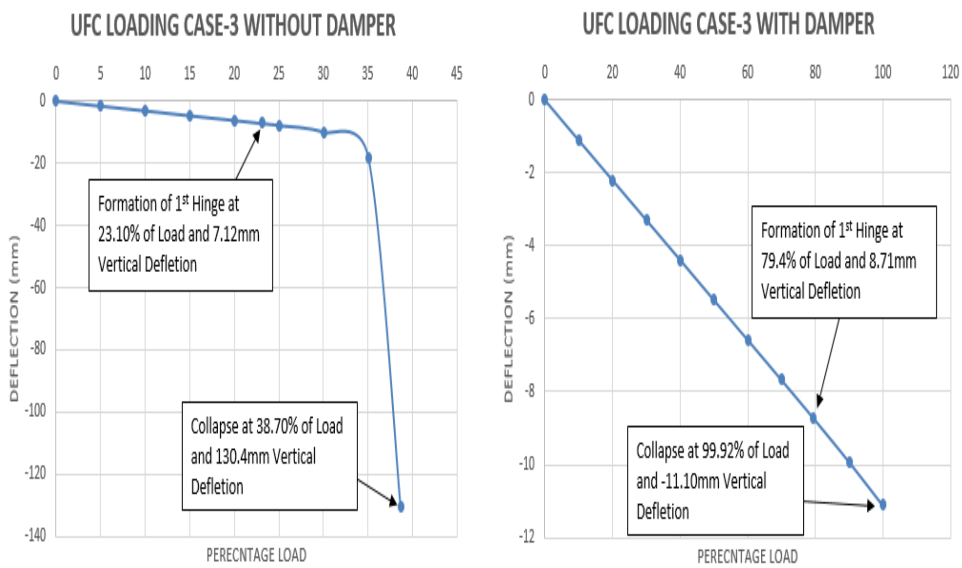
(a) without dampers (b) with dampers

Figure 4.41: Percentage Load v/s Vertical Deflection for UFC loading case-1



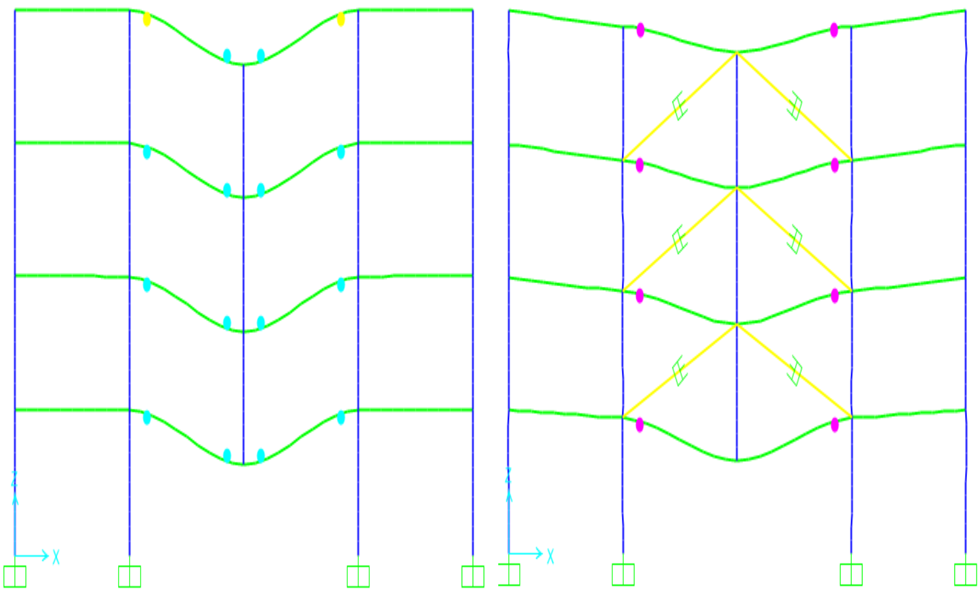
(a) without dampers (b) with dampers

Figure 4.42: Hinges at collapse for UFC loading case-3



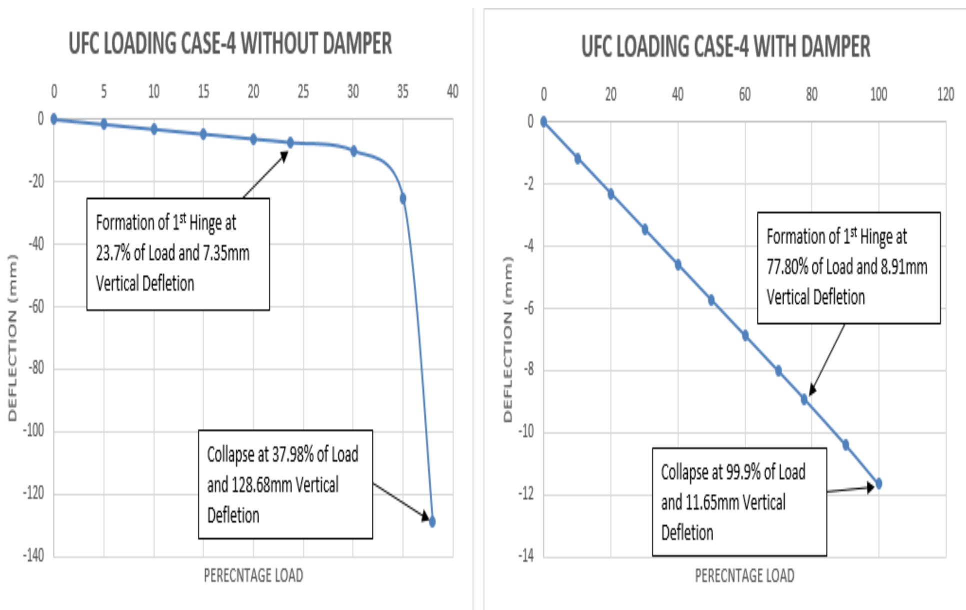
(a) without dampers (b) with dampers

Figure 4.43: Percentage Load v/s Vertical Deflection for UFC loading case-3



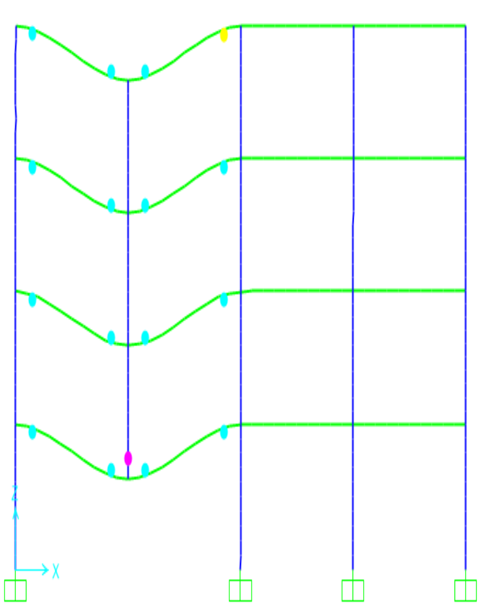
(a) without dampers (b) with dampers

Figure 4.44: Hinges at collapse for UFC loading case-4

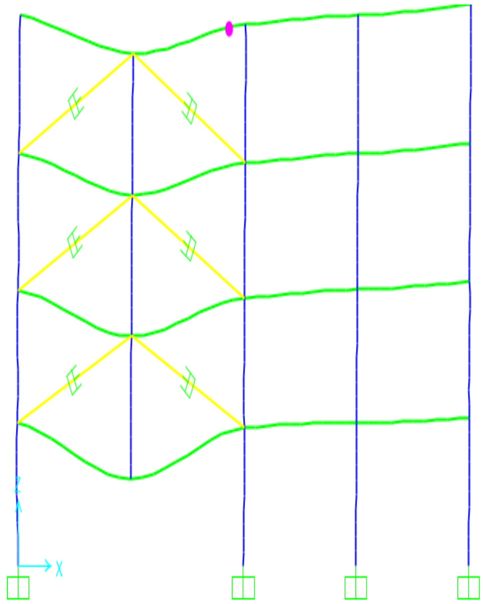


(a) without dampers (b) with dampers

Figure 4.45: Percentage Load v/s Vertical Deflection for UFC loading case-4

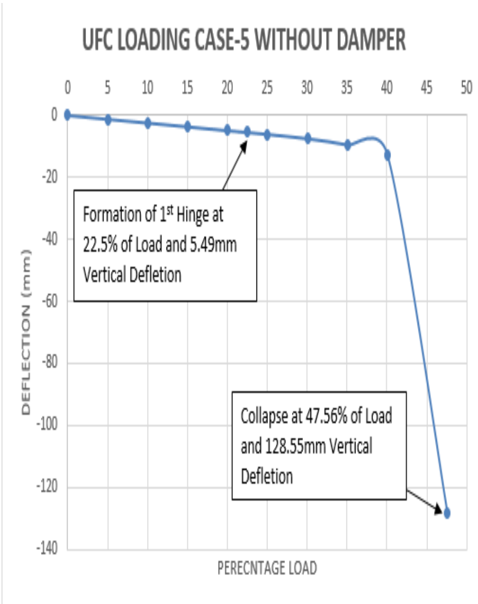


(a) without dampers

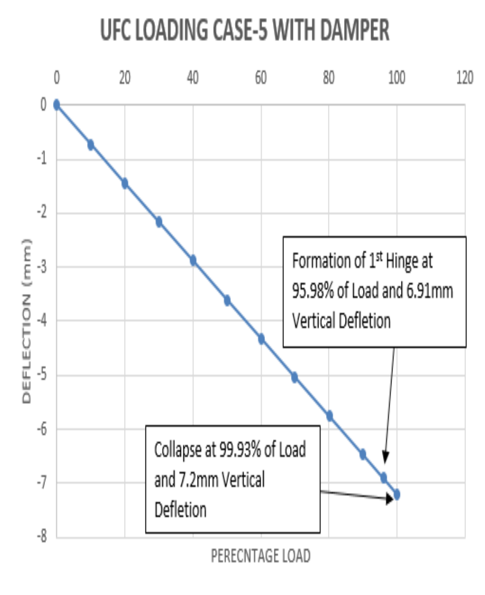


(b) with dampers

Figure 4.46: Hinges at collapse for UFC loading case-5



(a) without dampers



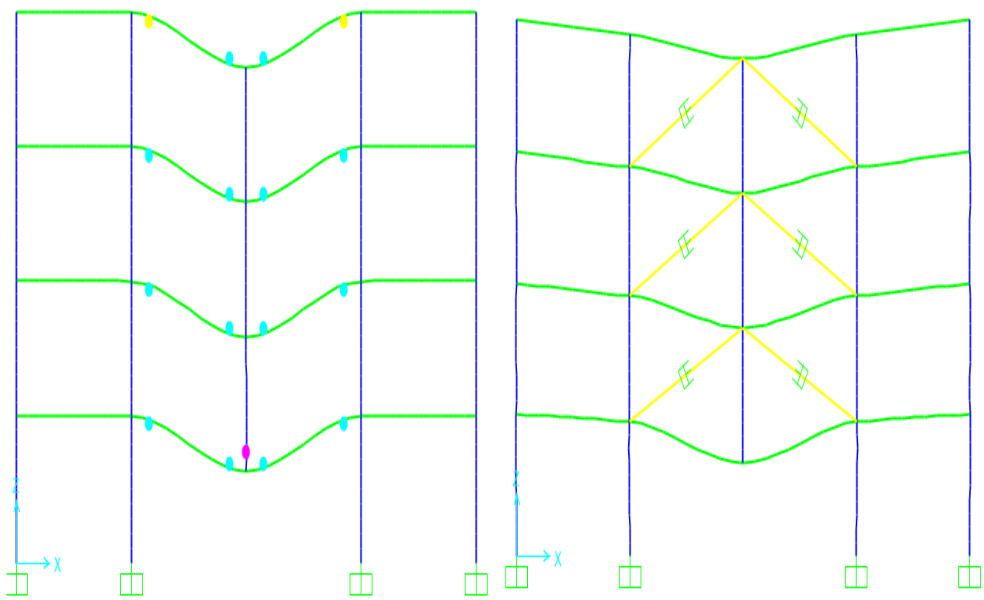
(b) with dampers

Figure 4.47: Percentage Load v/s Vertical Deflection for UFC loading case-5

Table 4.3 shows comparison of load carrying capacity of frame with and without dampers for UFC guidelines under five different column removal cases.

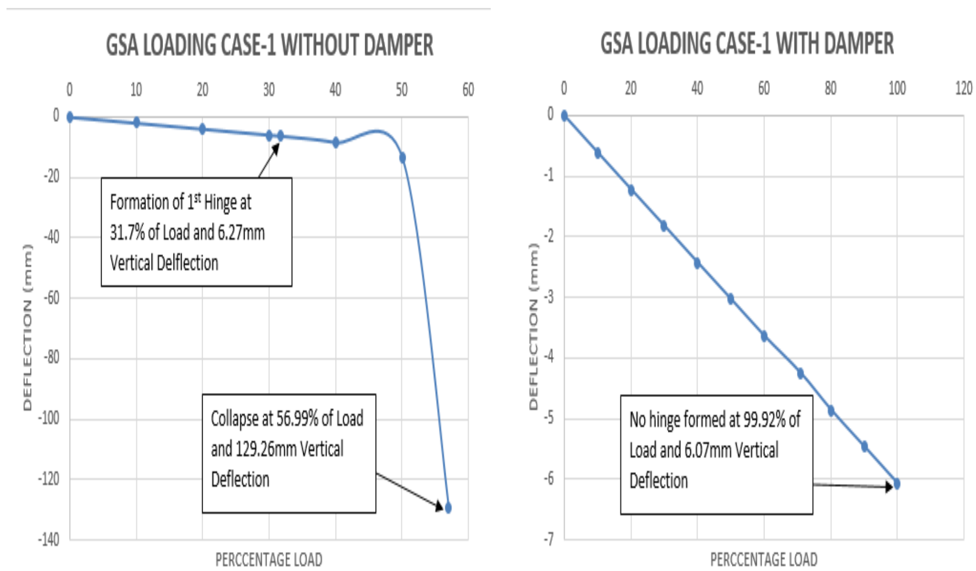
Table 4.3: Comparison of Load carrying capacity of frame UFC loading

Loading & Column Removal Case	**Percentage Load corresponding to formation of 1st Hinge**			**Percentage Collapse Load**	**Percentage Load at End of Analysis**	
	Without Damper	**With Damper**	**% Difference**	**Without Damper**	**With Damper**	**% Difference**
UFC Guidelines Case-1	25.70 %	95.73 %	**+70.03 %**	46.12 %	99.98 %	**+53.86 %**
UFC Guidelines Case-2	25.70 %	95.73 %	**+70.03 %**	46.12 %	99.98 %	**+53.86 %**
UFC Guidelines Case-3	23.10 %	79.40 %	**+56.30 %**	38.70 %	99.92 %	**+61.22 %**
UFC Guidelines Case-4	23.70 %	77.80 %	**+51.10 %**	37.98 %	99.90 %	**+61.92 %**
UFC Guidelines Case-5	22.50 %	95.98 %	**+73.48 %**	47.56 %	99.93 %	**+52.37 %**



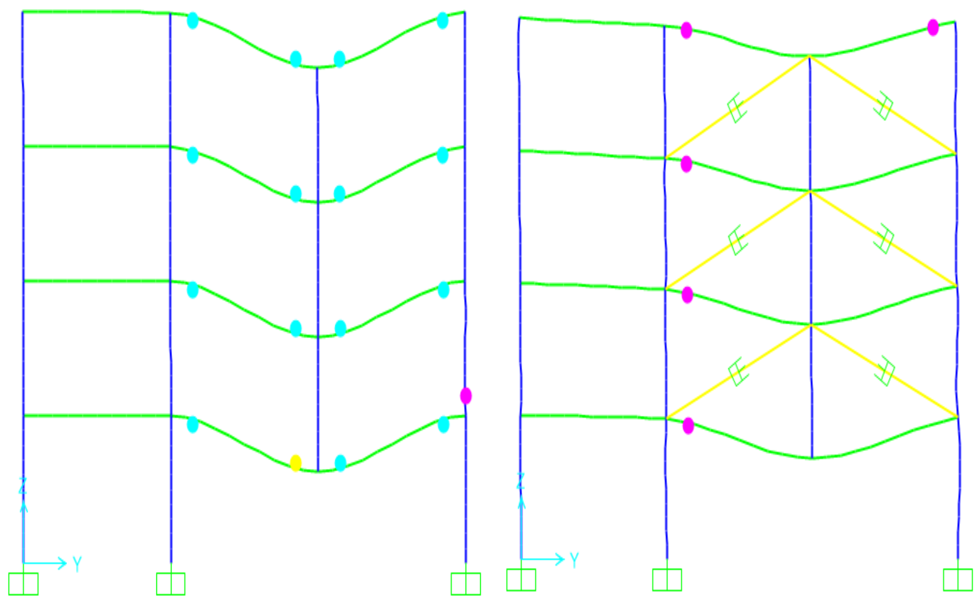
(a) without dampers (b) with dampers

Figure 4.48: Hinges at collapse for GSA loading case-1



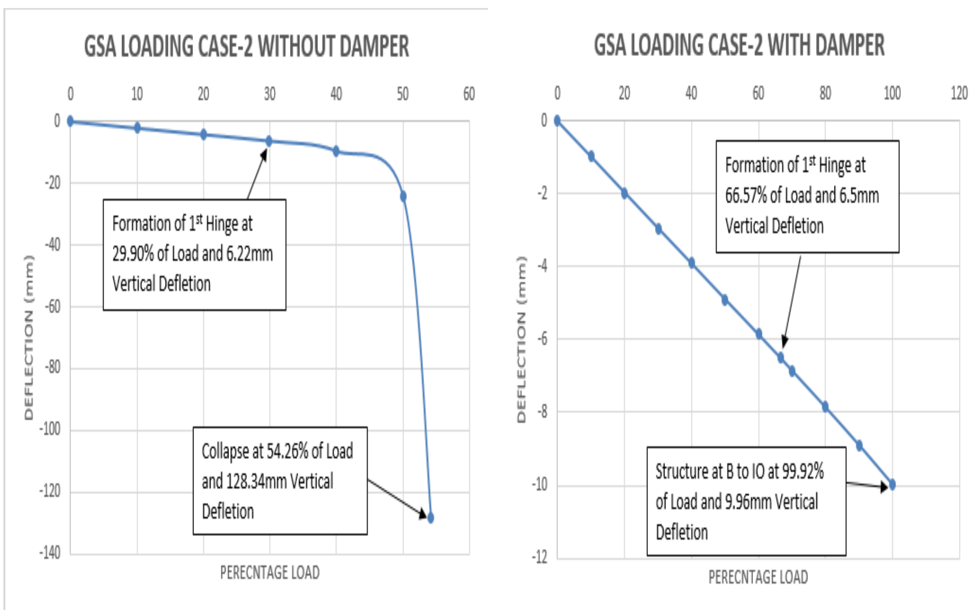
(a) without dampers (b) with dampers

Figure 4.49: Percentage Load v/s Vertical Deflection for GSA loading case-1



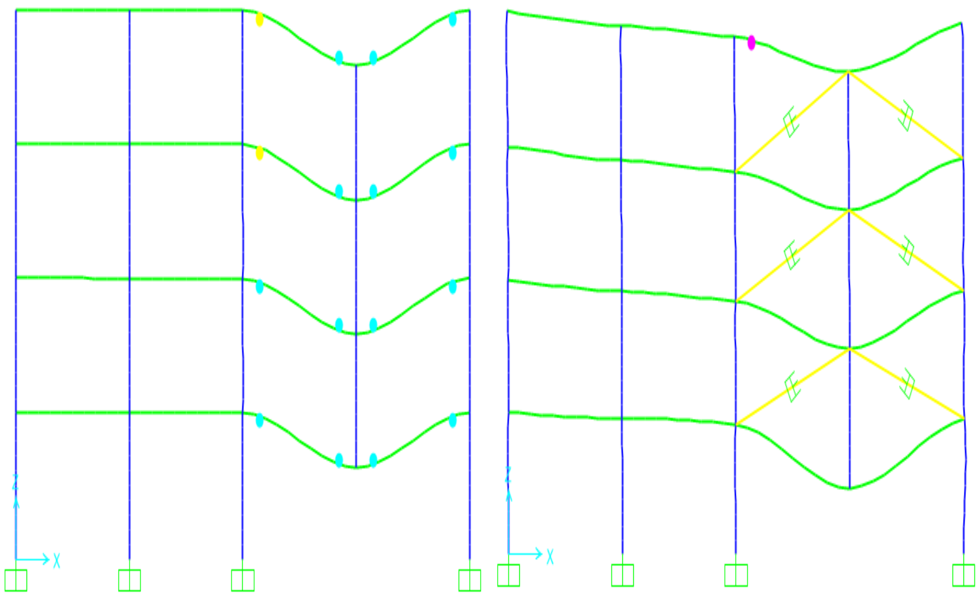
(a) without dampers (b) with dampers

Figure 4.50: Hinges at collapse for GSA loading case-2



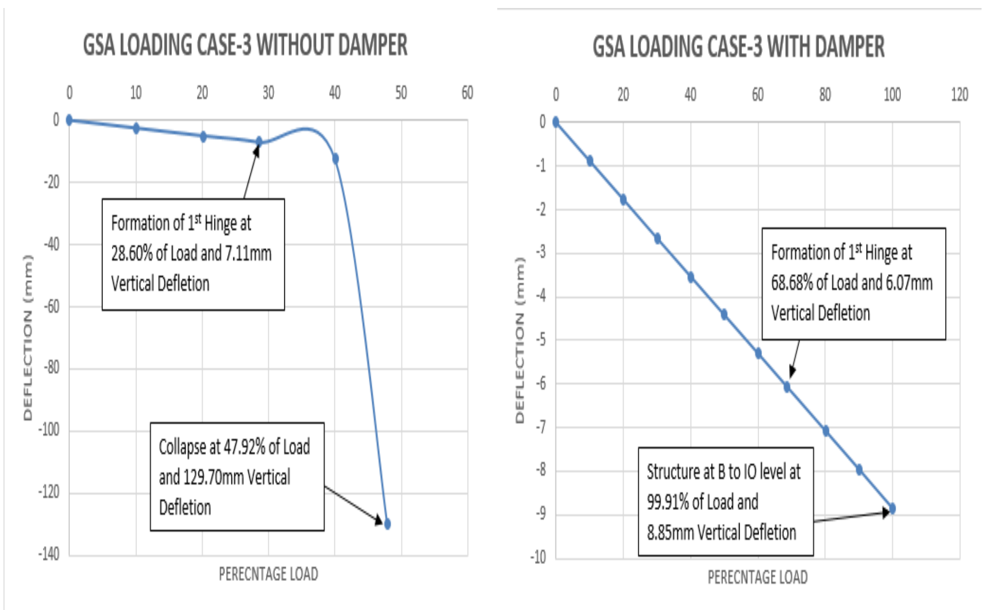
(a) without dampers (b) with dampers

Figure 4.51: Percentage Load v/s Vertical Deflection for GSA loading case-2



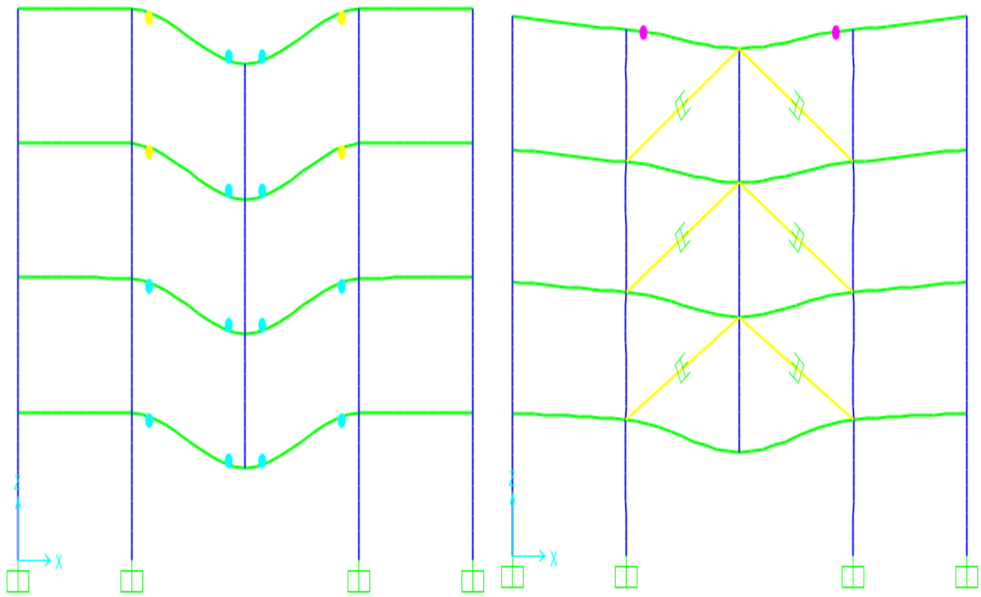
(a) without dampers (b) with dampers

Figure 4.52: Hinges at collapse for GSA loading case-3



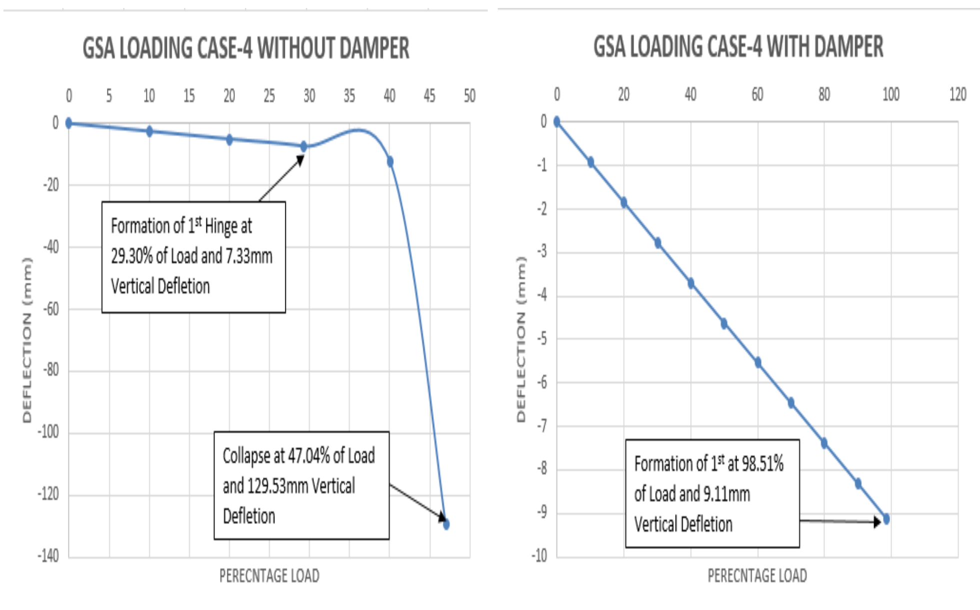
(a) without dampers (b) with dampers

Figure 4.53: Percentage Load v/s Vertical Deflection for GSA loading case-3



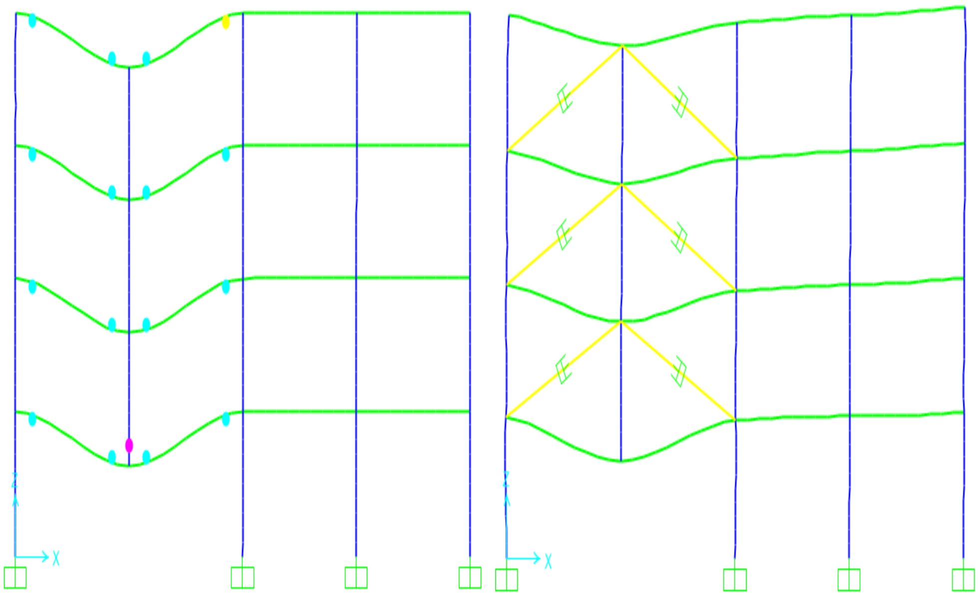
(a) without dampers (b) with dampers

Figure 4.54: Hinges at collapse for GSA loading case-4



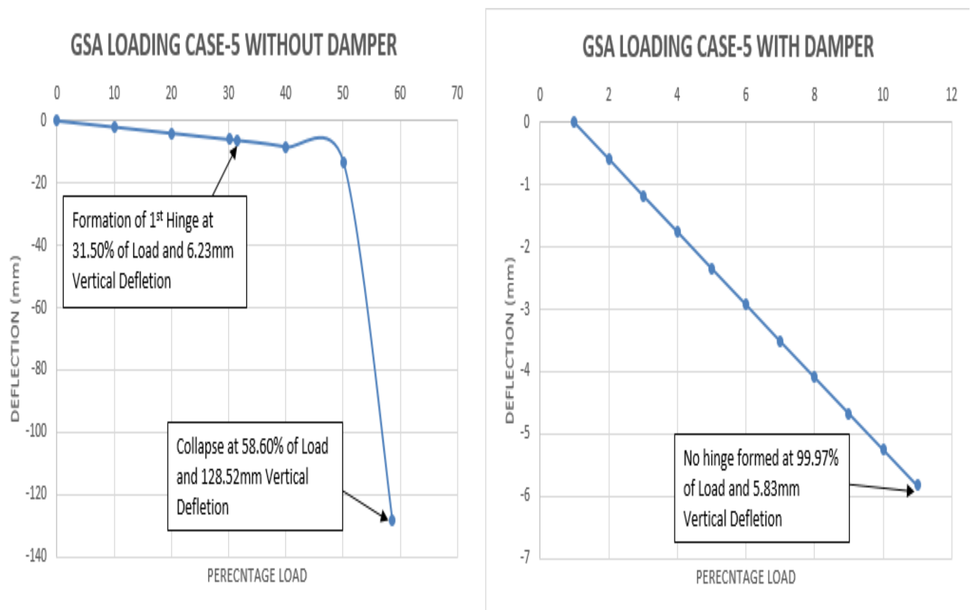
(a) without dampers (b) with dampers

Figure 4.55: Percentage Load v/s Vertical Deflection for GSA loading case-4



(a) without dampers (b) with dampers

Figure 4.56: Hinges at collapse for GSA loading case-5



(a) without dampers (b) with dampers

Figure 4.57: Percentage Load v/s Vertical Deflection for GSA loading case-5

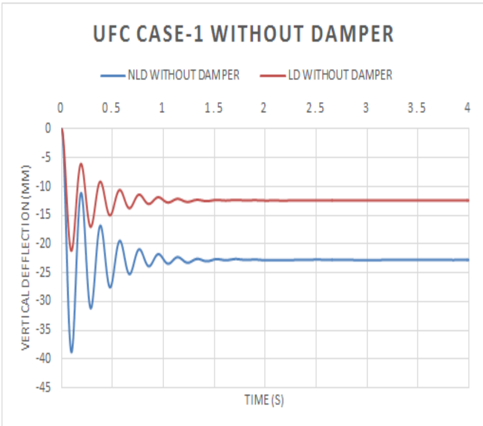
Table 4.4 shows comparison of load carrying capacity of frame with and without dampers for GSA guidelines under five different column removal cases.

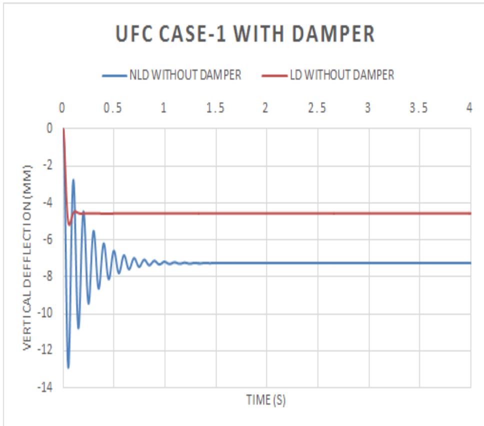
Table 4.4: Comparison of Load carrying capacity of frame GSA loading

Loading & Column Removal Case	**Percentage Load corresponding to formation of 1st Hinge**			**Percentage Collapse Load**	**Percentage Load at End of Analysis**	
	Without Damper	**With Damper**	**% Difference**	**Without Damper**	**With Damper**	**% Difference**
GSA Guidelines Case-1	31.70 %	No hinge Formed	**-**	56.99 %	No hinge Formed	**-**
GSA Guidelines Case-2	29.90 %	65.57 %	**+35.67 %**	54.26 %	99.92 %	**+45.66 %**
GSA Guidelines Case-3	28.60 %	68.68 %	**+40.08 %**	47.92 %	99.91 %	**+51.99 %**
GSA Guidelines Case-4	29.30 %	No hinge Formed	**-**	47.04 %	98.51 %	**+51.47 %**
GSA Guidelines Case-5	31.50 %	No hinge Formed	-	58.60 %	99.97 %	41.37 %

Comparison of Nonlinear Dynamic Analysis Result

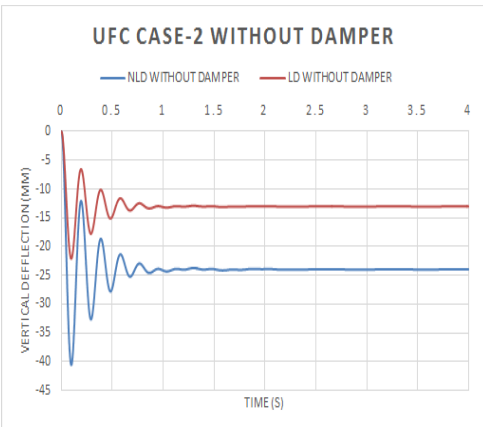
Nonlinear dynamic analysis is performed to obtain displacement at location of removed columns. Displacement at the location of column removal is maximum in case of frame without dampers and it is observed that after incorporating viscoelastic damper with suitable damping there is considerable reduction in the displacement at the location of removed column. Nonlinear dynamic and linear dynamic analysis result are plotted for the vertical deflection under column removal and is compared to that of frame with dampers as shown in Fig. 4.58.

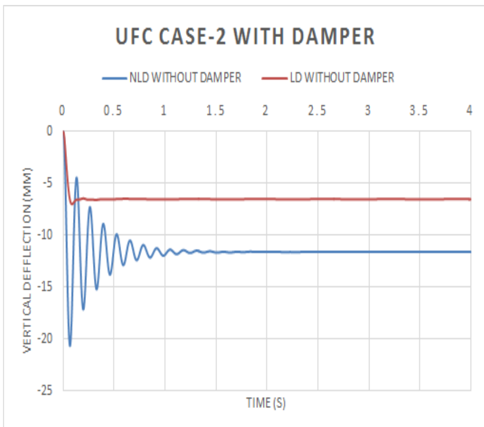




(a) Deflection for UFC loading (b) Deflection for GSA loading

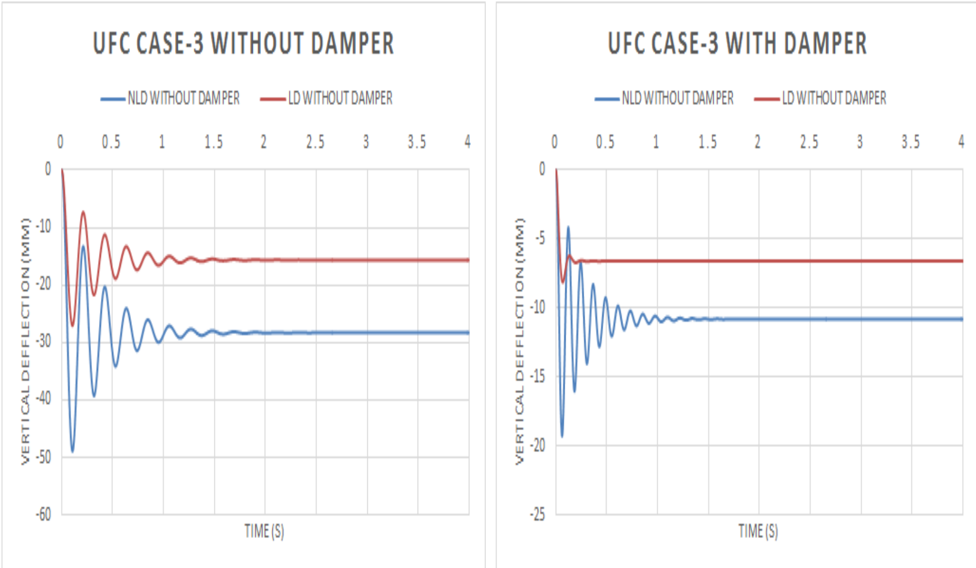
Figure 4.58: Deflection under column removal point for Case 1 UFC loading





(a) Deflection for UFC loading (b) Deflection for GSA loading

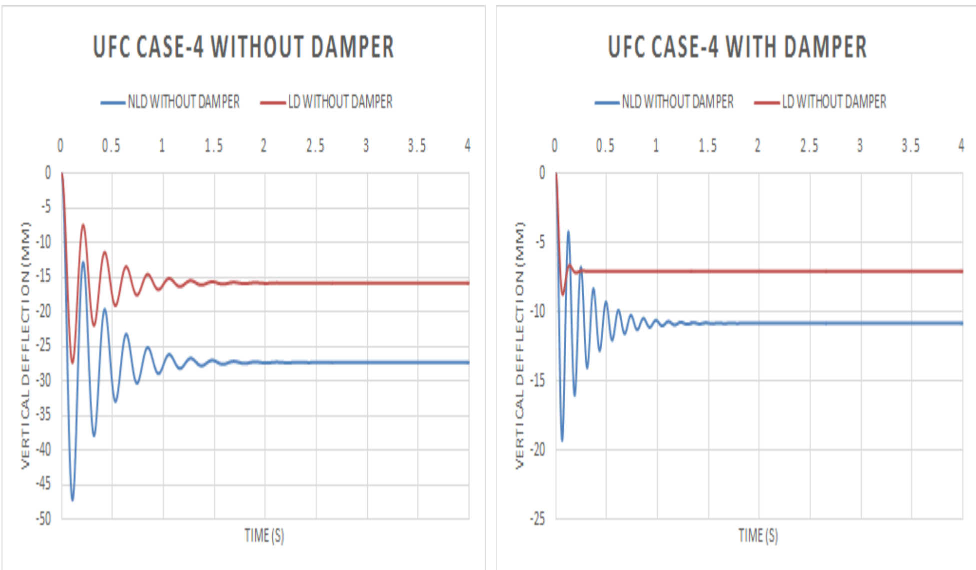
Figure 4.59: Deflection under column removal point for Case 2 UFC loading



(a) Deflection for UFC loading

(b) Deflection for GSA loading

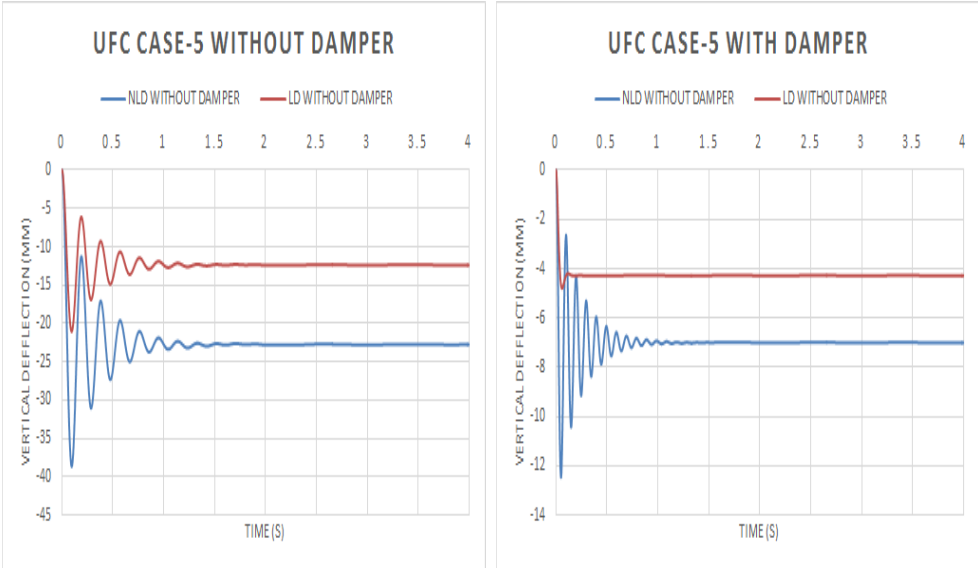
Figure 4.60: Deflection under column removal point for Case 3 UFC loading



(a) Deflection for UFC loading

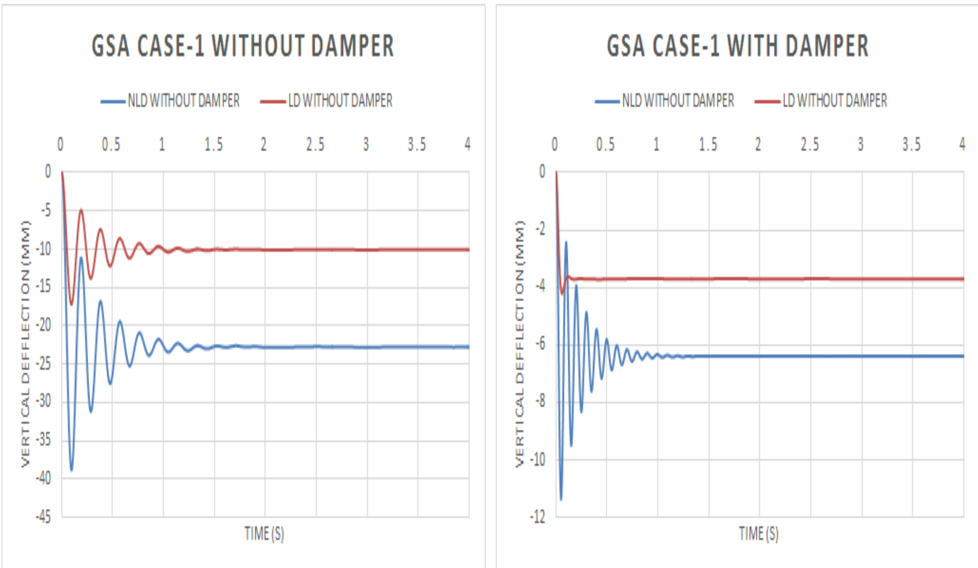
(b) Deflection for GSA loading

Figure 4.61: Deflection under column removal point for Case 4 UFC loading



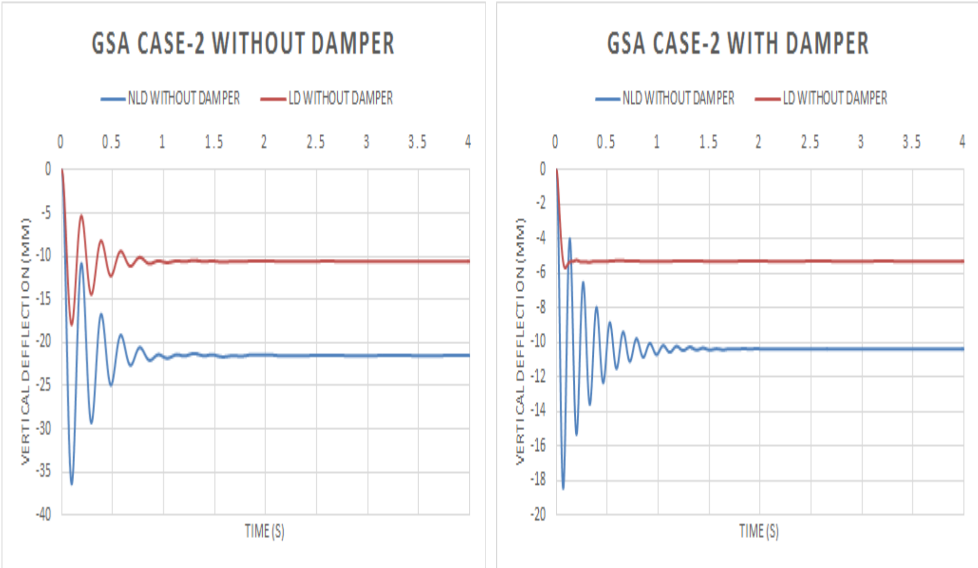
(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.62: Deflection under column removal point for Case 5 UFC loading



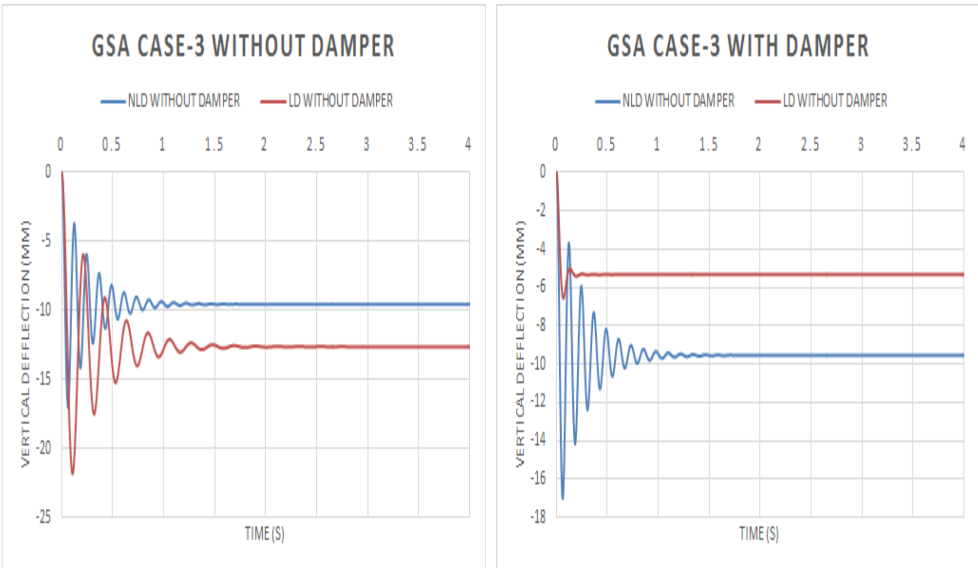
(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.63: Deflection under column removal point for Case 1 GSA loading



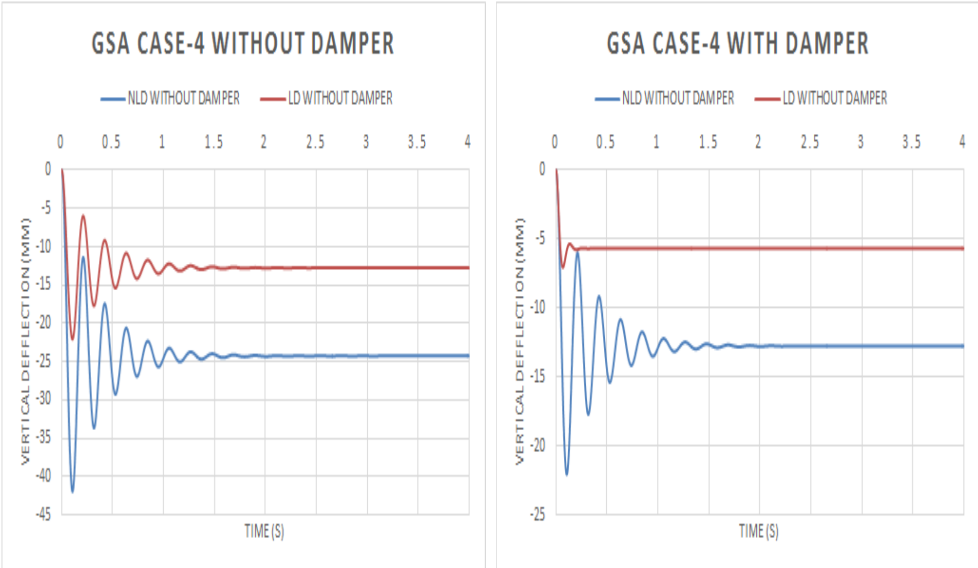
(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.64: Deflection under column removal point for Case 2 GSA loading



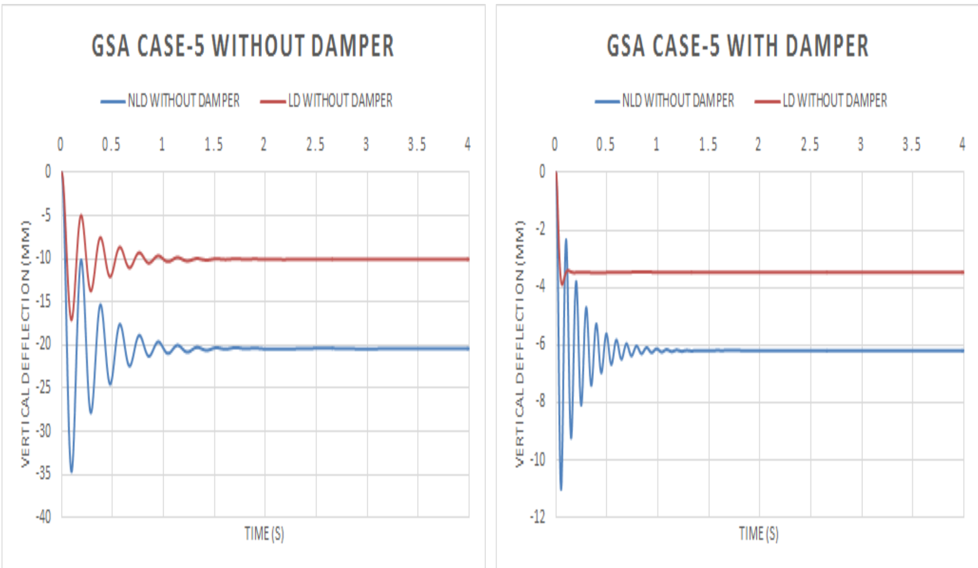
(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.65: Deflection under column removal point for Case 3 GSA loading



(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.66: Deflection under column removal point for Case 4 GSA loading



(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 4.67: Deflection under column removal point for Case 5 GSA loading

4.7 Summary

In this chapter, column is removed at five different locations and remaining structure is analyzed to evaluate the potential for progressive collapse of 4-storey reinforced concrete symmetric building using structural analysis and design software SAP2000. Linear static, Linear dynamic, Nonlinear static and Nonlinear dynamic analyses are performed by following the U.S. General Service Administration (GSA) and Department of Defense (DoD) guidelines using the alternate load path method. Demand Capacity Ratios (DCR) is calculated for flexure and shear in beams and axial force,biaxial moment in column and comparisons of results of frame without viscoelastic dampers and with damper. The DCR obtained by linear static analysis is compared for both with and without viscoelastic dampers. Vertical deflection for linear dynamic analysis and Non-linear dynamic analysis is obtained at the column removal location. For Non-linear static analysis load resistance capicity and deflection is plotted for both type of system.

Chapter 5

Progressive Collapse Analysis of 12 Storey Residential Building

5.1 General

To study the effect of failure of primary structural member on the structure, 12-storey residential building is analyzed for progressive collapse in SAP2000 software. Performance of gravity designed building and building designed for seismic and wind loading is evaluated under progressive collapse analysis. Linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis is performed to evaluate the potential for progressive collapse of gravity and seismic designed building. Guidelines given in the U.S. General Services Administration (GSA) [14] and Unified Facilities Criteria (UFC) [15], are used for progressive collapse analysis of buildings.

5.2 Building Configuration

Progressive collapse analysis of 12-storey residential building is discussed here. The building is having plan as shown in Fig. 5.1. Building is modeled in SAP2000 with the all storey height of 3.0 meter. Brick masonary of 230 mm thickness are considered on all periphery walls and 115mm on all interior walls. Building is analyzed and designed for seismic and wind loading using structural analysis and design software SAP2000. Progressive collapse potential for building is carried out for three different column lost scenarios as highlighted by a circle in Fig. 5.1.

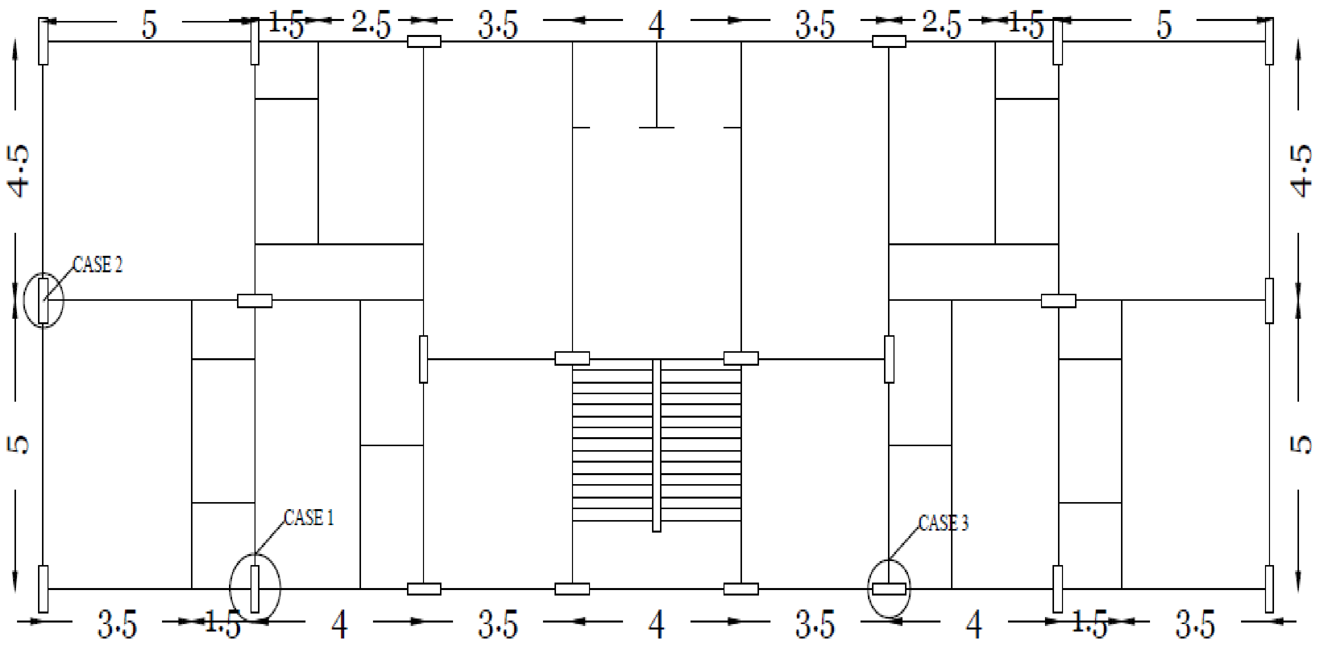


Figure 5.1: Plan of 12-storey residential building

5.3 Loading Data

12-storey residential building is analyzed and designed by considering following loading parameters and material properties. Building is analyzed and designed for seismic loading. Progressive collapse potential for building is carried out for three different column lost scenarios. Various loading data and size of the elements are as follows:

Gravity Loading Parameters:

- Dead Load - Self weight of the structural elements
- Live Load on roof = 1.5 kN/m^2
- Live Load on floors = 3.0 kN/m^2
- Floor Finish = 1.5 kN/m^2
- Wall Load on periphery beam = 13.80 kN/m^2

- Wall Load on interior beam = 6.90 kN/m^2

Seismic Loading Parameters:

- Seismic Zone - 5
- Soil type - Medium (II)
- Importance Factor - 1

Material Properties:

- Grade of Concrete - M25
- Grade of Steel - Fe415

Building Configuration:

- Slab Thickness = 150 mm
- Primary Beam Size = 350 mm × 650 mm
- Primary Beam Size = 230 mm × 400 mm
- Column Size 300 mm × 800 mm
- Wall Thickness = 230 mm & 115 mm
- Typical Storey height = 3.0 m

Seismic and wind loading design of the building is carried out for the maximum of following load combinations as suggested by IS 1893 (part 1) : 2002 [21].

- 1.5 (DL + LL)
- 1.2 (DL + LL ± EQx) and 1.2 (DL + LL ± EQy)
- 1.5 (DL ± EQx) and 1.5 (DL ± EQy)
- (0.9DL ± 1.5EQx) and (0.9DL ± 1.5EQy)
- 1.2 (DL + LL ± Wx) and 1.2 (DL + LL ± Wy)
- 1.5 (DL ± Wx) and 1.5 (DL ± Wy)
- (0.9DL ± 1.5Wx) and (0.9DL ± 1.5Wy)

5.4 Progressive Collapse Analysis

After designing of 12-storey residential building, the vertical member as shown in the Fig. 5.1 is removed separately from bottom storey level. These three cases have been considered based on exterior and interior condition given in guidelines.

5.4.1 Linear Static Analysis

Progressive collapse analysis using linear static method is been performed for all the three case of column failure. Demand capacity ratio is calculated for flexure and shear at critical locations for the building designed for seismic load and gravity load. With the help of calculated demand and capacity, check for the DCR in each structural member is carried out. Procedure to calculate DCR is illustrated in chapter 3. Various considerations regarding calculation of DCR and acceptance criteria of DCR suggested by guidelines are presented in previous chapter.

5.4.2 Linear Dynamic Analysis

The failure of vertical members under extreme events, such as blast and impact, is a highly dynamic phenomenon. So it is necessary to carry out the response of building structure by performing dynamic analysis. Linear dynamic analysis method involves real-time removal of load carrying structural elements. Thus it is more appropriate to refer to this method of analysis as a time history analysis. The detailed procedure to perform linear dynamic analysis is discussed in 5.4.2.

5.4.3 Nonlinear Static Analysis

Nonlinear static analysis is widely used to analyze a building for a lateral load and is known as "pushover analysis". In this study vertical pushover analysis procedure is adopted to understand the behaviour of building structure. In this method loads is applied step by step until maximum load is attained or until the structure collapses. In the nonlinear static analysis for progressive collapse, structural elements are allowed to deform beyond elastic limit, hence it undergoes in to the inelastic behavior. In progressive collapse analysis, vertical pushover is applied, using normal service loading, until the maximum load or the maximum displacement is attained. Procedure to perform nonlinear static analysis is presented in 3.4.3.

5.4.4 Nonlinear Dynamic Analysis

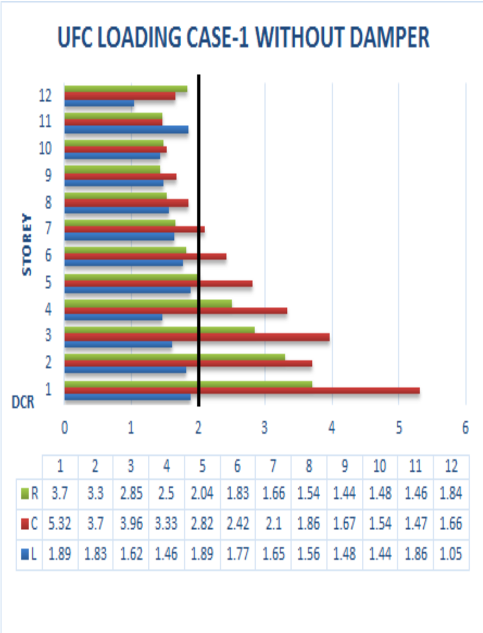
The nonlinear dynamic analysis method is the most detailed method for the progressive collapse analysis in which a primary load-bearing structural element is removed dynamically and the structural material is allowed to undergo nonlinear behavior. This allows larger deformations and energy dissipation through material yielding, cracking, and fracture. Nonlinear dynamic analysis includes both material and geometrical nonlinearities. Procedure to perform nonlinear dynamic analysis is explained in the chapter 3.

5.5 Result and Discussion

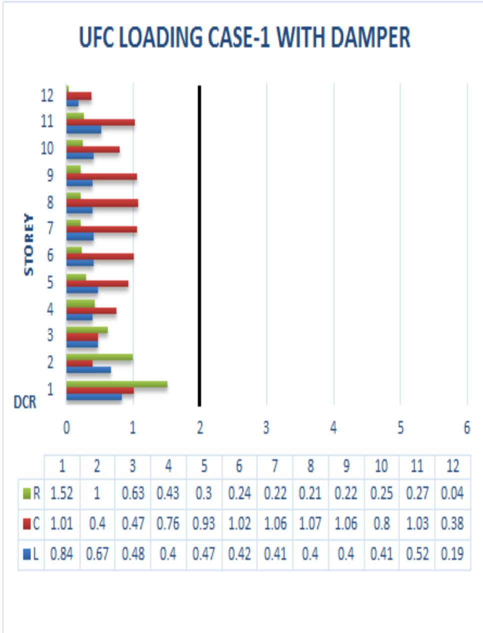
In this chapter, progressive collapse potential of 12-storey Residential building is examined in terms of Demand Capacity Ratio (DCR). The linear static analysis of 12-storey residential Reinforced concrete building has been performed by following the U.S. General Service Administration (GSA) and Department of Defense (DoD) guidelines. Demand Capacity Ratio (DCR) is found at critical locations by creating the member lost scenario for five different columns. Nonlinear static analysis is performed using SAP2000 to determine hinge formation pattern of building considered for the study. Nonlinear dynamic analysis is also performed to understand the behaviour of structure considering material and geometrical nonlinearities.

Comparison Demand Capacity Ratio (DCR) for Flexure

DCR for flexure is calculated at all the twelve stories for frame with damper and without damper and is compared as shown in Fig. 5.2 to Fig. 5.7 for both guidelines. From result it is observed that DCR in beams of buildings without dampers are exceeding the allowable limit i.e. 2 for all three column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 2, which indicates enhanced progressive collapse resistance of building.

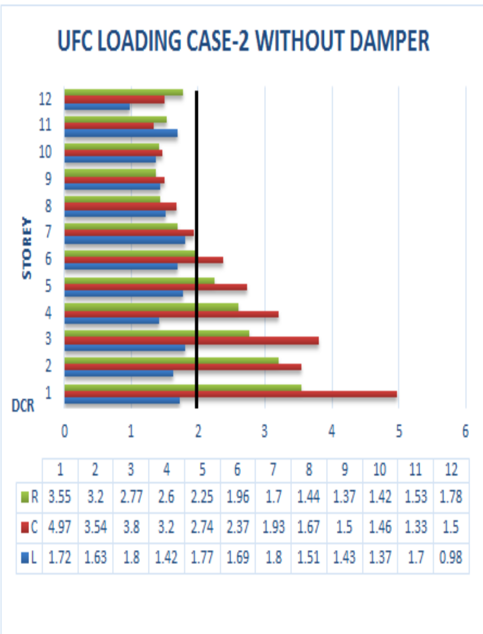


(a) DCR without damper

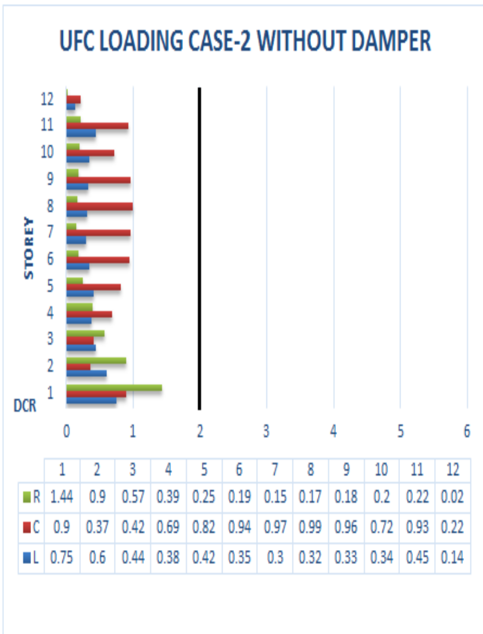


(b) DCR with damper

Figure 5.2: DCR for flexure for case 1 (UFC Loading)

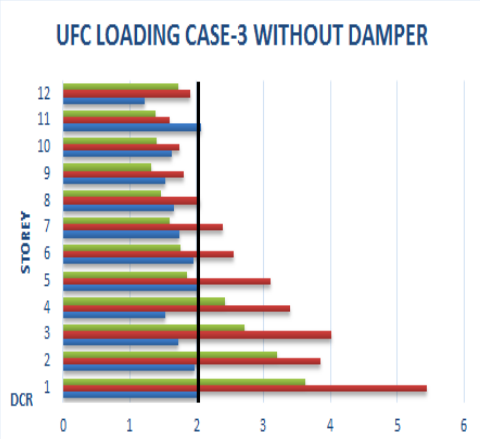


(a) DCR without damper



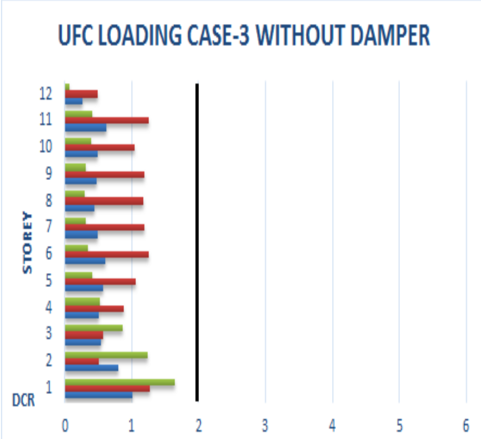
(b) DCR with damper

Figure 5.3: DCR for flexure for case 2 (UFC Loading)



	1	2	3	4	5	6	7	8	9	10	11	12
R	3.63	3.2	2.71	2.43	1.85	1.76	1.59	1.47	1.32	1.41	1.39	1.73
C	5.44	3.86	4.02	3.4	3.1	2.56	2.4	2.05	1.8	1.75	1.6	1.91
L	2	1.97	1.73	1.54	2.03	1.95	1.74	1.66	1.54	1.63	2.06	1.23

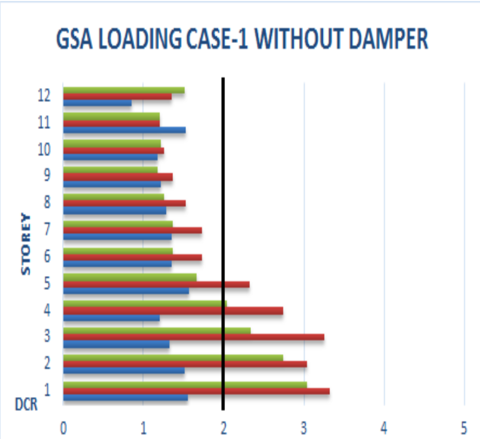
(a) DCR without damper



	1	2	3	4	5	6	7	8	9	10	11	12
R	1.64	1.24	0.87	0.52	0.41	0.34	0.31	0.3	0.31	0.4	0.42	0.065
C	1.27	0.51	0.57	0.88	1.06	1.26	1.19	1.18	1.19	1.04	1.25	0.49
L	1.02	0.8	0.54	0.51	0.57	0.6	0.49	0.45	0.47	0.5	0.63	0.26

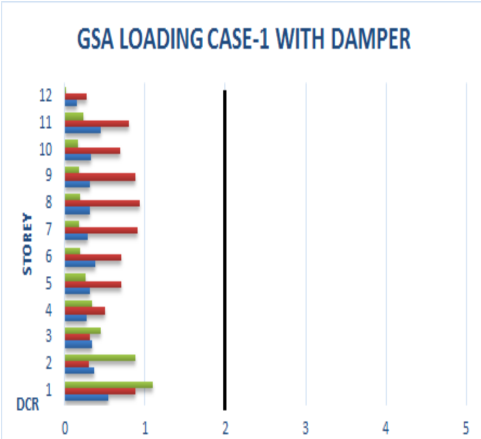
(b) DCR with damper

Figure 5.4: DCR for flexure for case 3 (UFC Loading)



	1	2	3	4	5	6	7	8	9	10	11	12
R	3.04	2.74	2.34	2.04	1.67	1.37	1.37	1.26	1.18	1.22	1.2	1.51
C	3.33	3.04	3.25	2.74	2.32	1.73	1.73	1.53	1.37	1.26	1.21	1.36
L	1.56	1.51	1.33	1.2	1.57	1.36	1.36	1.28	1.22	1.18	1.53	0.86

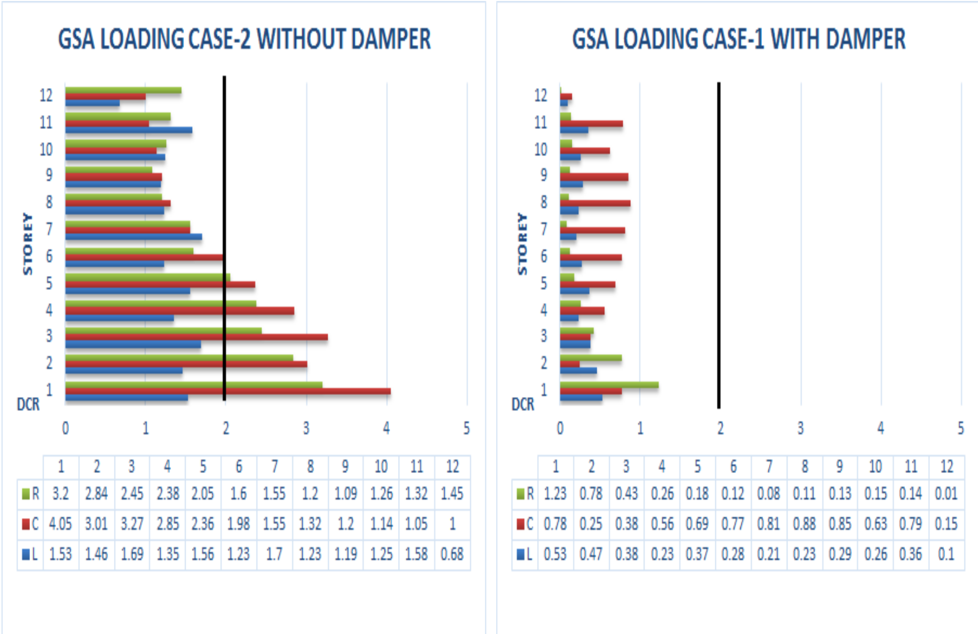
(a) DCR without damper



	1	2	3	4	5	6	7	8	9	10	11	12
R	1.1	0.88	0.45	0.34	0.26	0.19	0.18	0.2	0.18	0.17	0.23	0.02
C	0.88	0.3	0.32	0.5	0.7	0.7	0.91	0.93	0.88	0.69	0.8	0.27
L	0.54	0.37	0.34	0.27	0.32	0.38	0.29	0.31	0.32	0.33	0.45	0.15

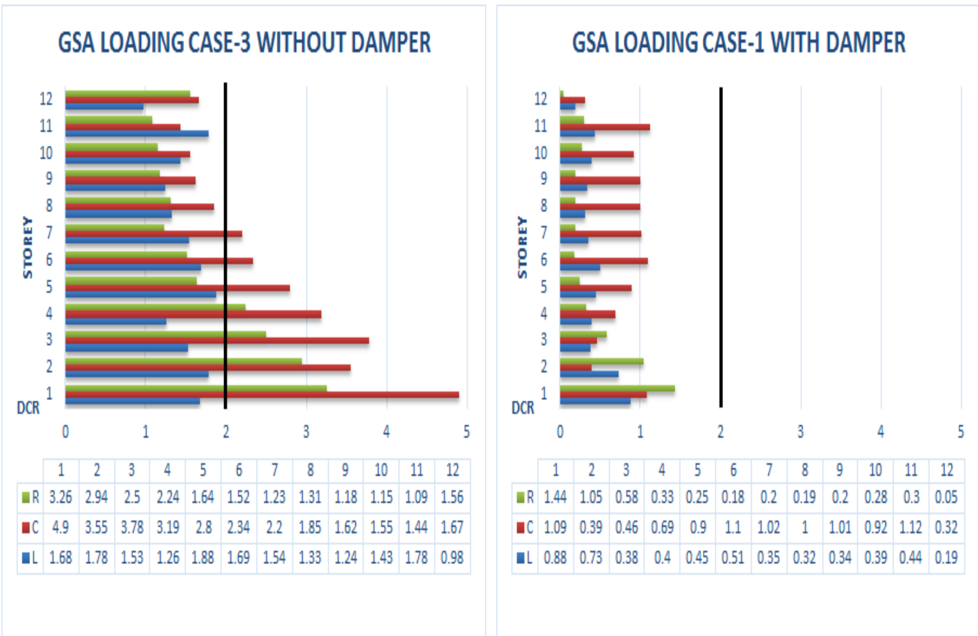
(b) DCR with damper

Figure 5.5: DCR for flexure for case 1 (GSA Loading)



(a) DCR without damper (b) DCR with damper

Figure 5.6: DCR for flexure for case 2 (GSA Loading)

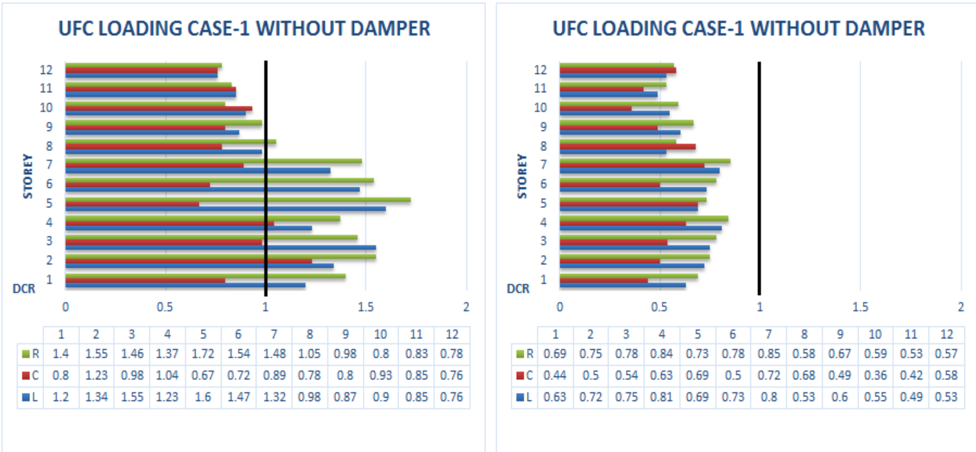


(a) DCR without damper (b) DCR with damper

Figure 5.7: DCR for flexure for case 3 (GSA Loading)

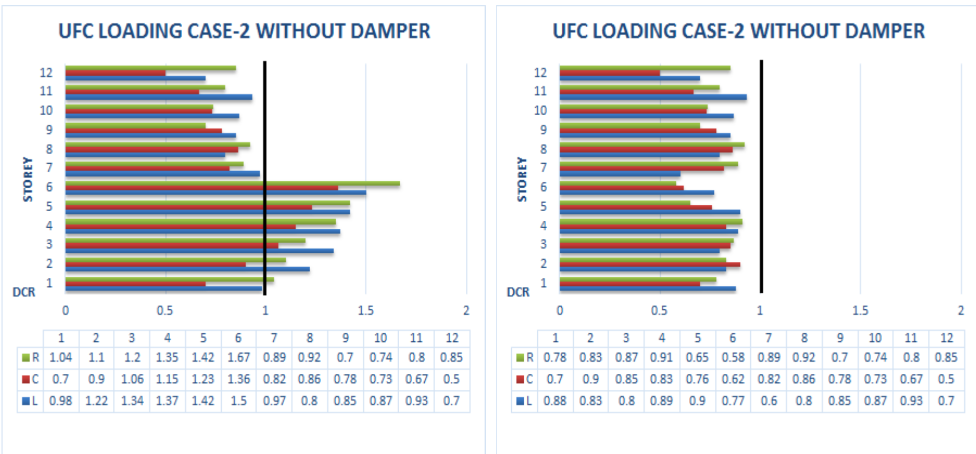
Comparison of Demand Capacity Ratio (DCR) for Shear

DCR for shear is calculated at all the twelve stories for frame with damper and without damper and is compared as shown in Fig. 5.8 to Fig. 5.13 for both guidelines. From result it is observed that DCR in beams of buildings without dampers are exceeding the allowable limit i.e. 1 for all three column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 1, which indicates enhanced progressive collapse resistance of building.



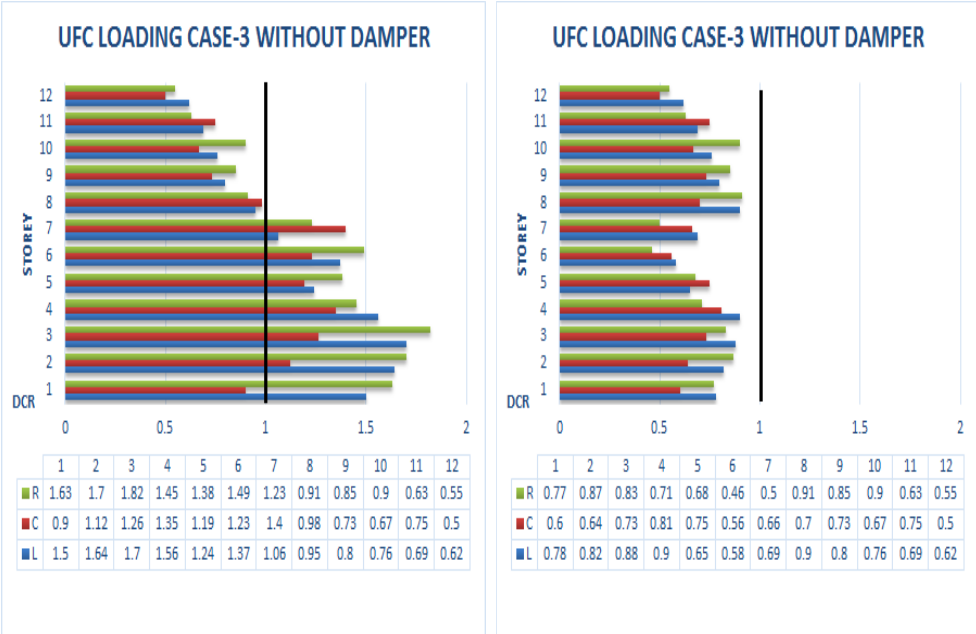
(a) DCR without damper (b) DCR with damper

Figure 5.8: DCR for shear for case 1 (UFC Loading)



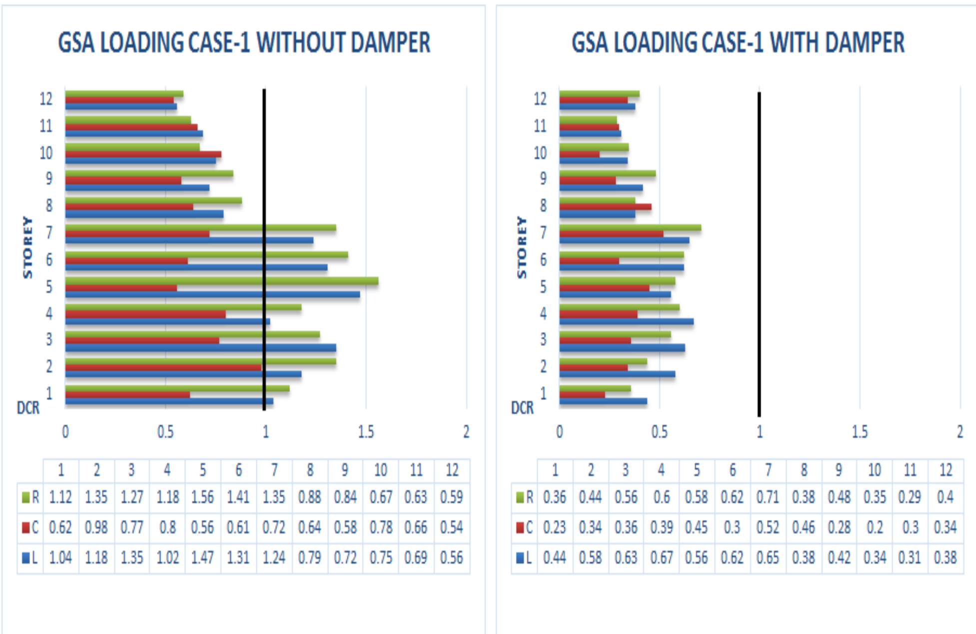
(a) DCR without damper (b) DCR with damper

Figure 5.9: DCR for shear for case 2 (UFC Loading)



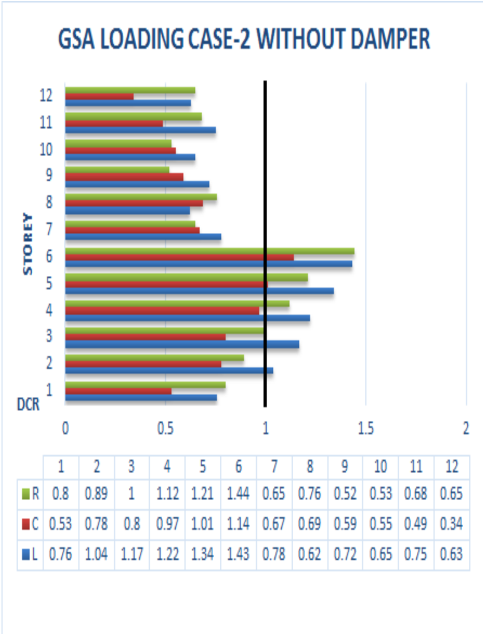
(a) DCR without damper (b) DCR with damper

Figure 5.10: DCR for shear for case 3 (UFC Loading)

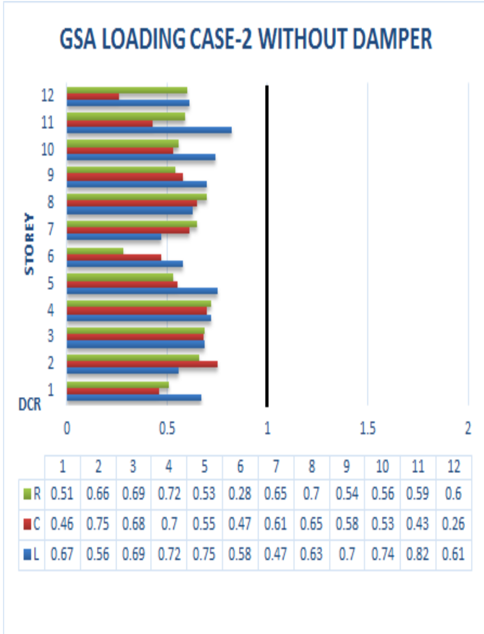


(a) DCR without damper (b) DCR with damper

Figure 5.11: DCR for shear for case 1 (GSA Loading)

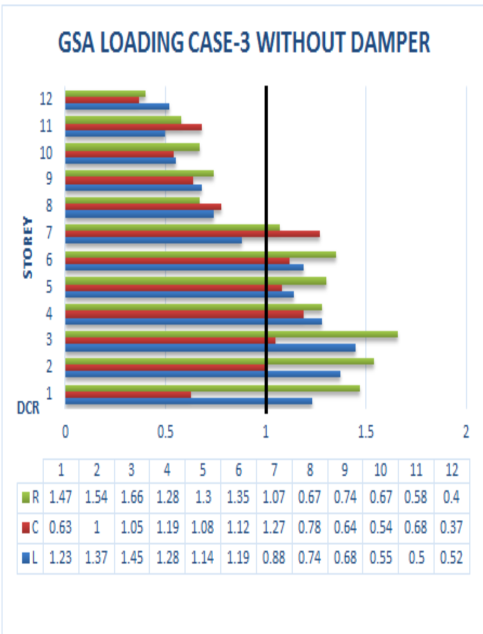


(a) DCR without damper

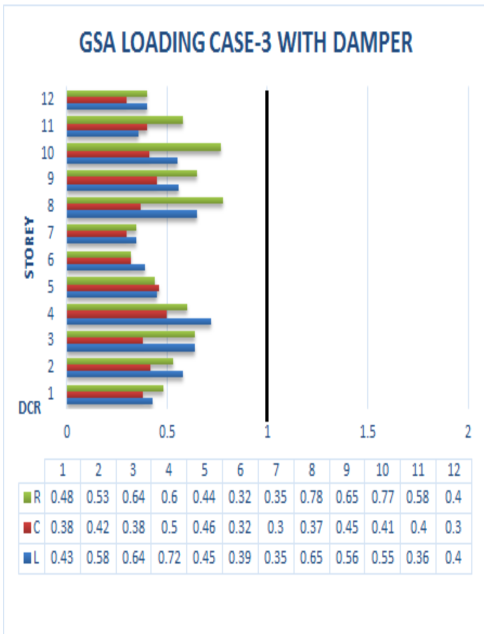


(b) DCR with damper

Figure 5.12: DCR for shear for case 2 (GSA Loading)



(a) DCR without damper

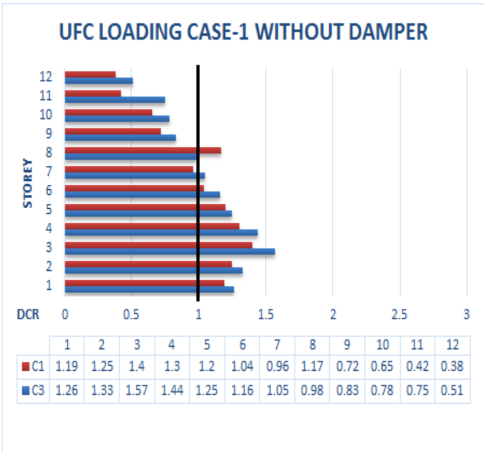


(b) DCR with damper

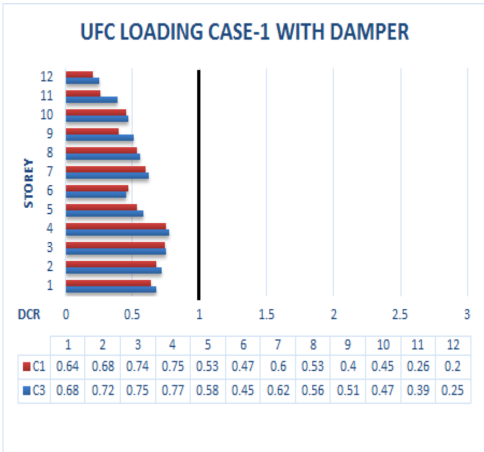
Figure 5.13: DCR for shear for case 3 (GSA Loading)

Comparison of Demand Capacity Ratio (DCR) for Column

DCR for column is calculated at all the twelve stories for frame with damper and without damper and is compared as shown in Fig. 5.14 to Fig. 5.19 for both guidelines. From result it is observed that DCR in columns of buildings without dampers are exceeding the allowable limit i.e. 1 for all three column removal scenarios, which indicates high potential of progressive collapse. When viscoelastic dampers are used, DCR for beams reduces significantly less than 1, which indicates enhanced progressive collapse resistance of building.

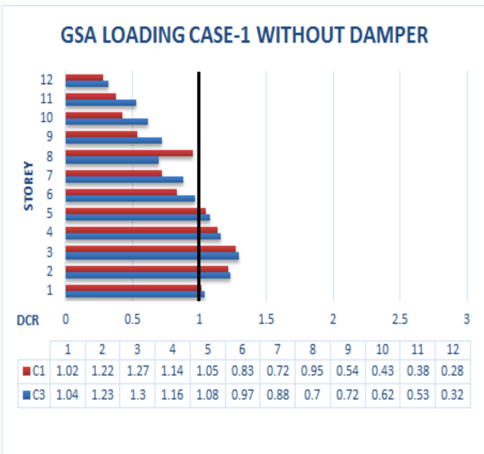


(a) DCR without damper

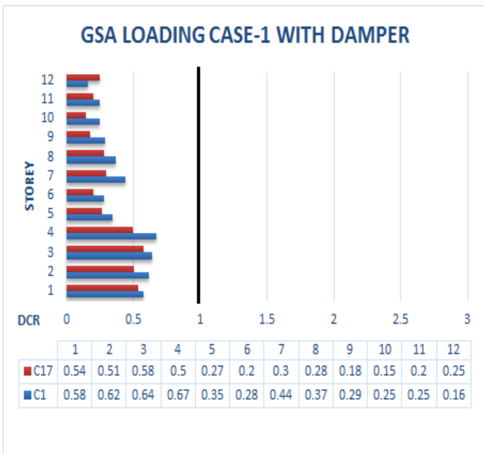


(b) DCR with damper

Figure 5.14: DCR for column for case 1 (UFC Loading)

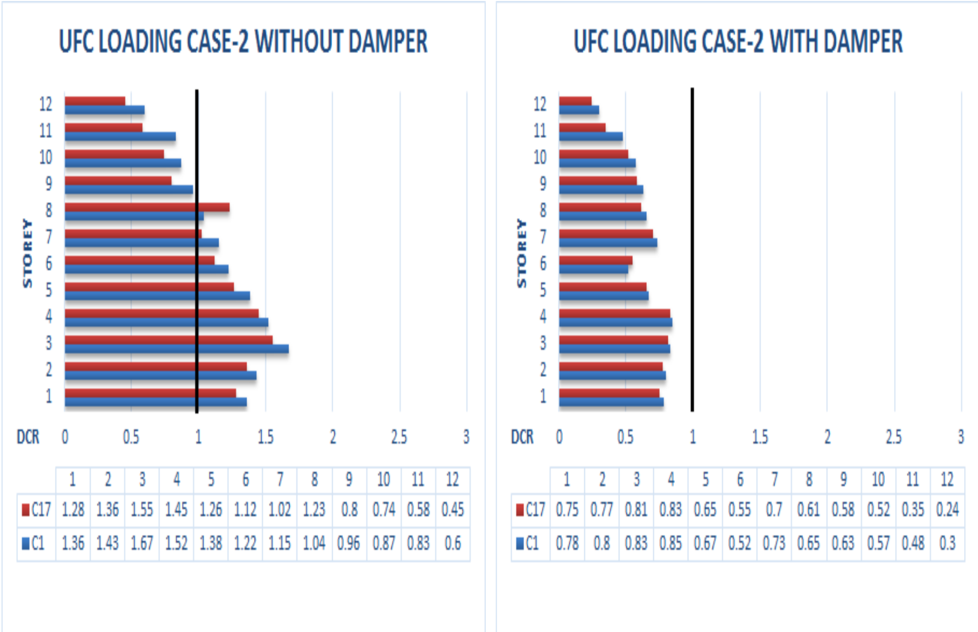


(a) DCR without damper



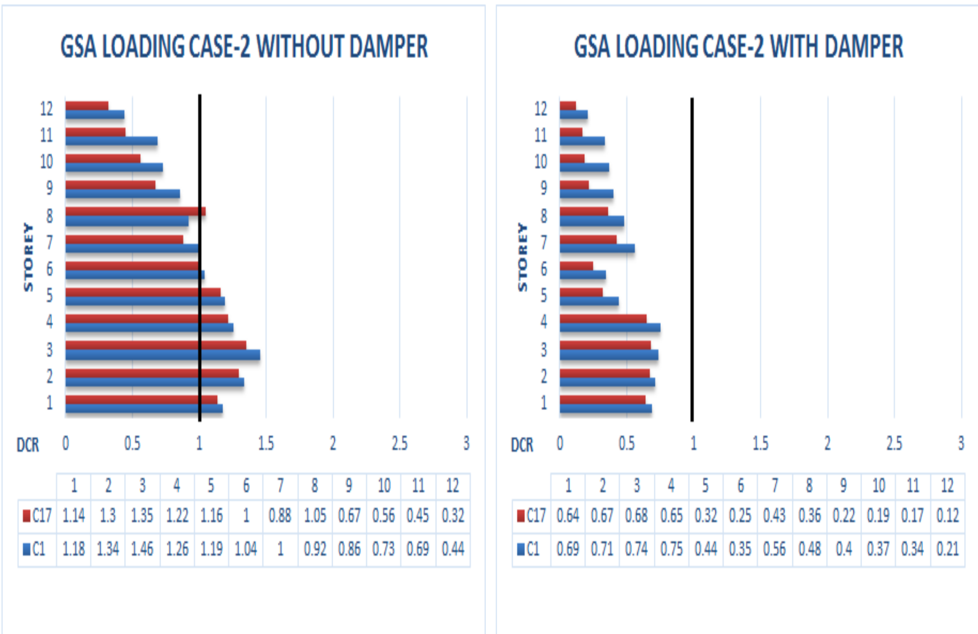
(b) DCR with damper

Figure 5.15: DCR for column for case 1 (GSA Loading)



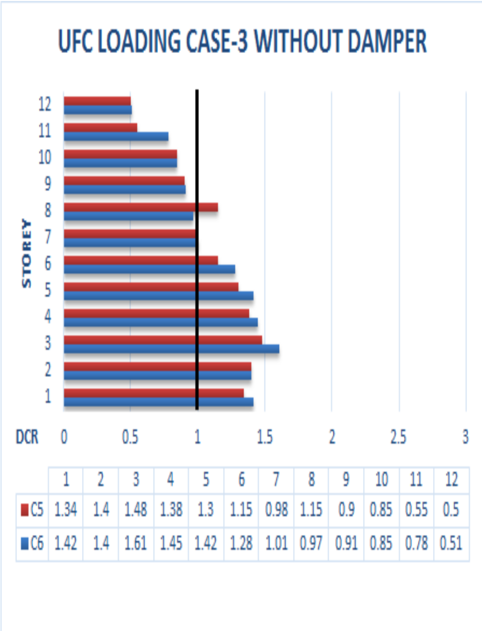
(a) DCR without damper (b) DCR with damper

Figure 5.16: DCR for column for case 2 (UFC Loading)

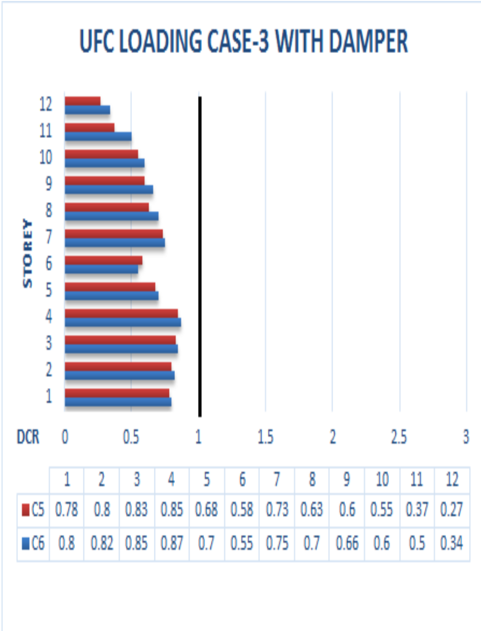


(a) DCR without damper (b) DCR with damper

Figure 5.17: DCR for column for case 2 (GSA Loading)

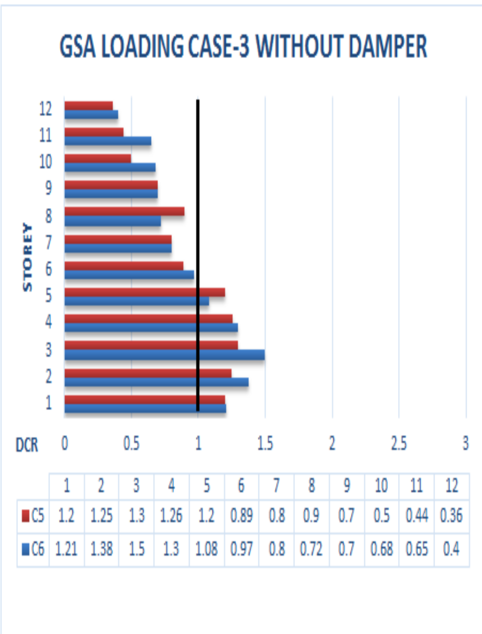


(a) DCR without damper

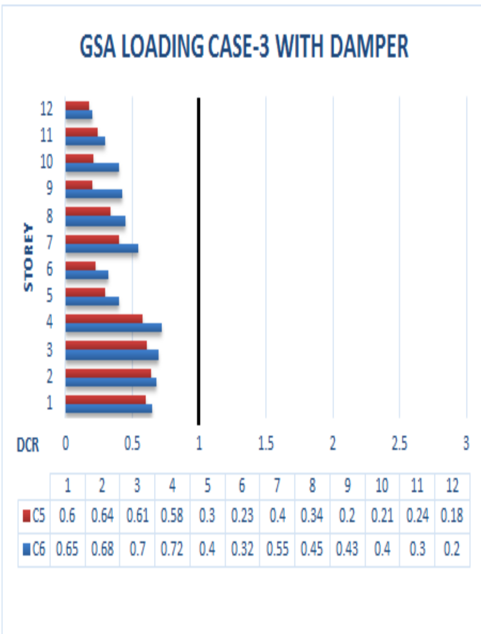


(b) DCR with damper

Figure 5.18: DCR for column for case 3 (UFC Loading)



(a) DCR without damper

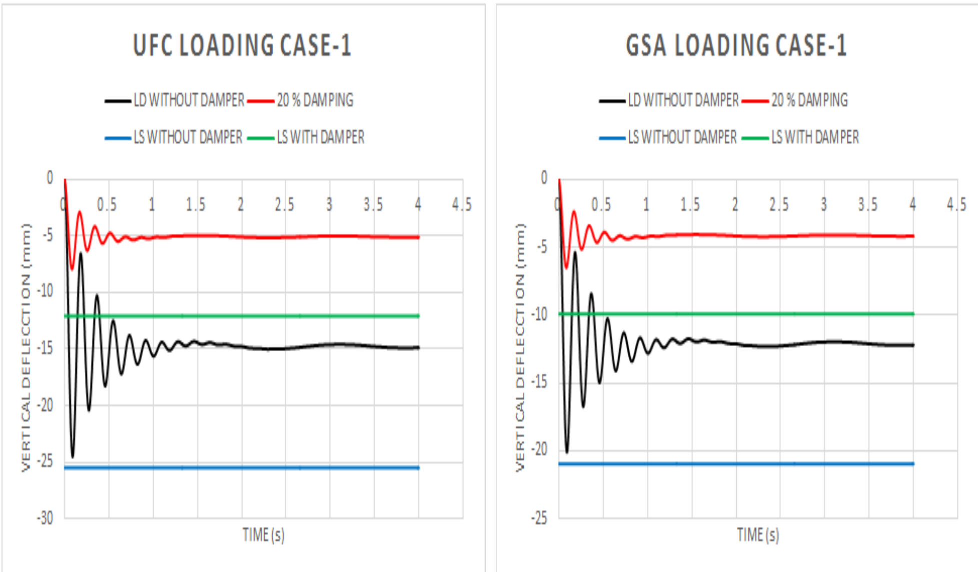


(b) DCR with damper

Figure 5.19: DCR for column for case 3 (GSA Loading)

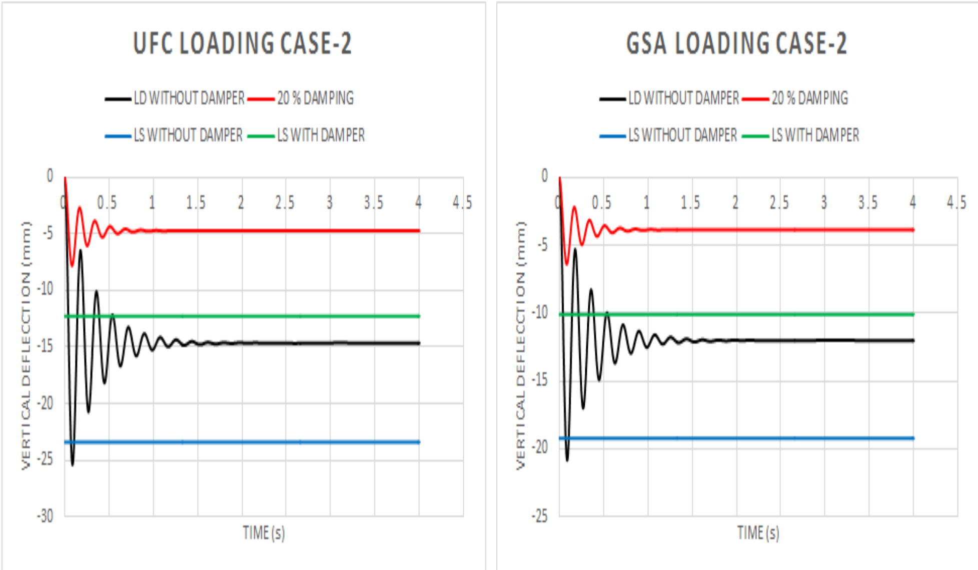
Comparison of Linear Dynamic Analysis Results

In linear dynamic analysis reaction is applied at the location of column removal and is make to zero after some elapsed time to incorporate dynamic effect. Linear dynamic analysis is performed to obtain displacement at location of removed columns. Displacement at the location of column removal is maximum in case of frame without dampers and it is observed that after incorporating viscoelastic damper with suitable damping there is considerable reduction in the displacement at the location of removed column as shown in Fig. 5.20 to Fig. 5.22 for all the three column removal cases.



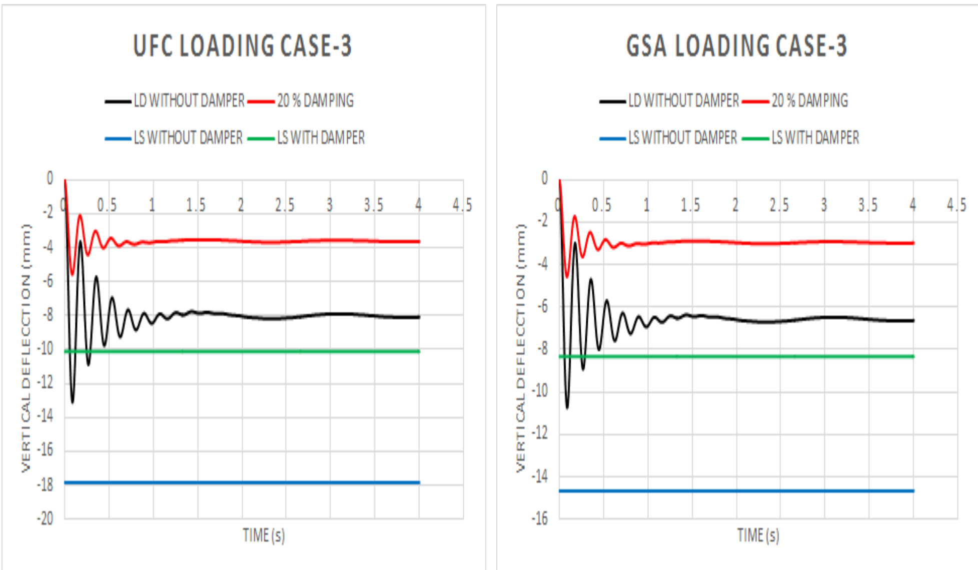
(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 5.20: Deflection under column removal point for Case 1



(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 5.21: Deflection under column removal point for Case 2



(a) Deflection for UFC loading (b) Deflection for GSA loading

Figure 5.22: Deflection under column removal point for Case 3

Here the Table 5.1 represents the vertival deflection at column removal location for both GSA and UFC guidelines. Vertical deflection is maximum for linear static analysis as compared to linear dynamic analysis. Incorporating viscoelastic damper with suitable damping, considerably reduces the displacement at the location of removed column.

Table 5.1: Comparison of Vertical deflection at column removal location

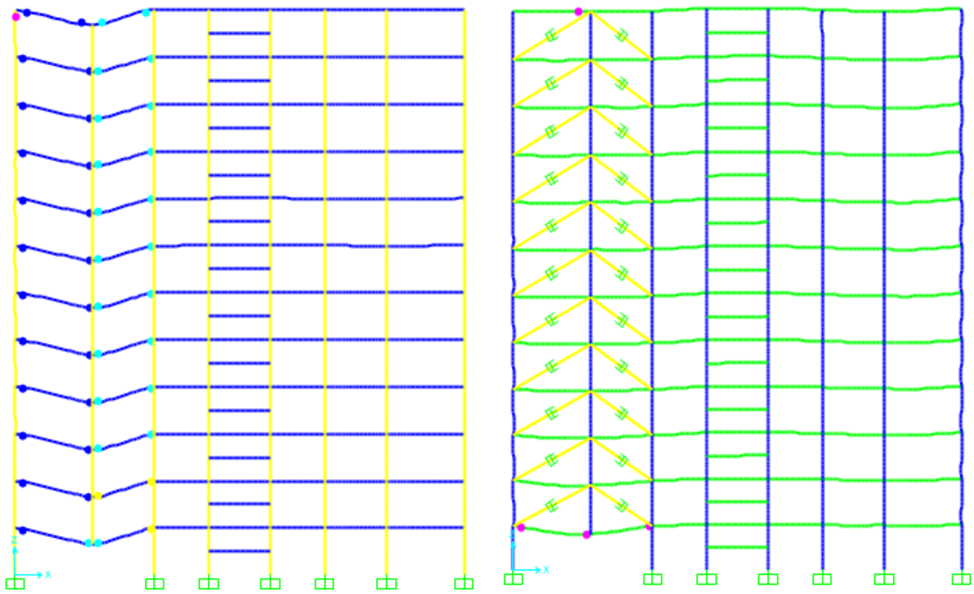
	Maximum Vertical Deflection (mm)			
Loading & Column Removal Case	Linear Static Without Damper	Linear Dynamic Without Damper	Linear Static With Damper	20 % Damping
UFC Guidelines Case-1	25.50 mm	24.51 mm	12.07 mm	7.93 mm
UFC Guidelines Case-2	23.40 mm	25.40 mm	12.27 mm	7.80 mm
UFC Guidelines Case-3	17.83 mm	13.11 mm	10.12 mm	5.58 mm
GSA Guidelines Case-1	20.95 mm	20.10 mm	9.92 mm	6.50 mm
GSA Guidelines Case-2	19.22 mm	20.83 mm	10.09 mm	6.39 mm
GSA Guidelines Case-3	14.66 mm	10.74 mm	8.33 mm	4.57 mm

Comparison of Nonlinear Static Analysis Result

In progressive collapse analysis, vertical pushover is applied, using normal service loading, until the maximum load or the maximum displacement is attained. In this method load is applied step by step until maximum load is attained or until the structure collapses. The failure of one load bearing element may likely to cause failure of other elements resulting in progressive collapse of entire structure. Therefore, nonlinear static analysis is useful for evaluation and to observe the hinge formation pattern in the building during column removal scenario.

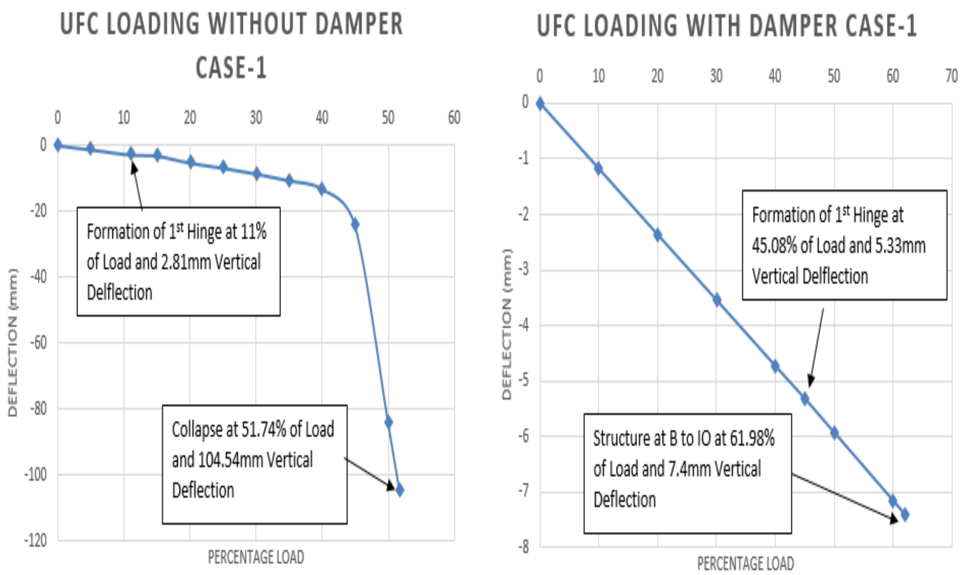
Results are observed in terms of vertical deflection under column removal scenario, formation of hinge pattern and load carrying capacity. Vertical deflection is measured at each step at an interval of 5% increase in load. A graph of vertical deflection at location of removed column corresponding to % load resisted by frame is plotted as shown in Fig. 5.24. From the results, it is observed that, viscoelastic dampers significantly increases load carrying capacity of the frame as well as drastically reduces the vertical deflection.From the hinge formation pattern, it is observed that less number of hinges are formed when viscoelastic dampers are provided, as compared to frame without dampers

Pattern of hinge formation at the collapse load for frames with and without dampers are presented. From the hinge formation pattern, it is observed that less number of hinges are formed when viscoelastic dampers are provided, as compared to frame without dampers as shown in Fig. 5.23.



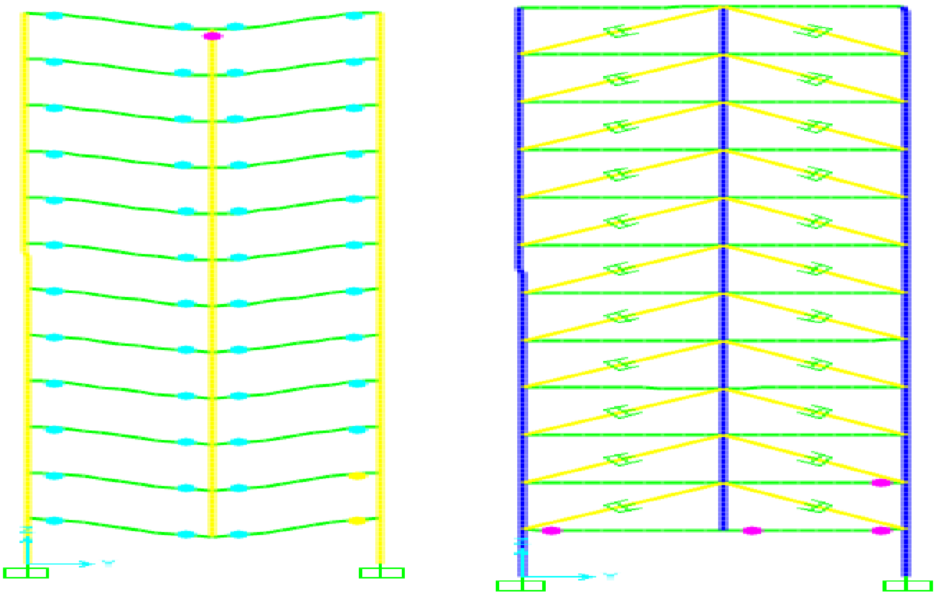
(a) without dampers (b) with dampers

Figure 5.23: Hinges at collapse for UFC loading case-1



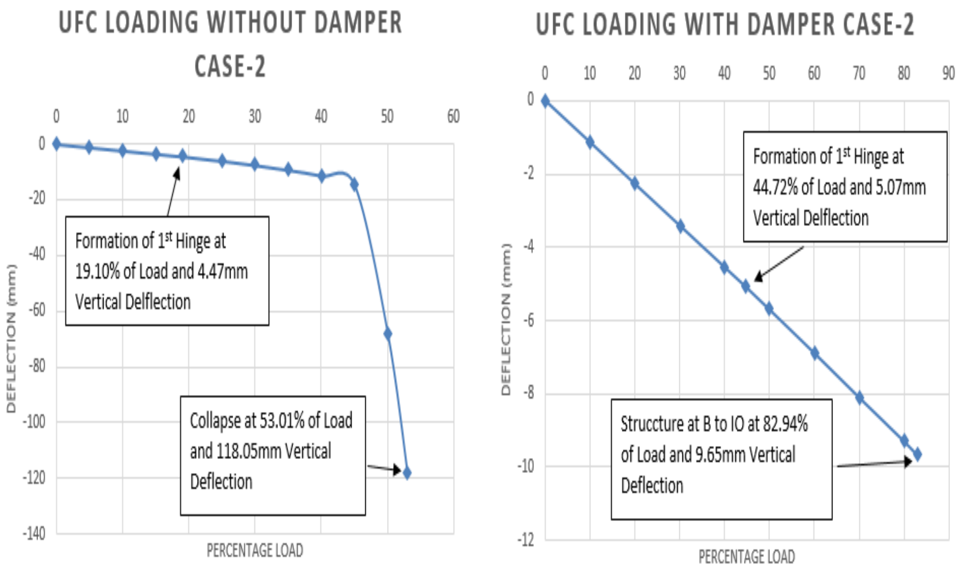
(a) without dampers (b) with dampers

Figure 5.24: Percentage Load v/s Vertical Deflection for UFC loading case-1



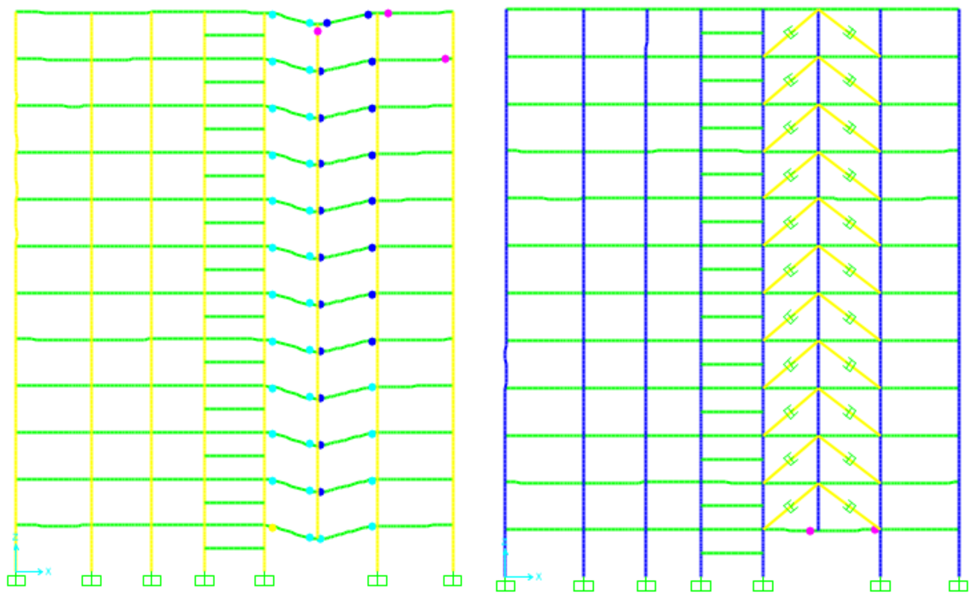
(a) without dampers (b) with dampers

Figure 5.25: Hinges at collapse for UFC loading case-2



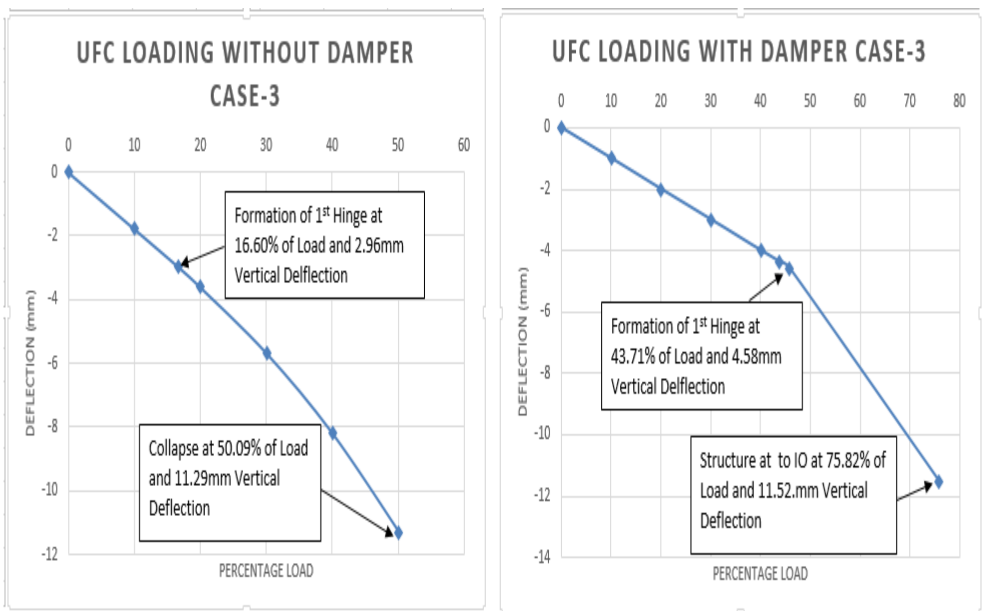
(a) without dampers (b) with dampers

Figure 5.26: Percentage Load v/s Vertical Deflection for UFC loading case-2



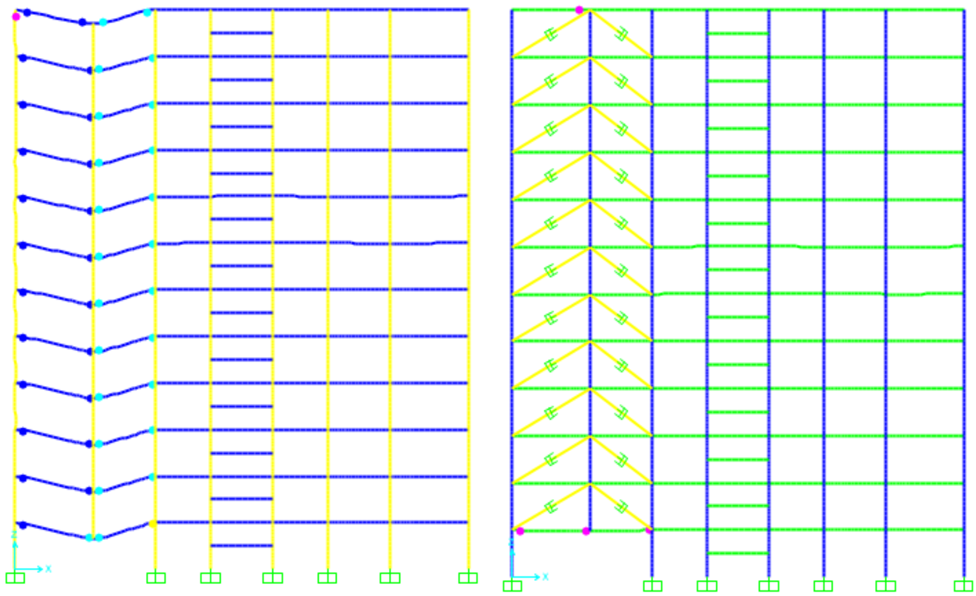
(a) without dampers (b) with dampers

Figure 5.27: Hinges at collapse for UFC loading case-3



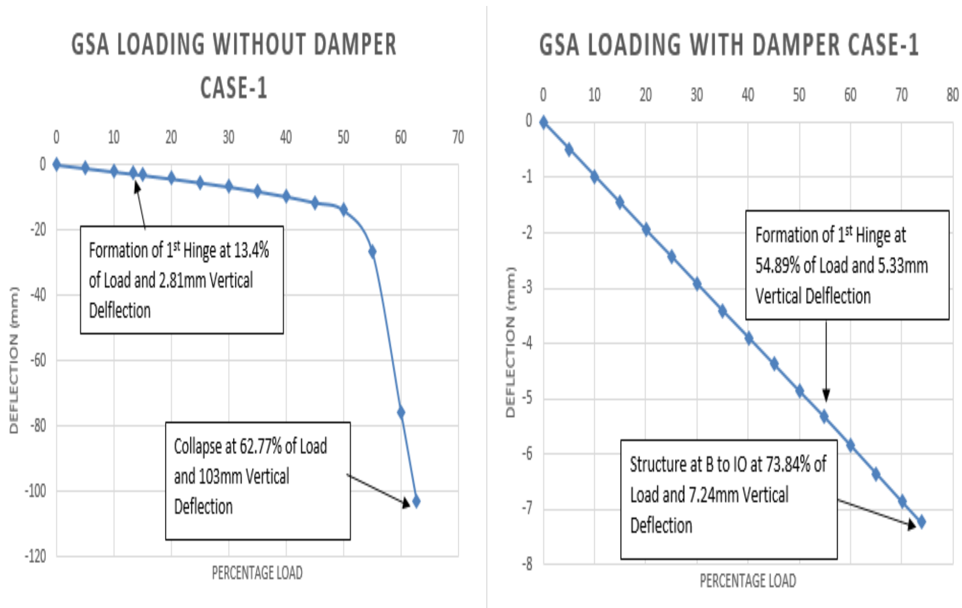
(a) without dampers (b) with dampers

Figure 5.28: Percentage Load v/s Vertical Deflection for UFC loading case-3



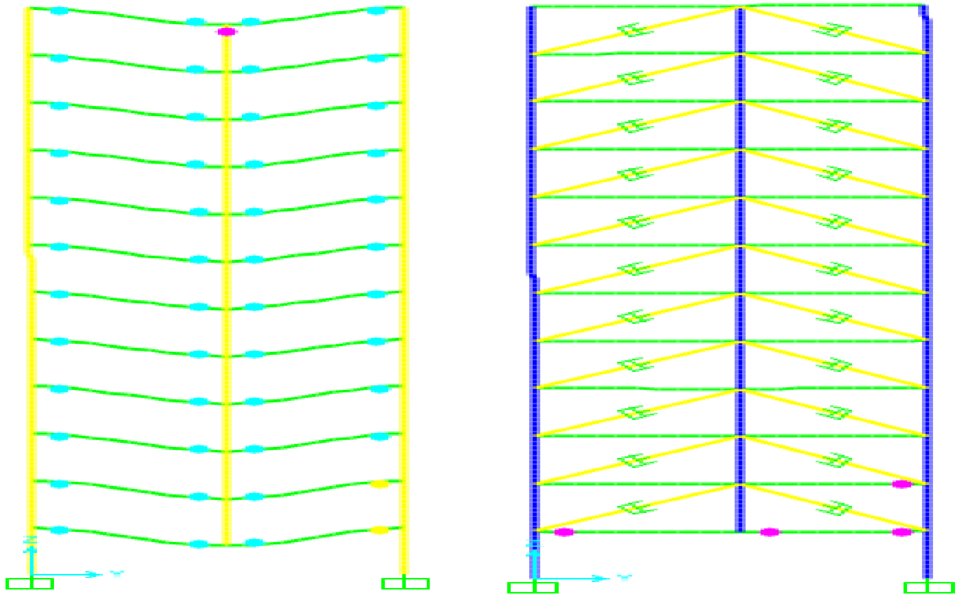
(a) without dampers (b) with dampers

Figure 5.29: Hinges at collapse for GSA loading case-1



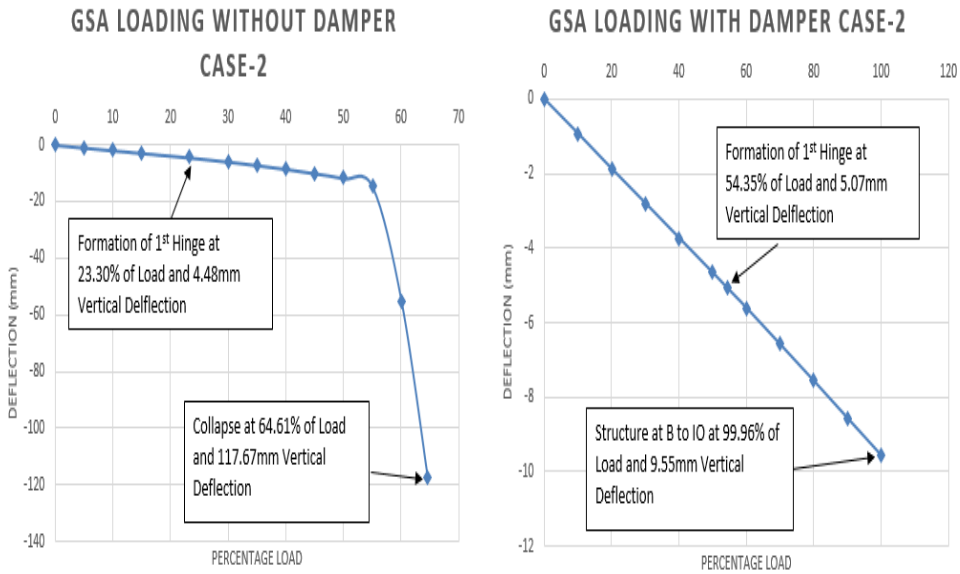
(a) without dampers (b) with dampers

Figure 5.30: Percentage Load v/s Vertical Deflection for GSA loading case-1



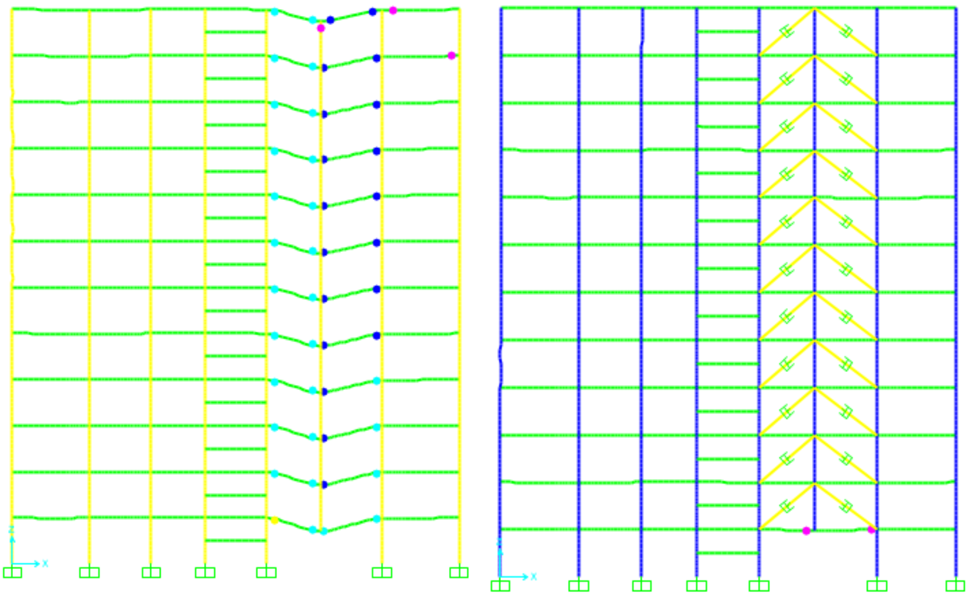
(a) without dampers (b) with dampers

Figure 5.31: Hinges at collapse for GSA loading case-2



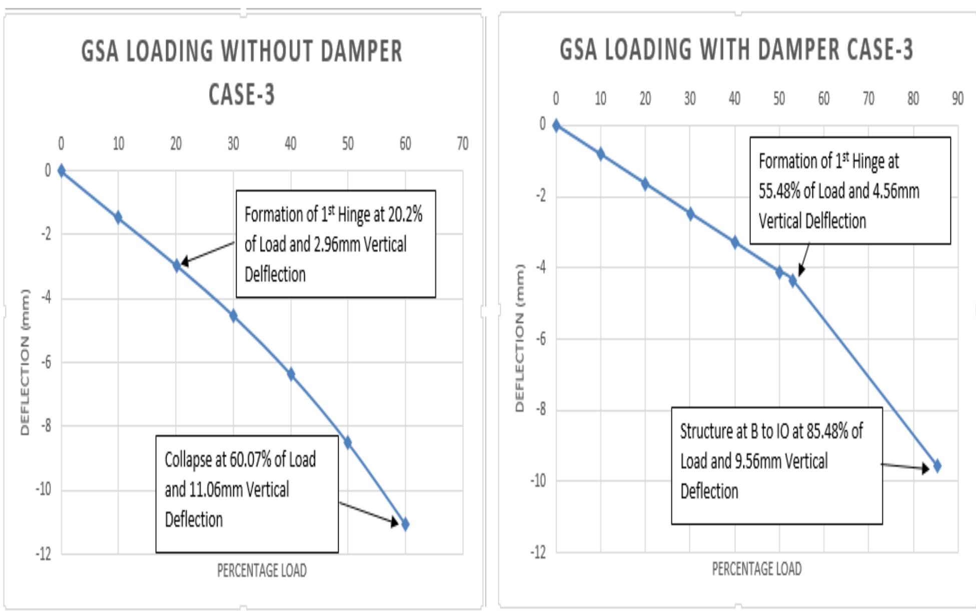
(a) without dampers (b) with dampers

Figure 5.32: Percentage Load v/s Vertical Deflection for GSA loading case-2



(a) without dampers (b) with dampers

Figure 5.33: Hinges at collapse for GSA loading case-3



(a) without dampers (b) with dampers

Figure 5.34: Percentage Load v/s Vertical Deflection for GSA loading case-3

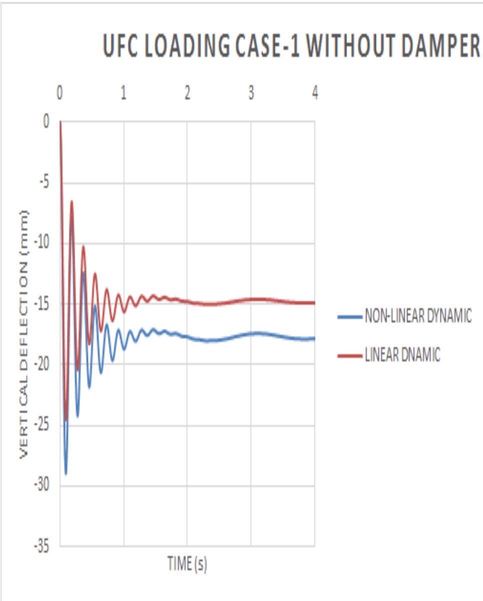
Table 5.2 shows comparison of load carrying capacity of frame with and without dampers for GSA and UFC guidelines under three different column removal cases.

Table 5.2: Comparison of Load carrying capacity of frame dampers

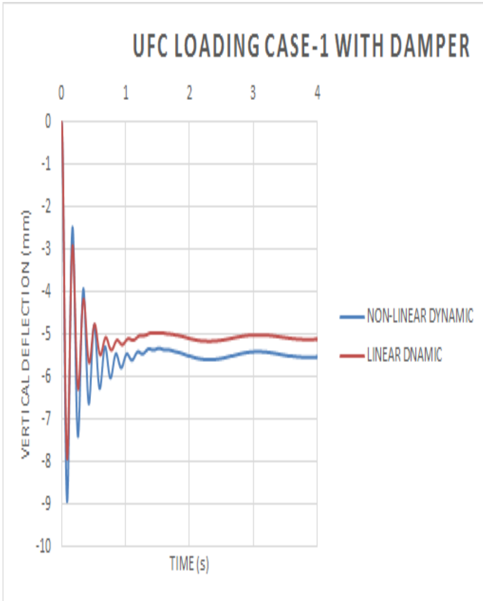
Loading & Column Removal Case	**Percentage Load corresponding to formation of 1st Hinge**			**Percentage Collapse Load**	**Percentage Load at End of Analysis**	
	Without Damper	**With Damper**	**% Difference**	**Without Damper**	**With Damper**	**% Difference**
GSA Guidelines Case-1	13.40 %	54.89 %	**+41.49 %**	62.77 %	73.84 %	**+11.07 %**
GSA Guidelines Case-2	23.30 %	54.35 %	**+31.05 %**	64.61 %	99.96 %	**+35.35 %**
GSA Guidelines Case-3	20.20 %	54.48 %	**+34.28 %**	60.37 %	85.48 %	**+51.11 %**
UFC Guidelines Case-1	11.0 %	45.08 %	**+34.08 %**	51.74 %	61.98 %	**+10.24 %**
UFC Guidelines Case-2	19.10 %	44.72 %	**+25.62 %**	53.01 %	82.94 %	**+29.93 %**
UFC Guidelines Case-3	16.60 %	43.71 %	**+27.11 %**	50.09 %	75.82 %	**+25.73 %**

Comparison of Nonlinear Dynamic Analysis Result

Nonlinear dynamic analysis is performed to obtain displacement at location of removed columns. Displacement at the location of column removal is maximum in case of frame without dampers and it is observed that after incorporating viscoelastic damper with suitable damping there is considerable reduction in the displacement at the location of removed column. Nonlinear dynamic and linear dynamic analysis result are plotted for the vertical deflection under column removal and is compared to that of frame with dampers as shown in Fig. 5.35.

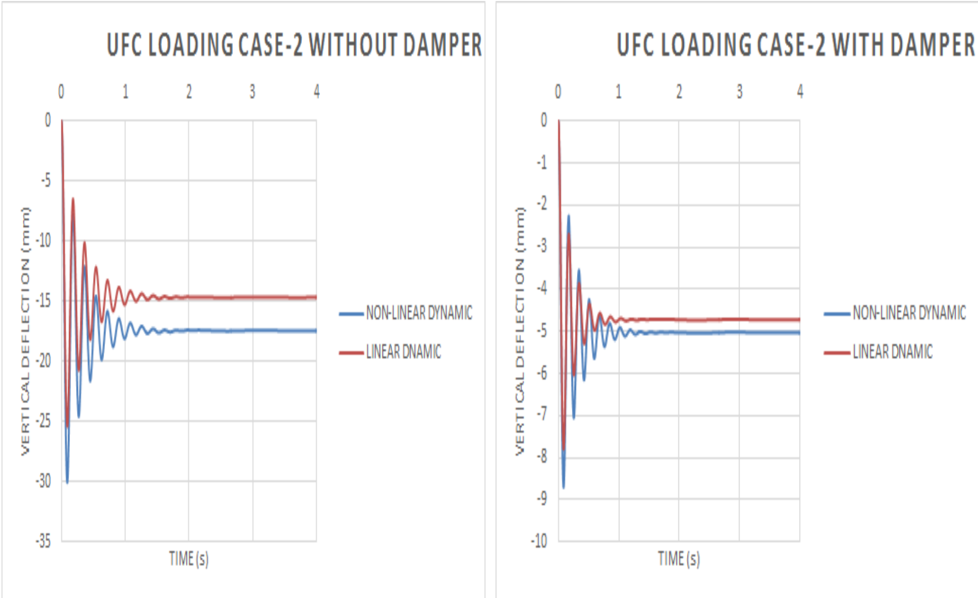


(a)



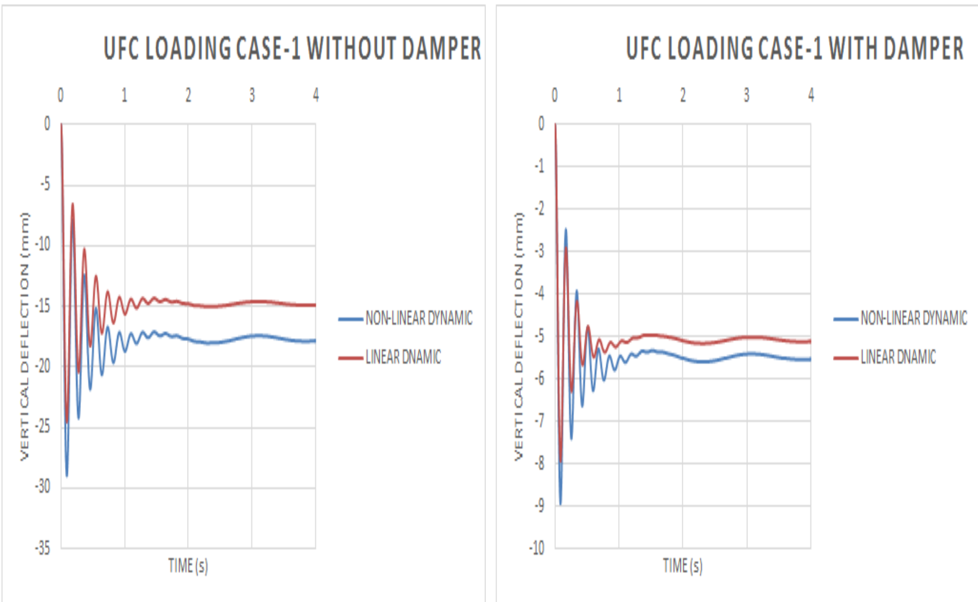
(b)

Figure 5.35: Deflection under column removal point for Case 1 UFC loading



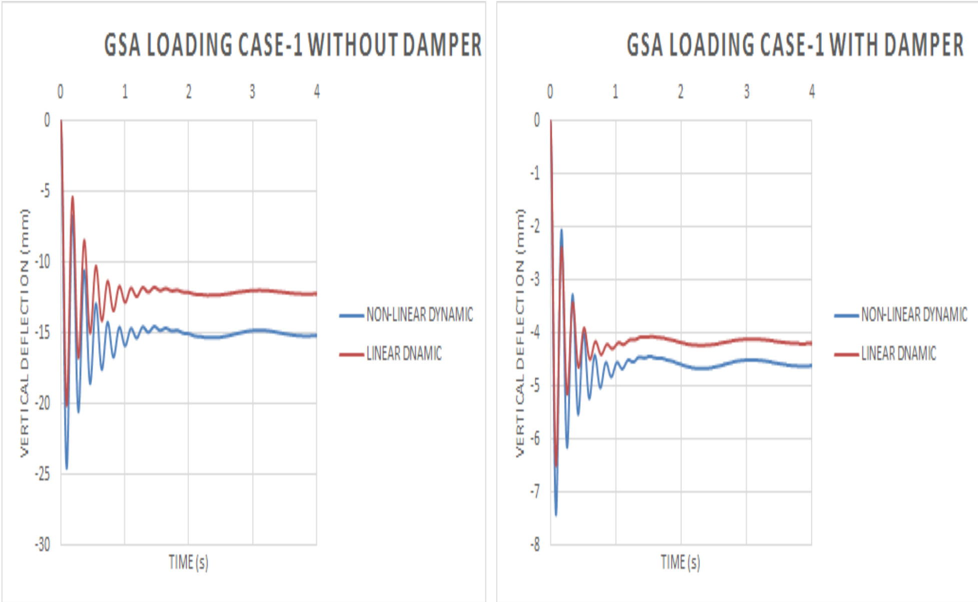
(a) (b)

Figure 5.36: Deflection under column removal point for Case 2 UFC loading



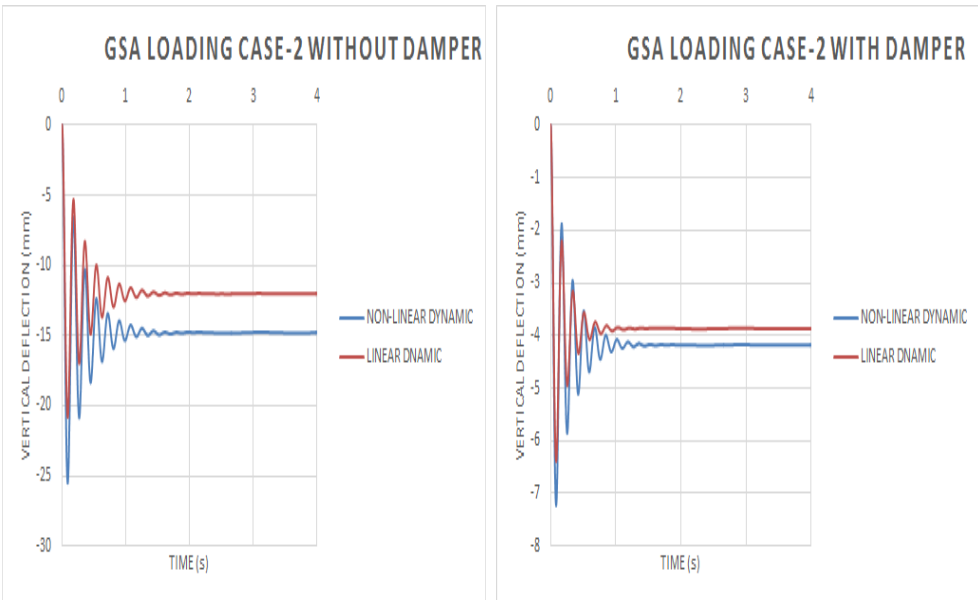
(a) (b)

Figure 5.37: Deflection under column removal point for Case 3 UFC loading



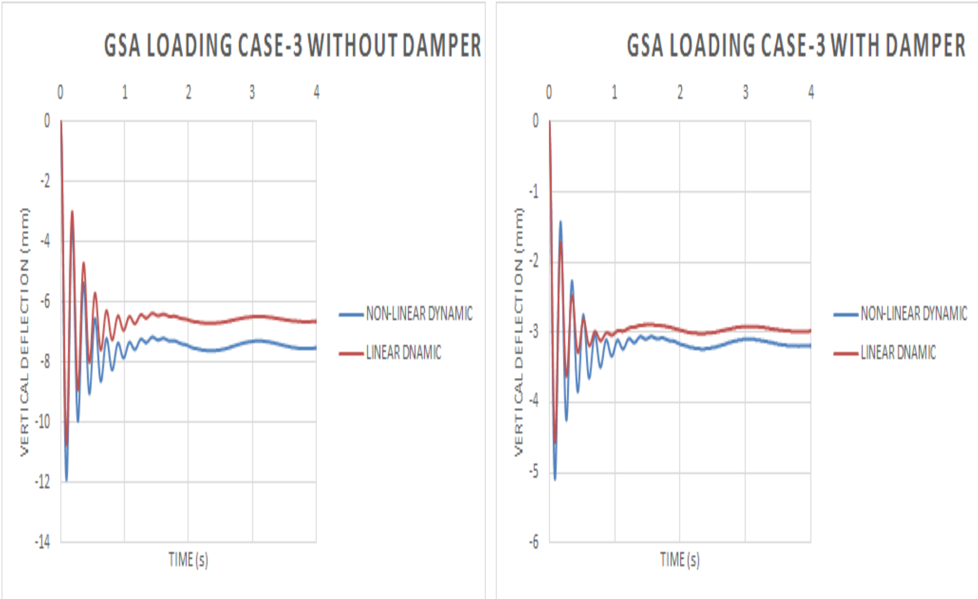
(a) (b)

Figure 5.38: Deflection under column removal point for Case 1 GSA loading



(a) (b)

Figure 5.39: Deflection under column removal point for Case 2 GSA loading



(a) (b)

Figure 5.40: Deflection under column removal point for Case 3 GSA loading

5.6 Summary

In this chapter, column is removed at five different locations and remaining structure is analyzed to evaluate the potential for progressive collapse of 12-storey reinforced concrete resedential building using structural analysis and design software SAP2000. Linear static, Linear dynamic, Nonlinear static and Nonlinear dynamic analyses are performed by following the U.S. General Service Administration (GSA) and Department of Defense (DoD) guidelines using the alternate load path method. Demand Capacity Ratios (DCR) is calculated for flexure and shear in beams and axial force,biaxial moment in column and comparisons of results of frame without viscoelastic dampers and with damper. The DCR obtained by linear static analysis is compared for both with and without viscoelastic dampers. Vertical deflection for linear dynamic analysis and Non-linear dynamic analysis is obtained at the column removal location. For Non-linear static analysis load resistance capicity and deflection is plotted for both type of system.

Chapter 6

Summary and Conclusions

6.1 Summary

Progressive collapse is defined as the spread of an initial local failure in a manner similar to a chain reaction that leads to partial or total collapse of a building. Progressive collapse is the expansion of local damage, from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a large part of it. It is also known as disproportionate collapse. Progressive collapse occurs when a structure has its loading pattern or boundary conditions changed such that structural elements are loaded beyond their capacity and fails. As a result, the remaining structure is forced to resist alternate load paths to redistribute the unbalanced force. These unbalanced redistributed forces will cause further collapse of structural members and it will continue until this additional forces are balanced.

In order to prevent the progressive collapse, structure should be capable for providing alternate load path to redistribute additional forces, when one or more column is failed. Prevention or mitigation of progressive collapse appears to be an important issue in the development of several structural design codes. U.S. General Service Administration (GSA) and Department of Defense guidelines (DoD) have issued design and analysis guidelines for progressive collapse evaluation of building structures. The aim of this study is to reduce the potential of progressive collapse of building using energy dissipation devices. Passive energy dissipation devices like viscoelastic dampers are primarily used to control the displacement during earthquakes.

In the present study, effect of viscoelastic dampers on progressive collapse resistance of 4-storey reinforced concrete (RC) frame, 4-storey symmetrical reinforced concrete (RC) building and 12-Storey residential building is performed by following both GSA and DoD guidelines. Modeling, analysis and design of the building are carried out using software SAP2000 for different threat-independent column removal conditions. Linear static, Linear dynamic, Nonlinear static and Nonlinear dynamic analysis have been recommended to estimate the potential of collapse under sudden column removal scenario from critical location.

Linear static analysis of 4-storey reinforced concrete (RC) frame, 4-storey symmetrical reinforced concrete (RC) building and 12-Storey residential building are performed by following alternate load path method. In this method original structure is designed for combination of gravity and seismic loading. Subsequently column at ground floor is removed depending on case. The structure is subjected to gravity loading as per guidelines and demand in terms of axial force, shear force and bending moment is evaluated from the analysis. Capacity at critical sections is obtained from original design and strength increase factor. If Demand Capacity Ratio (DCR) exceeds permissible values, it is considered as failed.

DCR for beam is calculated at three points left, center and right side of the column removal position. The comparison between DCR for flexure, shear and axial loading are compared for structure with and without viscoelastic dampers for linear static analysis. Study of vertical displacement under column removal point is carried out when column is removed from different locations. Displacement obtained by linear static analysis and linear dynamic analysis with and without viscoelastic damper for different damping are compared for both GSA and DoD load cases. Nonlinear Static (Push Down) analysis is performed and results are observed in terms of vertical deflection under column removal scenario, formation of hinge pattern and load carrying capacity.

6.2 Conclusions

Based on the study carried out in major project following conclusions are drawn.

- DCR obtained by DoD guidelines are having higher values compared to those obtained by GSA guidelines for all the four column removal cases. It is because of the difference in the load cases. Generally the DoD guidelines are used for military departments, the defence agencies and the structures of national importance. Therefore DoD guidelines use larger load factors and lateral loading compared to GSA guidelines.

1. **4 Storey Framed Structure**

    - In 4-storey reinforced concrete frame structure, DCR for flexure in beams exceeds the permissible value 2 for both middle column removal and interior column removal cases. Also DCR for shear and axial load and biaxial moment for column exceeds permissible value 1 for both the column removal cases.
    - When viscoelastic dampers are used, DCR for beams as well as columns reduces significantly at all the floor levels and are within the permissible limits for both the guidelines.
    - Deflection at the location of column removal is maximum in case of frame without dampers. Incorporation of viscoelastic damper with suitable damping, considerably reduces the deflection up to 50%-70% at the location of removed column.
    - Viscoelastic dampers significantly increases load resistance capacity of structure with significant reduction in vertical deflection at the location of removed column. Formation of 1^{st} hinge in the frames with viscoelastic dampers has 25%-35% more load resistance capacity as compared to frames without dampers. Similarly, load resistance capacity corresponding to collapse load increases by 40%-55% for frames with viscoelastic dampers as compared to frames without dampers.
    - From nonlinear dynamic analysis, it is observed that the deflection at the location of column removal reduces to 60%-75% after incorporating viscoelastic dampers.

2. **4 Storey Reinforced Concrete Symmetric Building**

   - In 4-storey reinforced concrete symmetric building DCR for flexure is very severe for interior column removal case-3 and case-4. For exterior column removal DCR for flexure in beams was exceeding permissible limits. DCR for shear in beam as well as for axial load and biaxial moment in column for case-3 and case-4 also created worst effect on building structure and are exceeding the permissible limits for all the five cases.
   - After incorporating viscoelastic dampers, DCR for flexure, shear and column reduces significantly and are within the limits of permissible values.
   - Deflection at the location of column removal is maximum in case-3 and case-4 considering interior column removal and with viscoelastic dampers deflection is considerably reduces by 70%-75% for both the guidelines.
   - Frames with viscoelastic damper shown the increase in load resistance capacity of structure. Formation of 1^{st} hinge in the frame with dampers has 50%-70% more load resistance capacity as compared to frames without dampers. Similarly, load resistance capacity corresponding to collapse load increases by 50%-60% for frames with viscoelastic dampers as compared to frames without dampers.
   - Nonlinear static analysis reveals that hinge formation starts from the location having maximum DCR value. Then formation of hinge continues through the locations where DCR values exceeds.
   - For nonlinear dynamic analysis deflection at the location of column removal reduces to 70%-80% after incorporating viscoelastic dampers.
   - The dynamic amplification factor of 2 is a good estimate for static analysis procedures since linear static and linear dynamic analysis procedures yield approximately the similar maximum deflections.

3. **12 Storey Reinforced Concrete Residential Building**

   - For 12-storey reinforced concrete residential building, DCR for flexure and shear in beam as well as for axial load and biaxial moment in column is exceeding the permissible limits for the bottom 5 to 7 stories and after introducing viscoelastic dampers the DCR values are within the permissible limits.

- Deflections at all the three column removal locations for linear dynamic analysis reduces to about 55%-70% as compared to frame without dampers.
- Nonlinear static (Push down) analysis shows that the load resistance capacity of frame with damper for 1^{st} hinge formation increases to 25%-40% as compared to frame without dampers. Also load resistance capacity corresponding to collapse load also increases by 25%-50%, in various column removal scenarios.

6.3 Future Scope of Work

The present study can be extended to include following aspects :

- Progressive collapse potential of important existing buildings can be evaluated.
- Different measures to mitigate progressive collapse potential of buildings such as providing frictional dampers and buckling restrained braces can be explored.
- Experimental testing of viscoelastic damper based model of building can be developed and results can be studied.
- Buildings having different structural configurations i.e. shear walled building, braced frame building etc can be undertaken to study its progressive collapse potential and its mitigation measures.

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Appendix A

Design of Viscoelastic Damper

- The design of Viscoelastic damper is an iterative process. The design is carried out according to R. D. Hanson and T. T. Soong [12] , which recommends Kelvin Model for analysis. To support the iterative calculations Microsoft Excel Sheet was used.
- Prior to design it is required to decide, desired damping ratio that should be achieved to reduce prescribed response level of building. In this study, the required structural damping ratio ζ is assumed for the initial goal.
- Here sample calculation of design of damper is carried out for required damping ratio ζ is equal to 20 percent.

Data taken:

- Fundamental frequency of the building ω = 18.4 rad/sec
- Inherent Damping ratio of the building = 5%
- Operating Temperature T = 25 C
- Storey Height h = 3m
- Required damping ratio ζ = 20%
- Angle between bracing member and floor θ = 31.78
- Target Added damping ratio = 15%

1. From modal strain energy method

$$K_d = \frac{2\zeta}{\eta_d - 2\zeta} \times K_s$$

   K_s is the storey stiffness of the structure without added dampers. K_s = 183570540 N/m. K_d = 363506602 N/m. Therefore for two dampers K_d = 181753301 N/m.

2. The thickness of viscoelastic material can be determined based on the maximum allowable damper deformation.This is Controlled by maximum allowable storey drift ratio. Damper thickness is also controlled by maximum allowable strain in material (γ). Final thickness can be given as,

$$t_d = \frac{0.005 \times h_s \times \cos\theta}{\gamma_d}$$

$$t_d = 0.03m$$

3. Simplified relationship for shear storage and shear loss modulus is given by

$$G' = 16.0 \times \omega^{0.51} \times \gamma^{-0.23} \times e^{(72.46/T)}$$

$$G" = 18.5 \times \omega^{0.51} \times \gamma^{-0.20} \times e^{(73.89/T)}$$

   From the above G' = 1429300 N/m^2 and G" = 1723000 N/m^2.

4. Area of viscoelastic damper is calculated using following equation

$$A = \frac{K_d \times t_d}{G'}$$

   Therefore, area of viscoelastic damper A = 0.2 m^2 for one layer. Assuming Width of damper pad B = 0.3 m, Length of damper B = 0.70 m, and thickness of damper t = 0.03 m.

5. The damping co-efficient C_d of viscoelastic damper can be determine from following equation,

$$C_d = \frac{A \times G"}{\omega_n \times t_d}$$

   C_d = 624.28 kNs/m

Appendix B

List of Paper Published/Communicated

- Javia Parth D., Patel Paresh V., Joshi Digesh D., "Enhancing Progressive Collapse Resistance Using Energy Dissipation Devices", NuTech, Institute of Technology, Nirma University, Ahmedabad, Feb-2016.

- Javia Parth D., Joshi Digesh D., Patel Paresh V., "Effect of Viscoelastic Dampers on Progressive Collapse Resistance of RC Building", Smart, Sustainable and Resilient Civil Infrastructure development (SSRCID-2016), The Northcap University, Gurgaon. (Full length paper accepted)

- Javia Parth D., Joshi Digesh D., Patel Paresh V., "Enhancing Progressive Collapse Resistance of Frame Using Viscoelastic Dampers", International Conference on Recent Developments in Design and Construction Technologies of Tall Structures (REDECON - 2016), Association of Consulting Civil Engineers, Bengalore. (Abstract accepted)

- Javia Parth D., Patel Paresh V., Joshi Digesh D., "Mitigation of Progressive Collapse of RC building using Viscoelastic Dampers", Structural Congress 2017, American Society of Civil Engineering (ASCE), USA. (Abstract Communicated)