Evaluation of Progressive Collapse Resistance of High Rise Building with Different Structural Systems

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Major Project

Submitted in partial fulfillment of the requirements for the degree of

Master of Technology

In

Civil Engineering

(Computer Aided Structural Analysis and Design)

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Declaration

This is to certify that

- The thesis 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.
- Due acknowledgement has been made in the text to all other materials used.

Jay A. Kapadiya

Certificate

This is to certify that the Major Project Report entitled **"Evaluation of Progressive Collapse Resistance of High Rise Building with Different Structural Systems"** submitted by Mr. Jay Kapadiya (Roll No: 13MCLC30) towards the partial fulfilment of the requirement 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 my 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

Structural safety has always been a key concern responsible for the design of civil engineering projects. One of the mechanisms of structural failure that has accumulated increased attention over the past few decades is referred to as progressive collapse. Progressive collapse of building structure is initiated when one or more vertical load carrying members particularly columns are seriously damaged or collapse during any of the abnormal loads i.e. vehicle impact, fire, earthquake, or other man-made or natural hazard. As a result, a substantial part of the structure may collapse, causing greater damage to the structure than the initial impact. Thus it is necessary to prevent progressive collapse of the building structure.

The aim of present study is to evaluate the progressive collapse resistance of multi storied building with different structural systems. Various causes for progressive collapse are presented. After the collapse of the World Trade Center (WTC) Tower many government and private authorities worked on developing design guidelines for progressive collapse resistant structures. Among all the guidelines the U.S. General Service Administrator (GSA)and Department of Defense (DoD)are most widely used by structural engineers. Various criteria to be considered to perform progressive collapse analysis as specified in these guidelines are discussed. These guidelines have suggested three different analysis methods. Comparison between these two guidelines is also presented.

Complete analysis and design of 10-storey concrete building is presented. A regular floor plan of 20 m x 16 m is considered. Linear static analysis is performed using analysis program Midas Gen-2012. The DCR (Demand Capacity Ratio) is calculated using linear static analysis. It is important to mitigate the vulnerability of progressive collapse if building is having high potential of progressive collapse. To reduce the progressive collapse three types of structural systems are explored i.e. bracing at top storey level of the building, bracing at side face of the building and bracing at top storey level and side face of the building. The DCR (Demand Capacity Ratio) is calculated using linear static analysis and compared with different structural systems. Also, displacement is compared with different structural systems.

For 10-storey building DCR in case of flexure exceeds the permissible limit of 2.0 in case of GSA and UFC load case which reveals that beams are not safe in flexure as per GSA guidelines and UFC guidelines. Also, DCR in case of shear exceeds the permissible limit of 1.0 in case of UFC load case which reveals that beams are safe in shear as per GSA guidelines but fails as per UFC guidelines.Demand capacity ratios for column exceed the allowable limit of 1.0 at bottom two to four stories.Displacement under column removal point for GSA loading and UFC loading is compared for all the cases and case 4 of column removal creates worst effect on the building structure. The three alternative structural systems are presented from which provision of bracing at top storey level and side face of the building is most economical solution to reduce the potential of progressive collapse.

Acknowledgement

I would like to thank my guide **Dr. P. V. Patel**, Professor & Head, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad whose keen interest and knowledge base helped me to carry out the major project work. His constant support and guidance during my project work equipped me with a great understanding of different aspects of the project work.

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Abbreviations

GSA .... General Services Administration
UFC .... Unified Facilities Criteria
USA .... United States of America
TF .... Tie Force Method
AP .... Alternate Path Method
ELR .... Enhanced Local Resistance
DoD .... Department of Defense
DCR .... Demand-to-Capacity Ratio
ϕ .... Strength reduction factor
R_n .... Nominal Tie Strength
X_i .... Load Factor
F_i .... Load Effect
W_f .... Floor Load

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Chapter 1

Introduction

1.1 General

The spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it has been known as “progressive collapse”.

Progressive collapse of existing building is initiated by the sudden failure of one or more of its major load bearing elements, typically columns or walls. Once a column is removed due to a vehicle impact, fire, earthquake, or other man-made or natural hazards, the buildings weight (gravity load) transfers to neighbouring columns in the structure. If these columns are not properly designed to resist and redistribute the additional gravity load, that part of the structure fails. The vertical load carrying elements of the structure continue to fail until the additional loading is stabilized. As a result, a substantial part of the structure may collapse, causing greater damage to the structure than the initial impact. Progressive collapse of a structure takes place when the structure has its loading pattern or boundary conditions changed such that structural elements are loaded beyond their ultimate capacities and fail. When any element fails, redistribution of the loads and failure of the next elements in the vicinity in a chain-like reaction until the failure of the whole building.

A progressive collapse of a building is a catastrophic partial or total failure. That arise from an initiating event that causes local damage that cannot be absorbed by the inherent continuity and ductility of the building structural system. Following this local damage or failure, a chain reaction of failures propagates vertically or horizontally and develops into an extensive partial or total collapse, where the resulting damage is disproportionate to the local damage caused by the initiating event. Such collapses can be initiated by many causes, including design and construction errors and events that are beyond the design basis or are not considered explicitly in design.

Such events would include abnormal loads not normally considered in design (e.g. gas explosions, vehicular collisions, and sabotage), severe fires, extreme values of environmental loads that stress the building system well beyond the design envelope, and misuse. Requirements for blast resistant design and progressive collapse prevention are now mandatory in specific buildings like airports, emergency management centers, and some critical governmental facilities which may be a target for terrorist attacks. The performance of buildings during progressive collapse event depends on many factors. Those factors include: the actual strength to the design strength, the level of redundancy in the structural system, the level of structural integrity of the individual members to form a whole system, and the types of structural details and the ductility existent in the system.

1.2 Historical Background

- **Ronan Point Apartment Building, London, England, May 1968:** Ronan Point was a development of apartment buildings in London. It was built between 1966 and 1968. On the morning of May 16, 1968, a gas leak caused an explosion in an apartment of the 18th floor of one of the buildings. The explosion blew out an exterior wall panel. The loss of an exterior wall triggered the collapse of the upper floors followed by the collapse of the floors below due to the impact of the falling upper floors.

Figure 1.1: Collapse of Ronan Point Apartment Building

- **Skyline Plaza, Virginia, March 2, 1973:** While concrete was being placed on the 24th floor and shoring removal was occurring on the 22nd floor, a collapse occurred for the full height of the tower. Impact of debris also caused horizontal progressive collapse of entire parking garage under construction adjacent to the tower. As a result 14 workers were killed, 34 injured.

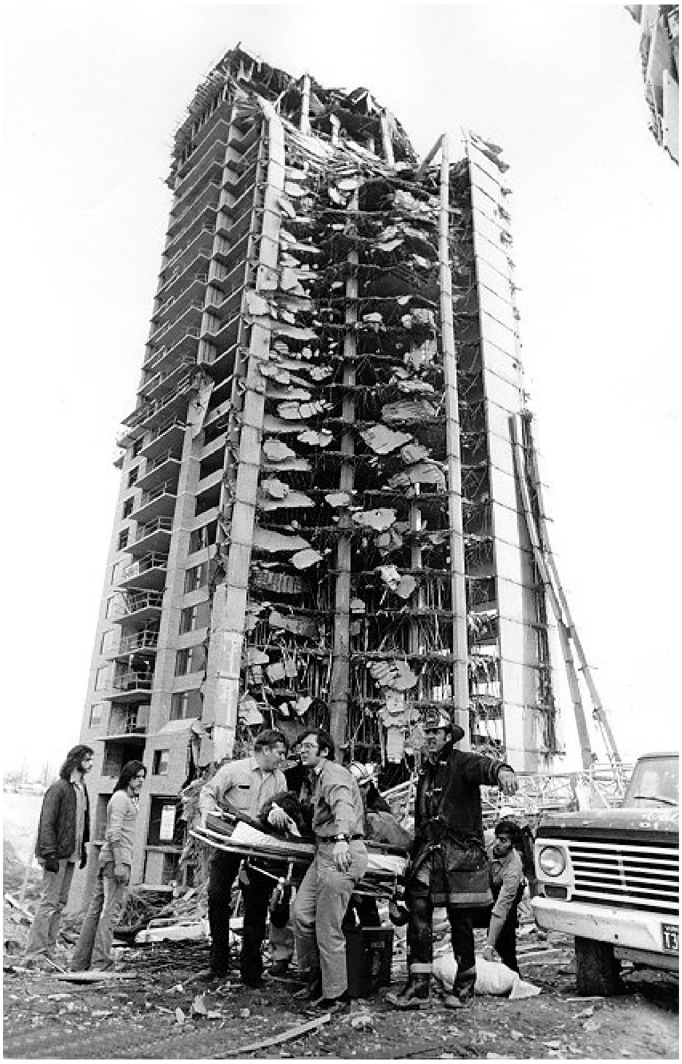


Figure 1.2: Collapse of Skyline Plaza

- **Khobar Towers, Saudi Arabia, June 25, 1996:** 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.

Figure 1.3: Collapse of Khobar Towers

- **P. Murrah Federal Building, Oklahoma, April 19, 1995:** The Alfred P. Murrah Building located in Oklahoma City, Oklahoma, was an office facility for the U.S. government. On the morning of April 19, 1995 the Murrah Building was the target of a terrorist attack in which a truck bomb was detonated in front of its north side. The explosion caused extensive structural damage to the building.

Figure 1.4: Collapse of P. Murrah Federal Building

- **World Trade Centre, New York City, September 11, 2001:** The September 11 attacks were a series of four coordinated terrorist attacks launched by the Islamic terrorist group al-Qaeda upon the United States in New York City and the Washington, D.C., on Tuesday, September 11, 2001. The attacks killed almost 3,000 people and caused at least $10 billion in property and infrastructure damage.

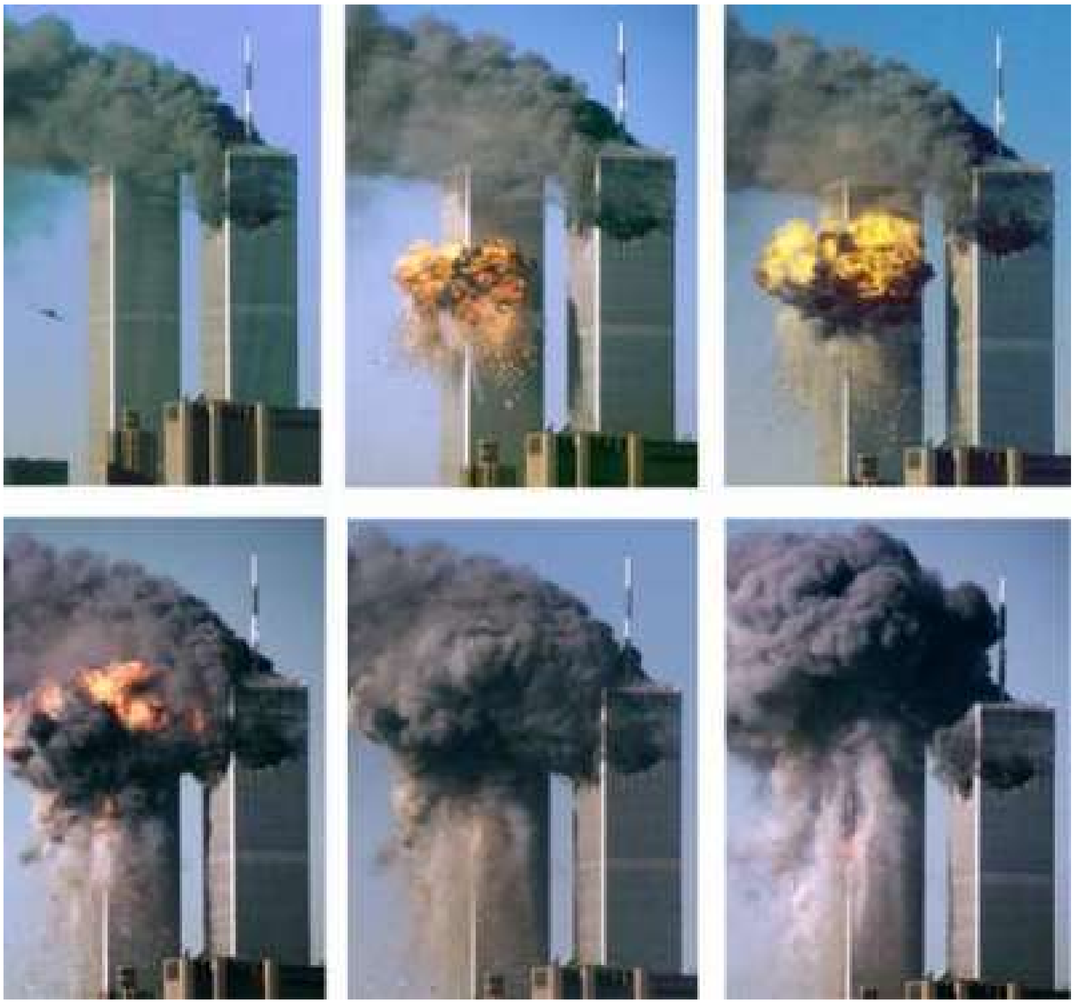


Figure 1.5: Collapse of World Trade Centre

1.3 Causes of Progressive Collapse

Progressive collapse of the building structure is generally occurred under the abnormal loads. A number of potential abnormal load hazards, which could trigger progressive collapse, are considered:

- Gas Explosions
- Bomb explosion (Blast load)

- Design/Construction error
- Fire
- Overload due to occupant misuse
- Vehicular collision

1.4 Objective of Study

The key objectives of study are as follows:

- To study the various causes of the progressive collapse.
- To study and compare the various guidelines for progressive collapse analysis.
- To study the various analysis approaches for evaluation of the progressive collapse resistance of high rise building.
- To study to reduces the progressive collapse of the building by providing various structural systems.

1.5 Scope of Work

In order to understand above objective the scope of work for major project is decided as follow.

- Study the various causes of the progressive collapse.
- Study and comparison of the various guidelines/specification for progressive collapse analysis of building.
  - U.S. General Service Administration (GSA-2003)
  - Unified Facilities Criteria, UFC 4-023-03(2009)

- Study of various analysis approaches for progressive collapse resistance design.
- Perform linear static analysis procedure using Midas-Gen 2012(v3.1) to study the behaviour of R.C.C. building by removing interior/external column.
- Study of different structural systems to reduce progressive collapse of the building.

1.6 Organization of Major Project

The content of major project is divided into different chapters as follows:

Chapter 1 include introduction and overview of the major project work. The various progressive collapse are discussed with the historical background. Also, causes of the progressive collapse are identified. It also includes objective of study and scope of work.

Chapter 2 covers literature review. In this chapter brief literature review is presented pertaining to progressive collapse resistant structural system of high rise building.

Chapter 3 includes U.S. General Services Administration (GSA 2003) guidelines for progressive collapse. Important parameters regarding progressive collapse analysis such as static and dynamic analysis loading, analysis procedure, internal and external column removal consideration and acceptance criteria are discussed as suggested by GSA guidelines.

Chapter 4 includes Unified Facilities Criteria (UFC) published by Department of Defense (DoD) for design of building to resist progressive collapse. Tie force method, Alternate load path method and Enhanced local resistance method is discussed as per revised UFC 4-023-03. Also, comparison between General Services Administration (GSA 2003) Guideline and Department of Defense (DoD) of United States of America (USA) Guideline to evaluate the progressive collapse potential is presented.

Chapter 5 includes progressive collapse analysis of 10-storey reinforced building. Building is analyzed by Midas-Gen 2012(v3.1). The demand capacity ratio is compared for linear static analysis and linear dynamic analysis with different types of column removal scenario.

Chapter 6 Three different mitigation strategies to prevent progressive collapse are illustrated. DCR is calculated at critical locations after considering various mitigation approaches for all cases of column removal for both the guidelines GSA and UFC. The displacement under column removal point for all the approaches is compared with displacement obtained before mitigation.

Chapter 7 summarizes the work done in the major project. Also, includes summary of work done, various conclusions obtained from the study and future scope of work.

Chapter 2

Literature Survey

2.1 General

Literature review related to Progressive collapse resistant structural system of high-rise buildings is presented in this chapter. Various research papers have been referred to understand the behaviour of the high-rise building during progressive collapse and different structural systems to be provided to defeat progressive collapse.

2.2 Literature Review

Kim and Jung[1] investigated the progressive collapse resisting capacity of mega frame structures composed of many identical subsystems based on column loss scenario. Also, nonlinear analyses of mega frames composed of various numbers of subsystems and mega columns were carried out by removing one of the first story mega columns. Based on the analysis results, various alternative schemes were investigated to enhance the progressive collapse resisting capacity of mega frame buildings. To enhance the resistance against progressive collapse, they redesigned the basic model structure with four mega columns by adding additional floor trusses in the transfer floors, adding moment resisting frames at the perimeter and adding vertical interior or exterior mega bracing. The pushdown analysis results showed that the schemes with additional mega bracing were most effective in increasing the progressive collapse resisting capacity of mega frame buildings with additional benefit of smaller requirement of structural steel. It was shown that by installing mega bracing, more structural members participate in resisting progressive collapse. Based on the analysis results, it is recommended that the exterior or interior mega bracing be used in the design of mega frame structures to enhance the overall redundancy and consequently the progressive collapse resisting capacity of the structure.

Tsai and Huang[2] investigated the influence of three types of exterior partially infilled walls (parapet, wing-type and panel types) on the column-loss response of an RC building. Linear static analysis results reveal that the sectional moment demand-to-capacity ratios (DCRs) of beams are generally reduced with consideration of the infilled walls. Moreover, nonlinear static analysis results indicate that the collapse resistance of the RC building under column loss may be significantly increased with the wing-type walls. Also, the deformation capacity corresponding to the collapse resistance is reduced with the infilled walls. From the aspect of structural behaviour, the wing-type wall is a better choice than the parapet and panel types for practical application.

Kim et al.[3] investigated the progressive collapse potential of braced frames using nonlinear static and dynamic analyses. Eight different bracing types were considered and their performances were compared with those of a special moment-resisting frame designed with the same design load. According to the pushdown analysis results, most braced frames designed as per current design codes satisfied the design guidelines for progressive collapse initiated by loss of a first storey interior column; however, most model structures showed brittle failure mode caused by buckling of braces and columns. The inverted-V type braced frames showed superior ductile behaviour during progressive collapse. The nonlinear dynamic analysis results showed that all the braced structures remained in stable condition after sudden removal of a column, and their deflections were less than that of the moment resisting frame.

Mashhadiali and Kheyroddin[4] investigated the progressive collapse resisting capacities of the tube-type diagrid and the newly developed hexagrid system buildings. Push down and time history analyses were carried out to evaluate the nonlinear static and dynamic behaviors, respectively. 28-storey and 48-storey buildings were studied for five removal members from the corner of the buildings. The various parameters, such as aspect ratio and plan geometry, can affect the selected structural system behaviour. The analysis results state that the hexagrid has enough potential of force redistribution to resist progressive collapse due to its special configuration. Push down curves report that hexagrid is ductile and the diagrid is brittle. The location of plastic hinge formations clarifies completely the behaviour of both structural systems and illustrate that the mega corner column increases the capacity of structure against progressive collapse. It is found that as buckling is prevented, behaviour of both structural systems improves and this effect is more significant in diagrids than hexagrids.

Tavakoli et al.[5] presented the three and two dimensional modelling and push-over analysis of seismically designed special dual system steel frame buildings with concentrically braced frames with complete lose of critical elements. The structures consist of 5 and 15 floors with 4 and 6 bays. Results indicate, when the number of stories and bays are increased, larger capacity to resist progressive collapse under lateral loading. It seems there is no concern about occurrence of progressive collapse under seismic loading in one column and adjacent brace lose scenario for steel special dual systems containing special moment resisting frame and X brace.

Kwon et al.[6] evaluated the progressive collapse potential of building structures designed for real construction projects based on arbitrary column removal scenario using various alternate path methods specified in the GSA guidelines. 22-storey reinforced concrete moment frames with core wall building and a 44-storey interior concrete core and exterior steel diagrid structure are analysed. The progressive collapse resisting capacities of the model structures were evaluated using the linear static, nonlinear static, and nonlinear dynamic analyses. The linear static analysis results showed that progressive collapse occurred in the 22-storey model structure when an interior column was removed. However the structure turned out to be safe according to the nonlinear static and dynamic analyses. Similar results were observed in the 44-storey diagrid structure. Based on the analysis results, it was concluded that, compared with nonlinear analysis procedures, the linear static method is conservative in the prediction of progressive collapse resisting capacity of building structure based on arbitrary column removal scenario.

Ren et al.[7] investigated the progressive collapse resistance of high-rise RC frame shear wall structures, two typical 15-storey building models are designed with equivalent overall lateral resistance to seismic actions. Building A is a weak wall-strong frame structure, whereas building B is a strong wall-weak frame system. Three dimensional (3D) finite-element models of the two structures are established using fiber beam and multilayer shell elements. The progressive collapse resistances of the frames and the shear walls in both structures are evaluated under various column (shear wall) removal scenarios. Results demonstrate that there is a difference in progressive collapse prevention performance for different structural layouts. The progressive collapse resistance tends to be inadequate for the strong wall-weak frame system. Such a system is subsequently redesigned using the linear static alternate path (AP) method proposed in GSA guidelines. The outcome of this study has provided a reference for progressive collapse prevention designs of typical and representative high-rise RC frame shear wall structures.

Kim and Hong[8] investigated the progressive collapse-resisting capacities of tilted and twisted buildings evaluated by nonlinear static and dynamic analyses. 30-storey tilted buildings with braced cores and 30-storey twisted buildings with reinforced concrete cores were prepared for analysis. The tilted structures were designed with steel braced cores, and the twisted buildings were designed with square RC cores. The progressive collapse of the model structures was initiated by removing one of the first-storey columns. The performances of the irregular structures were compared with those of the regular buildings designed without tilting or twisting. Results shows the progressive collapse potential of the tilted structures varied significantly, depending on the location of the removed column. Especially, the corner column located in the tilting direction needs to be strengthened or protected from possible damage to prevent progressive collapse of the whole structure. Also observed in the tilted structures that the plastic hinges formed not only in the bays from which a column was removed, but also in the nearby bays. Similar phenomenon was also observed in the twisted structures. However, the overall progressive collapse potentials of the tilted or twisted structures considered in this study were not particularly higher than those of the corresponding regular structures. This was partly because the tilted or twisted structures were designed with larger structural members considering their irregularities. Another reason seems to be that, compared with regular structures, more structural elements were involved in resisting progressive collapse when a structural member was eliminated.

Kim and Park[9] investigated the progressive collapse potential of 36-storey building structures with RC core walls and outrigger trusses as a major lateral load-resisting system. Two types of perimeter frames were designed: (i) perimeter frames with mega-columns and (ii) perimeter frames with belt trusses at the top storey. The static pushdown analysis of the structure with mega-columns and outrigger trusses showed that the maximum strength reached only about 20% of the load specified in the GSA guideline when a mega-column was removed. The dynamic analysis showed that the vertical displacement monotonically increased until collapse when a mega-column was suddenly removed. However, the structure with outrigger and belt trusses remained stable after a perimeter column was removed. In this case the maximum load factor obtained from pushdown analysis reached almost 1.0. The progressive collapse resisting capacity of the structure with mega-columns and core walls connected by outrigger trusses could be enhanced by providing additional redundancy to the key elements such as mega-columns. It was observed that redesigning the structure with additional belt trusses or with moment connected interior/exterior frames significantly enhanced robustness of the structure. Moreover, based on the comparison of static and dynamic analysis results, it was concluded that the dynamic amplification factor of 2.0 recommended in the guidelines provided reasonably conservative results.

Kim and Lee[10] investigated the effect of infill steel panels on enhancing progressive collapse resisting capacity of steel moment frames and a simple design procedure for infill steel panels was proposed to ensure safety against progressive collapse caused by sudden removal of a column. The progressive collapse potentials were evaluated based on arbitrary column removal scenario. The accuracy of the equivalent single brace modelling techniques of steel panels was investigated in comparison with the analysis results of finite element modelling. The analysis results showed that the infill steel panels were effective in reducing the progressive collapse potential of moment frames caused by sudden removal of a column. It was observed that as the thickness of the steel panels increased the progressive collapse resisting capacity also increased. However when the thickness of the steel panels increased higher than a certain level the increase in the progressive collapse resisting capacity did not increase proportionally because of yielding of columns. Even the partial infill panels or panels with perforation were somewhat effective in protecting the structures against progressive collapse. The simplified modelling of steel panels utilizing an equivalent single brace generally corresponded well with the finite element model, and the preliminary design procedure of steel panels using the single brace model turned out to be effective in estimating the minimum thickness of steel panels required to ensure safety against progressive collapse.

Kim and Lee[11] investigated the progressive collapse potential of high-rise tube-type structures in which lateral load-resisting systems are located at the perimeter of the structures. Two different types of diagrid structures, with and without corner columns, and a tubular structure with closely spaced external columns and deep spandrel girders, were considered for analysis. In the nonlinear static pushdown analysis, any dynamic amplification factor was not applied in the load combination based on the observation that the amplification of member force was not significant in the diagrid and tubular model structures. The analysis results showed that tube-type buildings generally had high resistance to progressive collapse caused by the sudden loss of external members. The progressive collapse of tube-type buildings tended to occur when perimeter columns corresponding to more than 11% of all vertical members were removed from a side of the diagrid and tubular structures. When the diagonals located around a corner were removed, the number was reduced to 8%. It was observed that the addition of corner columns in the diagrid system did not contribute significantly to an increase in maximum strength for progressive collapse, but helped prevent the failed members from propagating all around the perimeter. It was also observed that the progressive collapse-resisting capacity of 54-storey diagrid structures were slightly higher than that of 36-storey structures.

2.3 Summary

In this chapter brief literature review is presented pertaining to progressive collapse resistance structural system of high rise building.

Chapter 3

U.S. General Services Administration Guidelines

3.1 General

The Progressive Collapse Analysis and Design Guidelines is developed by the General Services Administration (GSA 2003)[13] to assist in reducing potential for progressive collapse in new buildings as well as existing ones.
It starts with a process to determine whether a building is exempt from progressive collapse considerations or not based on following factors:

- Building occupancy
- Building category (e.g., reinforced concrete building, steel frame building, etc.)
- Number of stories
- Seismic zone

The evaluation is done by performing structural analysis for the following, the removal of one column or a 30 ft length of bearing wall. GSA guideline suggests alternate load path method to evaluate the potential of progressive collapse. Overall flow for consideration of progressive collapse is given in the Figure 3.1.

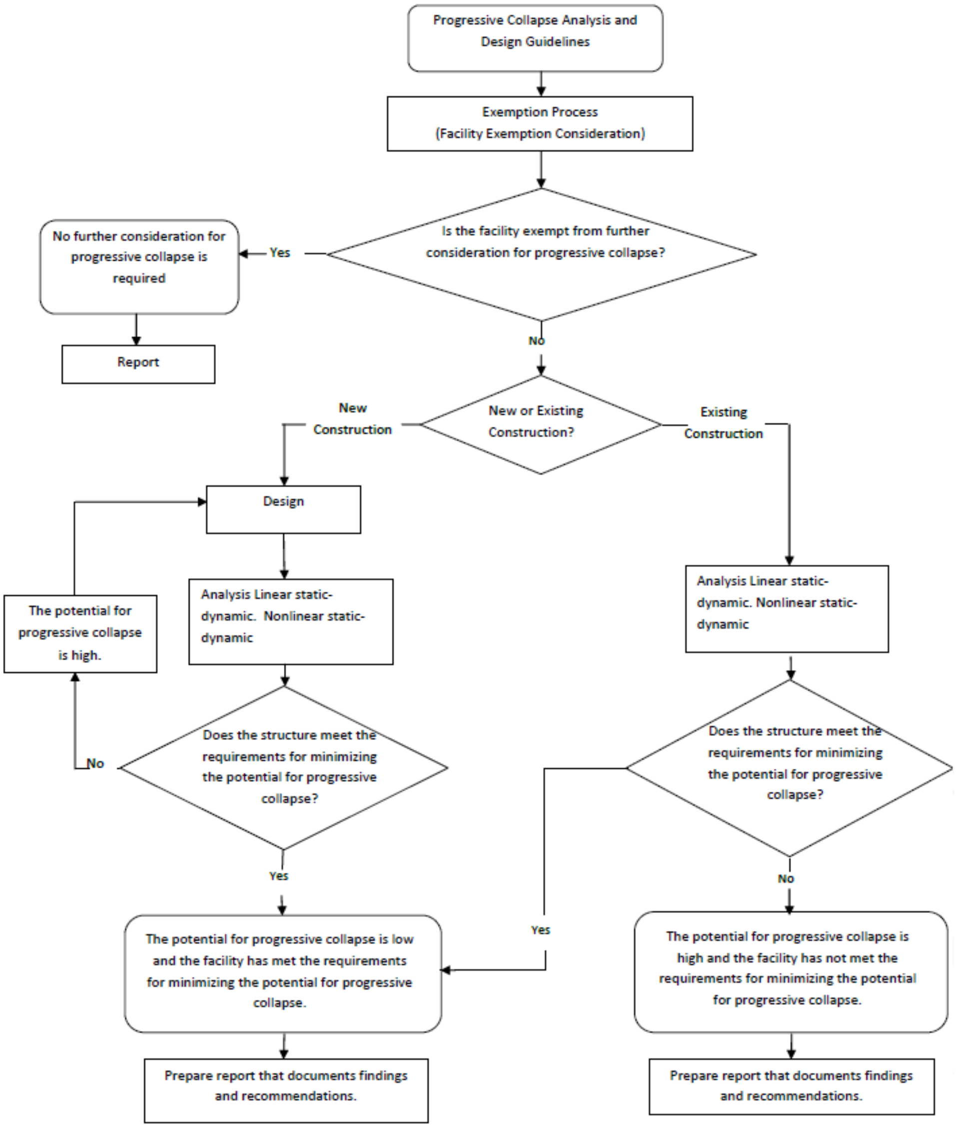


Figure 3.1: Overall Flow for Consideration of Progressive Collapse

3.2 Analysis Method

Progressive collapse of a structure takes place when the structure has its loading pattern or boundary conditions changed. When that structural elements are loaded beyond their ultimate capacities, the failure of the primary load resistance system leads to redistribution of force to the adjoining members. If the adjoining member cannot resist the additional load, then that member fails. This process continues in the structure and eventually building collapses.

3.3 Loading

The U.S. General Service Administration guidelines recommended that the following downward loads be applied when assessing the potential for progressive collapse.

3.3.1 Static Analysis Loading

For static analysis purposes the following vertical load shall be applied downward to the structure under investigation:

Load = 2(DL + 0.25 LL)

Where,

DL = dead load

LL = live load (higher of the design live load or the code live load)

3.3.2 Dynamic Analysis Loading

For dynamic analysis purposes the following vertical load shall be applied downward to the structure under investigation:

Load = (DL + 0.25 LL)
Where,
DL = dead load
LL = live load (higher of the design live load or the code live load)

3.4 Analysis Procedure

The static linear analysis approach may be used to assess the potential for progressive collapse. A linear static procedure may be used for determining the potential for progressive collapse.
The potential for progressive collapse can be determined by the following procedure.

Step 1: The components and connections of both the primary and secondary structural elements shall be analyzed for the case of an instantaneous loss in primary vertical support. The applied downward loading on the structure for static analysis purposes is 2(DL + 0.25LL) and for dynamic analysis purposes is (DL+0.25LL).

Step 2: The results from the analyses performed in Step 1 shall be evaluated by utilizing the analysis criteria specified by GSA guideline[13].

3.5 Analysis Considerations

The U.S. General Service Administration (GSA) guideline[13] suggests the following analysis consideration in the assessment of progressive collapse for symmetric and asymmetric structural configuration.

3.5.1 Typical Structural Configurations

Structure that have a relatively simple layout with no atypical structural configuration shall use the following analysis scenarios:

Exterior Considerations

The following exterior analysis cases shall be considered in the procedure outlined as given above.

- Analyze for the instantaneous loss of a column for one floor above grade (1st storey) located at or near the middle of the short side of the building.
- Analyze for the instantaneous loss of a column for one floor above grade (1st storey) located at or near the middle of the long side of the building.
- Analyze for the instantaneous loss of a column for one floor above grade (1st storey) located at the corner of the building.

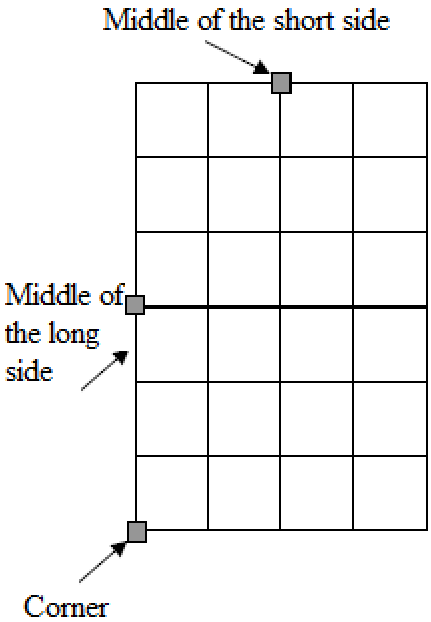


Figure 3.2: Exterior Column Removal Consideration

Interior Considerations

Facilities that have underground parking or uncontrolled public ground floor areas shall use the following interior analysis case in the procedure outlined as given above.

- Analyze for the instantaneous loss of 1st column that extends from the floor of the underground parking area or uncontrolled public ground floor area to the next floor (1st storey). The column consideration should be interior to the perimeter column lines.

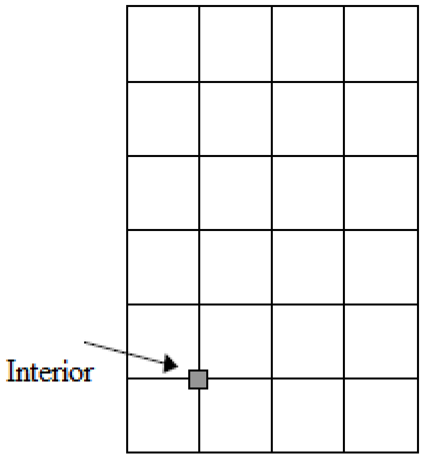


Figure 3.3: Interior Column Removal Consideration

3.5.2 Atypical Structural Configurations

All the structures are unique and are often not typical. GSA guideline gives different approach for atypical structural configurations. For such structures, the scenarios should consider cases where loss of a vertical support (column or wall) could lead to disproportionate damage. Possible structural configurations that may result in an atypical structural arrangement include, but are not limited to, the following configurations:

- Combination Structures
- Vertical Discontinuities/Transfer Girders
- Variations in Bay Size/Extreme Bay Sizes
- Plan Irregularities
- Closely Spaced Columns

Combination Structures : For facilities that utilize a combination of frame and wall systems for the primary supporting structure, one can apply considerations similar to that presented for typical building configurations. The user shall use engineering judgment to determine the critical situations that should be assessed for the potential for progressive collapse. The considerations may be similar to those utilized in typical building configurations, but additional configurations may be necessary depending on the structural composition.

Vertical Discontinuities/Transfer Girders : Structures that have vertical discontinuities may warrant additional consideration for progressive collapse. Examples of vertical discontinuities include discontinuous shear walls or columns such as the use of transfer girders. If vertical discontinuities are present in the primary structural configuration, analyses of the response of the building for a loss of primary vertical support in these areas shall be considered.

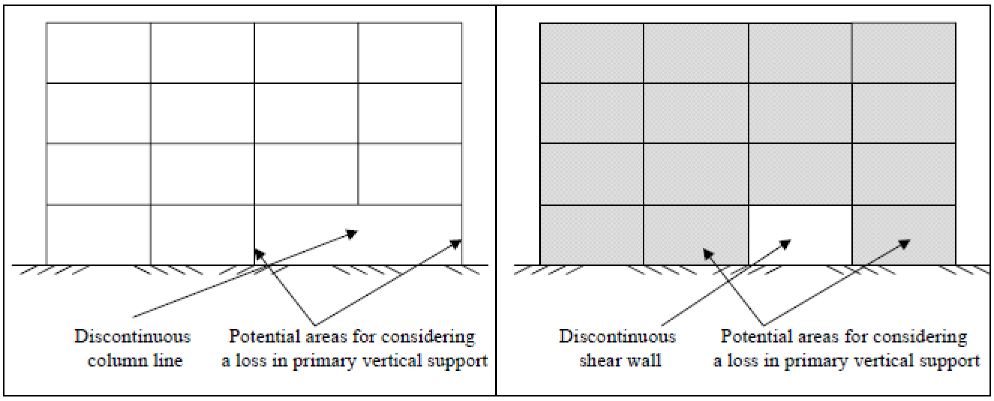


Figure 3.4: Vertical Discontinuities

Variations in Bay Size/Extreme Bay Sizes : A building configuration that contains structural bay that have a large variance in size or extremely large bay sizes should be considered vulnerable and an assessment of the potential for progressive collapse shall be performed in these areas. Structural bays that are greater than 30 ft in any direction are considered extreme.

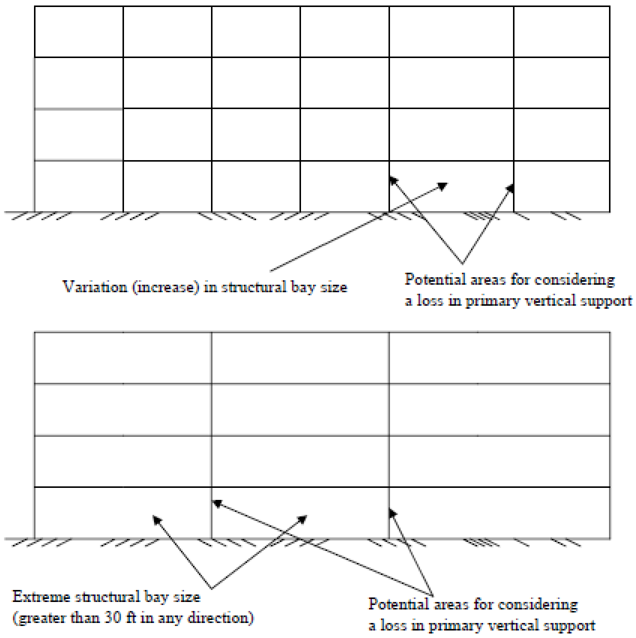


Figure 3.5: Building with Substantial Variation in Bay Size and Extreme Bay Size

Plan Irregularities : Plan irregularities such as re-entrant corners could present vulnerable areas in regards to the potential for progressive collapse. This type of structural configuration should be investigated regarding potential for progressive collapse. For example the removal of a primary support along the exterior of this structure could potentially collapse three structural bays from the ground floor level to the roof.

Closely Spaced Columns : Structures that have closely spaced columns may present uncertainty to the analyst when deciding on what primary vertical support to be removed in the analysis process. In this type of structure some of the columns are likely to be architectural and not a structural column. Structures that have this type of structural configuration shall be analyzed for a loss in support from both the architectural column as well as the structural column to assess the potential for progressive collapse. In the situation where structural columns are closely spaced, the structure should be analyzed for the loss of both columns if the distance between the columns is less than or equal to 30% of the longest dimension of the associated bay. Otherwise, only the loss of one column shall be required in the analysis.

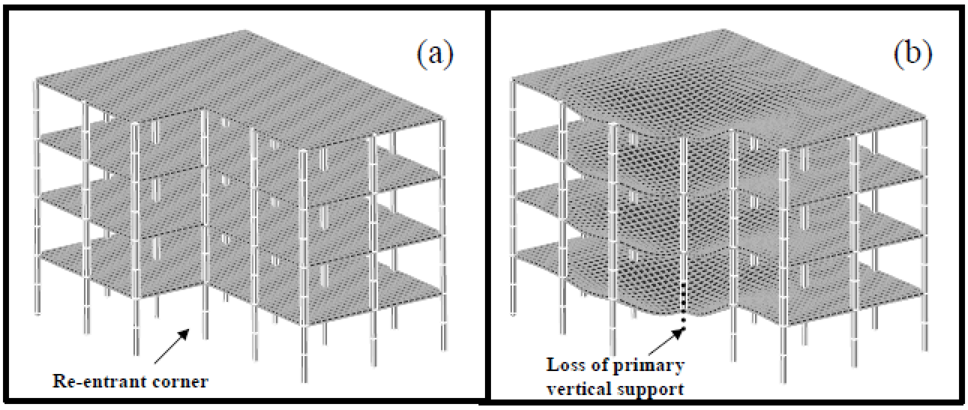


Figure 3.6: Response of the Structure before and after Loss of Vertical Support in the Re-entrant Corner

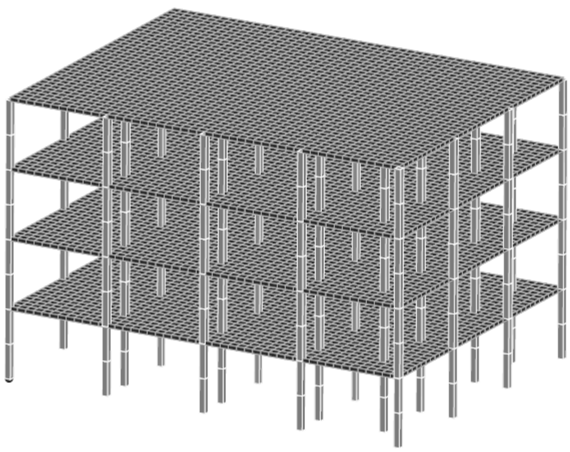


Figure 3.7: Building with Closely Spaced Columns

3.6 Acceptance Criteria

An examination of the linear elastic analysis results shall be performed to identify the magnitudes and distribution of potential demands on both the primary and secondary structural elements for quantifying potential collapse areas. The magnitude and distribution of these demands will be indicated by Demand-Capacity Ratios (DCR).

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{3.1}$$

Where,

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

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

Using the DCR criteria of the linear elastic approach, structural elements and connections that have DCR values that exceed the following allowable values are considered to be severely damaged or collapsed.

The allowable DCR values for primary and secondary structural elements are:

- DCR $<$ 2.0 for typical structural configurations
- DCR $<$ 1.5 for atypical structural configurations

3.7 Summary

Various consideration of General Services Administration (GSA 2003) guidelines[13] for progressive collapse evaluation are discussed in this chapter. Important parameters regarding progressive collapse analysis such as static and dynamic analysis loading, analysis procedure, internal and external column removal consideration and acceptance criteria are discussed as suggested by GSA guidelines.

Chapter 4

Unified Facilities Criteria (UFC) by DoD

4.1 General

Department of Defense (DoD) of United States of America (USA) published the document Unified Facilities Criteria (UFC) 4-023-03[12] "Design of Buildings to Resist Progressive Collapse" on 29th May 2002 for first time. Several changes are made in provisions of UFC 4-023-03 over a period of time. Department of Defense published revised copy of UFC 4-023-03 in 2005 and lastly in July 2009. The UFC provides detailed guidelines for analysis procedures for RC, steel, masonry and wood structures.

4.2 Analysis Approaches

The guideline provides three analysis approaches, namely, Tie Force Method, Alternate Path Method and Enhanced Local Resistance Method for progressive collapse design requirements.

4.2.1 Tie Force Method (TF) :

In the Tie Force approach, the building is mechanically tied together, enhancing continuity, ductility, and development of alternate load paths. Tie forces are typically provided by the existing structural elements and connections that are designed using conventional design procedures to carry the standard loads imposed upon the structure. Depending upon the construction type, there are several horizontal ties that must be provided: internal, peripheral, and ties to edge columns, corner columns, and walls. Vertical ties are required in columns and load-bearing walls. These Tie Forces are different from "Reinforcement ties" as defined in ACI 318 Building Code Requirements for Structural Concrete. The load path for peripheral ties must be continuous around the plan geometry and, for internal ties the load path must be continuous from one edge to the other.

Load Resistance Factor Design for Tie Force : As per Load and Resistance Factor Design (LRFD) approach, the design tie strength provided by a member or its connections to other members is taken as the product of the strength reduction factor ϕ and the nominal tie strength R_n calculated in accordance with the requirements and assumptions of applicable material specific codes. As per the LRFD approach, the design tie strength must be greater than or equal to the required tie strength:

$$\phi R_n \geq X_i F_i \tag{4.1}$$

Where,

ϕ = Strength reduction factor

R_n = Nominal Tie Strength calculated with the appropriate material specific code

ϕR_n = Required Tie Strength

X_i = Load Factor

F_i = Load Effect

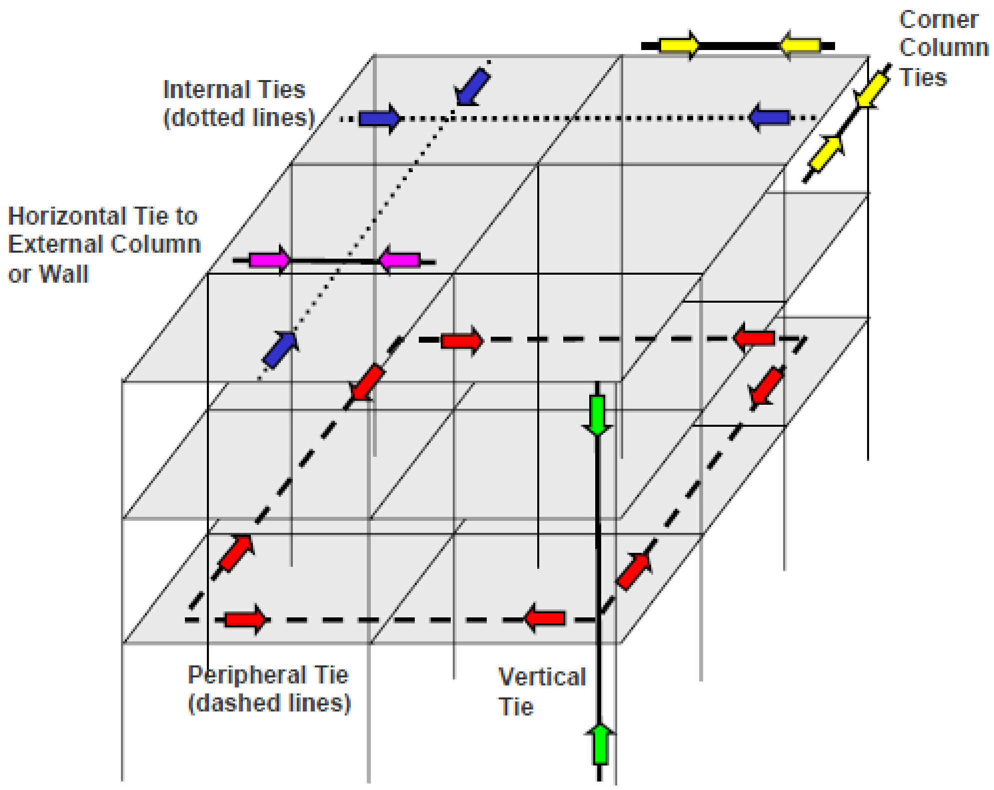


Figure 4.1: Tie Force in a Frame Structure

Uniform Floor Load : Floor load to determine the required strengths is calculated by:

$$W_f = (1.2DL + 0.5LL) \tag{4.2}$$

Where,

W_f = Floor Load (kN/m^2)

DL = Dead Load (kN/m^2)

LL = Live Load (kN/m^2)

Structural Elements and Connections With Inadequate Design Tie Strength:

If the vertical design tie strength of any structural element or connection is less than the vertical required tie strength, the designer must either revise the design to meet the tie force requirements or use the Alternate Path method to prove that the structure is capable of bridging over this deficient element. The Alternate Path shall not be applied to structural elements or connections that cannot provide the horizontal required tie strength. In this case, the designer must redesign or retrofit the element and connection such that a sufficient design tie strength is developed.

4.2.2 Alternate Path Method (AP) :

The Alternate Path method is used in two situations; (A) for Option 1 of Occupancy Category (OC) II and for Occupancy Category (OC) IV, when a vertical structural element cannot provide the required tie strength; the designer may use the alternate path (AP) method to determine if the structure can bridge over the deficient element after it has been notionally removed. (b) For Occupancy Category (OC) II Option 2, Occupancy Category (OC) III, and Occupancy Category (OC) IV, the Alternate Load Path method must be applied for the removal of specific vertical load-bearing elements. UFC suggests three analysis procedures to perform alternate load path method; (1) linear static (2) nonlinear static and (3) nonlinear dynamic.

Removal of Load-Bearing Element : Load-bearing elements are removed for the following two cases; (a) For OC II Option 1 and OC IV structures, where an element cannot provide the required vertical tie strength (b) For OC II Option 2, OC III, and OC IV structures, where Alternate Load Path is applied to elements for which the location and size are specified to verify that the structure has adequate flexural resistance to bridge over the missing element. For both external and internal column removal, for the purposes of AP analysis, beam-to-beam continuity is assumed to be maintained across a removed column as shown in Figure 4.2

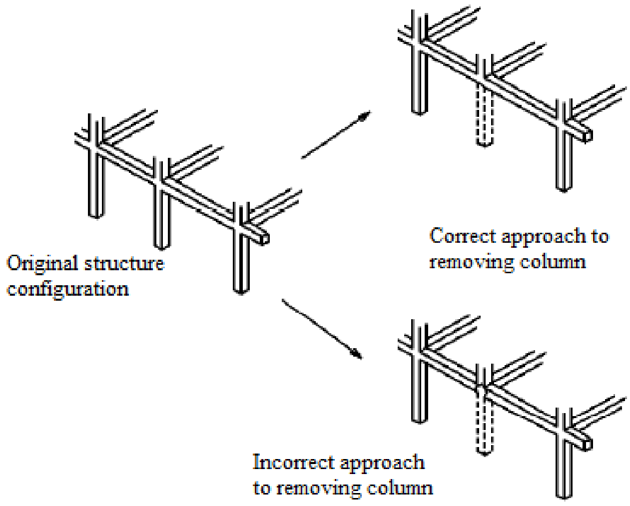


Figure 4.2: Correct and Incorrect Approach to Remove a Column

Consideration for column removal: For each plan location defined for element removal, an AP analysis is performed for:

A. First storey above grade

B. Storey directly below roof

C. Storey at mid-height

D. Storey above the location of a column splice or change in column size

For example, if a corner column is specified as the removed element location in a ten storey building with a column splice at the third storey, one AP analysis is performed for removal of the ground storey corner column; another AP analysis is performed for the removal of the corner column at the tenth storey; another AP analysis is performed for the fifth storey corner column (mid-height storey) and one AP analysis is performed for the fourth storey corner column (storey above the column splice). Figure 4.3 and Figure 4.4 shows the location of the external and internal column removal from the structure respectively.

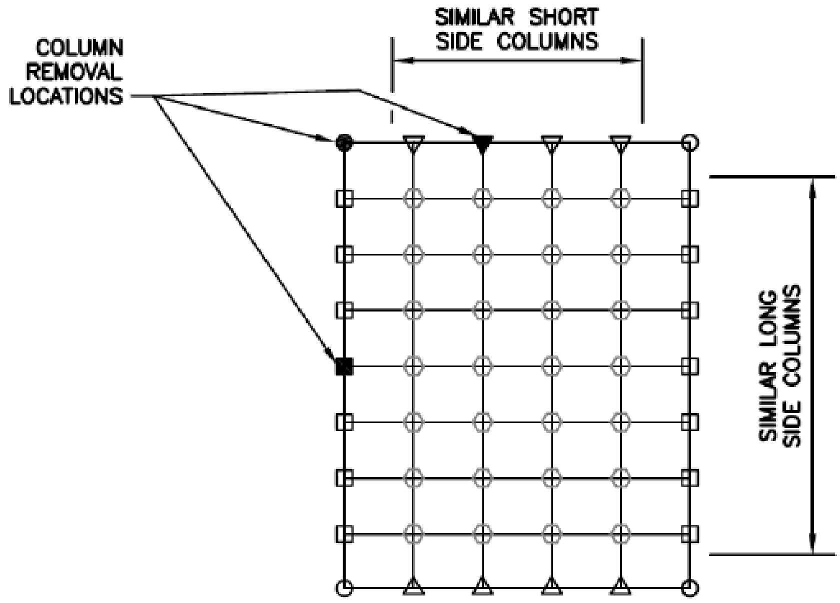


Figure 4.3: Location of External Column Removal for Structure

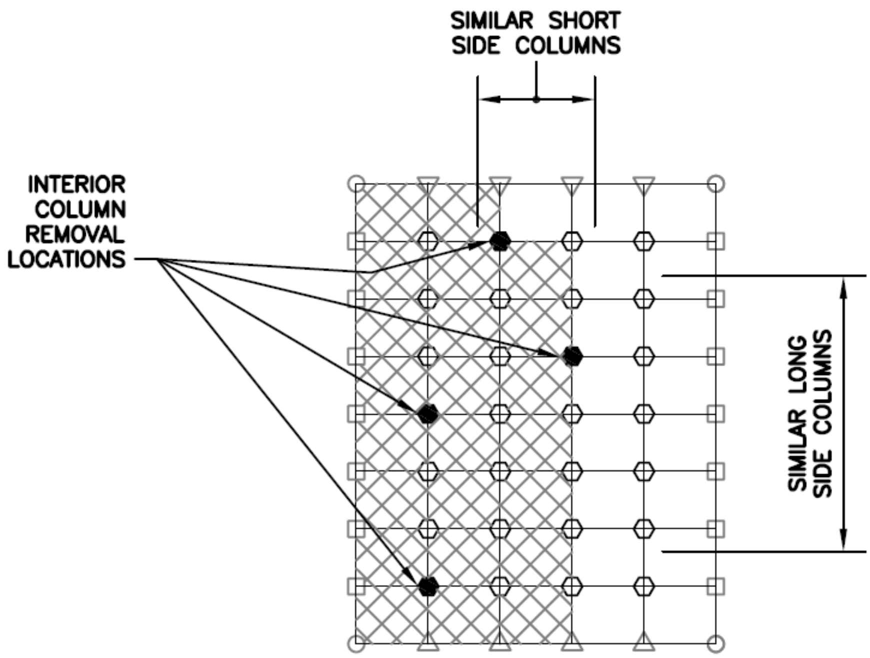


Figure 4.4: Location of Internal Column Removal for Structure

Analysis Loading : The U.S. General Service Administration guidelines recommended that the following downward loads be applied when assessing the potential for progressive collapse.
For static analysis :

$$G = 2(0.9 or 1.2)DL + (0.5LL or 0.2S) \tag{4.3}$$

For dynamic analysis :

$$G = (0.9 or 1.2)DL + (0.5LL or 0.2S) \tag{4.4}$$

Where,
G = Gravity Load
DL = Dead Load
LL = Live Load
S = Snow Load

4.2.3 Enhanced Local Resistance (ELR) :

The Enhanced Local Resistance (ELR) is required in three cases: OC II Option 1 (Tie Forces and ELR), OC III (Alternate Path and ELR), and OC IV (Tie Forces, Alternate Path and ELR). ELR is provided through the prescribed flexural and shear resistance of perimeter building columns and load bearing walls. The flexural resistance is defined as the magnitude of uniform load acting over the height of the wall or load-bearing column which causes flexural failure, i.e. the formation of a three hinge mechanism or similar failure mode. In calculating the flexural resistance, consider any effects (axial load, compression membrane behaviour, ends conditions, etc) that may act to increase the flexural resistance. The shear resistance of the column, load-bearing wall, and their connections must be equal to or greater than the shear capacity associated with the baseline flexural resistance, i.e., application of the uniform load that defines the baseline flexural resistance must not fail the column, load-bearing wall or their connections and splices in shear.

4.3 Comparison between Guidelines

In this section comparison between "General Services Administration (GSA 2003) Guideline[13]" and "Department of Defense (DoD) of United States of America (USA) Guideline[12]" to evaluate the progressive collapse potential is presented. Table compares load combinations from these two standards to perform progressive collapse analysis.

Table 1: Load combination For Progressive Collapse Analysis

Standards	**Load combination after column removal**
GSA	2(D + 0.25L) static analysis (D + 0.25L) dynamic analysis
DoD UFC 4-023-03	D +0.5L net floor uplift
	2(0.9 or 1.2) D + (0.5L or 0.2S) + 0.2W (NLD) static analysis (0.9 or 1.2) D + (0.5L or 0.2S) + 0.2W dynamic analysis

4.4 Summary

Various consideration of Unified Facilities Criteria (UFC) published by Department of Defense (DoD)[12] for design of building to resist progressive collapse is discussed in this chapter. Tie force method, Alternate load path method and Enhanced local resistance method is discussed as per revised UFC 4-023-03. Also, comparison between "General Services Administration (GSA 2003) Guideline[13]" and "Department of Defense (DoD) of United States of America (USA) Guideline[12]" to evaluate the progressive collapse potential is presented.

Chapter 5

Analysis of 10-storey Symmetric Building

5.1 General

To resist abnormal loadings progressive collapse analysis is necessary to evaluate the capacity of a structure. When a structure has its loading pattern changed progressive collapse occurs. To study the failure effect of primary structural component on the entire structure, one 10-storey symmetrical reinforced concrete (RC) building is analyzed for progressive collapse by using structural analysis and design software Midas-Gen-2012.

5.2 Building Configuration

In this chapter, progressive collapse analysis and its resistance structural systems are discussed. Building is modelled in Midas-Gen-2012.The building is having bay width 5 meter in X-direction and 4-meter in Y-direction as shown in Figure 5.1 Building having first storey height of 3.2 meter and all other storey height of 3 meter as shown in Figure 5.2 Wall of 115 mm thickness are considered on all the beams.

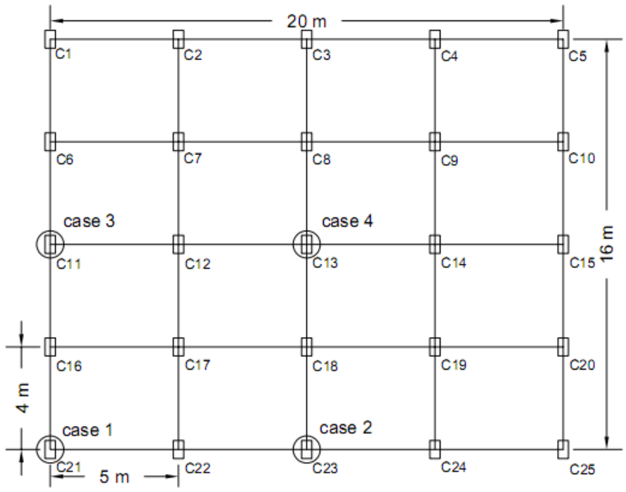


Figure 5.1: Plan of 10-storey building

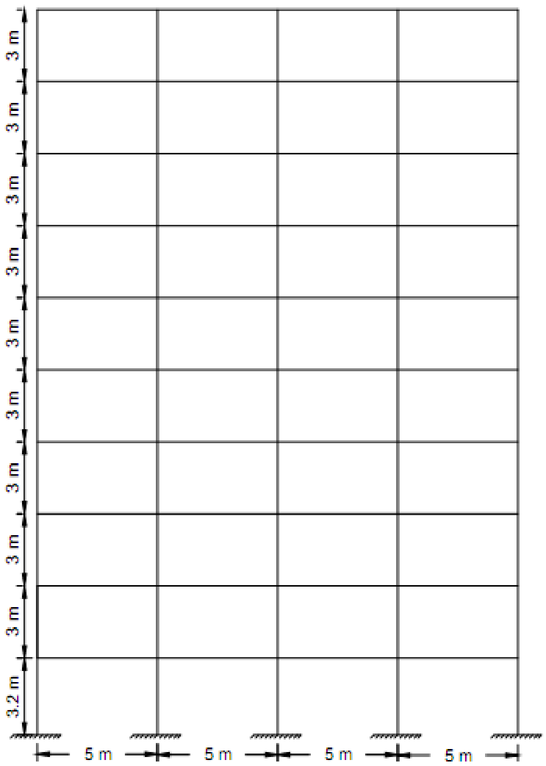


Figure 5.2: Elevation of 10-storey building

5.3 Loading Data

- **Loading parameters :**

a. Gravity loading :

Dead load : Self weight of the structural elements

Live load at typical floor : 3.0 (kN/m^2)

Live load at terrace floor : 2.0 (kN/m^2)

Floor finish at typical floor : 1.25 (kN/m^2)

Floor finish at terrace floor : 1.5 (kN/m^2)

Wall load : 7.5 (kN/m^2)

b. Seismic loading :

Z =0.16 (zone III) [Table 2 , IS 1893 (Part 1) : 2002]

Soil type =II (Ahmedabad)

I =1 [Table 6 , IS 1893 (Part 1) : 2002]

R =5 [Table 7 , IS 1893 (Part 1) : 2002]

T =0.075 $h^{0.75}$ = 0.9662

S_a/g =1.36/T =1.4075

A_h = 0.0225

c. Material properties :

Grade of concrete : M25

Grade of steel : Fe415

5.4 Load Combinations

Following load combinations of the member forces will be considered for arriving at the design forces.

- 1.5 (DL + LL)
- 1.2 (DL + $LL \pm EQx$) and 1.2 (DL + $LL \pm EQy$)
- 1.5 ($DL \pm EQx$) and 1.5 ($DL \pm EQy$)
- ($0.9DL \pm 1.5EQx$) and ($0.9DL \pm 1.5EQz$)

5.5 Preliminary Design of Building

Figure 5.1 shows the plan of 10-storey building and Figure 5.2 shows the elevation of 10-storey building.

- Beams : 300 x 400 M25 at all typical floors
- Columns : 400 x 700 M25 at all typical floors
- Slabs : 125 M25 - at all typical floors

The reinforcement detail for slab section is shown in Figure 5.3

The reinforcement detail of beam is shown in Figure 5.4 and also cross-section of beam is shown in Figure 5.5
The reinforcement detail of column is shown in Figure 5.6 and also cross-section of column is shown in Figure 5.7

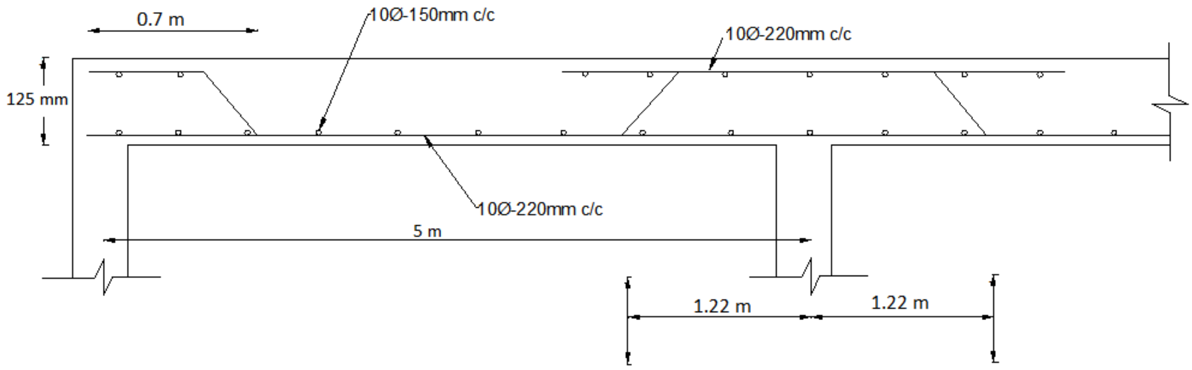


Figure 5.3: Section of slab

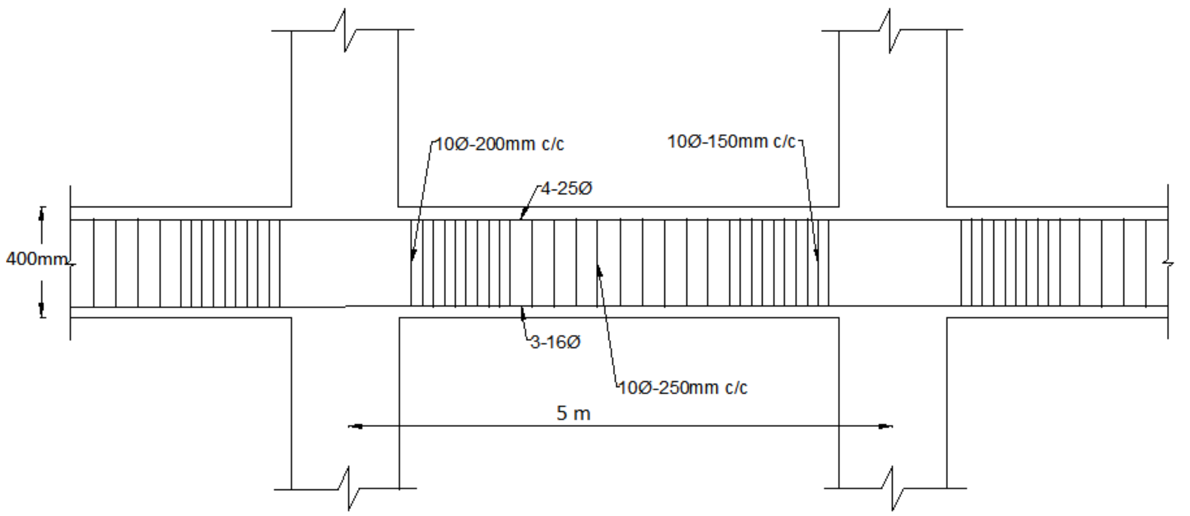


Figure 5.4: Longitudinal section of beam

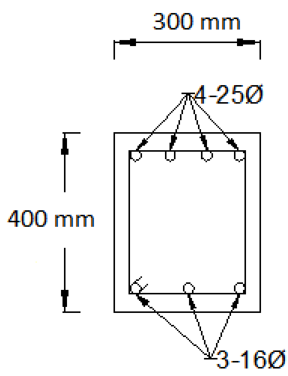


Figure 5.5: Cross-section of beam

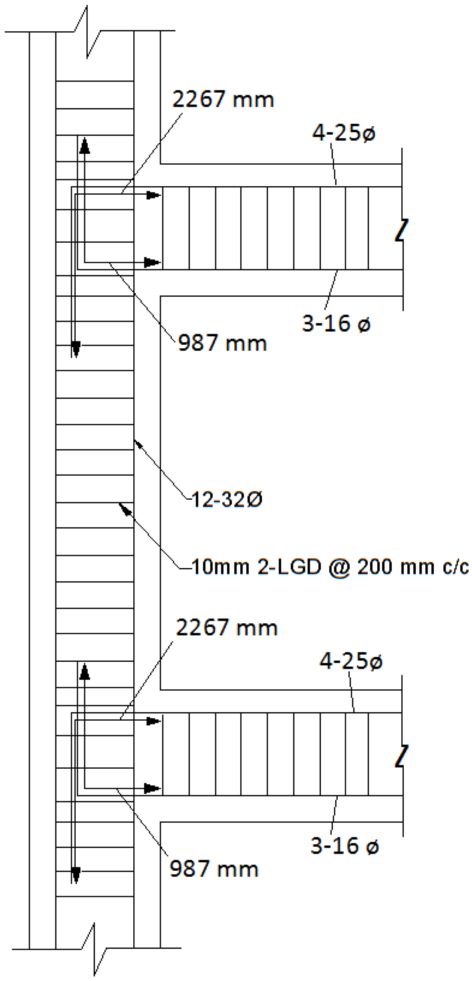


Figure 5.6: Longitudinal section of column

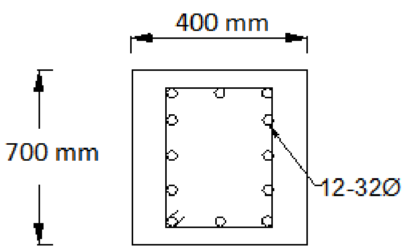


Figure 5.7: Section of column

5.6 Analysis for Progressive Collapse

The column removal scenario is created after completed the design of building. Based on exterior and interior column removal scenario four cases have been considered. Linear static analysis is performed using analysis program Midas Gen-2012.

5.6.1 Linear Static Analysis

In this analysis column or columns are removed from the considered location analysis is carried out for vertical load as per define in the guidelines.

GSA guideline, Load = 2(DL+0.25LL)
UFC guideline, Load = 2(1.2DL+0.5LL)

Steps:

- Build a finite element model,
- Apply the static load combination,
- Perform static linear analysis.

5.7 Calculation of Demand Capacity Ratio (DCR)

For the DCR determined the demand at critical point and capacity of the designed section. The Demand Capacity Ratio of each member is calculated from equation :

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{5.1}$$

Where,
Q_{UD} = Acting force (demand) determined in component or connection/joint

Q_{CE} = Expected ultimate, un-factored capacity of the component and/or connection/joint

The allowable DCR values for primary and secondary structural elements are:

- DCR < 2.0 for typical structural configurations
- DCR < 1.5 for atypical structural configurations

5.7.1 Demand Capacity Ratio (DCR) for Flexure

Figure 5.8 shows the bending moment in beams after column removal case-2.
Beams : 300 mm x 400 mm

Area of steel in beam above column removal for case-2,
A_{st} = 602.88 mm^2 (3-16ϕ)
f_{ck} = 1.25 x 25 = 31.25 N/mm^2
f_y = 1.25 x 415 = 518.75 N/mm^2

$$M_u = 0.87 f_y A_{st} d(1 - (\frac{A_{st} f_y}{b d f_{ck}})) \tag{5.2}$$

M_u = 99.758 kNm
DCR for flexure = $\frac{238.1}{99.758}$ = 2.38

5.7.2 Demand Capacity Ratio (DCR) for Shear

Figure 5.9 shows the shear force in beams after column removal case-2.
Beams : 300 mm x 400 mm

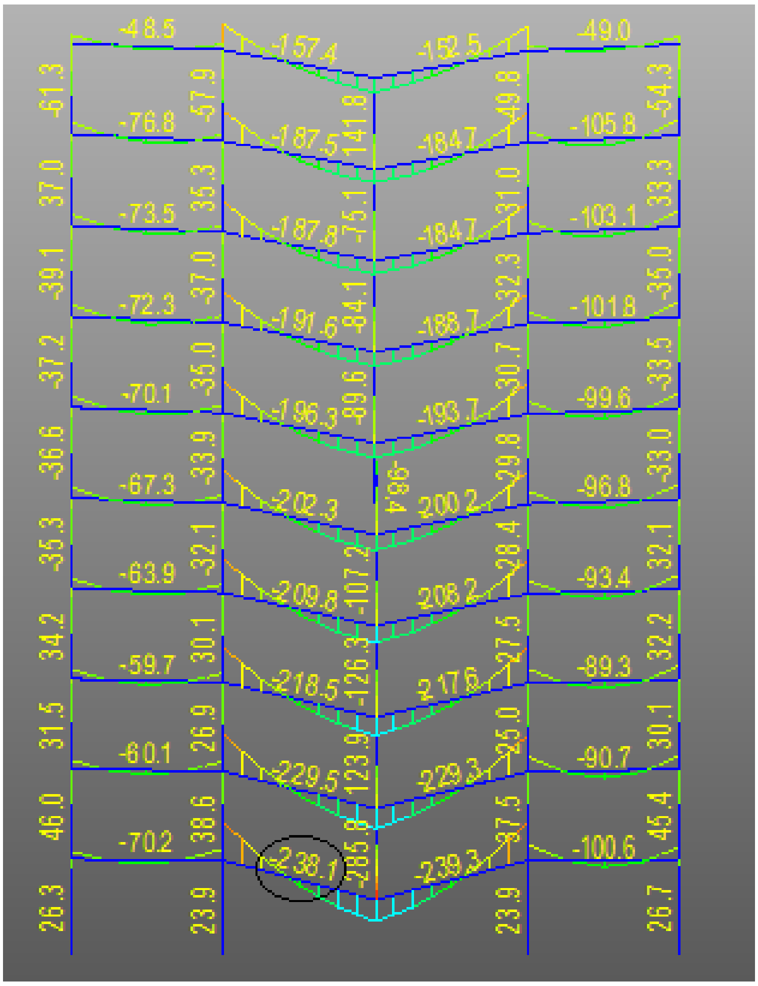


Figure 5.8: Bending moment in beams after column removal case 2

For shear, provide 10ϕ -2 lgd at 200 mm c/c

$A_{sv} = 157\ mm^2$

$S_v = 200$ mm

$f_y = 1.25$ x $415 = 518.75\ N/mm^2$

Shear resisted by shear reinforcement,

$$V_{us} = (\frac{0.87 f_y A_{sv} d}{S_v}) \tag{5.3}$$

$V_{us} = 141.715$ kN

Shear resisted by concrete,

$$V_c = (\tau_c bd) \tag{5.4}$$

$p_t = 0.528\%$

$\tau_c = 0.49$

$V_c = 58.8$ kN

Total shear, $V_s = V_{us} + V_c = 200.512$ kN

DCR for shear $= \frac{138.6}{200.512} = 0.691$

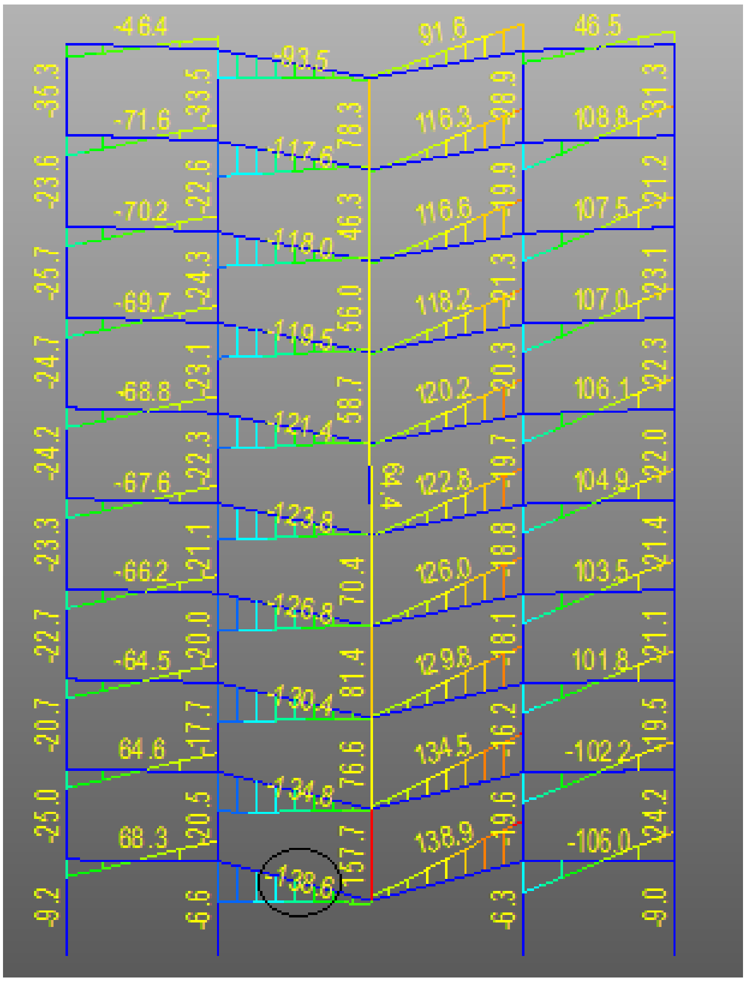


Figure 5.9: Shear force in beams after column removal case 2

5.7.3 Demand Capacity Ratio (DCR) for Column

Due to one column removal redistribution of forces occurs, so forces in nearby column i.e. axial force, moment about major and minor axis, changes.
Columns (c18 at top storey for case 2) : 400 mm x 700 mm

Area of steel in column, $A_{st} = 9650.4\ mm^2$
$f_{ck} = 25 = 25\ N/mm^2$
$f_y = 415 = 415\ N/mm^2$
$p_t = 3.4\ \%$

Load and Moment in column,

$P_u = 219.477$ kN	$P_{uz} = 5386.79$ kN
$M_{ux} = 73.17$ kNm	$M_{ux1} = 835.806$ kNm
$M_{uy} = 4.24$ kNm	$M_{uy1} = 47.4$ kNm

Demand capacity ratios for columns are calculated according to IS 456:2000 Clause 39.6 which states that the resistance of a member subjected to axial force and biaxial bending is represented by this equation.

$$(\frac{M_{ux}}{M_{ux1}})^{\alpha_n} + (\frac{M_{uy}}{M_{uy1}})^{\alpha_n} \leq 1.0 \tag{5.5}$$

$$(\frac{73.17}{835.806})^{0.734} + (\frac{4.24}{47.4})^{0.734} = 0.335 \leq 1.0 \tag{5.6}$$

Where,
M_{ux}, M_{uy} = moments about x and y axes due to design loads,
M_{ux1}, M_{uy1} = maximum uniaxial moment capacity for an axial load of P_u, bending about x and y axes respectively,
α_n is depends on P_u/P_{uz}

5.8 Analysis Results

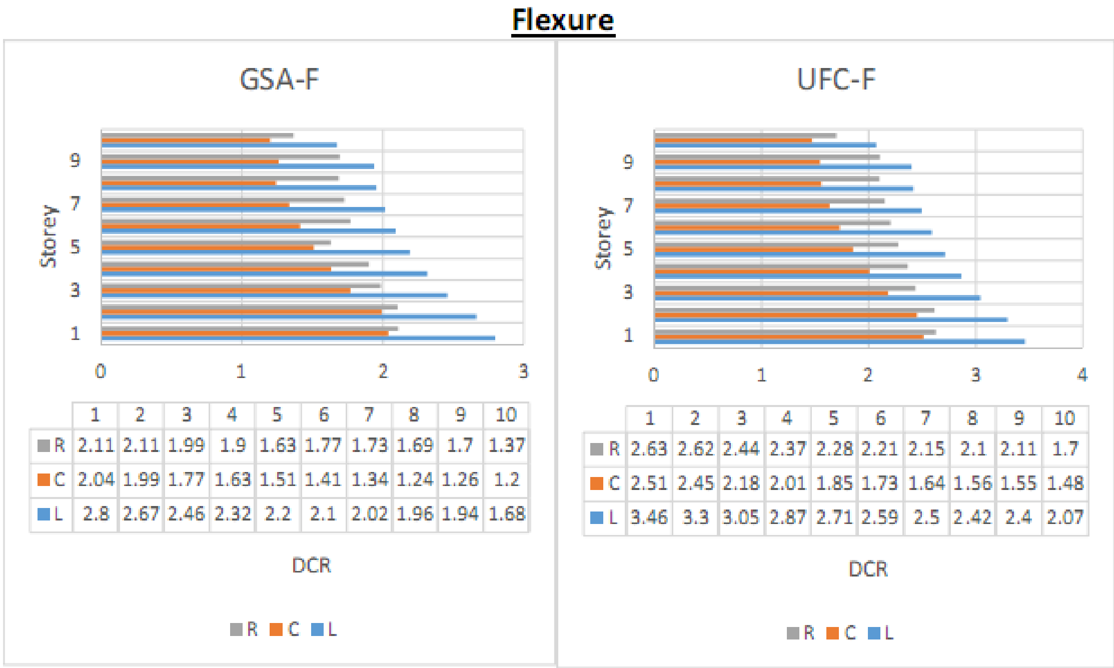
The linear static analysis of 10-storey RCC building has been perform using GSA and DoD guidelines. Demand Capacity Ratio (DCR) is obtain at critical location by alternate column removing scenario.

For case 1 corner column of the building, case 2 middle column from long side of the building, case 3 middle column from short side of the building, case 4 interior column of the building are removed.

DCR are calculated at each storey of the building. Also DCR are calculated at three point right (R), center (C) and left (L). From both the guidelines, GSA and DoD the permissible value of DCR for flexure is 2 and DCR for shear is 1.

5.8.1 Flexure Analysis

DCR for flexure for all the column removal cases are shown in figure 5.10 to figure 5.13. Result represented that value of DCR in flexure for beam exceed the limit of 2 for both GSA and UFC load cases. Value of DCR in flexure for beam by linear static analysis is higher on left and right side of the column removal point for most of cases.



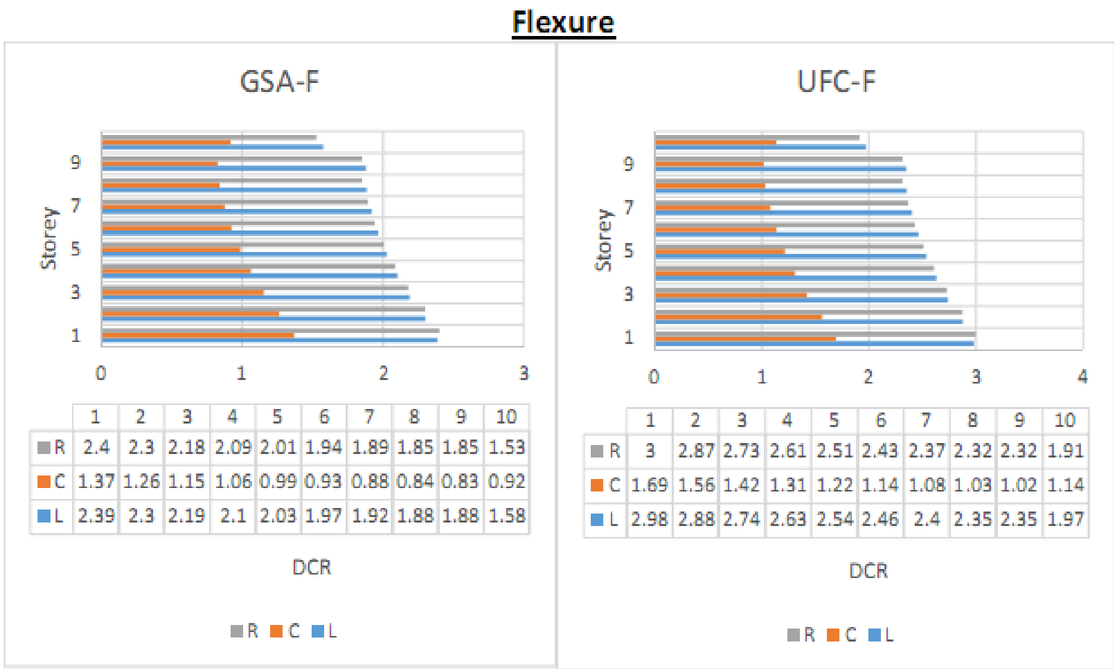
GSA-F

	1	2	3	4	5	6	7	8	9	10
R	2.11	2.11	1.99	1.9	1.63	1.77	1.73	1.69	1.7	1.37
C	2.04	1.99	1.77	1.63	1.51	1.41	1.34	1.24	1.26	1.2
L	2.8	2.67	2.46	2.32	2.2	2.1	2.02	1.96	1.94	1.68

UFC-F

	1	2	3	4	5	6	7	8	9	10
R	2.63	2.62	2.44	2.37	2.28	2.21	2.15	2.1	2.11	1.7
C	2.51	2.45	2.18	2.01	1.85	1.73	1.64	1.56	1.55	1.48
L	3.46	3.3	3.05	2.87	2.71	2.59	2.5	2.42	2.4	2.07

Figure 5.10: DCR for flexure for case 1



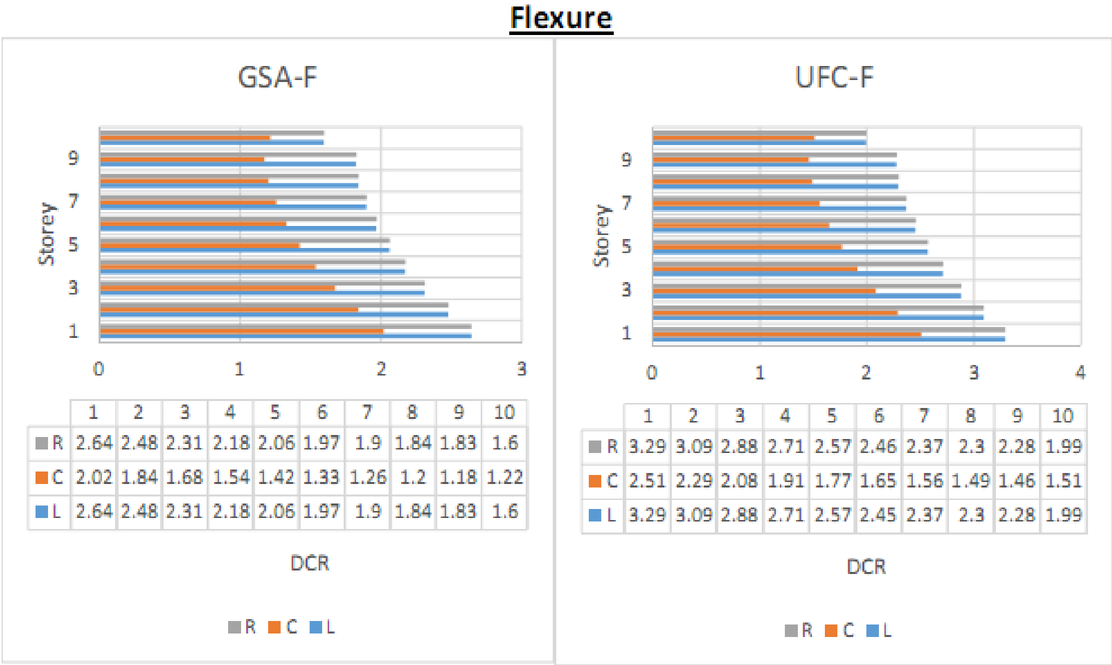
GSA-F

	1	2	3	4	5	6	7	8	9	10
R	2.4	2.3	2.18	2.09	2.01	1.94	1.89	1.85	1.85	1.53
C	1.37	1.26	1.15	1.06	0.99	0.93	0.88	0.84	0.83	0.92
L	2.39	2.3	2.19	2.1	2.03	1.97	1.92	1.88	1.88	1.58

UFC-F

	1	2	3	4	5	6	7	8	9	10
R	3	2.87	2.73	2.61	2.51	2.43	2.37	2.32	2.32	1.91
C	1.69	1.56	1.42	1.31	1.22	1.14	1.08	1.03	1.02	1.14
L	2.98	2.88	2.74	2.63	2.54	2.46	2.4	2.35	2.35	1.97

Figure 5.11: DCR for flexure for case 2



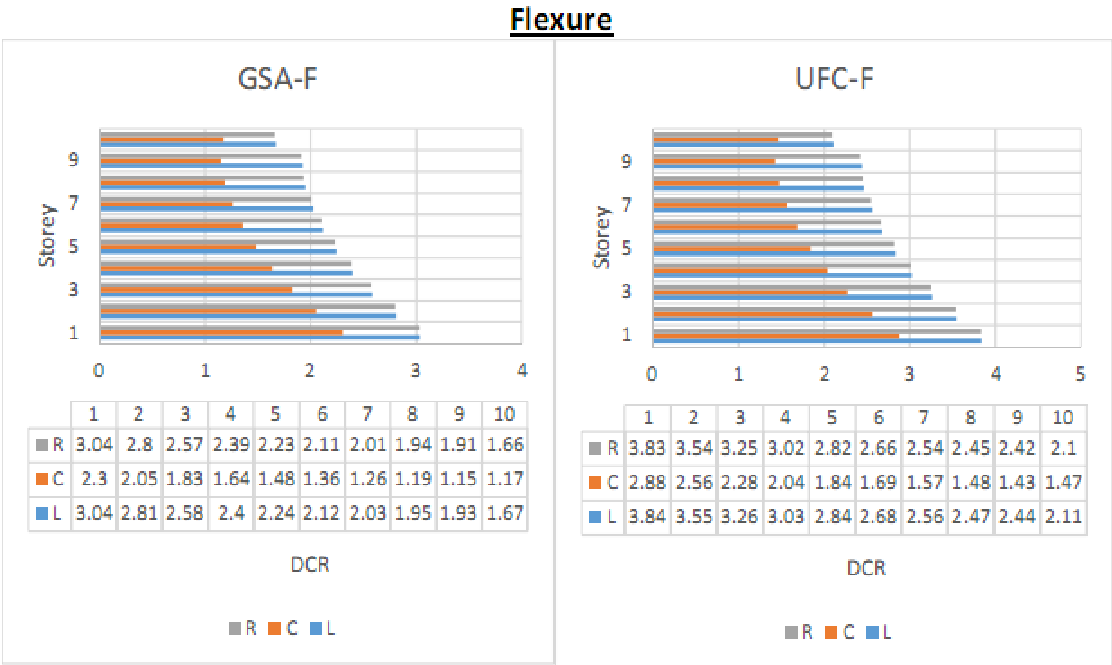
GSA-F

	1	2	3	4	5	6	7	8	9	10
R	2.64	2.48	2.31	2.18	2.06	1.97	1.9	1.84	1.83	1.6
C	2.02	1.84	1.68	1.54	1.42	1.33	1.26	1.2	1.18	1.22
L	2.64	2.48	2.31	2.18	2.06	1.97	1.9	1.84	1.83	1.6

UFC-F

	1	2	3	4	5	6	7	8	9	10
R	3.29	3.09	2.88	2.71	2.57	2.46	2.37	2.3	2.28	1.99
C	2.51	2.29	2.08	1.91	1.77	1.65	1.56	1.49	1.46	1.51
L	3.29	3.09	2.88	2.71	2.57	2.45	2.37	2.3	2.28	1.99

Figure 5.12: DCR for flexure for case 3



GSA-F

	1	2	3	4	5	6	7	8	9	10
R	3.04	2.8	2.57	2.39	2.23	2.11	2.01	1.94	1.91	1.66
C	2.3	2.05	1.83	1.64	1.48	1.36	1.26	1.19	1.15	1.17
L	3.04	2.81	2.58	2.4	2.24	2.12	2.03	1.95	1.93	1.67

UFC-F

	1	2	3	4	5	6	7	8	9	10
R	3.83	3.54	3.25	3.02	2.82	2.66	2.54	2.45	2.42	2.1
C	2.88	2.56	2.28	2.04	1.84	1.69	1.57	1.48	1.43	1.47
L	3.84	3.55	3.26	3.03	2.84	2.68	2.56	2.47	2.44	2.11

Figure 5.13: DCR for flexure for case 4

5.8.2 Shear Analysis

DCR for shear for all the column removal cases are shown in figure 5.14 to figure 5.17. Result represented that value of DCR in shear for beam is within the limit of 1 for GSA load cases and exceed the limit of 1 for UFC load cases.

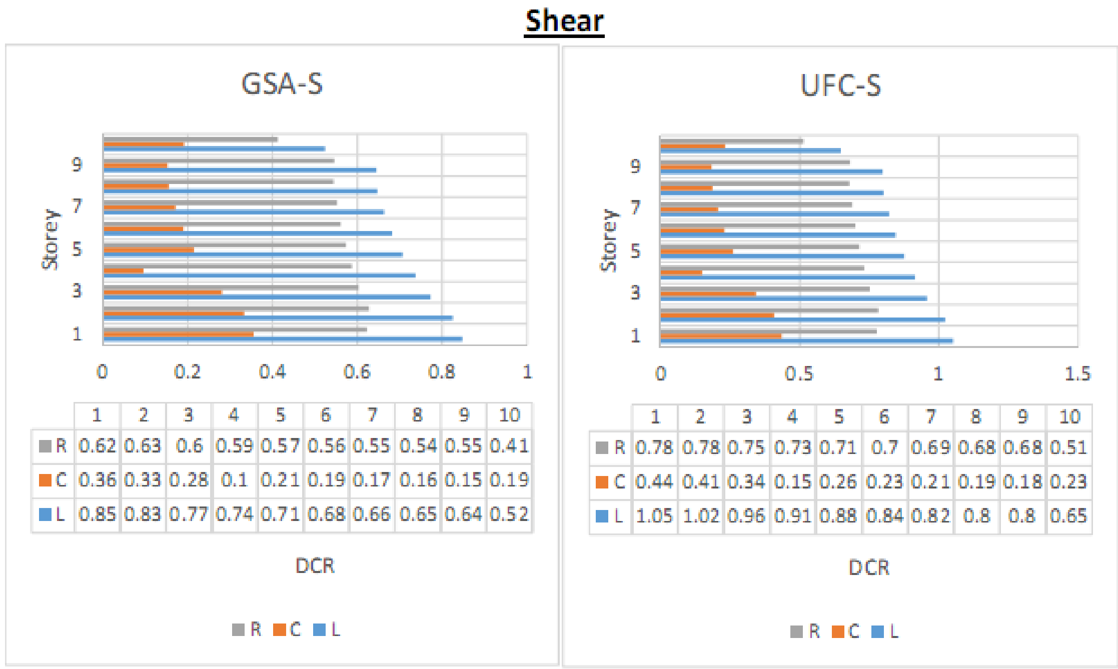


Figure 5.14: DCR for shear for case 1

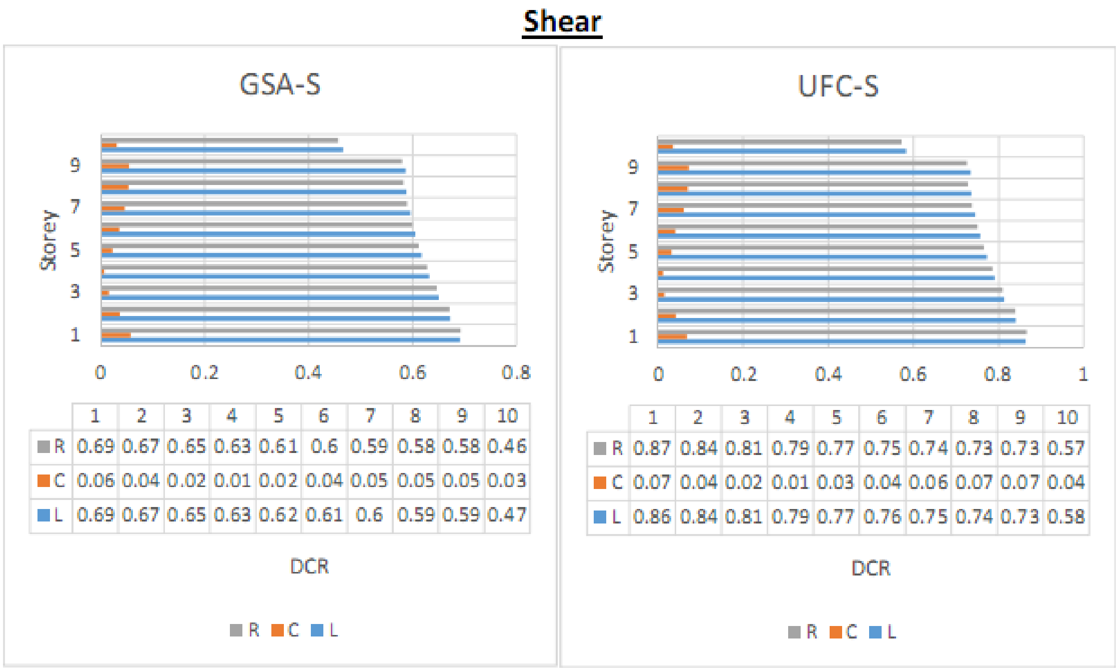


Figure 5.15: DCR for shear for case 2

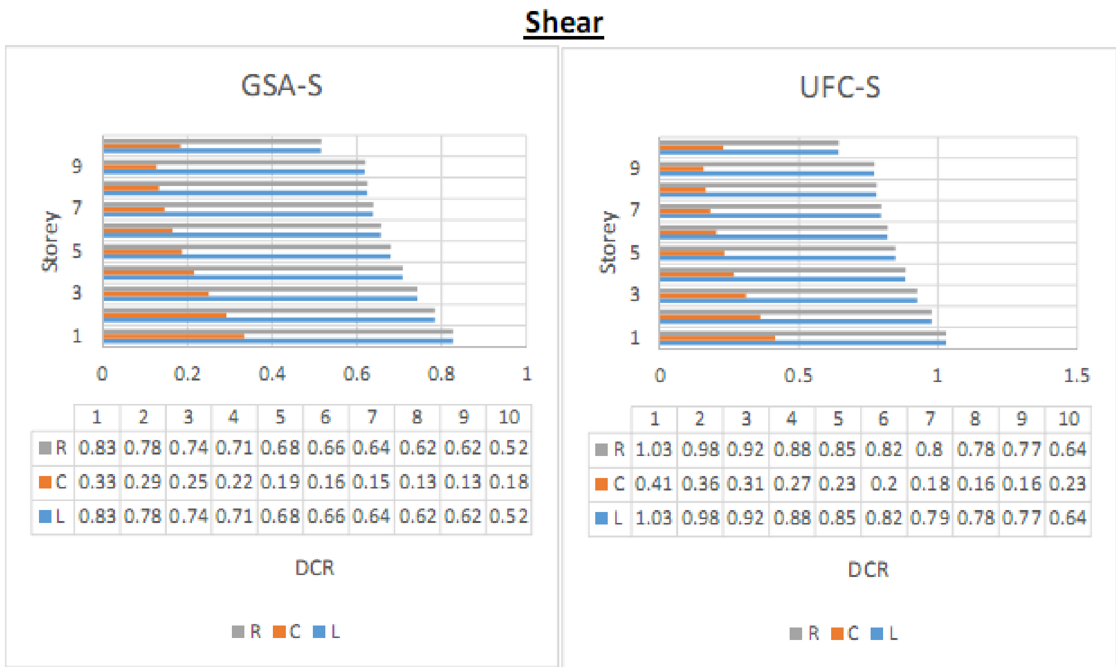


Figure 5.16: DCR for shear for case 3

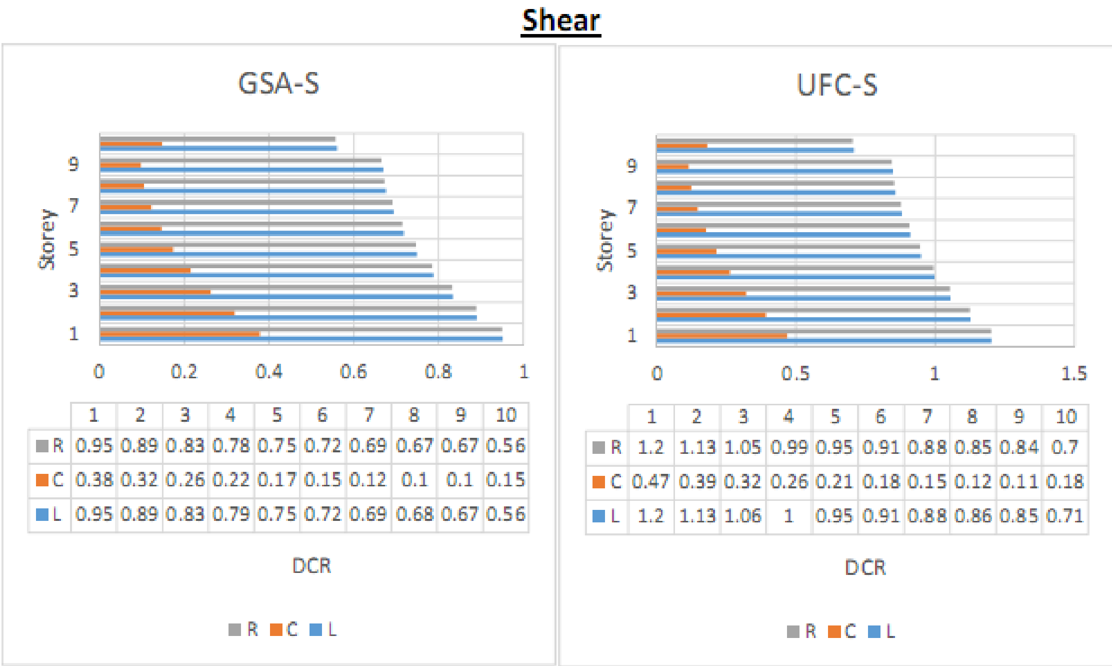


Figure 5.17: DCR for shear for case 4

5.8.3 Column Removal Analysis

Effect of column removal on other proximity column has been observed for ten storey building . Demand capacity ratios for all the four cases is calculated for one proximity column. Demand capacity ratios for column for all the cases is shown in figure 5.18 to figure 5.21.DCR for column exceed the allowable limit of 1.0 at bottom two to four stories. Also, values of DCR for column for all cases in UFC guideline is having higher values compared to GSA guideline.

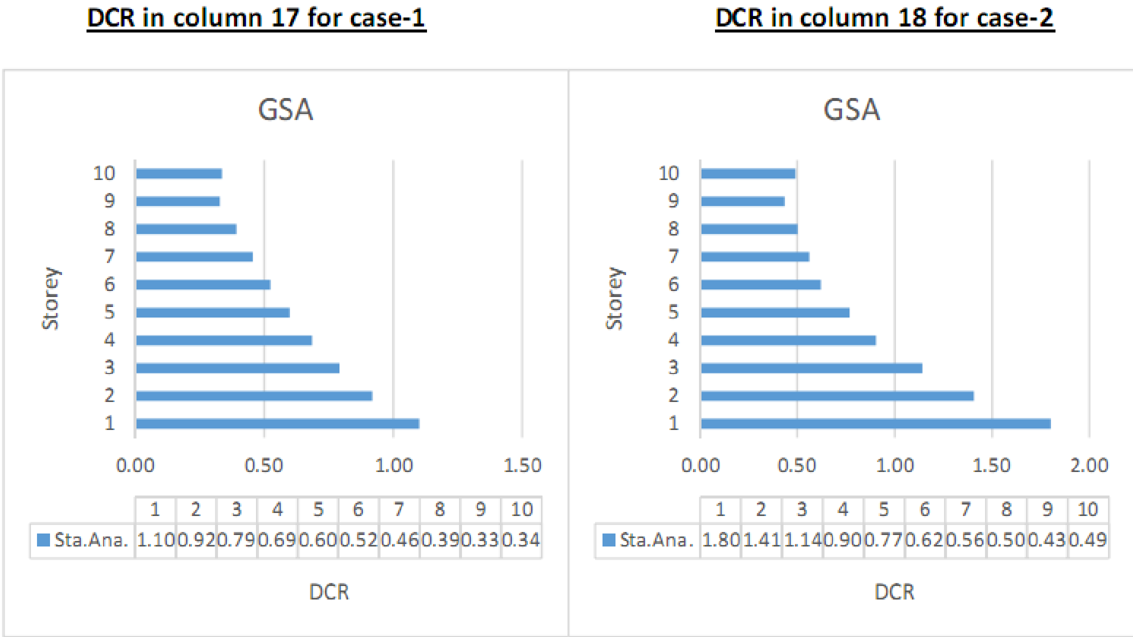


Figure 5.18: DCR in column for case 1 and case 2 for GSA

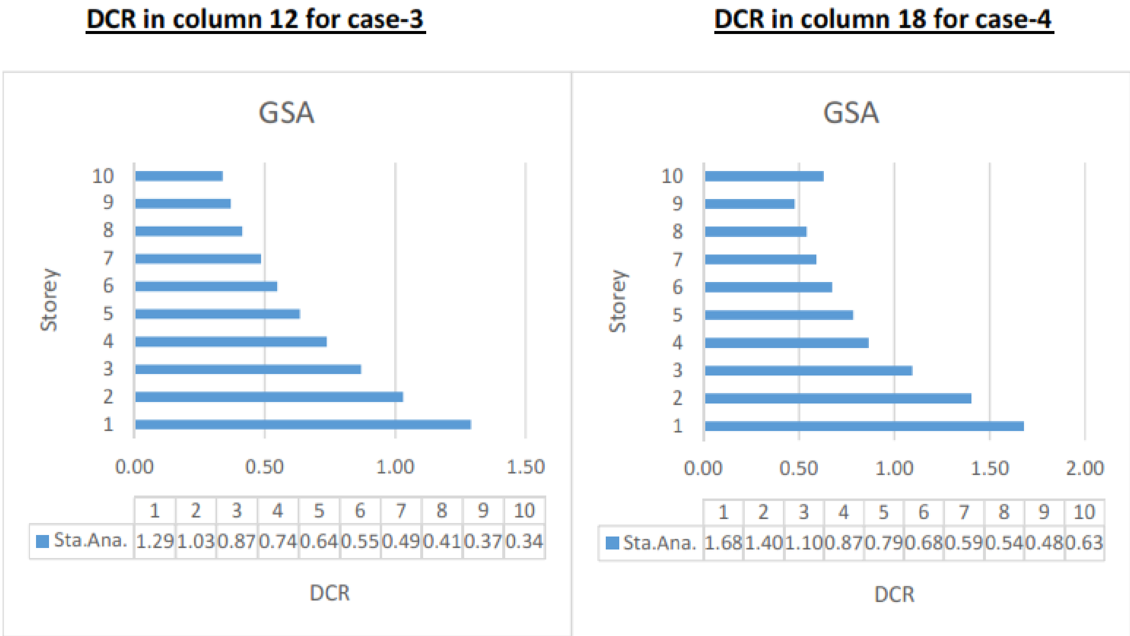


Figure 5.19: DCR in column for case 3 and case 4 for GSA

DCR in column 17 for case-1 | DCR in column 18 for case-2

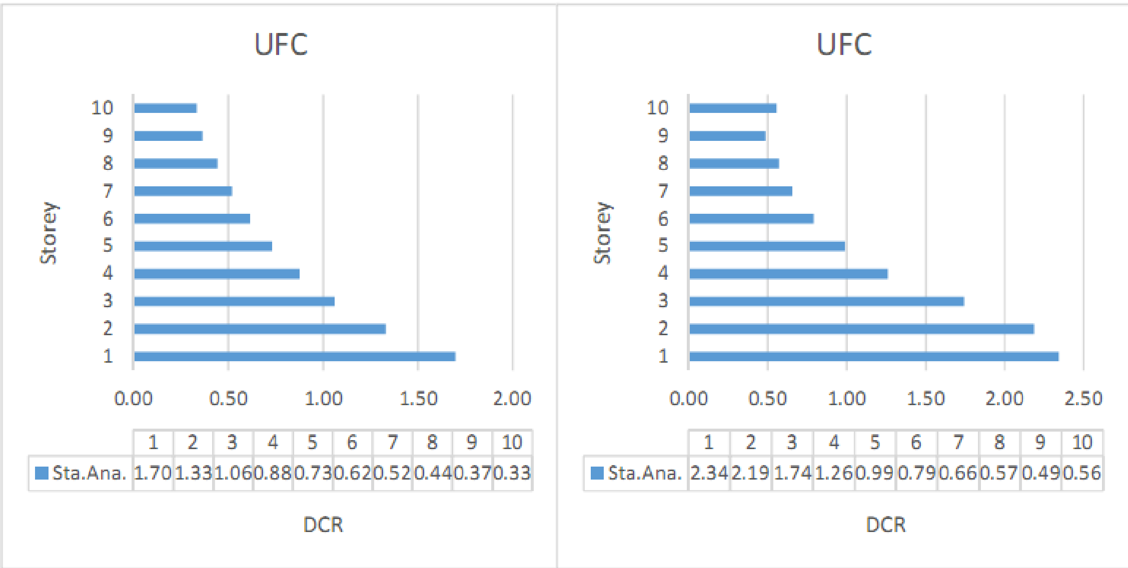


Figure 5.20: DCR in column for case 1 and case 2 for UFC

DCR in column 12 for case-3 | DCR in column 18 for case-4

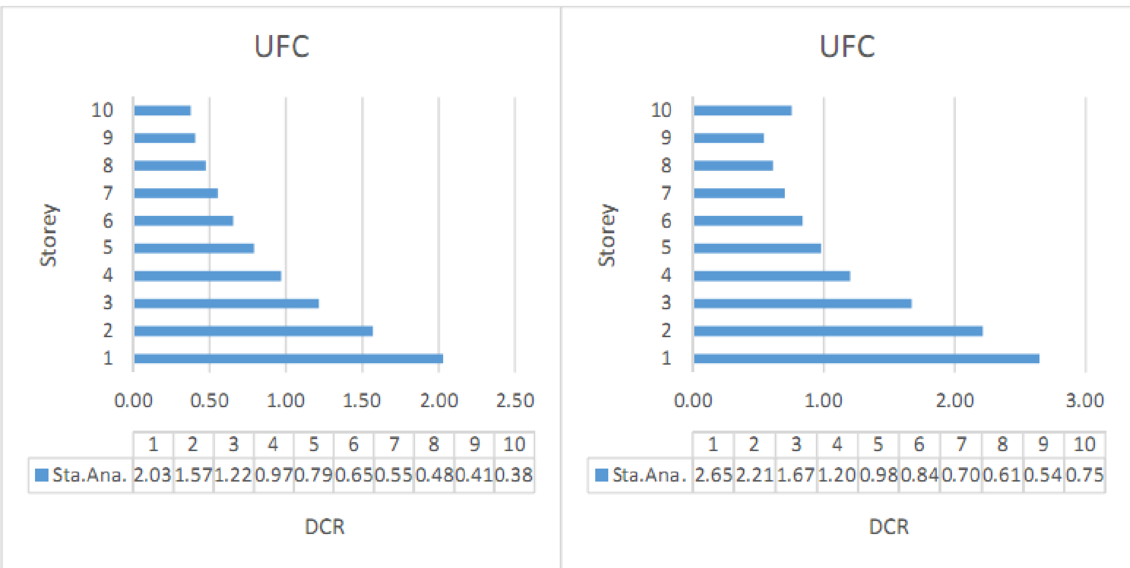


Figure 5.21: DCR in column for case 3 and case 4 for UFC

5.8.4 Displacement Analysis

Displacement under column removal point for all cases in UFC guideline is greater compare to GSA guideline as shown in figure 5.22.

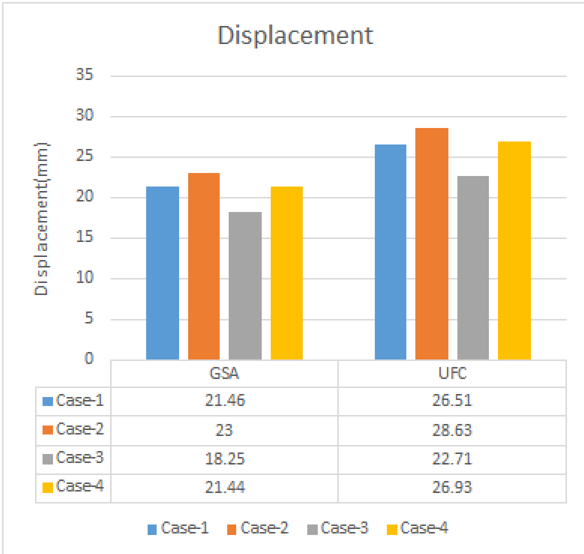


Figure 5.22: Displacement under column removal point for all cases

5.9 Summary

In this chapter, one 10-storey symmetrical reinforced concrete (RC) building is analyzed for progressive collapse potential by using structural analysis and design software Midas-Gen-2012. Linear static analysis is performed by U.S. General Service Administrator (GSA) and Department of Defense (DoD) guidelines using column removal scenario. Demand Capacity Ratio (DCR) is calculated for flexure and shear in beam are carried out for Symmetrical 10-storey reinforced concrete building. The DCR calculated by linear static analysis is compared by both the guideline GSA and UFC for all cases,case 1 corner column of the building, case 2 middle column from long side of the building, case 3 middle column from short side of the building, case 4 interior column. Effect of column removal on other proximity column is observed for ten storey building. The displacement calculated by linear static analysis is compared by both the guideline GSA and UFC for all cases.

Chapter 6

Structural System of 10-storey Symmetric Building

6.1 General

If building is having high potential of progressive collapse it is important to mitigate the vulnerability of progressive collapse. Mitigation is also referred as structural robustness. Structural robustness is an ability of structure to resist abnormal loadings. Progressive collapse analysis is necessary to evaluate the capacity of a structure.

In chapter 5, one 10-storey symmetrical reinforced concrete (RC) building is analyzed for progressive collapse potential by Midas-Gen-2012. It is observed that from the analysis result, DCR for all the cases exceed the permissible limit. As per the guideline the considered building is having high potential of progressive collapse. In this chapter three different types of alternative structural systems are considered to mitigate the progressive collapse of the building.

6.2 Mitigation of Progressive Collapse of 10-storey Building

It is important to mitigate the vulnerability of progressive collapse if building is having high potential of progressive collapse. For the important building it is very necessary to reduce the progressive collapse of the building. From the value of Demand Capacity Ratio (DCR) potential of progressive collapse of building is determined. Guidelines have specified acceptance criteria for different building configuration. Building is having high potential for progressive collapse if DCR for beam and column exceed the permissible value.

For minimization of potential for progressive collapse necessary structural changes are required. In this chapter three alternatives are executed to minimize the potential of progressive collapse of 10-storey symmetric RCC building. These three alternatives are provided as follows:

Alternative 1: Provision of bracing at top storey level of the building.

Alternative 2: Provision of bracing at side face of the building.

Alternative 3: Provision of bracing at top storey level and side face of the building.

Analysis and design of 10-storey symmetrical reinforced concrete (RC) building is analyzed for progressive collapse potential by using structural analysis and design software Midas-Gen-2012 having beam size 300 mm x 400 mm and column size 400 mm x 700 mm. Progressive collapse analysis is performed for four different column removal cases as shown in Figure 5.1. DCR for critical members exceed the permissible limit as specified by guideline. Therefore three different alternatives are executed to minimize the potential of progressive collapse of 10-storey symmetric RCC building.

Bracing of size 300 mm x 300 mm are provided for 10-storey symmetrical building having beam size 300 mm x 400 mm and column size 400 mm x 700 mm. Bracing are provided at top storey level of the building as an alternative 1, at side face of the building as an alternative 2, at top storey level and side face of the building as an alternative 3. Figure 6.1 shows typical elevation of all the three alternatives for 10-storey building.

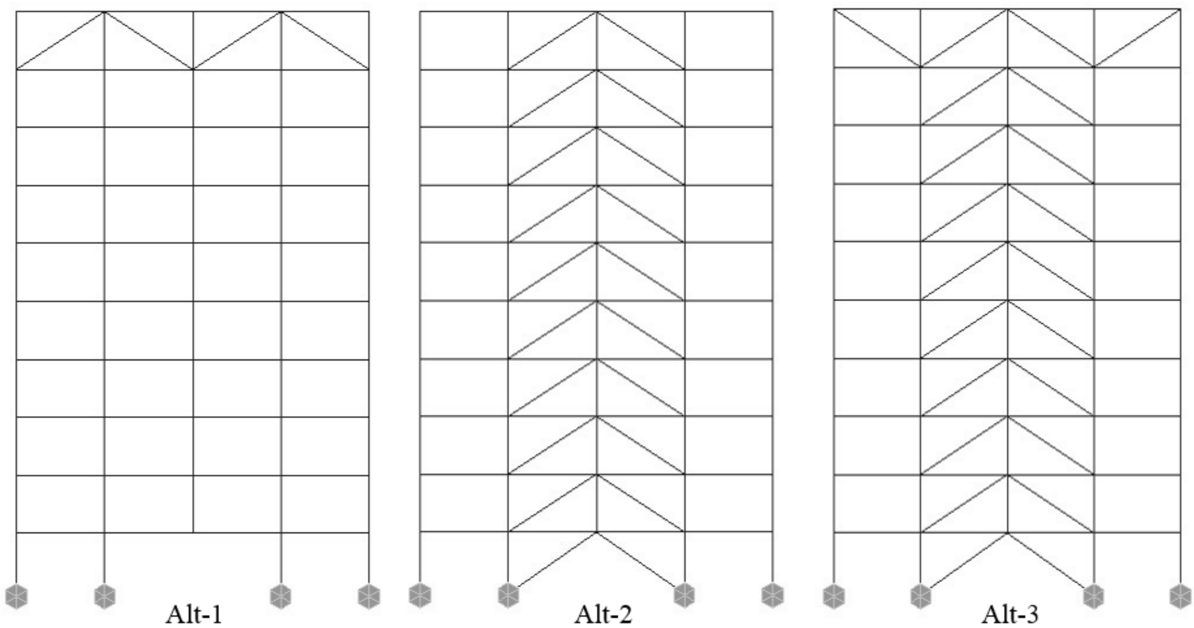


Figure 6.1: Various mitigation alternatives for 10-storey building (case-2)

6.3 Analysis Results

10-storey symmetrical reinforced concrete (RC) building is analyzed for progressive collapse potential. Linear static analysis is performed by U.S. General Service Administrator (GSA) and Department of Defense (DoD) guidelines using column removal scenario. DCR is calculated by considering four different column removal cases as shown in Figure 5.1

DCR obtained by UFC is having higher values compared to those obtained by GSA guidelines for all the above column removal case. Demand Capacity Ratio (DCR) is calculated for flexure and shear in beam are carried out for before and after the mitigation for 10-storey building. The displacement under column removal point is calculated by linear static analysis by both the guideline GSA and UFC for all cases before and after the mitigation for 10-storey building.

Comparison of DCR under column removal point for all the load case before and after mitigation for 10-storey building is shown in figure 6.2 to figure 6.26.

6.3.1 Flexure Analysis

Demand Capacity Ratio (DCR) is calculated for flexure in beam before and after the mitigation for 10-storey building. Figure 6.2 to figure 6.9 shows comparison of DCR for flexure for all load cases before and after the mitigation for 10-storey building. DCR in case of flexure is within the permissible limit of 2.0 in case of GSA and UFC load case which reveals that beams are safe in flexure as per GSA and UFC guidelines except case-4.

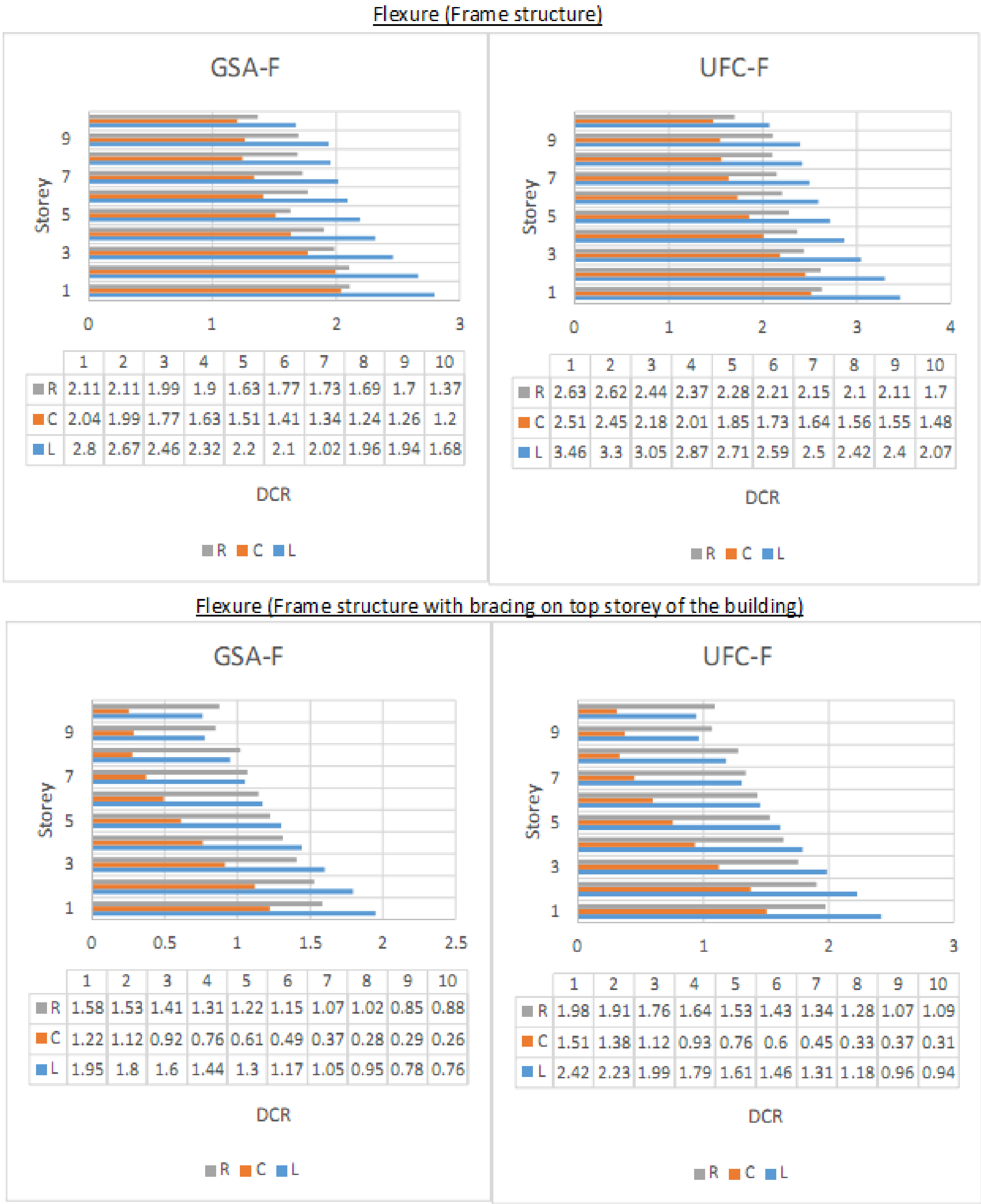


Figure 6.2: DCR for flexure for case 1 for frame structure & frame structure with bracing on top storey of building

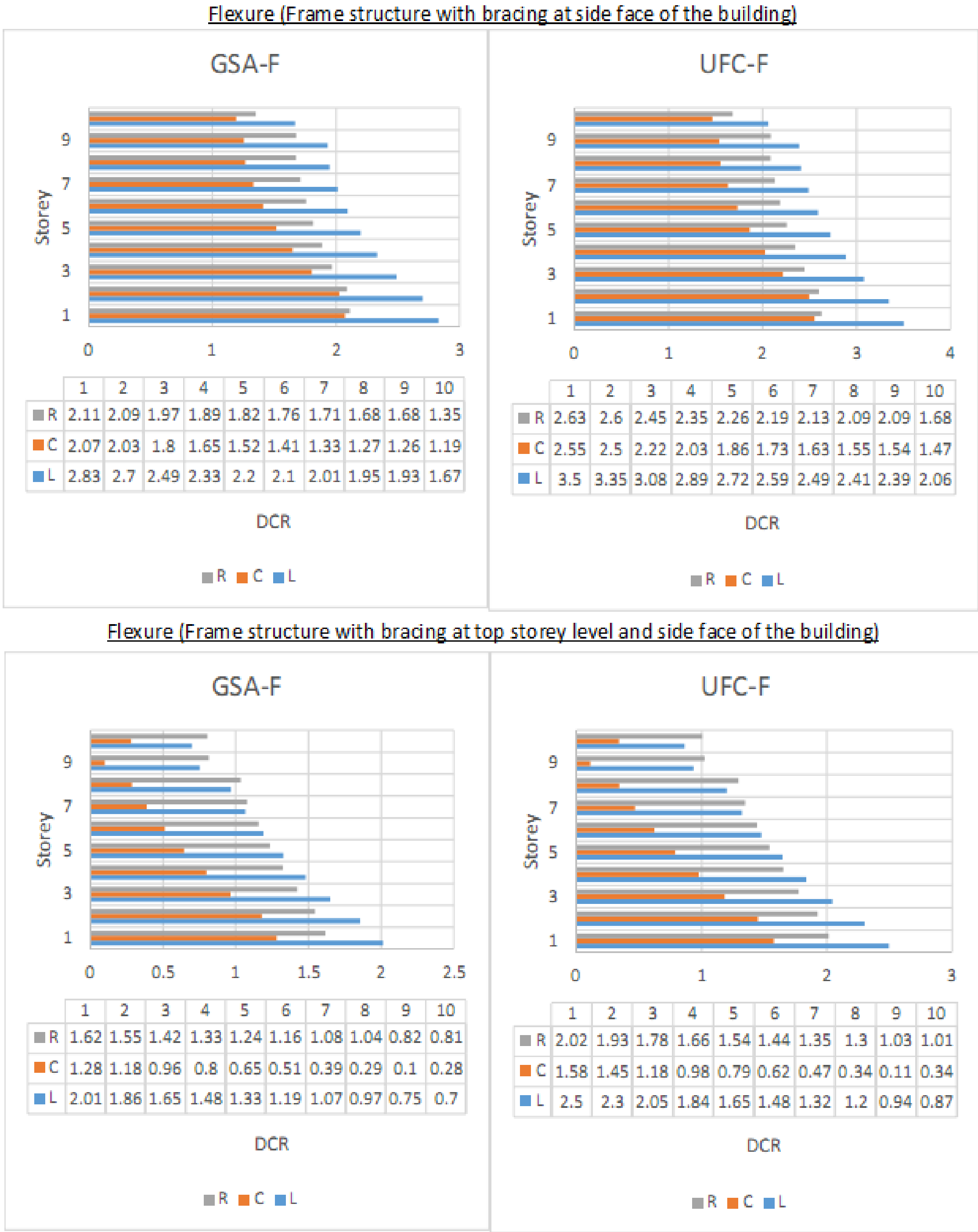
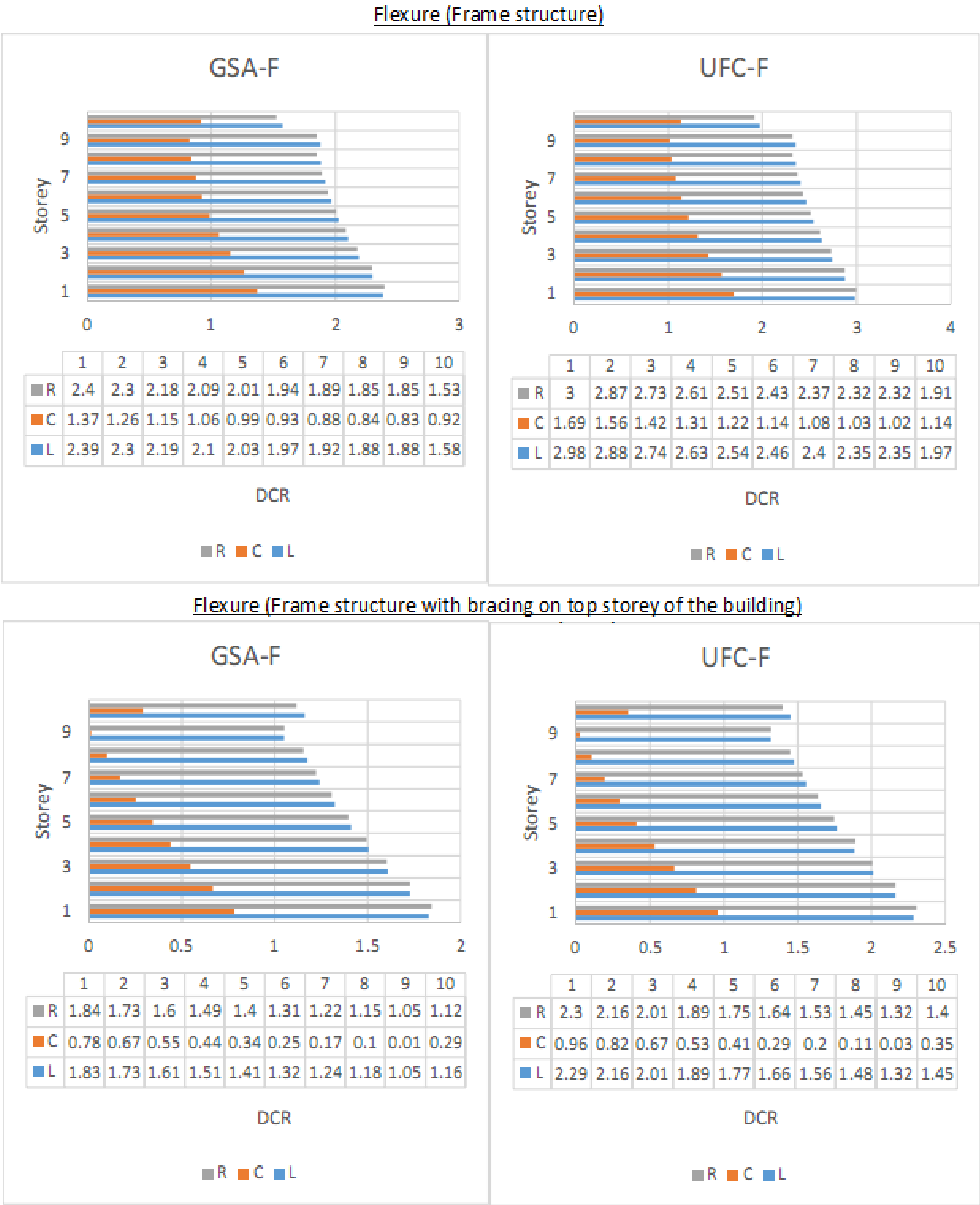


Figure 6.3: DCR for flexure for case 1 for bracing at side face of the building & bracing at top storey level and side face of the building



Flexure (Frame structure) — GSA-F

	1	2	3	4	5	6	7	8	9	10
R	2.4	2.3	2.18	2.09	2.01	1.94	1.89	1.85	1.85	1.53
C	1.37	1.26	1.15	1.06	0.99	0.93	0.88	0.84	0.83	0.92
L	2.39	2.3	2.19	2.1	2.03	1.97	1.92	1.88	1.88	1.58

Flexure (Frame structure) — UFC-F

	1	2	3	4	5	6	7	8	9	10
R	3	2.87	2.73	2.61	2.51	2.43	2.37	2.32	2.32	1.91
C	1.69	1.56	1.42	1.31	1.22	1.14	1.08	1.03	1.02	1.14
L	2.98	2.88	2.74	2.63	2.54	2.46	2.4	2.35	2.35	1.97

Flexure (Frame structure with bracing on top storey of the building) — GSA-F

	1	2	3	4	5	6	7	8	9	10
R	1.84	1.73	1.6	1.49	1.4	1.31	1.22	1.15	1.05	1.12
C	0.78	0.67	0.55	0.44	0.34	0.25	0.17	0.1	0.01	0.29
L	1.83	1.73	1.61	1.51	1.41	1.32	1.24	1.18	1.05	1.16

Flexure (Frame structure with bracing on top storey of the building) — UFC-F

	1	2	3	4	5	6	7	8	9	10
R	2.3	2.16	2.01	1.89	1.75	1.64	1.53	1.45	1.32	1.4
C	0.96	0.82	0.67	0.53	0.41	0.29	0.2	0.11	0.03	0.35
L	2.29	2.16	2.01	1.89	1.77	1.66	1.56	1.48	1.32	1.45

Figure 6.4: DCR for flexure for case 2 for frame structure & frame structure with bracing on top storey of building

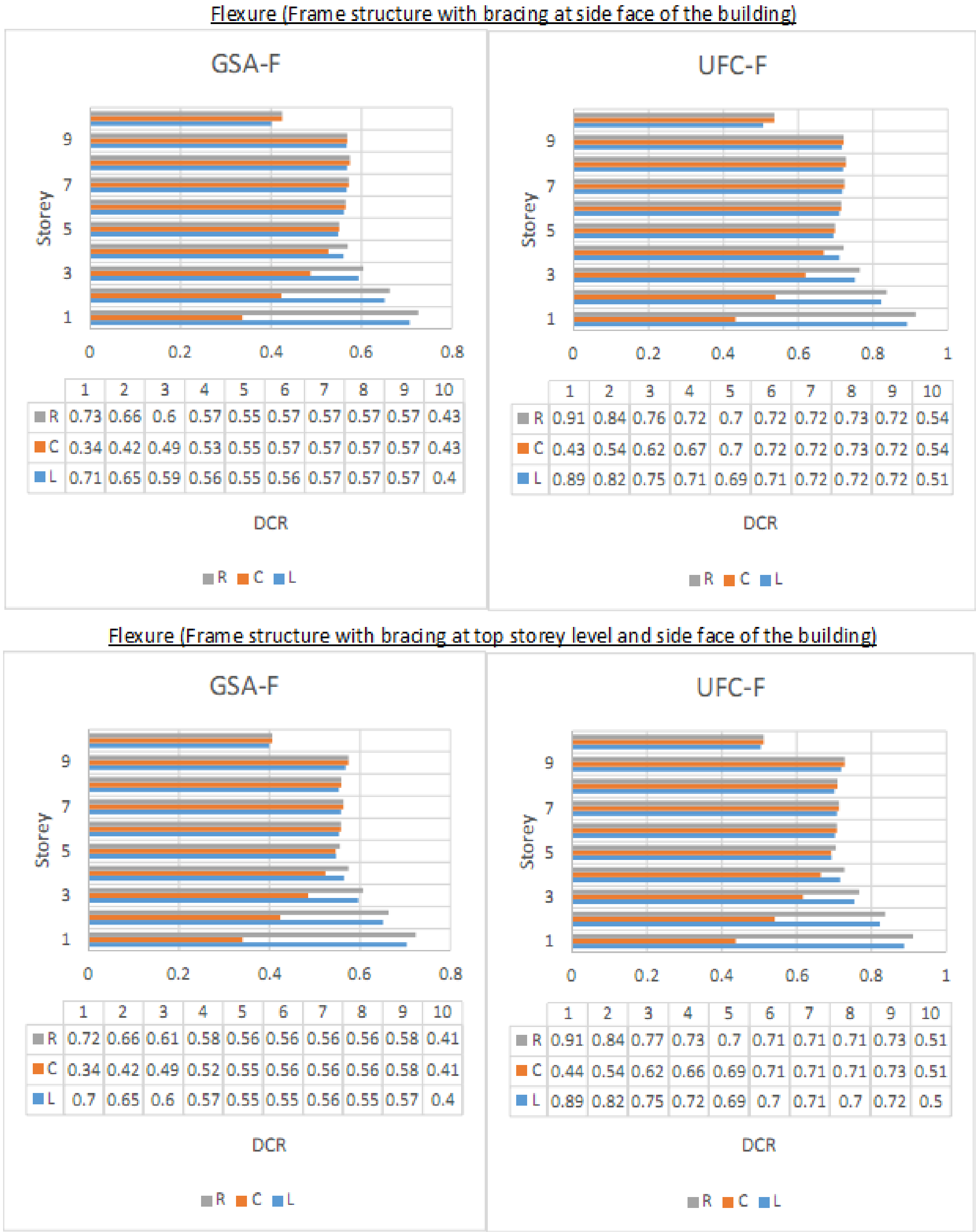
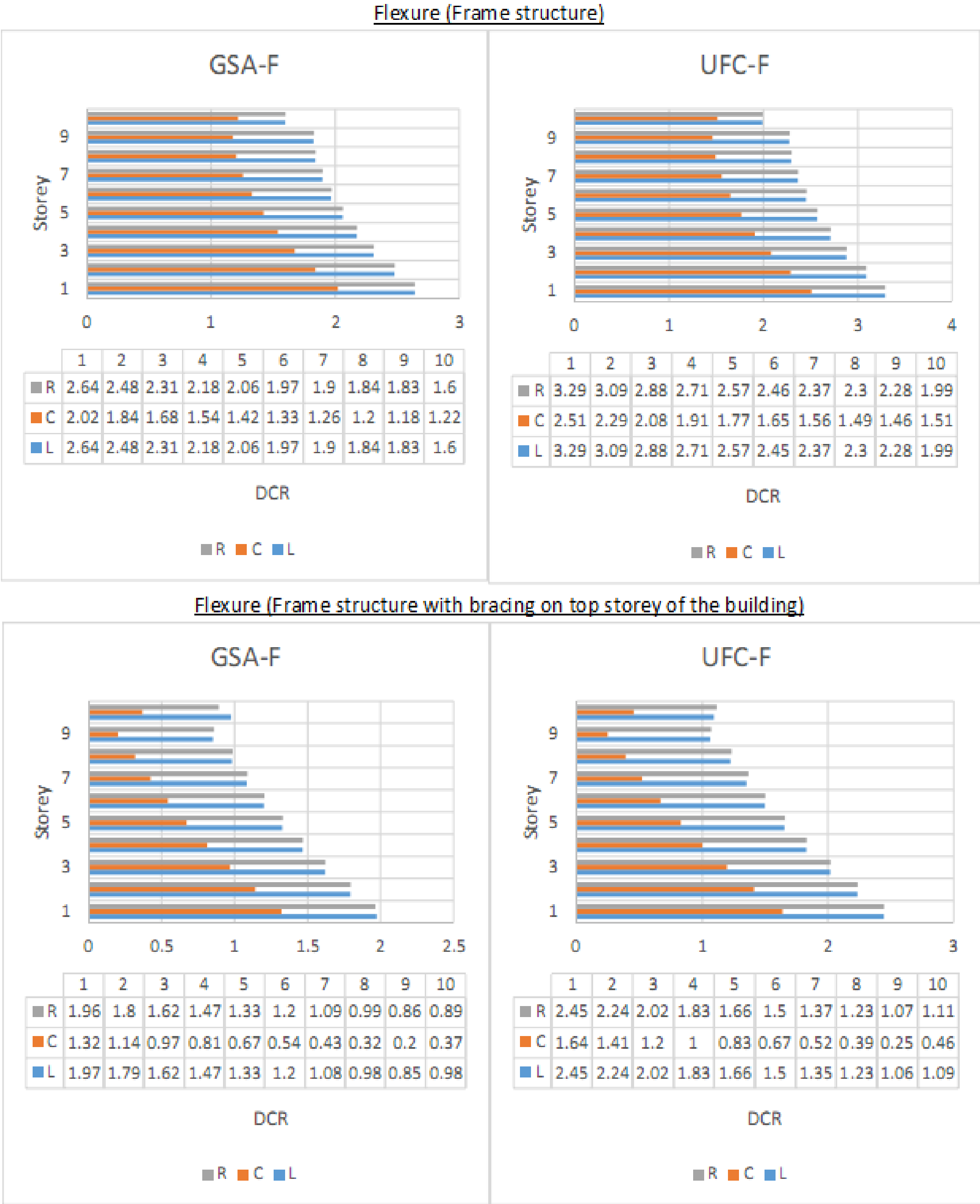


Figure 6.5: DCR for flexure for case 2 for bracing at side face of the building & bracing at top storey level and side face of the building



Flexure (Frame structure) — GSA-F

	1	2	3	4	5	6	7	8	9	10
R	2.64	2.48	2.31	2.18	2.06	1.97	1.9	1.84	1.83	1.6
C	2.02	1.84	1.68	1.54	1.42	1.33	1.26	1.2	1.18	1.22
L	2.64	2.48	2.31	2.18	2.06	1.97	1.9	1.84	1.83	1.6

Flexure (Frame structure) — UFC-F

	1	2	3	4	5	6	7	8	9	10
R	3.29	3.09	2.88	2.71	2.57	2.46	2.37	2.3	2.28	1.99
C	2.51	2.29	2.08	1.91	1.77	1.65	1.56	1.49	1.46	1.51
L	3.29	3.09	2.88	2.71	2.57	2.45	2.37	2.3	2.28	1.99

Flexure (Frame structure with bracing on top storey of the building) — GSA-F

	1	2	3	4	5	6	7	8	9	10
R	1.96	1.8	1.62	1.47	1.33	1.2	1.09	0.99	0.86	0.89
C	1.32	1.14	0.97	0.81	0.67	0.54	0.43	0.32	0.2	0.37
L	1.97	1.79	1.62	1.47	1.33	1.2	1.08	0.98	0.85	0.98

Flexure (Frame structure with bracing on top storey of the building) — UFC-F

	1	2	3	4	5	6	7	8	9	10
R	2.45	2.24	2.02	1.83	1.66	1.5	1.37	1.23	1.07	1.11
C	1.64	1.41	1.2	1	0.83	0.67	0.52	0.39	0.25	0.46
L	2.45	2.24	2.02	1.83	1.66	1.5	1.35	1.23	1.06	1.09

Figure 6.6: DCR for flexure for case 3 for frame structure & frame structure with bracing on top storey of building

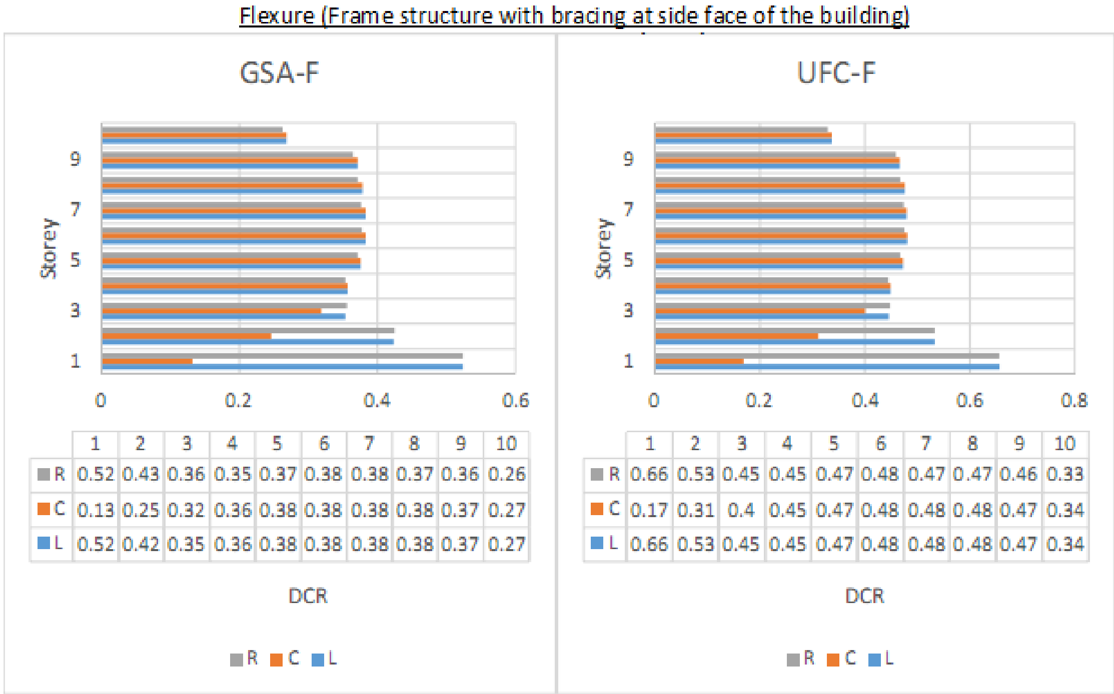


Figure 6.7: DCR for flexure for case 3 for bracing at side face of the building & bracing at top storey level and side face of the building

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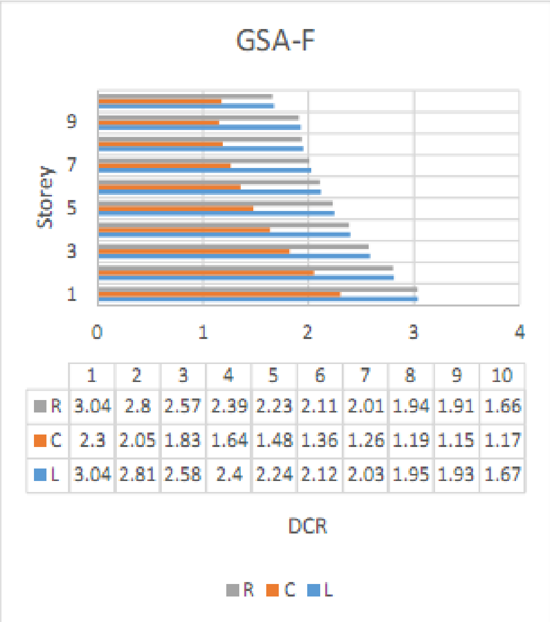
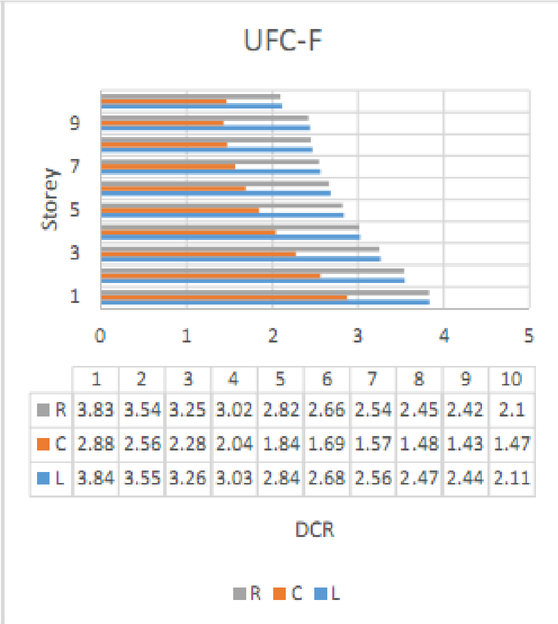
Flexure (Frame structure)

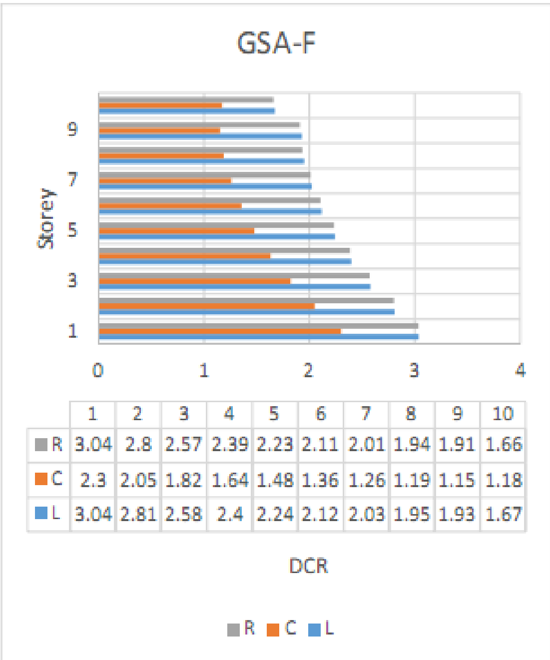


	1	2	3	4	5	6	7	8	9	10
R	3.04	2.8	2.57	2.39	2.23	2.11	2.01	1.94	1.91	1.66
C	2.3	2.05	1.83	1.64	1.48	1.36	1.26	1.19	1.15	1.17
L	3.04	2.81	2.58	2.4	2.24	2.12	2.03	1.95	1.93	1.67

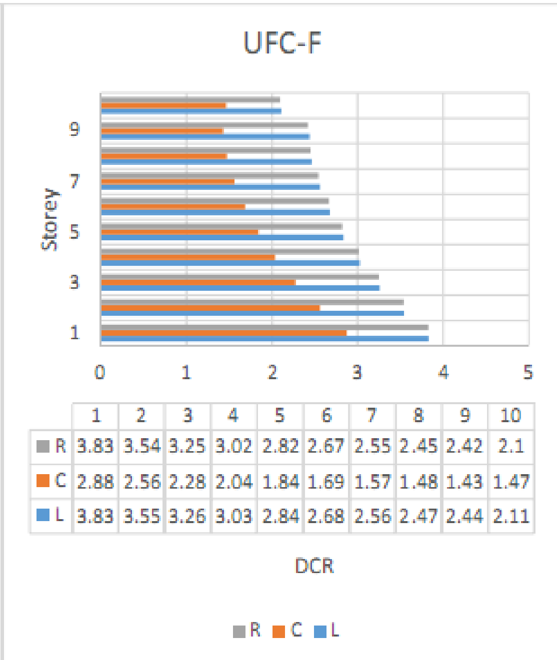


	1	2	3	4	5	6	7	8	9	10
R	3.83	3.54	3.25	3.02	2.82	2.66	2.54	2.45	2.42	2.1
C	2.88	2.56	2.28	2.04	1.84	1.69	1.57	1.48	1.43	1.47
L	3.84	3.55	3.26	3.03	2.84	2.68	2.56	2.47	2.44	2.11

Flexure (Frame structure with bracing on top storey of the building)



	1	2	3	4	5	6	7	8	9	10
R	3.04	2.8	2.57	2.39	2.23	2.11	2.01	1.94	1.91	1.66
C	2.3	2.05	1.82	1.64	1.48	1.36	1.26	1.19	1.15	1.18
L	3.04	2.81	2.58	2.4	2.24	2.12	2.03	1.95	1.93	1.67



	1	2	3	4	5	6	7	8	9	10
R	3.83	3.54	3.25	3.02	2.82	2.67	2.55	2.45	2.42	2.1
C	2.88	2.56	2.28	2.04	1.84	1.69	1.57	1.48	1.43	1.47
L	3.83	3.55	3.26	3.03	2.84	2.68	2.56	2.47	2.44	2.11

Figure 6.8: DCR for flexure for case 4 for frame structure & frame structure with bracing on top storey of building

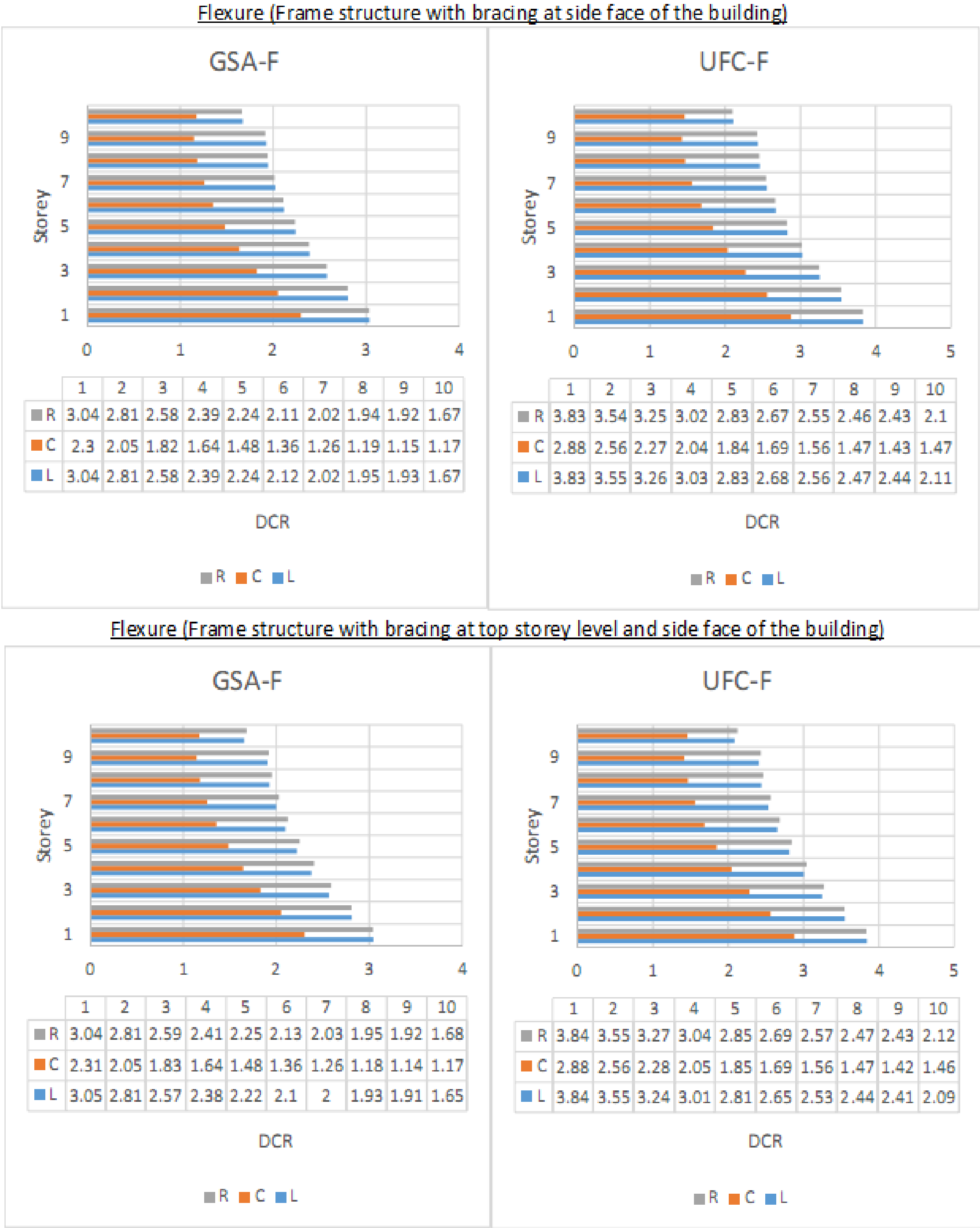
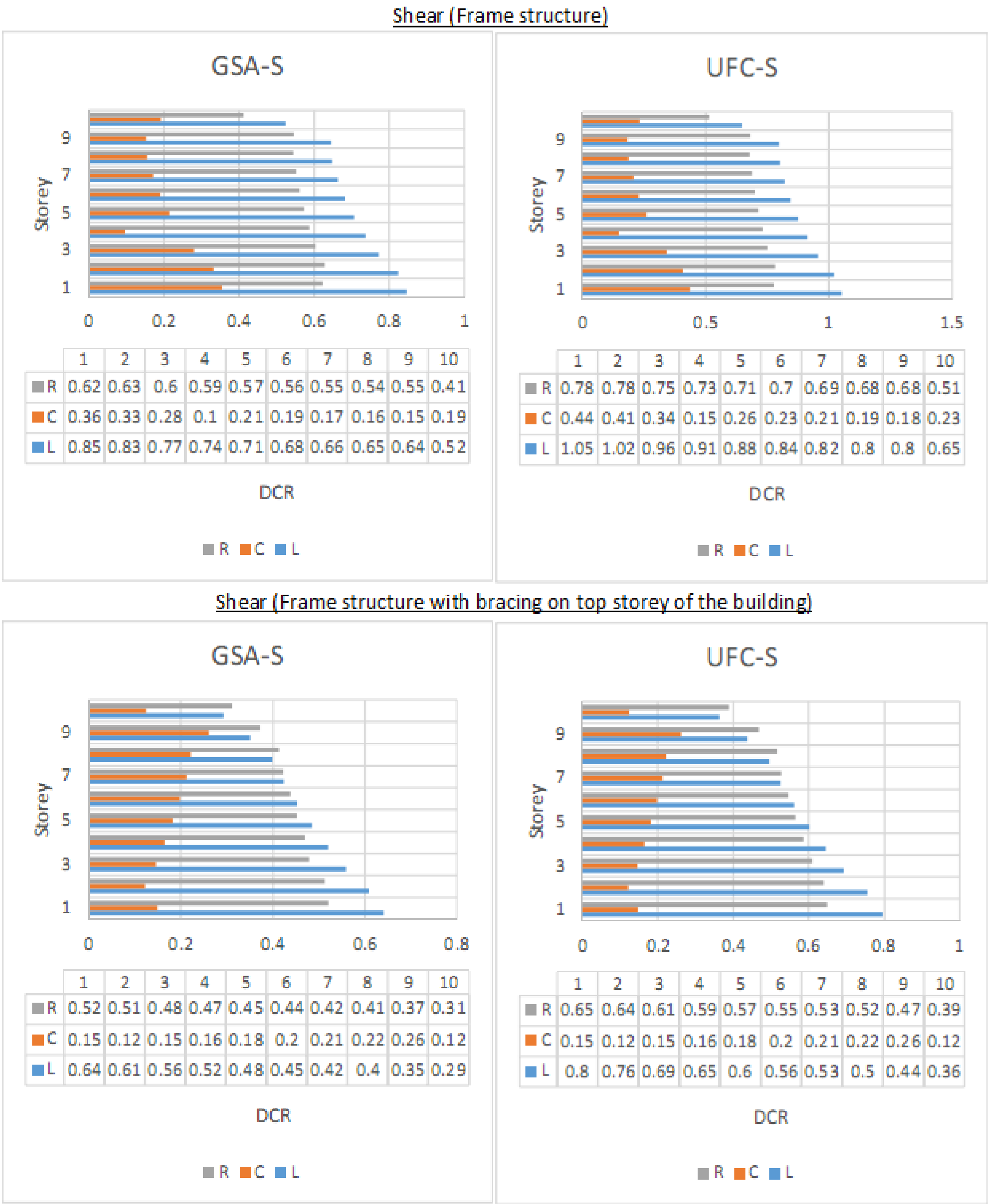


Figure 6.9: DCR for flexure for case 4 for bracing at side face of the building & bracing at top storey level and side face of the building

6.3.2 Shear Analysis

Demand Capacity Ratio (DCR) is calculated for shear in beam before and after the mitigation for 10-storey building. Figure 6.10 to figure 6.17shows comparison of DCR for shear for all load cases before and after the mitigation for 10-storey building. DCR in case of shear is within the permissible limit of 1.0 in case of GSA and UFC load case which reveals that beams are safe in shear as per GSA and UFC guidelines except case-4.



Shear (Frame structure) — GSA-S

	1	2	3	4	5	6	7	8	9	10
R	0.62	0.63	0.6	0.59	0.57	0.56	0.55	0.54	0.55	0.41
C	0.36	0.33	0.28	0.1	0.21	0.19	0.17	0.16	0.15	0.19
L	0.85	0.83	0.77	0.74	0.71	0.68	0.66	0.65	0.64	0.52

Shear (Frame structure) — UFC-S

	1	2	3	4	5	6	7	8	9	10
R	0.78	0.78	0.75	0.73	0.71	0.7	0.69	0.68	0.68	0.51
C	0.44	0.41	0.34	0.15	0.26	0.23	0.21	0.19	0.18	0.23
L	1.05	1.02	0.96	0.91	0.88	0.84	0.82	0.8	0.8	0.65

Shear (Frame structure with bracing on top storey of the building) — GSA-S

	1	2	3	4	5	6	7	8	9	10
R	0.52	0.51	0.48	0.47	0.45	0.44	0.42	0.41	0.37	0.31
C	0.15	0.12	0.15	0.16	0.18	0.2	0.21	0.22	0.26	0.12
L	0.64	0.61	0.56	0.52	0.48	0.45	0.42	0.4	0.35	0.29

Shear (Frame structure with bracing on top storey of the building) — UFC-S

	1	2	3	4	5	6	7	8	9	10
R	0.65	0.64	0.61	0.59	0.57	0.55	0.53	0.52	0.47	0.39
C	0.15	0.12	0.15	0.16	0.18	0.2	0.21	0.22	0.26	0.12
L	0.8	0.76	0.69	0.65	0.6	0.56	0.53	0.5	0.44	0.36

Figure 6.10: DCR for shear for case 1 for frame structure & frame structure with bracing on top storey of building

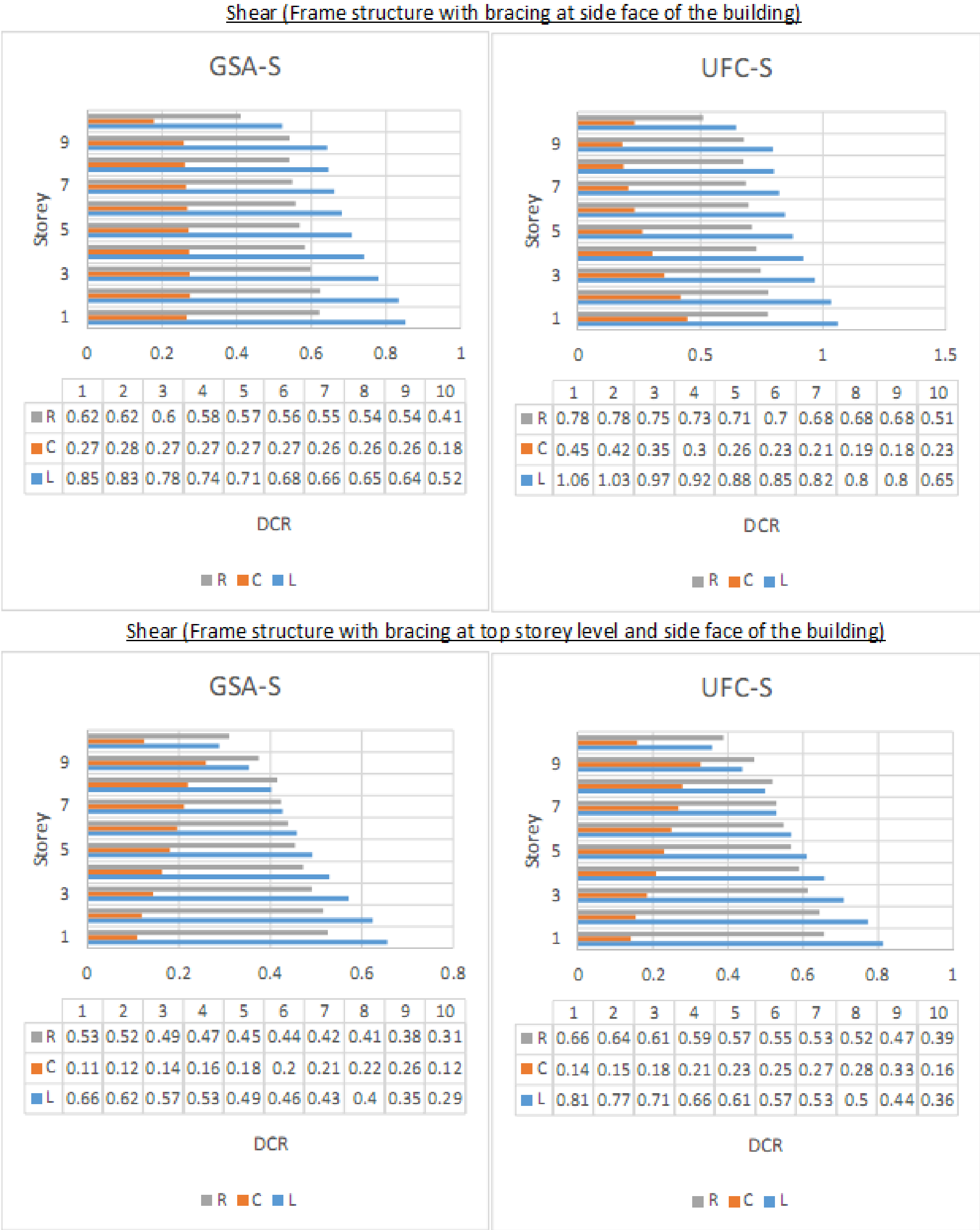
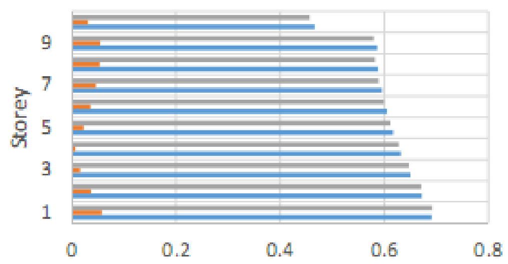


Figure 6.11: DCR for shear for case 1 for bracing at side face of the building & bracing at top storey level and side face of the building

Shear (Frame structure)

GSA-S

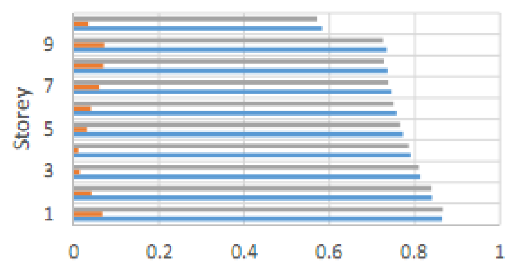


	1	2	3	4	5	6	7	8	9	10
R	0.69	0.67	0.65	0.63	0.61	0.6	0.59	0.58	0.58	0.46
C	0.06	0.04	0.02	0.01	0.02	0.04	0.05	0.05	0.05	0.03
L	0.69	0.67	0.65	0.63	0.62	0.61	0.6	0.59	0.59	0.47

DCR

R C L

UFC-S



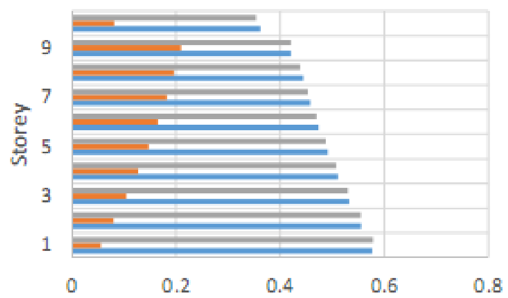
	1	2	3	4	5	6	7	8	9	10
R	0.87	0.84	0.81	0.79	0.77	0.75	0.74	0.73	0.73	0.57
C	0.07	0.04	0.02	0.01	0.03	0.04	0.06	0.07	0.07	0.04
L	0.86	0.84	0.81	0.79	0.77	0.76	0.75	0.74	0.73	0.58

DCR

R C L

Shear (Frame structure with bracing on top storey of the building)

GSA-S

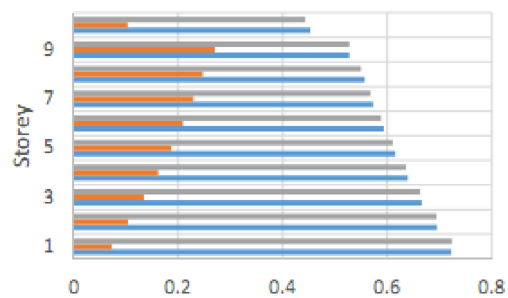


	1	2	3	4	5	6	7	8	9	10
R	0.58	0.56	0.53	0.51	0.49	0.47	0.45	0.44	0.42	0.35
C	0.06	0.08	0.1	0.13	0.15	0.17	0.18	0.2	0.21	0.08
L	0.58	0.56	0.53	0.51	0.49	0.47	0.46	0.44	0.42	0.36

DCR

R C L

UFC-S



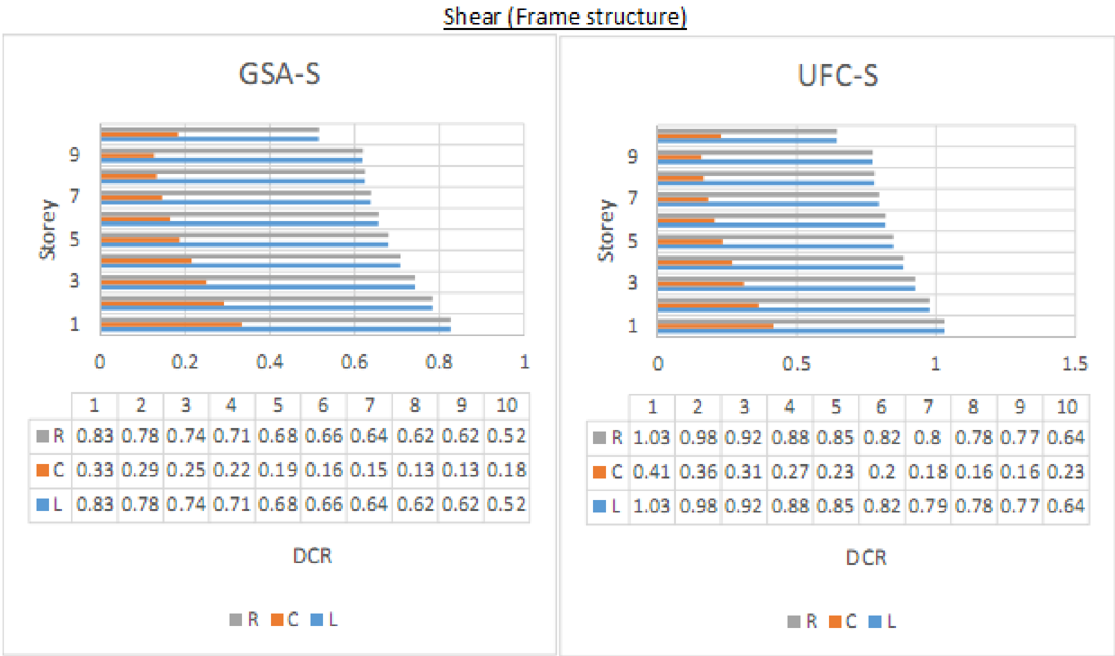
	1	2	3	4	5	6	7	8	9	10
R	0.72	0.69	0.66	0.64	0.61	0.59	0.57	0.55	0.53	0.44
C	0.07	0.1	0.13	0.16	0.19	0.21	0.23	0.25	0.27	0.1
L	0.72	0.7	0.67	0.64	0.62	0.59	0.57	0.56	0.53	0.45

DCR

R C L

Figure 6.12: DCR for shear for case 2 for frame structure & frame structure with bracing on top storey of building

Figure 6.13: DCR for shear for case 2 for bracing at side face of the building & bracing at top storey level and side face of the building



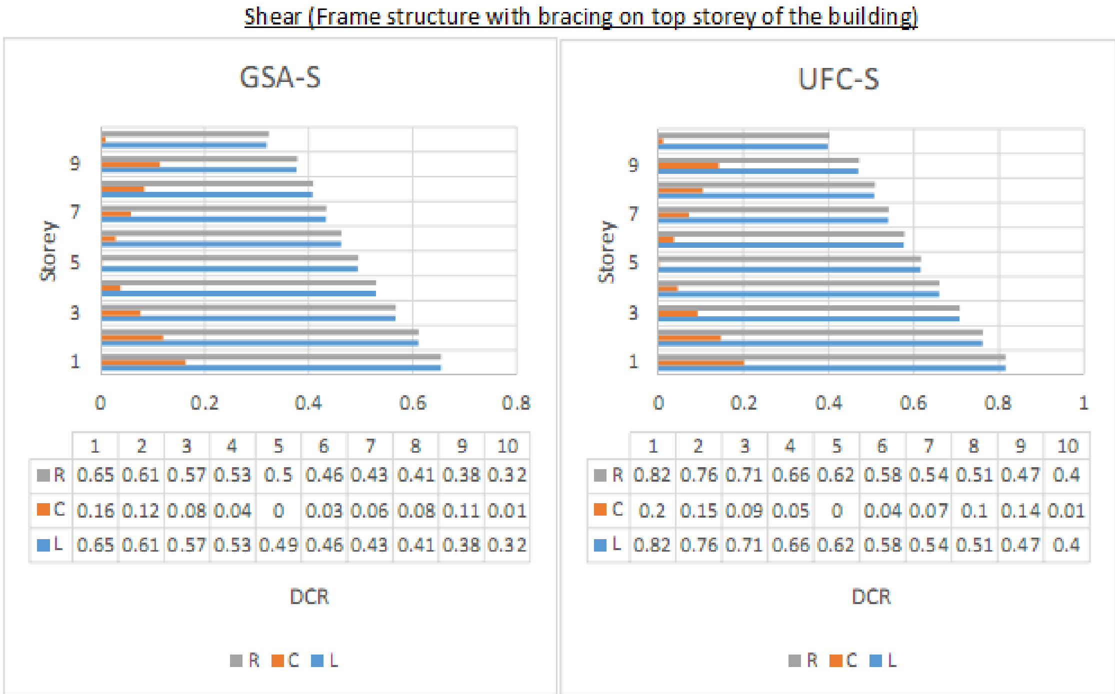


Figure 6.14: DCR for shear for case 3 for frame structure & frame structure with bracing on top storey of building

Shear (Frame structure with bracing at side face of the building) — GSA-S

	1	2	3	4	5	6	7	8	9	10
R	0.29	0.27	0.25	0.25	0.25	0.26	0.26	0.25	0.25	0.17
C	0.2	0.22	0.24	0.25	0.26	0.26	0.26	0.26	0.25	0.18
L	0.29	0.27	0.25	0.25	0.26	0.26	0.26	0.26	0.25	0.18

Shear (Frame structure with bracing at side face of the building) — UFC-S

	1	2	3	4	5	6	7	8	9	10
R	0.37	0.34	0.31	0.31	0.32	0.32	0.32	0.32	0.32	0.22
C	0.25	0.25	0.3	0.31	0.32	0.32	0.32	0.32	0.32	0.22
L	0.37	0.33	0.31	0.31	0.32	0.32	0.32	0.32	0.32	0.22

Shear (Frame structure with bracing at top storey level and side face of the building) — GSA-S

	1	2	3	4	5	6	7	8	9	10
R	0.29	0.27	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.17
C	0.2	0.22	0.24	0.25	0.25	0.26	0.25	0.25	0.25	0.17
L	0.29	0.27	0.25	0.25	0.25	0.26	0.25	0.25	0.25	0.17

Shear (Frame structure with bracing at top storey level and side face of the building) — UFC-S

	1	2	3	4	5	6	7	8	9	10
R	0.37	0.34	0.31	0.31	0.32	0.32	0.32	0.31	0.31	0.21
C	0.25	0.28	0.3	0.31	0.32	0.31	0.32	0.32	0.32	0.21
L	0.37	0.33	0.31	0.31	0.32	0.32	0.32	0.32	0.32	0.21

Figure 6.15: DCR for shear for case 3 for bracing at side face of the building & bracing at top storey level and side face of the building

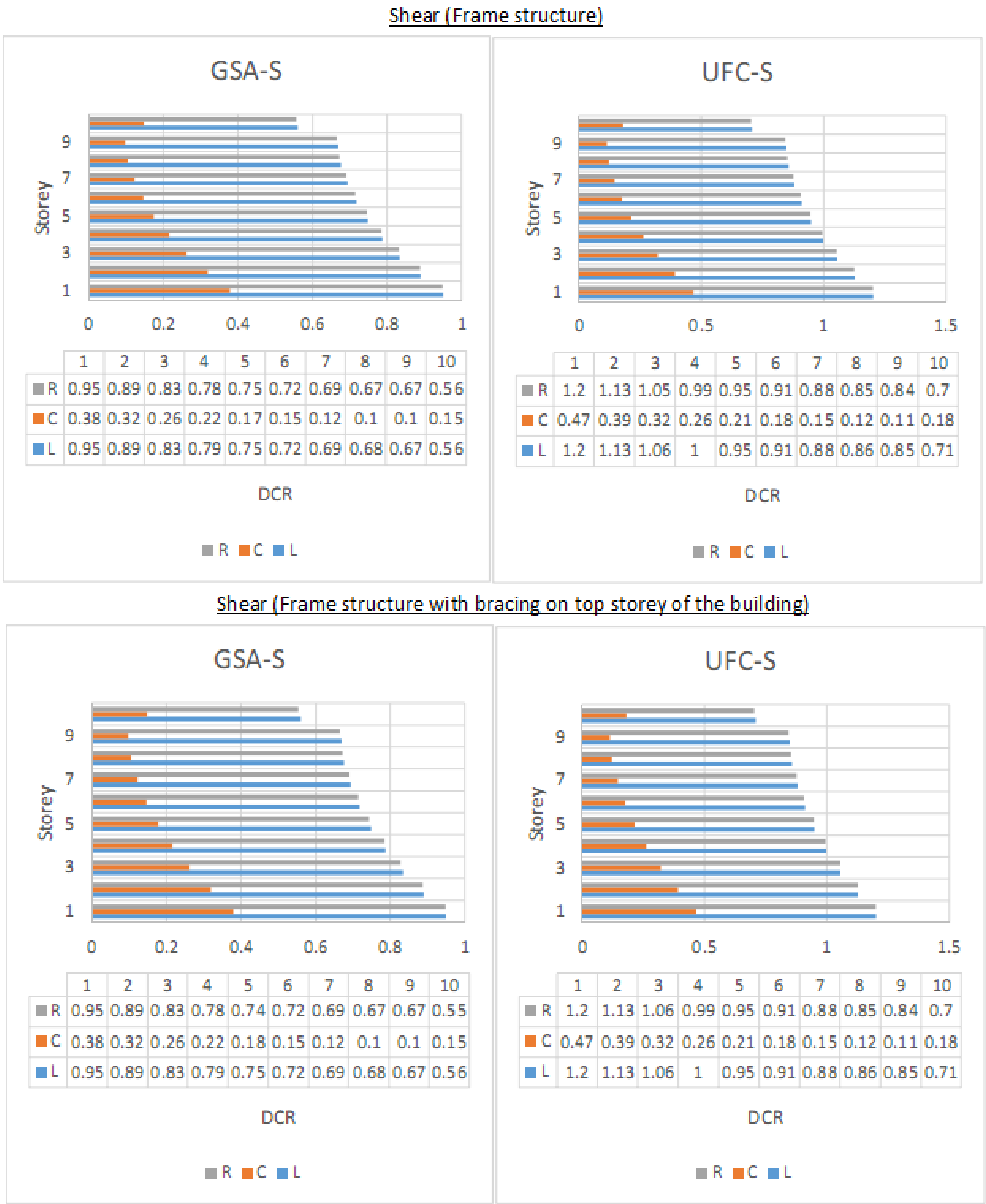
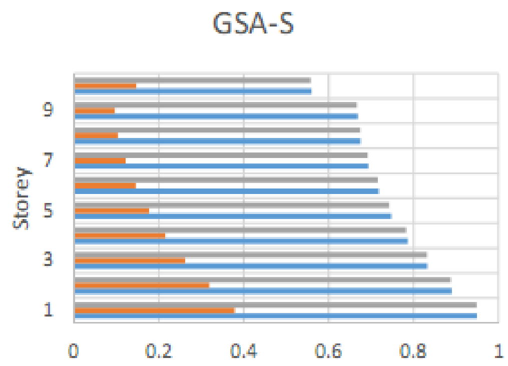


Figure 6.16: DCR for shear for case 4 for frame structure & frame structure with bracing on top storey of building

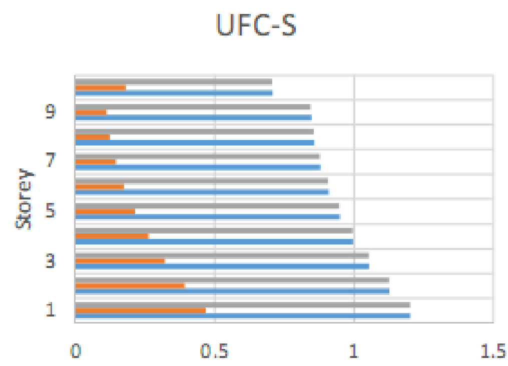
Shear (Frame structure with bracing at side face of the building)



	1	2	3	4	5	6	7	8	9	10
R	0.95	0.89	0.83	0.78	0.74	0.72	0.69	0.68	0.67	0.56
C	0.38	0.32	0.26	0.22	0.18	0.15	0.12	0.1	0.1	0.15
L	0.95	0.89	0.83	0.79	0.75	0.72	0.69	0.68	0.67	0.56

DCR

R C L

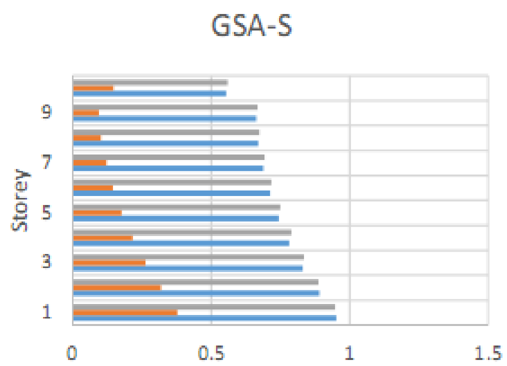


	1	2	3	4	5	6	7	8	9	10
R	1.2	1.13	1.05	1	0.95	0.91	0.88	0.85	0.84	0.71
C	0.47	0.39	0.32	0.26	0.21	0.18	0.15	0.12	0.11	0.18
L	1.2	1.13	1.05	1	0.95	0.91	0.88	0.86	0.85	0.71

DCR

R C L

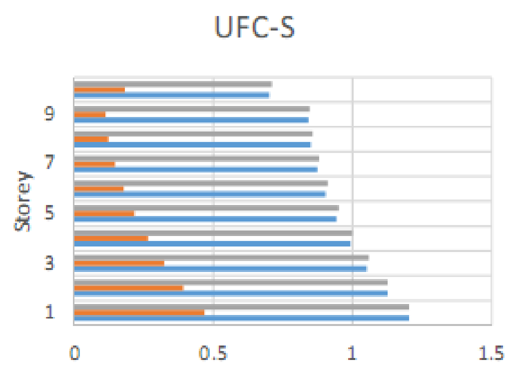
Shear (Frame structure with bracing at top storey level and side face of the building)



	1	2	3	4	5	6	7	8	9	10
R	0.95	0.89	0.84	0.79	0.75	0.72	0.69	0.68	0.67	0.56
C	0.38	0.32	0.26	0.22	0.18	0.15	0.12	0.1	0.1	0.15
L	0.95	0.89	0.83	0.78	0.74	0.71	0.69	0.67	0.66	0.56

DCR

R C L



	1	2	3	4	5	6	7	8	9	10
R	1.2	1.13	1.06	1	0.95	0.91	0.88	0.86	0.85	0.71
C	0.47	0.39	0.32	0.27	0.22	0.18	0.15	0.12	0.11	0.18
L	1.2	1.13	1.05	0.99	0.94	0.9	0.87	0.85	0.84	0.7

DCR

R C L

Figure 6.17: DCR for shear for case 4 for bracing at side face of the building & bracing at top storey level and side face of the building

6.3.3 Column Removal Analysis

For column removal analysis, the plan of 10 storey building is as shown in figure 6.18. Demand capacity ratios for all the four cases is calculated for one proximity column. DCR is calculated for 10-storey building before and after mitigation of all three alternatives is shown in figure 6.19 to figure 6.26. DCR for column exceed the allowable limit of 1.0 at bottom two to four stories. Also, values of DCR for column for all cases in UFC guideline is having higher values compared to GSA guideline.

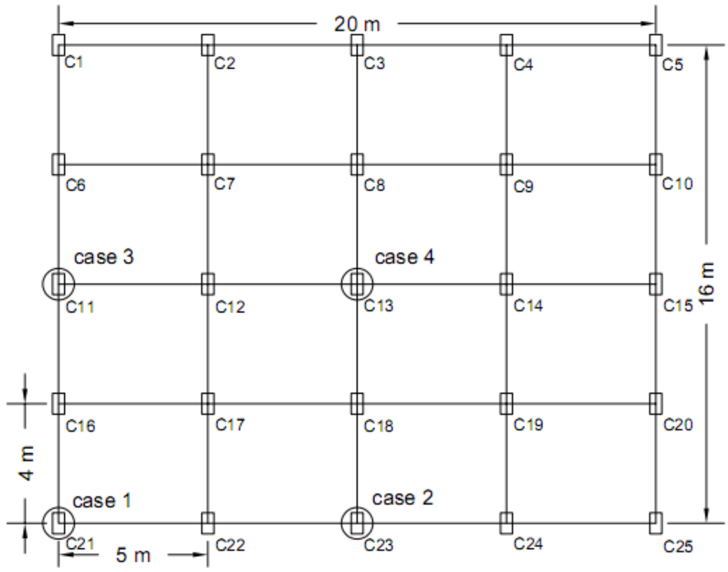


Figure 6.18: Plan of 10-storey building

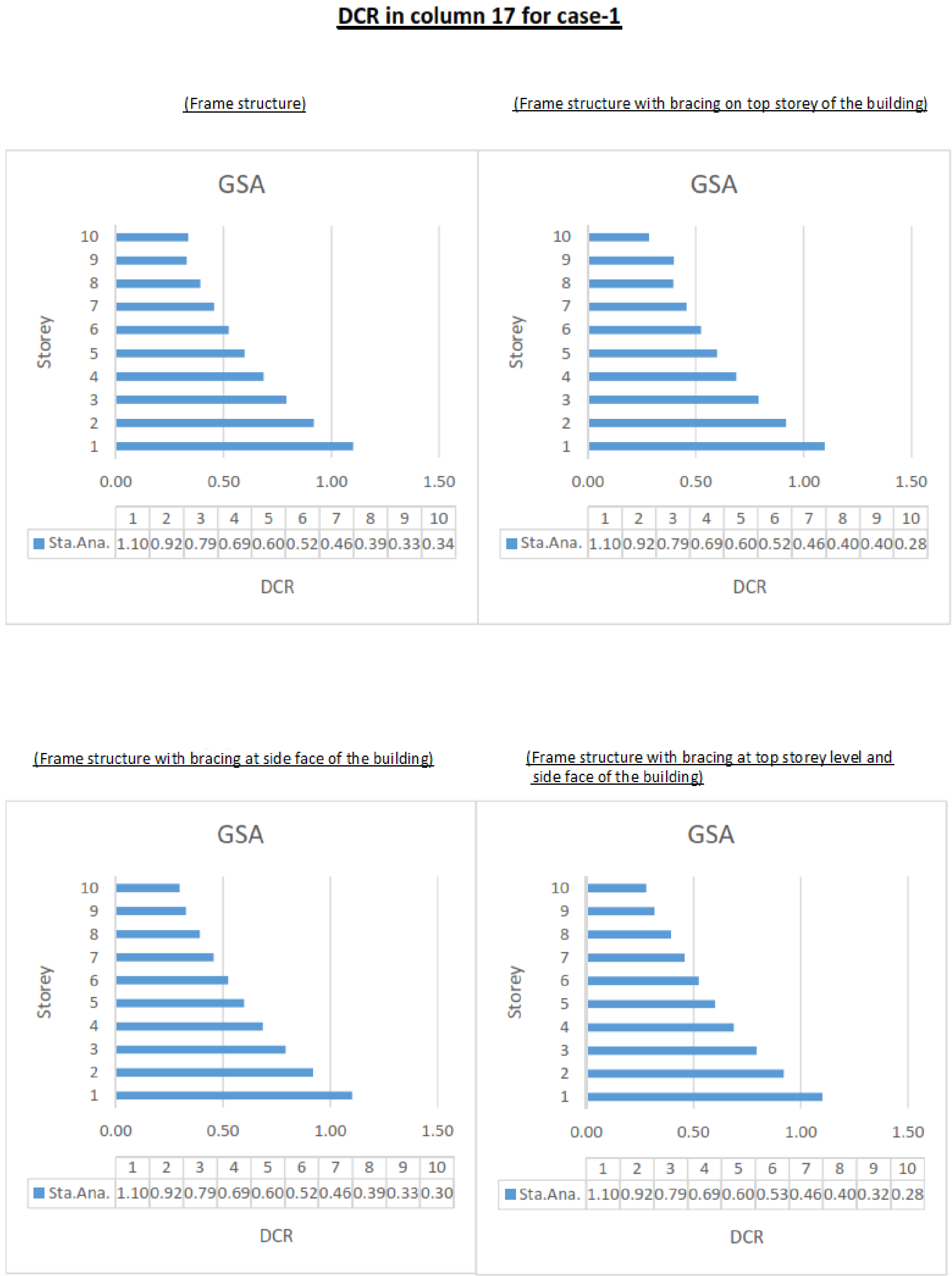


Figure 6.19: DCR in column 17 for case 1 for frame structure with bracing on top storey of building, for bracing at side face of the building & bracing at top storey level and side face of the building for GSA

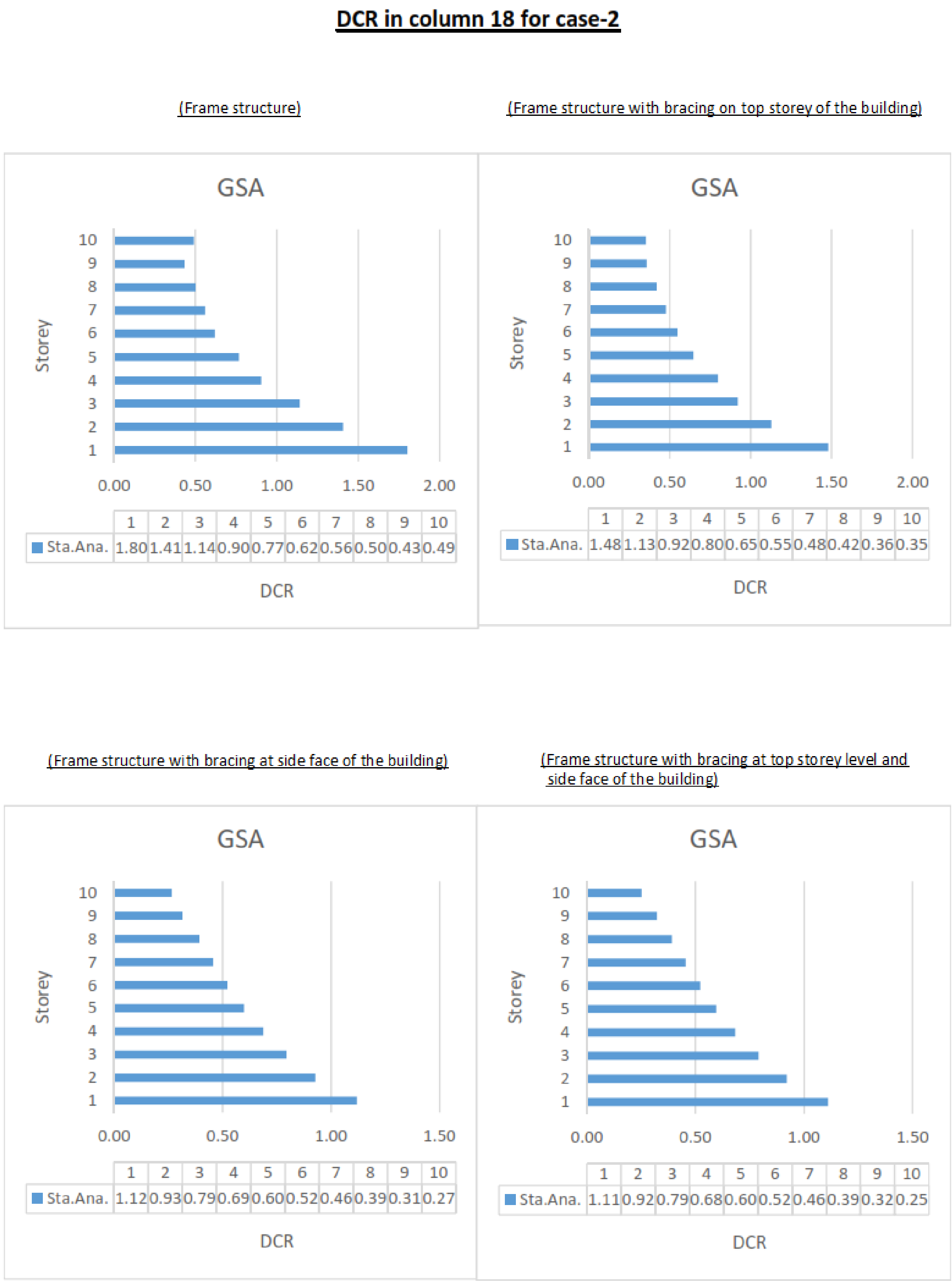


Figure 6.20: DCR in column 18 for case 2 for frame structure with bracing on top storey of building, for bracing at side face of the building & bracing at top storey level and side face of the building for GSA

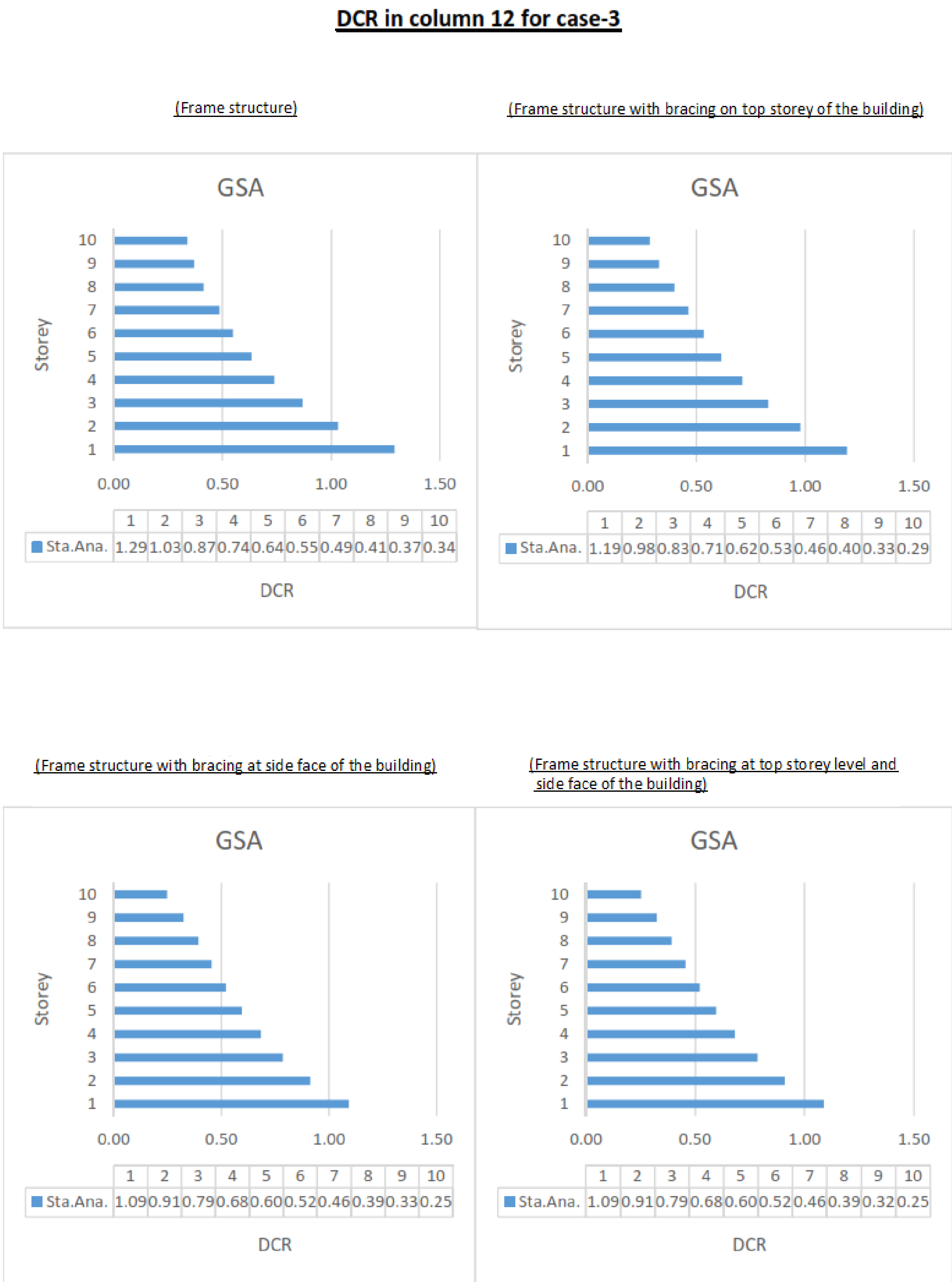


Figure 6.21: DCR in column 12 for case 3 for frame structure with bracing on top storey of building, for bracing at side face of the building & bracing at top storey level and side face of the building for GSA

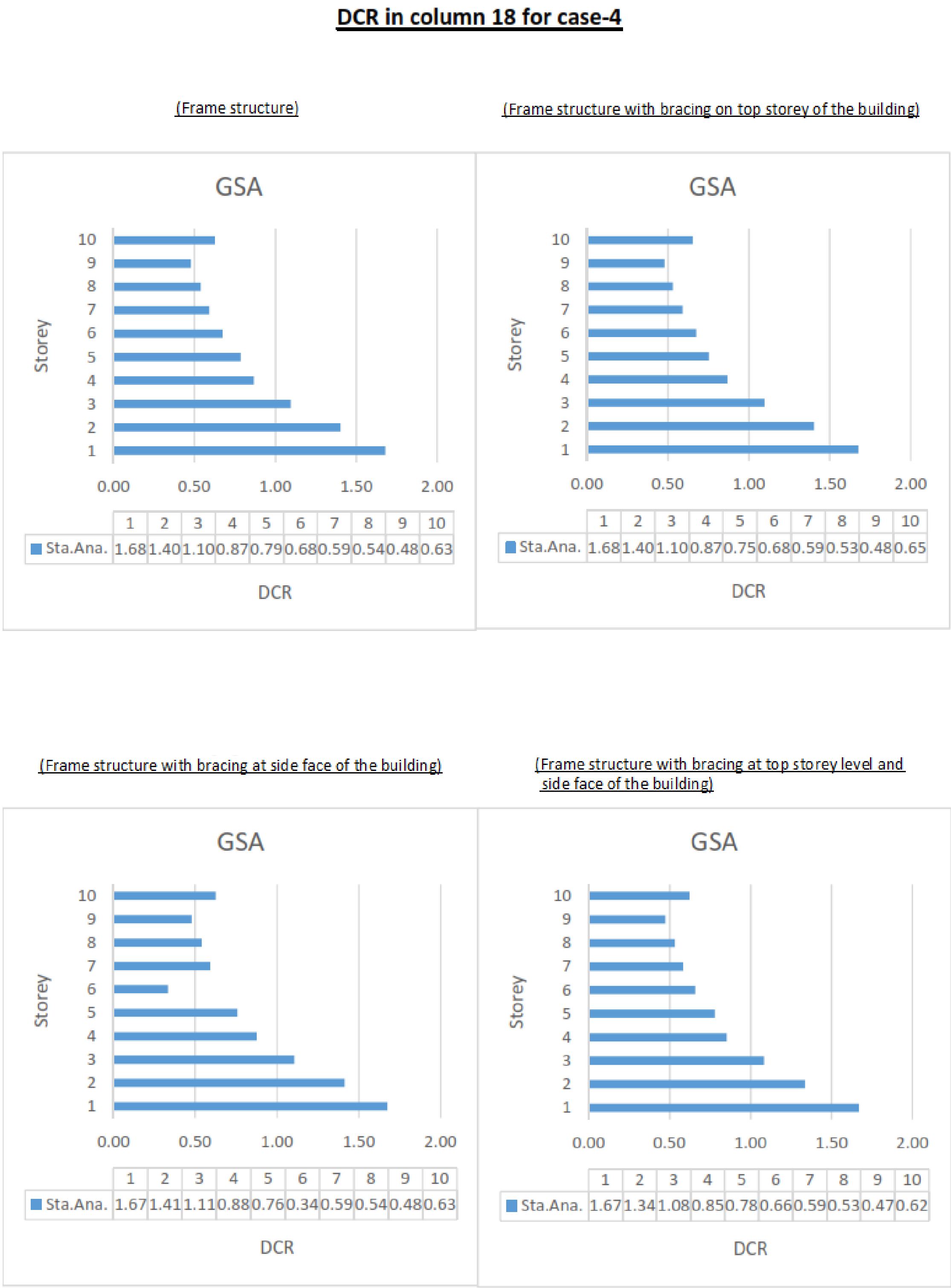


Figure 6.22: DCR in column 18 for case 4 for frame structure with bracing on top storey of building, for bracing at side face of the building & bracing at top storey level and side face of the building for GSA

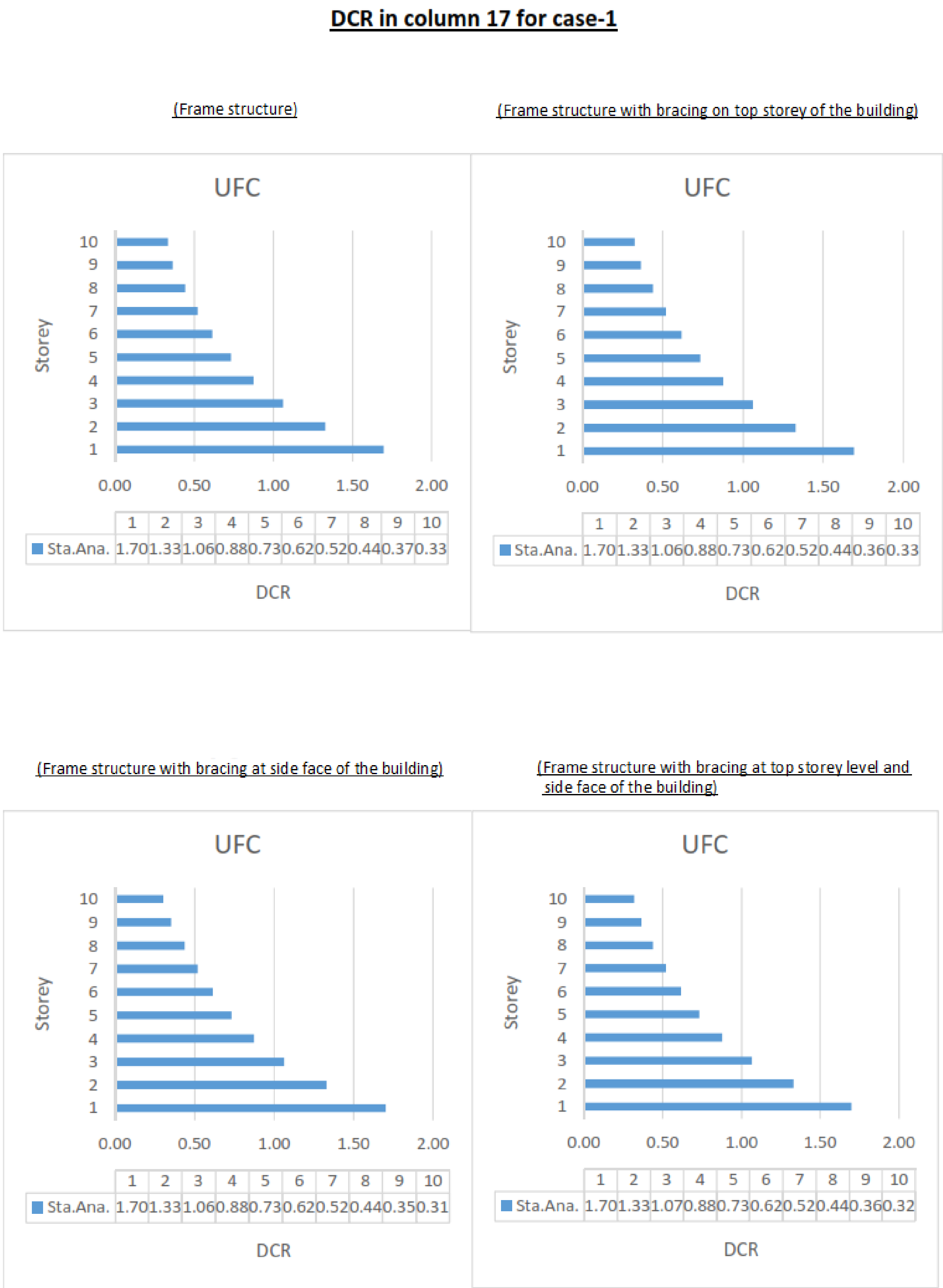


Figure 6.23: DCR in column 17 for case 1 for frame structure with bracing on top storey of building, for bracing at side face of the building & bracing at top storey level and side face of the building for UFC

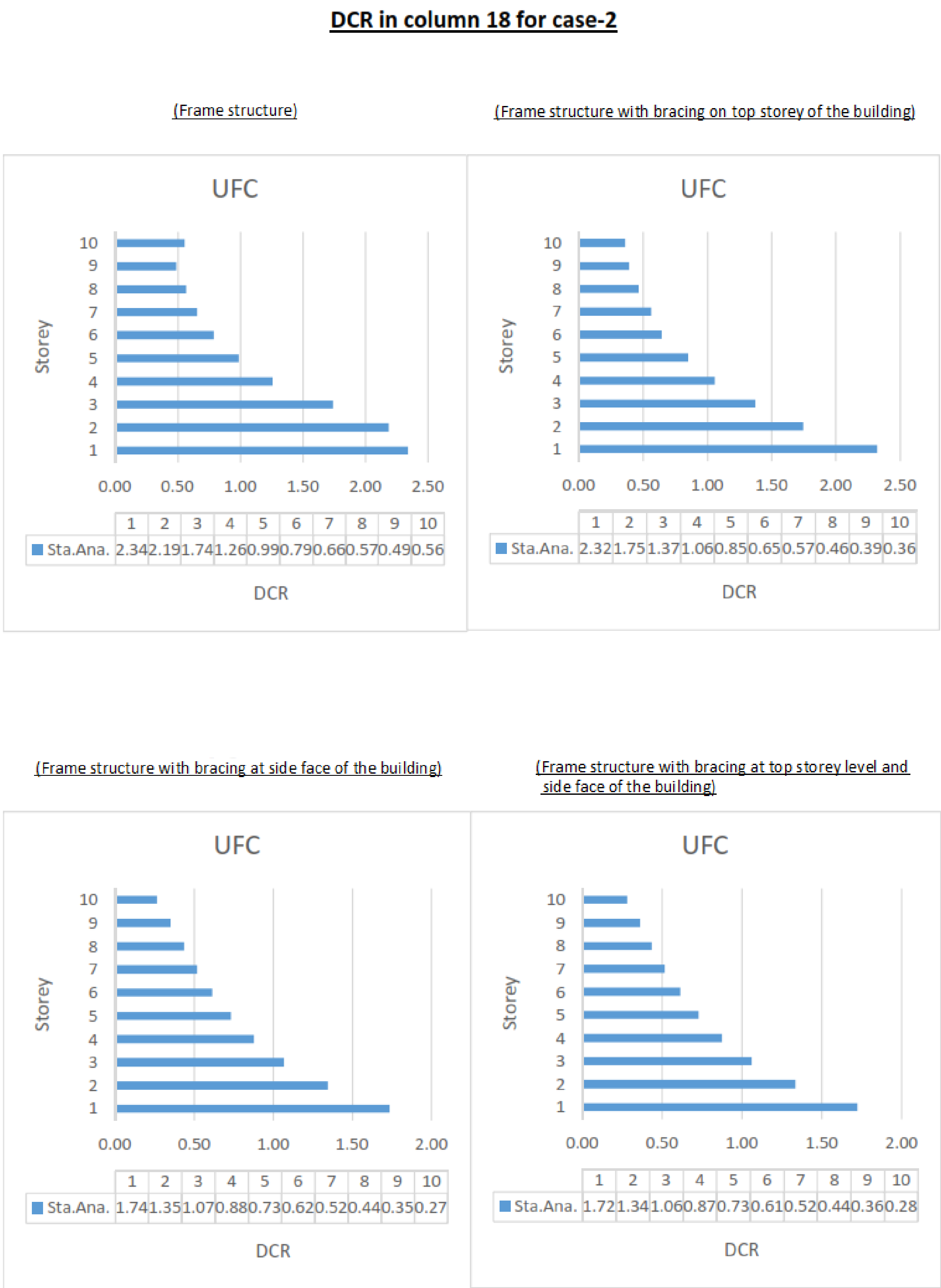


Figure 6.24: DCR in column 18 for case 2 for frame structure with bracing on top storey of building, for bracing at side face of the building & bracing at top storey level and side face of the building for UFC

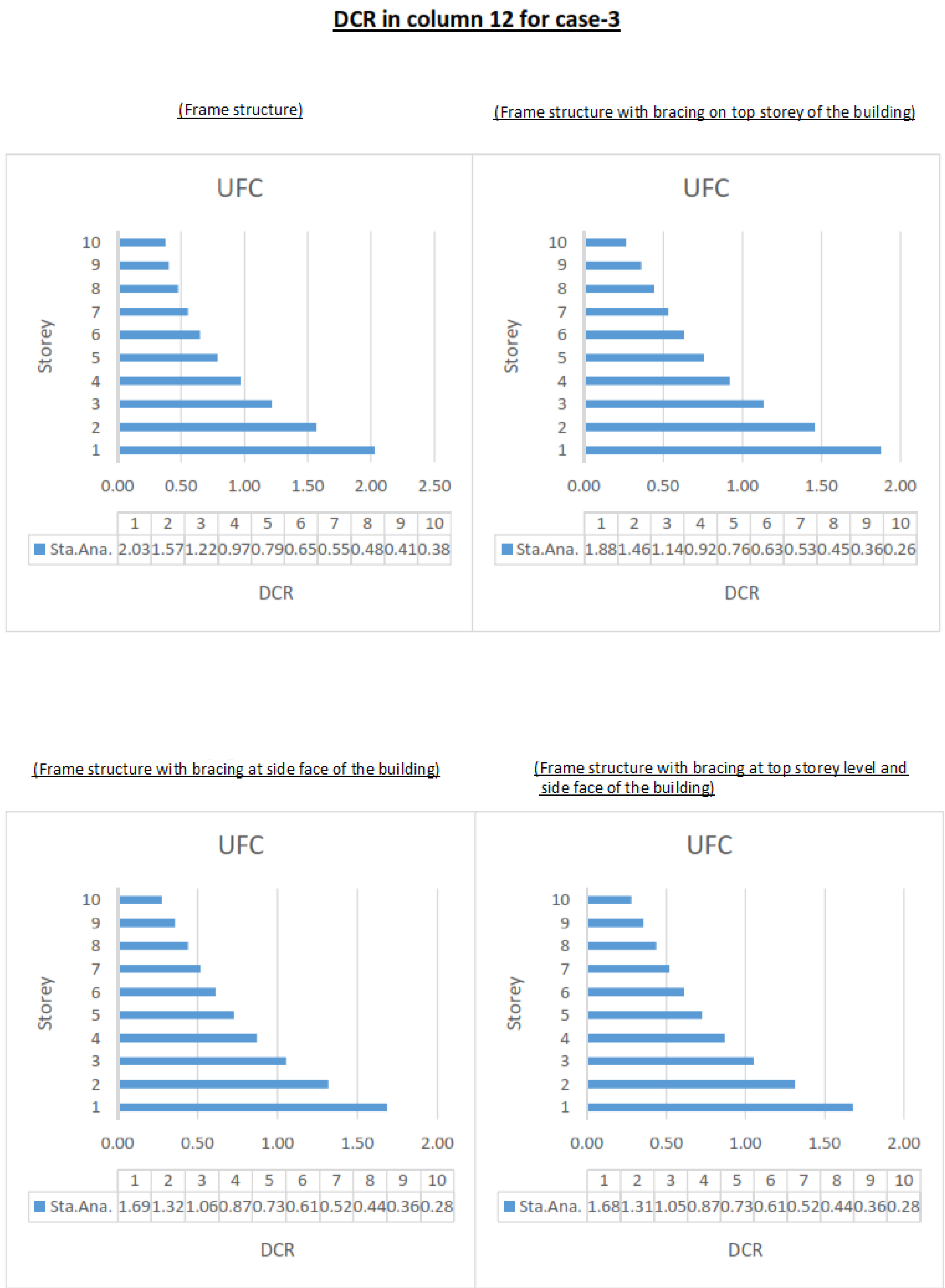


Figure 6.25: DCR in column 12 for case 3 for frame structure with bracing on top storey of building, for bracing at side face of the building & bracing at top storey level and side face of the building for UFC

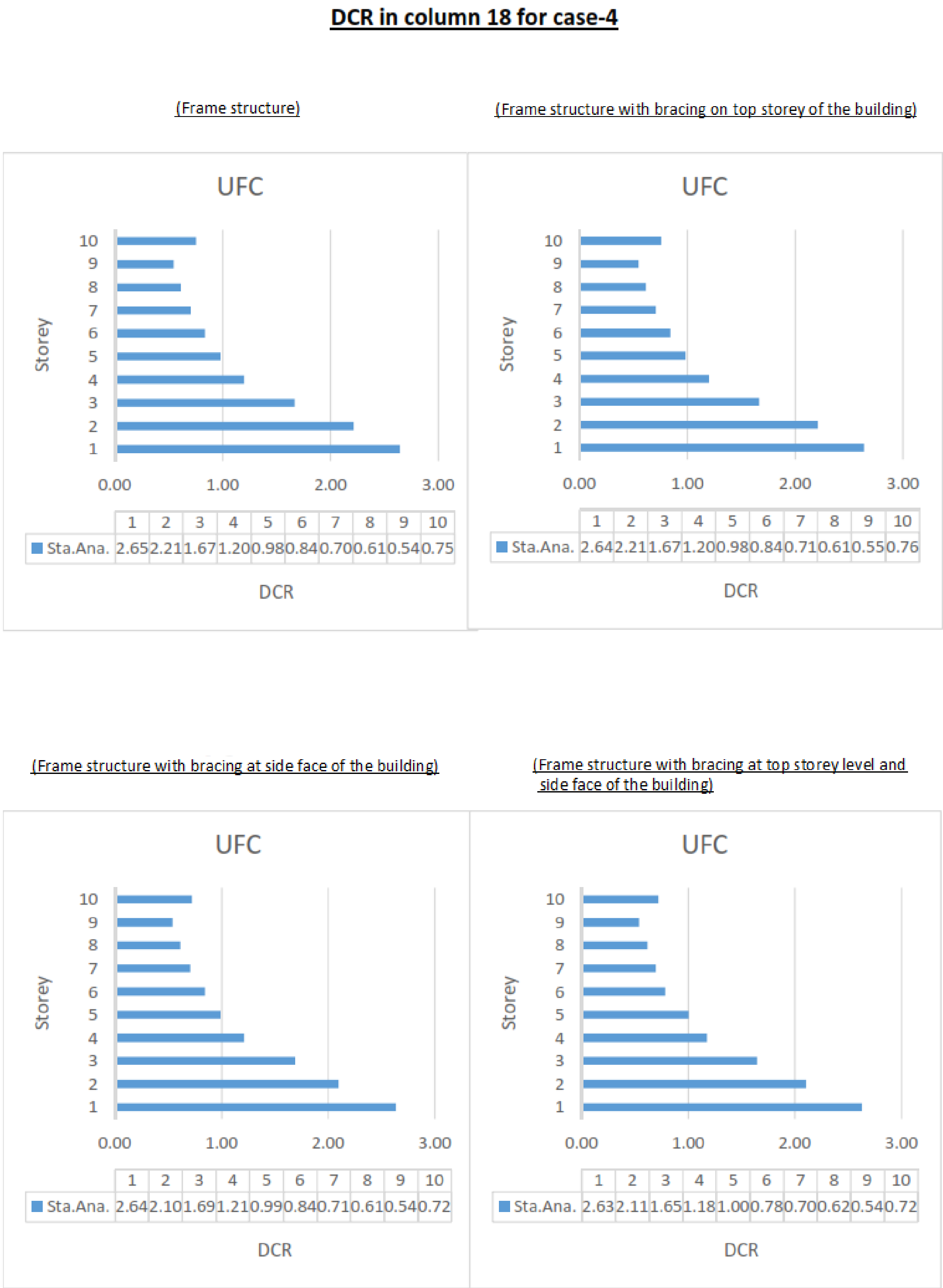


Figure 6.26: DCR in column 18 for case 4 for frame structure with bracing on top storey of building, for bracing at side face of the building & bracing at top storey level and side face of the building for UFC

6.3.4 Displacement Analysis

Figure 6.27 and figure 6.28 shows comparison of displacement under column removal point for GSA loading and UFC loading before and after mitigation of the building.

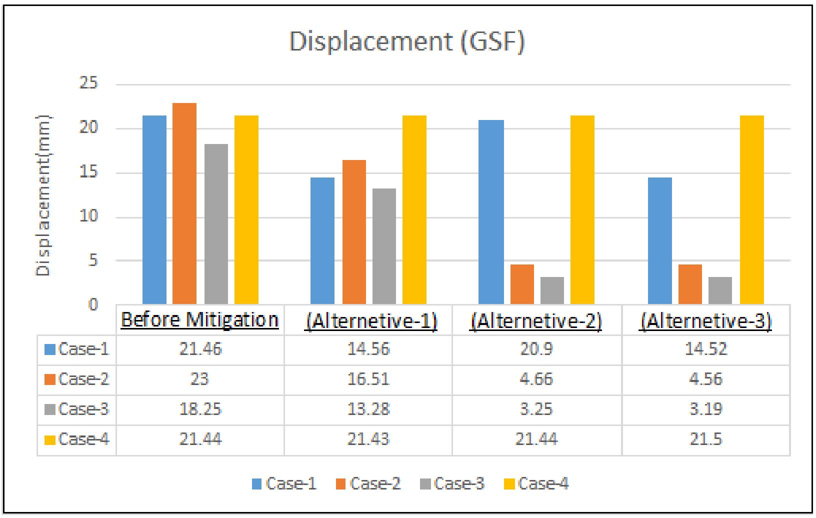


Figure 6.27: Displacement under column removal point for GSA loading

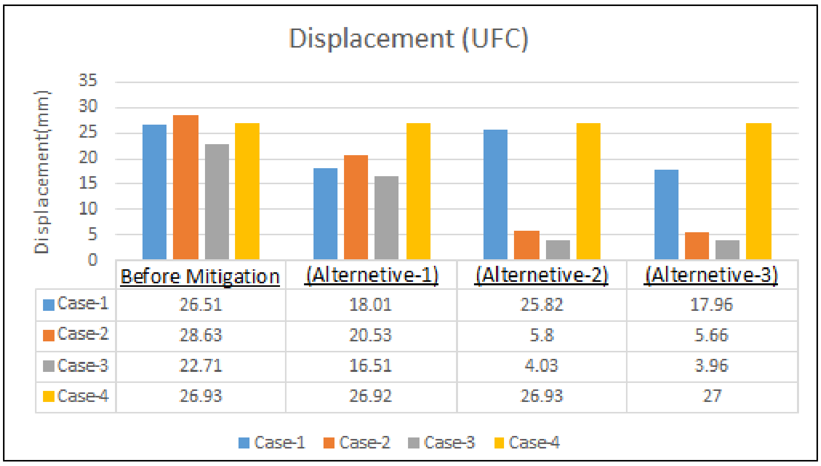


Figure 6.28: Displacement under column removal point for UFC loading

6.4 Summary

In this chapter, various mitigation systems are introduced to reduce the potential of 10-storey symmetrical reinforced concrete (RC) building. In this chapter three alternatives are executed i.e. provision of bracing at top storey level of the building, provision of bracing at side face of the building, Provision of bracing at top storey level and side face of the building. DCR under column removal point for all the load case before and after mitigation for 10-storey building is compared. Demand capacity ratios for all the four cases is calculated for one proximity column and compared before and after mitigation of all three alternatives. Also, the displacement under column removal point is compared for all the load cases before and after mitigation.

Chapter 7

Summary and Conclusions

7.1 Summary

Progressive collapse of existing building is initiated by the sudden failure of one or more of its major load bearing elements, typically columns or walls due to a vehicle impact, fire, earthquake, or other man-made or natural hazards. Structural members of building are not designed to withstand this type of abnormal loading and causes failure. Such type of failure is referred to as "Progressive Collapse".

Due to failure of column total gravity load from the structure transfers to neighboring members in the structure. If these columns are not properly designed to resist and redistribute the additional gravity load, that part of the structure fails. The vertical load carrying elements of the structure continue to fail until the additional loading is stabilized. As a result, a substantial part of the structure may collapse, causing greater damage to the structure than the initial impact.

In this study, cases various progressive collapse are discussed with the historical background and causes of the progressive collapse are identified. Various criteria to be considered to perform progressive collapse analysis are specified by U.S. General Service Administration (GSA) and Department of Defense (DoD) guidelines.

10-storey symmetrical reinforced concrete (RC) building is analyzed by both guidelines GSA and DoD. Modelling,analysis and design of the building are performed using structural analysis and design software Midas-Gen-2012. Progressive collapse analysis of 10-storey symmetrical reinforced concrete (RC) building is carriedout by following alternate load path method. In this method original structure is designed for gravity loading and seismic loading. Subsequently column is removed at ground floor level depending on cases. Structure is analysed under loading as specified in GSA and DoD guidelines, with column removal condition.

Capacity at critical sections is obtained from original design and considering strength increase factor. If Demand Capacity Ratio (DCR) exceeds permissible values as per the guidelines , it is considered as failed. DCR are calculated at each storey of the building. DCR are calculated at three point right (R), center (C) and left (L) of all column removal cases. The DCR for flexure, shear and axial force are calculated by linear static analysis are compared for both the guideline GSA and UFC for all cases.Effect of column removal on other proximity column is studied for ten storey building . Demand capacity ratios for all the four cases is calculated for one proximity column using both GSA and UFC guidelines.

Study of vertical displacement under column removal point is carried out for all the column removal cases. Displacement obtained by GSA and UFC load cases are compared for linear static analysis.

If building is having high potential of progressive collapse it is important to mitigate the vulnerability of progressive collapse. Mitigation is also referred as structural robustness. To minimization of potential for progressive collapse modification in structural system are required. Three alternatives are considered to minimize the potential of progressive collapse of 10-storey symmetric RC framed building. The alternatives include providing braces at various location i.e. bracing at top storey level of the building, bracing at side face of the building and bracing at top storey level and side face of the building.

7.2 Conclusions

From this study following conclusions can be drawn.

- For all the above column removal cases, DCR obtained by Department of Defense (DoD) Unified Facilities Criteria (UFC) is higher compared to those obtained by the U.S. General Service Administration (GSA) guidelines. In 10-storey building the value of Demand capacity ratio is governed for left side and right side of the column removal position in linear static analysis.

- For 10-storey building DCR in flexure exceeds the permissible limit of 2.0 with GSA and UFC load cases, which reveals that beams are unsafe in flexure as per GSA and UFC guidelines. Also, DCR in case of shear exceeds the permissible limit of 1.0 with UFC load case which indicates that beams are safe in shear as per GSA guidelines but unsafe as per UFC guidelines.

- In 10-storey building DCR for column considering axial force and bending moment exceed the allowable limit of 1.0 at bottom two to four stories. DCR values for column for all cases as per UFC guidelines are having higher values compared to that obtained with GSA guidelines.

- Displacement under column removal point for GSA loading and UFC loading is compared for all the cases and displacement is higher in case of UFC loading.

Out of all the four cases of column removal as suggested by guidelines, case 4 i.e. removal of column c13 creates worst effect on the building structure. 10-storey symmetrical building taken for the study is having high risk of progressive collapse.

- From all the three modified structural system presented, provision of bracing at top storey level and side face of the building is most economical solution to reduce the potential of progressive collapse. With alternative-3 mitigation strategies i.e. bracing at top storey level and side face of the building, DCR in case of flexure is within the permissible limit of 2.0 with GSA and UFC load cases which reveals that beams are safe in flexure as per GSA and UFC guidelines except case-4. Also, DCR in case of shear is within the permissible limit of 1.0 in case of GSA and UFC load case which reveals that beams are safe in shear as per GSA and UFC guidelines except case-4 i.e. removal of column c13.

7.3 Future Scope of Work

The study in this report is limited to progressive collapse analysis of 10-storey symmetrical reinforced concrete building. The present study can be extended to include following aspects.

- Progressive collapse analysis of symmetrical and Asymmetrical multi-storied steel building can be carried out.
- Progressive collapse potential of important existing buildings can be studied.
- Case study of existing building can be taken to study its vulnerability to blast and progressive collapse.

References

[1] Kim J and Jung M, "Progressive collapse resisting capacity of modular megaframe buildings", Structural Design of Tall and Special Buildings, 22:471-484, 2011.

[2] Tsai M and Huang T, "Progressive collapse analysis of an RC building with exterior partially infilled walls", Structural Design of Tall and Special Buildings, 22:327-348, 2011.

[3] Kim J,Lee Y and Choi H, "Progressive collapse resisting capacity of braced frames", Structural Design of Tall and Special Buildings, 20:257-270, 2010.

[4] Mashhadiali N and Kheyroddin A, "Progressive collapse assessment of new hexagrid structural system for tall buildings", Structural Design of Tall and Special Buildings, 23:947-961, 2013.

[5] H.R. Tavakoli,A. Rashidi Alashti and G.R. Abdollahzadeh, "3-D Nonlinear Static Progressive Collapse Analysis of Multi-story Steel Braced Buildings", 2012.

[6] Kwon K,Park S and Kim J, "Evaluation of Progressive Collapse Resisting Capacity of Tall Buildings", International Journal of High-Rise Buildings, 1:229-235, 2012.

[7] Ren P,Li Y,Guan H and Lu X, "Progressive Collapse Resistance of Two Typical High-Rise RC Frame Shear Wall Structures", American Society of Civil Engineers, 2014.

[8] Kim J and Hong S, "Progressive collapse performance of irregular buildings", Structural Design of Tall and Special Buildings, 20:721-734, 2010.

[9] Kim J and Park J, "Progressive collapse resisting capacity of building structures with outrigger trusses", Structural Design of Tall and Special Buildings, 21:566-577, 2012.

[10] Kim J and Lee H, "Progressive collapse-resisting capacity of framed structures with infill steel panels", Journal of Constructional Steel Research, 89:145-152, 2014.

[11] Kim J and Lee Y, "Progressive collapse-resisting capacity of tube-type structures", Structural Design of Tall and Special Buildings, 19:761-777, 2010.

[12] Design of Buildings to Resist the Progressive Collapse, Unified Facilities Criteria (UFC 4-023-03)published by Department of Defense (DoD), July 2009.

[13] Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects, U. S. General Service Administration (GSA), April 2013.

[14] IS: 456-2000, Plain and reinforced concrete code of practice, 4^{th} Revision, Bureau of Indian Standards, New Delhi.

[15] IS: 1893(Part-1)-2002, Criteria for earthquake resistance design of structures, 5^{th} Revision, Bureau of Indian Standards, New Delhi.