PROGRESSIVE COLLAPSE ANALYSIS OF STEEL STRUCTURE

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Major Project

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For the degree of

Master of Technology in Civil Engineering

(Computer Aided Structural Analysis & Design)

By

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Declaration

This is to certify that

a. 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.

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

Parikh Rushi D.

09MCL014

Certificate

This is to certify that the Major Project entitled "Progressive collapse analysis of steel structure" submitted by Rushi D. Parikh (09MCL014), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Progressive collapse occurs when a structure has its loading pattern or boundary conditions changed such that structural elements are loaded beyond their capacity and fail. The residual structure is forced to seek alternative load paths to redistribute the load applied. As a result, other elements may fail, causing further load redistribution. The process will continue until the structure can find equilibrium either by shedding load as a by-product of the failures of other elements or by finding stable alternative load paths. In the past, structures designed to withstand normal load conditions were over-designed and were usually capable of tolerating abnormal loads. Modern building designs and construction practices enabled engineers to build lighter and more optimized structural systems with considerably fewer over strength characteristics.

It is estimated that at least 15 to 20% of the total number of building failures are due to progressive collapse. Progressive collapse became an issue following the Ronan Point collapse. Shortly after the Ronan Point collapse, British Standards emphasized general tying of various structural elements of a building together, to provide continuity and redundancy. Eurocode recommended tying the building together and defined values for tie forces. The National Building Code of Canada contains a general statement about the need for structural integrity. After the collapse of World trade center (WTC) towers, many government and private authorities worked on developing design guidelines for progressive collapse resistant structures. In the U.S., the prominent documents are: Progressive collapse analysis and design guidelines developed by the General Services Administration (GSA 2003) and Design of Buildings to Resist Progressive collapse developed by the Department of Defense (DoD 2005). Among all the available guidelines GSA and DoD guidelines are widely adopted for progressive collapse analysis.

The present study includes various case studies of progressive collapse of structures around the world. It includes the evolution of various guidelines published by many government authorities and their comparison. Specifications of GSA and DoD guidelines are discussed in detail. 4-Storey and 9-storey moment resistant steel buildings are considered for evaluation of progressive collapse potential. Four analysis procedures are suggested by the

guidelines to evaluate the potential of progressive collapse namely linear static, linear dynamic, nonlinear static and nonlinear dynamic.

Linear static and linear dynamic procedures are carried out using structural analysis programme SAP2000 to find out demand capacity ratio (DCR) of beams and for highly stressed near by columns after the removal of load carrying elements from different locations. DCR found using linear static analysis are compared with the DCR calculated from linear dynamic analysis at each storey for different column removal cases.Displacements found under the column removal points by linear static analysis are compared with linear dynamic analysis for each column removal case.

Nonlinear static analysis procedure is carried out to understand the extent of damage in form of hinges in the structure at yield point and at collapse load. The graph of percentage of vertical load Vs. deflection is drawn after nonlinear static analysis procedure for different column removal cases and plastic hinge rotations are found out for maximum collapse load as per GSA and DoD guidelines. Nonlinear dynamic analysis procedure is also carried out to understand the behavior of the structure accurately. Nonlinear dynamic analysis is carried out to find displacement ductility, maximum support rotation and plastic hinge rotation for all the column removal cases.

Report also includes the mitigation strategies to resist or to reduce the chances of progressive collapse of an entire structure. Three retrofitting strategies are suggested for mitigation of progressive collapse. Comparison is made between the values of DCR, collapse load, displacement ductility and plastic hinge rotations for building without retrofitting and with retrofitting strategies.

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- Rushi D. Parikh

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Abbreviation and Notation

AISC .... American Institute of steel construction
APM .... Alternate path method
ASCE .... American Society of Civil Engineers
DCR .... Demand Capacity Ratio
DL .... Dead load
DoD .... Department of Defense
ELR .... Enhanced Local Resistance method
f_y .... Characteristic yeild strength of steel
FEMA .... Federal Emergency Management agency
GSA .... U.S. General Service Administration
LL .... Live load
NIST .... National Institute of Standards and Technology
NYC .... New York City Building Code
OC .... Occupancy Category
UFC .... Unified Facilities Criteria
SAP .... Structural Analysis Program
TF .... Tie Force method
WTC .... World Trade Center
ϕ .... Strength reduction factor
R_n .... Nominal Tie strength
W_F .... Floor Load
NBCC .... National building code of Canada
BS .... British Standards

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Chapter 1

Introduction

1.1 General

Progressive collapse can be defined as "the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it." The disproportionately refers to the situation in which failure of one member causes a major collapse of a larger magnitude compared to the initial event. Most definitions of progressive collapse encompass the "house of cards" effect as shown in Fig. 1.1, where by damage spreads beyond a local region, to an extent disproportionate to the initial cause.

Figure 1.1: House of cards effects

1.2 Mechanism of progressive collapse

Progressive collapse is triggered by localized damage that can not be restricted and leads to a chain reaction of failures resulting in a partial or total structural collapse, where the final damage is disproportionate to the local damage from the initiating event. Fig. 1.2 shows the mechanism of progressive collapse. Once a column is failed the building's weight (gravity load) transfers to neighboring members in the structure. If these members are not properly designed to resist and redistribute the additional load that part of the structure fails. The vertical load carrying elements of the structure continue to fail until the additional loading is stabilized.

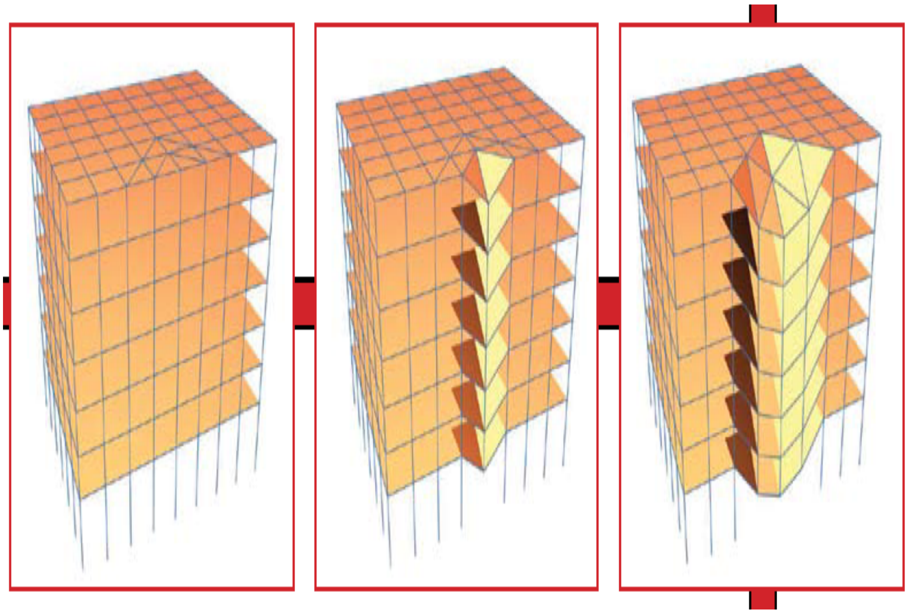


Figure 1.2: Mechanism of progressive collapse

1.3 Causes of progressive collapse

Buildings are generally designed for dead loads, live loads, earthquake loads and wind loads. Progressive collapse in the building structure mainly occurs due to accidental loadings on it for which building is not designed. An abnormal or accidental load is any loading condition a designer does not include in the normal and established practice of design. Abnormal loadings include explosions, sonic booms, wind-induced localized over-pressures, vehicle collisions, missile impacts, service system malfunctions, and impact of debris resulting from incidents. For certain abnormal loading events, the probability of an event occurring in a building increases with building size. In particular, high-rise buildings tend to be at a higher risk for gas and bomb explosions. In contrast, vehicular collisions affect primarily ground story areas.

1.4 Case studies of progressive collapse

Progressive collapse became an issue following the collapse of an apartment building at Ronan Point, London, U.K.[1], on May 16, 1968. A domestic gas explosion in a kitchen on the 18th floor of a 22-story precast building - estimated to be between 14 kPa and 83 kPa - blew out the exterior load bearing wall. The loss of support caused the floors above to collaspe, and the impact and weight of the falling debris caused the floors below to collapse as shown in Fig. 1.3. The particular type of joint detail used in the Ronan Point apartment building relied heavily on joint friction between precast panels.

Figure 1.3: Collapse of Ronan point apartment

Space trusses are highly redundant structures. This means that space truss structures are expected to survive even after losses of several members. However, the failure of the Hartford

(Connecticut) Coliseum space roof truss in 1978 (Ross, 1984)[2] showed that this assumption was not always correct. Progressive collapse can occur following the loss of one of several potentially critical members when a structure is subjected to full service loading. The collapse of arena at listowel, Ontario[3] under snow load on 28 february 1959 was the example of progressive collapse. The structure collapsed primarily because of defective workmanship in the glue-laminating of wood truss members. This is an example of a series of parallel trusses in which failure of one member initiated progressive collapse of the whole roof.Fig. 1.4.

Figure 1.4: Collapse of arena under snow load

The Jackson Skating Rink[1] was an unheated, covered skating rink in Durham, New Hampshire. After a heavy snow storm in 1996, the entire roof covering the ice collapsed completely. The roof structure was a pre-engineered rigid frame structure, approximately 210 ft long by 100 ft wide. There were 9 bents spaced at 21 ft, and column-and-beam end walls. At the time of failure the design load was approximately 1.9kPa due to accumulation of snow. The failure began at one end of the rink (Fig. 1.5) when anchorage for the thrust tie rods for one of the bends failed suddenly. The collapse of one bent caused the anchorage to fail at two adjacent bents.

Figure 1.5: Failed bents looking in direction of progressive collapse

Fig. 1.6[3] shows a case of progressive collapse where a truck took out the bracing for the upper chord of a bridge. It is an example of abnormal loadings due to vehicular collision.

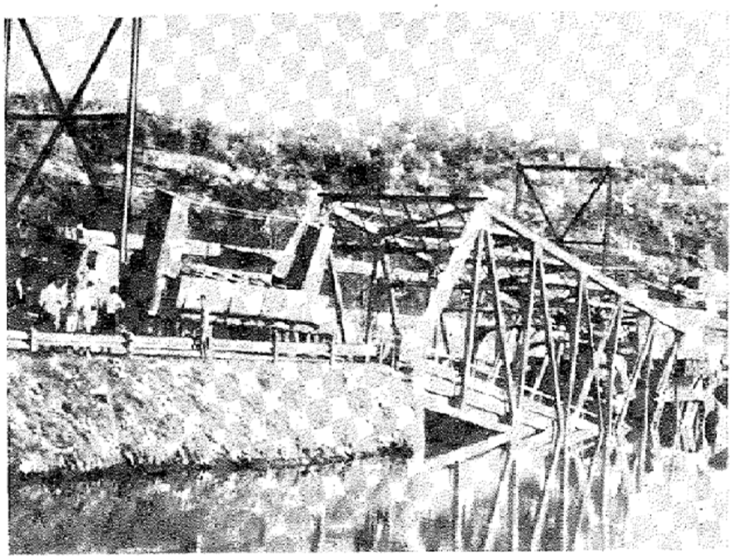


Figure 1.6: High truck load snaps top chord members and caused collapse

The collapse of Kansas City Hyatt Regency Hotel[1] in 1981 further attracted the attention of people to the issue of progressive collapse. Fig. 1.7 shows the collapse of Kansas City Regency Hotel, where collapse of 2^{nd} floor walkway occurred by following the 4^{th}

floor walkway collapse. The 2^{nd} and 4^{th} floor walkways shared a common suspension system, with the 2^{nd} floor walkway suspended directly below the 4^{th} floor walkway. The original design of the 2^{nd} and 4^{th} floor walkways called for them to be hung from the ceiling using continuous rods to support both walk-ways. In the final configuration the contractor elected not to support both walkways from the same steel rods but to hang the 4^{th} floor walkway from a set of rods that extended directly from the roof structure and terminated under the channels that were part of the framing system for the 4^{th} floor walkway. The 2^{nd} floor walkway was suspended, in turn, from separate rods which extended to the 4^{th} floor walkway and connected to the same framing channels there. This meant that the loads from the 2^{nd} floor walkway were transferred to the channels of the 4th floor walkway, and that the forces in the connections from the 4^{th} floor walkway to the rods to the roof were essentially twice as large as in the original design intent, which caused the collapse.

Figure 1.7: Collapse of Hyatt Regency Hotel

L'Ambiance Plaza was a 16-storey apartment building[1] under construction in Bridgeport, Connecticut. It was being erected using the lift-slab technique, which required the floor slabs to be cast on the ground and lifted into place by a jacking operation. In the afternoon of April 23, 1987, shortly after completion of one of the jacking operations, the building collapsed entirely. The structure had two-wings of post-tensioned concrete flat slabs supported on steel columns. Columns were installed in sections that were several stories high, through holes left for this purpose in the stack of slabs. Jacks were installed at the tops of these columns, and the slabs were jacked up the columns in groups. The lower slabs were connected in succession at their permanent locations on the columns. Upper-floor slabs could not be lifted to their final positions until the lower floors were in place and the full heights of the columns were installed. Hence, slabs needed to be temporarily parked at storage locations on the columns for periods of days while related construction proceeded. Parking was achieved by installing steel wedges under groups of three slabs. Failure began where workers were installing wedges. Apparently, there was sudden loss of support for one or more slabs, leading ultimately to the entire collapse of both wings of the building in a matter of seconds as shown in Fig. 1.8.

Figure 1.8: Collapse of L'Ambiance plaza

In case of World Trade Centre[1] the impact of the airplanes and the subsequent fires initiated local failures in the area of impact as shown in Fig. 1.9. This resulted in loss of vertical load carrying element in that area. This failed element moved in downward direction and created impact on the lower load carrying members. Failure progressed in the same manner and led to the total collapse of the building.

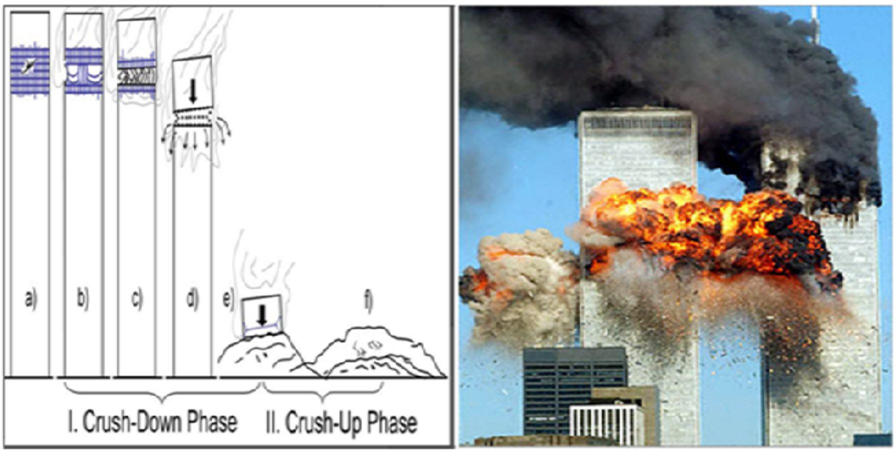


Figure 1.9: Collapse of world trade center

One experiment involved testing of a steel building scheduled for demolition in Northbrook, Illinois[4]. The demolition team tore out four selected columns from the building to simulate the sudden column removal that lead to progressive collapse. The structure was instrumented with strain gauges that recorded the change in strain in various structural members while the columns were removed. The strain values recorded in the field were compared with the results from a computer model of the building. The structure was a three story building located in Northbrook, Illinois. Built in 1968, the structure had reinforced concrete (RC) members in the basement, concrete slabs for the flooring, and was composed of steel framing on the first and second floors. The building had nine bays spanning 27 ft wide in the longitudinal direction, and 8 bays spanning 23 ft-6 in. in the transverse direction. The basement and first story are 10 ft-6 in. and 20 ft-6 in. in height. The heights of the lower and high points of the second story are 14 ft-8 in. and 15 ft-2 in., respectively. The entire experiment involved recording the strain on various structural members as four columns were removed from the north side of the building as shown in Fig. 1.10 and Fig. 1.11.

Figure 1.10: Bankers life and casualty company building

Figure 1.11: The circled columns were removed for experiment

The demolition team first exposed the columns and beams by removing the exterior brick wall. Then the surface of the columns and beam were grinded down to remove all paints and debris. Next, strain gauges were applied using an adhesive. The strain gauges were attached to a portable data acquisition scanner system and laptop. The strain values were recorded every tenth of a second during the column removal. During the column removal process, each column was weakened by a blow torch prior to its removal for safety reasons. The demolition team then melted a hole in each column between the torched lines. A chain was then attached to the hole and then the column was pulled out by a large backhoe. Fig. 1.12 shows the torched section of column being removed.

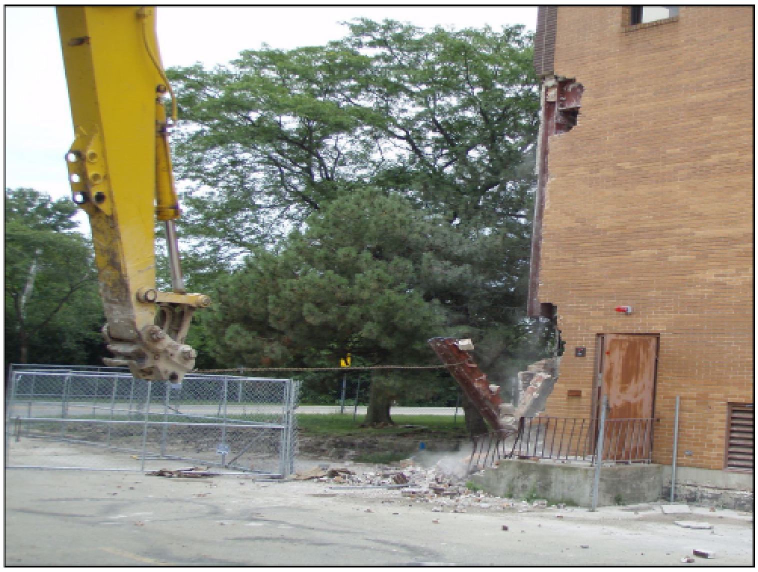


Figure 1.12: Torched section of column being removed

1.5 Objective of study

The objective of this study is to understand the progressive collapse analysis of steel building structure using GSA and DoD guidelines. The key objectives of study are as follows:

- To study the various causes of progressive collapse of steel building.
- To study and compare the various guidelines for progressive collapse analysis of steel building.
- To study the various analysis procedures for evaluation of potential of progressive collapse of moment resistant multistory steel building by considering various guidelines.
- To study the mitigation measures of progressive collapse and various techniques to improve the capacity of building to resist progressive collapse.

1.6 Scope of work

To achieve above objectives, the scope of work for major project is decided as follows:

- Understanding the causes of progressive collapse by studying various case studies of collapse of structures.
- Study of evolution of various guidelines for progressive collapse analysis.
- Comparison of various specifications of guidelines on progressive collapse analysis.
- Analysis and design of 4-storey and 9-storey regular moment resistant steel buildings.
- Progressive collapse analysis of 4-storey and 9-storey moment resistant steel building by linear static, linear dynamic, nonlinear static and nonlinear dynamic methods using GSA and DoD guidelines.
- Study of mitigation strategies for progressive collapse resistant of structures.

1.7 Organization of report

Content of the major project report is divided into following chapters.

Chapter-1 includes the definition and overview about progressive collapse phenomena. The mechanism of progressive collapse is discussed with the historical background. Various case studies of the progressive collapse of the buildings are also presented. It also includes objective of study, Scope of work and Organization of report.
In **chapter-2** brief literature review is presented pertaining to progressive collapse of steel structures. It includes of books, various guidelines and research papers.
Chapter-3 includes evolution and comparison of various guidelines. The comparison is organized by various provisions on Definition, threshold for consideration of progressive collapse, general strategy, loads, key elements and existing buildings.
Chapter-4 presents specifications of GSA and DoD guidelines. Progressive collapse analysis procedure, loading to perform static and dynamic analysis, internal and external column

removal consideration for regular structural configuration and acceptance criteria for Demand Capacity Ratio (DCR) as per GSA and DoD guidelines are discussed in this chapter.
In **chapter-5** analysis and design of 4-storey and 9-storey moment resistant steel buildings as per Indian standard is included. It includes progressive collapse analysis of 4-storey and 9-storey buildings using linear static and linear dynamic analysis as per GSA and DoD guidelines. Analysis is performed using structural analysis program SAP2000 by following alternate load path method. The DCR found using linear static analysis are compared with the DCR calculated from linear dynamic analysis at all storeys. The displacements found under the column removal locations from linear static and linear dynamic analysis are also compared.
Chapter-6 presents progressive collapse analysis of 4-storey and 9-storey steel buildings using nonlinear static and nonlinear dynamic analysis as per GSA and DoD guidelines.
In **chapter-7** various mitigation strategies to resist progressive collapse in steel building are presented. Three retrofit strategies (increasing strength only, increasing stiffness only and increasing both strength and stiffness) are discussed for mitigation of progressive collapse of the building.
Finally **chapter-8** summarizes the work carried out in the major project, important conclusions and future scope of work.

Chapter 2

Literature Review

2.1 General

Literature in form of research papers, books and guidelines regarding various aspects of progressive collapse analysis of steel structures are referred and review is presented in this chapter. The objective of literature review is to understand the current state of knowledge on progressive collapse analysis of buildings from a structural engineering point of view. The literature review includes results of research on various parameters related to progressive collapse such as causes, different analysis methods, limitations of various analysis methods, load combinations for various analysis methods, design philosophies, and mitigation methods.

2.2 Literature survey

A brief literature review is shown below for progressive collapse analysis of steel building structures.

2.2.1 Books and Guidelines

Krauthammer[5], in his book on "Modern Protective Structures", addresses a broad range of scientific and technical issues involved in mitigating the severe loading effects associated with blast, shock and impact. He discusses progressive collapse phenomena, progressive collapse of different types of structures like, precast concrete structures, monolithic concrete structures, truss structures and steel frame buildings. He also discusses the specifications of GSA and DoD guidelines. Example of 10-storey steel frame building is also considered for progressive collapse analysis using shear and moment connections. **Department of Defense**[6], of United States of America (USA) published the Unified Facilities Criteria UFC 4-023-03 (2009) for "Design of Buildings to Resist the Progressive Collapse". The guidelines incorporate the new knowledge related to design of buildings to resist progressive collapse. It includes steel beam-column connection, wood structure under blast damage and collapse loading, reinforced concrete slab response to large deformations. Guidelines are also provided for linear static, nonlinear static and nonlinear dynamic analysis methods.

Facts for steel buildings, blast and progressive collapse[7], published by (AISC) serves to provide the latest information and guidance available for commercial and industrial buildings subjected to extraordinary loads and responses. The document presents background and definitions for explosive loads and progressive collapse, general principles of blast loads and response prediction, recommendations for structures designed to resist blast and to mitigate progressive collapse, recent guidelines and Federal and DoD requirements, some observations from historical events, and some information on ongoing research.

General Services Administration[8], developed "Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects" to evaluate the potential of progressive collapse The guidelines provide a threat independent methodology for minimizing and assessing the progressive collapse potential in new and existing reinforced concrete and steel buildings.

National Institute of Standards and technology[1], (NIST) developed the document "Best Practices for Reducing the Potential for Progressive Collapse in Buildings". The main objective of the document is to provide best practices to engineers to minimize the progressive collapse of building in the event of abnormal loading. Practical means for reducing risk for new and existing buildings are presented in the document. The document also discusses the analysis methods for progressive collapse. A design consideration for different structural materials is summarized. The methodology for evaluating and mitigating progressive collapse potential in existing building is also discussed. Case studies of progressive collapse and progressive collapse provisions in various design standards are also presented.

2.2.2 Progressive collapse analysis

Vlassis et al.[9] proposed a new design-oriented methodology for progressive collapse assessment of floor systems within multi-storey buildings subject to impact from an above failed floor. The conceptual basis of the proposed framework was that the ability of the lower floor for arresting the falling floor depends on the amount of kinetic energy transmitted from the upper floor during impact. Three principal independent stages were employed in the proposed framework, including: (a) determination of the nonlinear static response of the impacted floor system, (b) dynamic assessment using a simplified energy balance approach, and (c) ductility assessment at the maximum level of dynamic deformation attained upon impact. The application of the proposed methodology was demonstrated by means of a case study, which considered the impact response of a floor plate within a typical multi-storey steel-framed composite building. Several possibilities regarding the location of the impacted floor plate, the nature of the impact event and the intensity of the gravity loads carried by the falling floor were examined.

Vlassis et al.[10] demonstrated the applicability of the new-design oriented methodology for progressive collapse assessment of multi-storey buildings by a case-study. A typical seven-storey steel framed composite building designed for office use was studied to demonstrate application of the proposed progressive collapse assessment method. The two principal scenarios investigated include removal of peripheral column and a corner column. To demonstrate the practicality of the proposed approach, assessment was based on the second lowest level of structural idealization associated with the response of a single floor plate. The case study had demonstrated that steel-framed composite buildings with typical structural configurations could be prone to progressive collapse initiated by local failure of a vertical support member. Susceptibility to progressive collapse was mainly related to the span sizes of the beams required to safely transfer the instantaneously applied gravity loads to the remaining undamaged structure as well as the joint detail used at the beam ends. The supply of additional slab reinforcements in the hogging moment regions could generally have a beneficial effect on the deformation capacities of the beams.

Lee et al.[11] proposed a parallel axial-flexural hinge model capable of representing post-yield flexural behavior and considering interaction effects of axial force and moment for a simplified nonlinear progressive collapse analysis of welded steel moment frames. The load-resisting mechanism of the column-removed double-span beams was investigated based on the material and geometric nonlinear parametric finite element analysis. A multi-linear parallel point hinge model was then proposed. The emphasis was to develop a reliable and computationally efficient macro model for practical collapse analysis. The application of the proposed hinge model to nonlinear dynamic progressive collapse analysis was illustrated by using OpenSEES program.

Fu[12] built a 3-D finite element model representing 20 storey building using the general purpose finite element package ABAQUS to perform the progressive collapse analysis. Shell elements and beam elements were used to simulate the whole building incorporating non-linear material characteristics and non-linear geometric behavior. The modeling techniques were described in detail. Numerical results were compared with the experimental data. Using this model, the structural behavior of the building under the sudden loss of columns for different structural systems and different scenarios of column removal were assessed in detail.

Kim and Park[13] studied progressive collapse potential of three- and nine-story special steel moment frames. Nonlinear static and dynamic procedures were followed It was observed that the model structures had high potential for progressive collapse when a first story column was suddenly removed. Then the size of beams required to satisfy the failure criteria for progressive collapse was obtained by the virtual work method; i.e., using the equilibrium of the external work done by gravity load due to loss of a column and the internal work done by plastic rotation of beams.

Kim and Kim[14] carried out the study on progressive collapse resisting capacity of steel moment resisting frames using GSA and DoD guidelines. In the study linear static, linear dynamic and nonlinear dynamic analysis procedures were used. 3-storey, 6-storey and 15-storey steel buildings were considered for progressive collapse analysis.

Kwasniewski[15] presented a case study of progressive collapse analysis of a selected multistory building. The numerical study was carried out for an existing 8-story steel framed structure built for fire tests in the Cardington Large Building Test Facility, UK. The problem was investigated using nonlinear dynamic finite element simulations carried out following the GSA guidelines. The paper focused on model development for global models subject to increasing vertical loading and notional column removal. Taking advantage of parallel processing on multiprocessor computers, a detailed 3D model with large number of finite elements had been developed for the entire structure.

Sasaki et al.[16] presented an analytical investigation into the effect on the redundancy of steel frame structures exerted by the loss of vertical structural members destroyed by aircraft crash and explosions. This examination was done to estimate the extent of a building's structural redundancy through an elasto-plastic analysis of three-dimensional frames based on the assumption that certain columns of the model building were lost. A typical high-rise steel-frame office building with a height of over 60 m was used as the model for analysis. Investigations were carried out on member loss at 4 separate locations. As a result, it was found that steel structural frames designed using joints with load-carrying capacity would remain standing even when multiple vertical load carrying members lost because the vertical loads could be redistributed to the remaining vertical structural members.

Marjanishvilli[17] discussed the four analysis procedures; linear static, linear dynamic, nonlinear static and nonlinear dynamic to evaluate the progressive collapse potential of 9-storied moment resistant steel building. Main objective of the study was to formulate an easy analysis procedure with reliable results using GSA guidelines. The advantages, disadvantages and limitations of each analysis procedure were discussed. Author concluded that most effective analysis procedure for progressive collapse evaluation incorporated the advantages of all the four analysis procedures.

Song and Sezen[18] carried out an experimental and analytical study of a 4-storey steel moment-resistant frame structure to investigate the progressive collapse performance. The linear static and nonlinear dynamic analysis were performed using the commercially available structural software SAP2000[19] by following the U.S. General Service Administration (GSA) guidelines. The results showed that columns at the top storey were most significantly in influenced by the column loss. The DCR (Demand Capacity Ratio) values in beams were smaller than the DCR values in columns due to the redistribution of loads to adjacent beams. The nonlinear dynamic analysis resulted in smaller displacements than linear static analysis.

Starossek[20] developed typology and classification of progressive collapse of structures that was founded on a study of the various underlying mechanisms of collapse. Six different types of collapse were described. (1) Pancake-type collapse: A pancake-type collapse exhibits features like initial failure of vertical load bearing elements, partial or complete separation and fall, in a vertical rigid body motion, of components, impact of separated and falling structural components on the remaining structure, collapse progression in the vertical direction. WTC collapse is the example of pancake-type collapse. (2) Zipper-type collapse: Characteristics features are the redistribution of forces into alternative paths, impulsive loading due to sudden element failure, and static and dynamic force concentration in the elements to fail next. The propagating action resulting from the failure of one element is the negative of the force in that element prior to failure acting as an impulsive loading at the point of failure. Impact forces do not occur. Principal forces in the failing elements and the propagating action, on the one hand, and the direction of failure propagation, on the other, are not parallel but more or less orthogonal. (3) Domino-type collapse: A trail of dominoes collapses in a fascinating chain reaction if one block falls at the push of a finger. A group of structures whose individual elements are at risk of overturning and are placed in a repetitive horizontal arrangement, just like a row of dominoes, could collapse in such a manner. (4) Section-type collapse: A beam under a bending moment or a bar under axial tension is considered. When a part of the respective cross section is cut, the internal forces transmitted by that part are redistributed into the remaining cross section. The corresponding increase in stress at some locations can cause the rupture of further cross sectional parts, and, in the same manner, a failure progression throughout the entire cross section. (5) Instability-type collapse: Instability of structures is characterized by small perturbations (imperfections, transverse loading) leading to large deformations or collapse. (6) Mixed-type collapse: Some collapses that have occurred in the past do not neatly fit into above categories are considered mixed-type collapse.

Bazant and Verdure[21] discussed the mechanism of progressive collapse from the case study of World Trade Center. After reviewing the mechanics of the towers, the motion during the crushing of one floor or group of floors and its energetics were analyzed, and a dynamic one-dimensional continuum model of progressive collapse was developed. Expressions for consistent energy potentials were formulated and an exact analytical solution of a special case was given. It was shown that progressive collapse would be triggered if the total (internal) energy loss during the crushing of one story (equal to the energy dissipated by the complete crushing and compaction of one story, minus the loss of gravity potential during the crushing of that story) exceeds the kinetic energy impacted to that story, regardless of the load capacity of the columns.

2.2.3 Mitigation of progressive collapse

Astaneh[22] carried out the experimental work to investigate the viability of a steel cable based system to prevent progressive collapse of building. Ten tests were conducted on full scale specimen of a one storey building. One side of the floor in the specimen had steel cables placed within the floor representing new construction and the other side had cables placed on the outside as a measure of retrofit of existing buildings.

Kim and Kim[23] carried out the study on the progressive collapse resisting capacity of the Reduced Beam Section (RBS), Welded cover plated flange (WCPF), and welded un reinforced flange welded web (WUF-W) connections, which were seismic connections recommended by the FEMA, was investigated. For progressive collapse analysis, two types of steel moment frame buildings were considered; one designed for high-seismic load and the other designed for moderate seismic load. The vertical displacement at the point of column removal and the plastic hinge rotation at beam ends were checked by using an alternative load path method proposed in the guidelines.

Khandelwal et al.[24] conducted study on previously designed 10 storied steel buildings by applying the alternate path method. In this methodology, critical columns and adjacent braces, if present, were instantaneously removed from an analysis model and the ability of the model to successfully absorb member loss was investigated. Two types of steel buildings were considered as per bracing configurations i.e. concentric and eccentric bracing systems for progressive collapse analysis of the structure.

Galal and El-Sawy[25] studied the effect of three retrofit strategies on enhancing the response of existing steel moment resisting frames designed for gravity loads using Alternate path methods recommended in GSA and DoD guidelines for resisting progressive collapse. The response was evaluated using 3-D nonlinear dynamic analysis. The studied models represent 6-bay by 3-bay 18-storey steel frames that were damaged by being subjected to six scenarios of sudden removal of one column in the ground floor. Four buildings with bay spans of 5.0m, 6.0m, 7.5m, and 9.0m were studied. The response of the damaged frames was evaluated when retrofitted using three approaches, namely, increasing the strength of the beams, increasing the stiffness of the beams, and increasing both strength and stiffness of the beams. The objective of this paper was to asses effectiveness of the retrofit strategies by evaluating the enhancement in three performance indicators which were chord rotation, tie forces, and displacement ductility demand for the beams of the studied building after being retrofitted.

Hamburger[26] suggested the use of moment-resisting framing at each floor level to redistribute loads away from failed elements to alternative load paths as the most commonly employed strategy to provide progressive collapse resistance. Design criteria commonly employed for this purpose typically rely on the flexural action of the framing to redistribute loads and account for limited member ductility and over strength using elastic analysis to approximate true inelastic behavior. This paper discussed the importance of catenary behavior in the framing elements. He also discussed the importance of the steel framing connections capable of resisting large tensile demands simultaneously with large flexural deformations.

Alashker et al.[27] discussed the progressive collapse resistance of steel-concrete composite floors in which steel beams were attached to columns through shear tabs. The study was conducted using computational simulation models validated through extensive comparisons to disparate test data. The effects of deck thickness, steel reinforcement, and the number of bolts in the shear tab connection on the behavior of the system were discussed as a function of a loading scheme. The concrete deck was modeled using eight-node brick elements. The inelastic behavior of concrete was represented using a three dimensional, three-invariant, non associative, concrete plasticity model. The welded wire fabric mesh in the slab was modeled using truss elements. The steel deck was modeled using shell elements. The shear tab connection was represented using one single row of shell elements with a thickness equal to that of the beam web. The shear studs connecting the beam top flanges to the concrete slab through the metal deck were modeled using beam elements that were embedded in the concrete slab and fully bonded to the concrete elements. The simulations were conducted using LS-DYNA; an explicit, general purpose, finite element software. The prototype steel framed building used in this study was designed by the NIST for the purpose of studying its response to an event which may cause progressive collapse. The building, which was a 10-story office structure, had plan dimensions of 30.5 m X 45.7 m and utilized moment-resisting frames located around the perimeter of the building for lateral load resistance. Gravity frames were used for the interior of the building. This paper studied the robustness of composite floor systems with single shear tab connections for the scenario of sudden removal of a center column.

2.3 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes research on various aspects related to progressive collapse such as causes, types, differences between analysis methods, limitations of analysis methods, load combinations for analysis, design philosophies, structural design solutions. It also includes studies on mitigation of progressive collapse and various retrofit techniques to reduce the potential of progressive collapse. This review helps to develop basic understanding of progressive collapse analysis of steel buildings.

Chapter 3

Comparison of various guidelines

3.1 General

Immediately following the Ronan Point collapse, some countries, such as the U.K. and Canada adopted some regulatory measures to address prevention of progressive collapse. In the 1980s, design standards in the U.S. began to incorporate requirements for "general structural integrity" to provide nominal resistance to progressive collapse. Recent terrorist attacks on buildings throughout the world, particularly U.S. owned and occupied buildings, several U.S. government agencies have developed their own requirements to provide resistance against progressive collapse. This chapter provides a survey and comparison of existing building standards on progressive collapse.

3.2 Comparison of progressive collapse provisions

The comparison of various provisions are based on the following criteria.

- Definition
- Threshold for consideration of progressive collapse
- General Strategy
- Loads

- Key elements
- Existing buildings

3.2.1 Definition

Most definitions of progressive collapse encompass the "house of cards" effect, where by damage spreads beyond a local region. Damage is assumed local if it is limited to 15% or 20% of floor or roof area, depending on the standards; or to one structural bay or the floors immediately adjacent to the initial damage.

Following are the definitions of progressive collapse, local collapse, and structural integrity used in various building standards;

British Standards BS 5950-1:2000:[28] The British Standards do not use the words progressive collapse but rather structural collapse disproportionate to the initial cause. This contrasts with the local collapse, which is limited to 15% of floor or roof area or $100m^2$, whichever is less, at the relevant level and at one immediately adjacent level, either above or below it.

National Building Code of Canada (NBCC):[29] "Progressive collapse is the phenomenon in which the spread of an initial local failure from element to element eventually results in the collapse of a whole building or disproportionately large parts of it."

American Society of Civil Engineers (2005), ASCE 7-05[30]: "Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage."

New York City Building Code (1998):[31] "Progressive collapse is interpreted as structural failure extending vertically over more than three stories, and horizontally over an area more than $100m^2$ or 20% of the horizontal area of the building whichever is less."

Department of Defense (2005)[6], "Design of Buildings to Resist Progressive Collapse": The definition of progressive collapse given in ASCE-7 is adopted.

General Services Administration (2003):[8] "Progressive collapse is a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause."

3.2.2 Threshold for consideration of progressive collapse

From the study of various building standards that contain provisions for structural integrity, it is observed that some standards do not mention thresholds, and by default apply these provisions to all buildings. Other standards recommend consideration of progressive collapse only for buildings that are above a certain height, or whose failure could cause severe loss of human life. Table 3.1 shows the threshold limits for consideration of progressive collapse in different guidelines.

3.2.3 General strategy

With varying emphasis, most standards refer to three methods of mitigating progressive collapse. The first is to reduce exposure to hazards, the second method considers resistance to progressive collapse during the design process and is therefore called the direct design method. The third method is the indirect design method. Following are the provisions on the strategy given by various building standards to resist progressive collapse.

British Standards: The standards recommend the following approaches to mitigate progressive collapse:

- Tie building together
- Remove notionally, one at a time, columns and horizontal force resisting elements, and investigate structural stability.
- Design structural members as key elements where necessary.

Eurocode: Eurocode makes the following general recommendations:

- Avoid, eliminate, or reduce hazards.
- Select structural form with low sensitivity to hazards considered.

Table 3.1: Threshold for consideration of progressive collapse

GSA Guidelines (2003)	Building occupancy building type,proximity of moving or parked vehicles, seismic design and others
Department of Defense UFC 4-010-01	Buildings > 3 stories
British Standards Steel Concrete Masonry Timber	 All Buildings All Buildings Buildings > 5 stories Buildings > 5 stories
Eurocode 2002	**Consequence Classes:** **Low** : 1 to 3 stories : No consideration **Medium** : 3 to 6 stories: Eurocode robustness and stability rules **High** : 7 to 10 stories: simplified static analysis, prescriptive detailing rules **Severe** : >10 stories: Dynamic nonlinear analysis, load-structure interaction
Swedish Regulations	**Safety Classes:** **Little risk of serious injury** : No consideration **Some risk of serious injuries** : Consider only in multi-story buildings **Great risk of serious injury** : Mandatory consideration
Precast Concrete Institute (1976)	Horizontal ties in all buildings Vertical ties in buildings over two stories

- Select structural form that can survive accidental removal of an individual element, a limited part of the structure, or the occurrence of acceptable localized damage.
- Avoid structural systems that may collapse without warning.
- Tie structure together.

NBCC: NBCC provide the following guidance:

- Lower the risk of accidents: Prevent storage of gas or other explosive materials. Provide fender against vehicles.
- Ductility: Design connections to be ”ductile and capable of large deformations and energy absorption under the effects of abnormal conditions.”

- Design for abnormal loads: Key elements, whose failure by a foreseeable abnormal event would initiate progressive collapse, should be designed to remain just functional under that condition.

- Alternate paths: Usually the safest method of coping with progressive collapse is to design the structure in such a way that it can bridge the gap left when a structural component is removed.

ASCE 7: It recommends following methods and approaches:

- Provide sufficient continuity, redundancy, or energy dissipating capacity or a combination thereof, in the members of the structures.

- Identify extraordinary events with a probability of occurrence in the range of 10^{-6}/years to 10^{-4}/year or greater, and ensure key load-bearing elements can withstand such events.

- Minimum tie force between structural elements should be 20 kN/m.

- As elastic analysis may vastly underestimate the capacity of the structure, non-linear or plastic analysis may be used.

DoD guidelines: It recommends following approaches:

- Maximize standoff distance.

- Design all additions to existing buildings to be structurally independent.

- Areas that do not meet criteria for inhabited buildings should be structurally independent from the habited areas;

- Avoid building overhangs.

- Use highly redundant structural system such as moment resisting frame.

- Provide continuity across joints equal to the capacity of the connected members.

- Design all exterior columns to sustain a loss of lateral support to any floor level by adding one storey height to the nominal unsupported length.

- This provision also applies to internal columns where parking beneath building is unavoidable.

GSA guidelines: A series of flowcharts guides the designer through an exemption process to help him decide if the buildings needs to be designed against progressive collapse or not. Following general guidance is offered:

- Use redundant lateral and vertical elements and detailing;
- Design against shear failure
- Design for an additional storey of unsupported length, columns along the perimeter of the facility, between the first and the third floor above grade, and all columns in public areas or uncontrolled parking areas;
- Design to resist load reversals for facilities with uncontrolled parking areas or public areas: at least one structural bay deep around the perimeter from ground level to roof level, and for all interior structural bays floor at least three floors above grade.
- Account for three dimensional effects.

World Trade Center Building Code Task Force (2003): It follows three design methods against progressive collapse.

- Indirect design: The principal feature of this method consists of tying the building together.
- Direct design: The alternate path method allows local failure to occur but provides alternate load paths to bridge over the damage and avoid collapse.
- Direct design: The specific local resistance method consists in strengthening locally key elements against unanticipated loads without failing the connections, or supporting members framing it. The structure shall be detailed to permit load reversals.

3.2.4 Loadings for progressive collapse analysis

Accidental loads may be grouped as pressure loads (e.g., explosions, detonations, tornado wind pressures), impact (e.g., vehicular collision, aircraft or missile impact, debris), or as faulty construction practice. Particularly for the mitigation of progressive collapse various provisions for accidental loads, lateral loads in zones where seismic and wind forces do not govern the design, and combinations of loads for which the building stability should be checked. The loads to be combined reflect the small probability of the accidental load and the design live, snow or wind loads. Table 3.2 compares load combinations from various standards.

Table 3.2: Load combinations for progressive collapse analysis

Standards	**Load combination after column removal**	**Accidental Load**
BS	(1+0.5) D + L/3 + W/3	34 kPa
Eurocode 2003		20 kPa
ASCE 7-98, 02, 05	(0.9 or 1.2) D + (0.5L or 0.2S) + 0.2 W	A_k
Canada 1977	D + L/3 + W/3	
DoD UFC 4-010-01	D + 0.5L net floor uplift	
DoD UFC 4-023-03	D + 0.5L net floor uplift (0.9 or 1.2) D + (0.5L or 0.2S) + 0.2 W (NLD) 2[(0.9 or 1.2) D + (0.5L or 0.2S)] + 0.2 W (static analysis)	
GSA	2(D + 0.25L) static analysis (D + 0.25L) dynamic analysis	
NYC 1998, 2003	2D + 0.25L + 0.2W	

where,

D, L, W, S = Dead, Live, Wind and Snow Load respectively

NLD = Nonlinear dynamic

A_k = extraordinary load

Debris falling from a damaged floor above justifies doubling the dead load “2D” in some of the load combinations above.

3.2.5 Key elements

Key elements are defined as structural elements whose notional removal could cause collapse of an unacceptable extent. Following are details of various building standards.

BS 5950-1:2000: "If the notional removal of column, or of an element of a system providing resistance to horizontal forces, would risk the collapse of a greater area that column or element should be designed as a key element. Any other steel member or other structural elements that provide lateral restraint vital to the stability of a key element should it self also be designed as a key element for the same accidental loading".

New York City Building Code: Any single element essential to the stability of the structure, together with its structural connections, shall not fail under the loads stipulated in this criterion after being subjected to a load equivalent to the uniform pressure of 720psf.

World Trade Center Building Code Task Force (2003): Key elements should be strengthened locally against unanticipated loads without failing the connections or supporting members framing it. The structure should be detailed to permit load reversals.

Department of Defense (2005): The following guidance is provided for designing to resist a specific threat:

"Where there is a known risk of terrorist attack, but no specific terrorist threat is defined; in this case, the goal is to reduce the risk of mass casualties in the event of an attack".

3.2.6 Existing buildings

Interagency Security Committee (ISC):[32] "Existing buildings will not be retrofitted to prevent progressive collapse unless they are undergoing a structural renovation, such as a seismic upgrade . Prior to the submission for funding, all structures shall be analyzed according to requirements for new construction, and a written report shall clearly state the potential vulnerability of the building to progressive collapse".

GSA guidelines: The GSA guidelines incorporate an exemption process that takes into account the use, occupancy, and type of the facility, proximity of moving or parked vehicles, as well as structural features such as seismic design, to help the user decide whether the potential for progressive collapse needs to be considered.

DoD guidelines: DoD guidelines are applicable equally to new and existing facilities. The following actions are specified for different levels of protection:

- Very low level of protection "If a structural element does not provide the required horizontal tie force capacity, it must be re-designed in the case of new construction or retrofitted in the case of existing construction".

- Low level of protection "For elements with inadequate horizontal tie force capacity, the alternate path method can not be used. In this case the designer must re-design the element in the case of new construction or retrofit the element in the case of existing construction".

- Medium and high level of protection "For elements with inadequate horizontal tie force capacity, the designer must re-design the element in the case of new construction or retrofit the element for existing construction".

3.3 Summary

From the study of various standards it is observed that a number of building standards around the world contain specific provisions for design against progressive collapse, whereas other standards rely on more general provisions dealing with structural integrity and robustness.

Chapter 4

GSA and DoD guidelines

4.1 General

From the review of various standards regarding progressive collapse analysis as discussed in chapter-3, two guidelines published by U.S. are considered in detail in this chapter. The "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" is developed by the U.S. General Service Administration to evaluate the potential of progressive collapse for existing reinforced concrete and steel framed buildings. Similarly, Department of Defense of United States of America published the document, Unified Facilities Criteria (UFC) 4-023-03 "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. This chapter includes important specifications of GSA and DoD guidelines for progressive collapse analysis.

4.2 GSA guidelines

This guideline provides a "threat independent" methodology for minimizing the potential for progressive collapse in the design of new buildings. A threat independent approach is, however, prescribed as it is not feasible to rationally examine all potential sources of collapse initiation. The approach taken (i.e., the removal of a column or other vertical load bearing member) is not intended to reproduce or replicate any specific abnormal load or assault on the structure. Rather, member removal is simply used as a “load initiator and serves as a means to introduce redundancy and resiliency into the structure.

4.2.1 Philosophy of guideline

This Guideline addresses the need to protect human life and prevent injury as well as the protection of buildings, functions and assets. The Guideline take a flexible and realistic approach to the reliability and safety of buildings. The approach described in the guideline utilizes a flow-chart methodology to determine if the facility under consideration might be exempt from detailed consideration for progressive collapse. In other words, a series of questions must be answered that identify whether or not further progressive collapse considerations are required. Following parameters must be considered.

- Building occupancy
- Building category
- Number of stories
- Seismic zone
- Detailed description of local structural attributes
- Description of significant global structural attributes

The outcome of these answers leads to either (1) an exemption (no further consideration required) or (2) the need to further consider the potential for progressive collapse. The detailed analysis required in the latter case is intended to reduce the probability of progressive collapse for new construction and identify the potential for progressive collapse in existing construction. For new construction and existing construction if the facility is determined not to be exempt from further consideration for progressive collapse following methodologies should be executed.

4.2.2 New construction

All newly constructed facilities shall be designed with the intent of reducing the potential for progressive collapse. The process presented in these Guidelines consists of an analysis/redesign approach. This method is intended to enhance the probability that if localized damage occurs as the result of an abnormal loading event, the structure will not progressively collapse to an extent disproportionate to the original cause of the damage.

Design guidance: Structural design guidance is provided for consideration during the initial structural design phase and prior to performing the progressive collapse analysis. It is critical that floor girders and beams be capable of spanning two full spans (i.e., a double span condition consisting of two full bays) as a minimum. This requires both beam-to-beam structural continuity across the removed column, as well as the ability of girders and beams to deform flexurally well beyond their elastic limit without experiencing structural collapse. It is therefore necessary that the local beam-to-column connection characteristics be implemented during the initial phases of structural design. The incorporation of these features will increase the probability of achieving a low potential for progressive collapse when performing the analysis.

Analysis: Linear static analysis approach may be used to assess the potential for progressive collapse in all new and upgraded construction. Analysis approach is coupled with the following:

- Criteria for assessing the analysis results
- A suite of analysis cases
- Specific loading criteria to be used in the analysis

The following analysis considerations shall be used in the assessment for progressive collapse for typical or symmetrical structural configurations.

Exterior consideration

The following analysis cases shall be considered for framed structure. Fig. 4.1 shows the exterior column removal positions.

a. Analyze for the instantaneous loss of a column for one floor above grade located at or near the middle of the short side of the building.

b. Analyze for the instantaneous loss of a column for one floor above grade located at or near the middle of the long side of the building.

c. Analyze for the instantaneous loss of a column for one floor above grade located at the corner of the building.

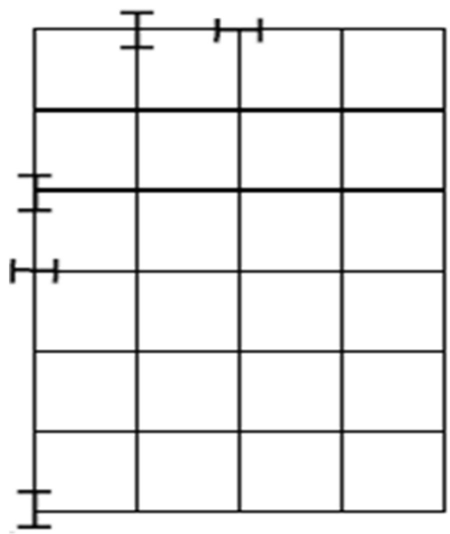


Figure 4.1: Exterior consideration for column removal

Interior consideration

Facilities that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis case. Fig. 4.2 shows the exterior column removal positions.

a. Analyze for the instantaneous loss of one column that extends from the floor of the underground parking area or uncontrolled public ground floor area to the next floor.

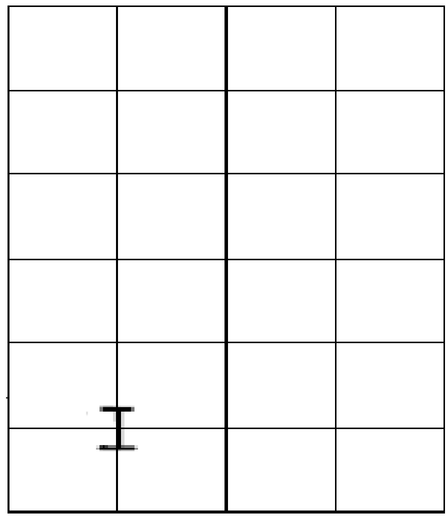


Figure 4.2: Interior consideration for column removal

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

Load = 2(DL + 0.25LL)

Where, DL= Dead load; LL = Live load

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

Load = DL + 0.25LL

Analysis criteria: Structural collapse resulting from the instantaneous removal of a primary vertical support shall be limited. The allowable extent of collapse for the instantaneous removal of a primary vertical support member along the exterior and within the interior of a building is defined as follows.

Exterior consideration

The maximum allowable extents of collapse resulting from the instantaneous removal of an exterior primary vertical support member one floor above grade shall be confined to:

- the structural bays directly associated with the instantaneously removed vertical member in the floor level directly above the instantaneously removed vertical member

- 1800 ft^2 at the floor level directly above the instantaneously removed vertical member whichever is less as shown in Fig. 4.3.

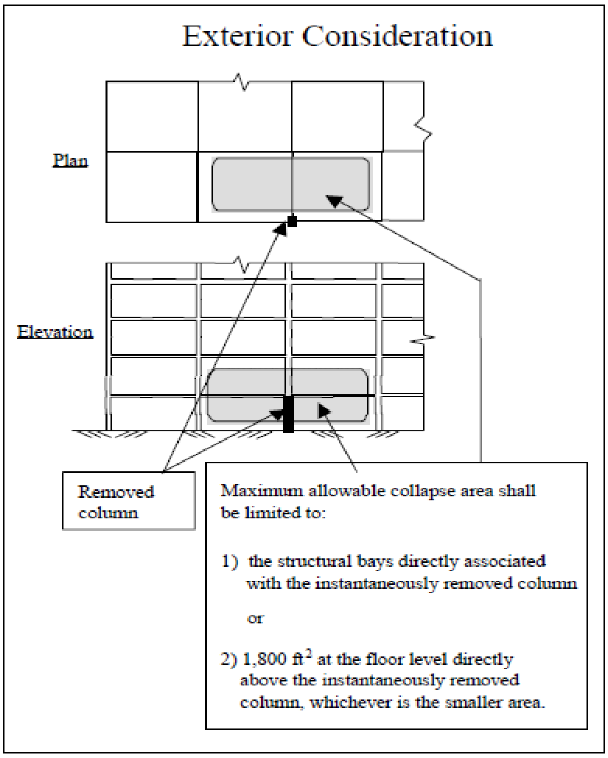


Figure 4.3: Maximum allowable collapse area for exterior consideration

Interior consideration

The allowable extents of collapse resulting from the instantaneous removal of an interior primary vertical support member in an uncontrolled ground floor area and/or an underground parking area for one floor level shall be confined to:

- the structural bays directly associated with the instantaneously removed vertical member
- 3600 ft^2 at the floor level directly above the instantaneously removed vertical member

whichever is less as shown in Fig. 4.4.

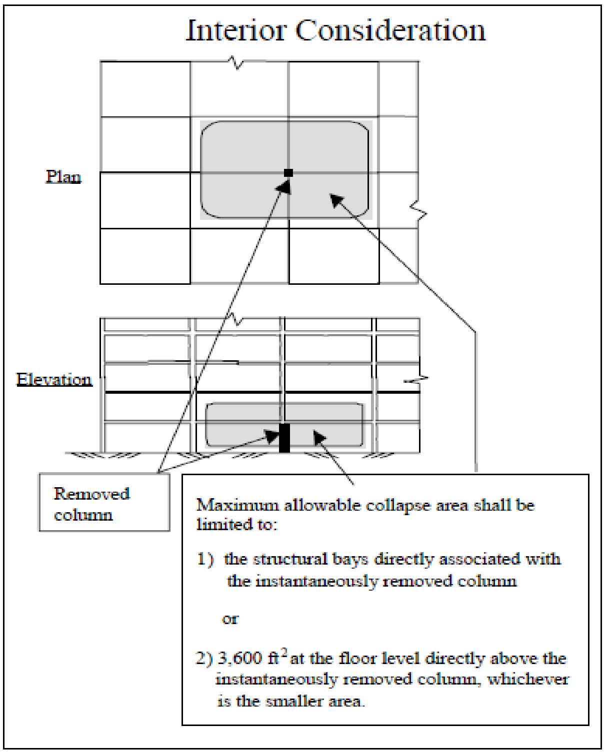


Figure 4.4: Maximum allowable collapse area for interior consideration

Analysis procedure: The step-by-step procedure for conducting the linear static analysis is as follows.

Step-1: Remove a vertical support from the location being considered and conduct a linear-static analysis of the structure. Load the model with loading defined earlier.

Step-2: Determine which members and connections have demand capacity ratio (DCR) values that exceed the acceptance criteria. For the primary and secondary structural components DCR can be determined as:

$$DCR = \frac{Q_{UD}}{Q_{UC}} \tag{4.1}$$

Where,

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

Q_{UC} = Expected ultimate, un-factored capacity of the member and connection (moment, axial force, shear and possible combined forces)

Step-3: For a member whose DCR exceeds the permissible values, place a hinge at the member end to release the moment. This hinge should be located at the center of flexural yielding for the member. Use rigid offsets from the connecting member as needed to model the hinge in the proper location. For yielding at the end of a member the center of flexural yielding should not be taken to be more than the depth of the member from the face of the intersecting member.

Step-4: At each inserted hinge, apply equal-but-opposite moments to the offset and member end to each side of the hinge as shown in Fig. 4.5. The magnitude of the moments should equal the expected flexural strength of the moment, and the direction of the moments should be consistent with direction of the moments in the analysis performed in Step 1.

Step-5: Re-run the analysis and repeat Steps 1 to 4. Continue this process until no DCR values are exceeded.

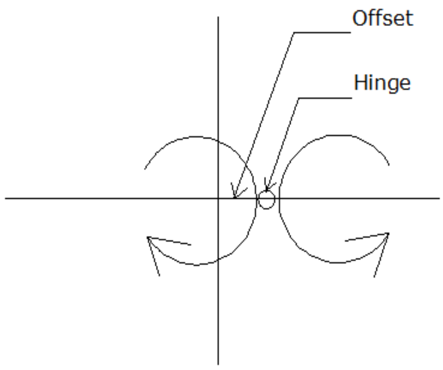


Figure 4.5: Placement of hinge

4.2.3 Acceptance Criteria

Linear static analysis results shall be performed to identify the magnitudes and distribution of potential demands on both the primary and secondary structural elements. Upon removing the selected column from the structure, an assessment is made as to which beams,

columns, joints, have exceeded their respective maximum allowable demands. The magnitude and distribution of demands will be indicated by DCR. Member ends exceeding their respective DCR values will then be released and their end moments re-distributed.
The allowable DCR values for primary and secondary structural elements are:

- DCR $\leq$ 2.0 for typical structural configurations.
- DCR $\leq$ (3/4)* DCR for atypical structural configurations.

4.2.4 Existing construction

Existing facilities undergoing modernization should be upgraded to new construction requirements when required by the project specific facility security risk assessment and when feasible. In addition, facilities undergoing modernization should, as a minimum, assess the potential for progressive collapse as the result of an abnormal loading event.

4.3 DoD guidelines

The UFC provides planning, design, construction, sustainment, restoration, and modernization criteria and applies to the Military departments, the defense agencies and the DoD field activities. This UFC provides the design requirements necessary to reduce the potential of progressive collapse for new and existing facilities that experience localized structural damage through normally unforeseeable events. UFC suggests that the level of progressive collapse analysis is based on the Occupancy Category (OC) of the structure for both new and existing buildings. These Occupancy Categories are as defined in UFC 3-310-01.
Definitions of OC are shown in Table 4.1.
The design requirements in this UFC are developed such that varying levels of resistance to progressive collapse are specified, depending upon the OC as shown in Table 4.2. These levels of progressive collapse employ three design/analysis approaches: Tie Forces (TF), Alternate Path (AP), and Enhanced Local Resistance (ELR).

Table 4.1: Definitions of Occupancy categories

Occupancy Category	Nature of occupancy
I	Agricultural facilities, certain temporary facilities, Minor storage facilities etc.
II	Buildings and other structures except those listed in categories I,III,IV and V
III	Buildings where more than 300 people congregate in one area schools etc.
IV	Hospitals and other health care facilities, police stations, fire stations etc.
V	Key national defense assets, emergency backup power generating facilities etc.

4.3.1 Tie force approach

In the Tie Force approach, the building is mechanically tied together, enhancing continuity, ductility, and development of alternate load paths. Fig. 4.6 illustrates these tie forces for frame construction.

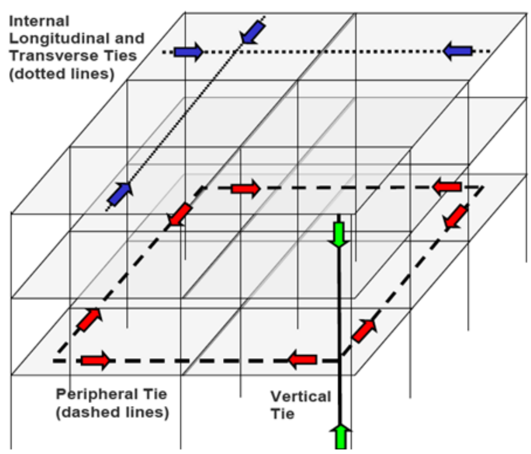


Figure 4.6: Tie forces in a frame structure

Table 4.2: Analysis Requirements for various Occupancy categories

Occupancy Category	**Analysis Requirements**
I	No specific requirements
II	Option 1: Tie Forces for the entire structure and Enhanced Local Resistance for the corner and Penultimate columns or walls at the first story. **OR** Option 2: Alternate Path for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first storey columns or walls
IV	Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.

There are three horizontal ties that must be provided: longitudinal, transverse, and peripheral. Unless the structural members (beams, girders, spandrels) and their connections can be shown capable of carrying the required tie force magnitudes while undergoing rotations of 0.20-rad (11.3-deg), tie forces are to be carried by the floor and roof system.

The floor load (uniform) to determine the required tie strengths is

W_F= 1.2 DL + 0.5LL

The required tie strength, distribution, and location for longitudinal, transverse, peripheral, and vertical ties are defined as follows.

Longitudinal and transverse ties:

Use the floor and roof system to provide the required longitudinal and transverse tie resistance. The longitudinal and transverse ties must be anchored to peripheral ties at each end. Spacing must not be greater than 0.2 L_T, or 0.2 L_L where L_T and L_L are the greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the transverse and longitudinal directions, respectively.

The required tie strength Fi (lb/ft or kN/m) in the longitudinal or transverse direction is

F_i = 3 W_F L_1

Where,

W_F = Floor load in (kN/m^2)

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

Peripheral Ties:

Use the floor and roof system to carry the required peripheral tie strength. Place peripheral ties within 3-ft (0.91-m) of the edge of a floor or roof and provide adequate development or anchors at corners, re-entrant corners or changes of construction. The required peripheral tie strength F_p (lb or kN) is

F_p = 6 W_F L_1 L_p

Where,

W_F = Floor load, determined (kN/m^2)

L_1 = for exterior peripheral ties, the greater of the distances between the centers of the columns, frames, or walls at the perimeter of the building in the direction under consideration (m or ft).

L_p = 3-ft (0.91-m)

Fig. 4.7 shows the peripheral and internal ties for non-uniform floor loads.

Vertical ties

Use the columns and load-bearing walls to carry the required vertical tie strength. Each column and load-bearing wall shall be tied continuously from the foundation to the roof level The vertical tie must have a design strength in tension equal to the largest vertical load received by the column or wall from any one story, using the tributary area and the floor load W_F.

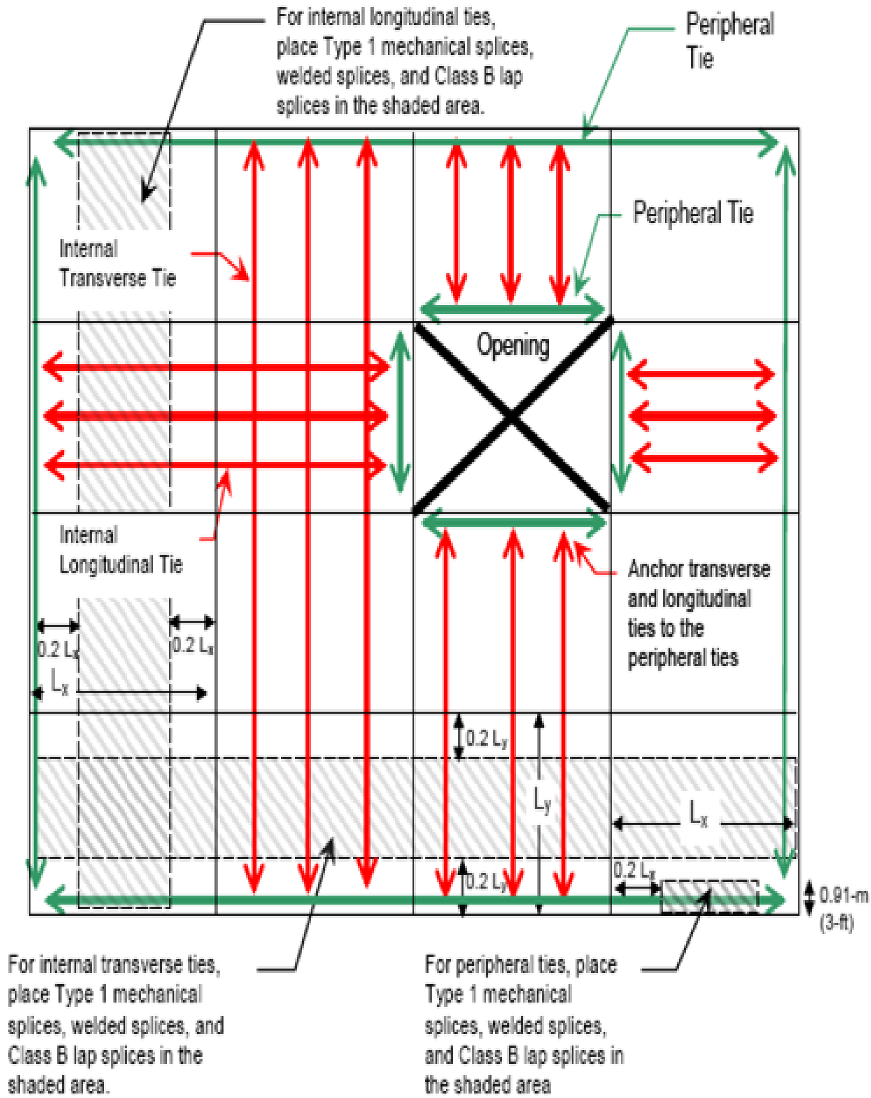


Figure 4.7: Peripheral and internal ties for non-uniform floor loads

4.3.2 Alternate path method

The structure is designed such that if any one member fails, alternate load paths are available for the load that was in that component and a general collapse does not occur. This method follows the Load and resistance factor design (LRFD) philosophy. Three analysis procedures are employed: Linear static (LSP), nonlinear static (NSP) and nonlinear dynamic (NDP). Following the LFRD approach, the design strength provided by a member and its connections to other members must be greater than or equal to the required strength.

Removal of load bearing element for alternate path method

For OC II (Option-1), each column that cannot provide the required vertical tie force, remove the clear height between the lateral restraints. For OC II, OC III and OC IV (Option-2), for each column remove the clear height between lateral restraints.

Location of removal of load bearing elements

For OC II (Option-1), remove the column that cannot provide the required vertical tie force. For OC II, OC III and OC IV (Option-2), remove external columns near the middle of the short side, near the middle of the long side, and at the corner of the building. For OC II Option 2, OC III and OC IV structures with underground parking or other areas of uncontrolled public access, remove internal columns near the middle of the short side, near the middle of the long side and at the corner of the uncontrolled space. Fig. 4.8 and 4.9 show external and internal column removal positions.

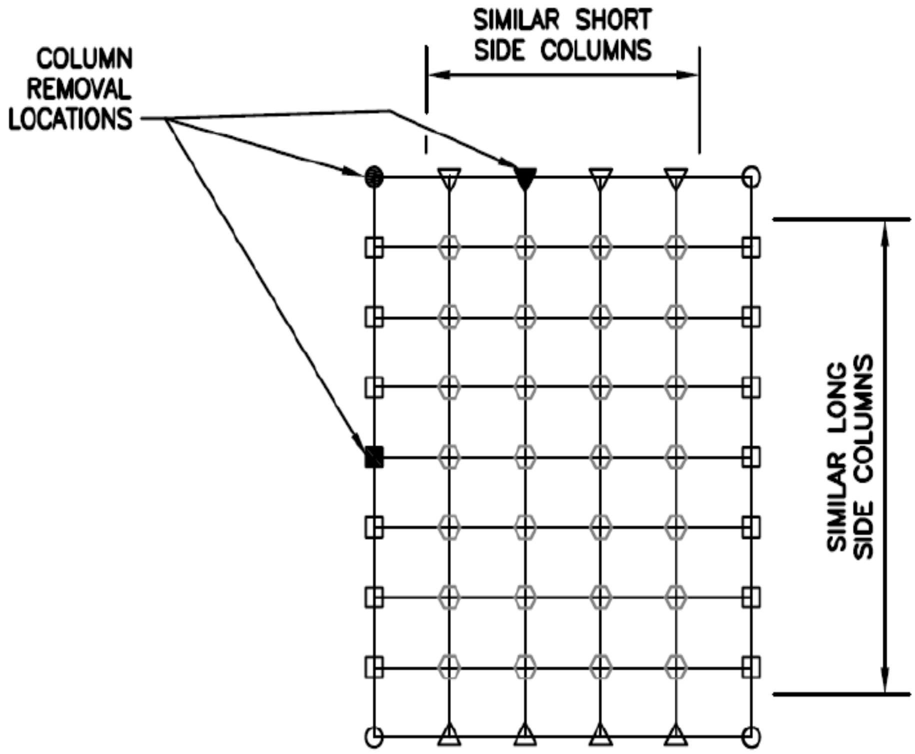


Figure 4.8: External column removal for OC III and IV

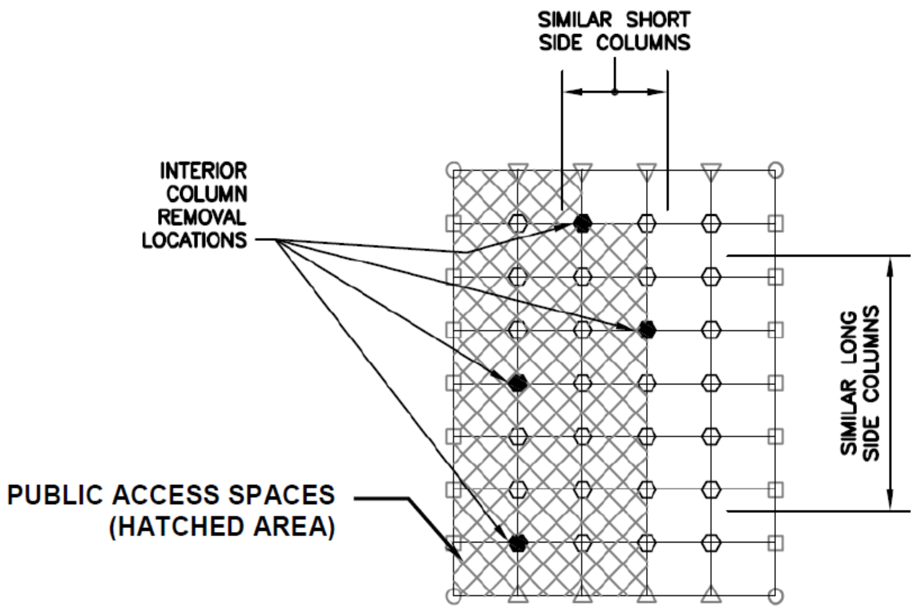


Figure 4.9: Internal column removal for OC III and IV

Structure acceptance criteria:

If there are no structural irregularities, a linear static procedure may be performed. If the structure is irregular, a linear static procedure may be performed if all of the component DCR determined are less than or equal to 2. If the structure is irregular and one or more of the DCR exceed 2, then a linear static procedure cannot be used.

Loadings for LSP

Due to the different methods by which deformation-controlled and force-controlled actions are calculated, two load cases will be applied and analyzed: one for the deformation-controlled actions, and one for the force-controlled actions.

To calculate the deformation-controlled actions, apply the following combination of gravity and lateral loads:

G_{LD} = Ω_{LD} [(0.9 or 1.2) D + (0.5 L or 0.2 S)]

G_{LD} = Increased gravity loads for deformation-controlled actions

D = Dead load including facade loads (kN/m^2)

L = Live load (kN/m^2)

S = Snow load (kN/m^2)

Ω_{LD}= Load increase factor (from ASCE41) for calculating deformation-controlled actions for Linear Static analysis

Apply the following gravity load combination to those bays not loaded with G_{LD}. G= (0.9 or 1.2) D + (0.5 L or 0.2 S)

where G= Gravity loads

Apply the following lateral load to each side of the building one side at a time, i.e., four separate analysis must be performed, one for each principal direction of the building, in combination with the gravity loads.

L_{LAT} = 0.002ΣP

Where,

L_{LAT} = Lateral load

0.002ΣP = Notional lateral load applied at each floor; this load is applied to every floor on each face of the building, one face at a time

ΣP = Sum of the gravity loads (Dead and Live) acting on only that floor; load increase factors are not employed.

To calculate the force-controlled actions, simultaneously apply the following combination of gravity and lateral loads.

G_{LF} = Ω_{LF} [(0.9 or 1.2) D + (0.5 L or 0.2 S)]

G_{LF} = Increased gravity loads for force-controlled actions.

Ω_{LF} = Load increase factor for calculating force-controlled actions; (2 for force-controlled action)

Apply the lateral loadings as defined above. Fig. 4.10 shows the loads and load locations for external and internal column removal.

Loadings for NSP

To calculate the deformation-controlled and force-controlled actions, apply the following combination of gravity and lateral loads.

G_N = Ω_N [(0.9 or 1.2) D + (0.5 L or 0.2 S)]

G_N = Increased gravity loads for force-controlled actions.

Ω_N = Dynamic increase factor for calculating deformation-controlled and force-controlled actions. Apply the lateral loadings same as defined for LSP.

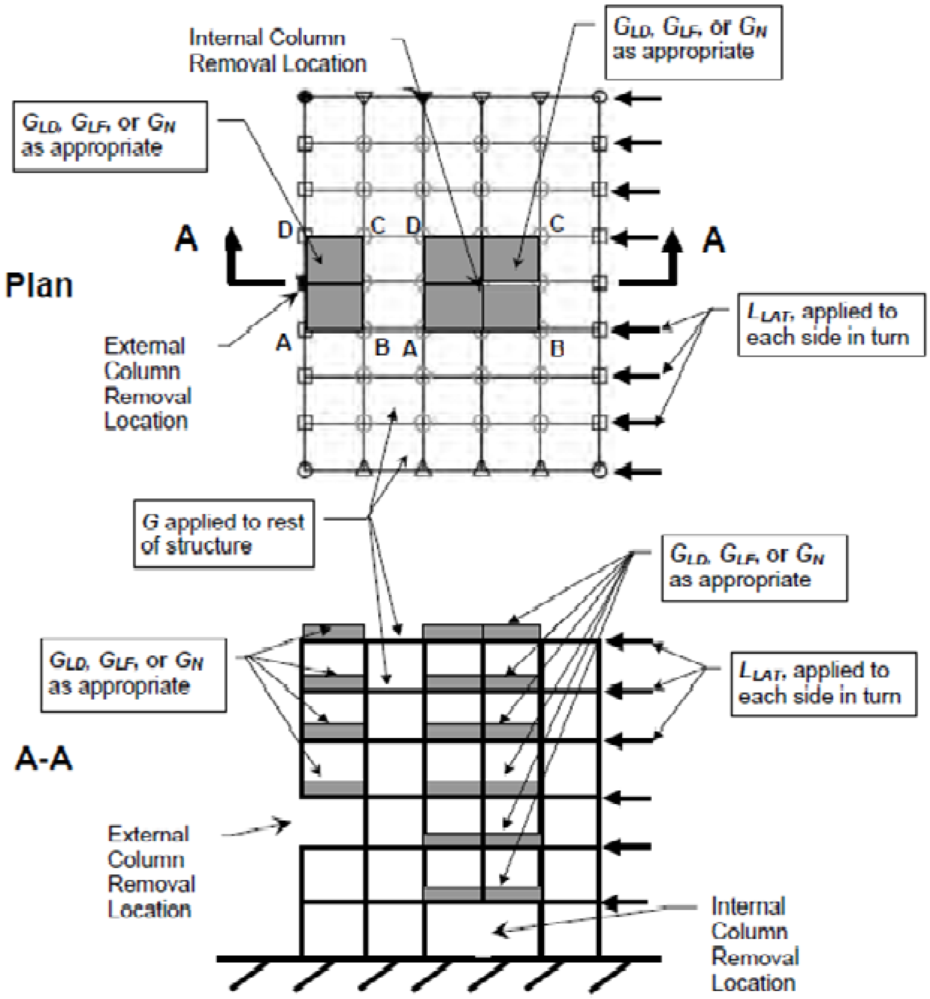


Figure 4.10: Loads and load locations for external and internal column removal

Apply the loads using a load history that starts at zero and is increased to the final values. Apply at least 10 load steps to reach the total load. The software must be capable of incrementally increasing the load and iteratively reaching convergence before proceeding to the next load increment.

Loadings for NDP

To calculate the deformation-controlled and force-controlled actions, apply the following combination of gravity and lateral loads. Apply the following gravity load combination to the entire structure.

G_{ND} = [(0.9 or 1.2) D + (0.5 L or 0.2 S)]

G_{ND}= Gravity loads for non linear dynamic analysis

Apply the lateral loadings same as defined for LSP.
Starting at zero load, monotonically and proportionately increase the gravity loads and lateral loads to the entire model (i.e., the column or wall section have not been removed yet) until equilibrium is reached. After equilibrium is reached, remove the column. While it is preferable to remove the column instantaneously, the duration for removal must be less than one tenth of the period associated with the structural response mode for the vertical motion of the bays above the removed column. The analysis shall continue until the maximum displacement is reached or one cycle of vertical motion occurs at the column removal location.

4.3.3 Enhanced local resistance

Enhanced local resistance (ELR) is provided through the prescribed flexure and shear resistance of perimeter building columns and load bearing walls.

ELR location requirements

For OC II option 1, ELR is applied to the perimeter corner and penultimate columns of the first storey above grade. For OC III, ELR is applied to all perimeter columns of the first storey above grade. For OC IV, ELR is applied to all perimeter columns of the first two stories above grade.

4.4 Summary

Various specifications of GSA and DoD guidelines are discussed in this chapter. Progressive collapse analysis procedures followed by both the guidelines are discussed. Various parameters for progressive collapse analysis like load cases for static and dynamic procedures, column removal locations and acceptance criteria are discussed.

Chapter 5

Linear static and dynamic analysis

5.1 General

To study the effect of failure of load carrying elements i.e. columns on the entire structure; 4-storey and 9-storey moment resistant steel buildings are considered. The buildings are modeled and analyzed for progressive collapse using the structural analysis and design software SAP2000. There are total four analysis procedures namely linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis. In this chapter linear static and linear dynamic analysis are presented to evaluate the potential for progressive collapse of buildings.

5.2 Building geometry

The buildings considered are 4-storey and 9-storey moment resistant steel buildings, with six bays in the longitudinal direction and three bays in the transverse direction. The longitudinal direction has a uniform column spacing of 8.25 m while transverse direction has a uniform column spacing of 9.75 m. Floor-to-floor height for each storey is 4.3 m. Fig. 5.1 shows the three dimensional model of 4-storey steel building.

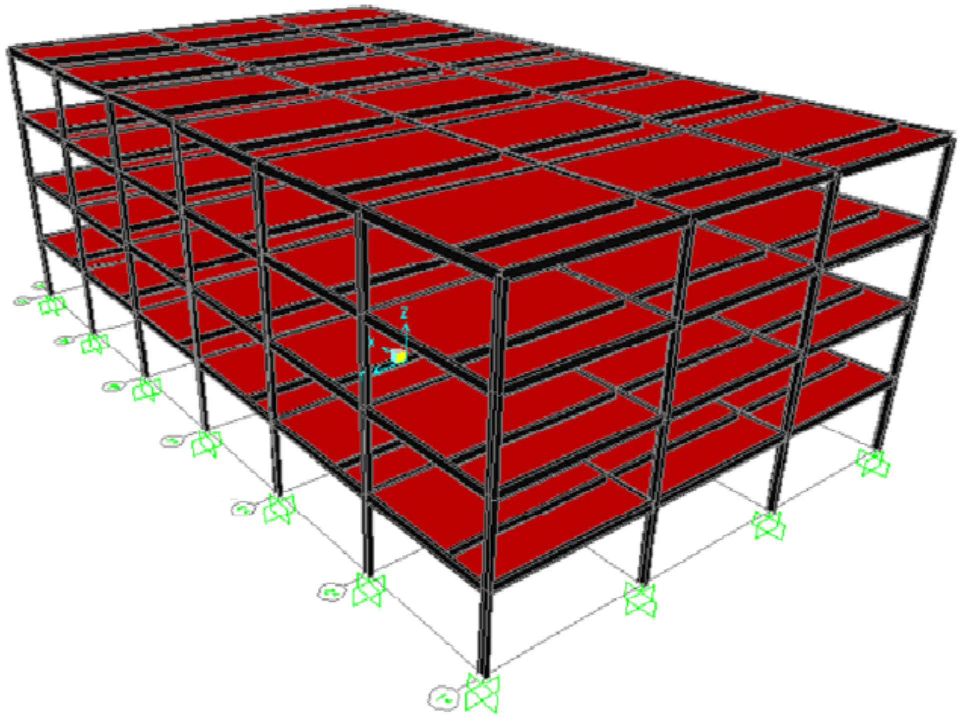


Figure 5.1: Three dimensional model of example building

5.3 Loadings

Dead load

- Self weight of the structure
- Thickness of the slab = 90mm
- Wall load = 19.7 kN/m at every floor except roof

Live load

- 1.9 kN/m^2 distributed uniformly across the entire floor area including roof

Seismic Loading parameters

- Location: Ahmedabad
- Zone: III
- Importance factor: 1

- Response reduction factor: 5
- Soil type:II

Material properties

- Yield strength: 250 MPa
- Modulus of elasticity: 2×10^5 MPa

Analysis and design of the building is carried out in STAAD Pro. Load combinations are as per from IS:800-1984[33] for analysis and design of the buildings. Fig. 5.2 shows the structural plan of the building. Table 5.1 shows column and beam schedule for 4-storey and 9-storey buildings.

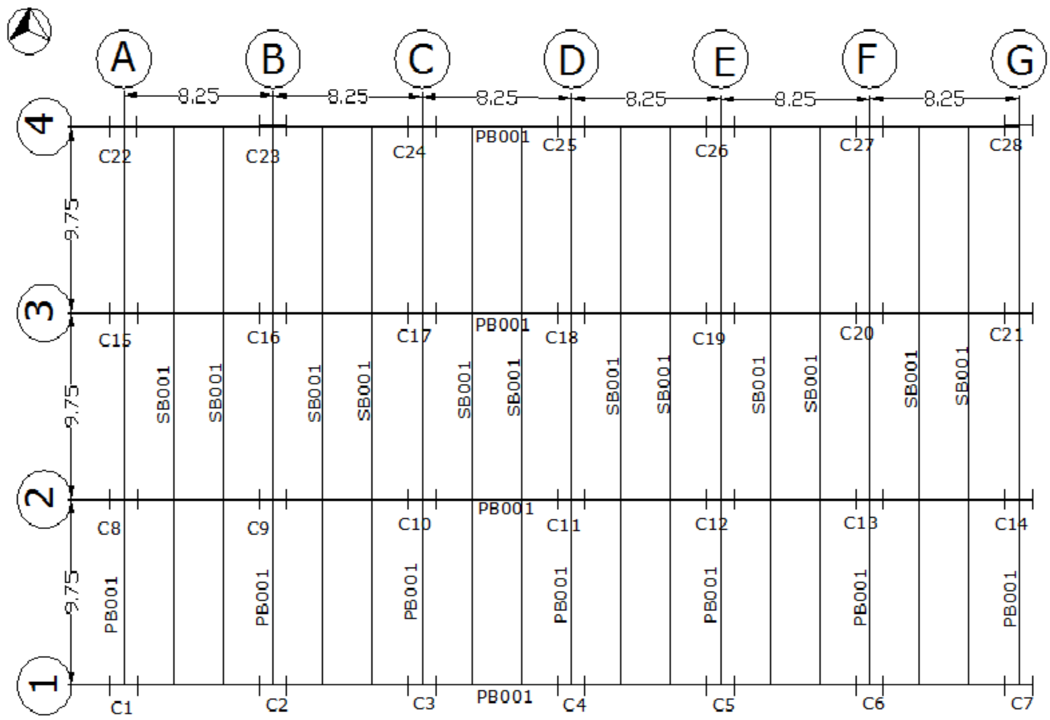
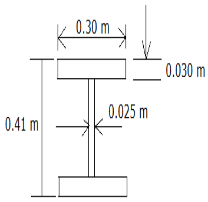
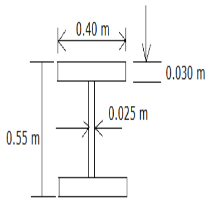


Figure 5.2: Plan of 4-storey and 9-storey moment resistant steel buildings

Table 5.1: Column and beam schedule for 4-storey and 9-storey buildings

	Primary beam	Secondary beam	Column
4-storey	PB001-PB004	SB001-SB004	
	ISMB500	ISMB450	
9-storey	PB001-PB009	SB001-SB009	
	ISMB500	ISMB450	

5.4 Progressive collapse analysis

Progressive collapse analysis is performed by instantly removing one or several columns and analyzing the building's remaining capability to absorb the damage. The key issue in progressive collapse is in understanding that it is a dynamic event, and that the motion is initiated by a release of internal energy due to the instantaneous loss of a structural member.This member loss disturbs the initial load equilibrium of external loads and internal forces, and the structure then vibrates until a new equilibrium position is found or until the structure collapses. Four column removal cases for progressive collapse analysis are considered. For case-1 middle column from long side of the building (C4) is removed, for case-2 column of shorter side of the building (C15) is removed, for case-3 corner column (C1) is removed, for case-4 interior column (C11) is removed. Fig. 5.3 shows the column removal locations. SAP2000 software is used to understand the behavior of structure under different "failed column" scenarios.

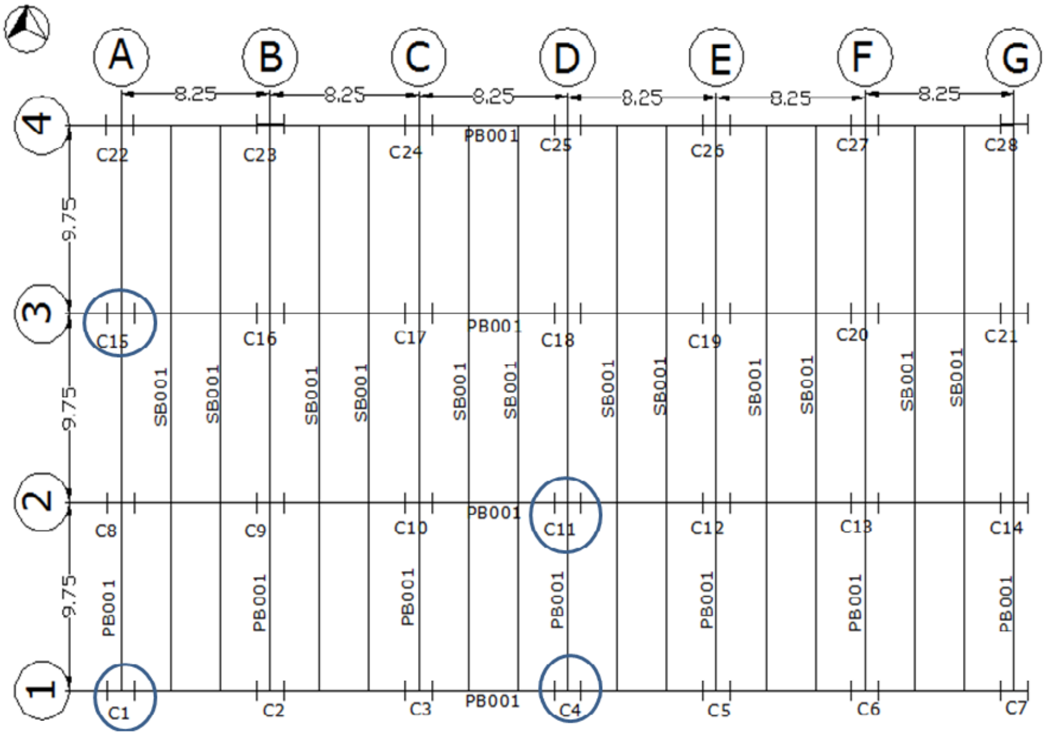


Figure 5.3: Column removal locations

5.4.1 Linear static analysis

The linear static analysis of a structure involves the solution of the system of linear equations represented by:

K u = r

Where K is the stiffness matrix, r is the vector of applied loads, and u is the vector of resulting displacements. For progressive collapse analysis, column is removed from the location being considered and analysis is carried out for following vertical loads which shall be applied downward on the structure.

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

As per DoD guideline, Load = 2(1.2DL + 0.5LL)

Where, factor 2 takes into account the dynamic effect of suddenly applied load.

Fig. 5.4 shows the application of GSA loadings for case-1 column removal. Loading of 2(DL + 0.25LL) is applied on the affected portion at all floors as shown in the figure and on the remaining portion (DL + 0.25LL) loading is applied.

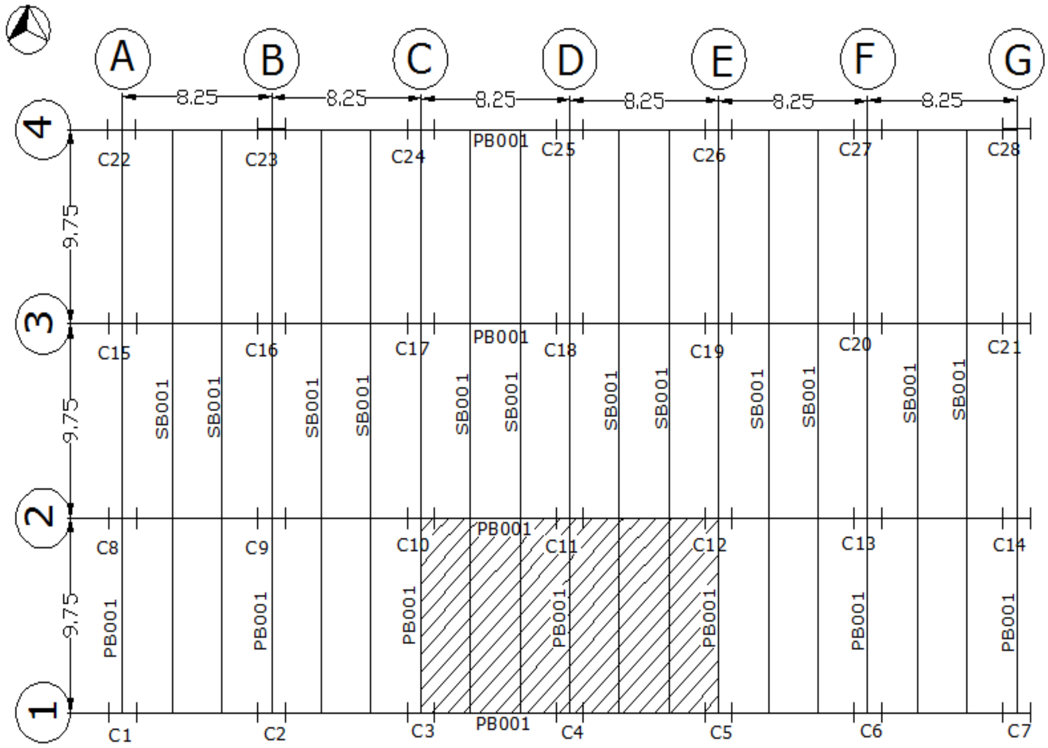


Figure 5.4: Loadings applied over the affected portion for case-1 by GSA

DoD guidelines include lateral loads also for lateral stability and P-Δ effects. Apply the following lateral load to each side of the building one side at a time. This requires four separate analysis cases, one for each principal direction of the building, in combination with the gravity loads.

$L_{LAT} = 0.002\Sigma P$

Where L_{LAT} = Lateral load

ΣP = Sum of the gravity loads (Dead and Live)

Fig. 5.5 shows the application of loadings as per DoD guidelines. Loading of 2(1.2DL + 0.5LL) is applied on the affected portion at all floors as shown in the figure and on the remaining portion (1.2DL + 0.5LL) loading is applied. In addition to gravity loads lateral loads are also applied in X-direction. Similarly Fig. 5.6 shows the application of gravity loads and lateral loads in Y-direction as per DoD guidelines.

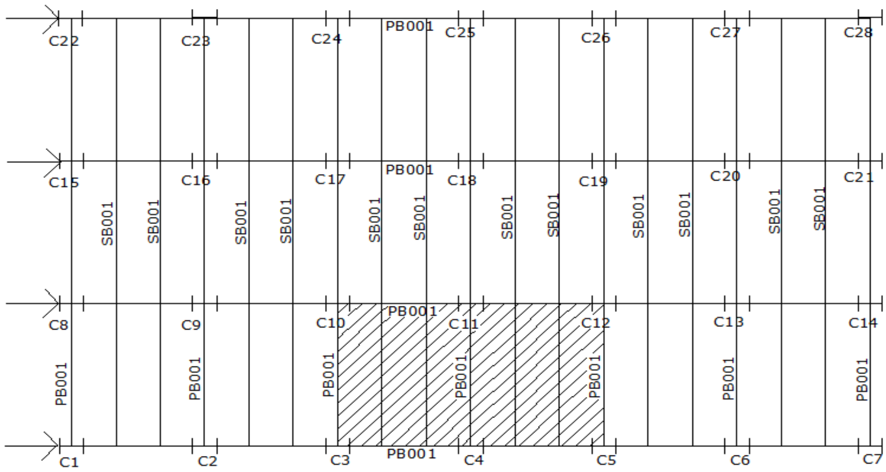


Figure 5.5: Loadings applied as per DoD guidelines for +X direction

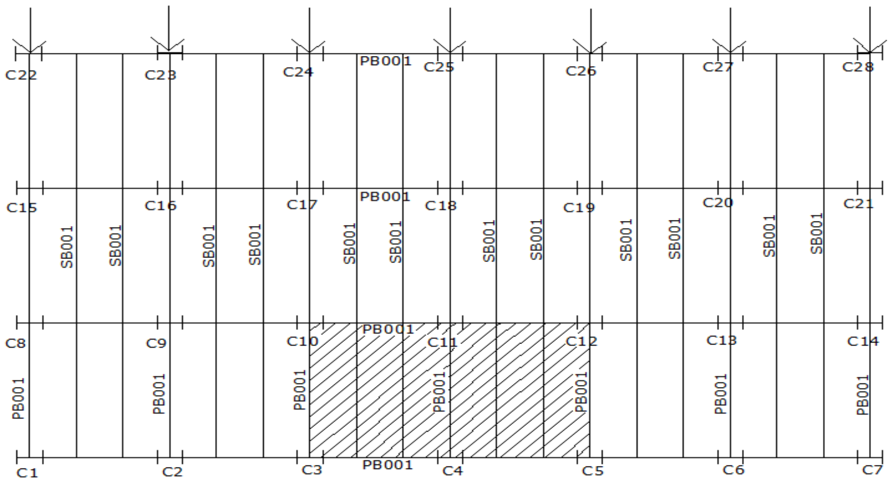


Figure 5.6: Loadings applied as per DoD guidelines for +Y direction

This analysis procedure is simple to perform and applicable to regular structures. From the analysis results demand to capacity ratios (DCR) are found. This analysis procedure involves the following steps:

a. Build a computer model.

b. Remove the column from the location being considered.

c. Apply the amplified static load combinations as per GSA and DoD guidelines.

d. Perform static linear analysis, a standard analysis procedure in SAP2000; and

e. Evaluate the results based on demand to capacity ratios (DCR).

Linear static analysis case has been defined in SAP2000 as per GSA and DoD guidelines as shown in the Fig.5.7.

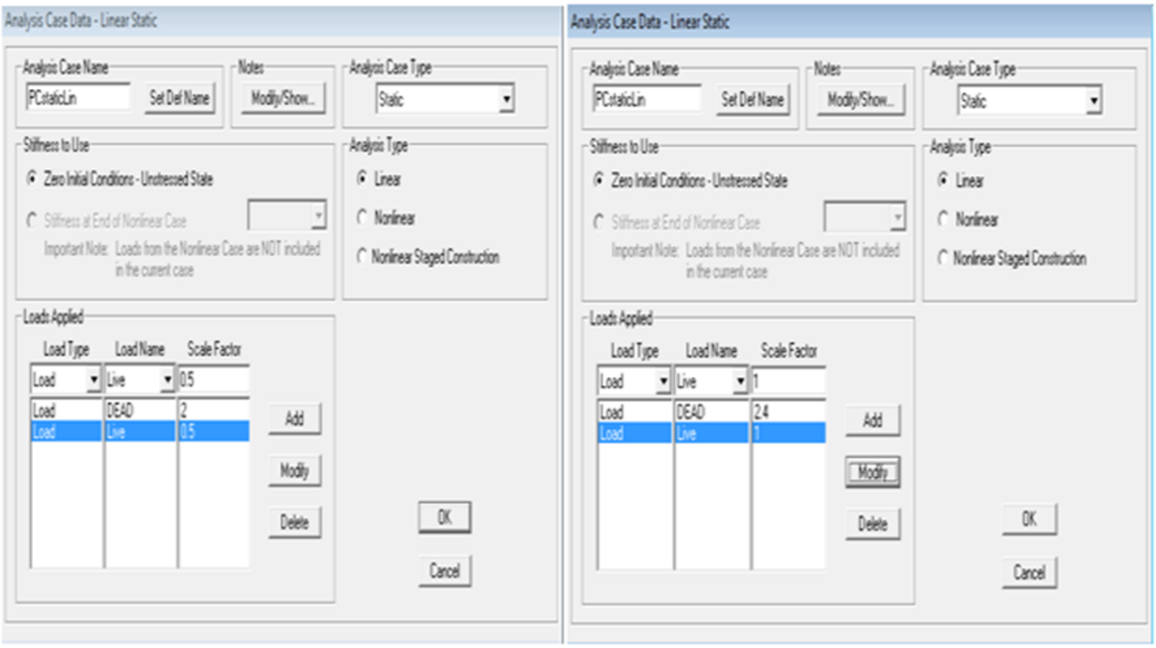


Figure 5.7: Linear static analysis case definition by GSA and DoD guidelines

Linear static analysis using staged construction SAP2000NL software is used for this analysis. The procedure followed for this example can be generally applied in any structural software capable of performing nonlinear static analysis. The "Staged Construction option in SAP2000 can be used to ensure proper redistribution of loads upon member removal. In this procedure, each column that is to be removed is assigned to a separate group. In this study, removal of four columns at ground floor is demonstrated. Columns are removed at four plan locations one at a time. The "Staged Construction" option in SAP2000 allows for the creation of separate analysis cases to automate the removal of columns. Create analysis cases which capture the stiffness for column removal. To do this, click staged construction button. In stage 1 add ALL, in stage 2 remove the column under investigation. Using these staged construction analysis cases as the initial stiffness, add a new analysis case for each column being removed. For example consider case-1 of column removal for 4-storey building. Now define new group that is GROUP1. Assign GROUP1 to the column being removed (C4 in this case). Fig. 5.8 shows the definition of staged construction analysis.

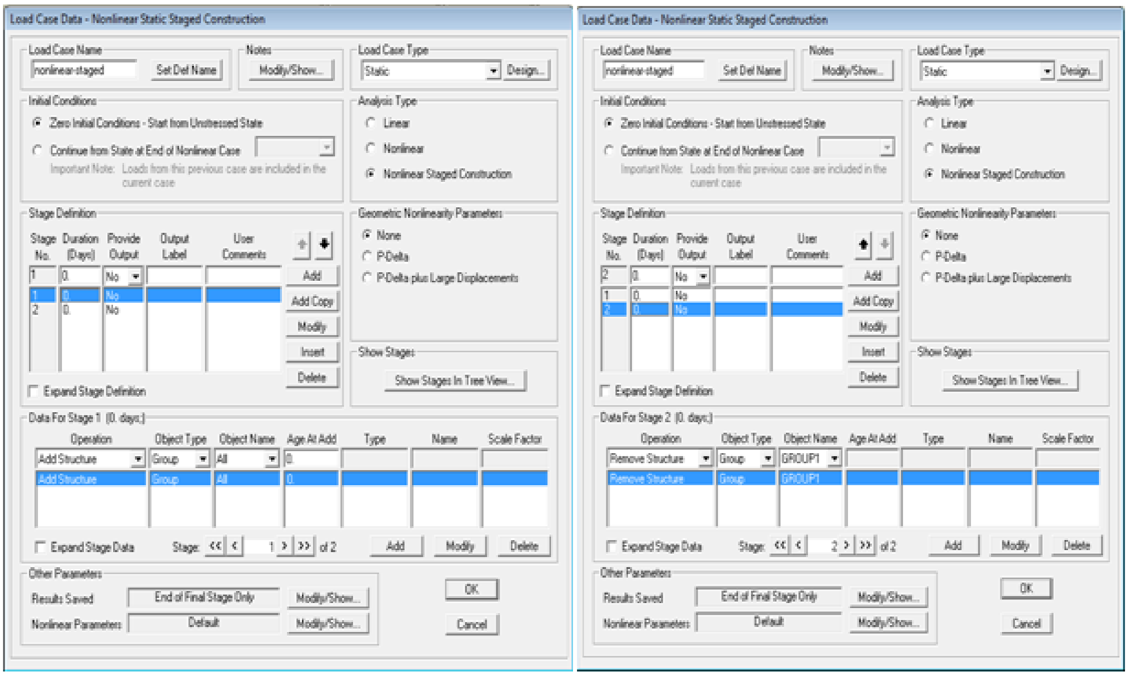


Figure 5.8: Staged construction definition

Fig. 5.9 shows the definition of linear static analysis case (as per GSA guideline) after staged construction. Use of staged construction option gives the similar results for linear static analysis case, defined without using staged construction.

5.4.2 Linear dynamic analysis

The failure of vertical members under abnormal loading is a highly dynamic phenomenon. So it is necessary to study the response of building structure by performing dynamic analysis. Dynamic analysis procedures (either linear or nonlinear) are usually avoided, as they

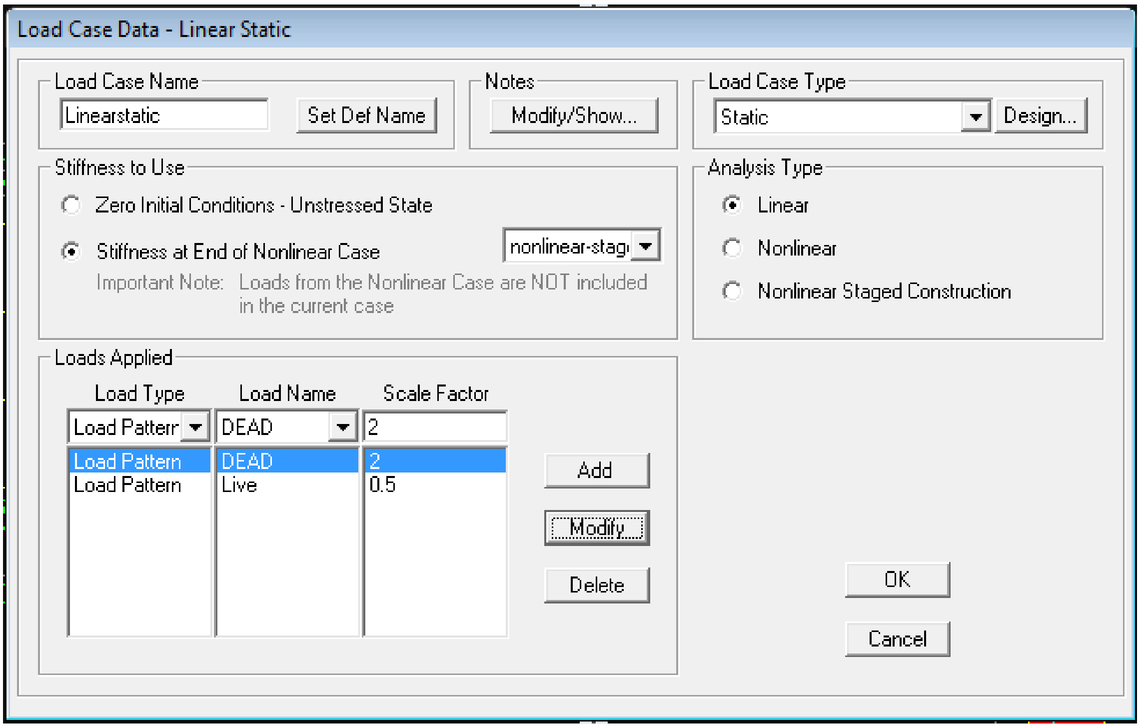


Figure 5.9: Definition of Linear static analysis case

are perceived to be excessively complex. But compared to static analysis procedures, their accuracy is much higher since dynamic procedures incorporate dynamic amplification factors, inertia, and damping forces. It is more appropriate to refer to this method of analysis as a time history analysis. Time-history analysis is a step-by-step analysis of the dynamic response of a structure to a specified loading that may vary with time. Time- history analysis is used to determine the dynamic response of a structure to arbitrary loading. The dynamic equilibrium equation to be solved is given by:

K u(t) + C $\dot{u}$ (t) + M $\ddot{u}$ (t) = r (t)

Where K is the stiffness matrix; C is the damping matrix; M is the diagonal mass matrix; u, $\dot{u}$, and $\ddot{u}$ are the displacement, velocity and acceleration of the structure respectively; and r is the applied load.

There are several options that determine the type of time-history analysis to be performed.

- Linear Vs. Nonlinear

- Modal Vs. Direct-integration : These are two different solution methods each with advantages and disadvantages. Under ideal circumstances, both methods should yield the same results for a given problem.

- Transient Vs. Periodic: Transient analysis considers the applied load as a one time event with a beginning and end. Periodic analysis considers the load to repeat indefinitely, with all transient response damped out.

Loading:
As per GSA guideline, Load = (DL + 0.25 LL)
As per DoD guideline, Load = (1.2DL + 0.5LL)
Apply lateral loads also for DoD loadings as defined earlier. This analysis is performed through following steps:

a. Build a computer model

b. Remove a column from the model

c. Apply the dynamic load combinations as per GSA and DoD guidelines. In SAP2000, there are two analysis options: direct integration and modal superposition. But it is found that the modal super position procedure runs much faster and hence analysis is performed using modal superposition method.

d. Perform time history analysis with zero initial conditions, a standard analysis procedure in SAP2000.

e. Evaluate the results based on demand-to-capacity ratio (DCR), where demand is taken as the peak value of response from the calculated time-history response.

Advantages of this analysis procedure include its accuracy, which derives from its ability to account for internal dynamic loading effects coupled with the effects of higher modes of vibration. The disadvantage of this methodology is its inability to account for material and geometric nonlinearity, which could be significant in complex structures where structure yield patterns cannot be easily identified. Linear dynamic analysis case has been defined in SAP2000 for GSA and DoD guidelines is shown in Fig. 5.10.

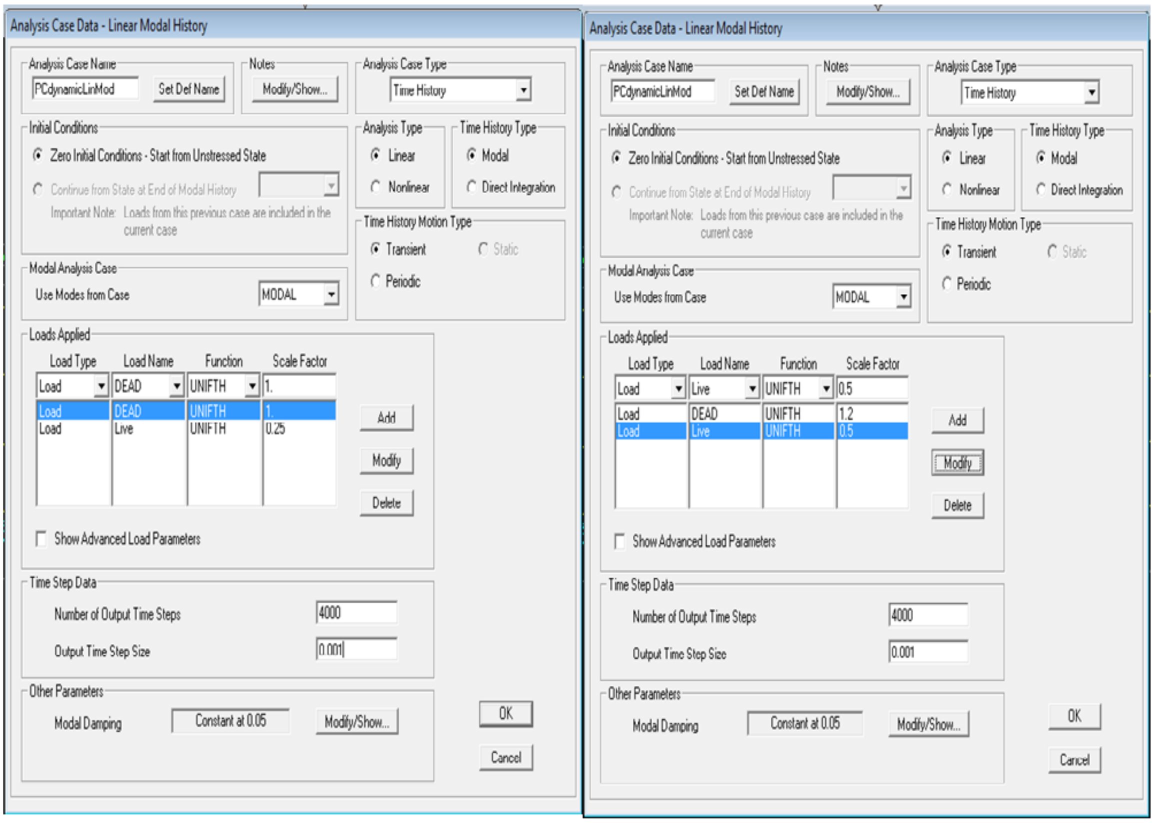


Figure 5.10: Linear dynamic analysis case definition by GSA and DoD guidelines

Linear dynamic analysis with staged construction and without staged construction: Initial conditions method can be used to perform linear dynamic analysis. This sets the initial conditions as the deflected shape of the undamaged structure under normal service loads. In most cases, however, these initial conditions are negligibly small, especially for linear dynamic analysis and can be neglected. Analysis procedure using initial condition is as follows:

- Build a computer model.
- Apply analysis loadings as per GSA and DoD guidelines.
- Run analysis with all columns.
- Find out column carrying capacity of column to be removed.
- Remove column and apply reaction as point load.

- Define analysis case as initial condition and apply loadings (Dead, Live and Point load).
- Define new analysis case and start the analysis after the end of initial analysis case.
- Remove the point load suddenly from the model.
- Perform linear dynamic analysis using time-history.

Fig. 5.11 shows the definition of initial analysis case.

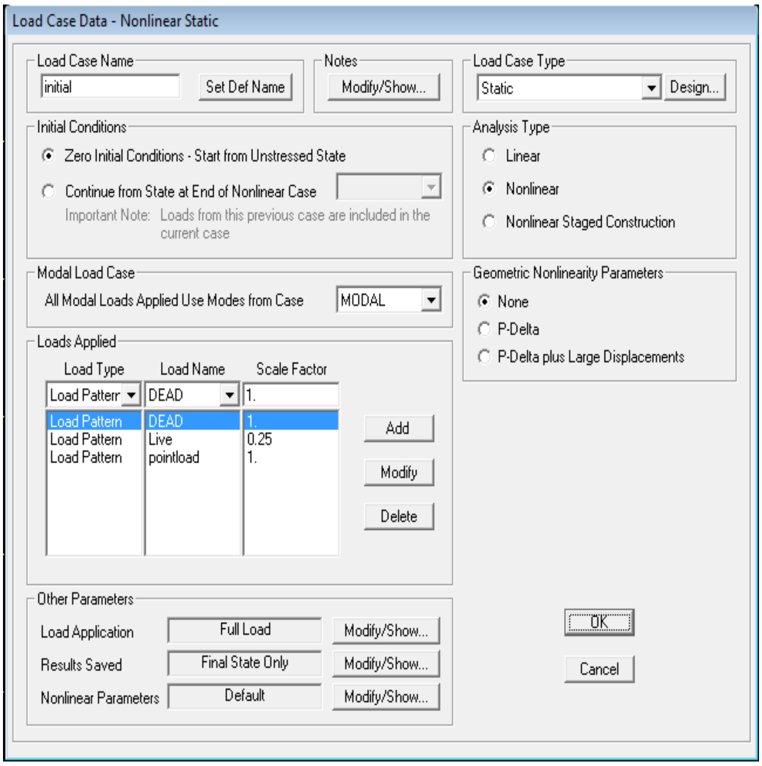


Figure 5.11: Definition of initial analysis case

Fig. 5.12 shows the time-history function definition. Fig. 5.13 shows the definition of linear dynamic analysis case definition.

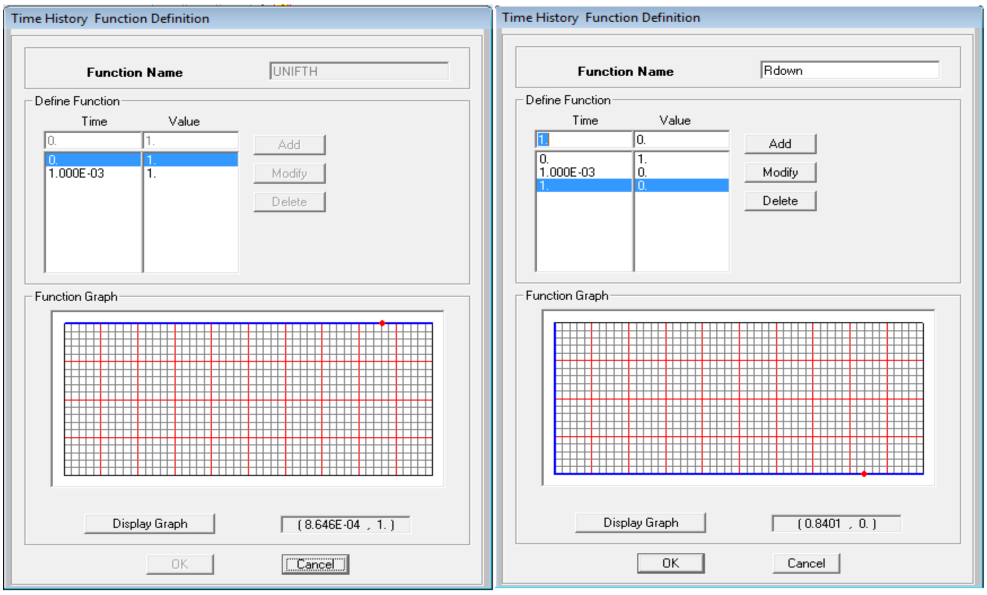


Figure 5.12: Time-history function definition for DL and LL

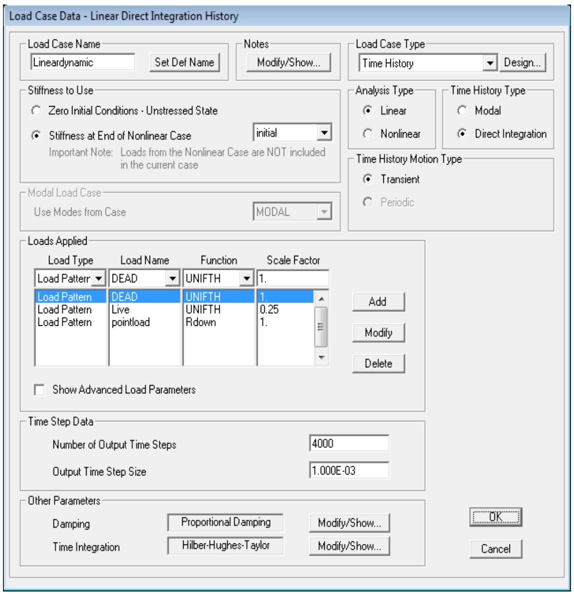


Figure 5.13: Definition of linear dynamic analysis case

Definition of staged construction for linear dynamic analysis is the same as per linear static analysis as explained earlier.

5.4.3 Calculation of DCR

DCR for beams and columns are found out for all the column removal cases at each floor. Permissible value of DCR for regular steel building is 2.0 in flexure. DCR is calculated at each storey for linear static and linear dynamic analysis. For beams, DCR is calculated at three points left, center and right side of the column removal position as shown in Fig. 5.14.

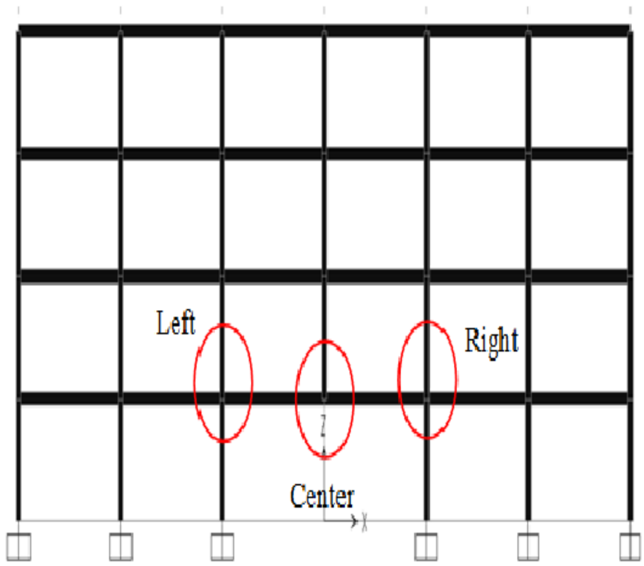


Figure 5.14: Calculation of DCR for case-1

DCR for flexure:

DCR for flexure in beam can be found by taking the ratio of the actual moment in the beam to its ultimate capacity as illustrated in equation below

$$DCR = \frac{M_{act}}{M_p} \tag{5.1}$$

Where, Mact= actual moment in the beam as obtained from analysis; Mp= ultimate moment capacity (i.e. plastic moment = fy Zp).

Moment diagram for case-1 is shown in Fig. 5.15 after the column removal. The ratio of moment to the flexural capacity of the beam (Zp fy) is known as DCR. Where Zp = Plastic section modulus; fy = yield strength. Thus DCR can be calculated at each storey.

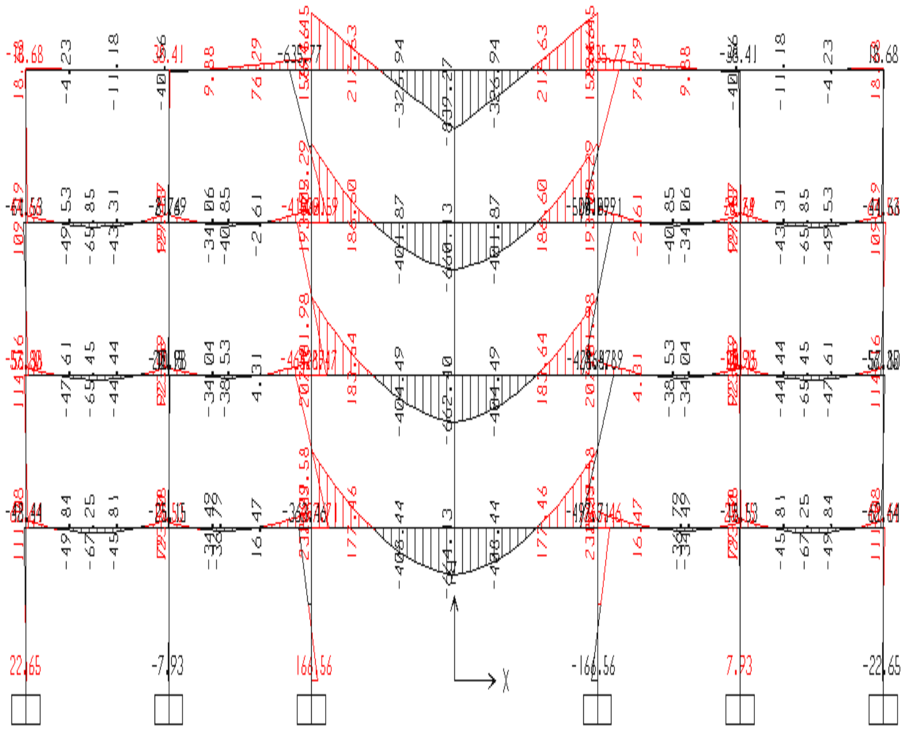


Figure 5.15: Moment diagram for case-1

Fig. 5.16, 5.17, 5.18, and 5.19 show the DCR for 4-storey and 9-storey steel buildings for column removal case-1, 2, 3 and 4 respectively as per GSA guidelines. The DCR are shown at critical locations.

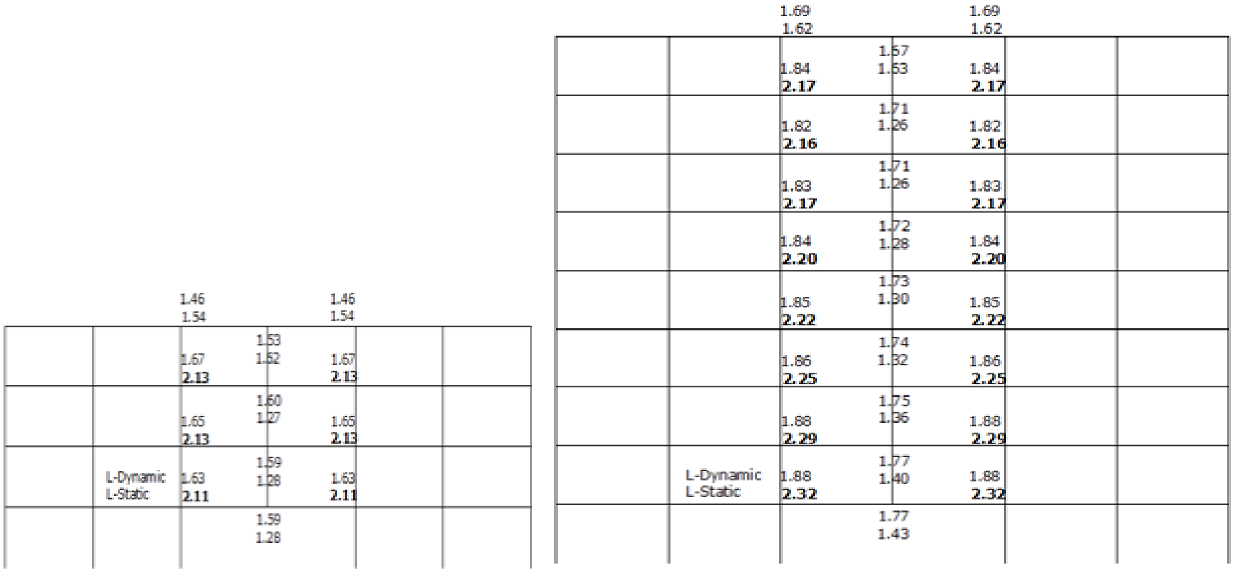


Figure 5.16: DCR for flexure for case-1 as per GSA guideline

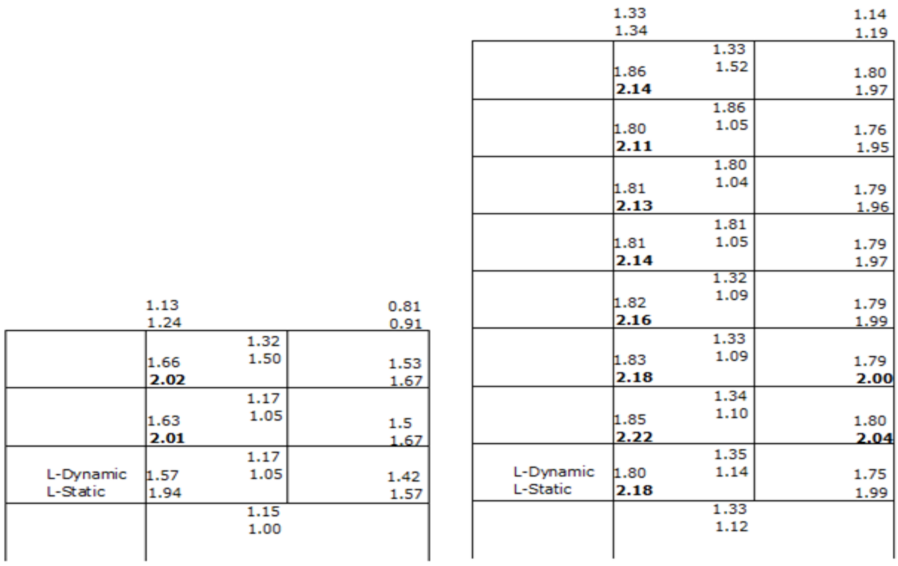


Figure 5.17: DCR for flexure for case-2 as per GSA guideline

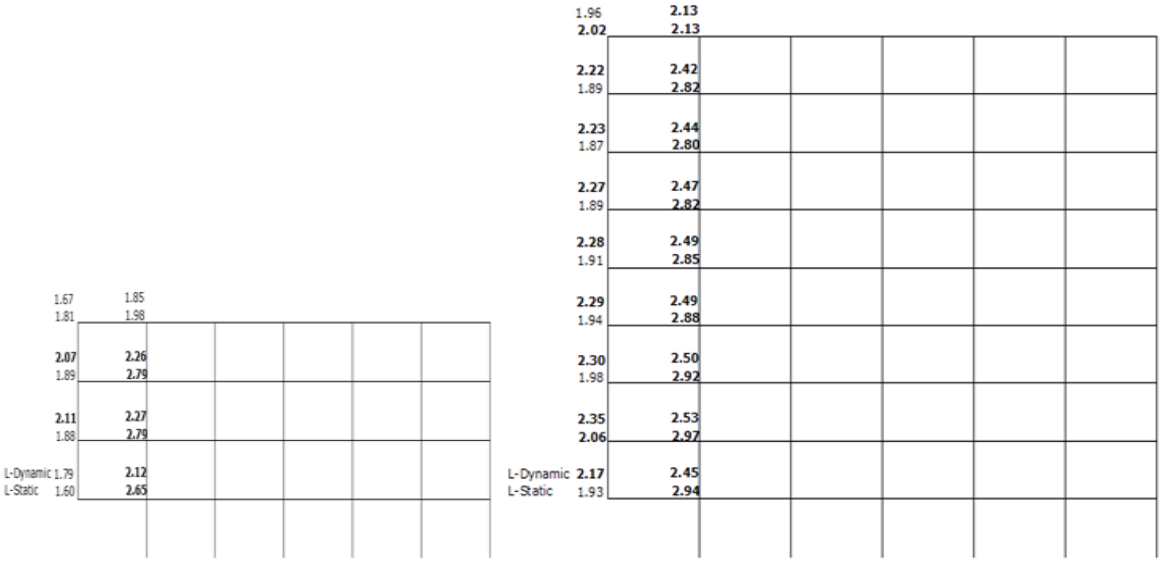


Figure 5.18: DCR for flexure for case-3 as per GSA guideline

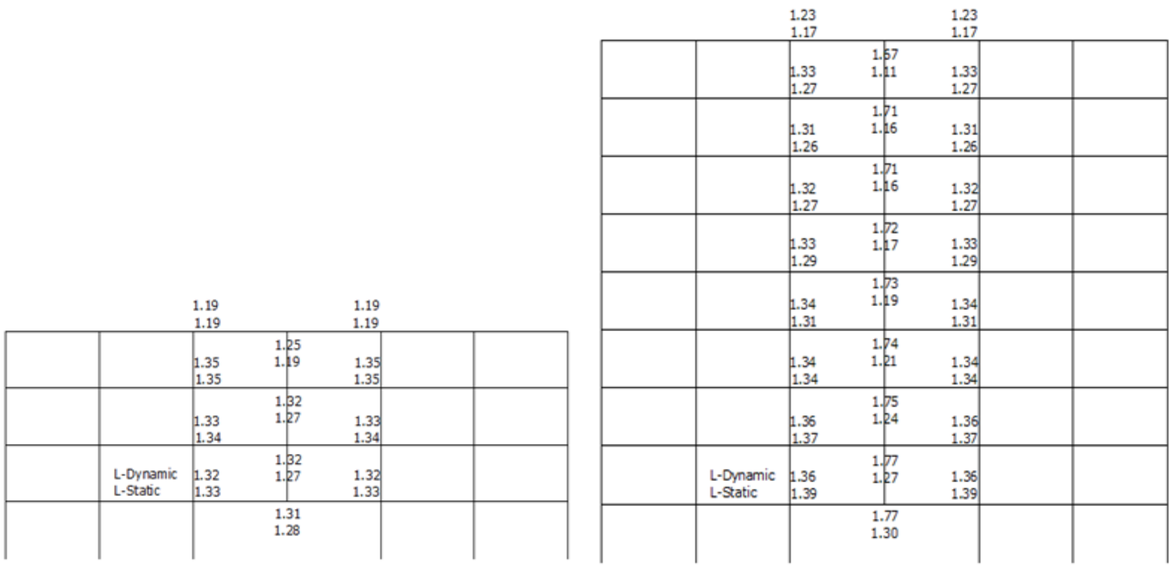


Figure 5.19: DCR for flexure for case-4 as per GSA guideline

Fig. 5.20 and 5.21 show the DCR for column removal case-1 as per DoD guideline for 4-storey and 9-storey buildings. Lateral loads are applied in ±X-direction.

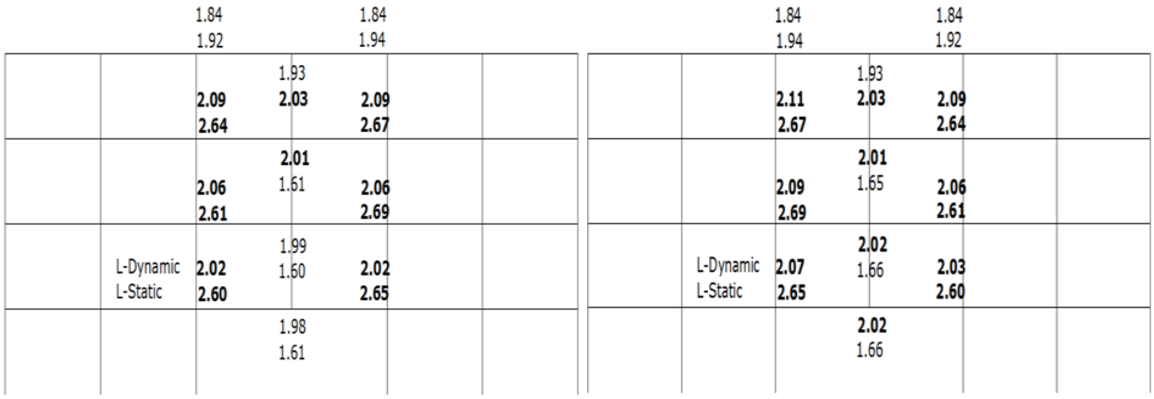


Figure 5.20: DCR for flexure for case-1 by DoD guideline for 4-storey (for ±X-direction)

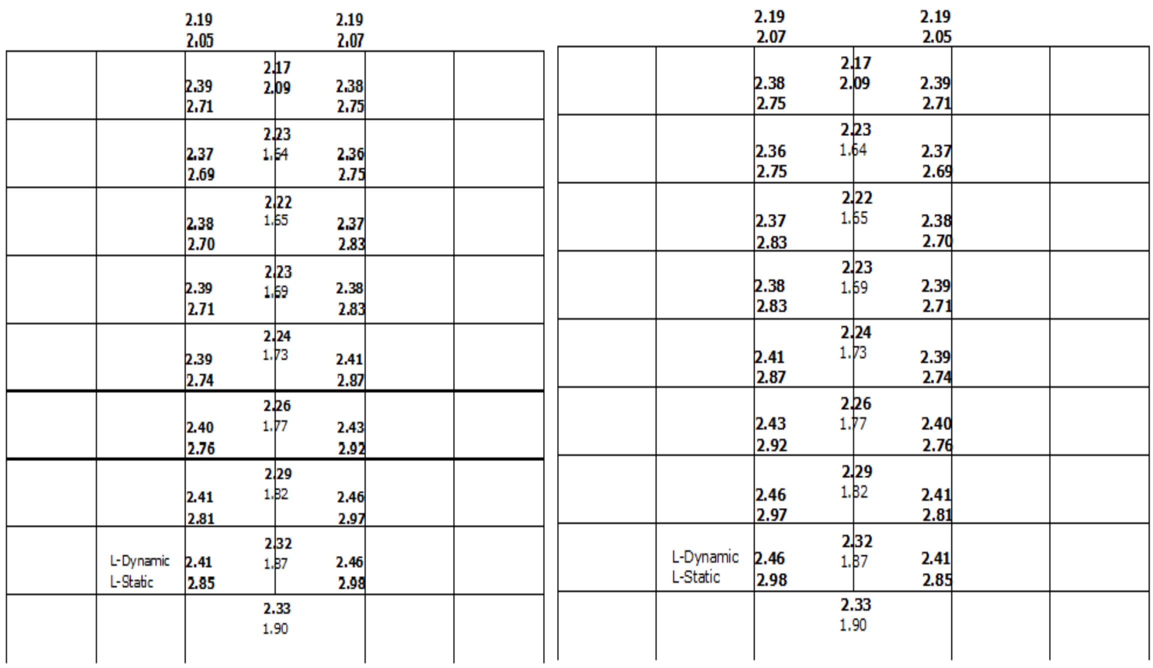


Figure 5.21: DCR for flexure for case-1 by DoD guideline for 9-storey (for ±X-direction)

Fig. 5.22 and 5.23 show the DCR for column removal case-1 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

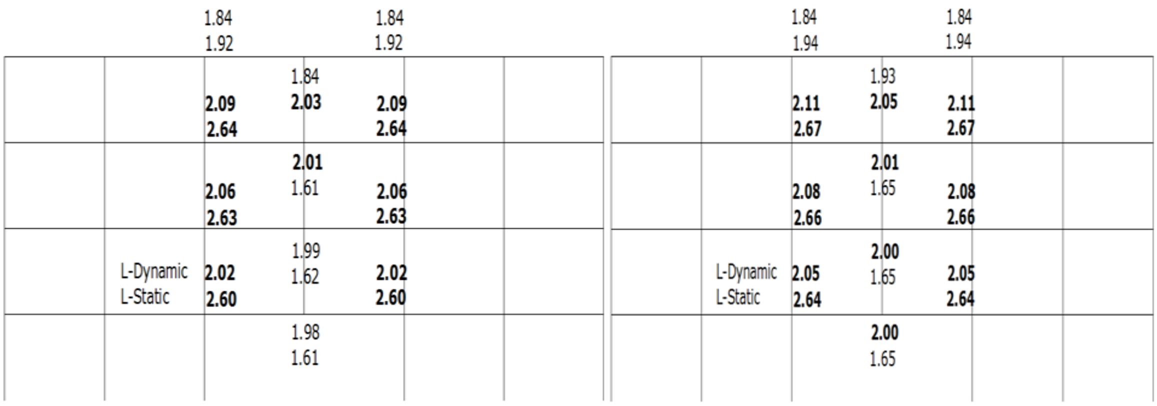


Figure 5.22: DCR for flexure for case-1 by DoD guideline for 4-storey (for ±Y direction)

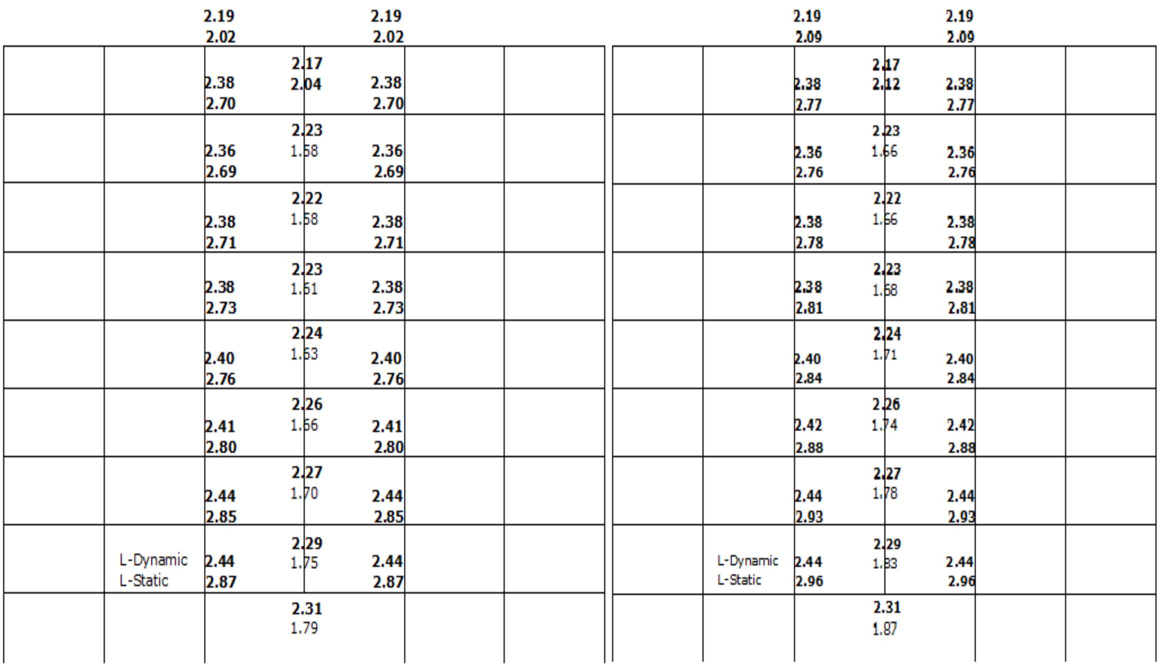


Figure 5.23: DCR for flexure for case-1 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.24 and 5.25 show the DCR for column removal case-2 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

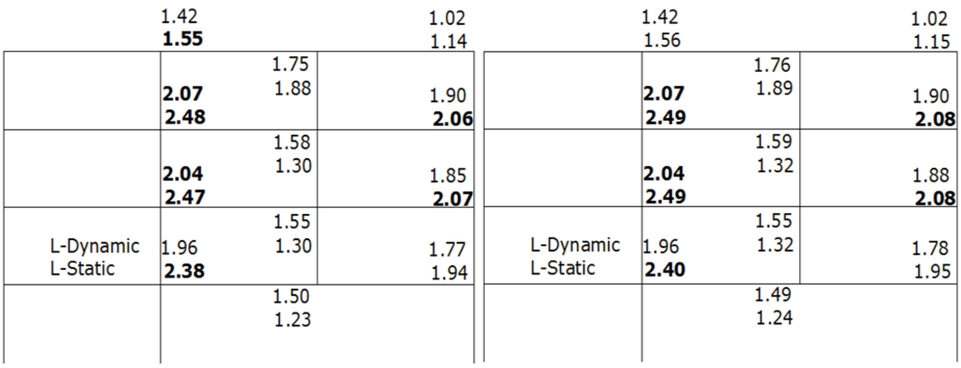


Figure 5.24: DCR for flexure for case-2 by DoD guideline for 4-storey (for ±X direction)

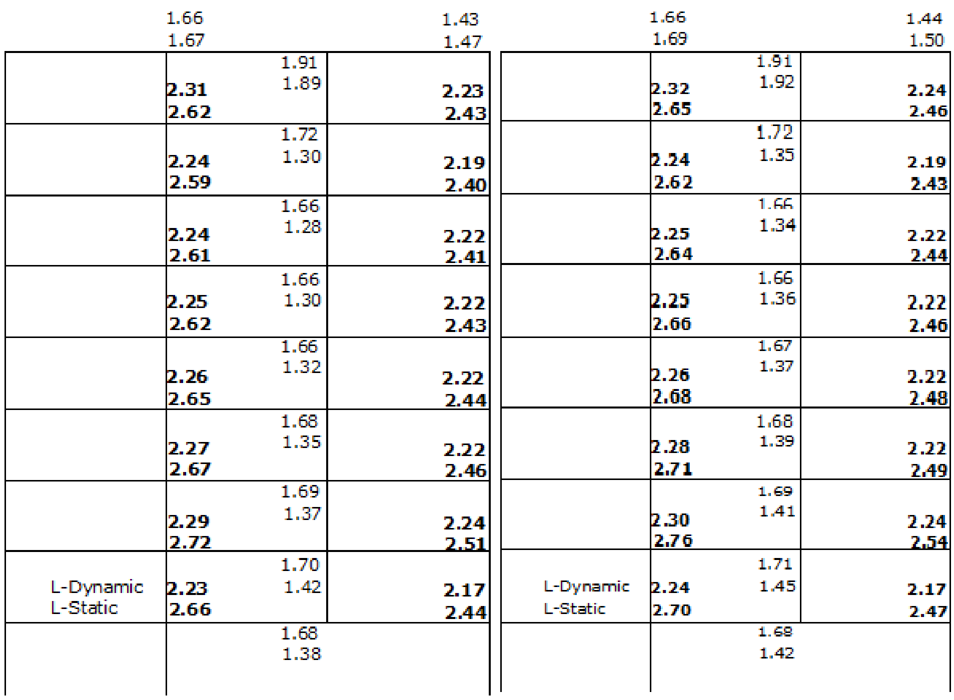


Figure 5.25: DCR for flexure for case-2 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.26 and 5.27 show the DCR for column removal case-2 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

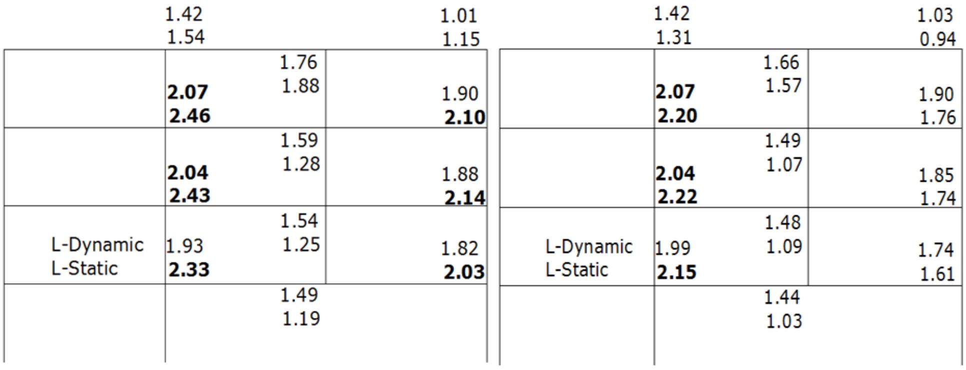


Figure 5.26: DCR for flexure for case-2 by DoD guideline for 4-storey (for ±Y direction)

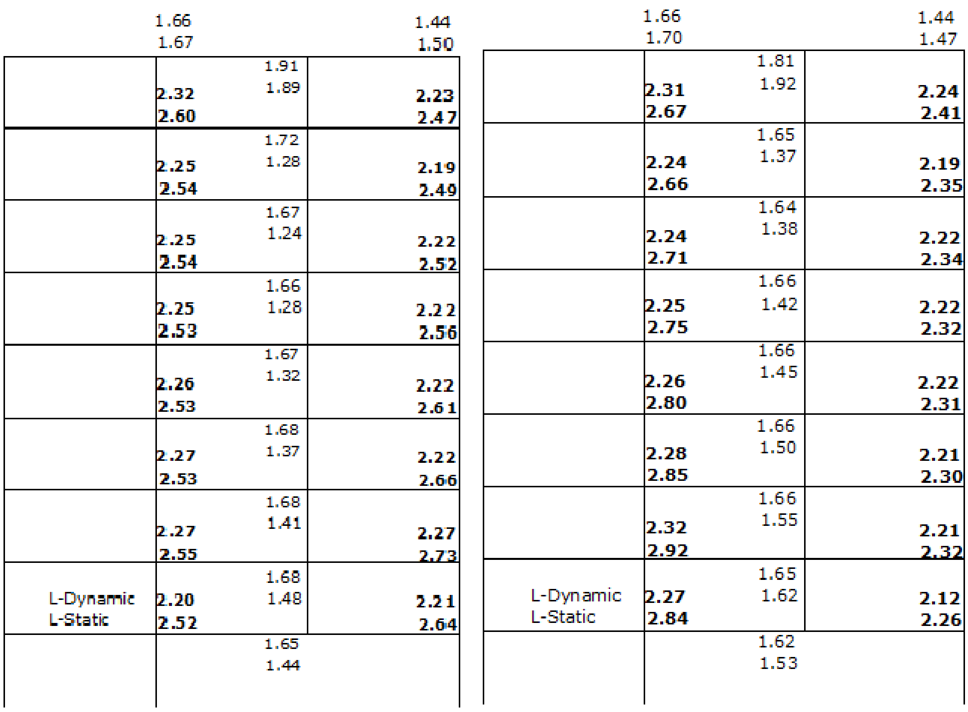


Figure 5.27: DCR for flexure for case-2 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.28 and 5.29 show the DCR for column removal case-3 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

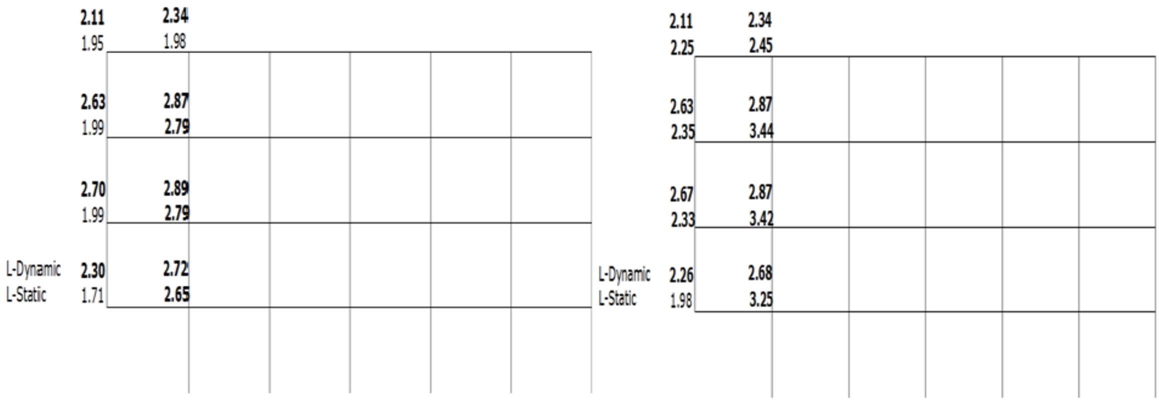


Figure 5.28: DCR for flexure for case-3 by DoD guideline for 4-storey (for ±X direction)

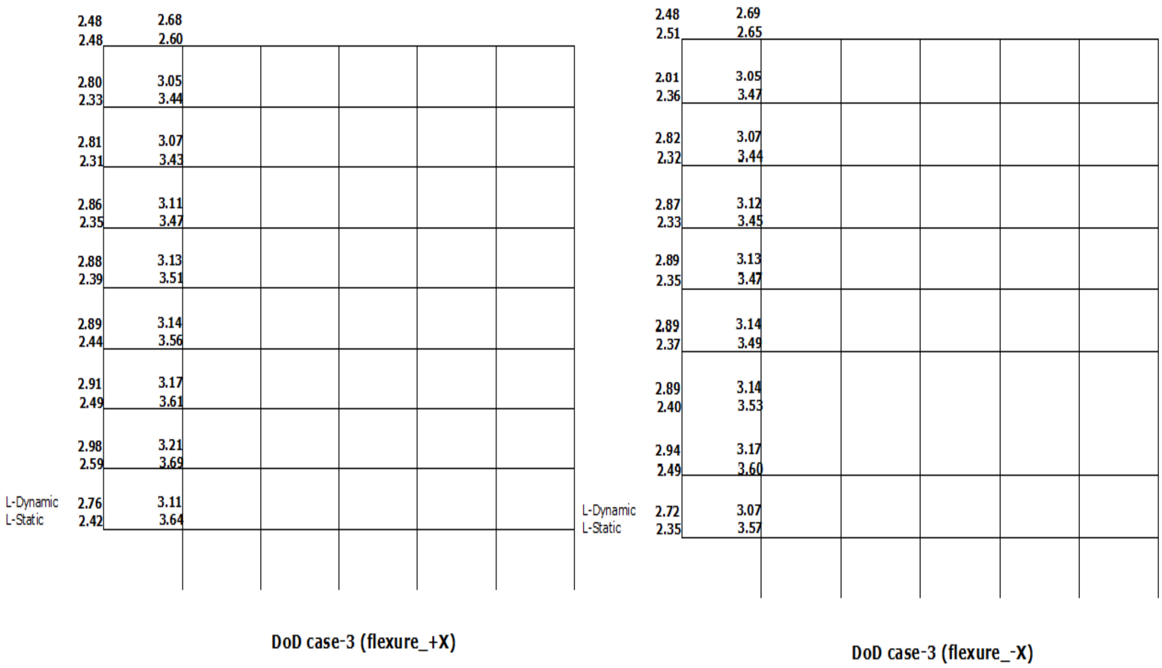


Figure 5.29: DCR for flexure for case-3 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.30 and 5.31 show the DCR for column removal case-3 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

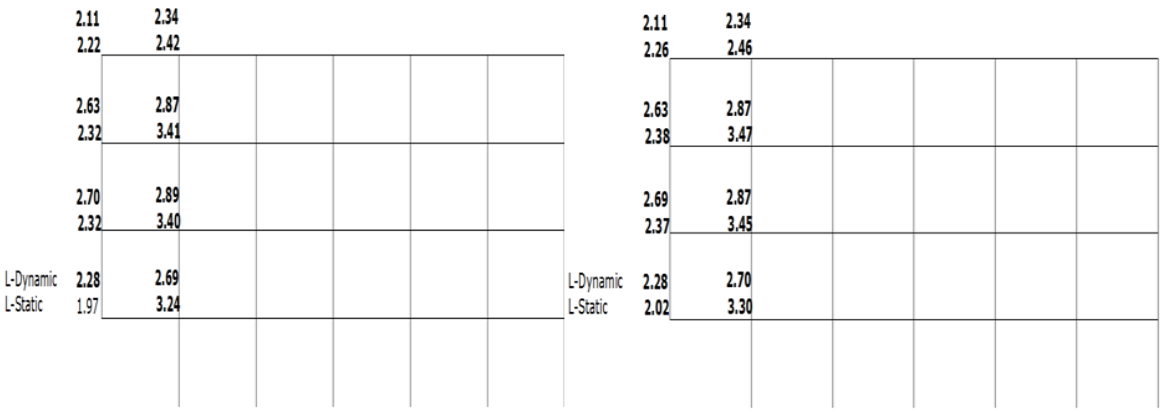


Figure 5.30: DCR for flexure for case-3 by DoD guideline for 4-storey (for ±Y direction)

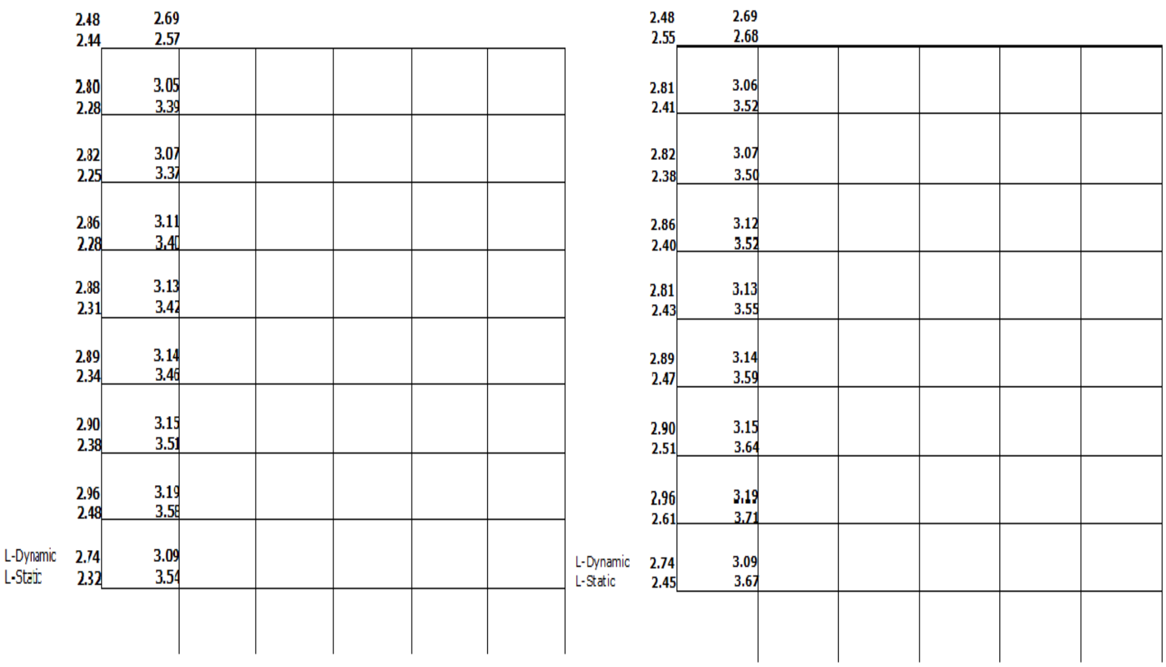


Figure 5.31: DCR for flexure for case-3 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.32 and 5.33 show the DCR for column removal case-4 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

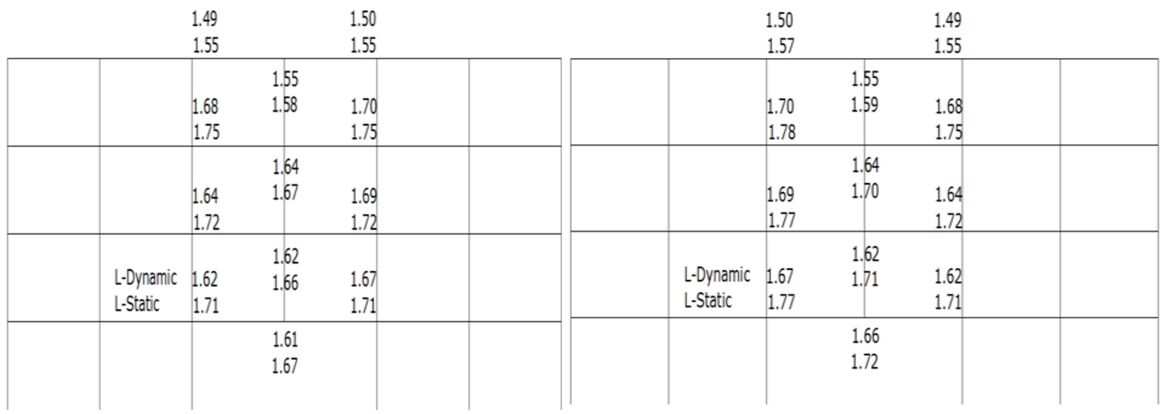


Figure 5.32: DCR for flexure for case-4 by DoD guideline for 4-storey (for ±X direction)

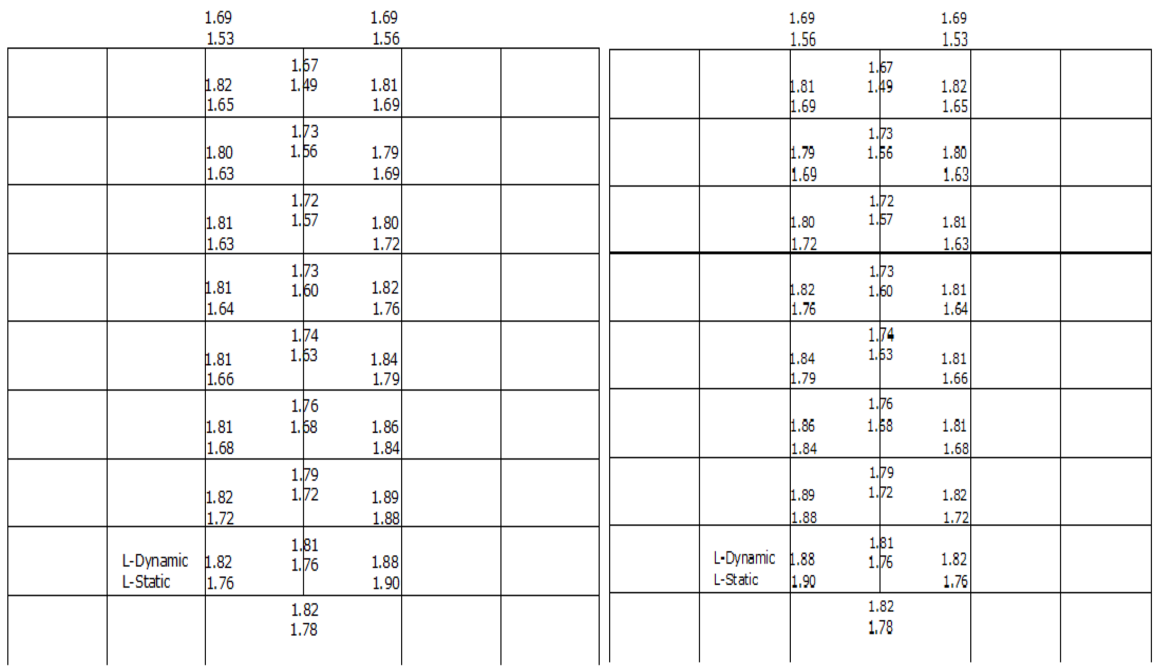


Figure 5.33: DCR for flexure for case-4 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.34 and 5.35 show the DCR for column removal case-4 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

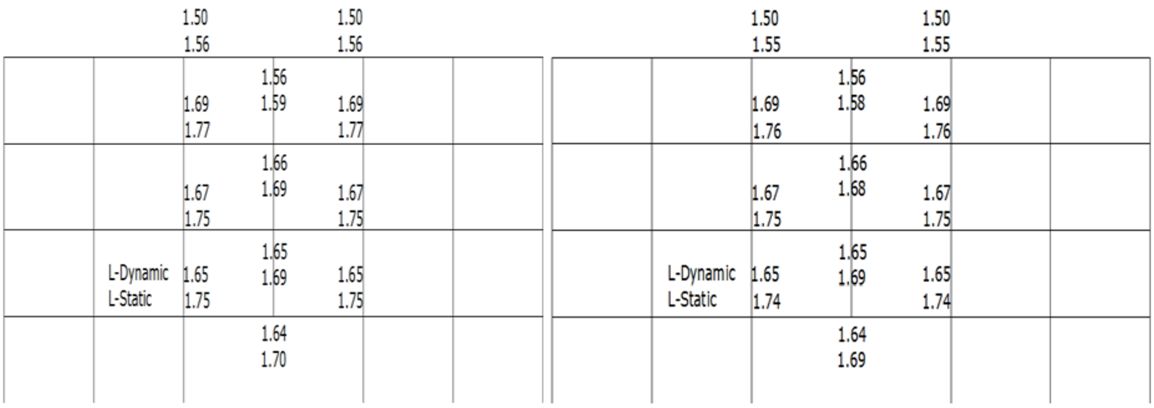


Figure 5.34: DCR for flexure for case-4 by DoD guideline for 4-storey (for ±Y direction)

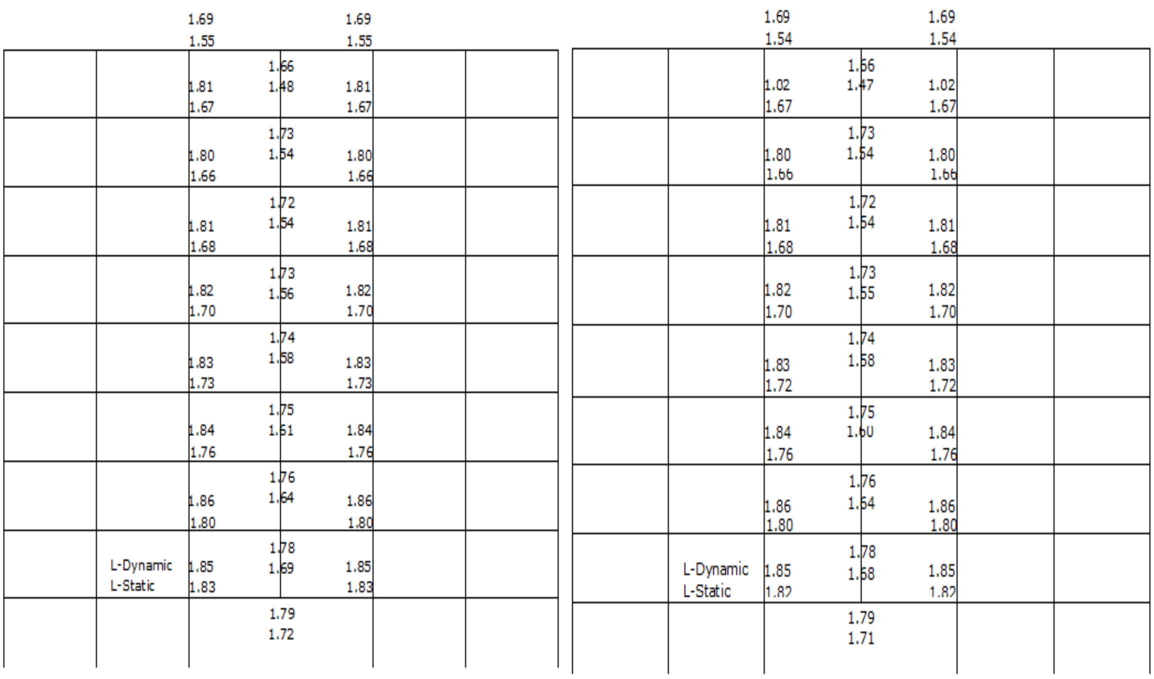


Figure 5.35: DCR for flexure for case-4 by DoD guideline for 9-storey (for ±Y direction)

DCR for shear:

DCR for shear in beam can be found by taking the ratio of the actual shear force in the beam to its ultimate capacity (i.e. plastic capacity) (0.55 Aw fy). Shear force diagram for case-1 is shown in Fig. 5.36 after the column removal. When shear force is divided with the shear capacity of the beam will give DCR. DCR is calculated same as flexure for linear static and linear dynamic analysis.

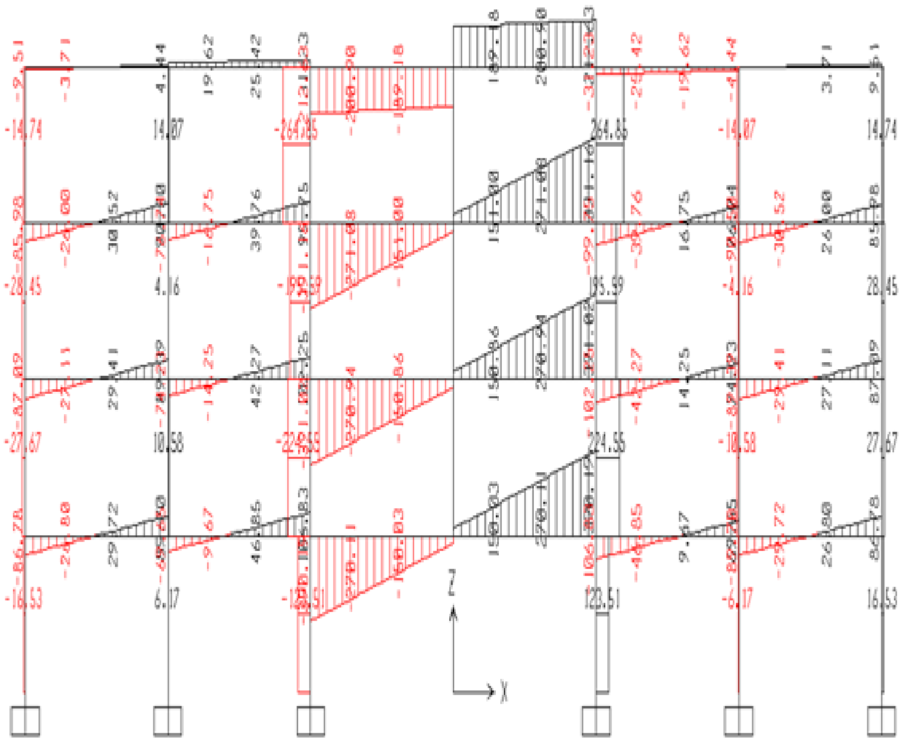


Figure 5.36: Shear force diagram for case-1

Fig. 5.37, 5.38, 5.39, and 5.40 show the DCR for 4-storey and 9-storey steel buildings for column removal case-1, 2, 3 and 4 respectively as per GSA guidelines.

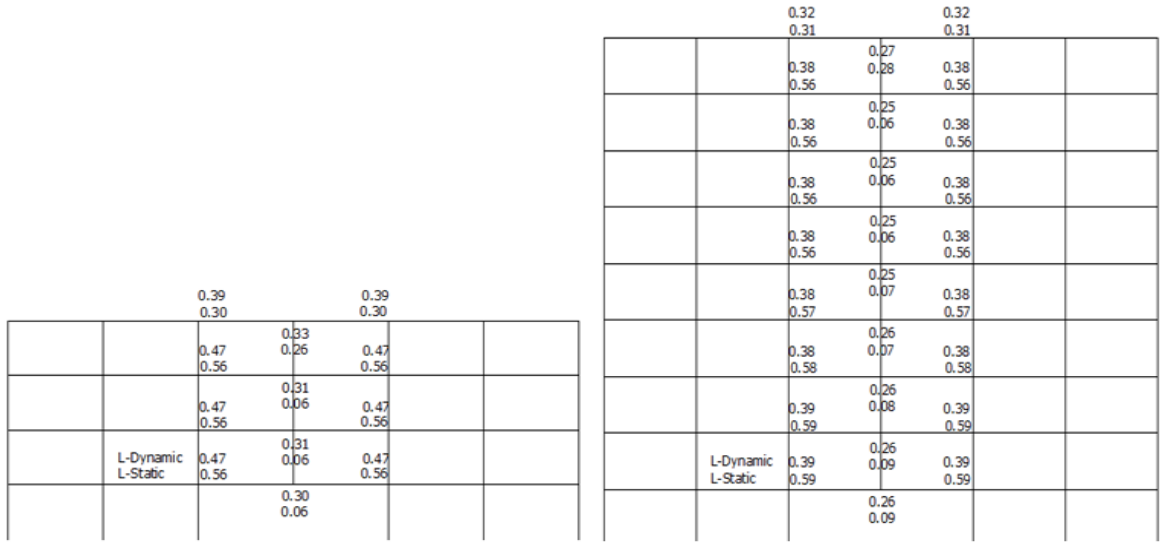


Figure 5.37: DCR for shear for case-1 by GSA guideline

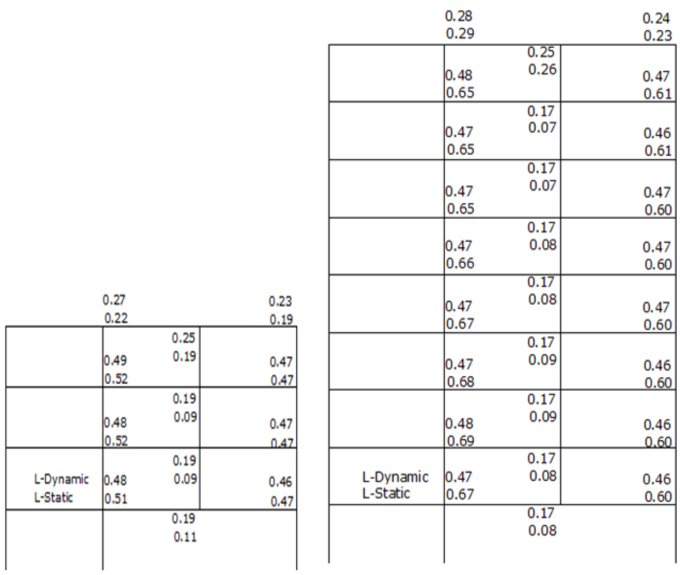


Figure 5.38: DCR for shear for case-2 by GSA guideline

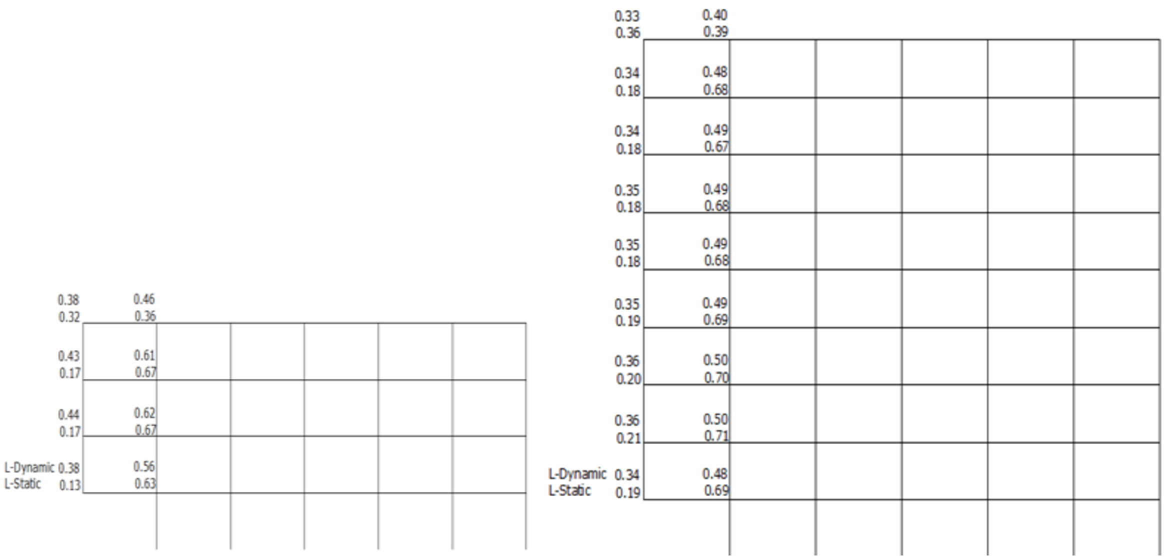


Figure 5.39: DCR for shear for case-3 by GSA guideline

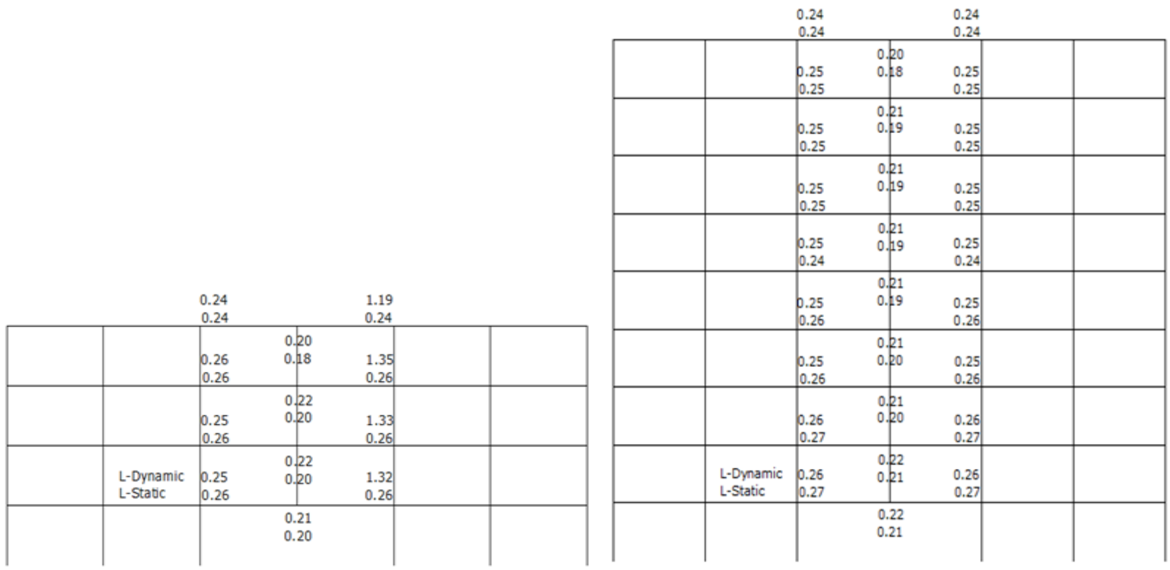


Figure 5.40: DCR for shear for case-4 by GSA guideline

Fig. 5.41 and 5.42 show the DCR for column removal case-1 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

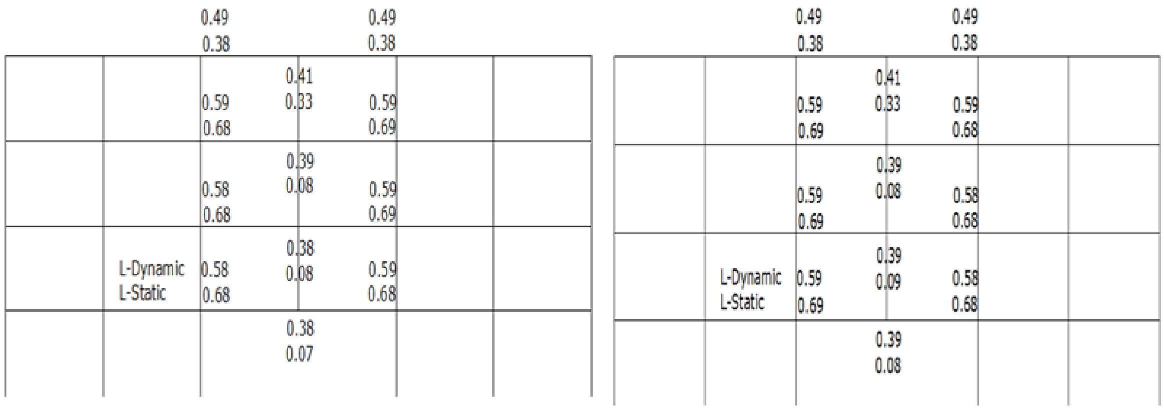


Figure 5.41: DCR for shear for case-1 by DoD guideline for 4-storey (for ±X direction)

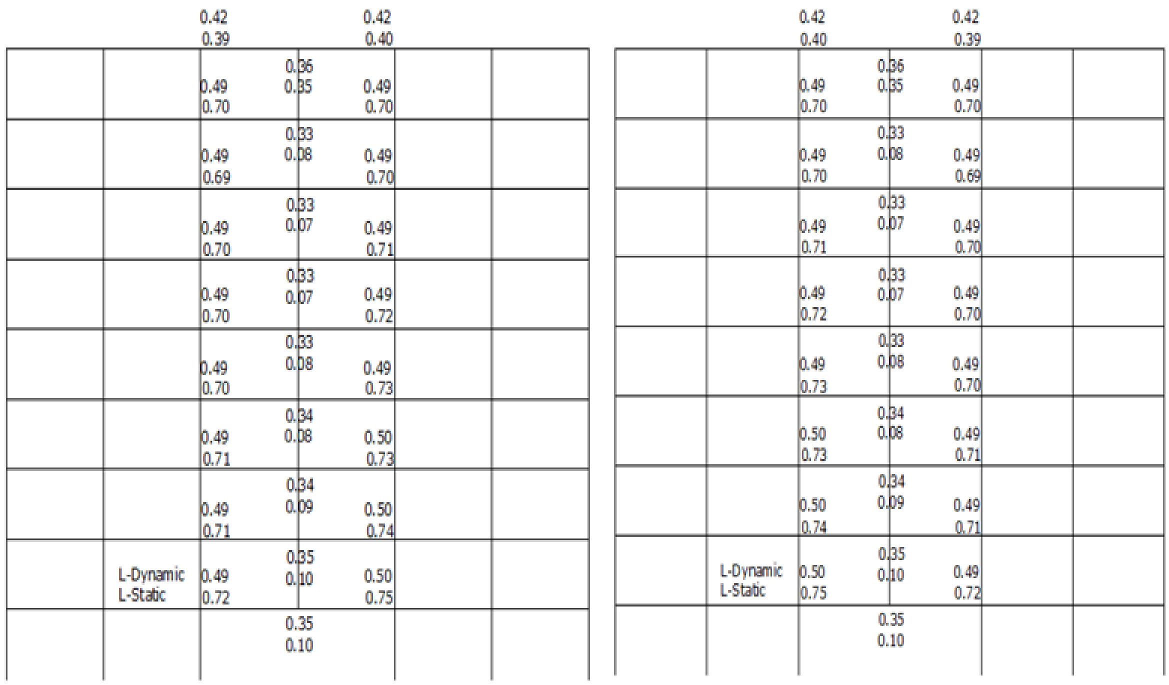


Figure 5.42: DCR for shear for case-1 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.43 and 5.44 show the DCR for column removal case-1 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

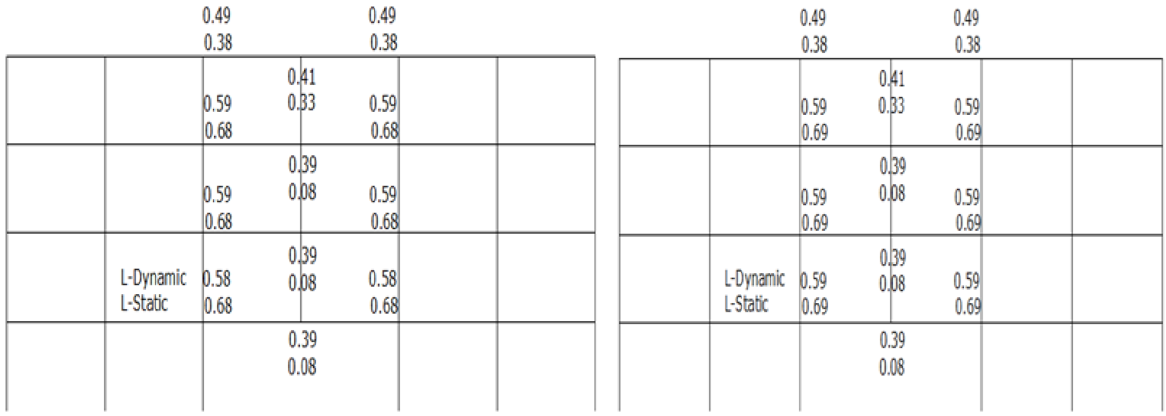


Figure 5.43: DCR for shear for case-1 by DoD guideline for 4-storey (for ±Y direction)

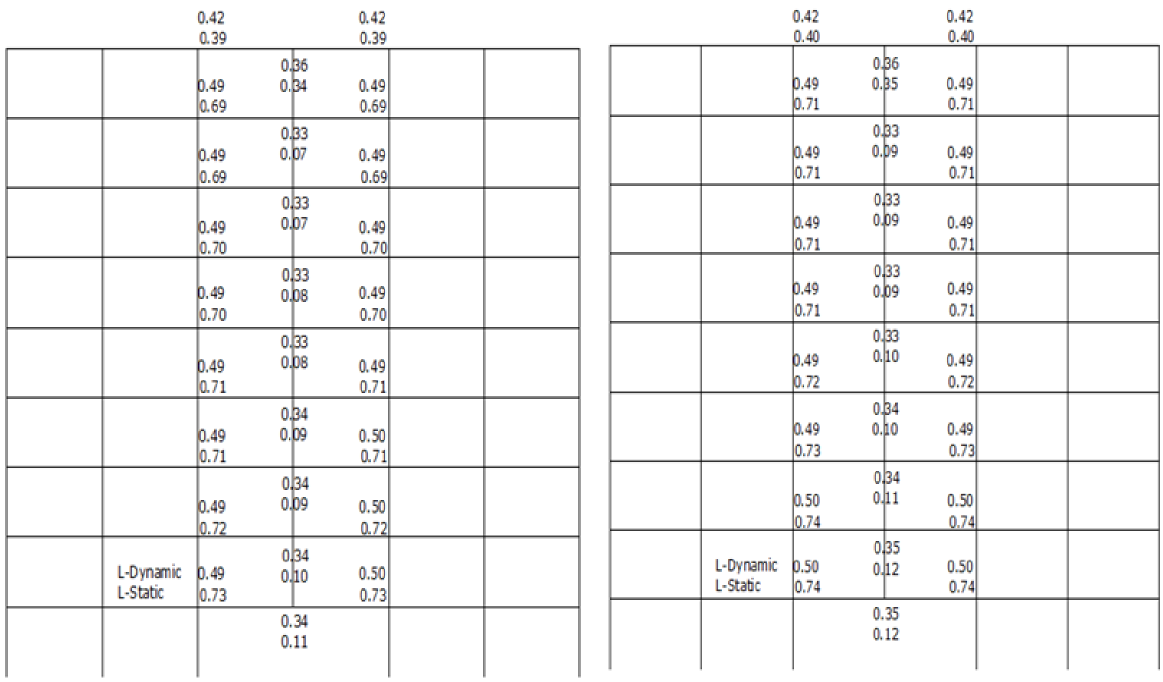


Figure 5.44: DCR for shear for case-1 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.45 and 5.46 show the DCR for column removal case-2 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

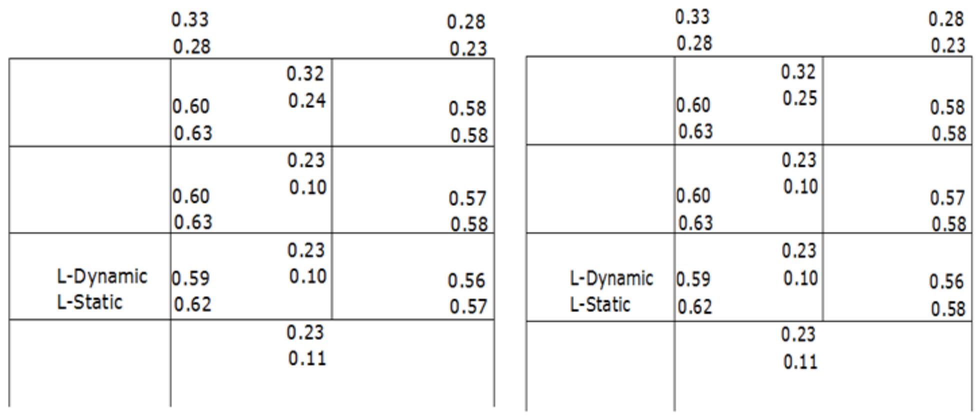


Figure 5.45: DCR for shear for case-2 by DoD guideline for 4-storey (for ±X direction)

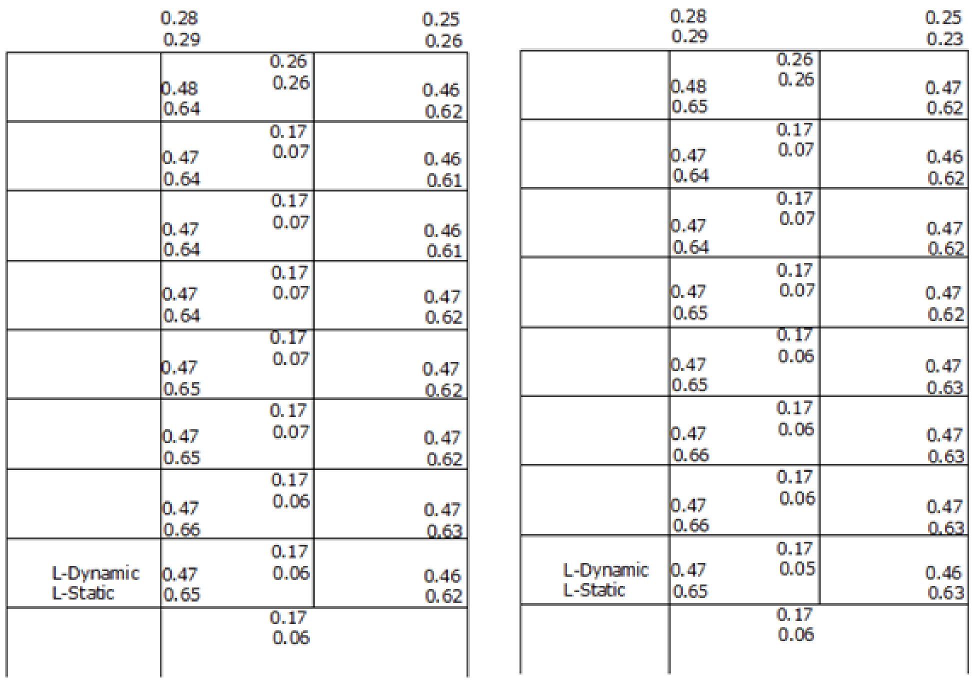


Figure 5.46: DCR for shear for case-2 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.47 and 5.48 show the DCR for column removal case-2 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

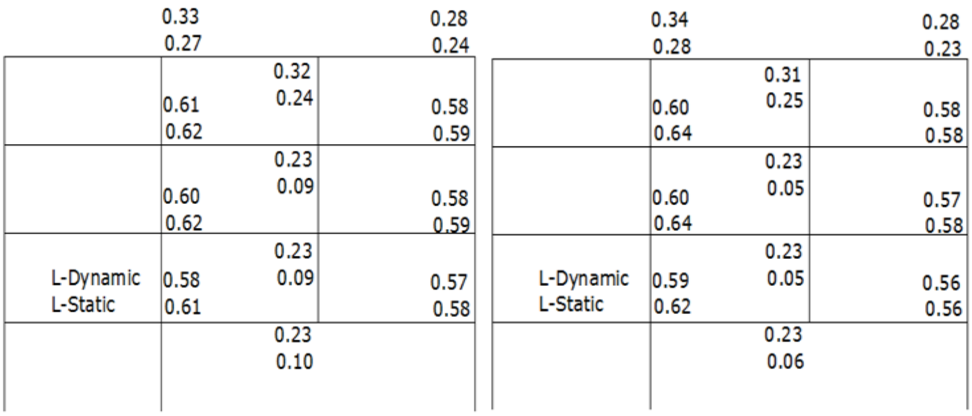


Figure 5.47: DCR for shear for case-2 by DoD guideline for 4-storey (for ±Y direction)

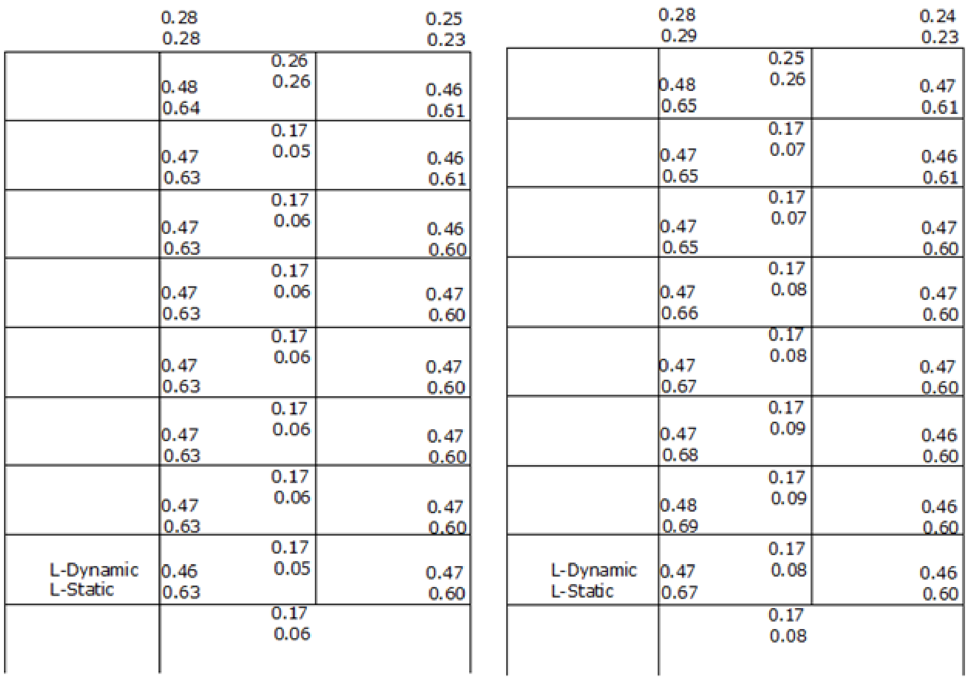


Figure 5.48: DCR for shear for case-2 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.49 and 5.50 show the DCR for column removal case-3 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

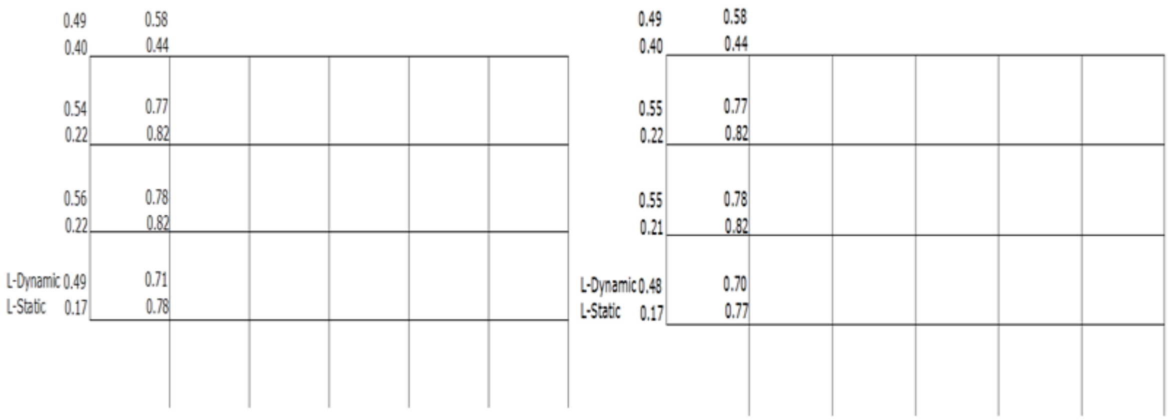


Figure 5.49: DCR for shear for case-3 by DoD guideline for 4-storey (for ±X direction)

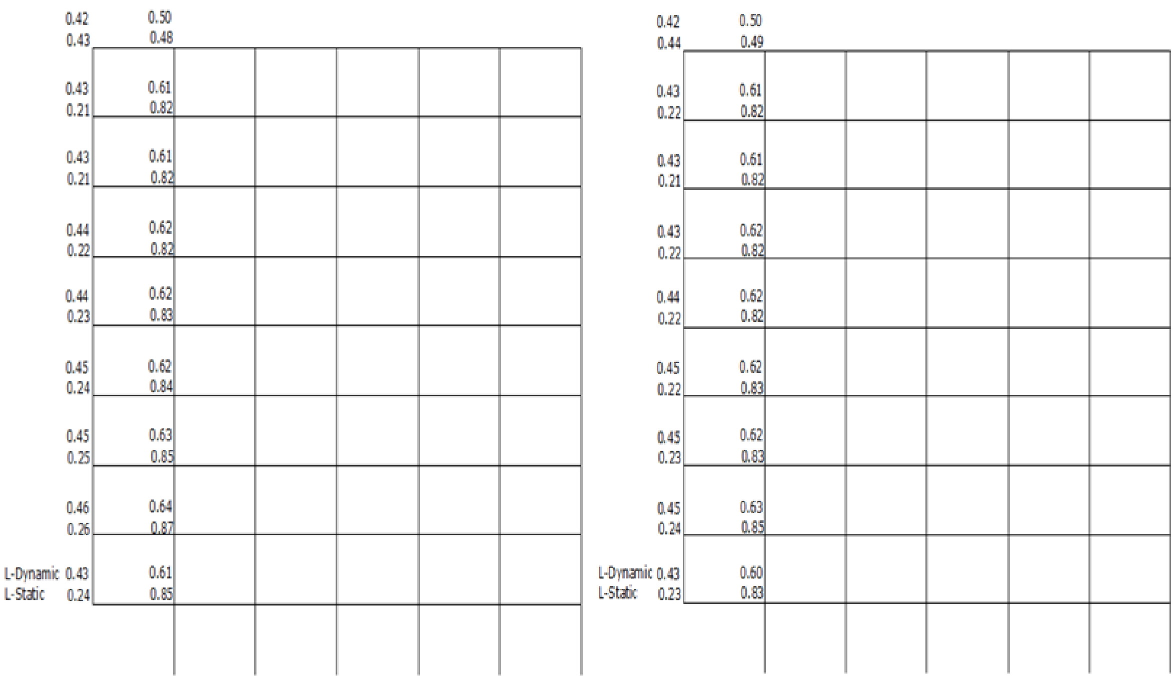


Figure 5.50: DCR for shear for case-3 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.51 and 5.52 show the DCR for column removal case-3 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

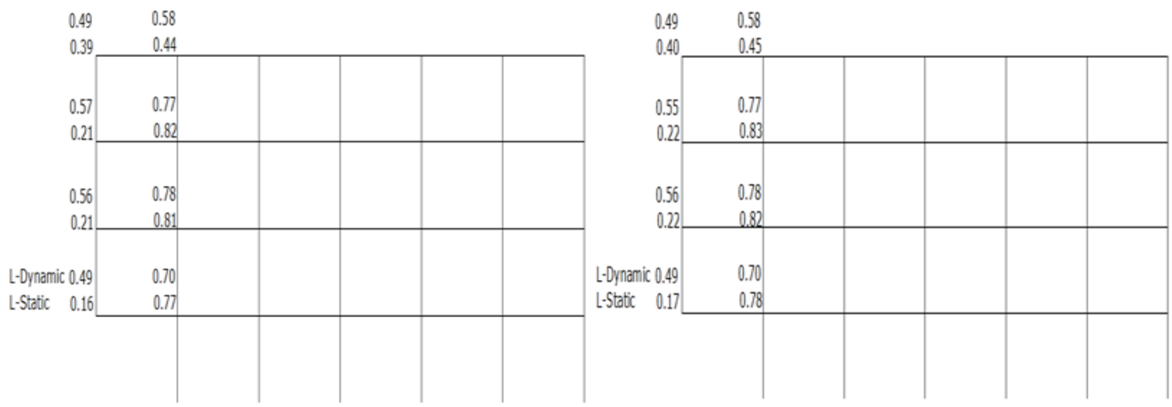


Figure 5.51: DCR for shear for case-3 by DoD guideline for 4-storey (for ±Y direction)

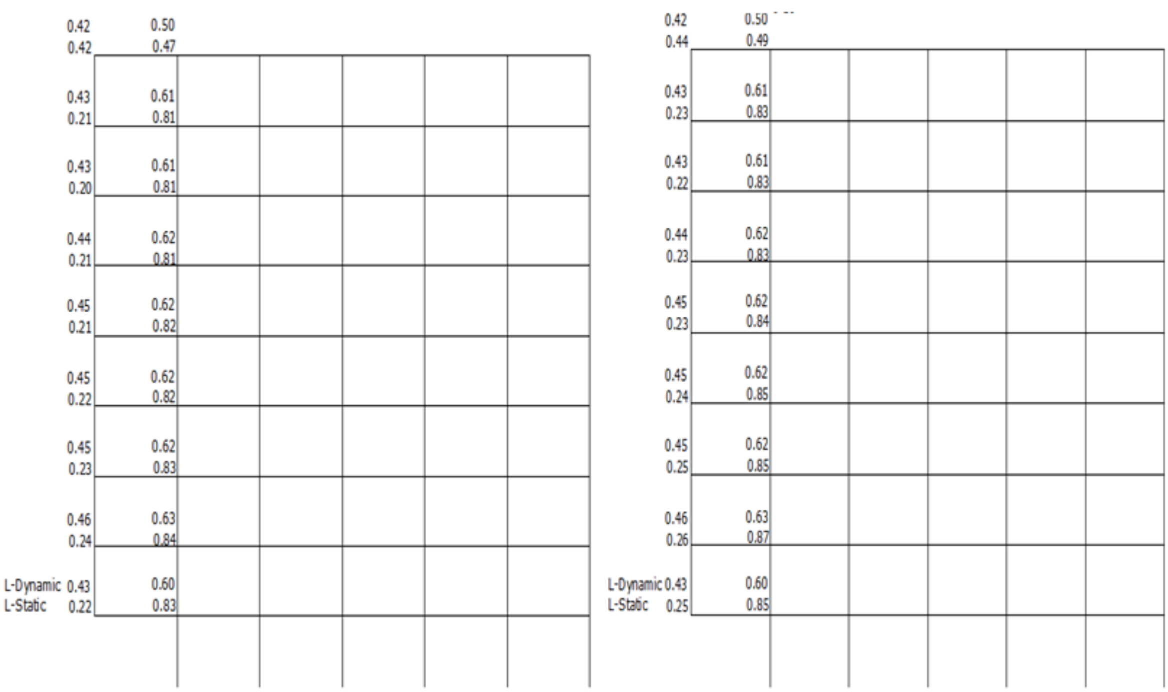


Figure 5.52: DCR for shear for case-3 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.53 and 5.54 show the DCR for column removal case-4 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

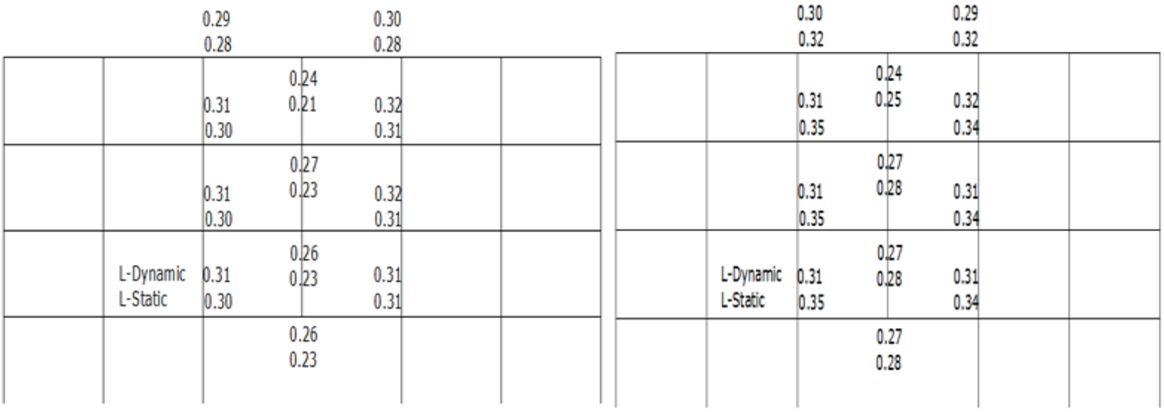


Figure 5.53: DCR for shear for case-4 by DoD guideline for 4-storey (for ±X direction)

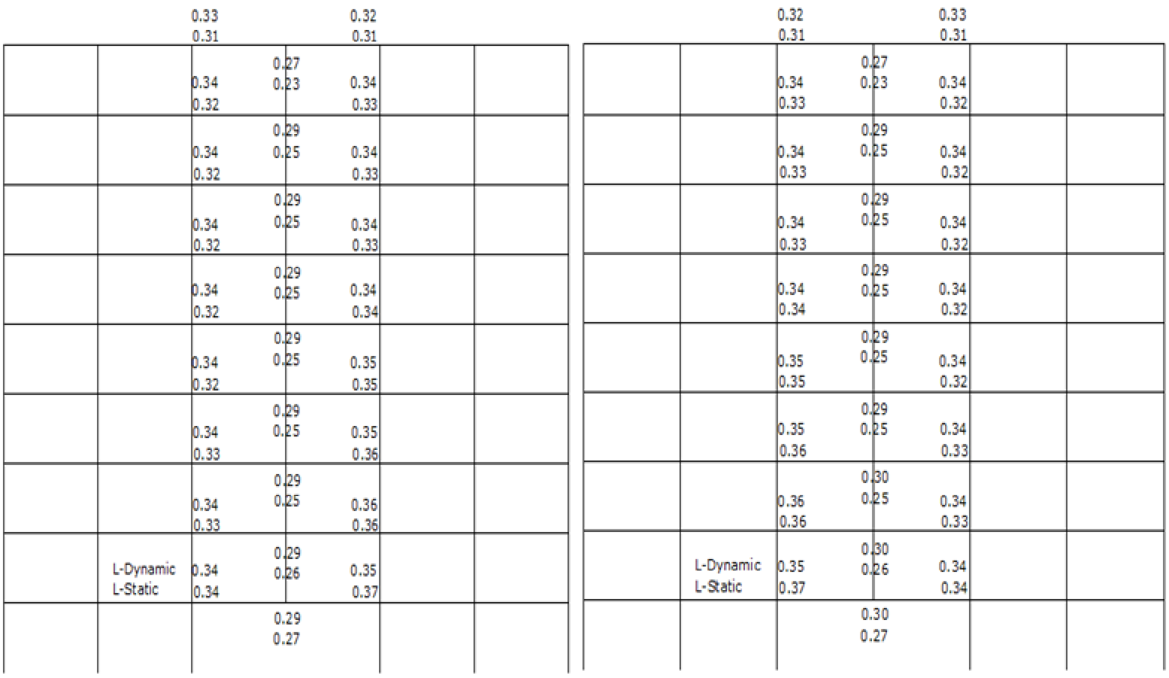


Figure 5.54: DCR for shear for case-4 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.55 and 5.56 show the DCR for column removal case-4 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

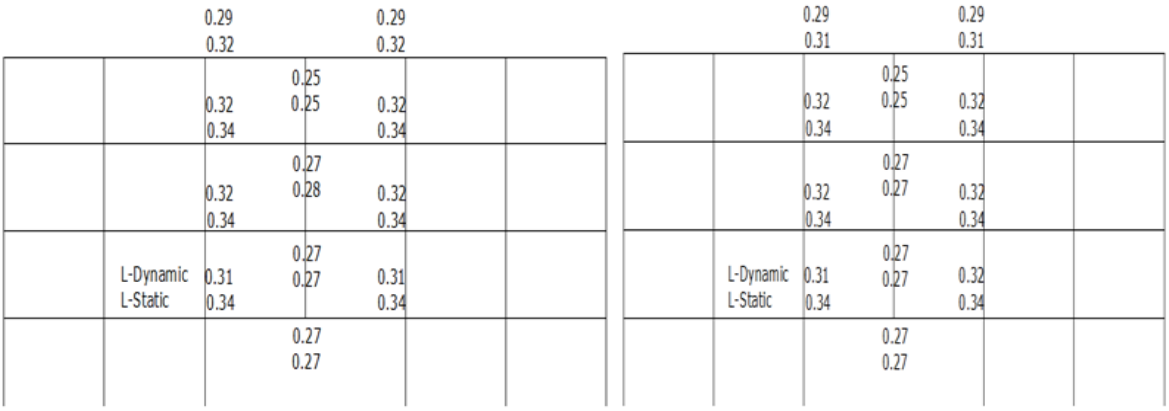


Figure 5.55: DCR for shear for case-4 by DoD guideline for 4-storey (for ±Y direction)

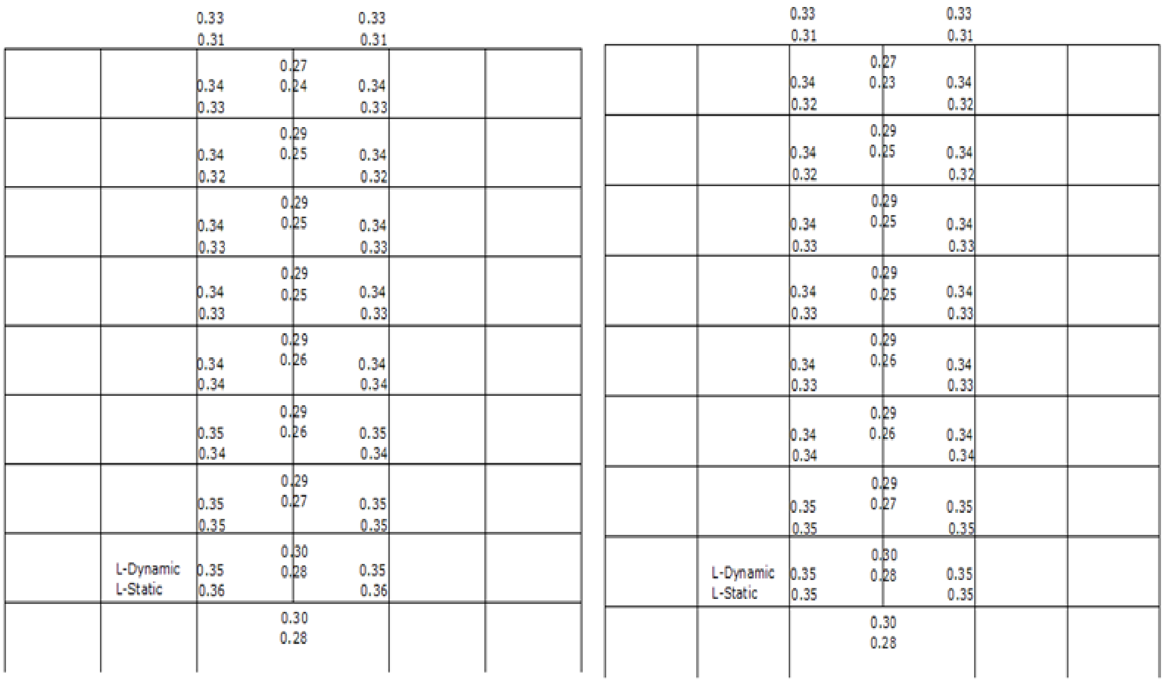


Figure 5.56: DCR for shear for case-4 by DoD guideline for 9-storey (for ±Y direction)

DCR for column:

Due to removal of one column redistribution of forces takes place in the structure, so forces in the column i.e. axial force, moment about major axis and moment about minor axis, change and can affect the adequacy of the existing columns.Demand capacity ratios for columns are calculated as per following equation. If it exceeds unity column can be considered as failed.

$$\frac{P}{P_y} + \frac{M_{pc}}{1.18M_p} \leq 1 \tag{5.2}$$

Where,

P is an axial force, compressive or tensile in a member as obtained from analysis results;

Py is yield strength of axially loaded section = As fy;

As is effective cross-section area of the member;

Mpc is maximum moment acting in a member as obtained from analysis results;

Mp is plastic moment capacity of the section.

For each column removal case DCR is calculated for highly stressed nearby columns. DCR for columns C3 in case-1, C16 in case-2, C2 in case-3 and C10 in case-4 are found out for linear static and linear dynamic analysis using GSA and DoD guidelines. Fig. 5.57, 5.58, 5.59, and 5.60 show the DCR for 4-storey and 9-storey steel buildings for column removal case-1, 2, 3 and 4 respectively as per GSA guidelines.

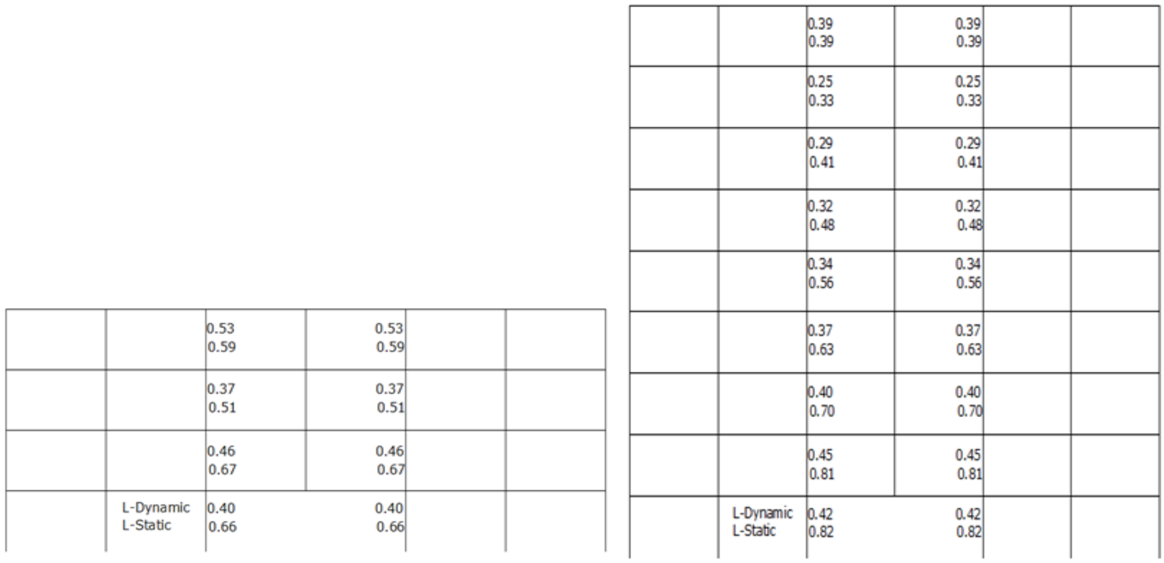


Figure 5.57: DCR for column for case-1 by GSA guideline

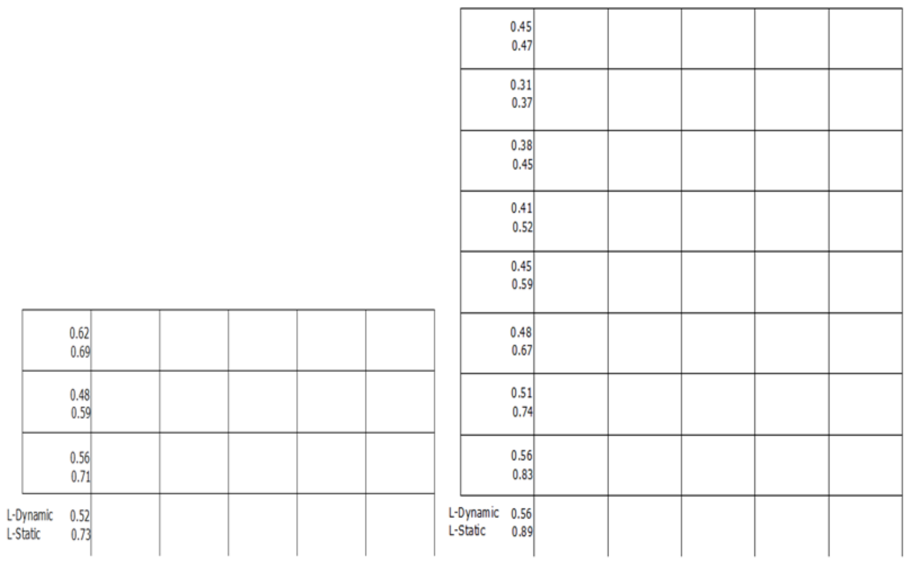


Figure 5.58: DCR for column for case-2 by GSA guideline

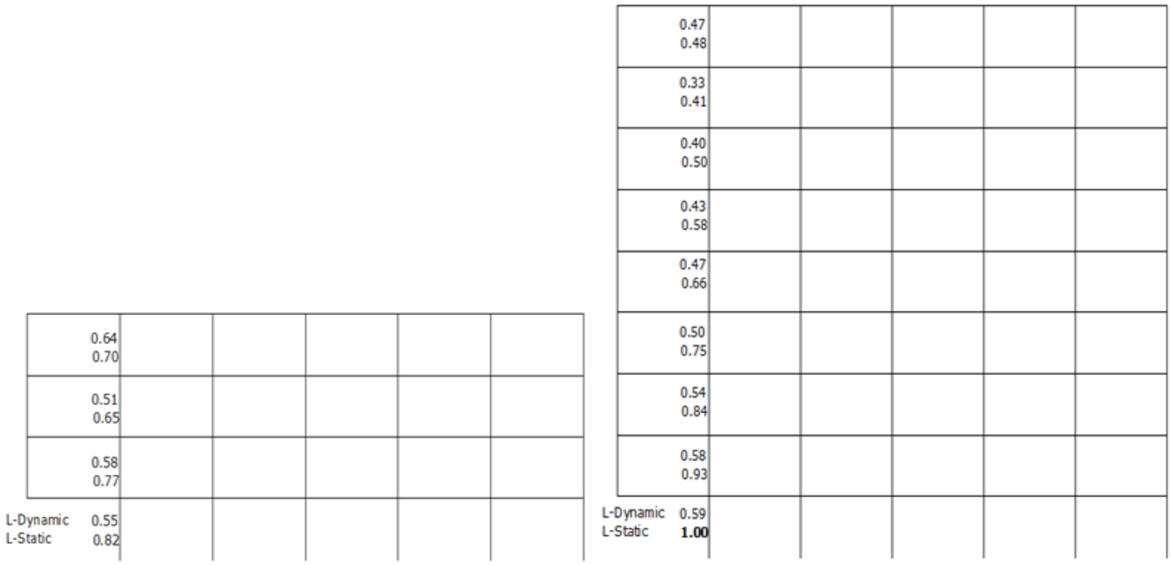


Figure 5.59: DCR for column for case-3 by GSA guideline

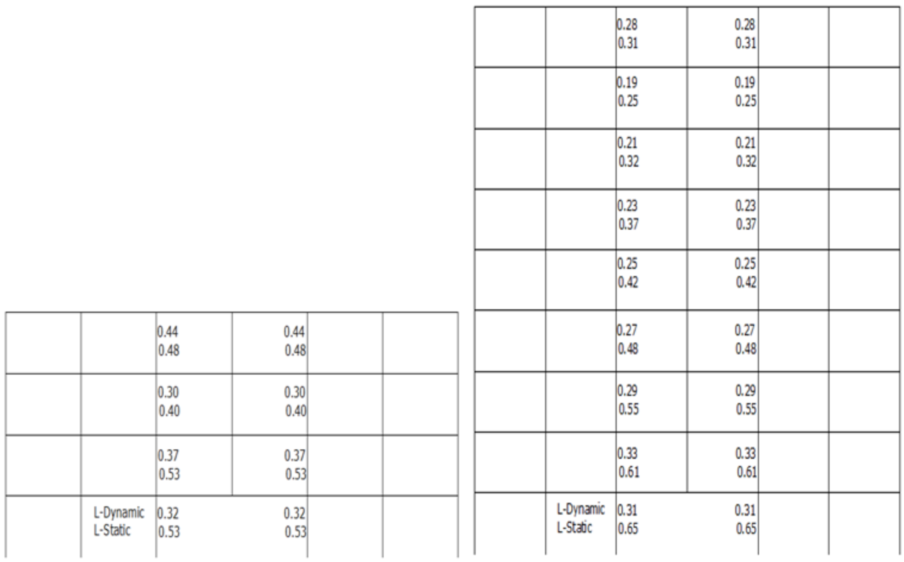


Figure 5.60: DCR for column for case-4 by GSA guideline

Fig. 5.61 and 5.62 show the DCR for column removal case-1 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

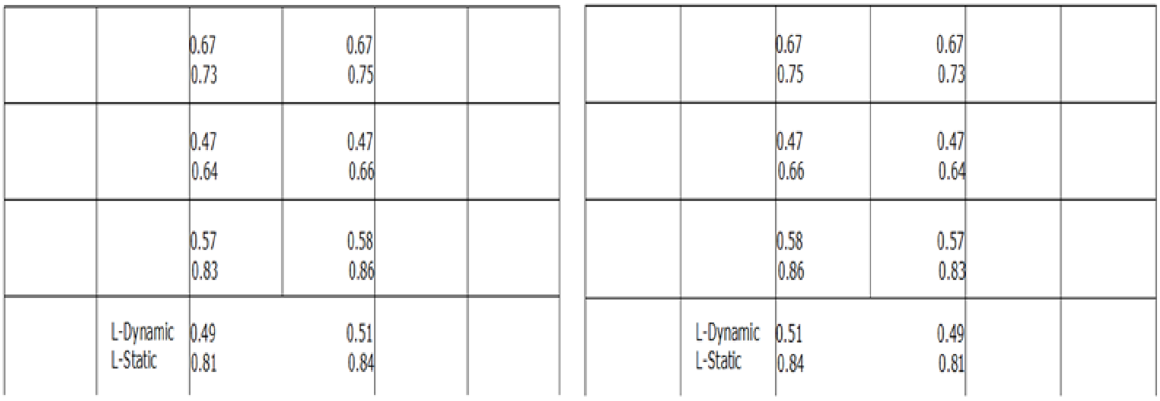


Figure 5.61: DCR for column for case-1 by DoD guideline for 4-storey (for ±X direction)

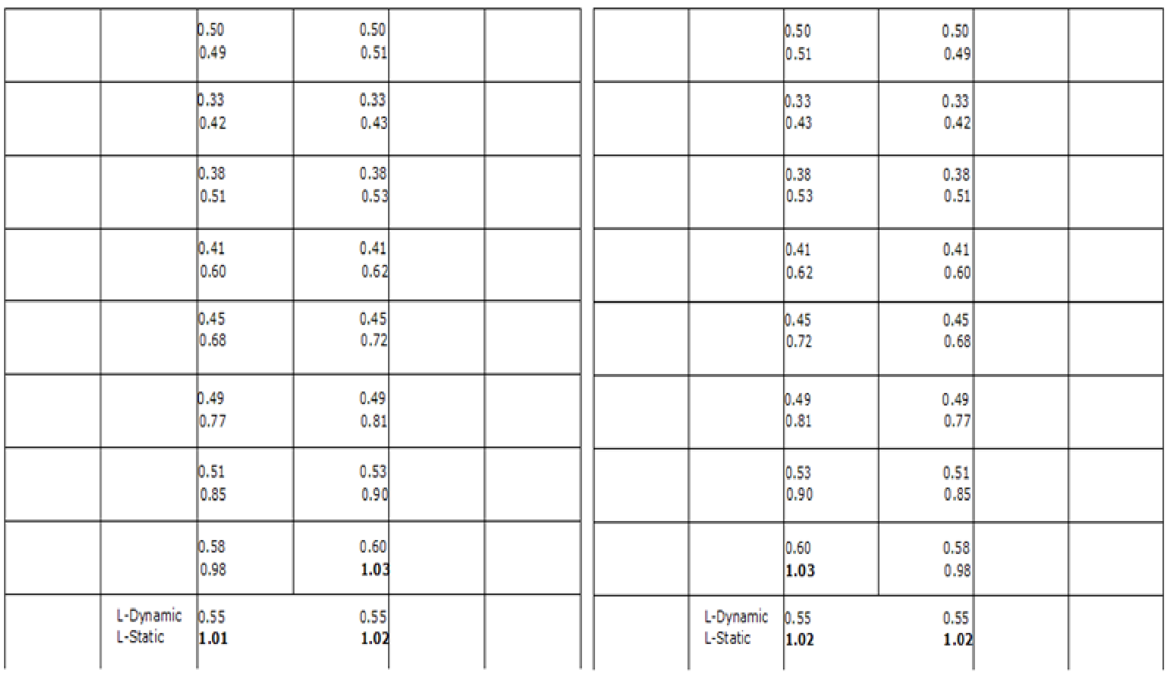


Figure 5.62: DCR for column for case-1 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.63 and 5.64 show the DCR for column removal case-1 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

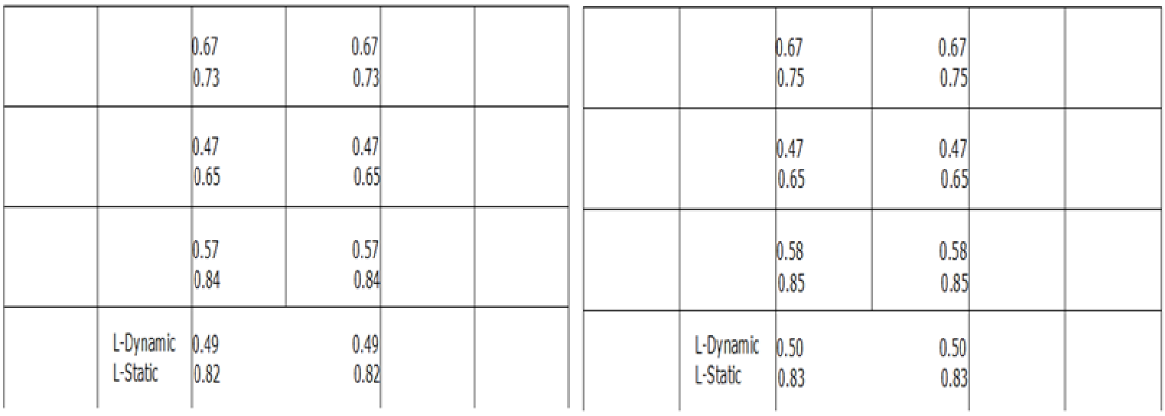


Figure 5.63: DCR for column for case-1 by DoD guideline for 4-storey (for ±Y direction)

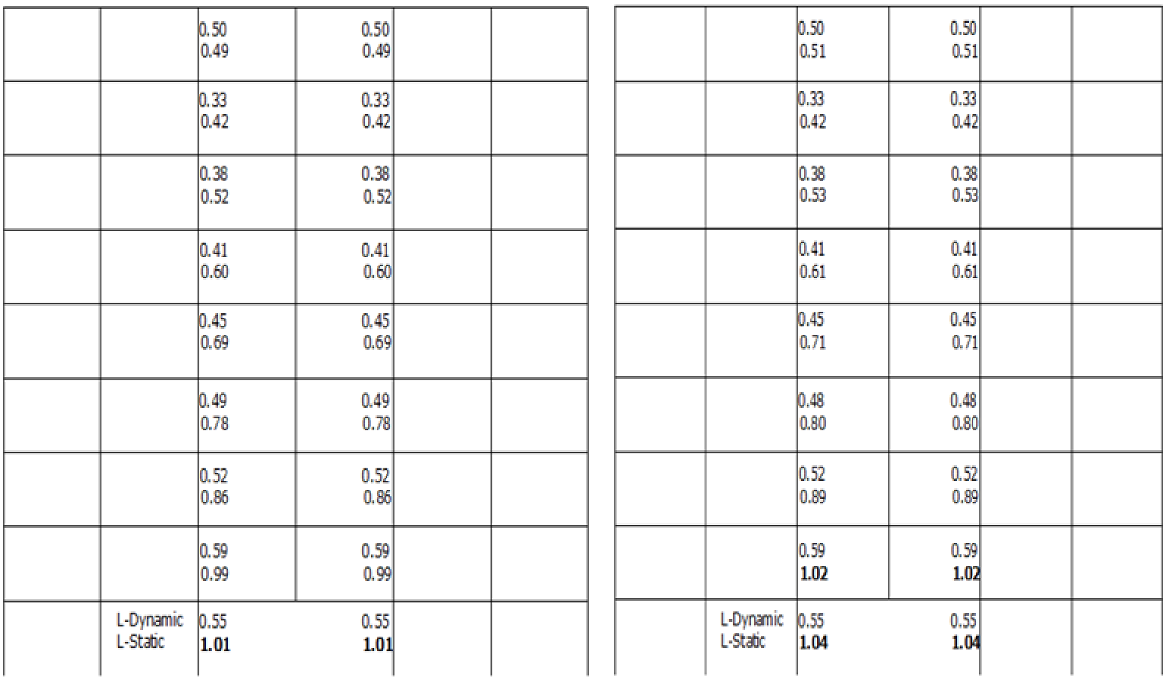


Figure 5.64: DCR for column for case-1 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.65 and 5.66 show the DCR for column removal case-2 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

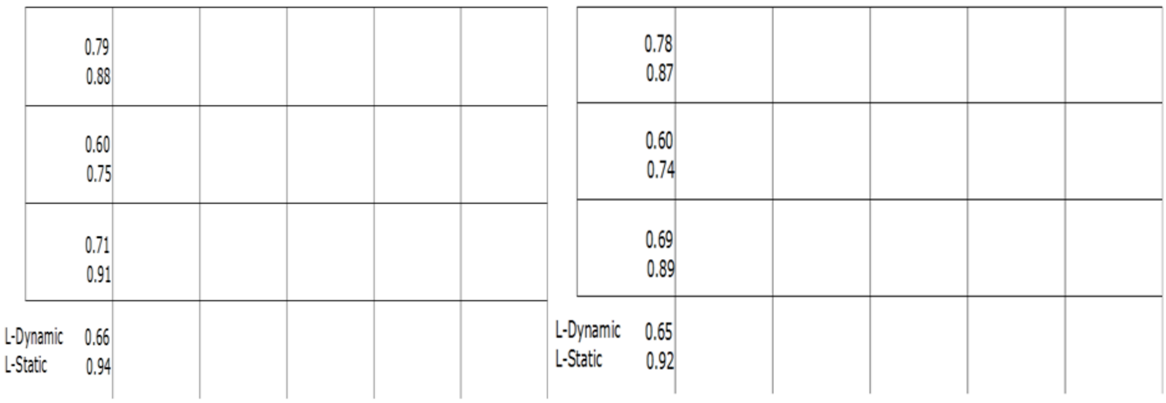


Figure 5.65: DCR for column for case-2 by DoD guideline for 4-storey (for ±X direction)

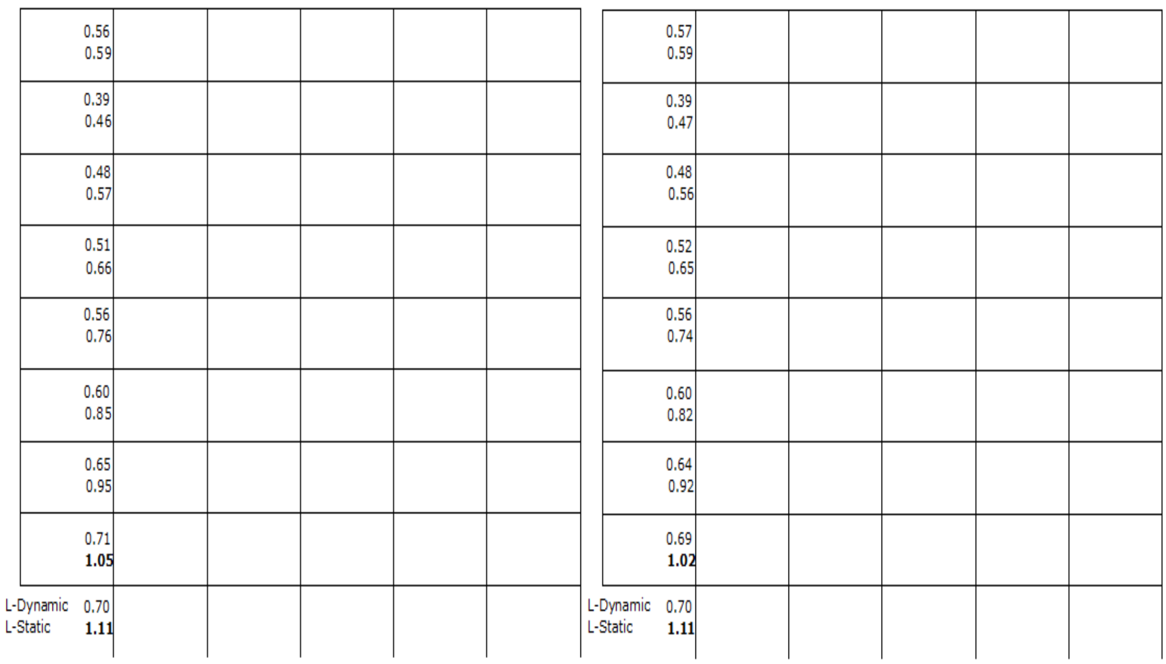


Figure 5.66: DCR for column for case-2 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.67 and 5.68 show the DCR for column removal case-2 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

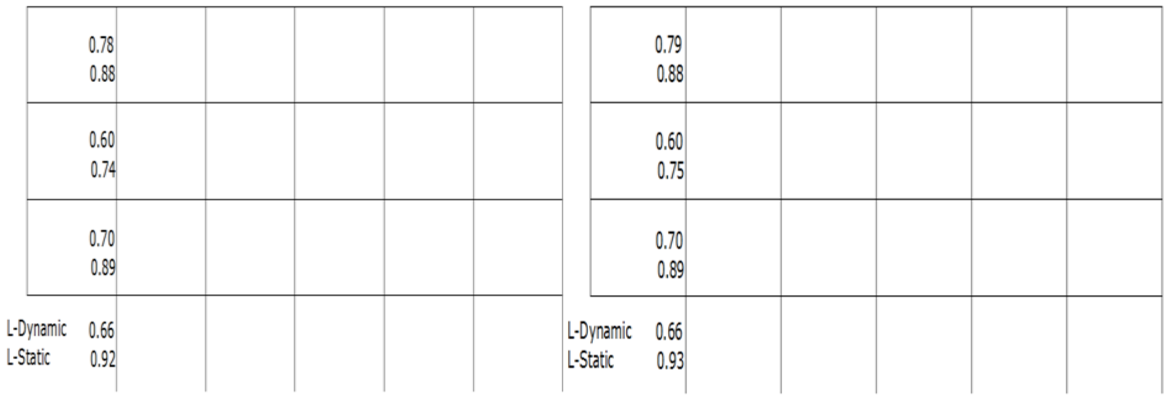


Figure 5.67: DCR for column for case-2 by DoD guideline for 4-storey (for ±Y direction)

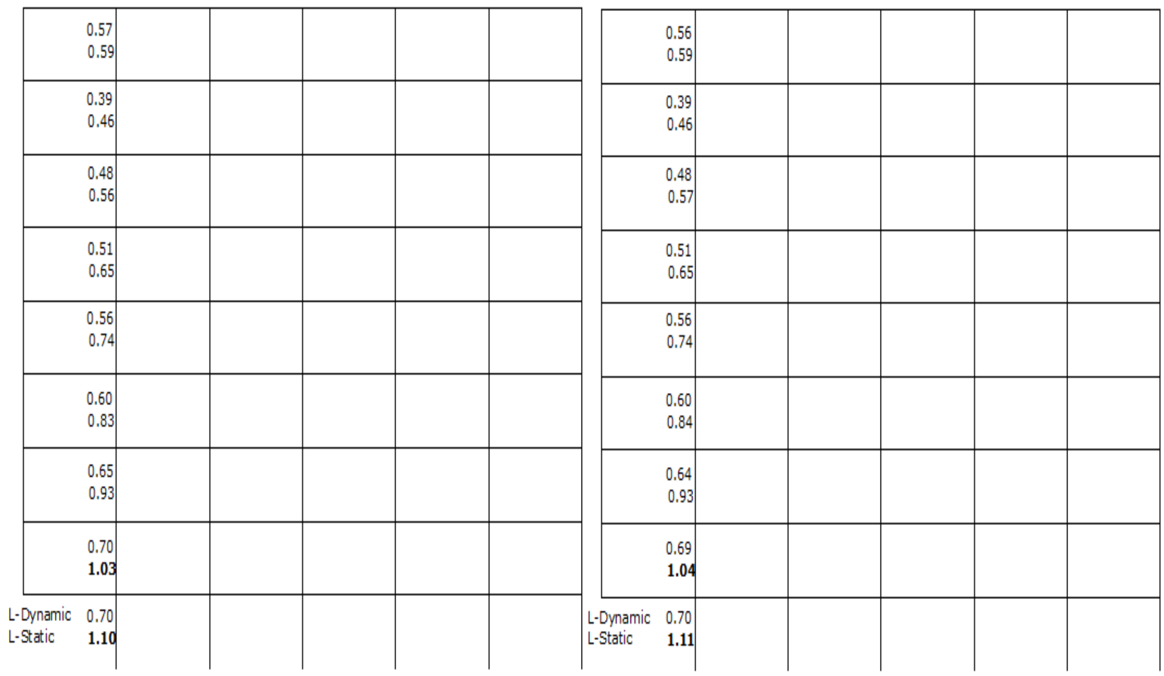


Figure 5.68: DCR for column for case-2 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.69 and 5.70 show the DCR for column removal case-3 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

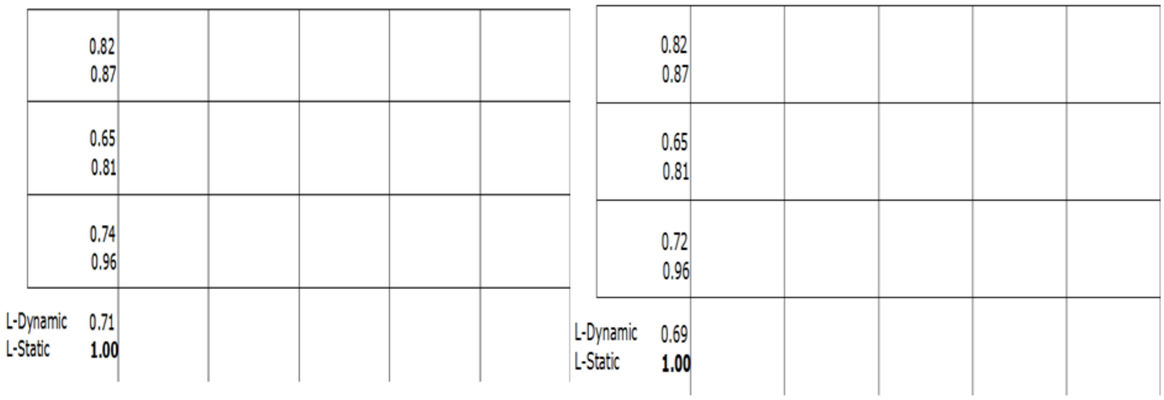


Figure 5.69: DCR for column for case-3 by DoD guideline for 4-storey (for ±X direction)

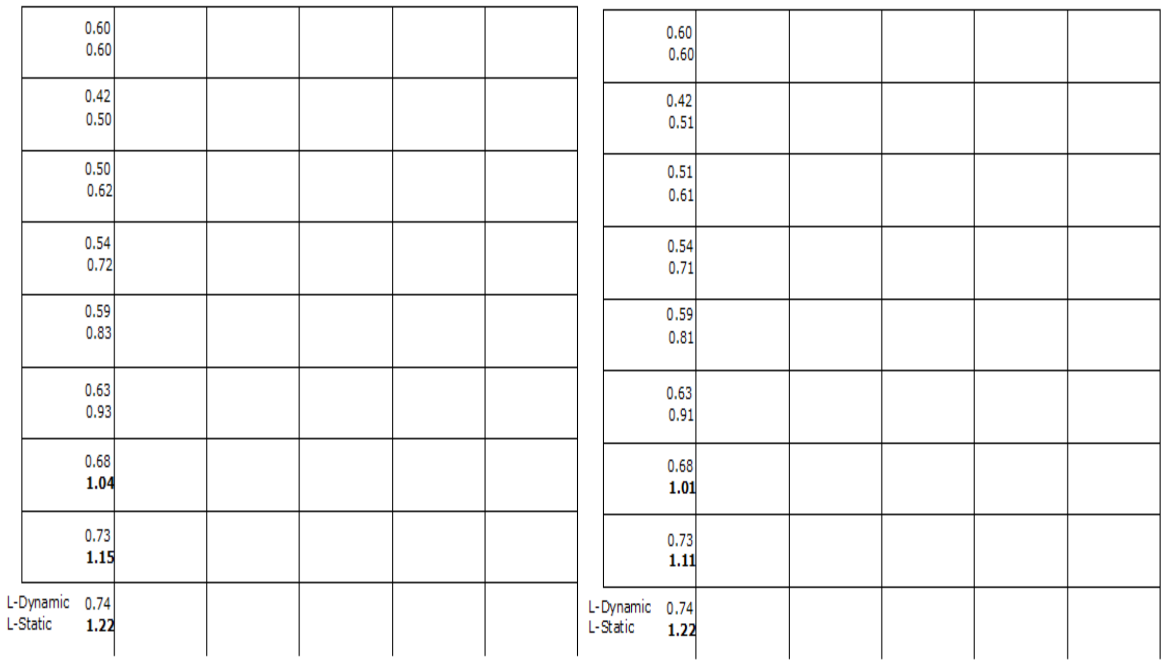


Figure 5.70: DCR for column for case-3 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.71 and 5.72 show the DCR for column removal case-3 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

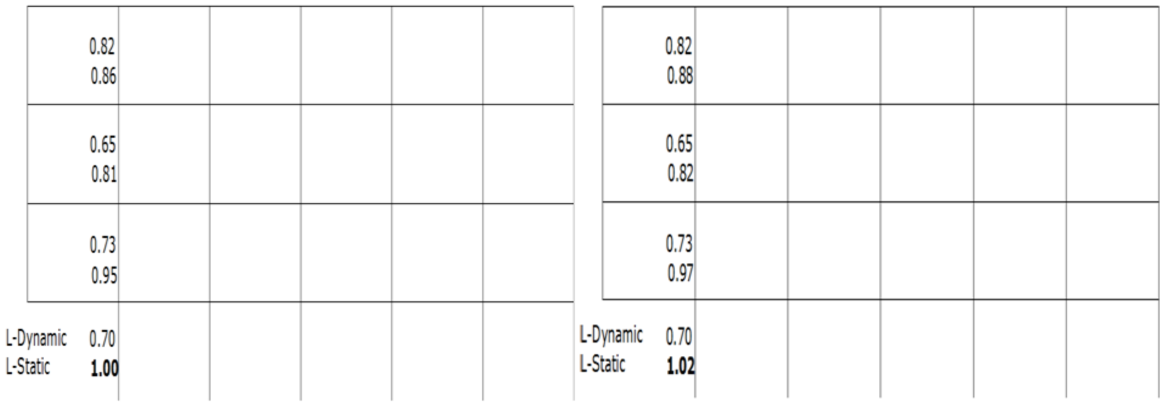


Figure 5.71: DCR for column for case-3 by DoD guideline for 4-storey (for ±Y direction)

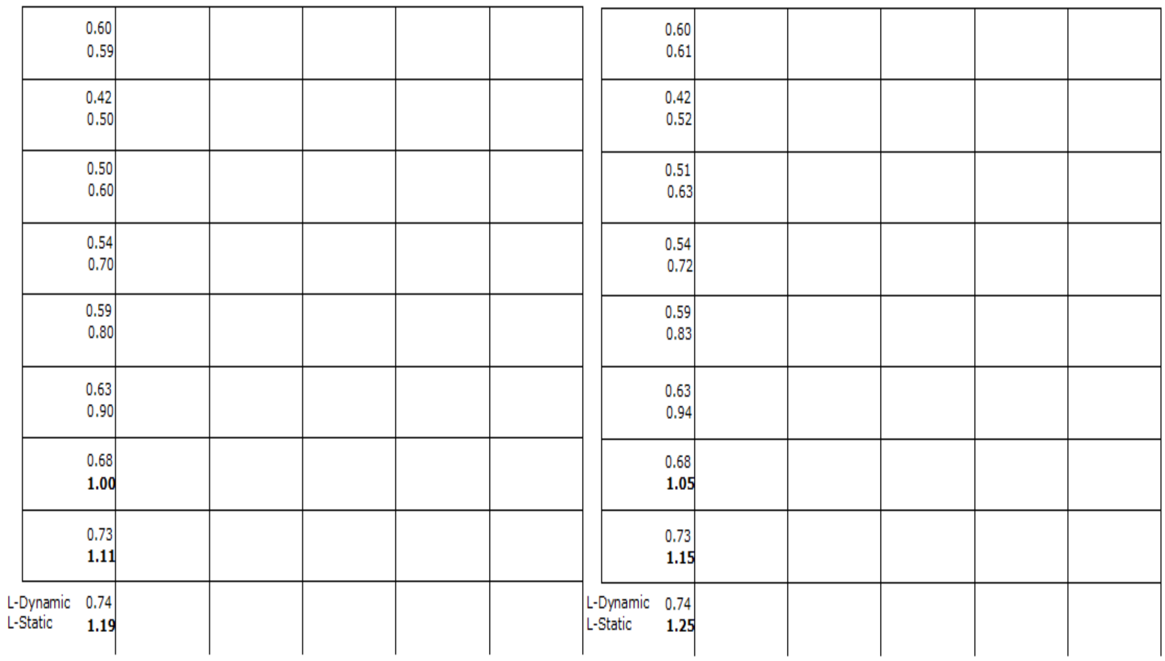


Figure 5.72: DCR for column for case-3 by DoD guideline for 9-storey (for ±Y direction)

Fig. 5.73 and 5.74 show the DCR for column removal case-4 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±X-direction.

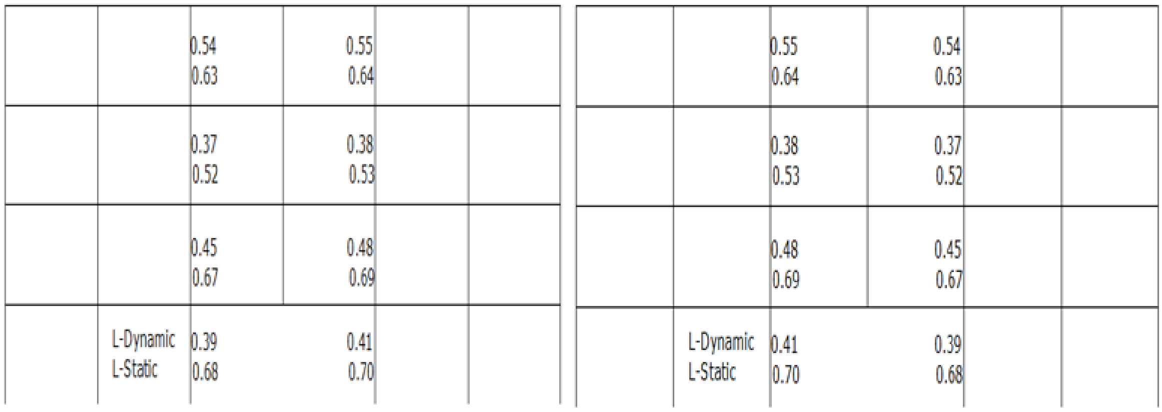


Figure 5.73: DCR for column for case-4 by DoD guideline for 4-storey (for ±X direction)

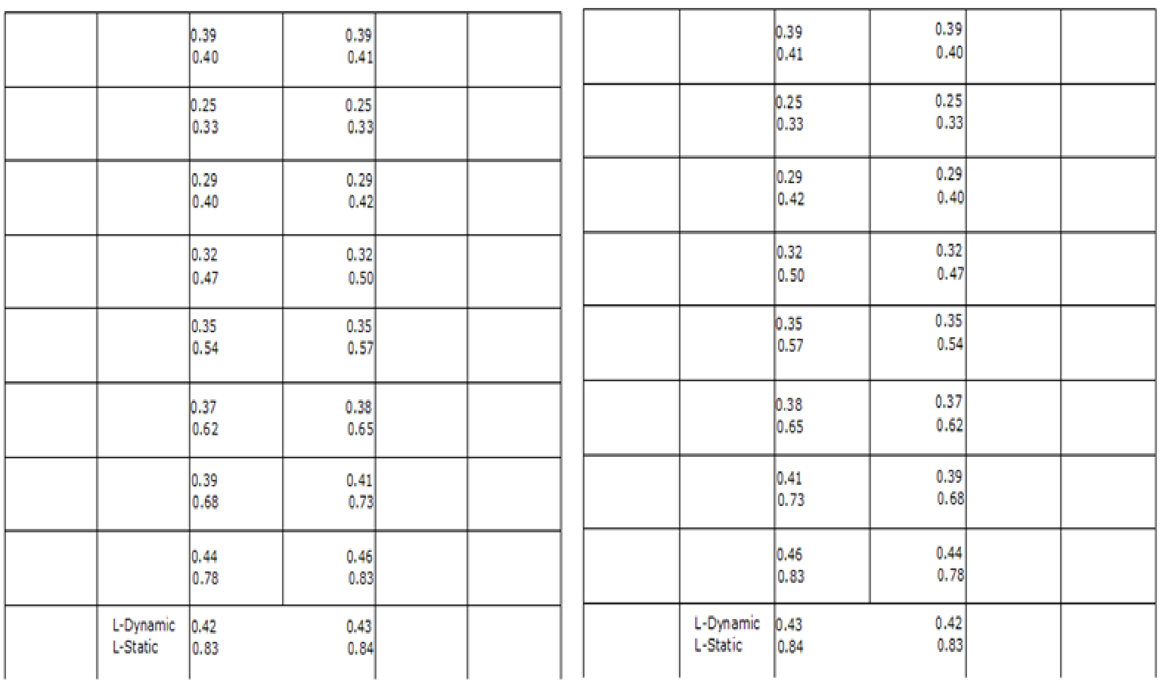


Figure 5.74: DCR for column for case-4 by DoD guideline for 9-storey (for ±X direction)

Fig. 5.73 and 5.74 show the DCR for column removal case-4 as per DoD guideline for 4-storey and 9-storey buildings. Lateral load is applied in ±Y-direction.

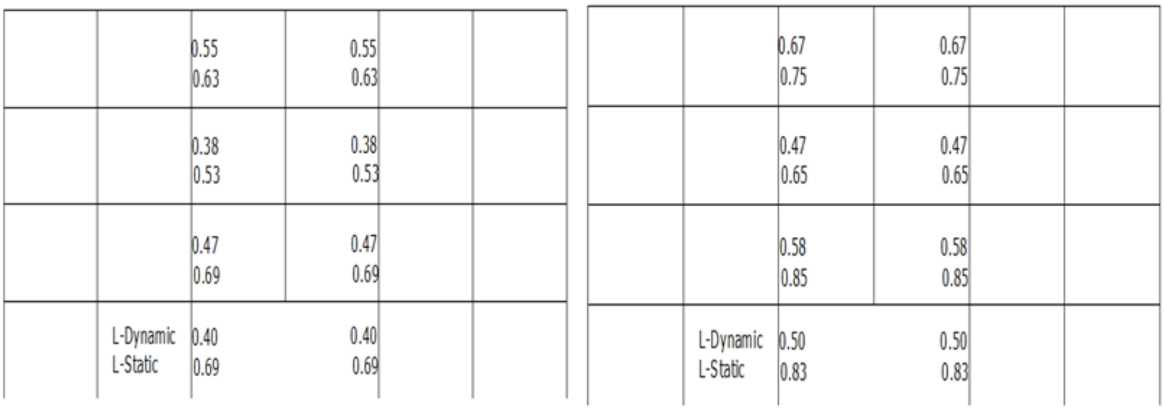


Figure 5.75: DCR for column for case-4 by DoD guideline for 4-storey (for ±Y direction)

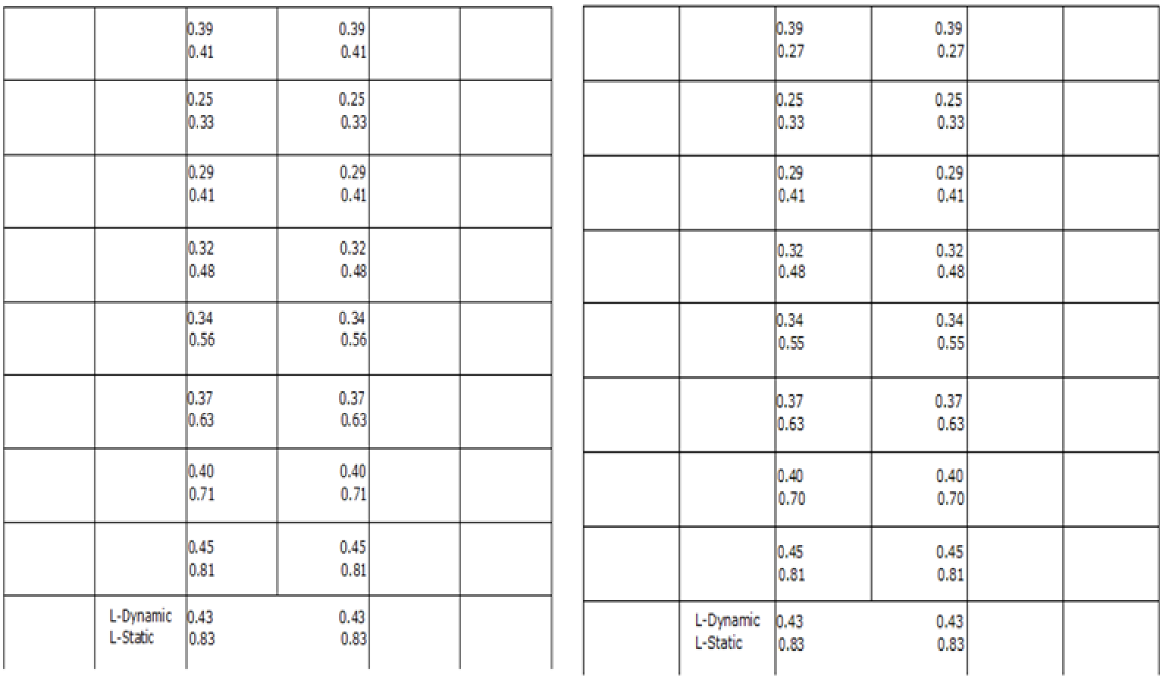


Figure 5.76: DCR for column for case-4 by DoD guideline for 9-storey (for ±Y direction)

5.4.4 Comparison of displacements for linear static and linear dynamic analysis

Displacements are calculated for different column removal cases for linear static and linear dynamic analysis for 4-storey and 9-storey buildings. Fig. 5.77 and Fig. 5.78 show the displacement for case-1 column removal for 4-storey and 9-storey buildings respectively. Displacements are calculated using GSA and DoD guidelines.

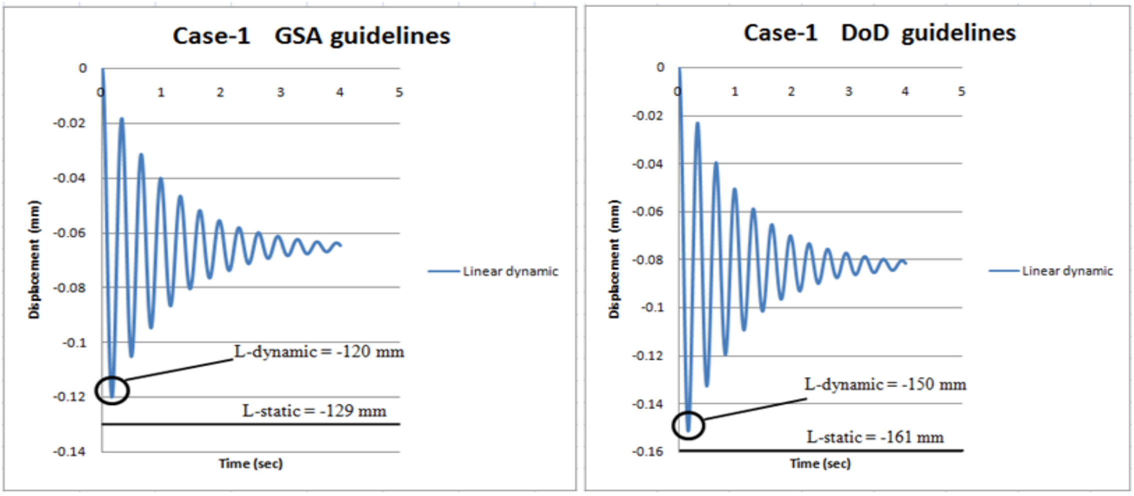


Figure 5.77: Displacement for case-1 by GSA and DoD guidelines for 4-storey

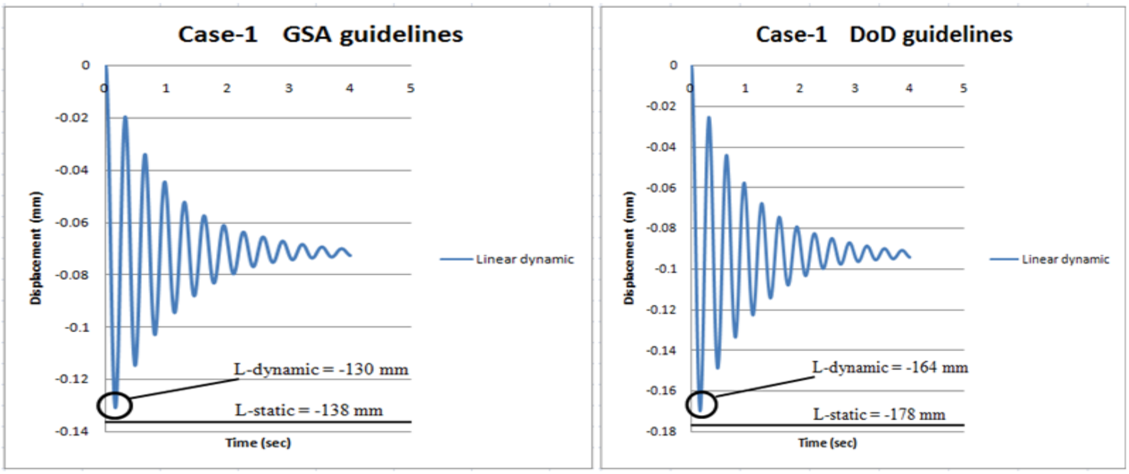


Figure 5.78: Displacement for case-1 by GSA and DoD guidelines for 9-storey

Fig. 5.79 and Fig. 5.80 show the displacement for case-2 column removal for 4-storey and 9-storey buildings respectively.

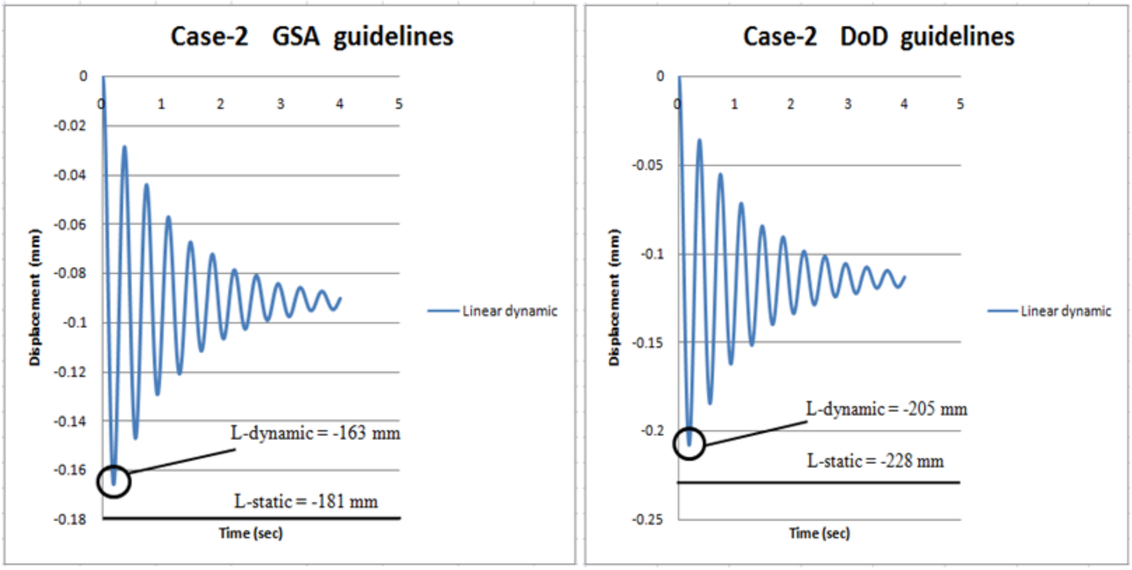


Figure 5.79: Displacement for case-2 by GSA and DoD guidelines for 4-storey

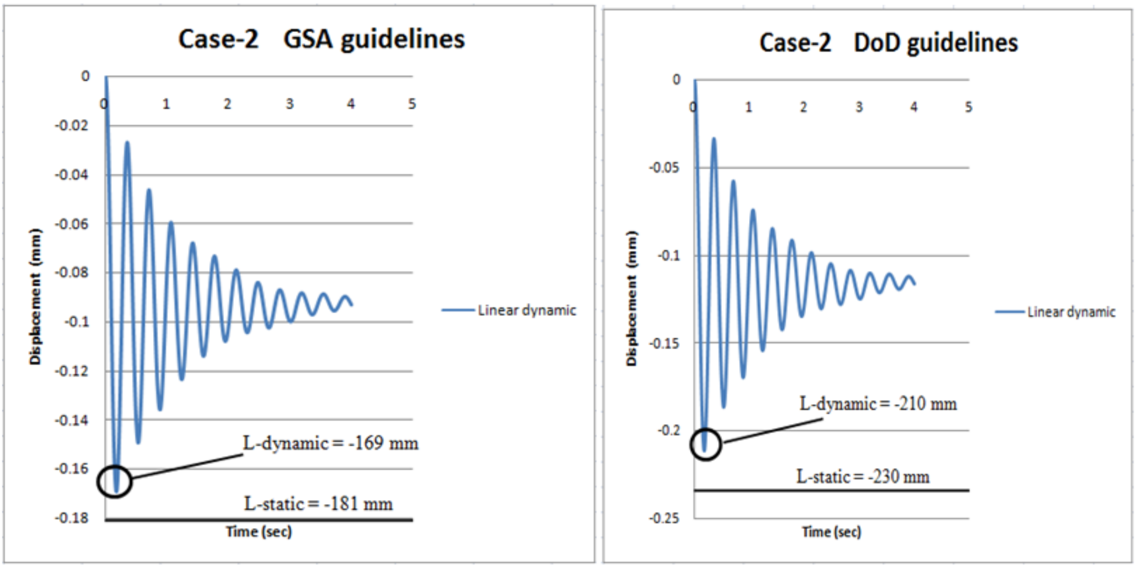


Figure 5.80: Displacement for case-2 by GSA and DoD guidelines for 9-storey

Fig. 5.81 and Fig. 5.82 show the displacement for case-3 column removal for 4-storey and 9-storey buildings respectively.

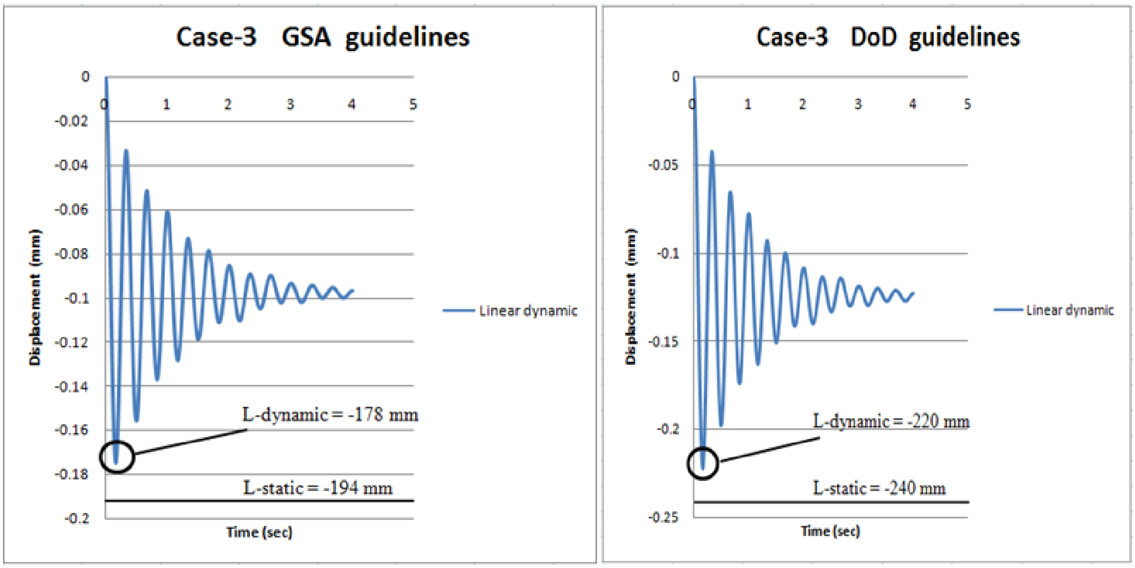


Figure 5.81: Displacement for case-3 by GSA and DoD guidelines for 4-storey

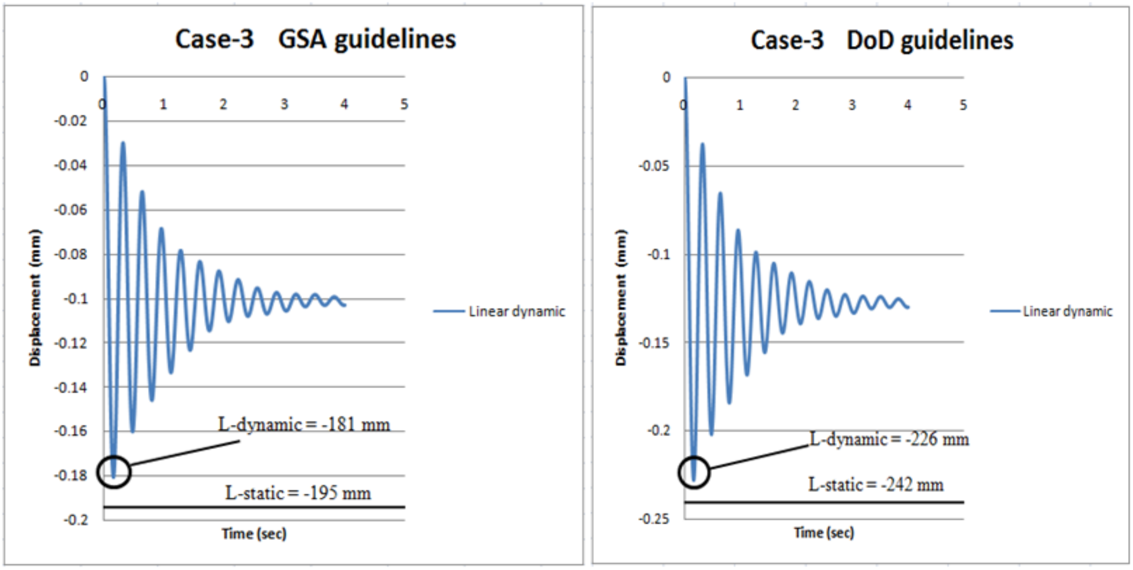


Figure 5.82: Displacement for case-3 by GSA and DoD guidelines for 9-storey

Fig. 5.83 and Fig. 5.84 show the displacement for case-3 column removal for 4-storey and 9-storey buildings respectively.

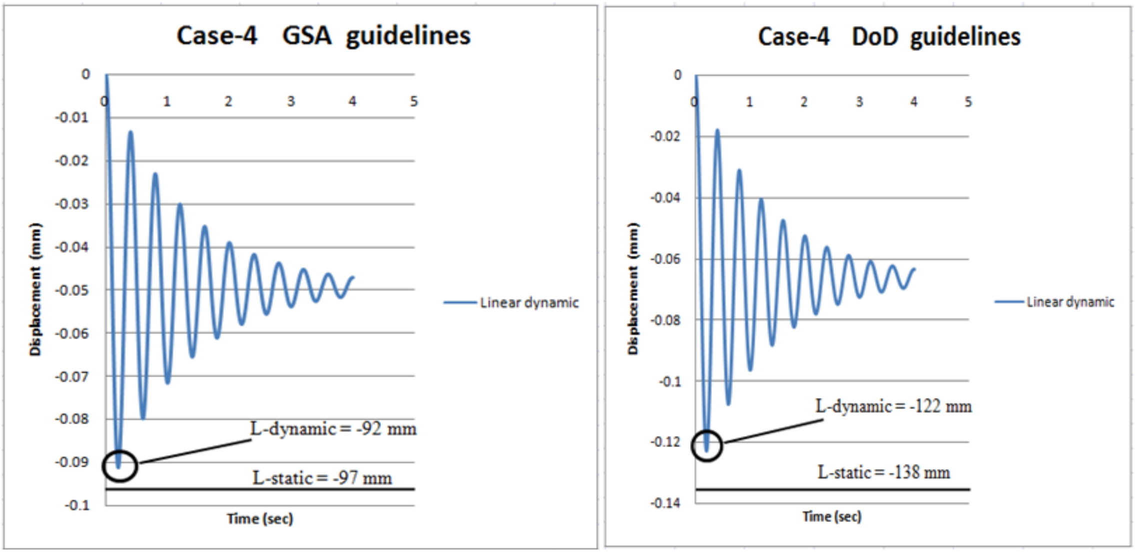


Figure 5.83: Displacement for case-4 by GSA and DoD guidelines for 4-storey

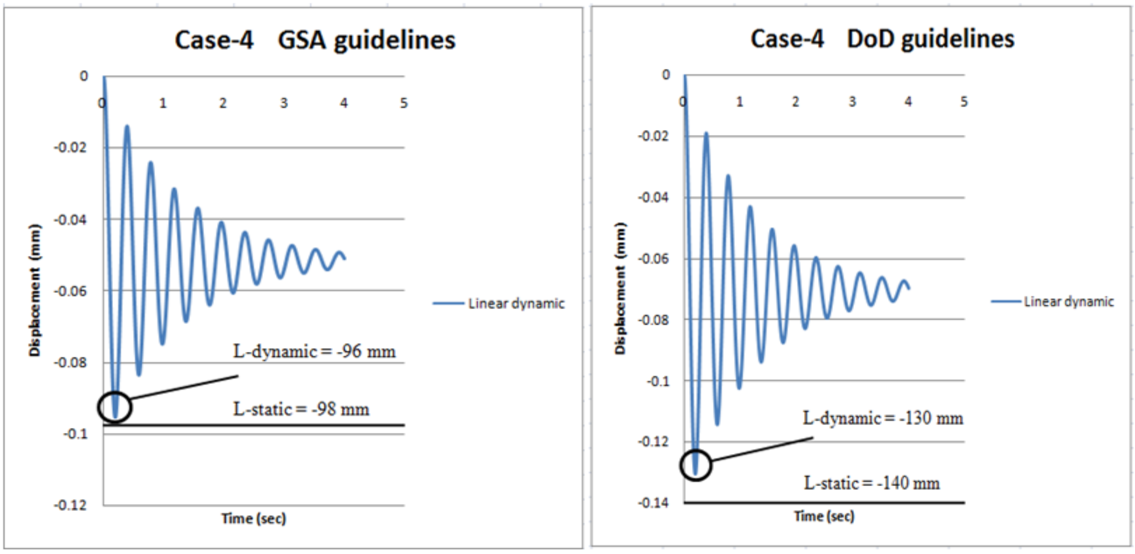


Figure 5.84: Displacement for case-4 by GSA and DoD guidelines for 9-storey

5.5 Results and discussion

In this chapter linear static and linear dynamic analysis procedures are carried out for progressive collapse analysis of 4-storey and 9-storey moment resistant steel buildings. DCR is found out for beams and highly stressed near by columns at all storey for four column removal cases. Comparison of DCR by GSA and DoD guidelines is carried out for each column removal case. Study of the vertical displacement under the column removal locations is carried out for all the column removal cases using linear static and dynamic analysis.

It is observed that DCR in flexure in beam exceeds permissible limit of 2 in all storey of building. DCR calculated by linear dynamic analysis is having values nearer to DCR calculated by linear static analysis. DCR calculated by DoD guidelines is having higher values compared to DCR calculated by GSA guidelines because of difference in loadings. DoD guidelines use larger load factors compared to GSA guidelines. In DoD guidelines lateral loads of 0.2% of sum of vertical loads are also applied while GSA guidelines do not specify any lateral loads. DCR calculated in flexure and shear for beams by linear static analysis is higher on left and right side of column removal points while on center generally linear dynamic analysis gives higher value. So for better results both the analysis methods should be followed for progressive collapse analysis. From the study it is observed that out of all the four column removal cases, case-3 column removal is having worst effect on the building for both 4-storey and 9-storey buildings. So it can be concluded that when the corner column is removed from the building, it will have high potential for progressive collapse for the type of building considered in this study. DCR values for flexure, shear and column increase as the height of the building increases. So potential for progressive collapse of the building increases as the height of the building increases for the studied buildings.

Displacements under the column removal locations found from linear static analysis are compared with displacements obtained by linear dynamic analysis for all the four column removal cases. Maximum calculated deflection due to linear dynamic analysis is 5-10% smaller than the deflection obtained by static linear analysis.

5.6 Summary

In this chapter linear static and linear dynamic analysis are carried out for progressive collapse analysis for 4-storey and 9-storey steel buildings. DCR are found for beams and highly stressed near by columns for all the four column removal cases as per GSA and DoD guideline. Linear static analysis procedure is also explained using staged construction option available in SAP2000. Two methods are explained for carrying out linear dynamic analysis using without initial condition and with initial condition. Displacements under column removal locations are also compared for linear static analysis and linear dynamic analysis.

Chapter 6

Nonlinear static and dynamic analysis

6.1 General

Out of four analysis procedures; nonlinear static and nonlinear dynamic analysis procedures are discussed in this chapter. To understand the nonlinear static analysis, illustrative examples of simply supported beam and continuous beams are studied using nonlinear static analysis procedure (vertical pushover analysis) of SAP2000. Collapse loads of the beams, placement of hinges at appropriate locations, formation of the hinges at yield and at collapse, formation of the mechanism etc. are studied before doing nonlinear static analysis of the entire building. Nonlinear static analysis is performed for 4-storey and 9-storey steel buildings using SAP2000. Illustrative example of nonlinear dynamic analysis procedure is also carried out using SAP2000. After the understanding of nonlinear dynamic analysis of continuous beam, analysis is carried out for 4-storey and 9-storey steel buildings. Displacement ductility and support rotations are found out using nonlinear static and dynamic analysis procedures.

6.1.1 Difference between linear and nonlinear analysis cases

Every analysis case is considered to be either linear or nonlinear. The difference between these two options is very significant in SAP2000. Table 6.1 shows the difference between linear and nonlinear analysis cases in SAP2000.

Table 6.1: Difference between linear and nonlinear analysis cases

	Linear analysis case	Nonlinear analysis case
Structural properties	constant	may vary with time
Initial conditions	The analysis starts with zero stress	The analysis may continue from a previous nonlinear analysis
Structural response and superposition	The response is proportional. The results may be superposed	The response is not proportional. The results cannot be superposed

6.1.2 Nonlinear static analysis of simply supported beam

Nonlinear static analysis of ISMB500 beam is carried out. Fig. 6.1 shows the simply supported beam of 4m length subjected to point load in the center. Hence collapse load (Wc) as per plastic analysis is (4Mp/L), where Mp is plastic moment capacity of the section.

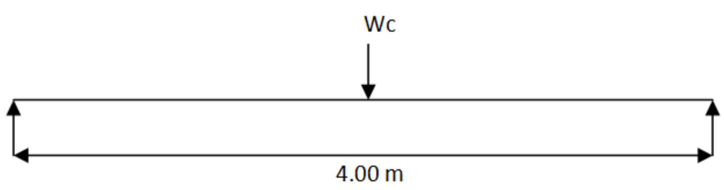


Figure 6.1: Simply supported beam

Mp = Zp.fy

Mp = 2070.4 X 250 X 100 X 10^{-6} = 517.6 kN-m

So, Wc = 517.6 kN

Modeling in SAP2000

When simply supported beam is subjected to point load in the center, hinge will form in the center under the application of load. SAP2000 provides two types of hinge properties

(1) Automatic hinge properties and (2) user-defined hinge properties. There are five default hinge options available: Axial (P), Torsion (T), Moment (M2 or M3), shear (V2 or V3), and coupled (P-M2-M3). For this example automatic moment (M3) hinge is assigned to beam. Point load of 517.6 kN is applied as live load in the center of the beam. Self weight of the beam is neglected. Step-wise increase of the load is applied. Fig. 6.2 shows the nonlinear analysis case definition in SAP2000.

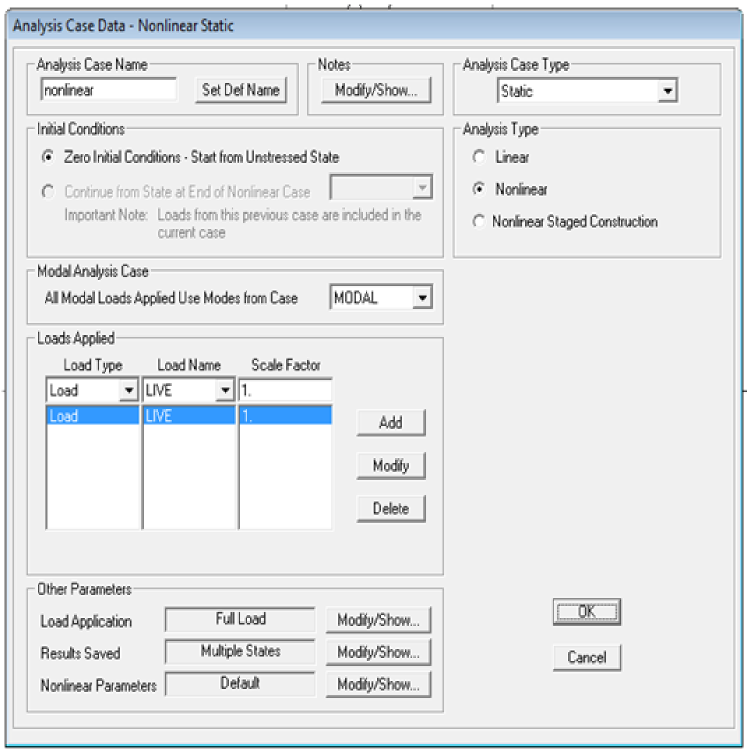


Figure 6.2: Non-linear analysis case definition for simply supported beam

Results

Vertical deflection is found out for each analysis step. A graph of percentage of load Vs vertical deflection under the application of point load is drawn. Percentage of load is found by summation of the reactions obtained at the supports for each analysis step divided by total point load applied initially. Table-6.2 shows the reaction obtained in each step of nonlinear analysis and corresponding vertical deflection at each step. First hinge formation occurred at the 9^{th} step of the analysis when the beam reaches at the elastic limit and beam collapsed at 13^{th} step of the analysis when the percentage of load reached to 100%. Fig. 6.3 shows a graph of vertical deflection vs percentage of load.

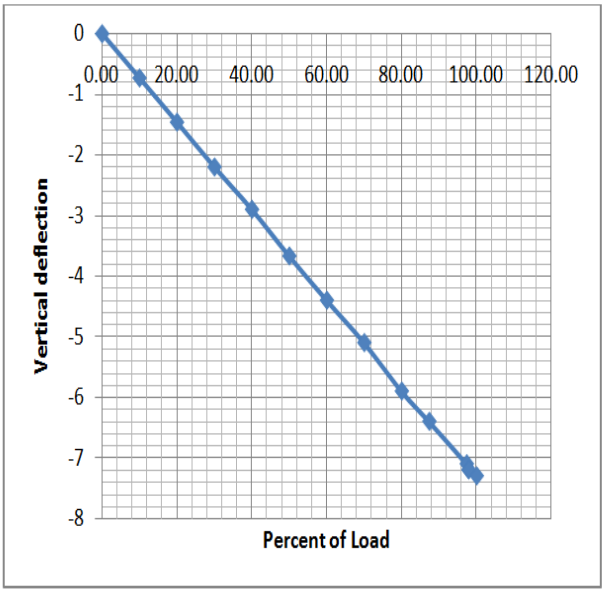


Figure 6.3: Vertical deflection vs percentage of load

6.1.3 Nonlinear static analysis of continuous beam

An ISMB500 is taken as two span continuous beam of 4m span each subjected to 2000 kN load in the center as shown in Fig. 6.4. Consider left part of beam as AB and right part of beam as BC.

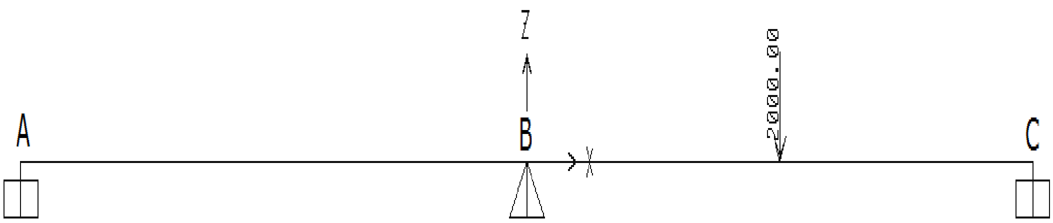


Figure 6.4: Continuous beam

Table 6.2: Reaction obtained in each step of analysis and corresponding deflection

Step	Vertical deflection	Reaction	Percentage of load
	(mm)	(kN)	%
1	-0.73	51.76	10
2	-1.46	103.52	20
3	-2.2	155.28	30
4	-2.9	207.04	40
5	-3.67	258.80	50
6	-4.4	310.56	60
7	-5.1	362.32	70
8	-5.9	414.08	80
9 (1^{st})	-6.4	452.18	87.36
10	-7.1	503.94	97.36
11	-7.2	506.44	97.84
12	-7.2	506.44	97.84
13 (fail)	-7.3	517.60	100

A linear static analysis is carried out first. Fig. 6.5 shows the bending moment diagram after linear static analysis. Linear static analysis does not show any hinge formation in the beam at yielding or collapse of the beam at its ultimate flexural capacity. In other words bending moment diagram is independent of capacity of beam.

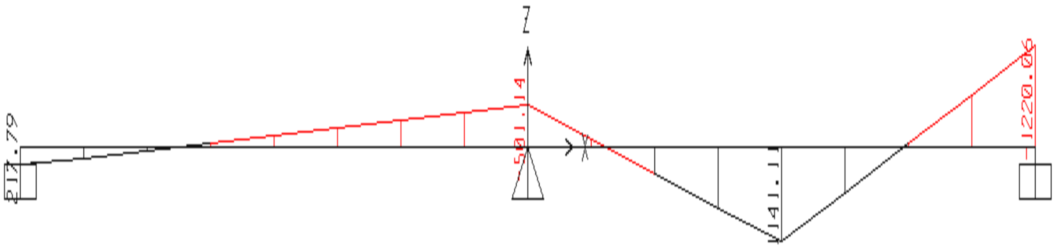


Figure 6.5: Bending moment after linear static analysis

Bending moment capacity of the section is 518 kN-m but the linear static analysis procedure does not give any indication of the capacity of the section provided while nonlinear static analysis procedure gives the idea about the elastic limit of the section as well as the ultimate load carrying capacity of the section in terms of hinge formation as explained in the example of simply supported beam. To perform nonlinear static analysis default M3 hinges at both the ends of the beam and in the center of the beam are assigned. Fig. 6.6 shows the bending moment diagram after nonlinear analysis.

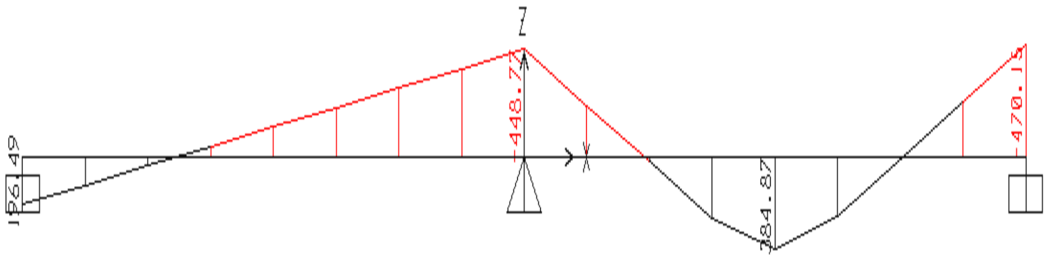


Figure 6.6: Bending moment after nonlinear static analysis

Bending moment diagram shown here is for collapse load of 950 kN. At the collapse load, mechanism is formed in the section and the beam collapses. After the formation of mechanism, further analysis is not carried out by software unlikely linear static analysis. Fig. 6.7 shows the deflected shape of the beam and hinge formation when mechanism is formed.

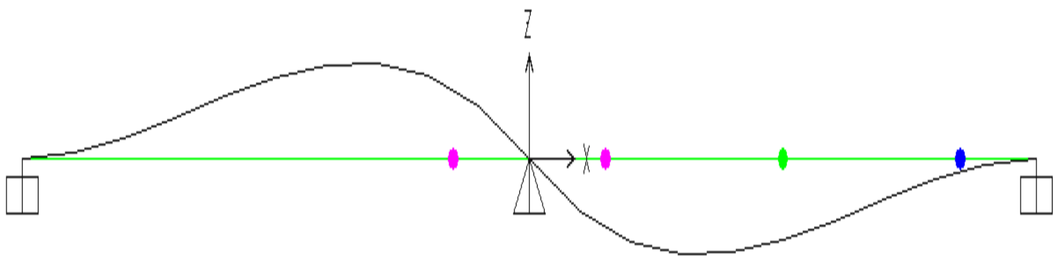


Figure 6.7: Deflected shape after nonlinear static analysis

Now, in the same example if hinge in the middle of the beam is not provided then mechanism will not form in the section and beam will take the 100% load. In this case collapse load achieved is equal to the load applied on the beam. In this case though beam is taking full load without failure, hinges will not take the load beyond their load carrying capacity and mechanism will not be formed. So, the proper location for assignment of hinges is very important to get the true behavior of the structure.

6.1.4 Nonlinear dynamic analysis of continuous beam

As mentioned above, dynamic analysis procedures, especially nonlinear dynamic, are usually avoided due to the complexity of the analysis. Additionally, evaluation and validation of the results can be very time consuming. Now for nonlinear dynamic analysis procedure of a continuous beam, remove the intermediate support and apply a point load corresponding to reaction carried by intermediate support. Reaction carried by intermediate support for linear static analysis is 1003.41 kN. To observe the behavior corresponding to progressive collapse, a sudden removal of point load is required. Define an analysis case and apply all the load cases as shown in Fig. 6.8. After equilibrium is reached, remove the intermediate support by ramping down the point load. The duration for removal of point load should be less than one tenth of the period associated with the structural response mode for the element removal. Fig. 6.9 shows the time-history function definition for removal of point load.

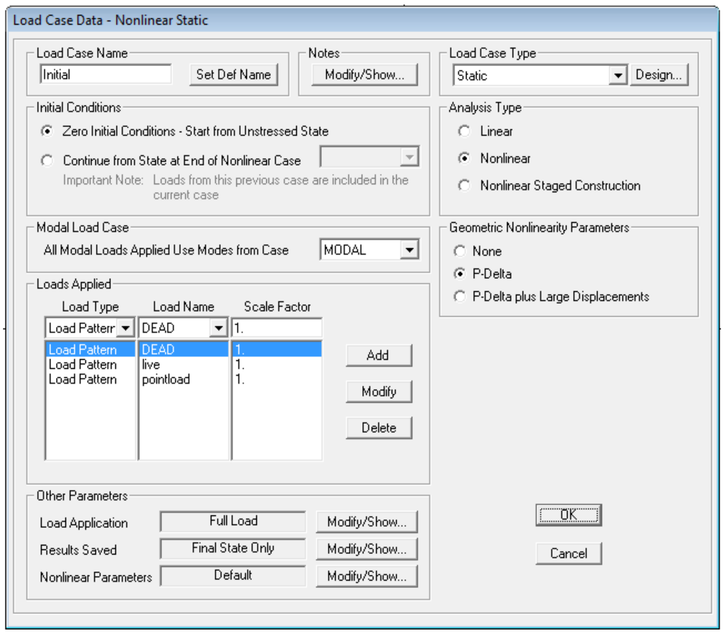


Figure 6.8: Analysis case definition for initial condition

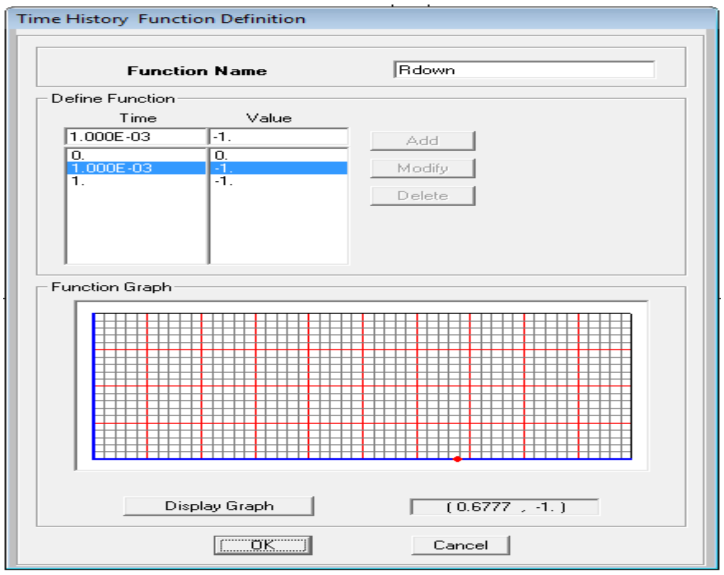


Figure 6.9: Time-history function definition for support removal

Fig. 6.10 shows the analysis case definition for nonlinear dynamic analysis.

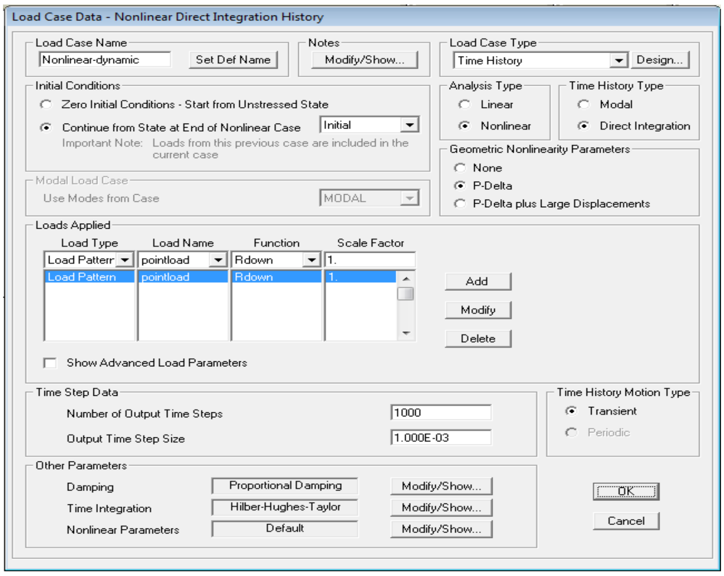


Figure 6.10: Nonlinear-dynamic analysis case definition

After the analysis, it is seen that mechanism is formed in beams and due to that beams collapse and nonlinear static analysis case could not complete successfully, hence nonlinear dynamic analysis case could not be started. In this case collapse load is equal to 950 kN as obtained in nonlinear static analysis procedure. Now to perform nonlinear dynamic analysis, do not allow the mechanism to be formed in individual beams and delete moment hinges from the middle of both the beams and repeat the entire procedure as discussed above. Fig. 6.11 shows the maximum deflection of beam before collapse by nonlinear dynamic analysis. Ductility can be found out by dividing the ultimate displacement to yield displacement.

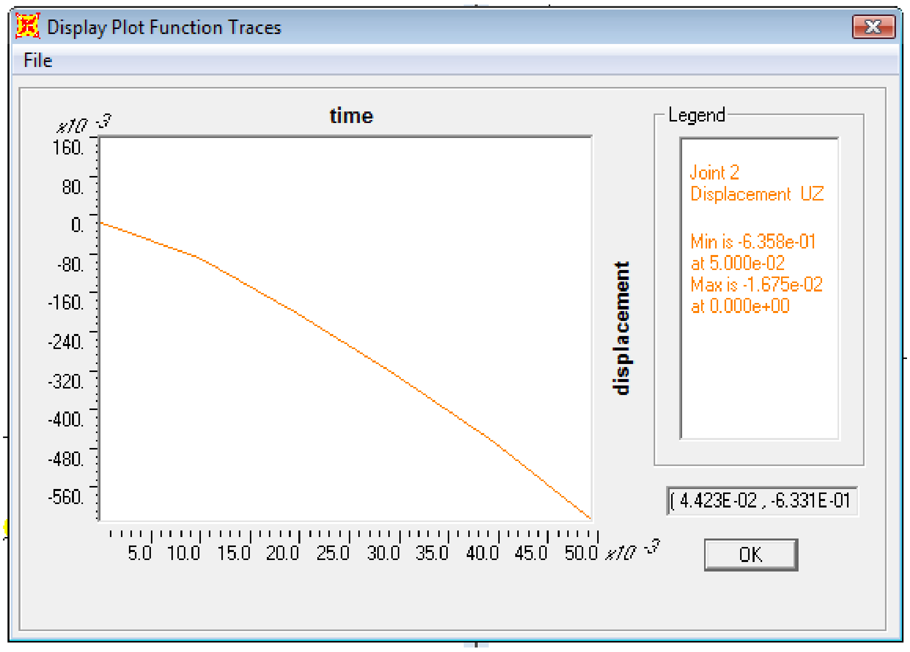


Figure 6.11: Deflection of beam by nonlinear dynamic analysis

6.2 Iterative static analysis procedure as per GSA guideline

GSA guideline describes the step by step linear static analysis procedure for progressive collapse analysis of the steel structure. In this method, for a member whose DCR exceeds the permissible value, a hinge is placed at the member end to release the moment. This hinge should be located at the center of flexural yielding for the member. Use rigid offsets from the connecting member as needed to model the hinge in the proper location. At each inserted hinge, apply equal-but-opposite moments to the offset and member end to each side of the hinge as shown in Fig. 6.12. The magnitude of the moments should be the expected flexural strength of the member. Continue this process until no DCR values are exceeded.

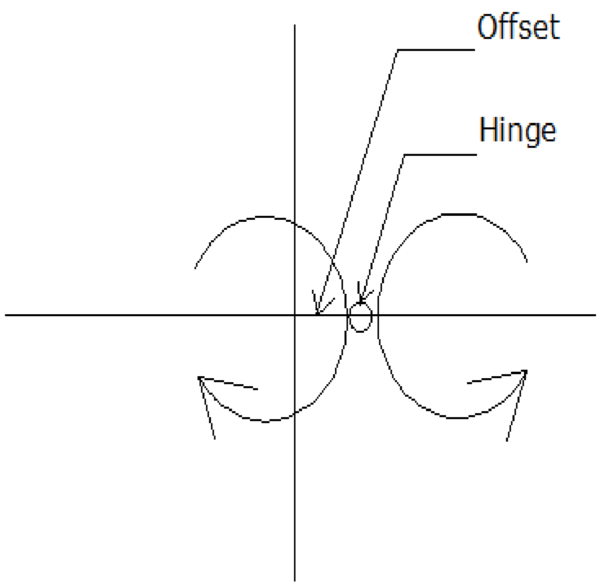


Figure 6.12: Placement of hinge

6.2.1 Analysis procedure

Consider the example of 4-storey building. For case-1 column removal, DCR for flexure is found using linear static analysis procedure in previous chapter. Fig.6.13 shows the DCR for flexure after linear static analysis procedure. Permissible value of DCR for flexure is 2. Members, whose DCR value is greater than 2 are considered failed members. Now in failed members a hinge is placed at a distance of 0.25m (half the depth of the beam) from the support at both the ends. Offset provided for hinge model is a rigid offset. Apply equal and opposite bending moment of 518 kN-m to both the side of the hinge and run the analysis. Fig.6.14 shows the values of DCR after redistribution. Continue this process until no DCR values are exceeded. Here as per the GSA guidelines, permissible collapse area should be limited to 1800 ft^2 at the floor level directly above the instantaneously removed column or the structural bays directly associated with the instantaneously removed column. Here for this building, failure area of the building increases than the permissible area, so the building is said to having high potential for progressive collapse.

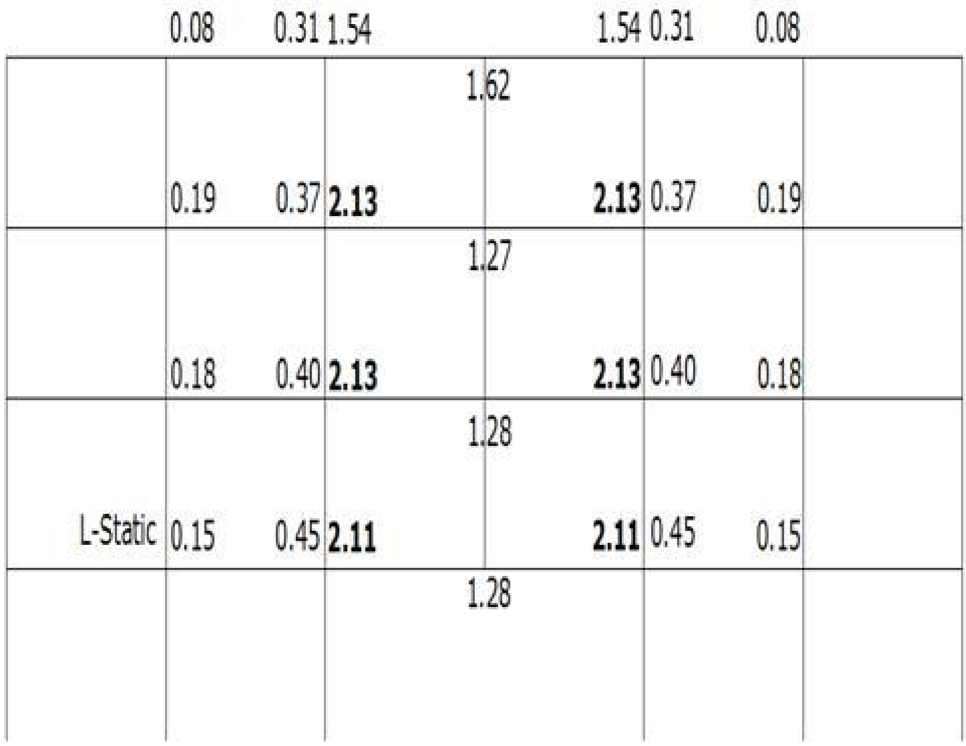


Figure 6.13: DCR after linear static analysis

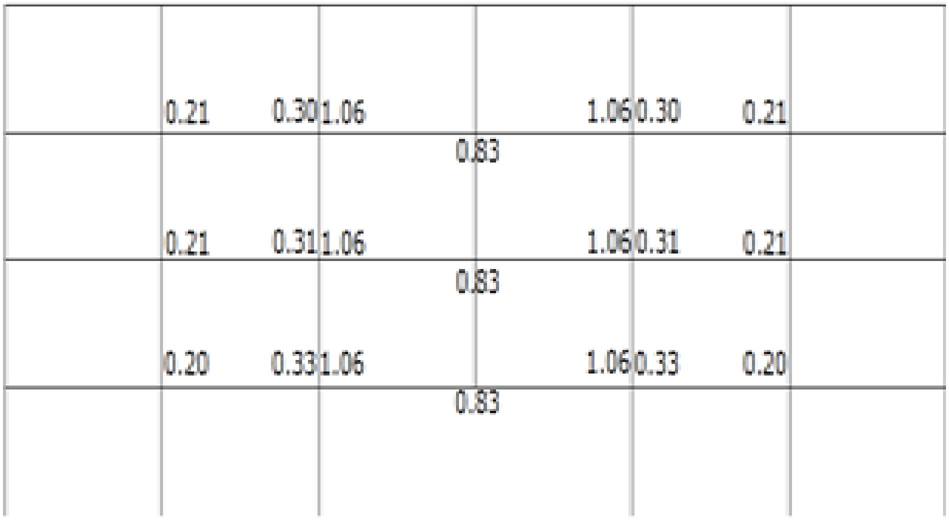


Figure 6.14: DCR after redistribution

This method is performed when the analysis software does not have the capacity to perform non-linear static analysis or push over analysis. Now to perform vertical pushover analysis using SAP2000, default M3 hinges are assigned to beams at both the ends. Default P-M2-M3 hinges are assigned to columns at both the ends. After that nonlinear static analysis is performed using SAP2000 as explained in illustrations. DCR calculated using nonlinear static analysis is shown in Fig. 6.15.

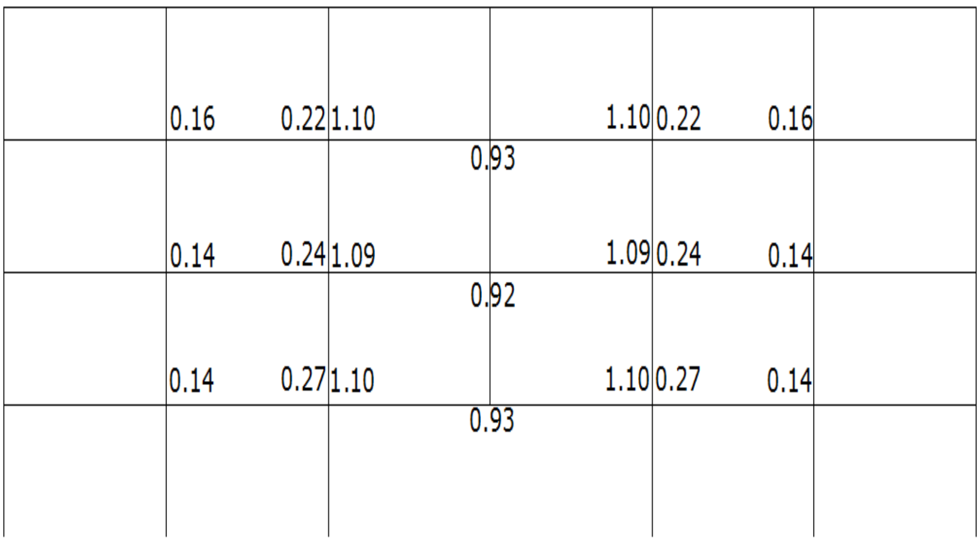


Figure 6.15: DCR after vertical pushover analysis

From Fig. 6.14 and Fig. 6.15, it is seen that DCR found from both the analysis procedures are approximately same. The advantage of nonlinear static (vertical pushover) analysis is its ability to account for nonlinear effects. This method gives the better understanding about the yielding capacity and ultimate capacity of the sections. The method described by GSA guideline does not give any indication of yielding or ultimate capacity of the section.

6.3 Nonlinear static analysis

For progressive collapse analysis, a nonlinear static analysis method implies a stepwise increase of amplified (by a factor of 2) vertical loads until maximum amplified loads are attained or until the structure collapses. This method is also called vertical pushover analysis. This method of vertical pushover analysis is load controlled. Nonlinear static analysis procedure involves following steps.

a. Build a computer model of a structure

b. Define and assign nonlinear plastic hinge properties

c. Apply static load combinations as per GSA and DoD guidelines

d. Perform nonlinear static analysis using SAP2000

e. Verify and evaluate the results based on maximum load resisted as well as maximum plastic rotation of hinge values.

Verification of nonlinear analysis is a somewhat complicated process and highly dependent on analysis parameters (such as load step, tolerance and integration methods), and it may involve several computer analysis reruns. Fig.6.16 shows the nonlinear static analysis case definition as per GSA and DoD guidelines.

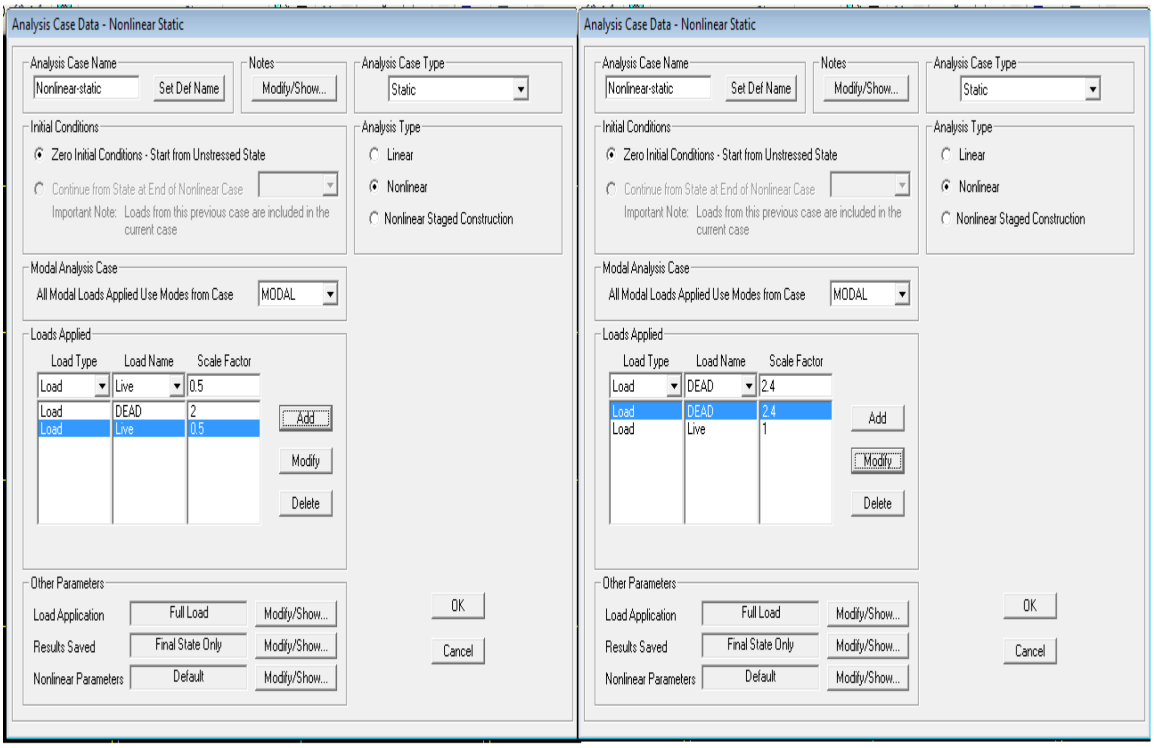


Figure 6.16: Nonlinear static analysis case definition as per GSA and DoD guideline

For nonlinear analysis automatic hinge properties are assigned to a frame element. For default moment hinges, SAP2000 uses Tables 5-6 of FEMA-356[34]. For each degree of freedom, there is a force-displacement (moment-rotation) curve that gives the yield value and the plastic deformation following yield. This is done in terms of a curve with values at five points, A-B-C-D-E, as shown in Fig.6.17. Point A is always origin. B represents yielding. Point C represents the ultimate capacity for pushover analysis. Point D represents a residual strength. Point E represents total failure. There are additional deformation measures at points IO (immediate occupancy), LS (Life safety), and CP (Collapse prevention). These are informational measures. FEMA defines permissible values for plastic rotation of hinges at each stage i.e. IO, LS and CP. Default M3 hinges are assigned to beams at both the ends. Default (P-M2-M3) hinges are assigned to columns at both the ends. Nonlinear static analysis is performed for all the column removal cases.

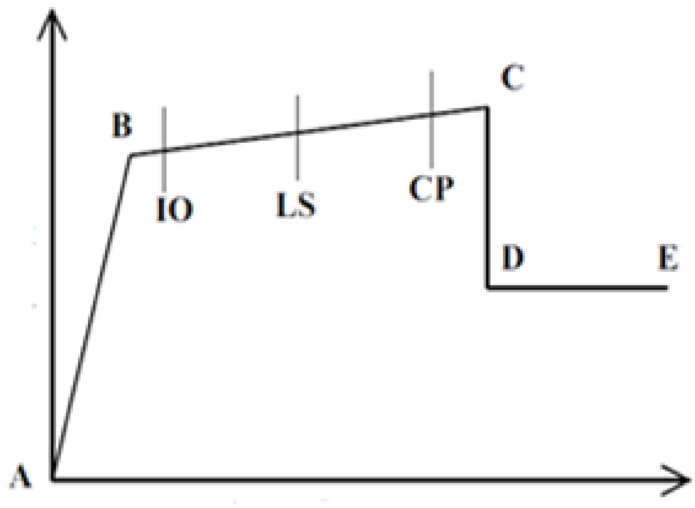


Figure 6.17: Moment rotation curve

After nonlinear static analysis, plastic hinge rotation is found out at the collapse state and compared with the permissible value of plastic hinge rotation. The graph of vertical deflection Vs percentage of load is plotted for all the column removal cases. Percentage of load is found by summation of the reactions obtained at the supports for each analysis step divided by total load applied.

Fig. 6.18 shows the formation of first plastic hinge (at yield capacity of the member) and plastic hinges at collapse for case-1 column removal and corresponding displacement. Fig. 6.19 shows the graph of vertical displacement Vs. percentage of load for case-1 and corresponding plastic hinge rotations as per GSA guideline for 4-storey building.

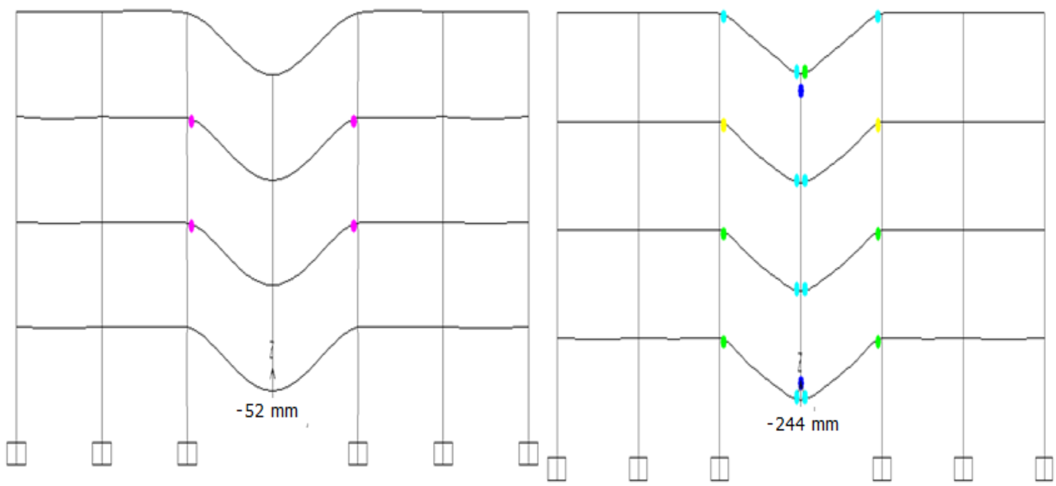


Figure 6.18: Hinges at yield and at collapse for case-1 by GSA guidelines for 4-storey

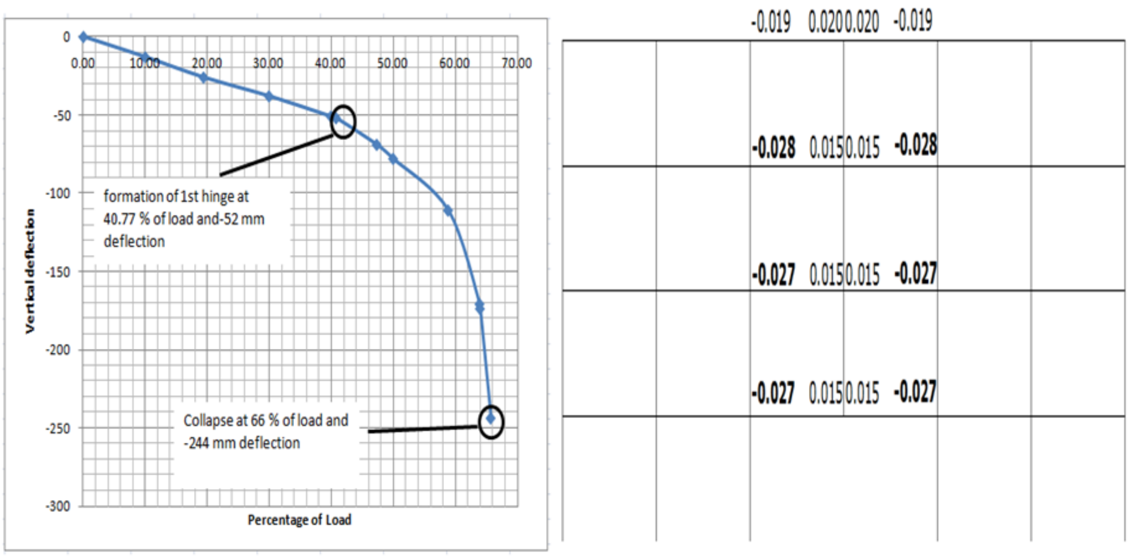


Figure 6.19: Vertical displacement Vs percentage of load for case-1 by GSA guidelines for 4-storey

Fig. 6.20 shows the formation of first plastic hinge (at yield capacity of the member) and plastic hinges at collapse for case-1 column removal and corresponding displacement. Fig. 6.21 shows the graph of vertical displacement Vs. percentage of load for case-1 and corresponding plastic hinge rotations as per DoD guideline for 4-storey building.

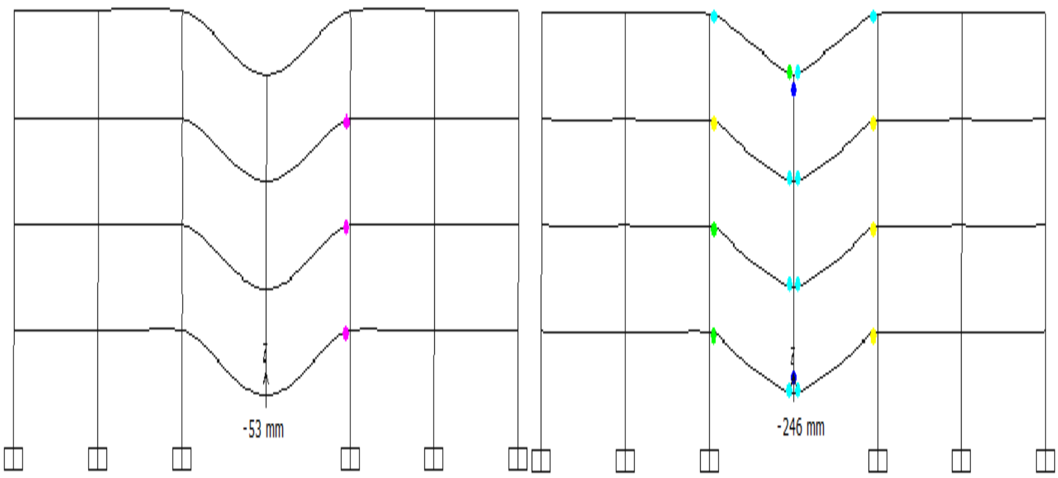


Figure 6.20: Hinges at yield and at collapse for case-1 by DoD guidelines for 4-storey

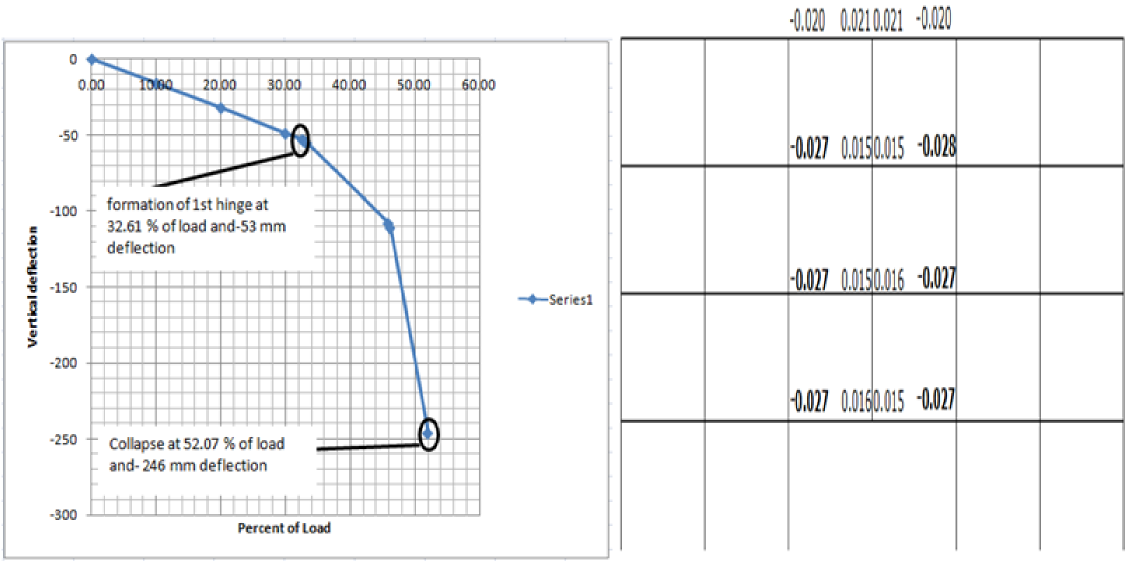


Figure 6.21: Vertical displacement Vs percentage of load for case-1 by DoD guidelines for 4-storey

Fig. 6.22 shows the formation of first plastic hinge and plastic hinges at collapse for case-1 column removal and corresponding displacement. Fig. 6.23 shows the graph of vertical displacement Vs. percentage of load for case-1 and corresponding plastic hinge rotation as per GSA guideline for 9-storey building.

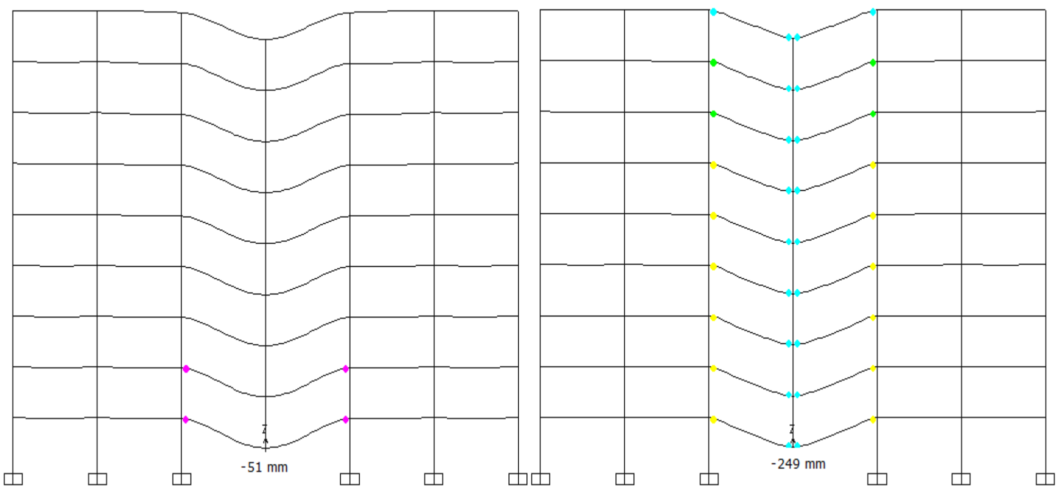


Figure 6.22: Hinges at yield and at collapse for case-1 by GSA guidelines for 9-storey

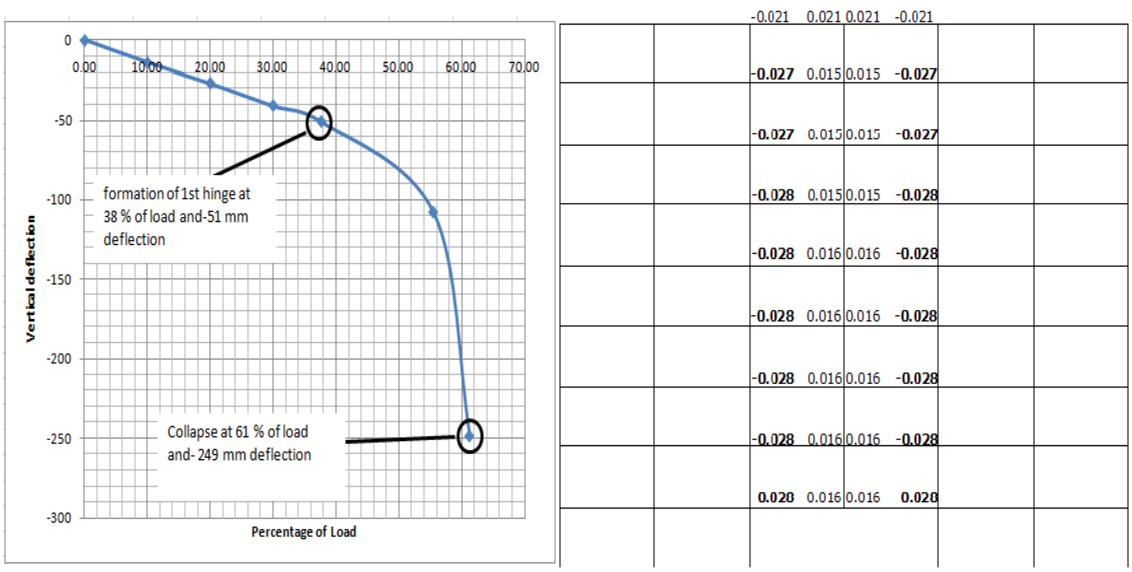


Figure 6.23: Vertical displacement vs percentage of load for case-1 by GSA guidelines for 9-storey

Fig. 6.24 shows the formation of first plastic hinge and plastic hinges at collapse for case-1 column removal and corresponding displacement. Fig. 6.25 shows the graph of vertical displacement Vs. percentage of load for case-1 and corresponding plastic hinge rotation as per DoD guideline for 9-storey building.

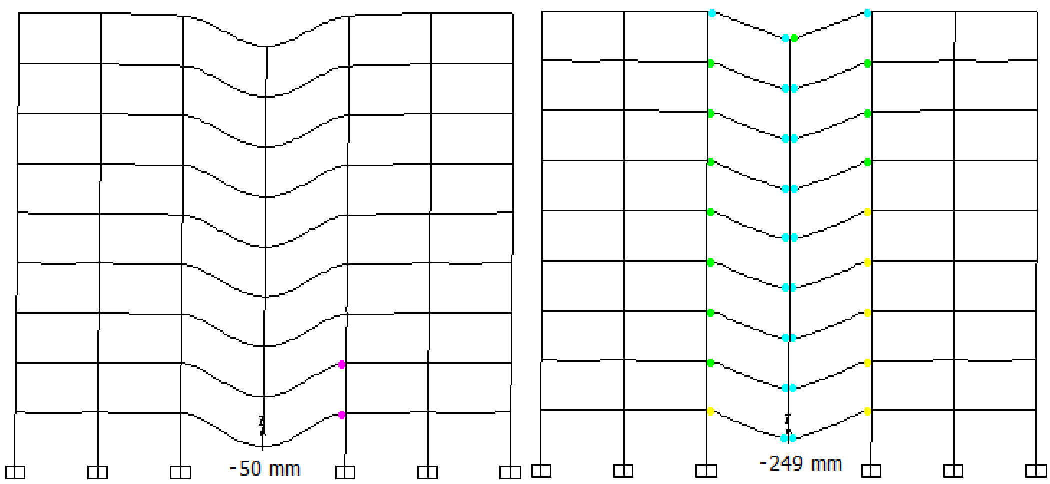


Figure 6.24: Hinges at yield and at collapse for case-1 by DoD guidelines for 9-storey

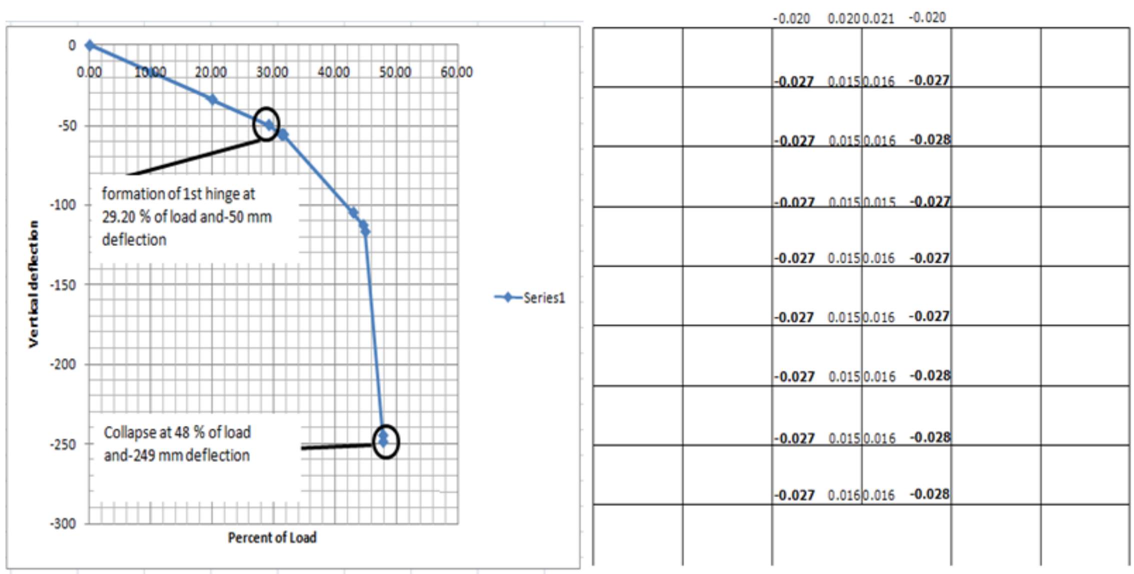


Figure 6.25: Vertical displacement vs percentage of load for case-1 by DoD guidelines for 9-storey

Fig. 6.26 shows the formation of first plastic hinge and plastic hinges at collapse for case-2 column removal and corresponding displacement. Fig. 6.27 shows the graph of vertical displacement Vs. percentage of load for case-2 and corresponding plastic hinge rotation as per GSA guideline for 4-storey building.

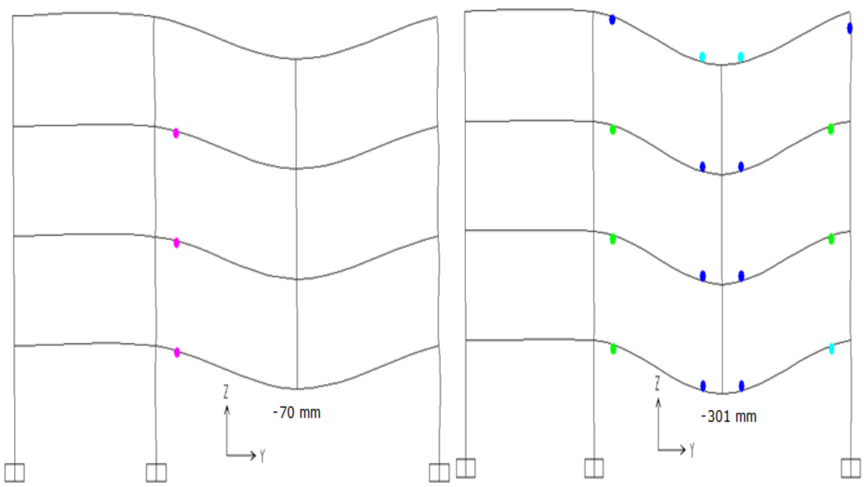


Figure 6.26: Hinges at yield and at collapse for case-2 by GSA guidelines for 4-storey

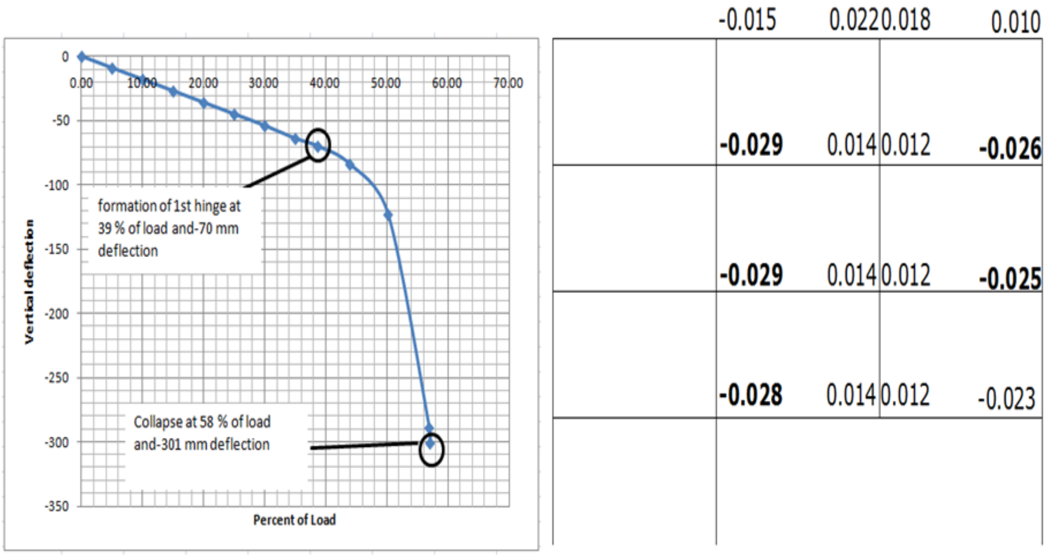


Figure 6.27: Vertical displacement vs percentage of load for case-2 by GSA guidelines for 4-storey

Fig. 6.28 shows the formation of first plastic hinge and plastic hinges at collapse for case-2 column removal and corresponding displacement. Fig. 6.29 shows the graph of vertical displacement Vs. percentage of load for case-2 and corresponding plastic hinge rotation as per DoD guideline for 4-storey building.

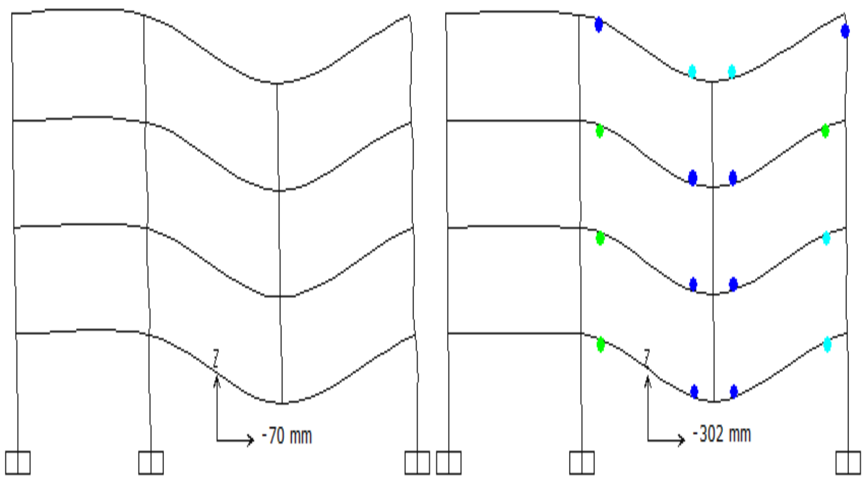


Figure 6.28: Hinges at yield and at collapse for case-2 by DoD guidelines for 4-storey

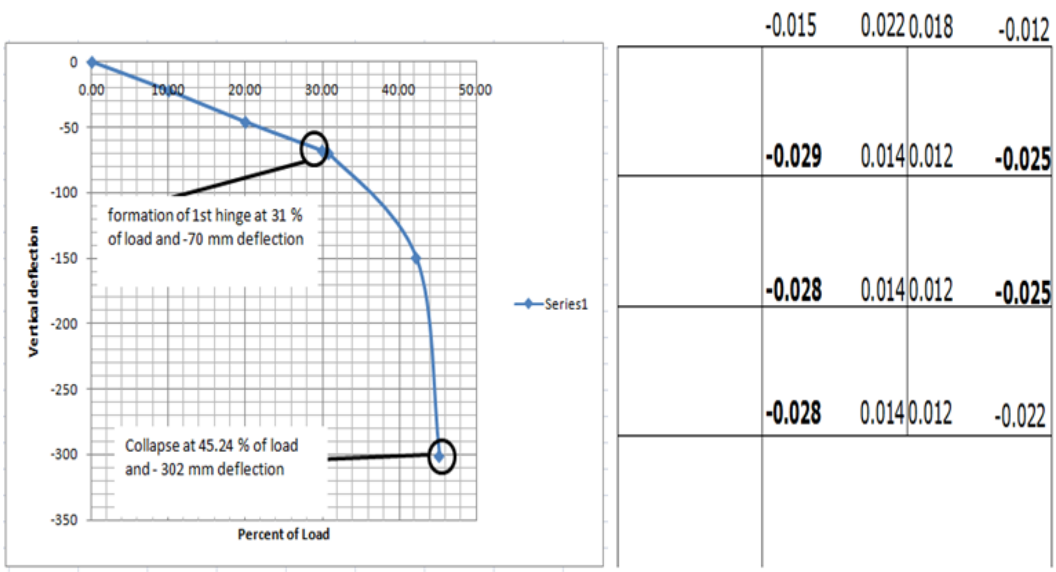


Figure 6.29: Vertical displacement vs percentage of load for case-2 by DoD guidelines for 4-storey

Fig. 6.30 shows the formation of first plastic hinge and plastic hinges at collapse for case-2 column removal and corresponding displacement. Fig. 6.31 shows the graph of vertical displacement Vs. percentage of load for case-2 and corresponding plastic hinge rotation as per GSA guideline for 9-storey building.

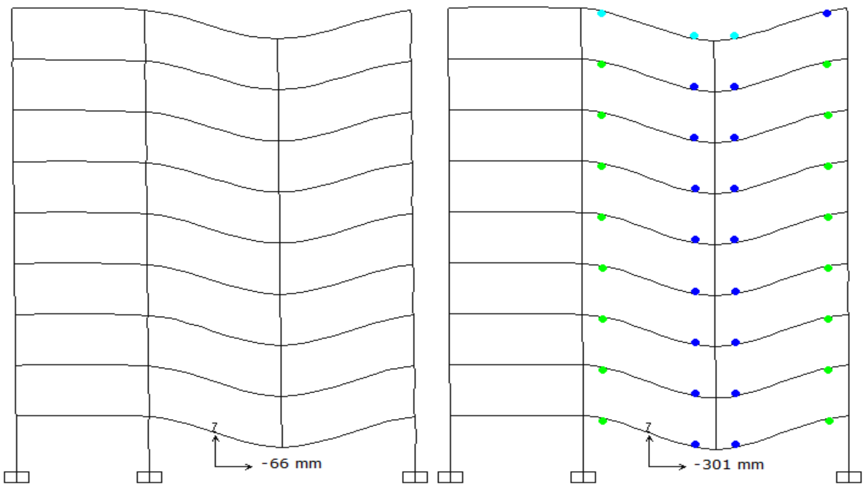


Figure 6.30: Hinges at yield and at collapse for case-2 by GSA guidelines for 9-storey

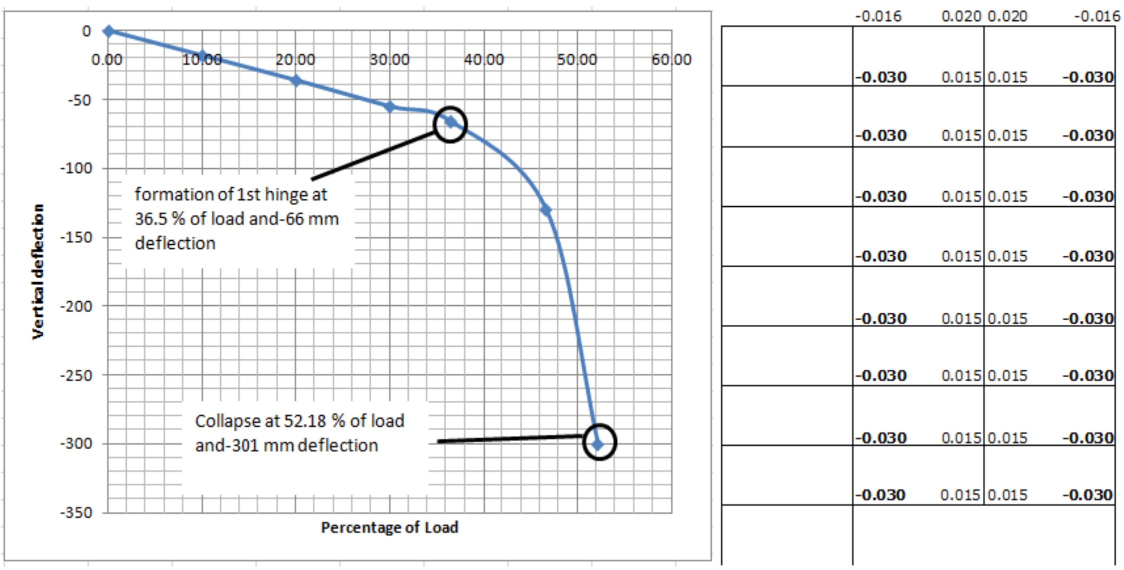


Figure 6.31: Vertical displacement vs percentage of load for case-2 by GSA guidelines for 9-storey

Fig. 6.32 shows the formation of first plastic hinge and plastic hinges at collapse for case-2 column removal and corresponding displacement. Fig. 6.33 shows the graph of vertical displacement Vs. percentage of load for case-2 and corresponding plastic hinge rotation as per DoD guideline for 9-storey building.

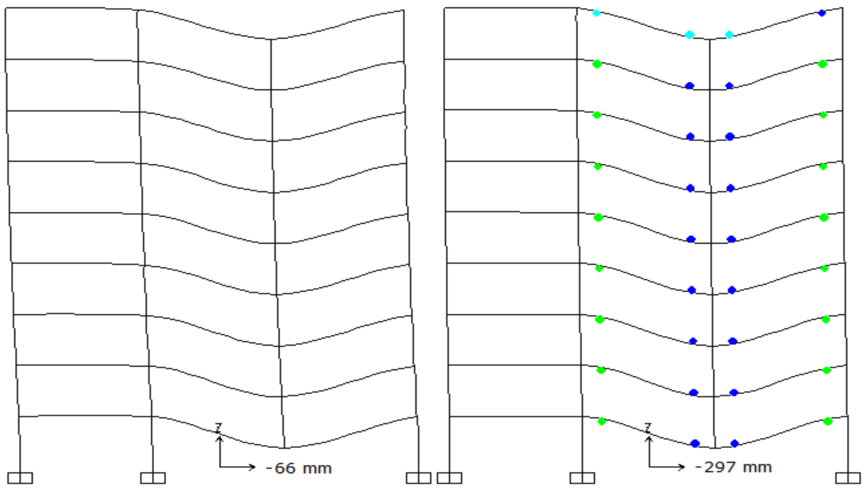


Figure 6.32: Hinges at yield and at collapse for case-2 by DoD guidelines for 9-storey

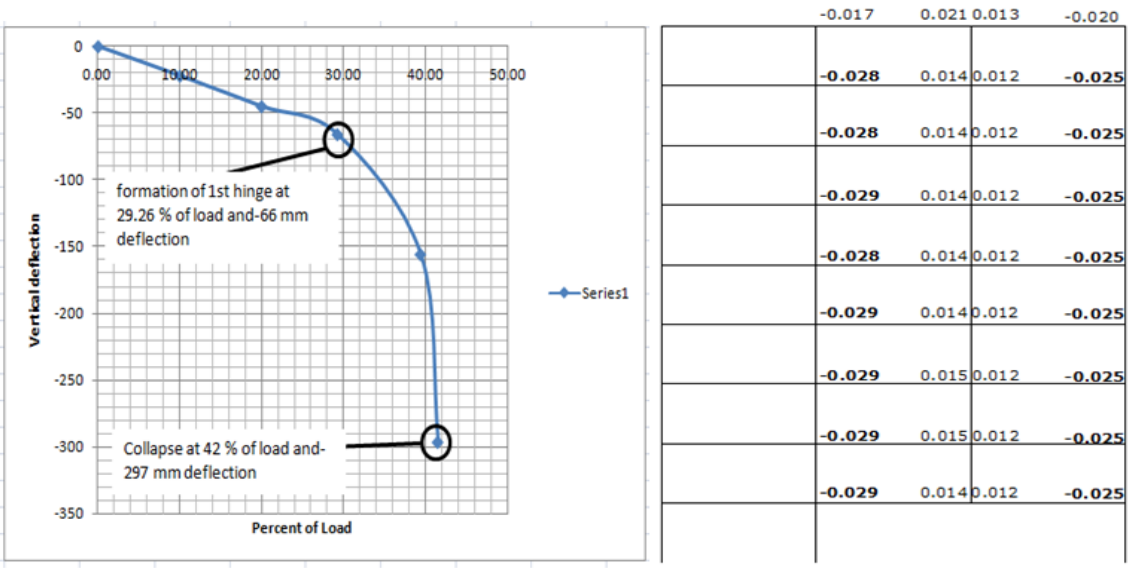


Figure 6.33: Vertical displacement vs percentage of load for case-2 by DoD guidelines for 9-storey

Fig. 6.34 shows the formation of first plastic hinge and plastic hinges at collapse for case-3 column removal and corresponding displacement. Fig. 6.35 shows the graph of vertical displacement Vs. percentage of load for case-3 and corresponding plastic hinge rotation as per GSA guideline for 4-storey building.

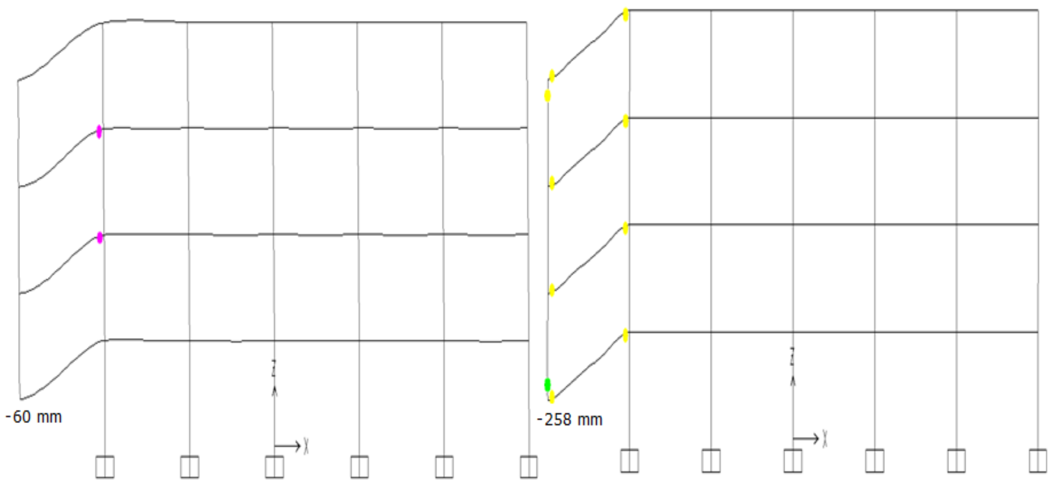


Figure 6.34: Hinges at yield and at collapse for case-3 by GSA guidelines for 4-storey

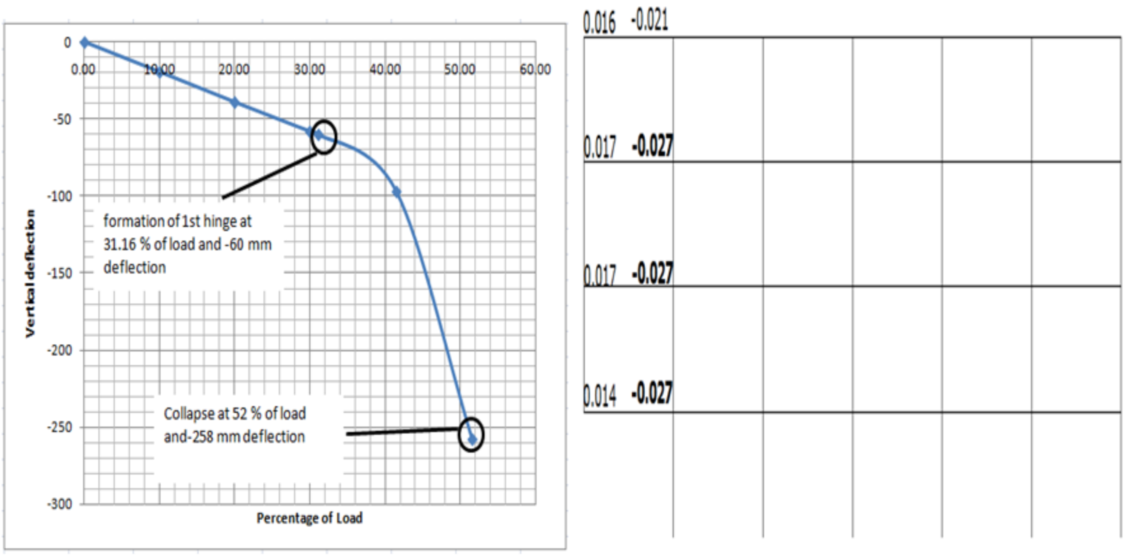


Figure 6.35: Vertical displacement vs percentage of load for case-3 by GSA guidelines for 4-storey

Fig. 6.36 shows the formation of first plastic hinge and plastic hinges at collapse for case-3 column removal and corresponding displacement. Fig. 6.37 shows the graph of vertical displacement Vs. percentage of load for case-3 and corresponding plastic hinge rotation as per DoD guideline for 4-storey building.

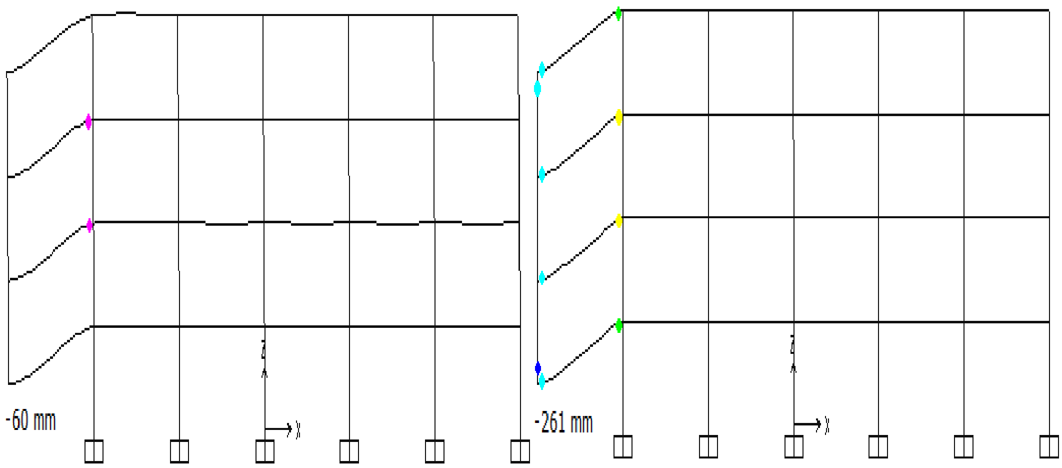


Figure 6.36: Hinges at yield and at collapse for case-3 by DoD guidelines for 4-storey

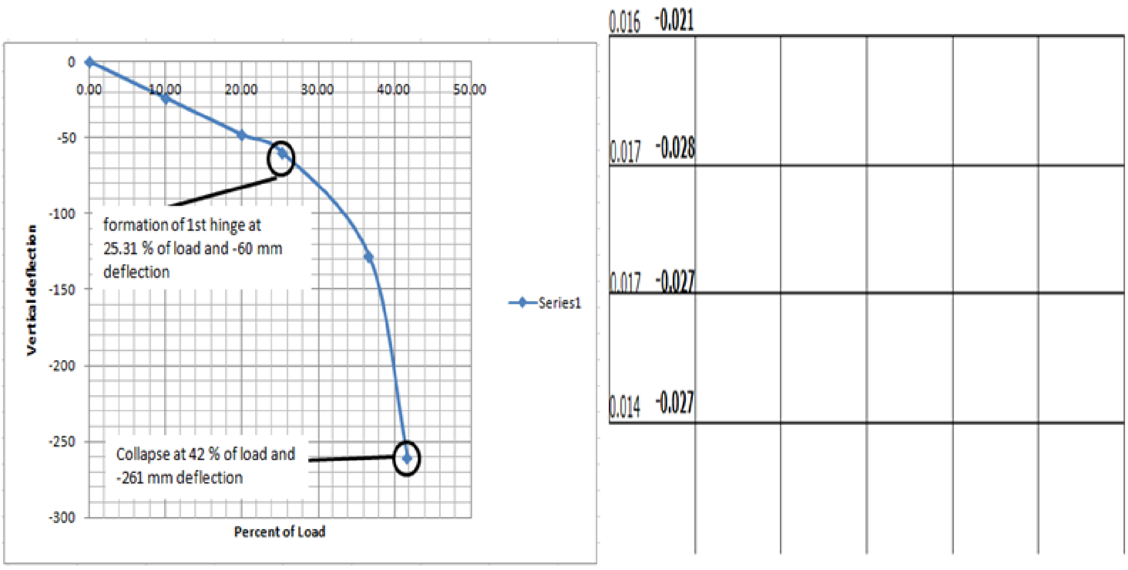


Figure 6.37: Vertical displacement vs percentage of load for case-3 by DoD guidelines for 4-storey

Fig. 6.38 shows the formation of first plastic hinge and plastic hinges at collapse for case-3 column removal and corresponding displacement. Fig. 6.39 shows the graph of vertical displacement Vs. percentage of load for case-3 and corresponding plastic hinge rotation as per GSA guideline for 9-storey building.

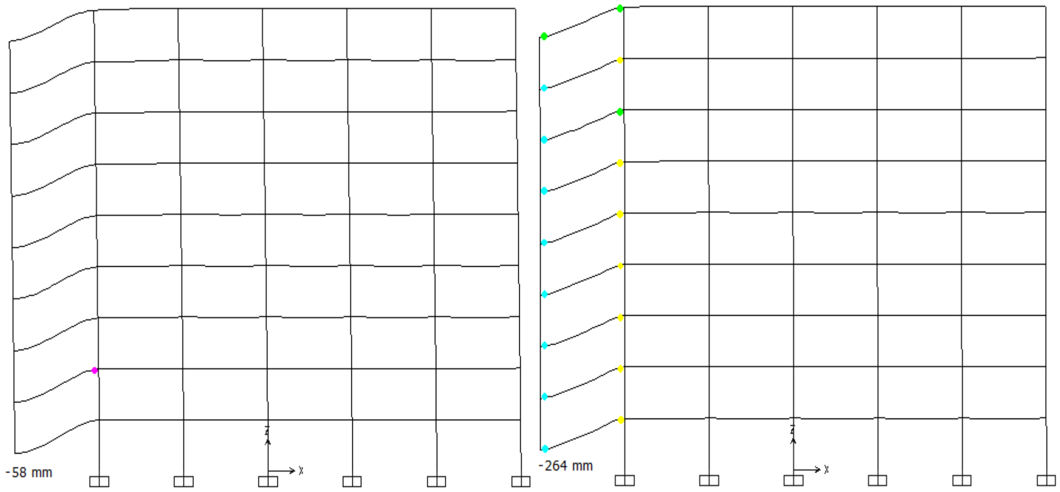


Figure 6.38: Hinges at yield and at collapse for case-3 by GSA guidelines for 9-storey

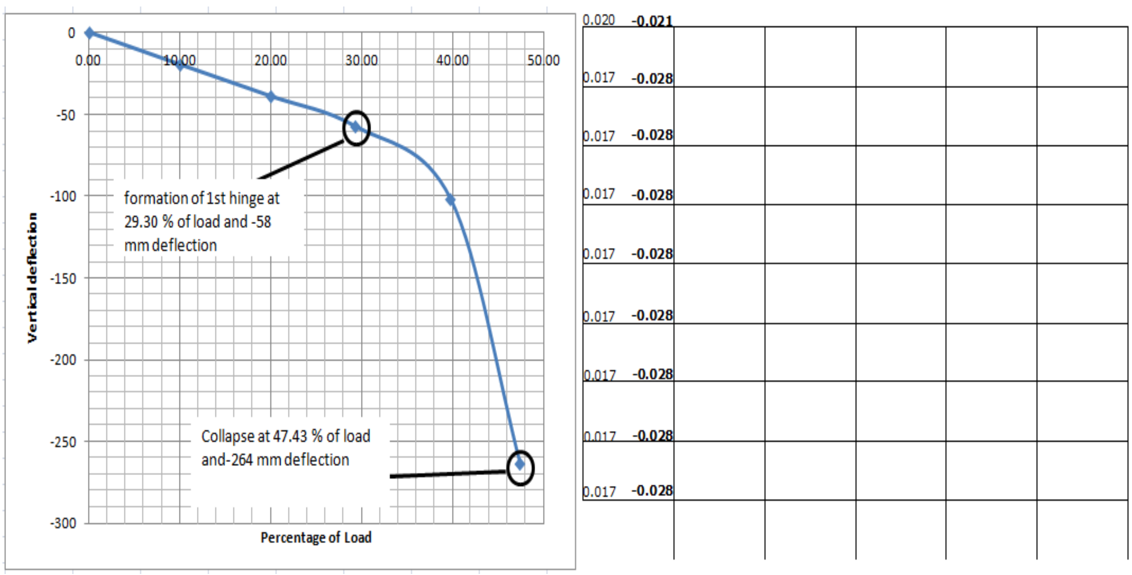


Figure 6.39: Vertical displacement vs percentage of load for case-3 by GSA guidelines for 9-storey

Fig. 6.40 shows the formation of first plastic hinge and plastic hinges at collapse for case-3 column removal and corresponding displacement. Fig. 6.41 shows the graph of vertical displacement Vs. percentage of load for case-3 and corresponding plastic hinge rotation as per DoD guideline for 9-storey building.

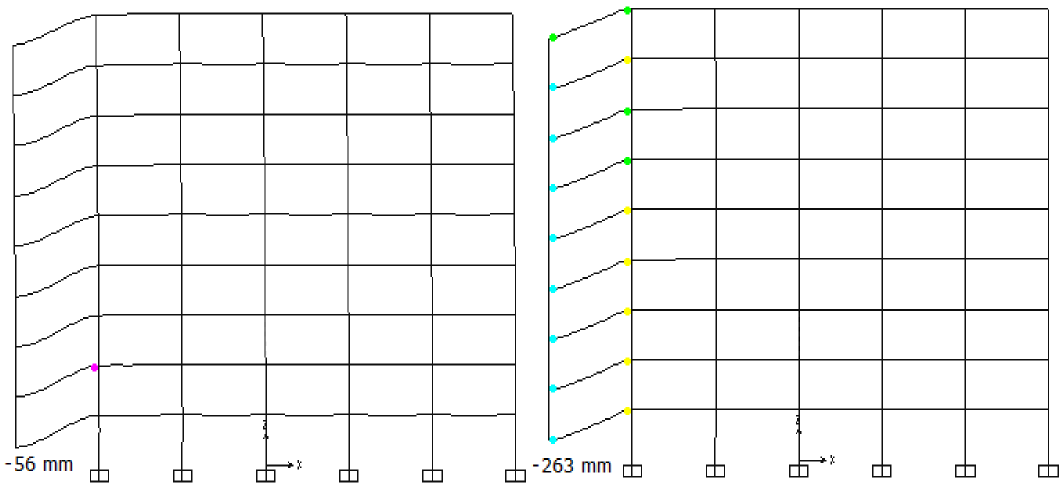


Figure 6.40: Hinges at yield and at collapse for case-3 by DoD guidelines for 9-storey

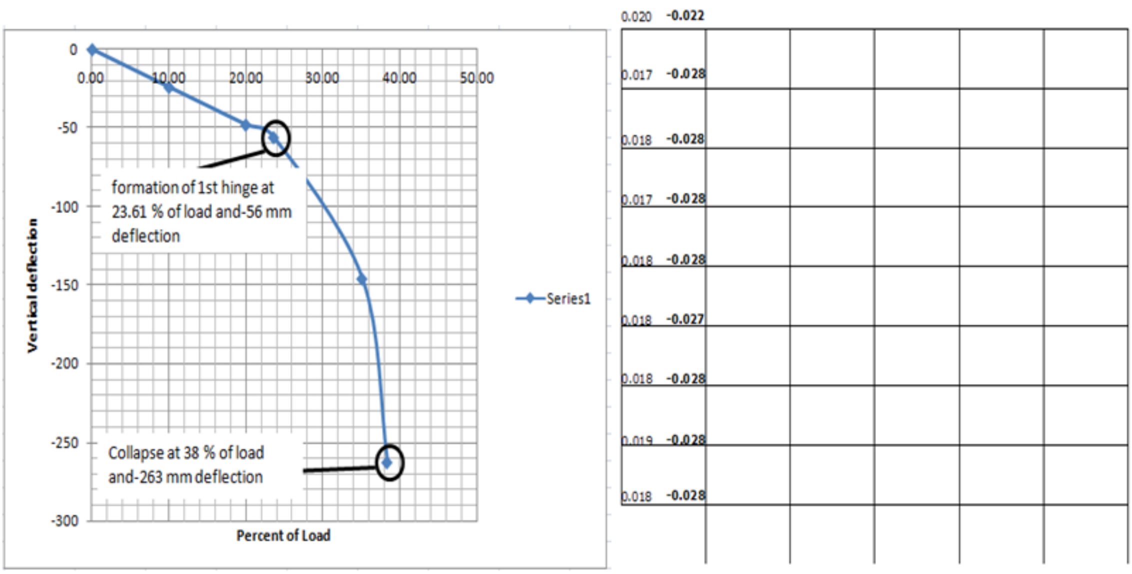


Figure 6.41: Vertical displacement vs percentage of load for case-3 by DoD guidelines for 9-storey

Fig. 6.42 shows the formation of first plastic hinge and plastic hinges at collapse for case-4 column removal and corresponding displacement. Fig. 6.43 shows the graph of vertical displacement Vs. percentage of load for case-4 and corresponding plastic hinge rotation as per GSA guideline for 4-storey building.

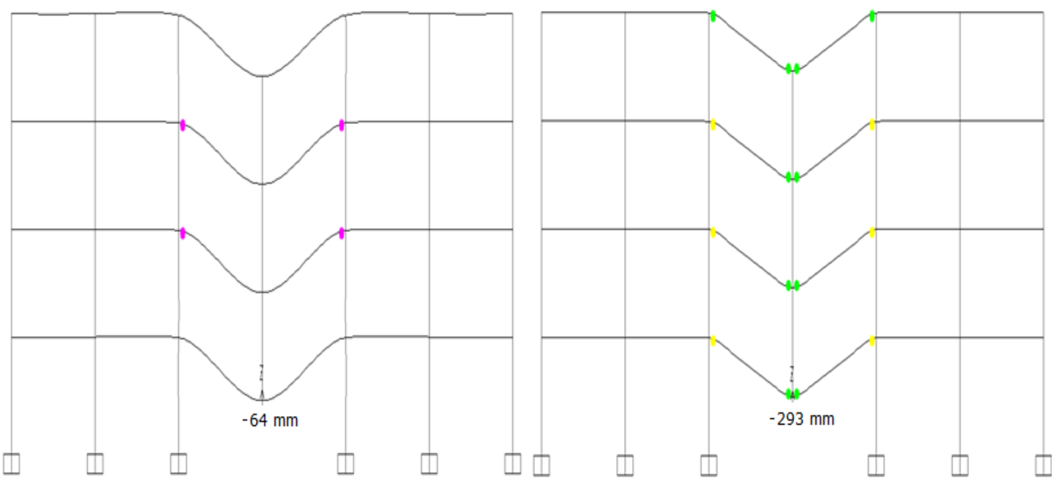


Figure 6.42: Hinges at yield and at collapse for case-4 by GSA guidelines for 4-storey

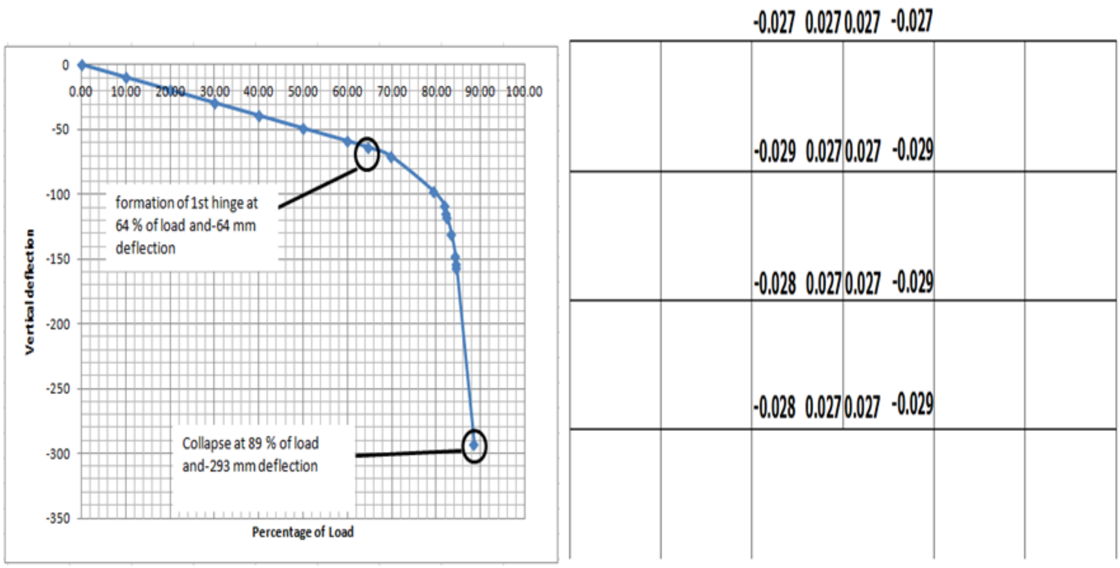


Figure 6.43: Vertical displacement vs percentage of load for case-4 by GSA guidelines for 4-storey

Fig. 6.44 shows the formation of first plastic hinge and plastic hinges at collapse for case-4 column removal and corresponding displacement. Fig. 6.45 shows the graph of vertical displacement Vs. percentage of load for case-4 and corresponding plastic hinge rotation as per DoD guideline for 4-storey building.

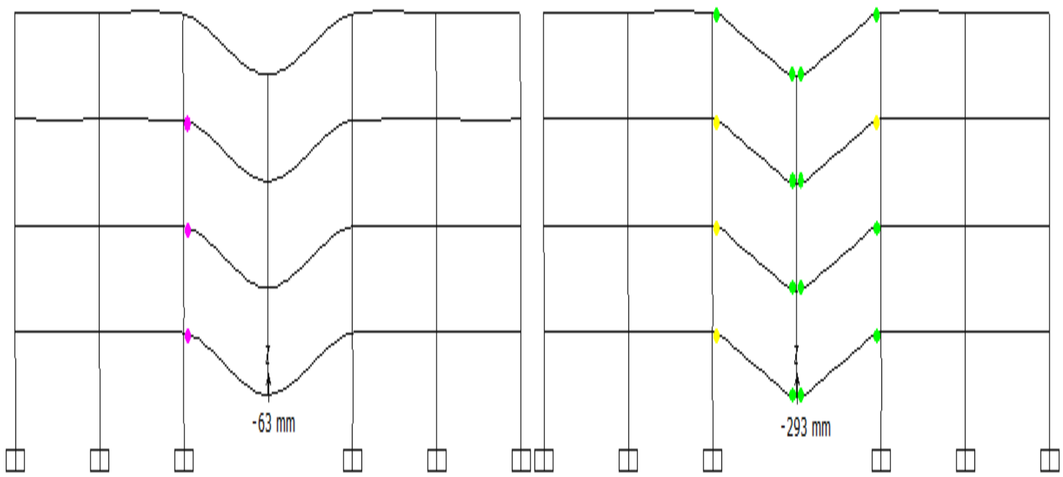


Figure 6.44: Hinges at yield and at collapse for case-4 by DoD guidelines for 4-storey

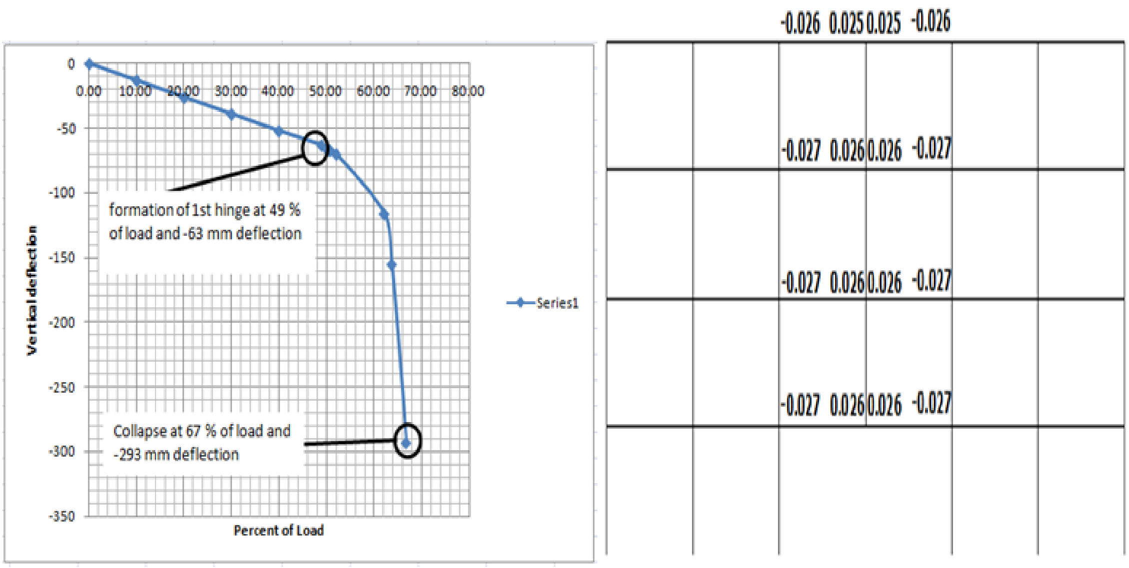


Figure 6.45: Vertical displacement vs percentage of load for case-4 by DoD guidelines for 4-storey

Fig. 6.46 shows the formation of first plastic hinge and plastic hinges at collapse for case-4 column removal and corresponding displacement. Fig. 6.47 shows the graph of vertical displacement Vs. percentage of load for case-4 and corresponding plastic hinge rotation as per GSA guideline for 9-storey building.

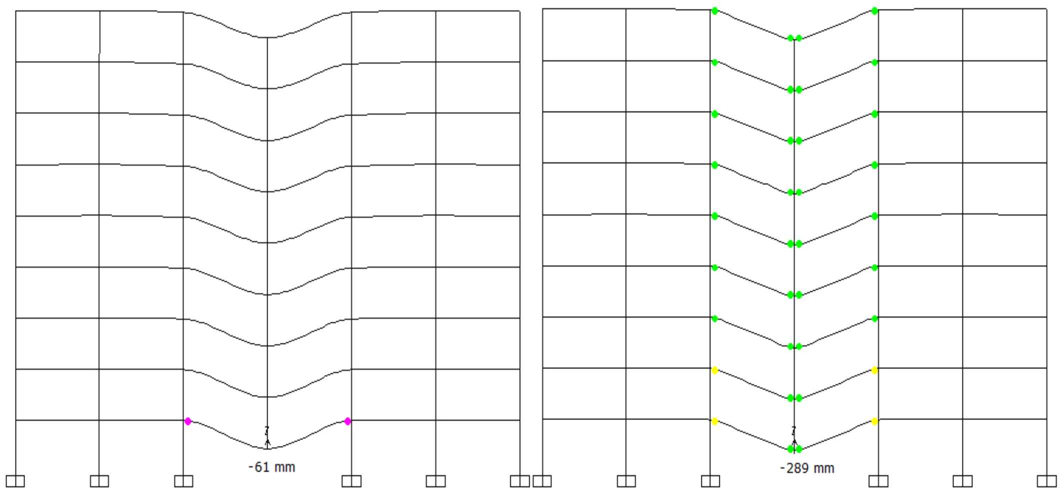


Figure 6.46: Hinges at yield and at collapse for case-4 by GSA guidelines for 9-storey

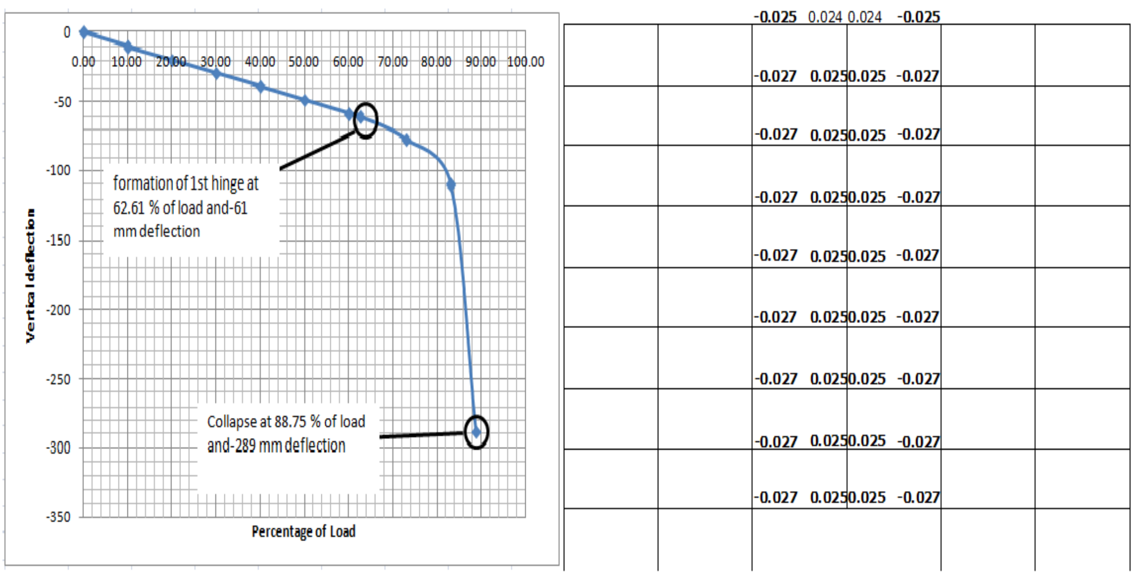


Figure 6.47: Vertical displacement vs percentage of load for case-4 by GSA guidelines for 9-storey

Fig. 6.48 shows the formation of first plastic hinge and plastic hinges at collapse for case-4 column removal and corresponding displacement. Fig. 6.49 shows the graph of vertical displacement Vs. percentage of load for case-4 and corresponding plastic hinge rotation as per DoD guideline for 9-storey building.

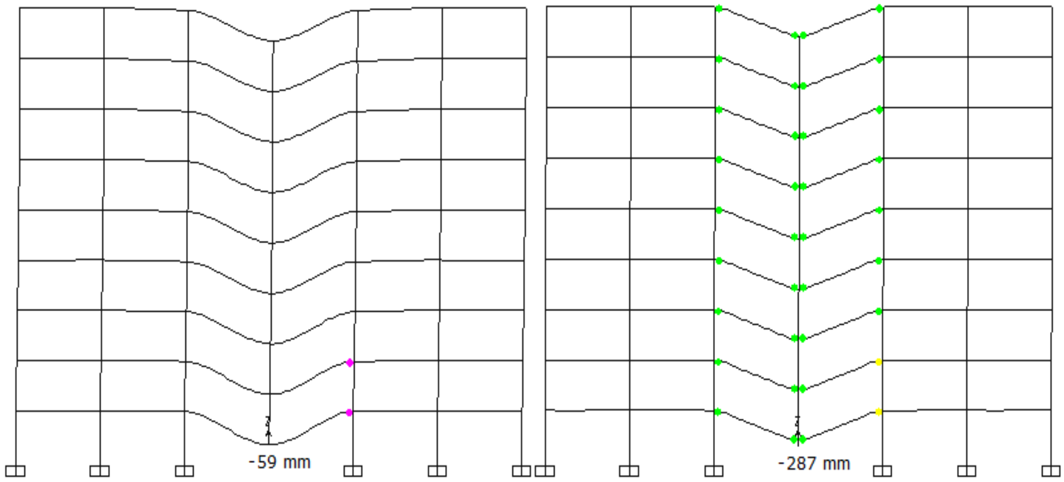


Figure 6.48: Hinges at yield and at collapse for case-4 by DoD guidelines for 9-storey

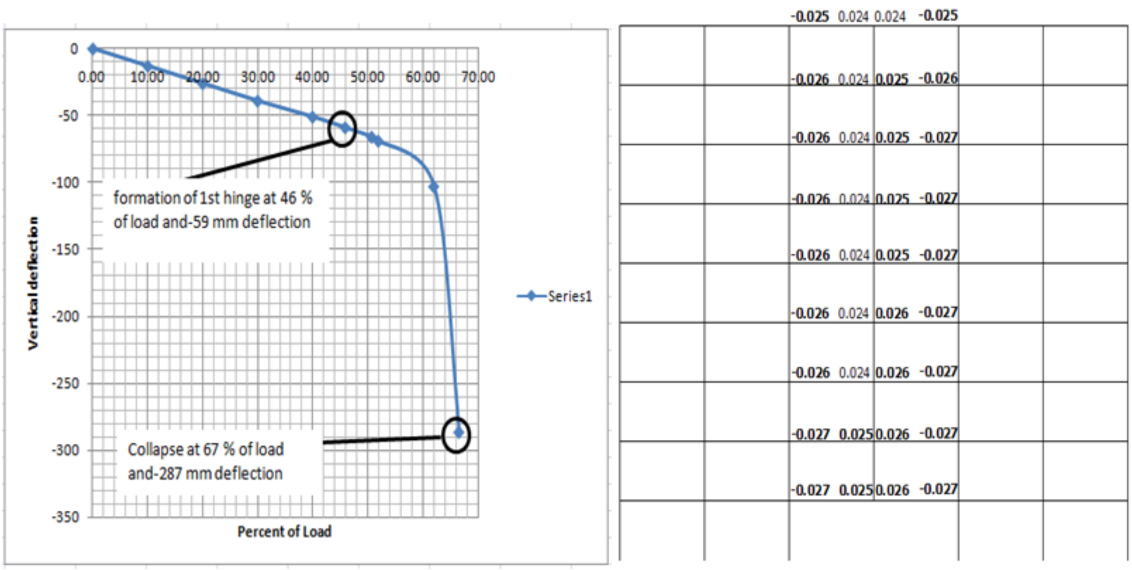


Figure 6.49: Vertical displacement vs percentage of load for case-4 by DoD guidelines for 9-storey

6.4 Nonlinear dynamic analysis procedure

The nonlinear dynamic procedure for progressive collapse analysis is the most efficient method of analysis in which a primary load-bearing structural element is removed dynamically and the structural material is allowed to undergo nonlinear behavior. This allows larger deformations and energy dissipation through material yielding, cracking, and fracture. Nonlinear dynamic analysis is performed similarly to linear dynamic analysis procedure with the exception that in this case the structural elements are allowed to enter in their inelastic range. Following types of nonlinearity are available in SAP2000:

- Material nonlinearity
  - Various type of nonlinear properties in Link/ Support elements
  - Tension and/ or compression limits in frame elements
  - Plastic hinges in Frame elements
- Geometric nonlinearity
  - P-delta effects
  - Large displacement effects

Time-history method with initial condition methodology is used to carry out nonlinear dynamic analysis. The initial conditions describe the state of the structure at the beginning of a time-history case. These include:

- Displacements and velocities
- Internal forces and stresses
- Internal state variables for nonlinear elements
- Energy values for the structure
- External loads

For nonlinear analysis, initial conditions can be specified at the start of the analysis. there are two options for specifying initial conditions:

- Zero initial conditions: the structure has zero displacement and velocity, all elements are unstressed, and there is no history of nonlinear deformation.

- Continue from a previous nonlinear analysis: the displacements, velocities, stresses, loads, energies, and nonlinear state histories from the end of a previous analysis are carried forward.

There are some restrictions when continuing from a previous nonlinear case as follows:

- Nonlinear static and nonlinear direct-integration time-history cases may be chained together in any combination, i.e., both types of analysis are compatible with each other.

- Nonlinear modal time-history (FNA) cases can only continue from other FNA cases that use modes from the same modal analysis case.

When continuing from a previous case, all applied loads specified for the present analysis case are incremental, i.e., they are added to the loads already acting at the end of the previous case. For nonlinear direct-integration analysis, it is possible to continue from a nonlinear static analysis case. This procedure is carried out as follows:

- Build a computer model

- The nonlinear dynamic procedure requires several analysis cases for each column removal. Analysis cases are created in order to determine the forces present at equilibrium in each column to be removed.

- For each column removal, the column member is deleted in the structural model and the internal forces determined from the equilibrium model are applied to the structure as a load case to the joint or joints at each column end. These static nonlinear analysis cases are used as the starting conditions for the column removals.

- Within these analysis cases assign all loads to be used in this analysis case as per the load combinations defined in GSA and DoD guidelines.

- Click Nonlinear parameters button and choose P-delta option. It is possible to use P-delta + large displacements, but it is not necessarily needed for this analysis.

- After equilibrium is reached for the structure, remove the column by ramping down the column forces under a duration for removal of less than one tenth of the period associated with the structural response mode for the element removal.

- The analysis shall continue until the maximum displacement is reached or one cycle of vertical motion occurs at the column or wall section removal location.

Fig. 6.50 shows the interface for definition of analysis cases and their assigned loads. Fig. 6.51 shows the time-history function definition for removal of column. Fig. 6.52 shows the column removal analysis case definition.

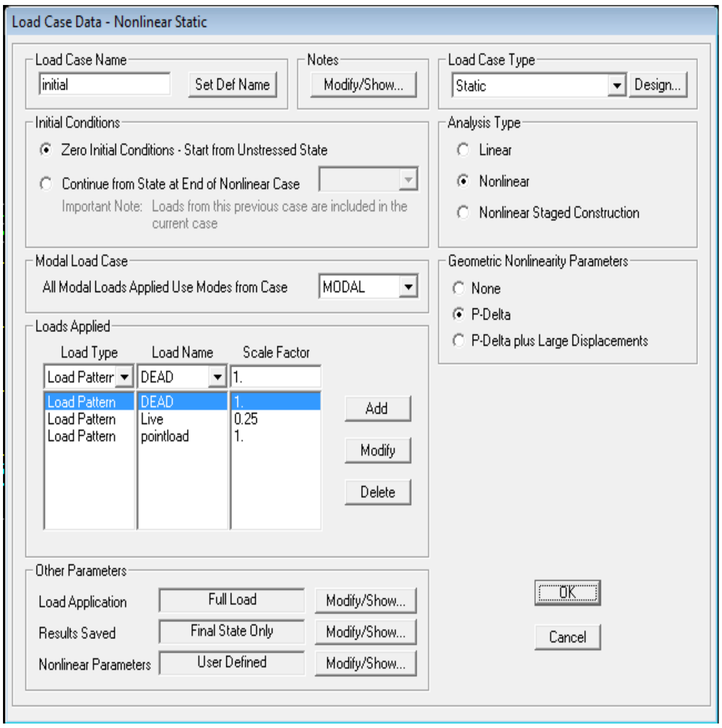


Figure 6.50: Initial analysis case definition as per GSA guideline

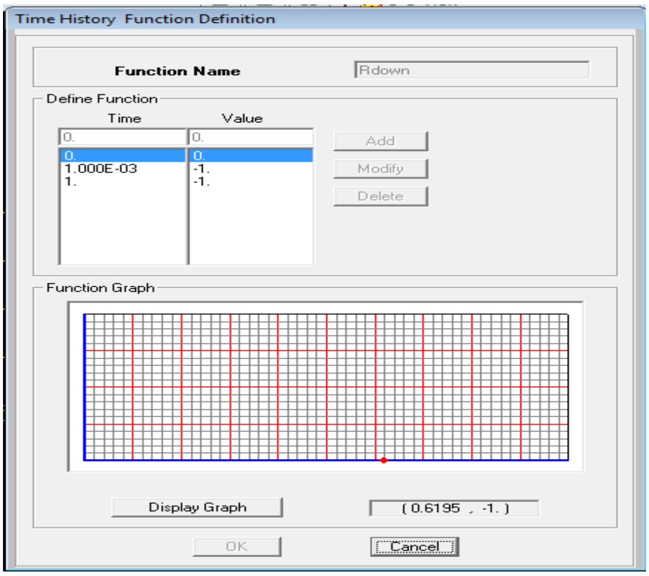


Figure 6.51: Time-history function definition for removal of column

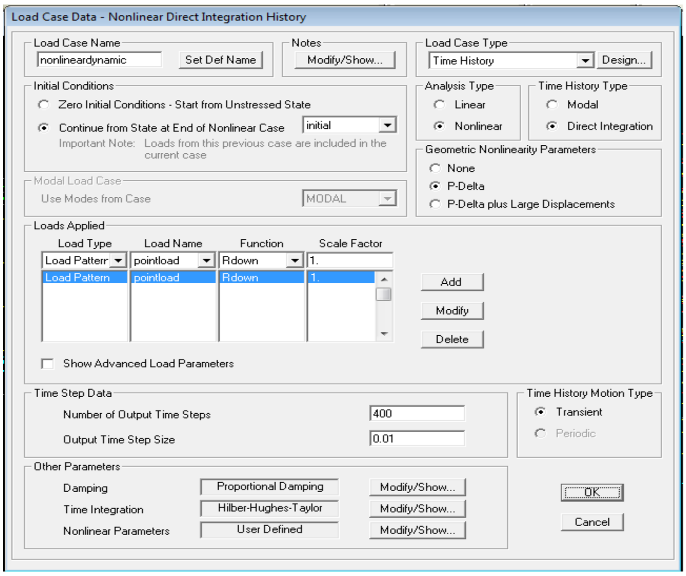


Figure 6.52: Column removal analysis case definition as per GSA guideline

Fig. 6.53 shows the displacement under the column removal location for case-1 as per GSA guideline and corresponding plastic hinge rotations at collapse. Fig. 6.54 shows displacement and plastic hinge rotations as per DoD guidelines for 4-storey building.

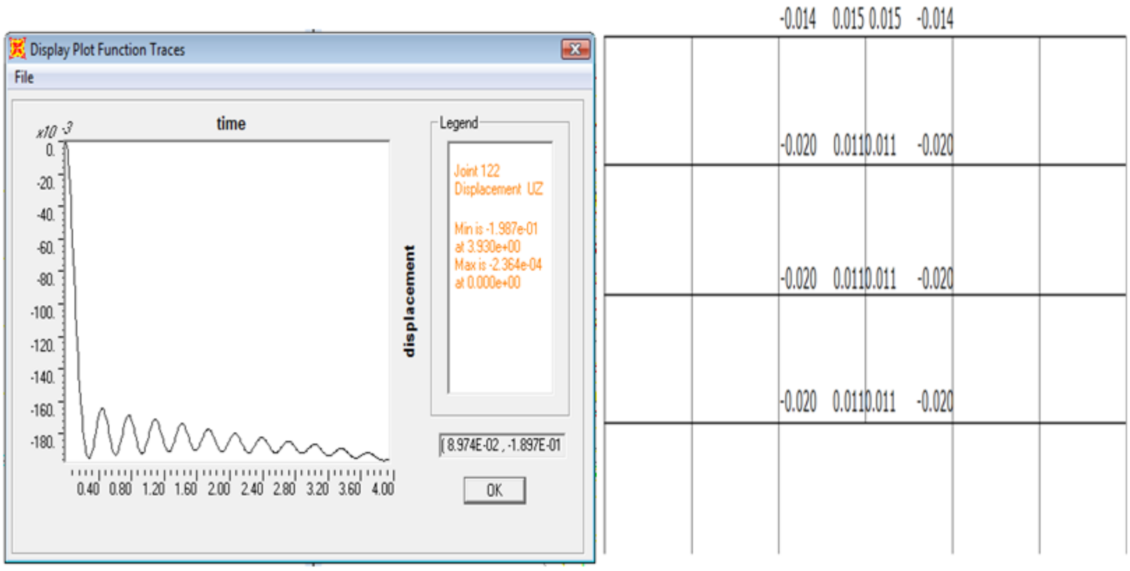


Figure 6.53: Displacement and hinge rotations for case-1 by GSA guideline for 4-storey

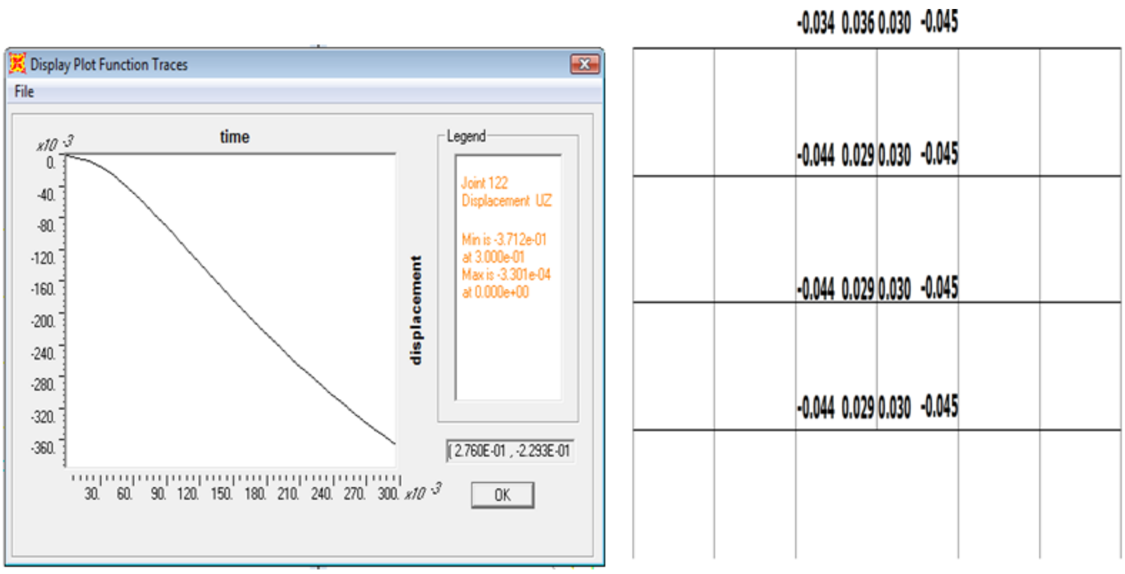


Figure 6.54: Displacement and hinge rotations for case-1 by DoD guideline for 4-storey

Summary of nonlinear static analysis and nonlinear dynamic analysis for case-1 column removal is given in tabular form. Table 6.3 shows the summary for GSA and DoD guidelines. Table shows the maximum collapse load as per nonlinear static analysis, displacement ductility as per nonlinear dynamic analysis, maximum support rotations and plastic hinge rotations. Collapse load is found out from nonlinear static analysis. Displacement ductility is found by taking the ratio of ultimate displacement to yield displacement. For example Fig. 6.53 shows the maximum displacement is 198 mm and yield displacement (when the first plastic hinge occurs) is 60 mm. Ratio of both the values give displacement ductility. Fig. 6.54 shows that nonlinear dynamic analysis could not successfully completed. The reason is that the model has a plastic hinge that failed or mechanism has formed. Fig. 6.54 shows the maximum displacement just before the failure. Maximum support rotation is found out by taking the ratio of maximum displacement to the length of the adjacent beam from where the column is removed. Plastic hinge rotation is found out from the software.

Table 6.3: Summary for case-1 by GSA and DoD guidelines for 4-storey

	GSA guidelines	DoD guidelines
Nonlinear static analysis	Collapse at 66% of load	Collapse at 52% of load
Displacement ductility	(198 mm/60 mm)= 3.3	(371 mm/47 mm)= 7.89
Permissible ductility	20	20
Maximum support rotation	0.024 radians	0.045 radians
Plastic hinge rotation	0.020 radians	0.044 radians

Fig. 6.55 shows the displacement under the column removal location for case-1 as per GSA guideline and corresponding plastic hinge rotations at collapse. Fig. 6.56 shows displacement and plastic hinge rotations as per DoD guidelines for 9-storey building.

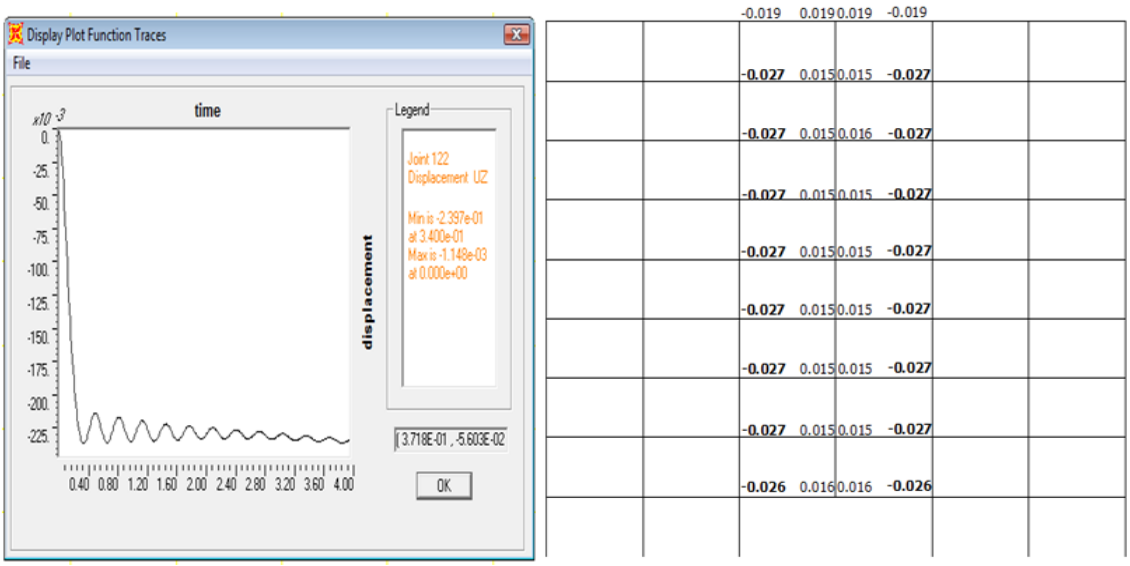


Figure 6.55: Displacement and hinge rotations for case-1 by GSA guideline for 9-storey

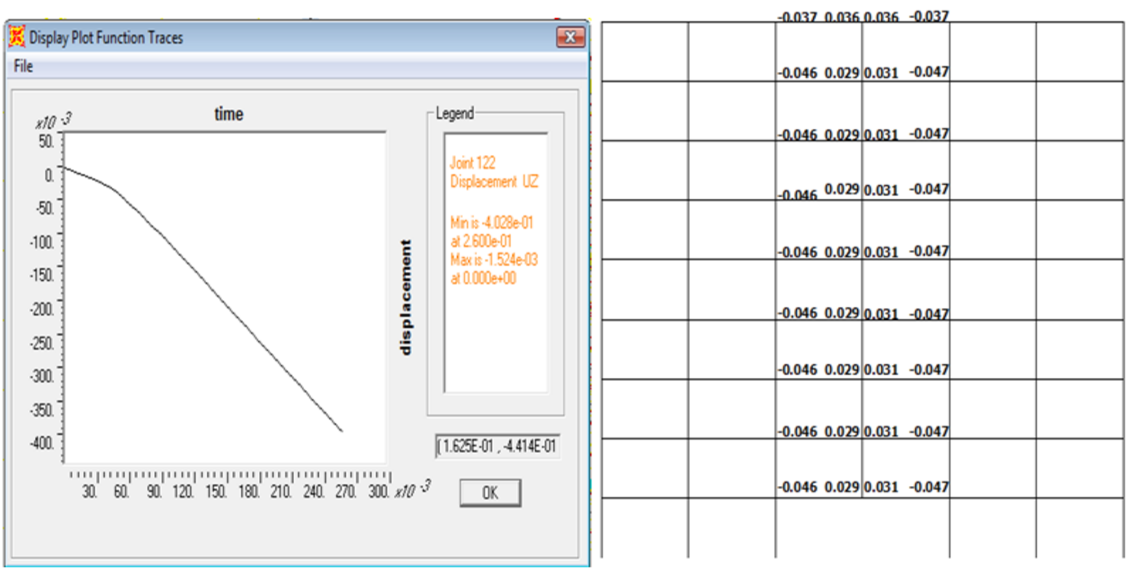


Figure 6.56: Displacement and hinge rotations for case-1 by DoD guideline for 9-storey

Summary of nonlinear static analysis and nonlinear dynamic analysis for case-1 column removal is given in tabular form. Table 6.4 shows the summary for GSA and DoD guidelines. Table shows maximum collapse load as per nonlinear static analysis, displacement ductility as per nonlinear dynamic analysis, maximum support rotations and plastic hinge rotations. From Fig. 6.55 it is seen that maximum displacement is 237 mm and maximum displacement for Fig. 6.56 is 402 mm.

Table 6.4: Summary for case-1 by GSA and DoD guidelines for 9-storey

	GSA guidelines	DoD guidelines
Nonlinear static analysis	Collapse at 61% of load	Collapse at 48% of load
Displacement ductility	(237 mm/55 mm)= 4.31	(402 mm/50 mm)= 8.04
Permissible ductility	20	20
Maximum support rotation	0.029 radians	0.049 radians
Plastic hinge rotation	0.027 radians	0.047 radians

Fig. 6.57 shows the displacement under the column removal location for case-2 as per GSA guideline and corresponding plastic hinge rotations at collapse. Fig. 6.58 shows displacement and plastic hinge rotations as per DoD guidelines for 4-storey building.

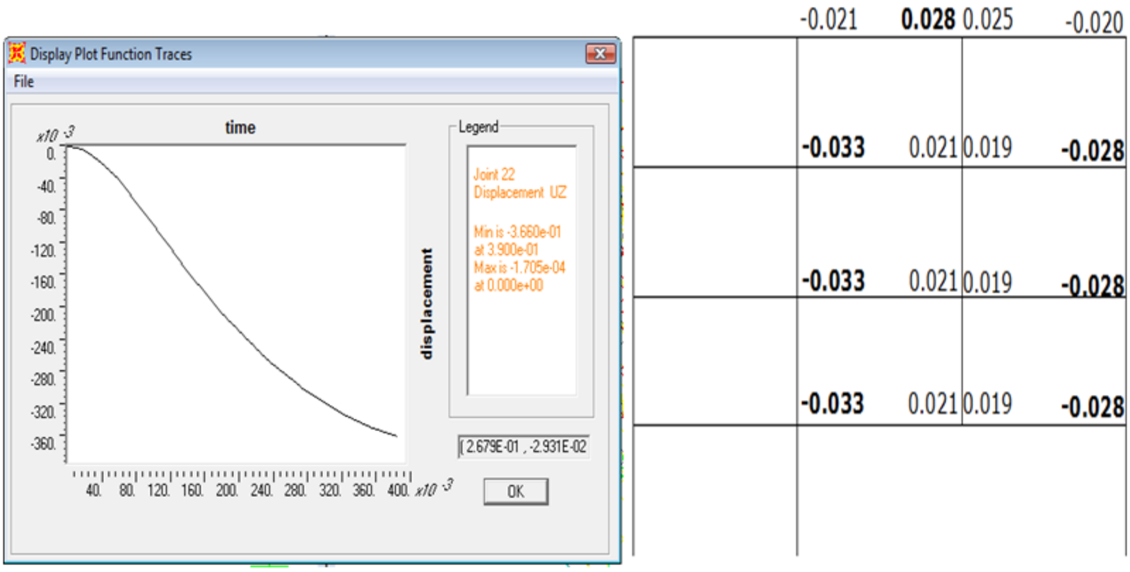


Figure 6.57: Displacement and hinge rotations for case-2 by GSA guideline for 4-storey

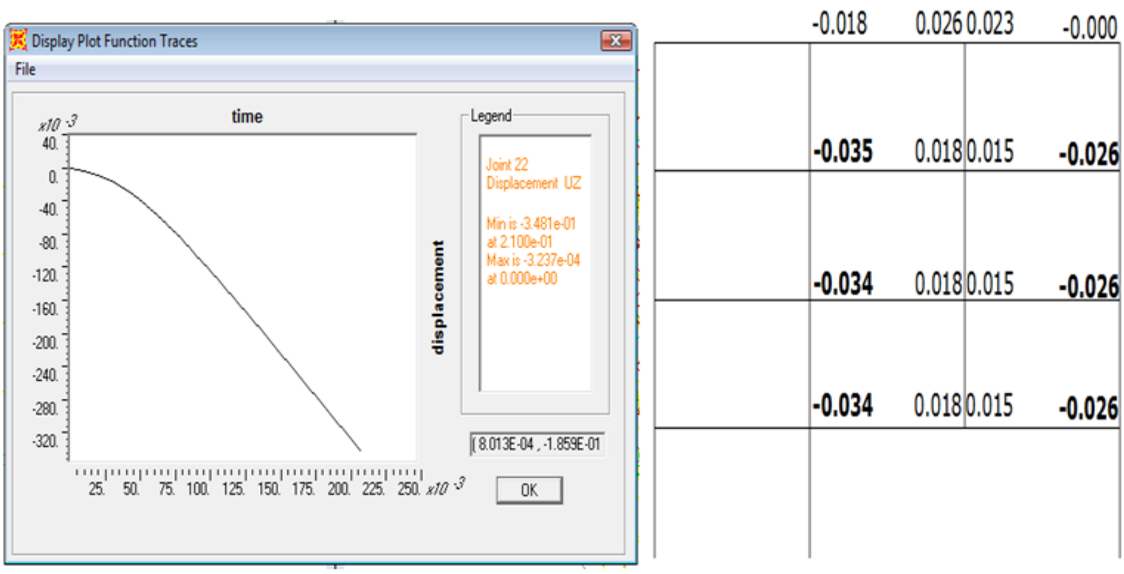


Figure 6.58: Displacement and hinge rotations for case-2 by DoD guideline for 4-storey

Summary of nonlinear static analysis and nonlinear dynamic analysis for case-2 column removal is given in tabular form. Table 6.5 shows the summary for GSA and DoD guidelines. Table shows the maximum collapse load as per nonlinear static analysis, displacement ductility as per nonlinear dynamic analysis, maximum support rotations and plastic hinge rotations. From Fig. 6.57 it is seen that maximum displacement is 345 mm and maximum displacement for Fig. 6.58 is 348 mm.

Table 6.5: Summary for case-2 by GSA and DoD guidelines for 4-storey

	GSA guidelines	DoD guidelines
Nonlinear static analysis	Collapse at 58% of load	Collapse at 45% of load
Displacement ductility	(345 mm/83 mm)= 4.16	(348 mm/82 mm)= 4.24
Permissible ductility	20	20
Maximum support rotation	0.035 radians	0.036 radians
Plastic hinge rotation	0.033 radians	0.034 radians

Fig. 6.59 shows the displacement under the column removal location for case-2 as per GSA guideline and corresponding plastic hinge rotations at collapse. Fig. 6.60 shows displacement and plastic hinge rotations as per DoD guidelines for 9-storey building.

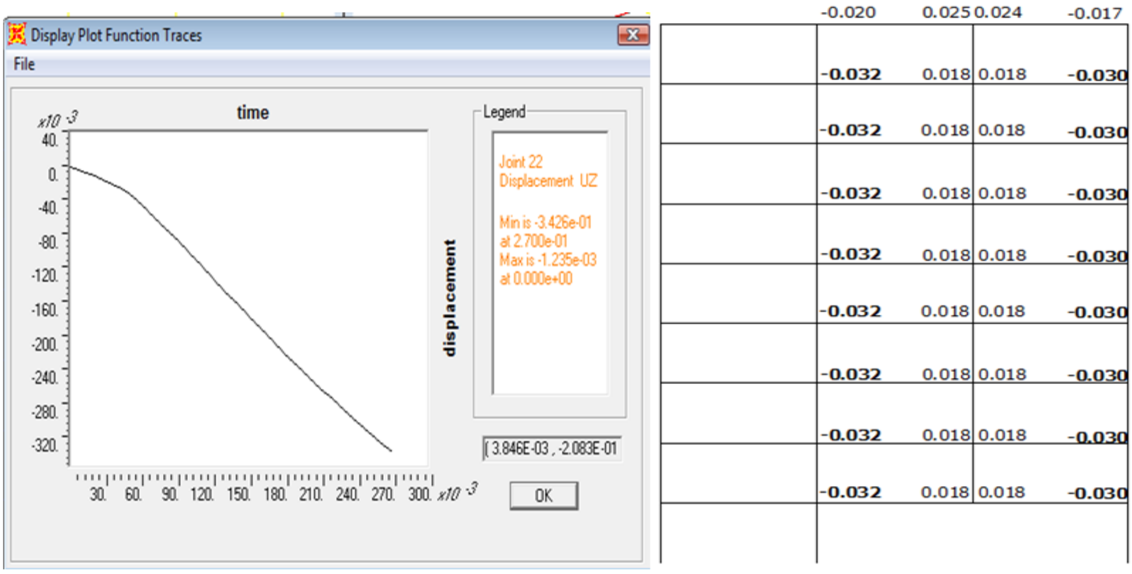


Figure 6.59: Displacement and hinge rotations for case-2 by GSA guideline for 9-storey

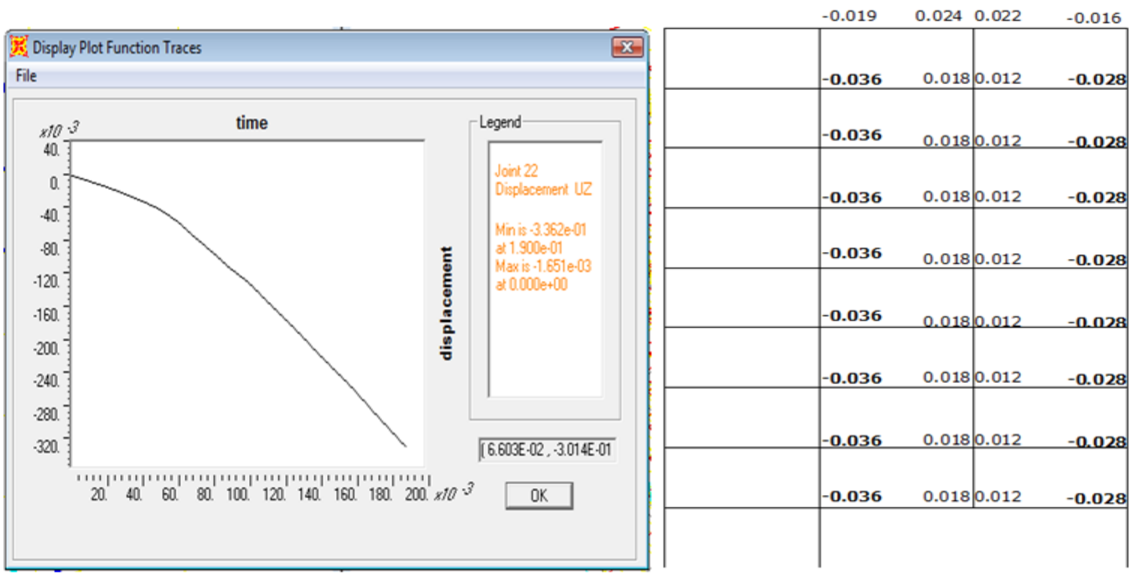


Figure 6.60: Displacement and hinge rotations for case-2 by DoD guideline for 9-storey

Summary of nonlinear static analysis and nonlinear dynamic analysis for case-2 column removal is given in tabular form. Table 6.6 shows the summary for GSA and DoD guidelines. Table shows the maximum collapse load as per nonlinear static analysis, displacement ductility as per nonlinear dynamic analysis, maximum support rotations and plastic hinge rotations. From Fig. 6.59 it is seen that maximum displacement is 342 mm and maximum displacement for Fig. 6.60 is 336 mm.

Table 6.6: Summary for case-2 by GSA and DoD guidelines for 9-storey

	GSA guidelines	DoD guidelines
Nonlinear static analysis	Collapse at 52% of load	Collapse at 42% of load
Displacement ductility	(342 mm/62 mm)= 5.52	(336 mm/59 mm)= 5.69
Permissible ductility	20	20
Maximum support rotation	0.039 radians	0.039 radians
Plastic hinge rotation	0.032 radians	0.036 radians

Fig. 6.61 shows the displacement under the column removal location for case-3 as per GSA guideline and corresponding plastic hinge rotations at collapse. Fig. 6.62 shows displacement and plastic hinge rotations as per DoD guidelines for 4-storey building.

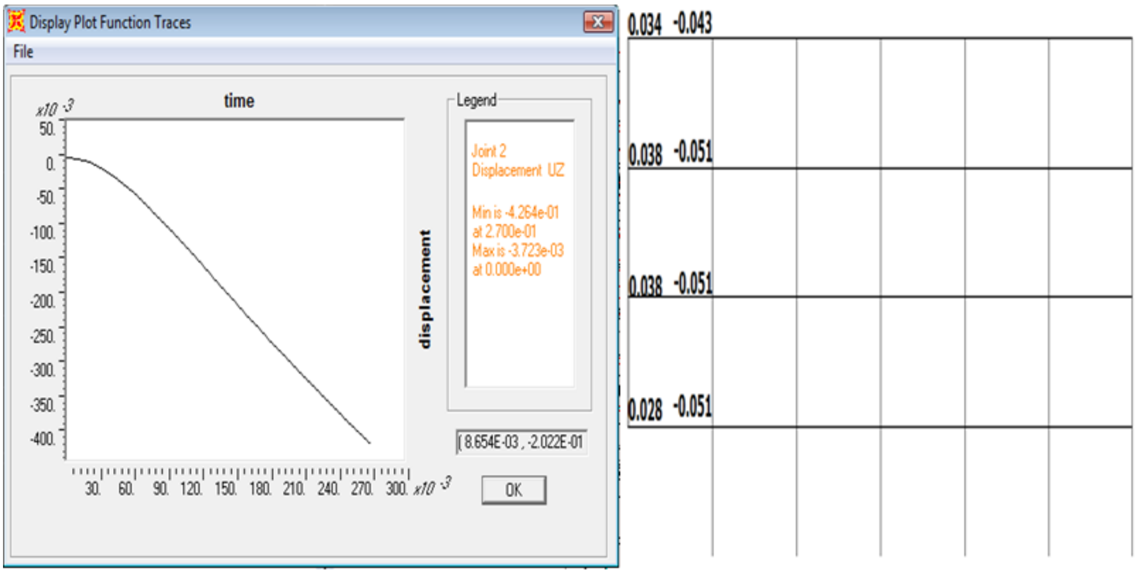


Figure 6.61: Displacement and hinge rotations for case-3 by GSA guideline for 4-storey

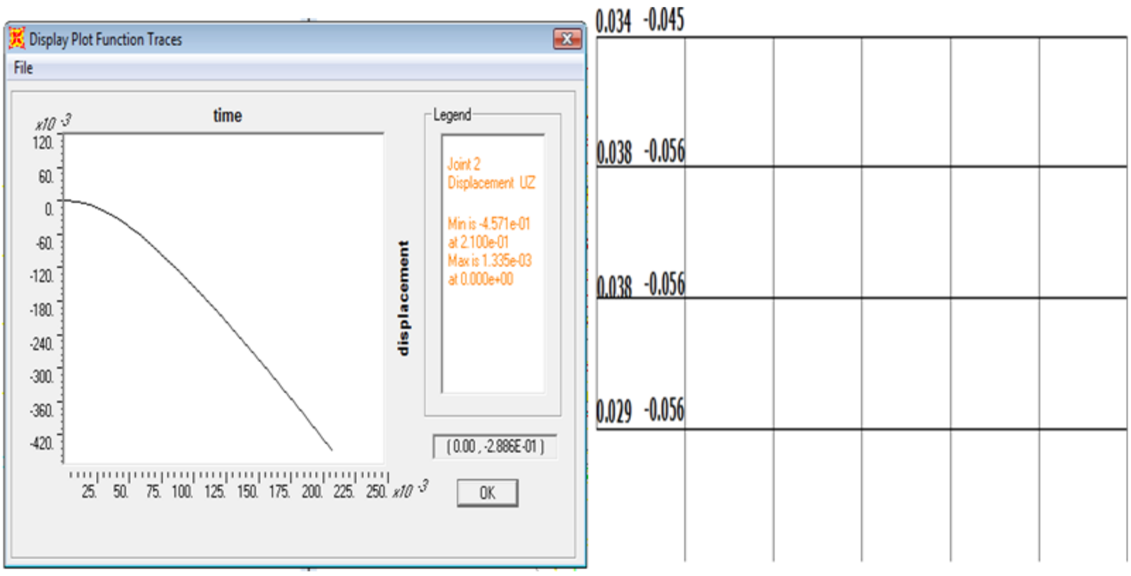


Figure 6.62: Displacement and hinge rotations for case-3 by DoD guideline for 4-storey

Summary of nonlinear static analysis and nonlinear dynamic analysis for case-3 column removal is given in tabular form. Table 6.7 shows the summary for GSA and DoD guidelines. Table shows the maximum collapse load as per nonlinear static analysis, displacement ductility as per nonlinear dynamic analysis, maximum support rotations and plastic hinge rotations. From Fig. 6.61 it is seen that maximum displacement is 426 mm and maximum displacement for Fig. 6.62 is 457 mm.

Table 6.7: Summary for case-3 by GSA and DoD guidelines for 4-storey

	GSA guidelines	DoD guidelines
Nonlinear static analysis	Collapse at 52% of load	Collapse at 42% of load
Displacement ductility	(426 mm/56 mm)= 7.61	(457 mm/46 mm)= 9.93
Permissible ductility	20	20
Maximum support rotation	0.051 radians	0.055 radians
Plastic hinge rotation	0.051 radians	0.055 radians

Fig. 6.63 shows the displacement under the column removal location for case-3 as per GSA guideline and corresponding plastic hinge rotations at collapse. Fig. 6.64 shows displacement and plastic hinge rotations as per DoD guidelines for 9-storey building.

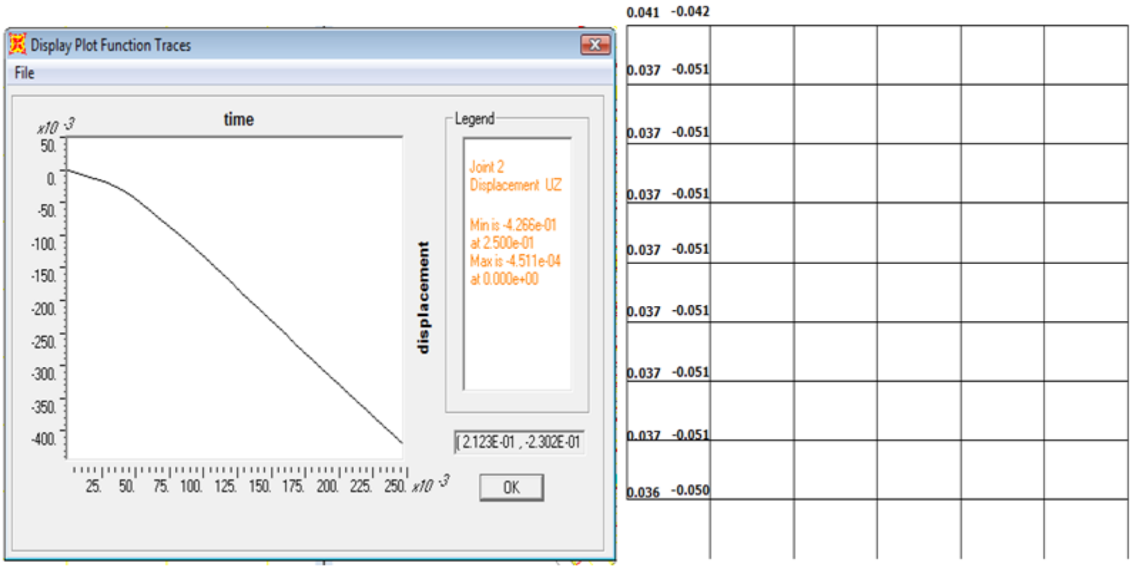


Figure 6.63: Displacement and hinge rotations for case-3 by GSA guideline for 9-storey

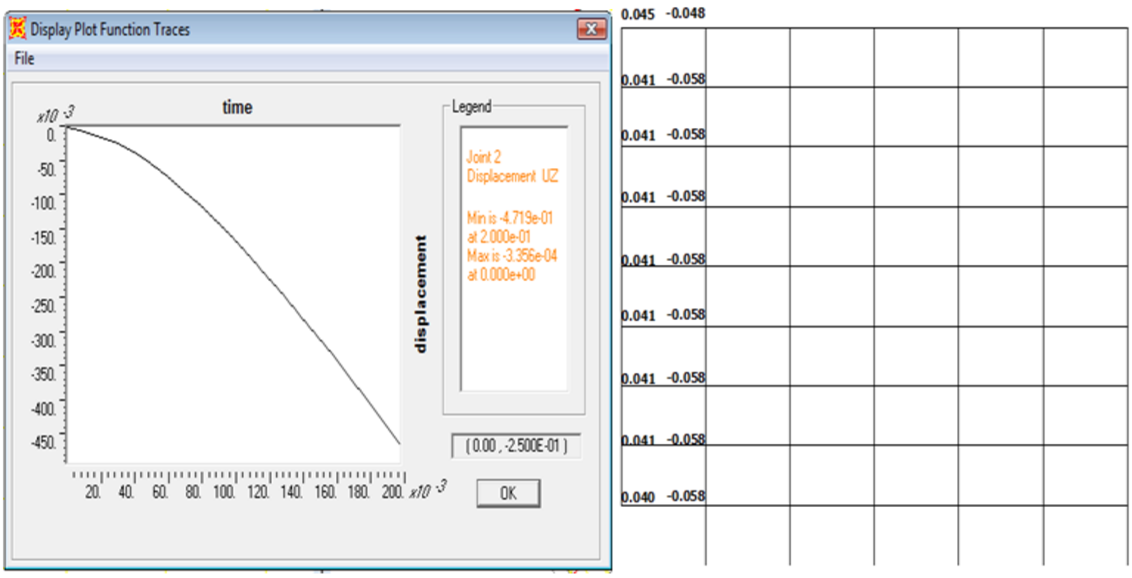


Figure 6.64: Displacement and hinge rotations for case-3 by DoD guideline for 9-storey

Summary of nonlinear static analysis and nonlinear dynamic analysis for case-3 column removal is given in tabular form. Table 6.8 shows the summary for GSA and DoD guidelines. Table shows the maximum collapse load as per nonlinear static analysis, displacement ductility as per nonlinear dynamic analysis, maximum support rotations and plastic hinge rotations. From Fig. 6.63 it is seen that maximum displacement is 427 mm and maximum displacement for Fig. 6.64 is 471 mm.

Table 6.8: Summary for case-3 by GSA and DoD guidelines for 9-storey

	GSA guidelines	DoD guidelines
Nonlinear static analysis	Collapse at 47% of load	Collapse at 38% of load
Displacement ductility	(427 mm/54 mm)= 7.91	(471 mm/45 mm)= 10.47
Permissible ductility	20	20
Maximum support rotation	0.052 radians	0.057 radians
Plastic hinge rotation	0.050 radians	0.057 radians

Fig. 6.65 shows the displacement under the column removal location for case-4 as per GSA guideline and corresponding plastic hinge rotations at collapse. Fig. 6.66 shows displacement and plastic hinge rotations as per DoD guidelines for 4-storey building.

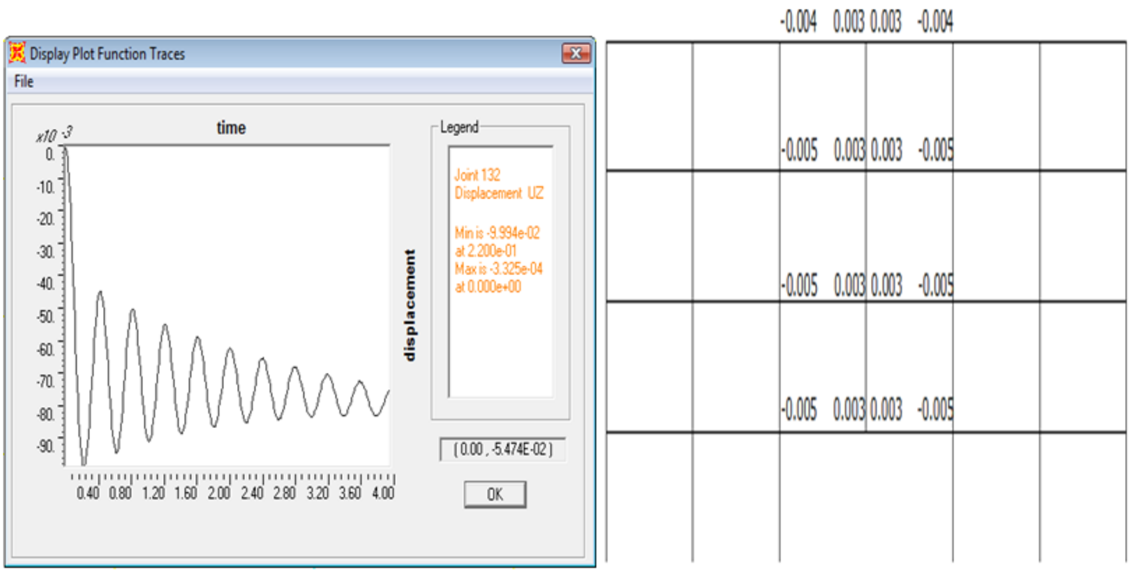


Figure 6.65: Displacement and hinge rotations for case-4 by GSA guideline for 4-storey

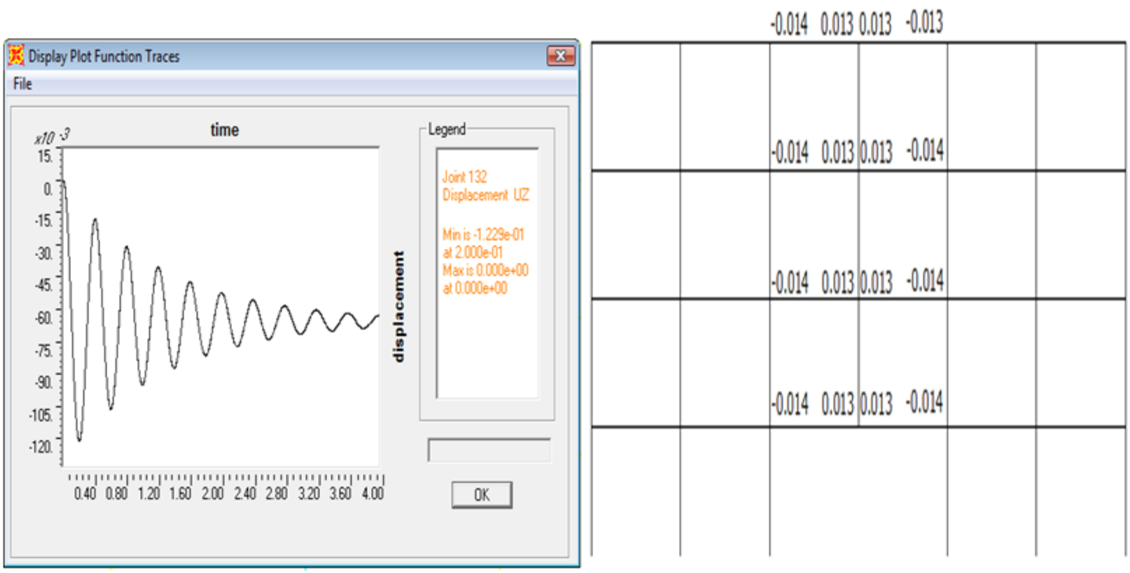


Figure 6.66: Displacement and hinge rotations for case-4 by DoD guideline for 4-storey

Summary of nonlinear static analysis and nonlinear dynamic analysis for case-4 column removal is given in tabular form. Table 6.9 shows the summary for GSA and DoD guidelines. Table shows the maximum collapse load as per nonlinear static analysis, displacement ductility as per nonlinear dynamic analysis, maximum support rotations and plastic hinge rotations. From Fig. 6.65 it is seen that maximum displacement is 94 mm and maximum displacement for Fig. 6.66 is 123 mm.

Table 6.9: Summary for case-4 by GSA and DoD guidelines for 4-storey

	GSA guidelines	DoD guidelines
Nonlinear static analysis	Collapse at 89% of load	Collapse at 67% of load
Displacement ductility	(94 mm/68 mm)=1.38	(123 mm/64 mm)=1.92
Permissible ductility	20	20
Maximum support rotation	0.011 radians	0.015 radians
Plastic hinge rotation	0.005 radians	0.014 radians

Fig. 6.67 shows the displacement under the column removal location for case-4 as per GSA guideline and corresponding plastic hinge rotations at collapse. Fig. 6.68 shows displacement and plastic hinge rotations as per DoD guidelines for 9-storey building.

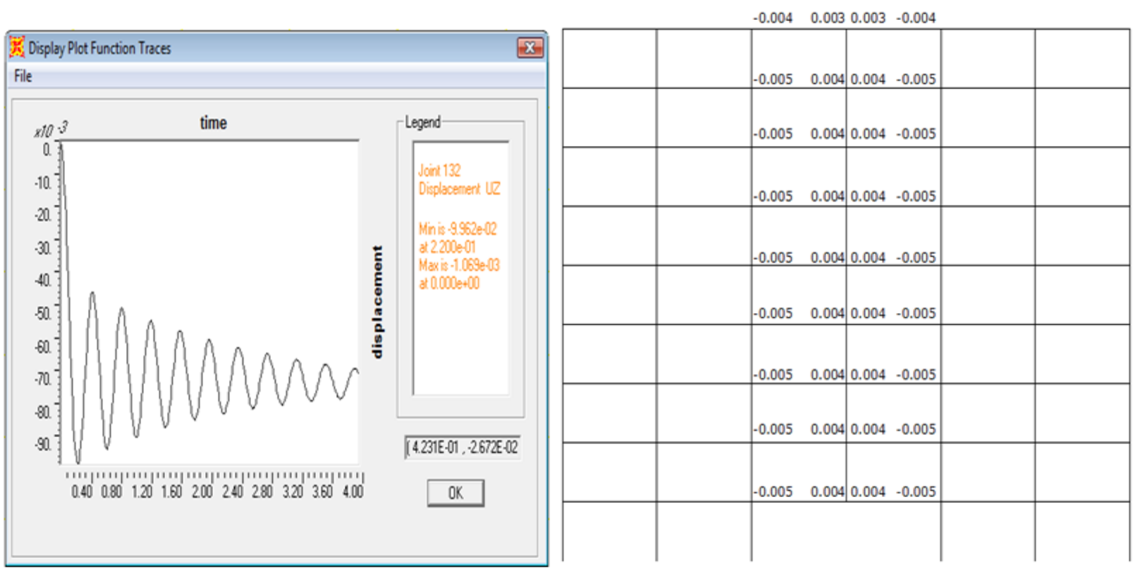


Figure 6.67: Displacement and hinge rotations for case-4 by GSA guideline for 9-storey

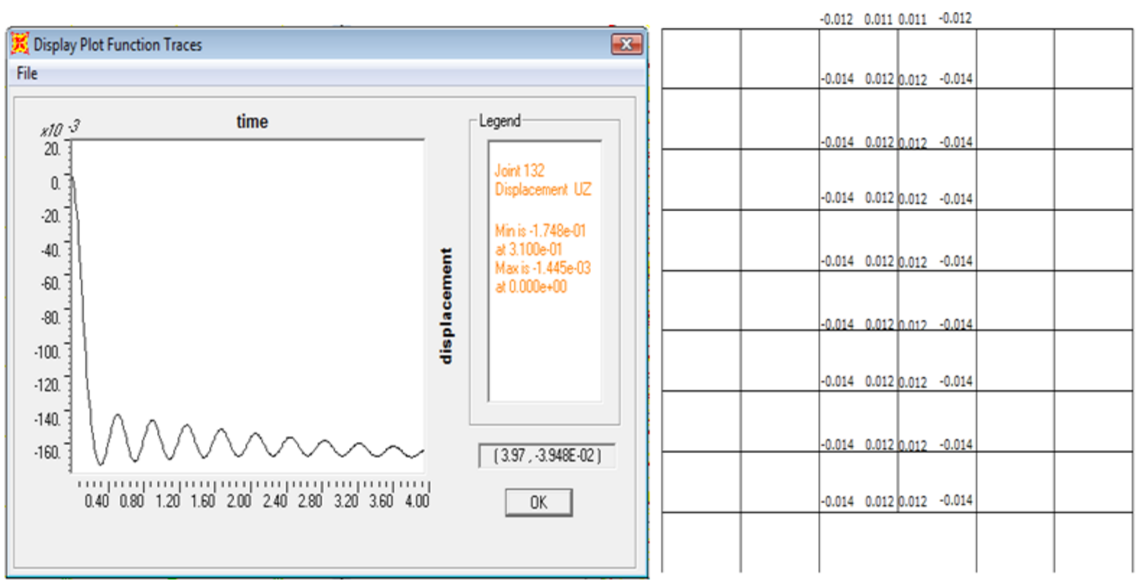


Figure 6.68: Displacement and hinge rotations for case-4 by DoD guideline for 9-storey

Summary of nonlinear static analysis and nonlinear dynamic analysis for case-4 column removal is given in tabular form. Table 6.10 shows the summary for GSA and DoD guidelines. Table shows maximum collapse load as per nonlinear static analysis, displacement ductility as per nonlinear dynamic analysis, maximum support rotations and plastic hinge rotations. From Fig. 6.67 it is seen that maximum displacement is 100 mm and maximum displacement for Fig. 6.68 is 175 mm.

Table 6.10: Summary for case-4 by GSA and DoD guidelines for 9-storey

	GSA guidelines	DoD guidelines
Nonlinear static analysis	Collapse at 89% of load	Collapse at 67% of load
Displacement ductility	(100 mm/64 mm)=1.56	(175 mm/68 mm)=2.57
Permissible ductility	20	20
Maximum support rotation	0.012 radians	0.021 radians
Plastic hinge rotation	0.005 radians	0.014 radians

6.5 Results and discussion

In this chapter nonlinear static (vertical pushover analysis) and nonlinear dynamic analysis is carried out for 4-storey and 9-storey steel buildings for all the four column removal cases as per GSA and DoD guidelines.

Nonlinear static analysis is carried out to understand the hinge formations at yield and at collapse. It also helps to understand the moment redistribution. Nonlinear static analysis gives maximum collapse load for all the column removal cases. Since the GSA and DoD mandated load combination includes a factor of 2 and 100% of the total load should be attainable through analysis but vertical pushover analysis indicates, 100% of vertical load is not attained at the time of collapse in any of the column removal cases. Collapse load achieved in DoD guidelines is lesser than the collapse load achieved using GSA guideline due to difference in the load combinations of both the guidelines. Minimum collapse load is attained for case-3 column removal and maximum collapse load is attained for case-4 column removal. So it states that potential for progressive collapse is high when the corner column is removed and potential for progressive collapse is low when the interior column is removed. These results are same as obtained from linear static and linear dynamic analysis. Similarly collapse load values decrease as the height of the building increases. So potential for progressive collapse of the building increases as the height of the building increases. Plastic hinge rotations are also found out at collapse load to see the state of hinges i.e. IO, LS, CP etc. When the hinges go beyond the CP state, hinges are considered to be collapsed. Plastic hinge rotations beyond CP state is 0.025 radians. So when the plastic hinge rotations are more than 0.025 radians for any member, it is considered as collapsed.

Nonlinear dynamic analysis is also carried out for all the four column removal cases using GSA and DoD guidelines. From nonlinear dynamic analysis displacement ductility is found out by dividing maximum displacement at ultimate capacity to displacement at yield capacity under the column removal locations. Permissible value for displacement ductility is given by the guidelines is 20. But in any column removal case the value of displacement ductility does not increase than the permissible value. Maximum value of displacement ductility is obtained for case-3 column removal and minimum value of displacement ductility is obtained for case-4 column removal. Nonlinear dynamic analysis could not completed successfully for case-2 and case-3 column removal. The reason is that mechanism has formed or plastic hinge has failed at this point, the model can not support the load. Since the analysis did not complete, members must be redesigned. So from nonlinear dynamic analysis it can be concluded that building is having high potential for progressive collapse when the column is removed from near the middle of shorter side and from the corner.

Maximum support rotations are also found out from nonlinear dynamic analysis. Maximum support rotations can be found out by taking the ratio of maximum displacement obtained from nonlinear dynamic analysis to the length of the adjacent beam from where the column is removed. Maximum support rotations are compared with the plastic hinge rotations obtained using SAP2000. Maximum value of plastic hinge rotations are obtained for case-3 column removal and minimum plastic hinge rotations are obtained for case-4 column removal.

6.6 Summary

In this chapter nonlinear static (vertical pushover analysis) and nonlinear dynamic analysis is carried out for 4-storey and 9-storey steel buildings for all the four column removal cases as per GSA and DoD guidelines. Illustrative examples are discussed using simply supported beam and continuous beam to understand the nonlinear static and dynamic analysis procedure. Iterative static analysis procedure is also explained as per GSA guideline. Iterative static analysis procedure is carried out if the analysis software does not have the facility to perform nonlinear static (pushover) analysis. Collapse load and maximum plastic hinge rotations are found out using nonlinear static analysis. Displacement ductility is found out using nonlinear dynamic analysis as per GSA and DoD guidelines.

Chapter 7

Mitigation measures of progressive collapse

7.1 General

Many government agencies and some private building owners today require that new buildings be designed and existing buildings evaluated and upgraded to provide an ability to resist the effects of potential blasts and other incidents that could cause extreme local damage without sustaining large scale collapse. While it may be possible to design buildings to resist such attacks without sustaining extreme damage, the loading effects associated with these hazards are so intense that design measures necessary to provide such performance would result in high costs as well as impose unacceptable limitations on the architectural design of such buildings.

7.2 Protection approaches for progressive collapse resistance

The following strategies are the most effective and economical for protecting structures from progressive collapse.

Alternate path approach

As a Typical approaches for collapse-resistant building design involves demonstration that not more than specified portions of a building will be subjected to collapse if the gravity load carrying capability of one or more vertical load carrying elements are suddenly lost. The initial loss of load carrying capability could be the result of an explosion, vehicle impact, fire or other cause. The actual cause of the initial damage to the gravity load-bearing system is typically not specified in the design procedure. However, the damage is assumed to be sudden and permanent. The engineer must determine that once the hypothetical damage has occurred, the structure is capable of redistributing the gravity loads, through alternative load paths and that collapse does not progress beyond certain specified limits.

Maximize standoff distance

The primary design strategy is to keep terrorists as far away from inhabited buildings as possible. The easiest and least costly approach for achieving the appropriate levels of protection against terrorist threats is to incorporate sufficient standoff distance into project designs. While sufficient standoff distance is not available to provide the minimum standoff distances required for conventional construction, maximizing the standoff distance results in the most cost-effective solution.

Minimize hazardous flying debris

In past explosive events where there was no building collapse, a high number of injuries resulted from flying glass fragments and debris from walls, ceilings, and fixtures (nonstructural features). Flying debris can be minimized through building design and avoidance of certain building materials and construction techniques. For example the glass used in most windows breaks at very low blast pressures. Minimizing those hazards through reduction in window numbers and sizes and through enhanced window construction has a major effect on limiting mass casualties.

Provide effective building layout

Effective design of building layout and orientation can significantly reduce opportunities for terrorists to target building occupants or injure large number of people. Place less important assets around a building perimeter, especially facing the side of the building that could be more exposed to an attack. More important assets should be placed farther from the building perimeter and/or a side that could face an attack. Other measures include attention to hallways and stairways to enable effective evacuation, rescue, and recovery.

Limit airborne contamination

Effective design of heating, ventilation, and air conditioning (HVAC) systems can significantly reduce the potential for chemical, biological, and radiological agents being distributed throughout buildings.

Provide mass notification

Providing a timely means to notify building occupants of threats and what should be done in response to those threats reduces the risk of mass casualties.

7.3 Retrofit strategies on mitigating progressive collapse of steel structures

The retrofit strategy may involve repair of deficient members, providing systems to increase stiffness and strength. Providing redundant load carrying systems such as mega truss or vierendeel trusses at the top of the building or by using bracing systems that redistribute the loads through the entire structure. Progressive failure in steel buildings occurs due to insufficient strength in the beams that are needed to bridge the load from the removed column location to the adjacent columns. Upon column removal, the vertical load is transferred to the adjacent columns, where the resulting increase in the flexure and shear demand on the adjacent beams. As such, upgrading the beams by increasing their strength and/ or stiffness is expected to be lost, upgrading both beams and columns might be needed.

A retrofit strategy using Fibre-Reinforced polymer (FRP) composites to strengthen the existing beam is expected to contribute to the strength, without significant contribution to the stiffness of the beam. A retrofit strategy that strengthens an existing beam using additional continuous steel plates will increase both strength and stiffness of the beam. On the other hand, strengthening a beam using intermittent steel plates will result in an increase in the stiffness without altering the strength of the beam.

In the analysis, the increase of strength is achieved by changing the yield strength. On the other hand, increasing the stiffness of the beam is achieved by increasing both modulus of elasticity and shear modulus. Finally increase of both strength and stiffness is conducted by increasing the thickness of flanges.

7.3.1 Retrofitting of 4-storey steel building

Progressive collapse analysis of 4-storey building is carried out using all the four analysis procedures for four column removal cases as discussed in chapters 5 and 6. Out of four column removal cases, column removal cases for case-2 and case-3 are considered for retrofitting. There are total three retrofitting strategies adopted as follows:

Retrofit strategy-1:

In this strategy, yield strength of beams and columns are increased. Initially the yield strength adopted for analysis was 250 MPa. In this strategy, the values of yield strengths considered for retrofitting are 300 MPa and 340 MPa. So in this strategy, the increase of yield strength is achieved up to 20% and 36% respectively.

Retrofit strategy-2:

In this strategy, stiffness of beams and columns are increased. Increase in the stiffness is achieved by increasing the modulus of elasticity and shear modulus up to 20% and 36%.

Retrofit strategy-3:

In this strategy, both stiffness and strength of the beams and columns are increased. Increase in the stiffness and strength of the sections is achieved by increasing the thickness of the flanges of the section by 20% and 36%.

7.3.2 Retrofit strategy-1

This strategy is applied for column removal case-2 and case-3 of 4-storey building. For column removal case-2, highly stressed columns C8, C22, C15 and C16 are retrofitted and affected beams above the column removal positions are retrofitted. Similarly for column removal case-3, highly stressed columns C1, C2, and C8 are retrofitted and affected beams above the column removal positions are retrofitted. The effect of retrofitting strategy on the DCR, collapse load, plastic hinge rotation and displacement ductility of the beams are evaluated. Fig. 7.1 and 7.2 show the DCR for flexure and column before retrofitting and after retrofitting for column removal case-2 for 20% strength increase.

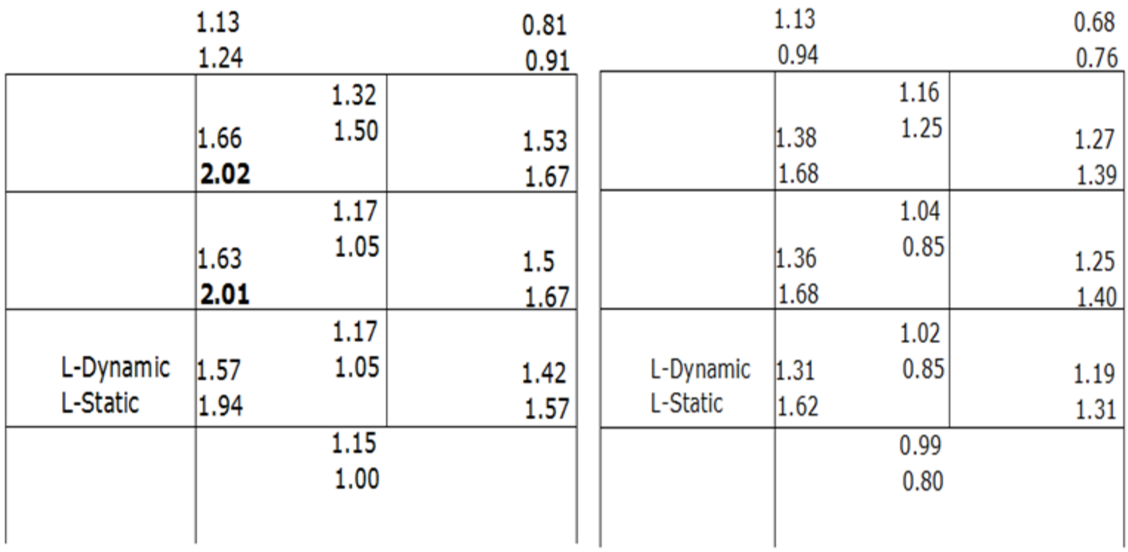


Figure 7.1: DCR before and after retrofitting for beam for 20% strength increase for case-2

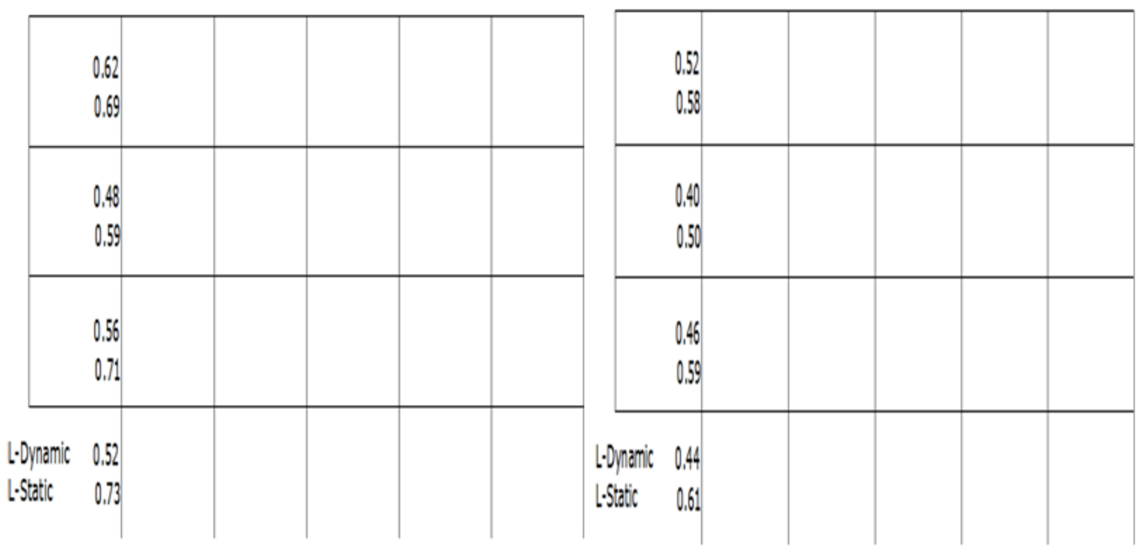


Figure 7.2: DCR before and after retrofitting for column for 20% strength increase for case-2

Fig. 7.3 and 7.4 show the DCR for flexure and column before retrofitting and after retrofitting for column removal case-2 for 36% strength increase.

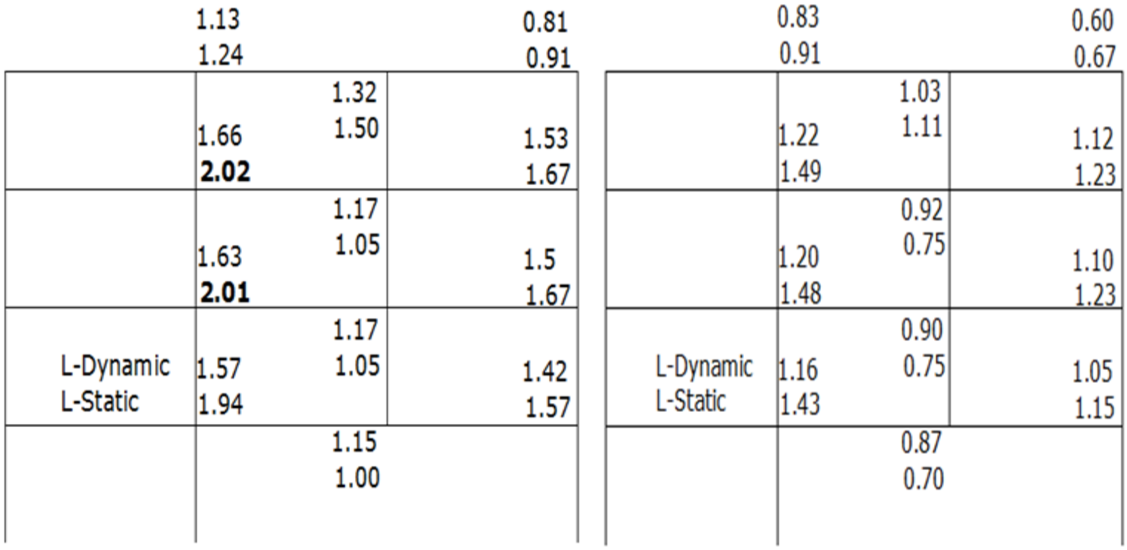


Figure 7.3: DCR before and after retrofitting for beam for 36% strength increase for case-2

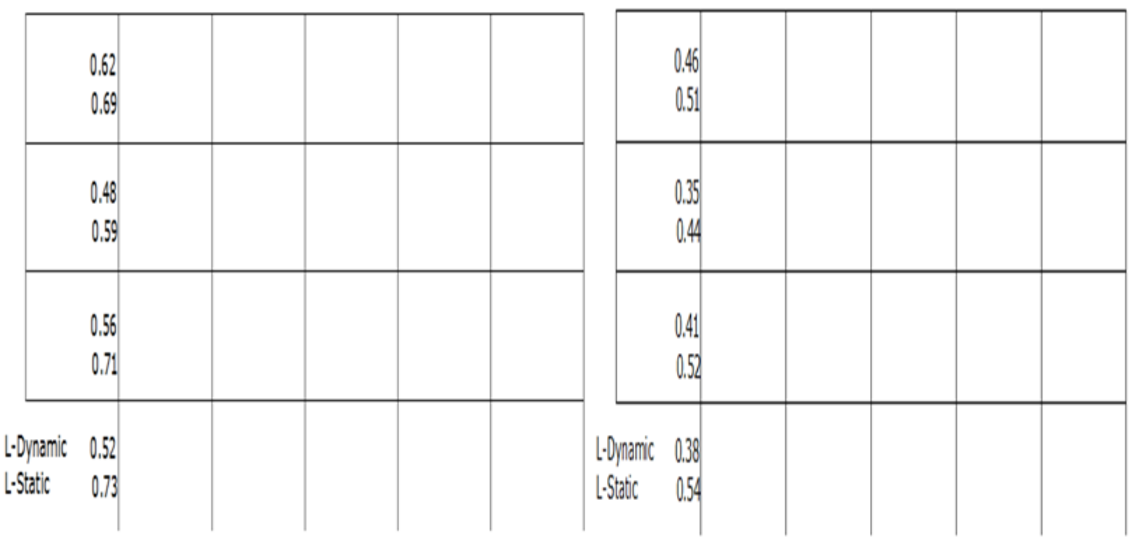


Figure 7.4: DCR before and after retrofitting for column for 36% strength increase for case-2

Fig. 7.5, 7.6 show the displacement for nonlinear dynamic analysis before and after retrofitting for column removal case-2 for 20% strength increase and 36% strength increase respectively.

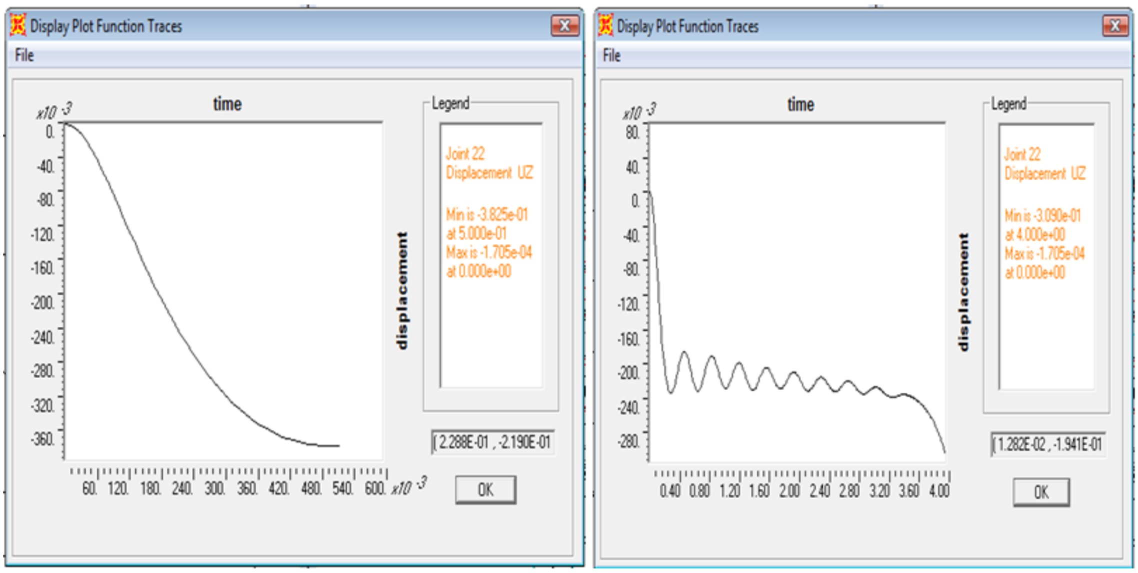


Figure 7.5: Displacement before and after retrofitting for 20% strength increase for case-2

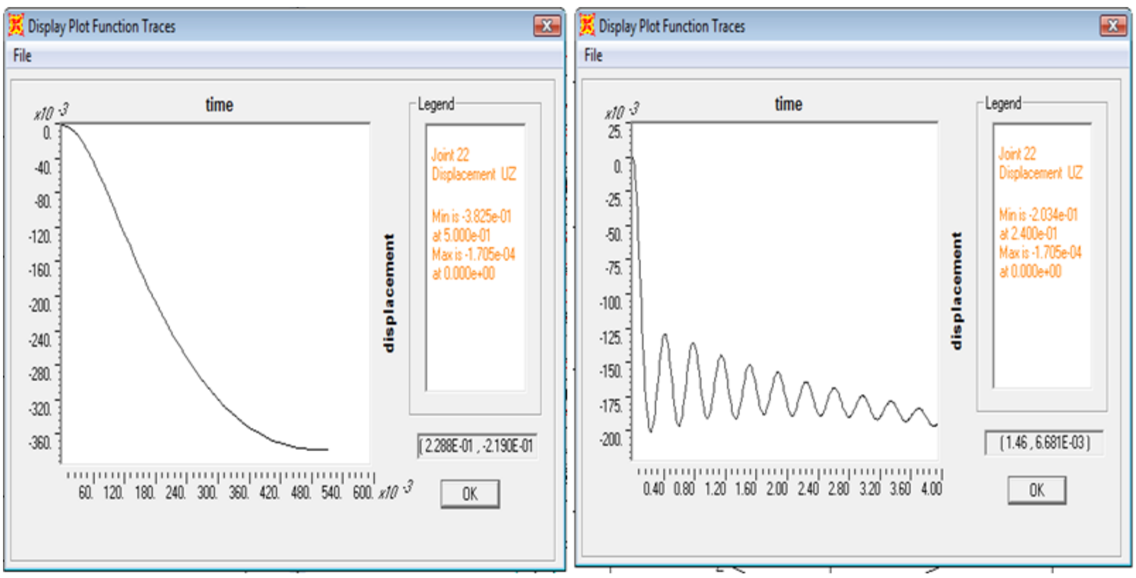


Figure 7.6: Displacement before and after retrofitting for 36% strength increase for case-2

Fig. 7.7 and 7.8 show the DCR for flexure and column before retrofitting and after retrofitting for column removal case-3 for 20% strength increase.

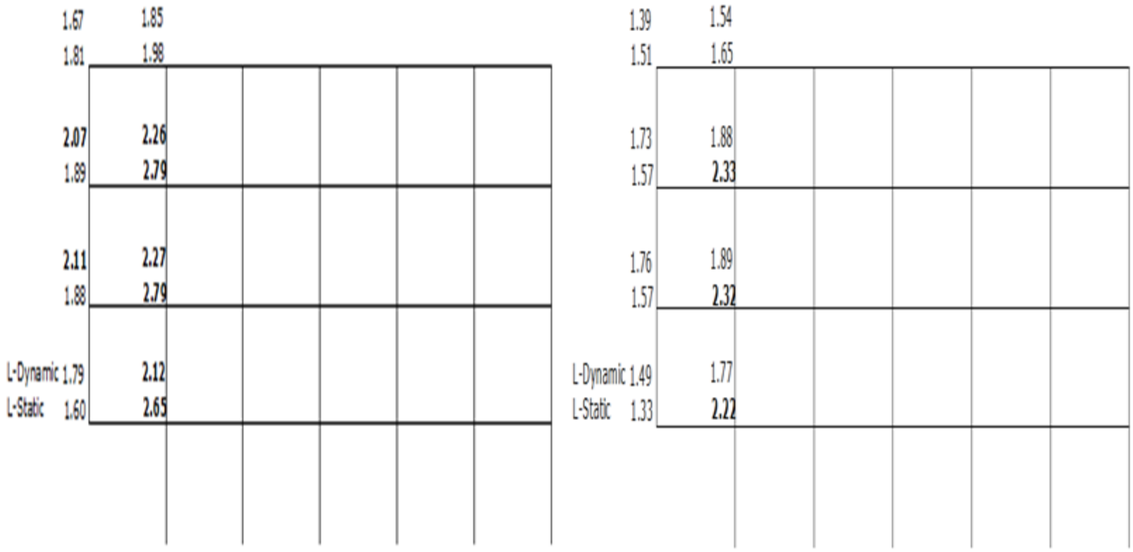


Figure 7.7: DCR before and after retrofitting for beam for 20% strength increase for case-3

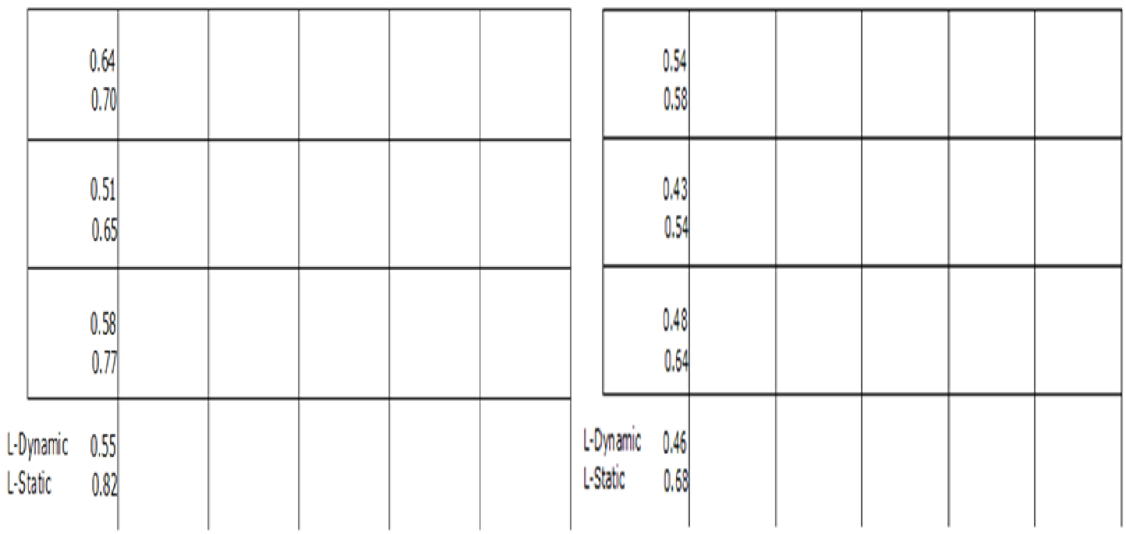


Figure 7.8: DCR before and after retrofitting for column for 36% strength increase for case-3

Fig. 7.9 and 7.10 show the DCR for flexure and column before retrofitting and after retrofitting for column removal case-3 for 36% strength increase.

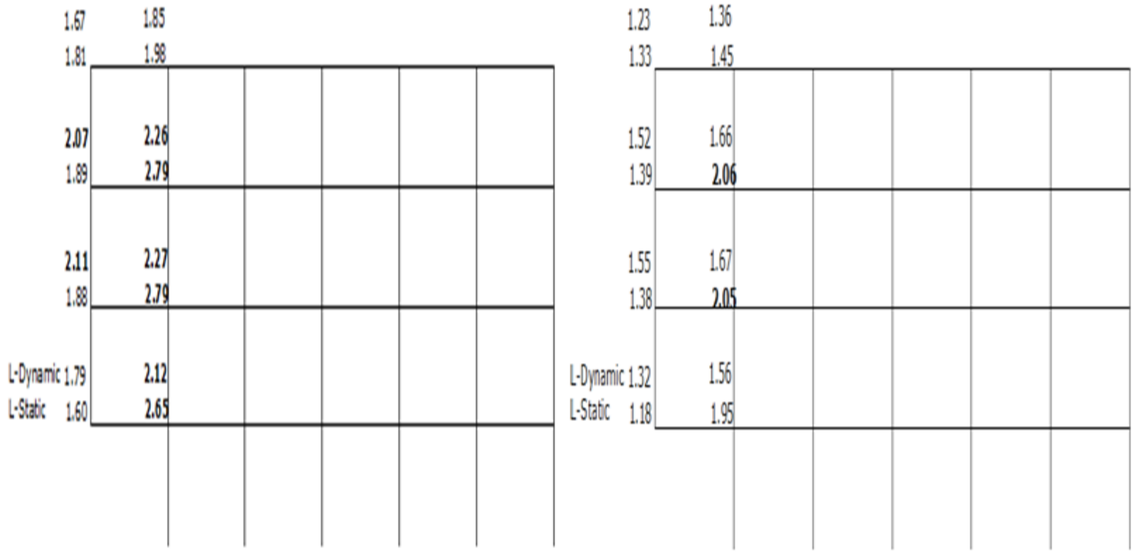


Figure 7.9: DCR before and after retrofitting for beam for 36% strength increase for case-3

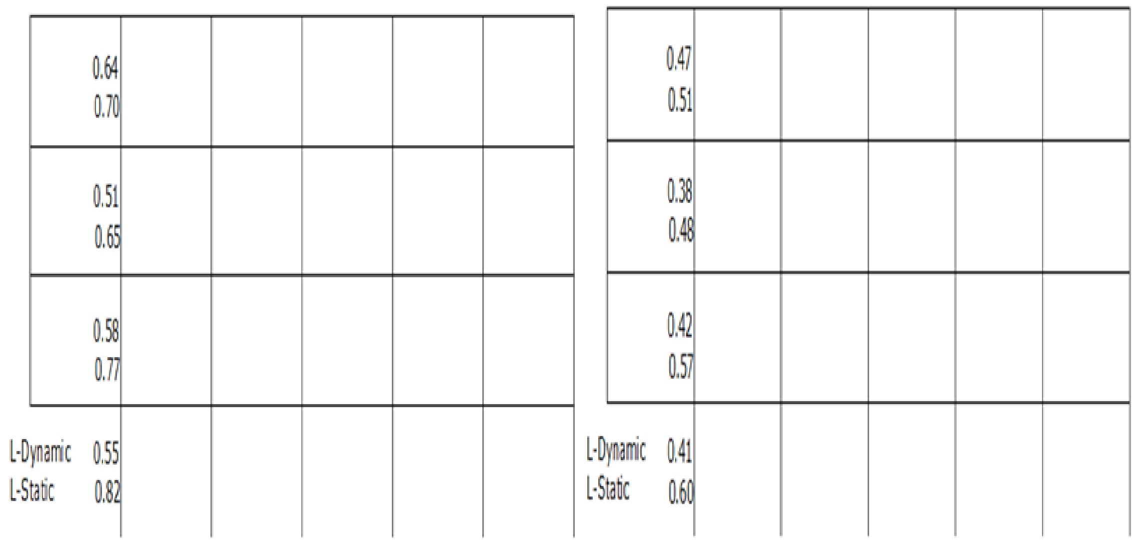


Figure 7.10: DCR before and after retrofitting for column for 36% strength increase for case-3

Fig. 7.11, 7.12 show the displacement for nonlinear dynamic analysis before and after retrofitting for column removal case-3 for 20% strength increase and 36% strength increase respectively.

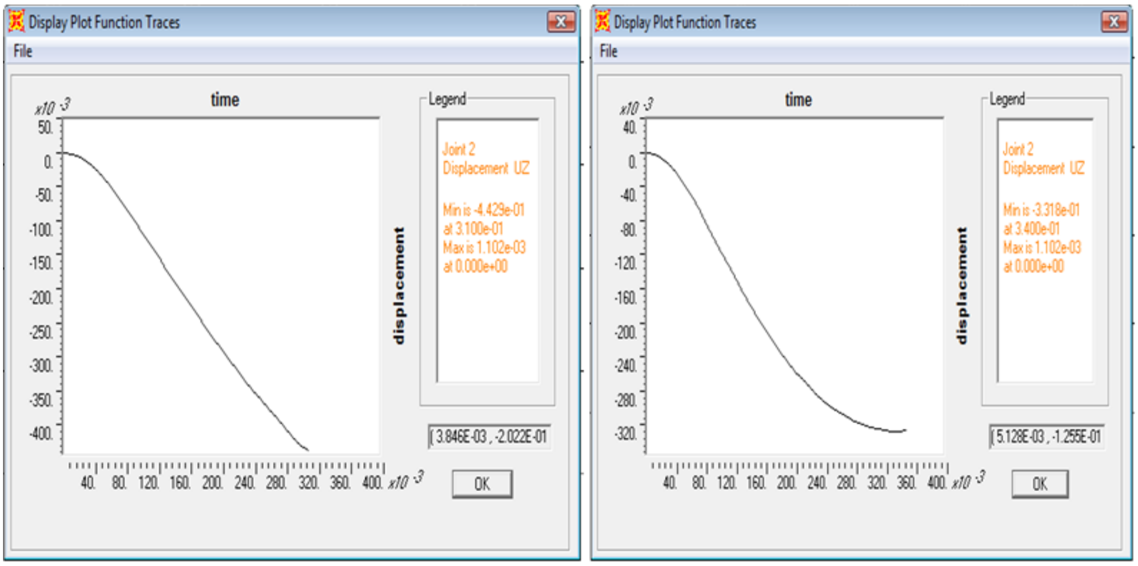


Figure 7.11: Displacement before and after retrofitting for 20% strength increase for case-3

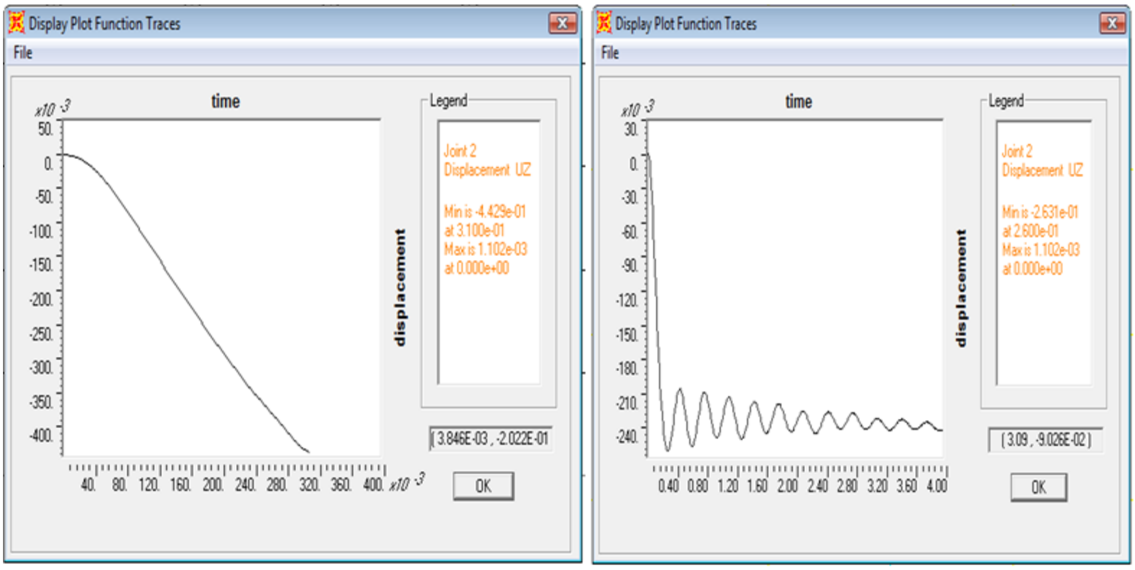


Figure 7.12: Displacement before and after retrofitting for 36% strength increase for case-3

Summary of case-2 and case-3 column removal after retrofitting is given in tabular form. Table 7.1 shows the maximum value of DCR for beam and column, collapse load, displacement ductility and plastic hinge rotation before and after retrofitting for case-2 column removal. Collapse load is found out from nonlinear static analysis as explained in previous chapter. Displacement ductility is found by taking the ratio of ultimate displacement to displacement at yield. Similarly Table 7.2 shows the summary for case-3 column removal.

Table 7.1: Summary for case-2 column removal for strength increase

	W/o retrofitting	20% strength increase	36% strength increase
DCR for beam	2.02	1.68	1.43
DCR for column	0.73	0.61	0.54
Collapse load	58%	68.3%	77%
Displacement ductility	5.23	3.72	1.99
Plastic hinge rotation	0.033	0.026	0.013

Table 7.2: Summary for case-3 column removal for strength increase

	W/o retrofitting	20% strength increase	36% strength increase
DCR for beam	2.79	2.32	2.05
DCR for column	0.82	0.68	0.60
Collapse load	52%	62%	70%
Displacement ductility	7.61	4.76	2.76
Plastic hinge rotation	0.051	0.032	0.024

7.3.3 Retrofit strategy-2

The cases considered for retrofitting strategies are column removal case-2 and case-3. The effect of retrofitting strategy on the DCR, collapse load, plastic hinge rotation and displacement ductility of the beams are evaluated. Fig. 7.13 and 7.14 show the DCR for flexure and column before and after retrofitting for column removal case-2 for 20% stiffness increase.

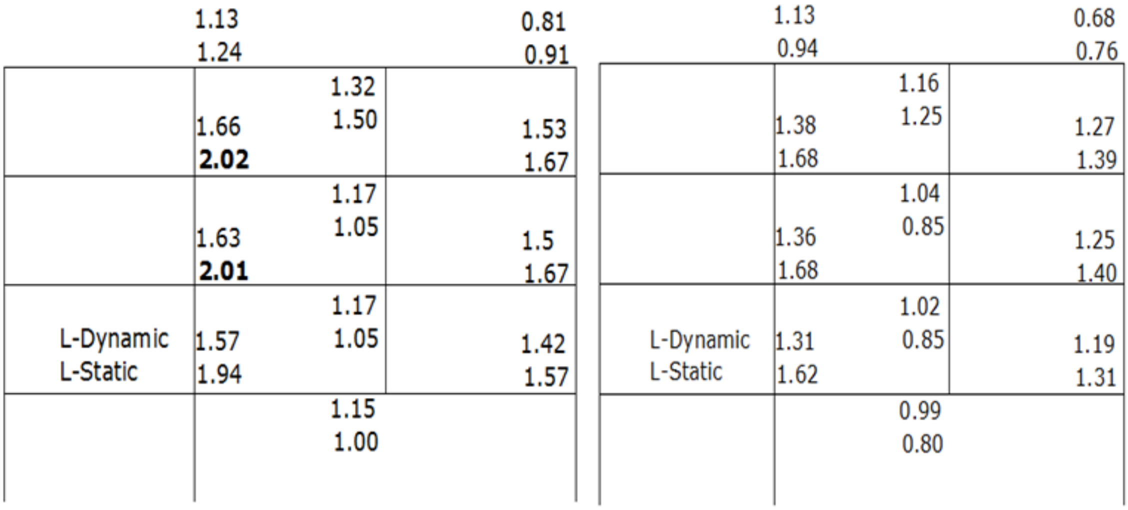


Figure 7.13: DCR before and after retrofitting for beam for 20% stiffness increase for case-2

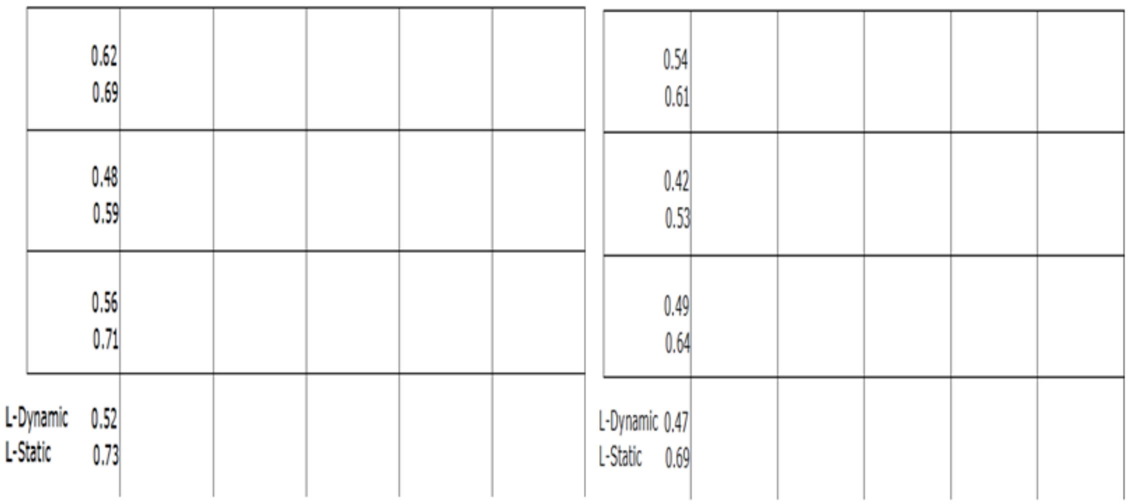


Figure 7.14: DCR before and after retrofitting for column for 20% stiffness increase for case-2

Fig. 7.15 and 7.16 show the DCR for flexure and column before retrofitting and after retrofitting for column removal case-2 for 36% stiffness increase.

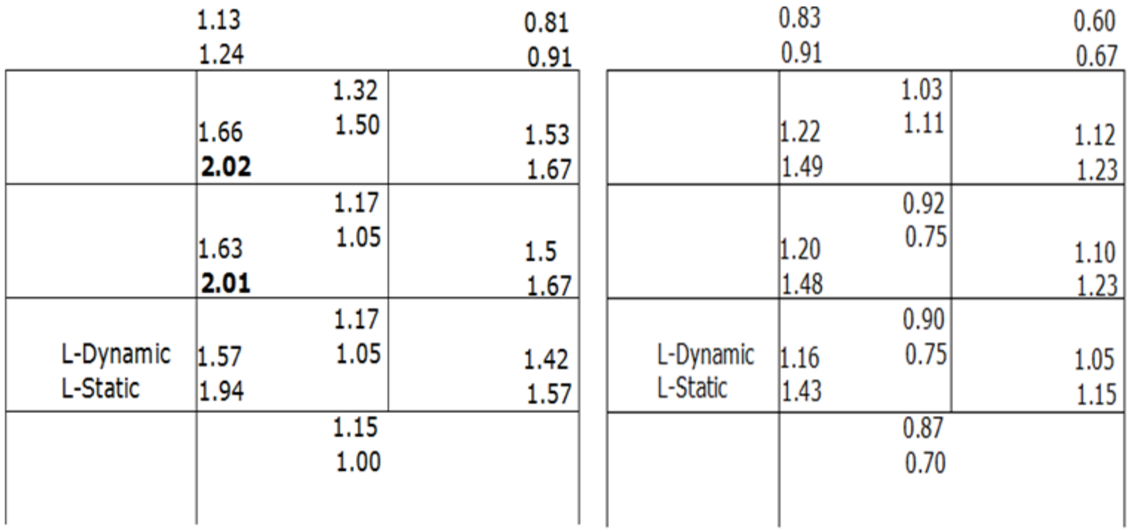


Figure 7.15: DCR before and after retrofitting for beam for 36% stiffness increase for case-2

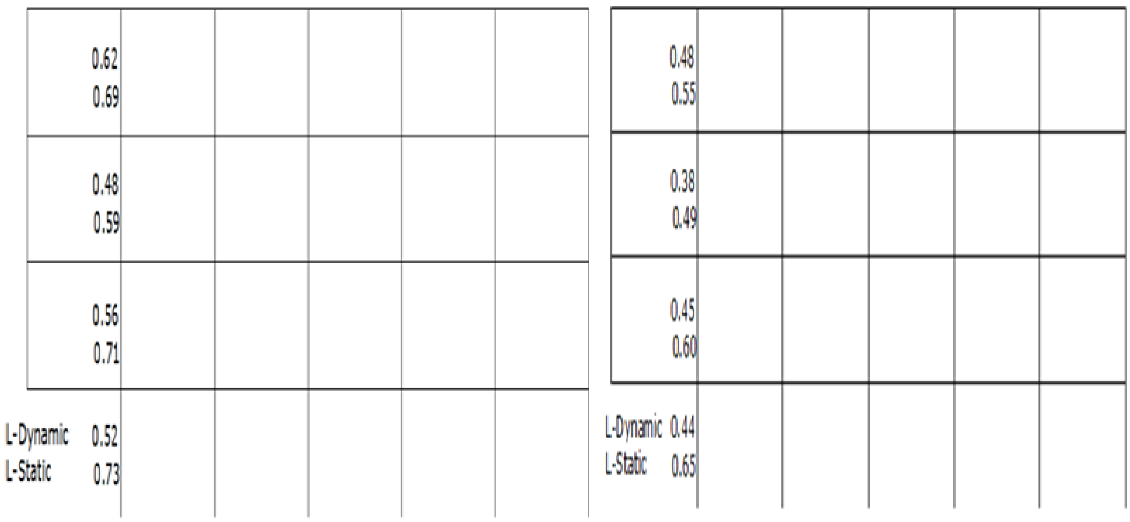


Figure 7.16: DCR before and after retrofitting for column for 36% stiffness increase for case-2

Fig. 7.17, 7.18 show the displacement for nonlinear dynamic analysis before and after retrofitting for column removal case-2 for 20% stiffness increase and 36% stiffness increase respectively.

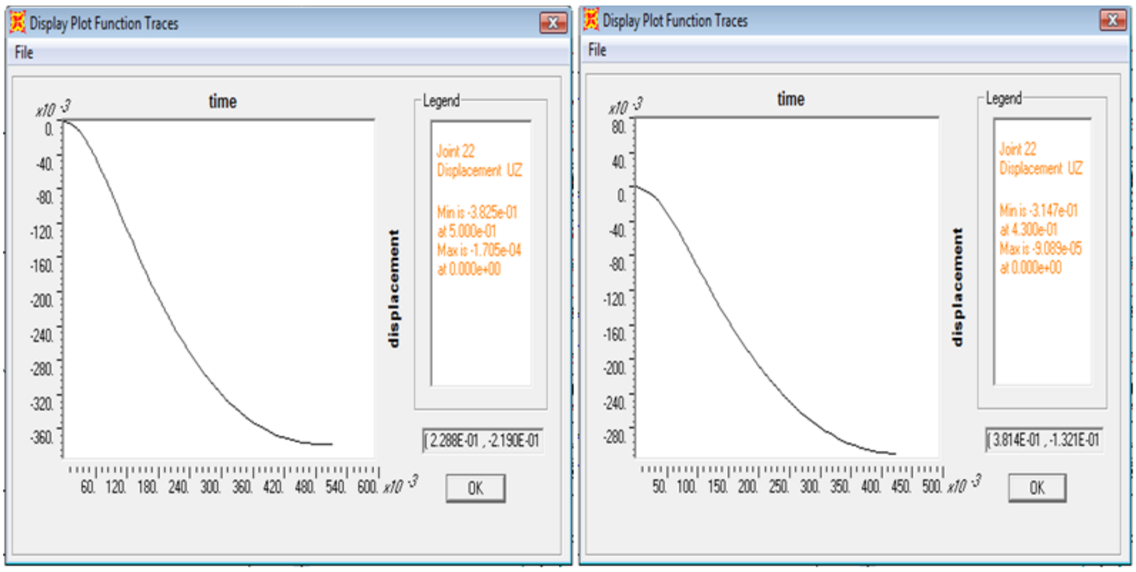


Figure 7.17: Displacement before and after retrofitting for 20% stiffness increase for case-2

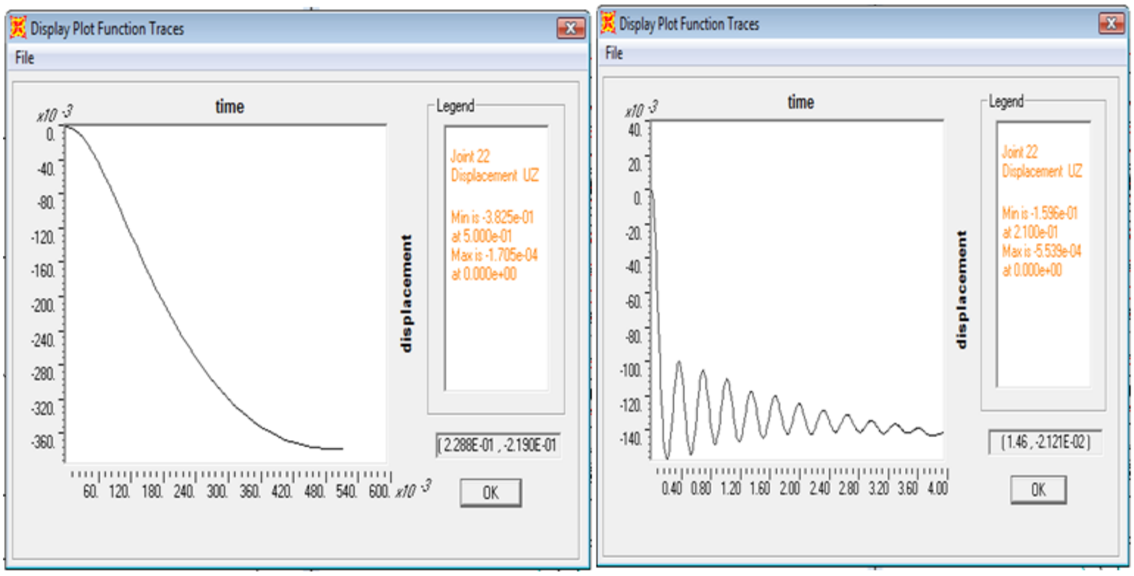


Figure 7.18: Displacement before and after retrofitting for 36% stiffness increase for case-2

Fig. 7.19 and 7.20 show the DCR for flexure and column before retrofitting and after retrofitting for column removal case-3 for 20% stiffness increase.

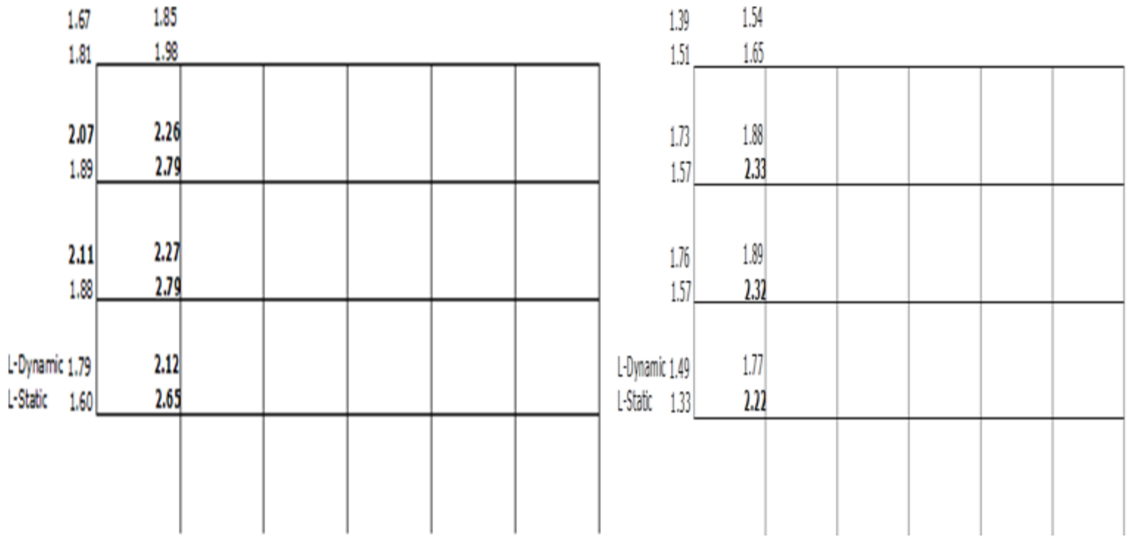


Figure 7.19: DCR before and after retrofitting for beam for 20% stiffness increase for case-3

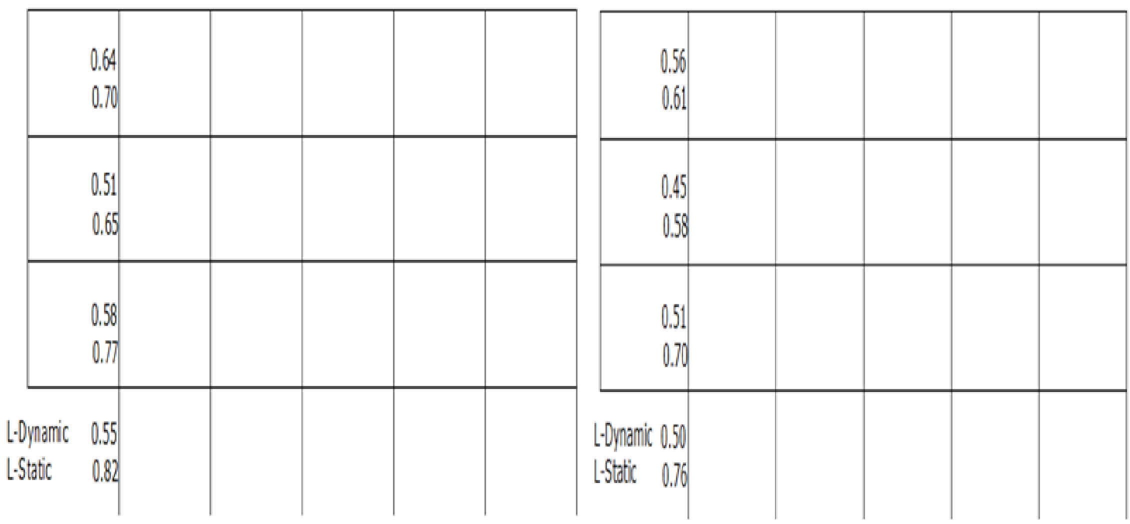


Figure 7.20: DCR before and after retrofitting for column for 20% stiffness increase for case-3

Fig. 7.21 and 7.22 show the DCR for flexure and column before retrofitting and after retrofitting for column removal case-3 for 36% stiffness increase.

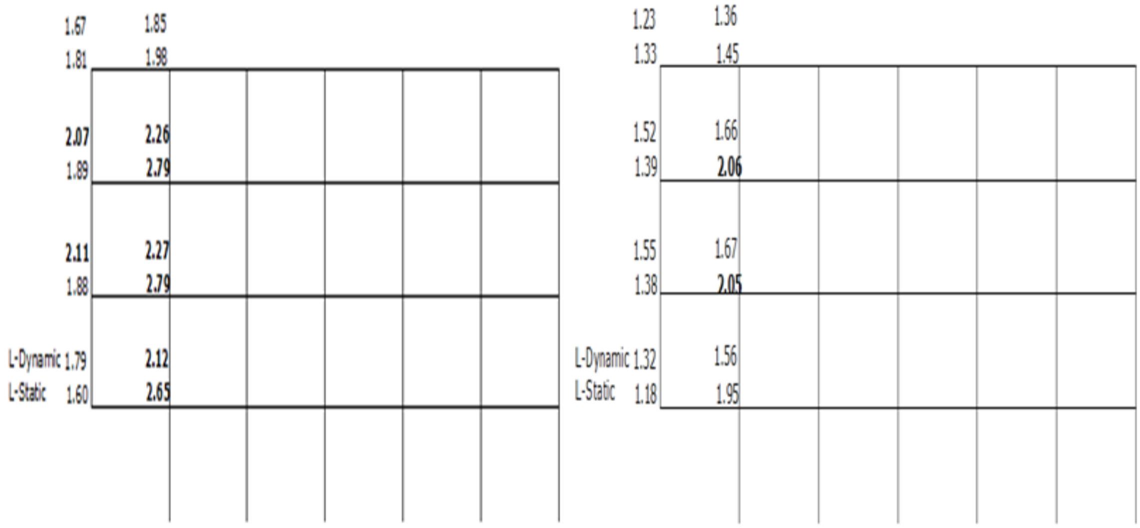


Figure 7.21: DCR before and after retrofitting for beam for 36% stiffness increase for case-3

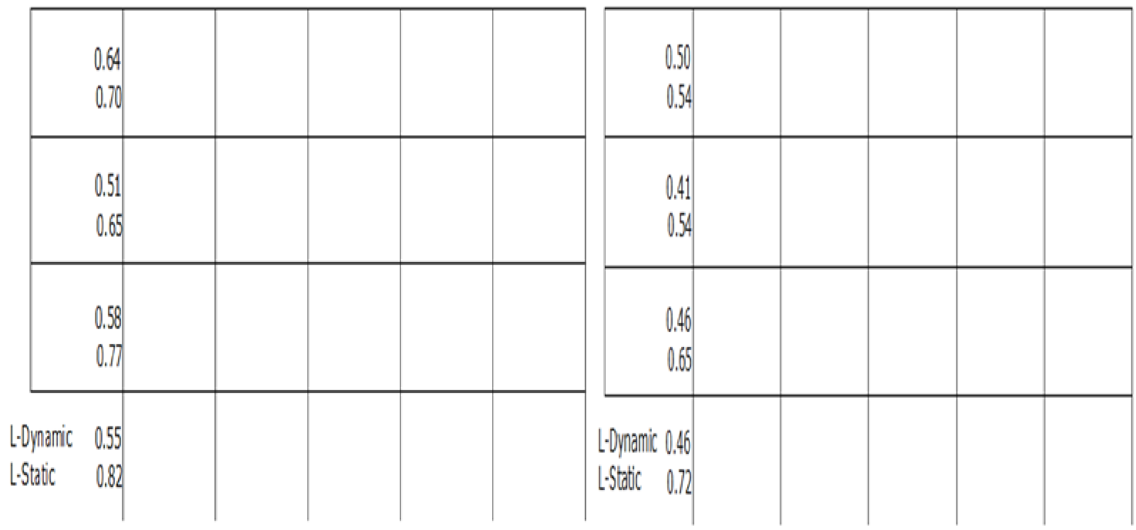


Figure 7.22: DCR before and after retrofitting for column for 36% stiffness increase for case-3

Fig. 7.23, 7.24 show the displacement for nonlinear dynamic analysis before and after retrofitting for column removal case-3 for 20% stiffness increase and 36% stiffness increase respectively.

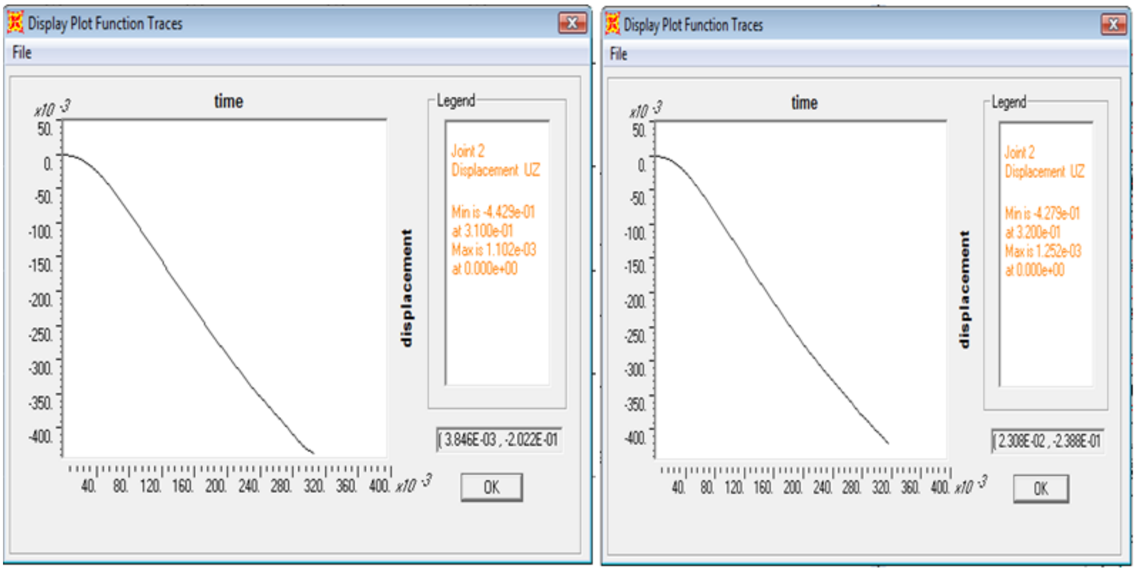


Figure 7.23: Displacement before and after retrofitting for 20% stiffness increase for case-3

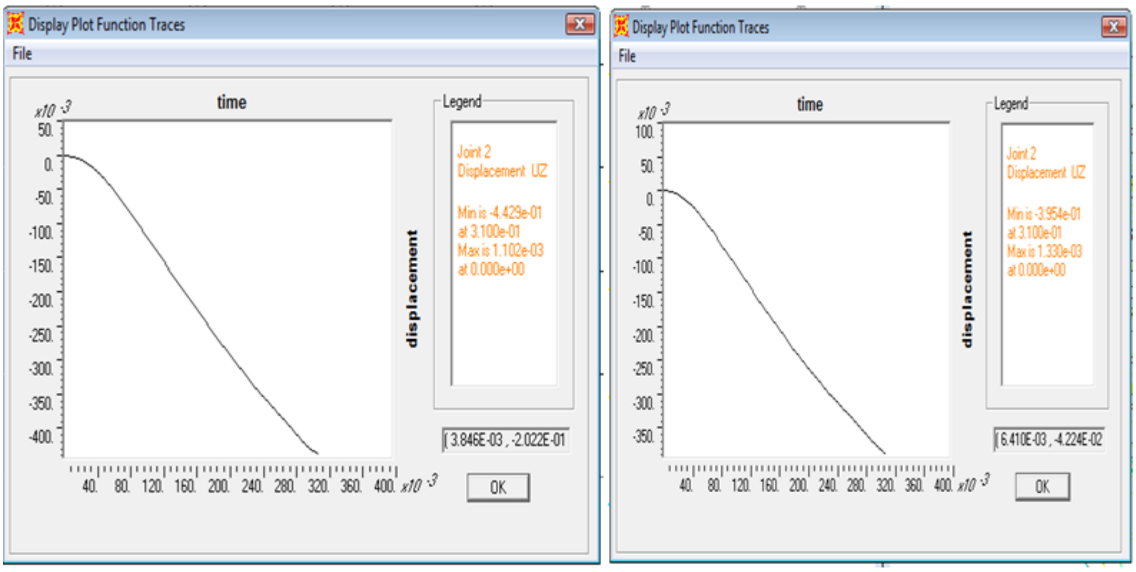


Figure 7.24: Displacement before and after retrofitting for 36% stiffness increase for case-3

Summary of case-2 and case-3 column removal after retrofitting is given in tabular form. Table 7.3 shows the maximum value of DCR for beam and column, collapse load, displacement ductility and plastic hinge rotation before and after retrofitting for case-2 column removal. Collapse load is found out from nonlinear static analysis as explained in previous chapter. Displacement ductility is found by taking the ratio of ultimate displacement to displacement at yield. Similarly Table 7.4 shows the summary for case-3 column removal.

Table 7.3: Summary for case-2 column removal for stiffness increase

	W/o retrofitting	20% stiffness increase	36% stiffness increase
DCR for beam	2.02	1.68	1.43
DCR for column	0.73	0.69	0.65
Collapse load	58%	58%	58%
Displacement ductility	5.23	4.35	3.80
Plastic hinge rotation	0.033	0.026	0.023

Table 7.4: Summary for case-3 column removal for stiffness increase

	W/o retrofitting	20% stiffness increase	36% stiffness increase
DCR for beam	2.79	2.32	2.05
DCR for column	0.82	0.76	0.72
Collapse load	52%	52%	52.2%
Displacement ductility	7.61	7.20	6.54
Plastic hinge rotation	0.051	0.042	0.038

7.3.4 Retrofit strategy-3

The cases considered for retrofitting strategies are column removal case-2 and case-3. Retrofitting of the sections is achieved by increasing the thickness of the flanges of the section by 20% and 36%. The effect of retrofitting strategy on the DCR, collapse load, plastic hinge rotation and displacement ductility of the beams are evaluated same as discussed in retrofit strategy-1 and 2. Summary of case-2 and case-3 column removal after retrofitting is given in tabular form. Table 7.5 shows the maximum value of DCR for beam and column, collapse load, displacement ductility and plastic hinge rotation before and after retrofitting for case-2 column removal. Similarly Table 7.6 shows the summary for case-3 column removal.

Table 7.5: Summary for case-2 column removal for strength and stiffness increase

	W/o retrofitting	20% strength and stiffness increase	36% strength and stiffness increase
DCR for beam	2.02	1.83	1.68
DCR for column	0.73	0.66	0.61
Collapse load	58%	72.5%	79%
Displacement ductility	5.23	2.43	2.09
Plastic hinge rotation	0.033	0.014	0.010

Table 7.6: Summary for case-3 column removal for strength and stiffness increase

	W/o retrofitting	20% strength and stiffness increase	36% strength and stiffness increase
DCR for beam	2.79	2.51	2.30
DCR for column	0.82	0.73	0.68
Collapse load	52%	65.4%	71.4%
Displacement ductility	7.61	3.85	3.16
Plastic hinge rotation	0.051	0.025	0.019

7.4 Results and discussion

Three retrofitting strategies are applied for column removal case-2 and case-3 for 4 storey steel building. Column removal case-2 and case-3 are selected because they found to be more critical from nonlinear dynamic analysis as it could not completed successfully. Comparison between DCR, maximum collapse load, displacement ductility and plastic hinge rotations are carried out for building without retrofitting and with retrofitting. Upgrading the beams and columns by increasing their strength only is more effective than increasing their stiffness only. DCR found out after applying the retrofit strategy-1 gives better improvement in progressive collapse resistance than retrofit strategy-2. Collapse load found from nonlinear static analysis increases significantly for retrofit strategy-1 while there is very negligible increase in collapse load for retrofit strategy-2. Similarly there is significant decrease in displacement ductility for retrofit strategy-1 compared to retrofit strategy-2. Nonlinear dynamic analysis completed successfully for case-2 column removal for retrofit strategy-1 and for 36% stiffness increase in retrofit strategy-2. While nonlinear dynamic analysis could not completed successfully for case-3 column removal for retrofit strategy-2. So it can be concluded that increase in strength gives better result than increase in stiffness only. The value of DCR decreases in retrofit strategy-3 but reduction in values are lesser compared to retrofit strategy-1. While collapse load increases more in the case of strategy-3 compared to strategy-1. Similarly ductility and plastic hinge rotations decrease more for strategy-3 compared to strategy-1. From the above observations, it can be said that the choice of the most suitable rehabilitation scheme to safeguard against the progressive collapse should consider the loading criteria, the targeted level of safety, and the desired performance parameter needed to be enhanced.

7.5 Summary

Three retrofitting strategies are applied for column removal case-2 and case-3 for 4 storey steel building. DCR are found using three retrofitting strategies for beams and columns. Maximum collapse load is found using nonlinear static analysis with all the three retrofitting strategies. Displacement ductility is found using nonlinear dynamic analysis. Comparison of DCR, maximum collapse load, displacement ductility and plastic hinge rotations are carried out for buildings without retrofitting and with retrofitting.

Chapter 8

Summary and Conclusion

8.1 Summary

The major project deals with progressive collapse analysis of regular steel buildings using various guidelines. The report includes definition and overview regarding progressive collapse phenomena. The mechanism of progressive collapse is discussed with the historical background. Various case studies of the progressive collapse of the buildings are also discussed in detail to understand the various causes. It also includes evolution and comparison between various guidelines available to evaluate the vulnerability of building for progressive collapse. The comparison is organized based on Definition, threshold for consideration of progressive collapse, general strategy, loads, key elements and existing buildings.

Progressive collapse analysis procedure, loading to perform static and dynamic analysis, internal and external column removal consideration for symmetrical structural configuration and acceptance criteria for DCR, for displacement ductility, for plastic hinge rotation as per GSA and DoD guidelines are discussed.

Progressive collapse analysis of 4-storey and 9-storey regular moment resisting steel building is performed using both the GSA and DoD guidelines. Progressive collapse analysis using linear static and linear dynamic analysis of 4-storey and 9-storey regular moment resistant steel buildings are performed using SAP2000 software. DCR for beams and for highly stressed nearby columns are calculated at all storey for all the four column removal cases. Study of vertical displacement under column removal point is carried out when column is removed from different locations. Displacement obtained by linear static analysis are compared with displacement obtained by linear dynamic analysis for both GSA and DoD guidelines.

Report also includes progressive collapse analysis of 4-storey and 9-storey steel buildings using nonlinear static and nonlinear dynamic analysis as per GSA and DoD guidelines. Illustrative examples of simply supported beam and continuous beam are discussed to understand the concept of nonlinear static and nonlinear dynamic analysis. Iterative static analysis procedure is also explained as per GSA guideline. Iterative static analysis procedure is carried out if the analysis software does not have the facility to perform nonlinear static (pushover) analysis. Maximum collapse loads are found out using nonlinear static analysis as per both the guidelines. Displacement ductility is found using nonlinear dynamic analysis and values are compared with maximum permissible value specified in the guidelines. Similarly plastic hinge rotations are also found using nonlinear dynamic analysis.

Mitigation measures are discussed to reduce progressive collapse potential of the structure. Various protection strategies to reduce progressive collapse are discussed. Three retrofit strategies are adopted. In retrofit strategy-1 yield strength of the beams and columns are increased. In retrofit strategy-2 stiffness of beams and columns are increased. In retrofit strategy-3 both stiffness and strength of the beams and columns are increased. DCR are found using retrofitting strategies for beams and columns. Maximum collapse load is found using nonlinear static analysis with all the three strategies. Displacement ductility is found using nonlinear dynamic analysis. Comparisons of DCR, maximum collapse load, displacement ductility and plastic hinge rotations are carried out for the buildings without retrofitting and with retrofitting.

8.2 Conclusions

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

- DCR obtained by DoD guidelines are having higher values compared to those obtained by GSA guidelines for all the four column removal cases. The reason is the difference in the load cases. Generally the DoD guidelines are used for military departments, the defense agencies and the structures of national importance. Therefore DoD guidelines use larger load factors compared to GSA guidelines. In DoD guidelines lateral loads of 0.2% of vertical loads are also applied while GSA guidelines do not specify any lateral loads.

- In 4-storey and 9-storey steel buildings, DCR for flexure exceeds the permissible value for each column removal case.

- In 4-storey building linear static analysis governed the value of DCR as compared to linear dynamic analysis for left side and right side of the column removal position, while values of DCR is higher for linear dynamic analysis as compared to linear static analysis for center point of the column removal position in most of the cases.

- In 9-storey building linear static analysis governed the value of DCR as compared to linear dynamic analysis for left side and right side of the column removal position, while values of DCR is higher for linear dynamic analysis as compared to linear static analysis for center point of the column removal position in most of the cases. But for case-4 column removal, DCR obtained at center point of column removal position by linear static analysis governs.

- In 4- storey and 9-storey buildings, maximum values of DCR for flexure are observed when the corner column is removed. So it can be concluded that the potential for progressive collapse increases when the corner column is failed. Similarly the values of DCR for flexure comes minimum when the interior column is removed. So it can be concluded that the potential for progressive collapse analysis reduces when the interior column is collapsed.

- In 4- storey building, DCR found out for the columns for different cases are within permissible limits. Linear static analysis gives higher values compared to linear dynamic analysis except at top level. At top level DCR value becomes approximately equal for both the analysis approaches.

- In 9- storey building, DCR found out for the columns for different cases exceed the permissible value for near by columns for case-3 using GSA guideline and for case-1, 2 and 3 using DoD guidelines. Linear static analysis gives higher values compared to linear dynamic analysis except at top level. At top level DCR value becomes approximately equal for both the analysis approaches.

- In 4-storey and 9-storey buildings, DCR found out for shear for different cases are within permissible limits.

- DCR values for flexure, shear in beams and in columns increase as the height of the building increases. So potential for progressive collapse of the building increases as the height of the building increases for the types of buildings considered in this study.

- Displacements found out under column removal is estimated to be 5-10% more in linear static analysis compared to linear dynamic analysis.

- Since the GSA and DoD mandated load combination includes a factor of 2 and 100% of the total load should be attainable through analysis but vertical pushover analysis indicates, 100% of vertical load is not attained at the time of collapse in any of the column removal cases.

- Collapse load achieved in DoD guidelines is lesser than the collapse load achieved using GSA guideline due to difference in the load combinations of both the guidelines. Minimum collapse load is attained for case-3 column removal and maximum collapse load is attained for case-4 column removal for both 4-storey and 9-storey buildings. So potential for progressive collapse is high when the corner column is removed and potential for progressive collapse is low when the interior column is removed.

- Collapse load values decrease as the height of the building increases. So potential for progressive collapse of the building increases as the height of the building increases for the buildings considered in this study.

- From nonlinear dynamic analysis, displacement ductility is found out under the column removal locations. Permissible value for displacement ductility is given by the guidelines is 20. But in any column removal case the value of displacement ductility does not increase than the permissible value. Maximum value of displacement ductility is obtained for case-3 column removal (10.47 by DoD guideline for 9-storey) and minimum value of displacement ductility is obtained for case-4 column removal (1.38 by GSA guideline for 4-storey). Nonlinear dynamic analysis could not completed successfully for case-2 and case-3 column removal due to inadequate strength and stiffness.

- For retrofitting, Upgrading the beams and columns by increasing their strength only (retrofit strategy-1) is more effective than increasing their stiffness only (retrofit strategy-2). DCR found out after applying the retrofit strategy-1 gives better progressive collapse resistance than retrofit strategy-2 for both the column removal cases.

- Collapse load found from nonlinear static analysis increases from 58% to 77% for case-2 and 52% to 70% for case-3 for retrofit strategy-1 while there is very negligible increase in collapse load from 58% to 58.1% for case-2 and 52% to 52.2% for case-3 for retrofit strategy-2.

- There is significant decrease in displacement ductility from 5.23 to 1.99 for case-2 and 7.61 to 2.76 for case-3 for retrofit strategy-1 compared to decrease in displacement ductility from 5.23 to 3.80 for case-2 and from 7.61 to 6.54 for case-3 for retrofit strategy-2.

- The value of DCR for beam decreases from 2.02 to 1.68 for case-2 and from 2.79 to 2.30 for case-3 when strength and stiffness both are increased (retrofit strategy-3). But reduction in values are lesser compared to retrofit strategy-1. Similarly for columns, reduction in the DCR is less from 0.73 to 0.61 for case-2 and from 0.82 to 0.68 for case-3 for retrofit strategy-3 compared to retrofit strategy-1.

- Collapse load increases more from 58% to 79% for case-2 and from 52% to 71.4% for case-3 for retrofit strategy-3 compared to retrofit strategies-1 and 2.

- Displacement ductility decreases more from 5.23 to 2.09 for case-2 and from 7.61 to 3.16 for case-3 for retrofit strategy-3 compared to retrofit strategies-1 and 2.

8.3 Future scope of work

The study in this report is limited to progressive collapse analysis of 4-storey and 9-storey regular moment resistant steel buildings. The present study can be extended to include following points:

- Progressive collapse analysis of irregular building. Different kinds of irregularities like plan irregularity, elevation irregularities etc. can be considered.
- Progressive collapse analysis with semi-rigid connections.
- Progressive collapse analysis with elastic supports to take into account different types of soil conditions.
- Progressive collapse analysis of building during the loss of load bearing wall.
- Progressive collapse analysis of building using applied element method (AEM)
- More emphasis can be given on mitigation measures for progressive collapse of building.
- Different types of structures (like bridges) except buildings can be considered for progressive collapse situation.

Appendix A

List of Papers Published/ Communicated

- Rushi Parikh, "Comparison of Various Guidelines on Progressive Collapse of Steel Structure", A National Symposium - CONTECH'10, Department of Civil Engineering, Nirma University, Ahmedabad, 24-25 September 2010.

- Rushi Parikh and Dr. Paresh V. Patel, "Progressive collapse analysis of steel structure", A National conference, AMAS-2011, Department of civil engineering, Pondicherry Engineering College, Pondicherry, 03-04 February 2011. Page No. 147-156. ISBN: 978-81-920-623-1-0.

- Rushi Parikh and Dr. Paresh V. Patel, "Nonlinear progressive collapse analysis of steel building", Research and Project Fair, RPF-2011, Gujarat Technological University, L.D. College of Engineering Ahmedabad, 12-13 May 2011. (Received 2^{nd} prize)

- Rushi Parikh and Dr. Paresh V. Patel, "Linear and nonlinear progressive collapse analysis of steel structure", International conference, AMTID-2011, Department of civil engineering, NIT calicut, 22-24 June 2011 (Accepted for presentation).

- Rushi Parikh and Dr. Paresh V. Patel, "Effect of retrofit strategies on mitigating progressive collapse of steel structure", International conference, NUiCONE-2011, Department of civil engineering, Nirma University, Ahmedabad. (Abstract communicated)

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