PROGRESSIVE COLLAPSE ANALYSIS OF RCC BUILDING

A DISSERTATION AS MAJOR PROJECT

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE
OF

MASTERS OF TECHNOLOGY

IN

STRUCTURAL ENGINEERING

Submitted by:

NIKITA SHAH

(ROLL No 2K20/STE/15)

under the guidance of

Dr. PRADEEP KUMAR GOYAL
(ASSOCIATE PROFESSOR)

DEPARTMENT OF CIVIL ENGINEERING
Delhi Technological University
(Formerly Delhi College of Engineering)
Bawana Road, Delhi-110042

MAY, 2022

DELHI TECHNOLOGICAL UNIVERSITY
(Formerly Delhi College of Engineering)
Bawana Road, Delhi-110042

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I, Nikita Shah, Roll no. 2K20/STE/15, student of M.tech (Structural Engineering), hereby declare that the project Dissertation titled " **PROGRESSIVE COLLAPSE ANALYSIS OF RCC BUILDING** " which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associate, Fellowship or other similar title or recognition.

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Nikita Shah

DELHI TECHNOLOGICAL UNIVERSITY

(Formerly Delhi College of Engineering)

Bawana Road, Delhi-110042

CERTIFICATE

I hereby certify that the Dissertation as Major Project titled " **PROGRESSIVE COLLAPSE ANALYSIS OF RCC BUILDING**" which is submitted by Nikita Shah, Roll no 2K20/STE/15, Department of Civil engineering , Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Masters of Technology, is a record of the project work carried out by the students under my supervision. To the best of my knowledge and based on student's declaration.,this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere.

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Date : 3.|5|2022

(Dr. Pradeep Kumar Goyal)

Associate Professor

Deptt. Of Civil Engineering

ABSTRACT

Progressive collapse of a structure occurs when a local elemental failure activates successive failure leading to either partial or total collapse of the structure. Prevention of progressive collapse in a structure is a significant issue in the improvement of several design codes. To control successive elemental failures, it is significant to study structural elemental behaviour under removal of column at critical locations. In this project, progressive collapse analysis have been done using a finite element linear static and non linear static anlysis on a 6 storey building. Though there are dynamic methods also for analysis but static method has been used owing to its practicality and simplicity in engineering world. Dynamic method is more complicated and involves number of iteration before it can give any close to accurate result, hence is not much in demand nowadays. The work focuses on progresive collapse analysis of RCC structure under column removal using commercially available computer program ETABS.

A G+5 RCC flat slab building is modelled as per Indian standard and analysed using two important methodologies. The results of linear static analysis have been stated in the form of DCR and the results of non linear static analysis have been stated in the form of support rotation at joints. In both the cases, the structure is not susceptible to progressive collapse both when the central column is removed and also when corner column is removed. The critical coloumns were removed to trigger progressive collapse.

ACKNOWLEDGEMENT

I would like to express my gratitude to Dr. Pradeep Kumar Goyal for his valuable guidance in M.Tech program. He gave me valuable suggestions and mentorship when required. I wish to recognize Department of Civil Engineering of Delhi Technological University for providing the generous support without which my postgraduate studies would not have been possible.

Also thankful to central library, DTU, Delhi. Also my sincere thanks to all others for their support and suggestions during the completion of the work.

Finally, I would like to thank my parents and my friends for their continual support, encouragement and moral support to complete this work for my post graduate years of study.

Date : NIKITA SHAH

Place: Delhi 2K20/STE/15

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LIST OF SYMBOLS

A_h = Design horizontal earthquake acceleration coefficient

g = gravitational acceleration

I= Importance factor

R= Response reduction factor

L= Building dimension in a considered direction

S_a/g = Response acceleration coefficient

T= Undamped natural period of oscillation of the structure

LIST OF ABBREVIATIONS

LSA : Linear Static Analysis

NLSA: Non Linear Static Analysis

LDA: Linear Dynamic Analysis

NLDA: Non Linear Dynamic Analysis

DCR : Demand Capacity ratio

GSA: General Services Administration

LIST OF EQUATIONS

CHAPTER 1

INTRODUCTION

Progressive collapse occurs when a local failure activates successive failure leading to either partial or total collapse of the structure. Progressive failure starts from an elemental failure which in turn is responsible for complete failure of the structure. Famous examples of progressive collapse of such a failure are RONAN POINT residential apartment tragedy in london in 1968 which occured due to natural gas explosion. The famous terrorist attack on WORLD TRADE CENTRE on 9/11 is another example of progressive collapse on any high rise structure. These famous incidents have led to many research activities being conducted on designing and prevention of progressive collapse. The observations and findings have helped in limiting failure by making structure more ductile and redundant.

To deal with the anaysis of the progressive collapse in the structure two approaches have been used which are direct and indirect approach. For first indirect approach, the structure satisfies some prescribed rules of design i.e integration and ductile nature of the structure or the availaibility of horizontal and vertical ties. For second direct approach, it is seen that local failure of the structure is allowed or not. If there is a possibility of local failure, then the evaluation in the structure is done by using alternate load path method by removing load carrying element in the structure. If no local failure possibility is there, important elements in the structure must be strong enough to bear accidental load such as explosion or collisions.

1.1 FLAT SLABS

A flat slab is defined as a beamless slab. It is a two-way reinforced concrete slab, in which loads are directly transferred to the supporting column . It is a framing system utilising a slab of uniform thickness, the simplest of structural slabs. The loads are directly transferred to the supporting concrete columns.

1.1.1 General **-**For the purpose of the clause, the below definition shall apply:

1. Column strip- The design strip in this has a width of $.25l_2$ and not exceeding $.25l_1$ at each side of columns's centre line, where l_1 is the length in where momnets are being determined and l_2 is perpendicular to l_1.

2. Middle strip- It is a design strip enclosed on each of its reverse side by column strip.

(Plain and reinforced concrete-code of practice (fourth revision ,2000)

Panel

It is a slab's part which is enclosed by centre line of column at each sides or at the centre line of the adjacent spans.

1.1.2 Flat slab thickness

The thickness of flat slab is controlled by span to depth ratio. IS456 provision number 23.2. The minimum thickness of slab is taken as 125mm.

1.1.3 Drop

Rectangular plan shall be taken for drop width having length not less than $1/3^{rd}$ of panel length .The width of the drop, perpendicular to the edge which is non-continuous and measured from column's centre line is equivalent to drops width half for interior columns.

1.1.4 Column heads

The portion where heads of the column are provided, it lies within the largst circular cone or pyramid having a vertex angle of 90 degree and can be within the outlines of the column and for design purpose only column head shall be considered.

1.1.5 Shear in Flat slab

From column's periphery or capital or drop at 'd/2' distance, critical section for shear is taken. It is taken as perpendicular to the slab panel, where 'd' is the effective depth of the section.

If shear stress calculated is not provide, the shear stress calculated shall not exceed $.25(f_{ck})^{.5}$ where k_s=1.

1.2 Non-linear seismic response of the structure

The forces acting on the structure for which they are designed must be very less than the seismic forces. When any earthquake forces hits the structure, it deforms inelastically. Even if any collapse doesn't occur in the structure but the damage it causes is beyond repairs. According to IS:13920-1993, in a structural system ductility can be maximised by reinforcing it with the required steel. In the inelastic region, a sufficient amount of ductile structural system undergoes very large deformations. To understand the characterstics of the structures, non-linear dynamic analysis of different SDOF (Single degree of freedom) and MDOF (Multi degree of freedom) having non-linear characterstics is required to be performed. From the results, prediction for collapse is done.

1.2.1 Non-linear Force-deformation behaviour

Linear methods are used to analyse those frame system of structures which have linear damping, linear restoring and linear inertia forces. When the frame structural system has any of the three reaction reactive forces (inertia, damping and stifnesses) having a non-linear relationship with their respective response parameters such as displacements, velocity and acceleration, then a set of non-linear differential equations are set-up. In order to get the correct response, these equations are to be solved. The most common linearity are stiffness and damping. Leaving some cases, inertia forces are generally linear.

The stiffness non-linearity are of two types: (i) Geometric non-linearity (ii) Material non-linearity

For material non-linearity, restoring action depicts hysterical behaviour under tha action of cyclic loading. For geometric non-linearity no such hysterical behaviour is observed.

Damping non-linearity, is generally found in problems realted with dynamics which are related to structural control, offshore structures and aerodynamics of structures. Mostly, damping non-lineariities are of non-hystteretic type. Hence, it is important to discuss material non-linearity exhibiting hysteritic behaviour. For structural system, having linear behaviour (subjected to ground motions weak in nature) of inertial forces, and linear damping characterstics, linear methods of analysis can be employed. The most important and latest method to obtain the response is Newmark's Beta method.

1.2.2 Pushover Analysis

Earthquake is the most catastrophic amongst all the natural hazards. Since, they are the most unpredictable and random in nature, the engineering tools needs to be enhanced for doing the analysis of the structures. Earthquake loads should be carefully taken so as to predict the exact behaviour of the structure to understand the damage and the ways to regulate it. With the advent of performance based design, the non linear static method i.e pushover method is in the forefront.

Pushover analysis is a static non-linear procedurein which lateral loading is increased gradually in accordance with certain predefined pattern. It has been assumed that the fundamental mode controls structure's behaviour and predefined patterns are shown in the form of fundamental mode or story shear. As the magnitude of load increases, the non-linear behaviour of various structural elements is observed, modes of failure and weak links of the structure is identified.

The pushover method is also used to analyse the response spectrum. There have been modifications in the procedure of pushover analysis so as to calculate the contribution of higher modes of vibration of the structure, change in story shear distribution corresponding to yielding of the structure members, etc. With the advent of popular analytical tools like ETABS or SAP2000, pushover has gained significant popularity.

Two types of Pushover analysis are: a) Force controlled b) Displacement controlled

a) Force Controlled Method

In this force is applied in small increments to the structure. It can only be used when the load is known i.e gravity load.

b) Displacement Controlled Method

In this method, displacement of the topmost storey of the structure is gradually increased step by step, such that horizontal force pushes the structure laterally. The displacement due to pushing of the structure is directly proportional to fundamental horizontal translational structural mode.

The structure’s seismic performance can be asessed in terms of displacement ductility, plastic hinge creation, pushover curve, performance point etc. The capacity curve can be converted into capacity spectrum, by SAP2000 using ATC 40 and thus the response spectrum is obtained. The performance point in the structure analysed is obtained by the intersection of demand and capacity spectrum.

The responses of the structure must be checked through certain acceptance criteria at the performance point. The performance point which is assesed from pushover is compared to calculated target displacement.

In ATC 40, three procedures have been explained to determine the performance point.

In Procedure A, set of equations have been defined.

In Procedure B, a recurring method is used to determine the performance point.In Procedure C, which is a graphical method is suitable for both hand and the software analysis. This method is suitable for determining performance point in SAP2000.

Following are the steps to obtain performance point in Procedure C. For each and every point on the pushover curve, demand spectrum curve is drawn.

a) Through point (P) on the curve, a constant period line is drawn by drawing a radial line.

b) The area which is based under the curve up to the point, the damping involved with point (P) on the curve can be calculated.

c) The demand spectrum is obtained by plotting it for same damping level as compared to point (P) on the pushover curve.

d) Point on the Single Demand Spectrum (Variable Damping Curve) is represented by the intersection point (‘P’) for the radial line and associated demand spectrum.

1.3 Objectives

Following are the objectives of the present work done:

1. To perform linear static analysis on a G+5 RCC flat slab building to check for progressive collapse.

2. To calculate Moment and Demand capacity ratio in a structure after removal of column at critical locations.

3. To perform Non-linear static analysis and calculate support rotation at the joints after column removal in a structure.

1.4 Motivation

Buildings may be subjected to any form of extreme events such as a natural disaster like earthquake or any kind of accident which may include explosion due to gas cylinders or any kind of vehicular collision, or bomb blasting due to increased terror attacks and sabotages. Due to the above mentioned events the term ‘progressive collapse’ was coined and studied in detail but not much research has been done on this until now. Also most of the research done is on framed structure incorporating progressive collapse but the focus in this project is also given on value engineering that is designing a robust structure with optimal economic criteria hence flat slab system has been taken instead of framed structure. Flat slab building has advantage over a normal slab framed structure in terms of economy as building height gets reduced, moreover approximately 10% of vertical member is saved, as a result of which foundation load is also reduced which also helps us in cost cutting of the structure without compromising with the strength of the building. Also after such an analysis several design techniques can be developed which will help find an alternate load paths, so that whenever a column fails the load or the stress gets distributed evenly in the remaining members as a result of which building will withstand extreme conditions of terror attack or accident or any form of natural disaster.

CHAPTER 2

LITERATURE REVIEW

This section deals with the literature that has been studied during the making of the project on progressive collapse analysis.

Marjanishvili A. et.al. [3] studied a nine-storey building and analysed for progressive collapse failure. In this paper four methods have been used namely, the linear static, nonlinear static, linear dynamic and finally the non linear dynamic. Each of the methods is deeply investigated and results are evaluated. General Services Administration guidelines have been used throughout the paper with an objective to provide clear concept of the various procedures used and thereby come to the conclusion as to which is the best and the most accurate method of analysis. SAP2000 has been used for the investigation.The conclusion drawn is that dynamic analysis be it linear or non-linear gives more accurate result, though a bit complicated but dynamic analysis includes dynamic factors and yield more and hence accuracy increases for them.It is also shown that the structure designed with allowable design criteria may exceed ductily or rotation.

Qian K. et. al. [4] has taken seven flat slab frame building is casted using glass fiber reinforced polymer as the reinforcement and then the building is tested. Furthermore the failure pattern is studied and the differences are noted down. Test results showed that the above mentioned strengthening scheme significantly increased strength of the structure. However, they were not succeed in enhancing the post failure resistance of the structure and also its deformation capacity: there was a huge loss in bonding when the structure was in its large deformation stage.

P & Solomos G. et.al. [5] studied a flat slab building in which columns were removed suddenly at three different locations and calculation of response is being done.. Also to account for dynamic effects example explosion or impacts, dynamic linear and non linear time history analysis is performed next. Result has been compiled after which susceptibility of the building to progressive collapse failure is checked according to GSA 2003 guidelines. When non-linear analysis was performed no failure was observed but when linear and dynamic produced progressive collapse. Also advantages and disadvantages of various methods have been discussed.

Keyvani L. et.al. [6] studied post-tensioned parking garge and investigated it experimentally and numerically. On removing the interior column behavior was observed for the structure. The structure which a permanent maximum displacement of atleast 60 mm vertically. And for the analytical analysis, model of the garage is developed using a CSI software Etabs and nonlinear dynamic analyses are further done for evaluation.The results concludes that the coloumn sorrounded by slab have no reinforcement. Flexural strength is increased when compressive member forces are developed in the slab and it is pushed down.

Eren N. et. al. [7] studied that very fwe studies has been done on this subject neither experimentally nor numerically. Progreesive collapse can be trigerred by explosions, any natural or man -made disasters. When study of infill walls gets incorporated iwth this subject, the study on the subject gets more narrower. Infill walls are usually considered as non-structural elemnets. It is considered when any earthquake hits the structure for the first time. Their resiistance is not taken into account. This assumption make design and anlysis of the structure very easy. But there are chances of conservative results.This paper presents the results of simulations done to check for progressive collapse on the structure. This was done to check the presence of infill walls in the structure for checking load carrying capacity of the structure. A macro-model has been developed and its effectiveness was investigated analysied by comparing its reuslts with the experimental results. After validation of the results, the above mentioned model was further use to check behavioural changes in the structure. For comparing resistance and lost energy,bare frames are also being invetstigated.

Parisi F. et.al. [8] studied progressive collapse on RCC building designed using Eurocode is numerically investigated considering failure of columns one after the other. The study is focused on a RC frame which is taken as benchmark in used in a previous investigation on a loss of a single column, using non-linear fibre based capacity modelling and incremental dynamic analysis. It depicts that the instant loss of two adjacent columns can reduce the load capacity drastically which results in the progressive collapse of the structure. A successive loss of columns induced negative or positive variation in load capacity, depending on the ratio between removal times, whereas drift capacity is reduced significantly.

Weng Y. et. al .[9] studied investigation of the load or stress distribution while subjecting the structure, to a central coloumn scenarios. Furthermore validation of the numerical model is done with the help of experiments. It is also noted that the continuous surface cap model (CSCM) incorporated with an erosion criterion keeping in mind both the maximum principal and shear strain will definitely and precisely predict the two way shear failure at all the slab-coloumn connections.In order to know the effect of boundary conditions the validated finite element model was developed, reinforcement amount, thickness of the slab on the load distribution tank of the structures. The sub-structure were designed to capture the load distribution behaviour of flat slab structure. Ignoring the constraints from sorrounding slabs may undermine the load redistribution capacity of flat slab substructures.It is recommended that the future experimental or numerical studies, rigid horizontal constraints must be applied at the slab edgeof the superstructure to well represent the constrainsts from sorrounding slabs. It is also noted that the amount of reinforcement would significantly affect the post-punching performance of flat slab structure.

Azim I. et.al.[10] studied the factors which influence progresive collapse resistance of the strcuture is taken into account such as beam dimensions. Those factors which influence the resistance of the progressive collapse, are identified and are discussed. A comprehensive data of experimental results related to progressive collapse of RC beam and RC column is compared through a broad literature review. Dynamic behaviour of the structure is also studied in this paper. Numerical modelling is also done in this paper. In the beam mechanism stage. CAA capacity of specimens can be enhanced if we increase the bottom reinforcement and delay the bottom fracture of the bars. Corners are given special attention, since interior column failure is taken as the penultimate column faiures.

Kwasniewski L. et.al. [11] studied **p**rogressive collapse of the multi-story building . An eight storey steel framed building is studied for a test of fire located in United Kingdom. A finite element non-linear dynamic analysis is carried out . The paper centres around development of global models with increasing vertical loading and column removal. The outcome of analysis was positive for all the three cases . The calculation should be repetitive for various parameters to visualse the probabilistic nature of the input, for example, failue strain for bolts and for concrete slabs under tension. However, probabilstic approach like this should be firmly established on the experimenal validation.

Izzuddin, B. A. et.al. [12] studied the assessment of multistoreyed structure, with instantaneous column loss as a main criterion. This offers a pragmatic means for examining robustness of structure at varied levels of idealisation. A crucial characteristic of this new approach is its capacity to accommodate simplified and detailed models of non-linear structural response, with the added advantage of permitting increasing assessment over consecutive levels of structural idealisation. This study clearly thrwos light on the suitability of the measures and indicators of robustness of the structures. This paper highlights the approach to progressive collapse determination of real single framed composite structure reulsing in the intrinsic robustness of the structure and use of current provisions.

Vlassis, A. G. et.al., [13] studied the principles of a methodology which is design-based for progrssive collapse evaluation of various structures. The suggestive procedure, assesses ductlity demand with supply in determining the potential for progressive collapse initiated by sudden loss of a support member vertical in nature. This paper primarily gives the suitability of the approach through a case study, by instantaneous removal of a ground floor column in a steel-framed composite structures. The study determines that such buildings/ structures can be susceptible to progressive collapse, more specifically because of collapse of the support joints of internal secondary beam to safely transfer the gravity loads to the undamaged members of surrounding if a flexible pin-plate joint detail is employed.

CHAPTER 3

MODELLING AND ANALYSIS

This chapter embodies tha main work done in the project including the procedures involved, plan and the section of the building investigated and the major data obtained from the analysis.

3.1 Plan and Section

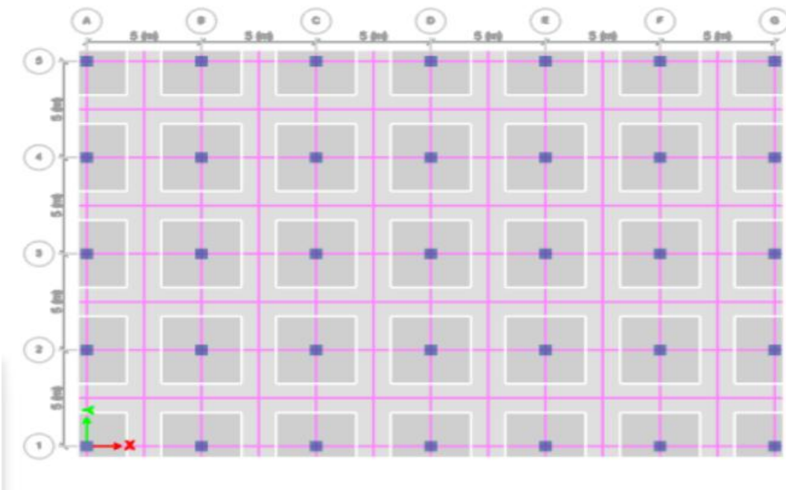

Figure: 3.1 Plan of the building

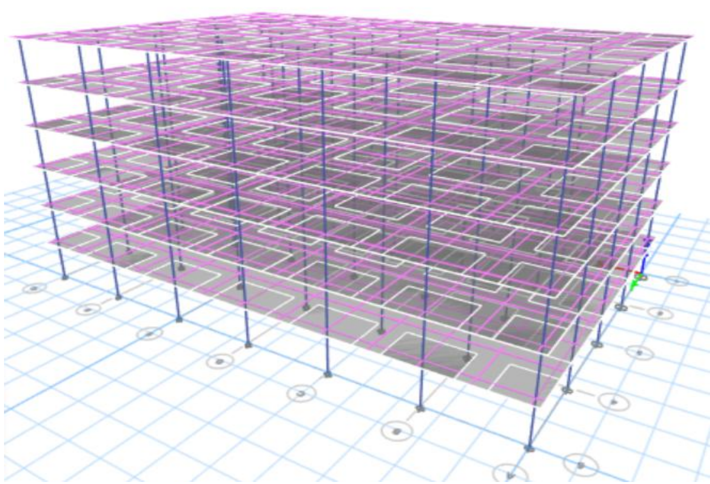

Figure: 3.2 3D image of the RCC buiding

3.1.1 Structural Data

3.1.1.1 General data of the building

a) Modelling details

The building in the project consists of 364 joints, 209 frames, 6 shells and 24 edges. Frame section details are as shown along with the reinforcement details. Columns are symmetric and reinforcement is 2% of the gross area in the columns which is well below the maximum limit.

Table 3.1 General data of the building

Type of the Structure	**6 storey RCC Building**
Zone	IV
Floor to Floor height	3m
Slab thickness	200mm
Live load	$3KN/m^2$
Floor Finish	$1KN/m^2$
Column sizes	550mm*550mm

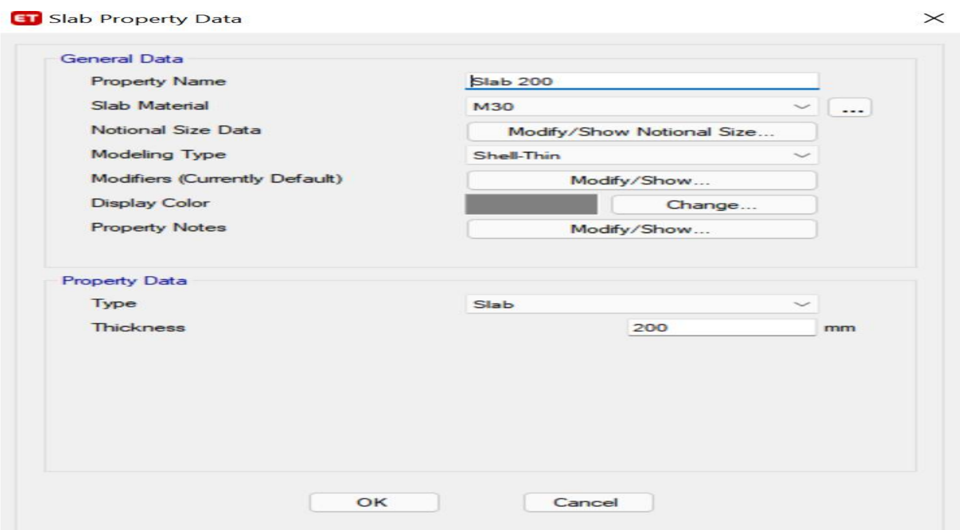

Figure 3.3 Slab property details

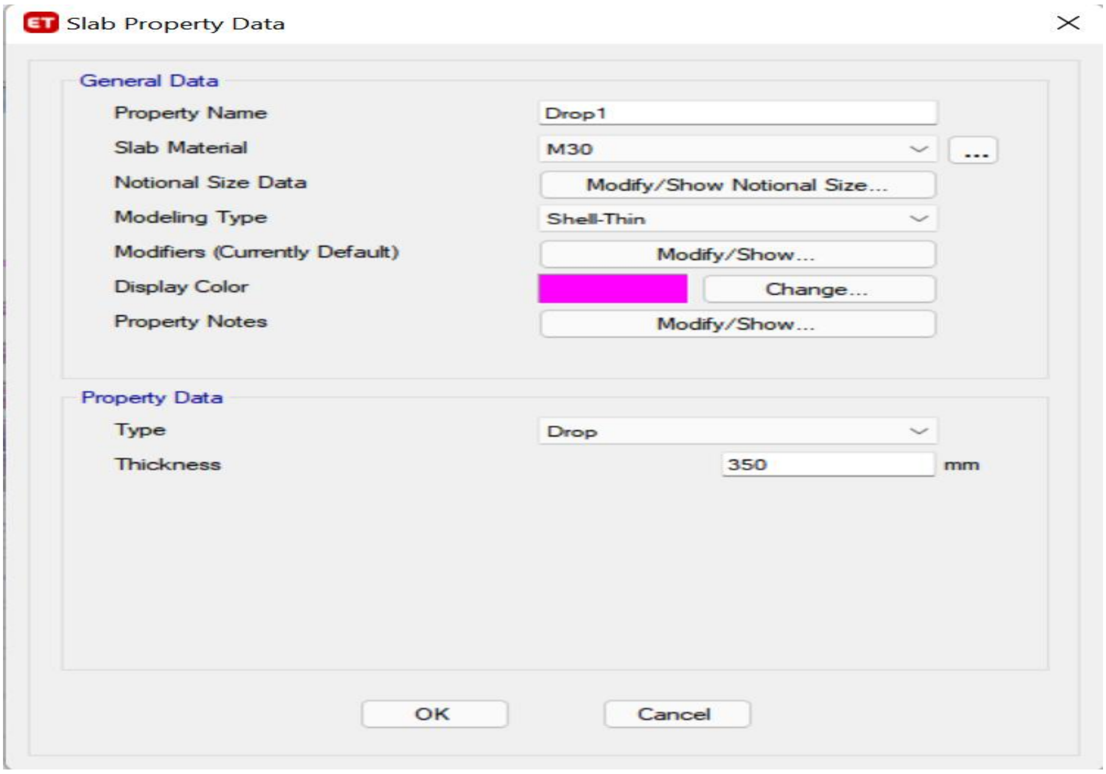

Figure 3.4 Drop panel property details

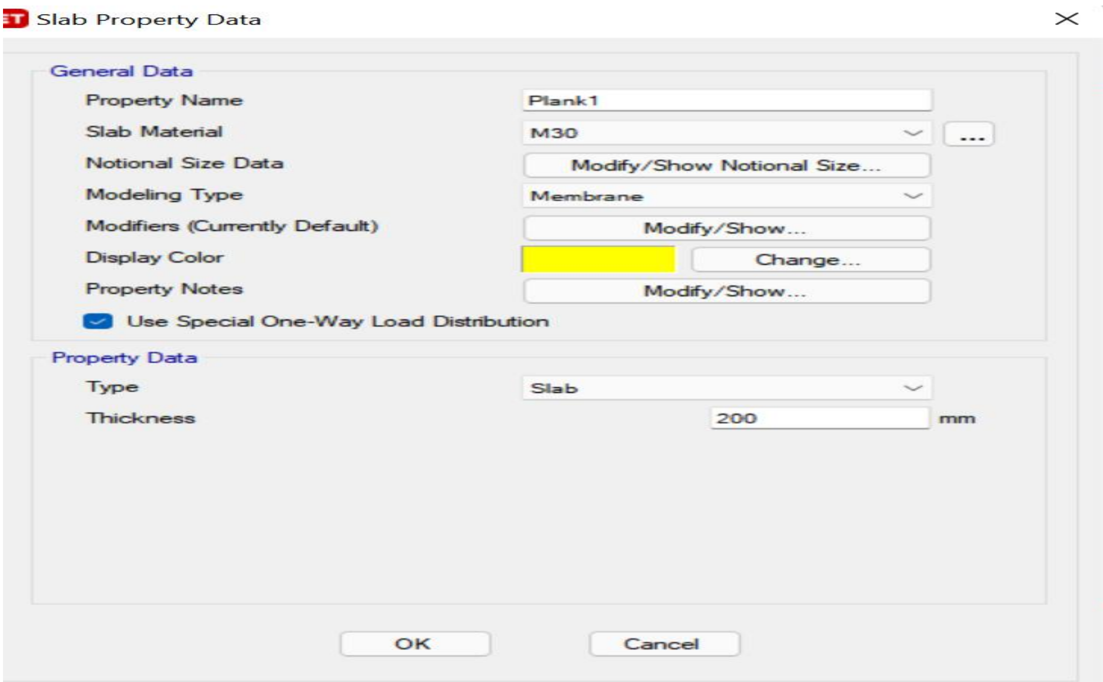

Figure 3.5 Slab properties details

The above table and figures gives the details of the building taken during the modelling and designing of the building. The above pictures shows the column, slab and reinforcement details used in modelling and analysis.

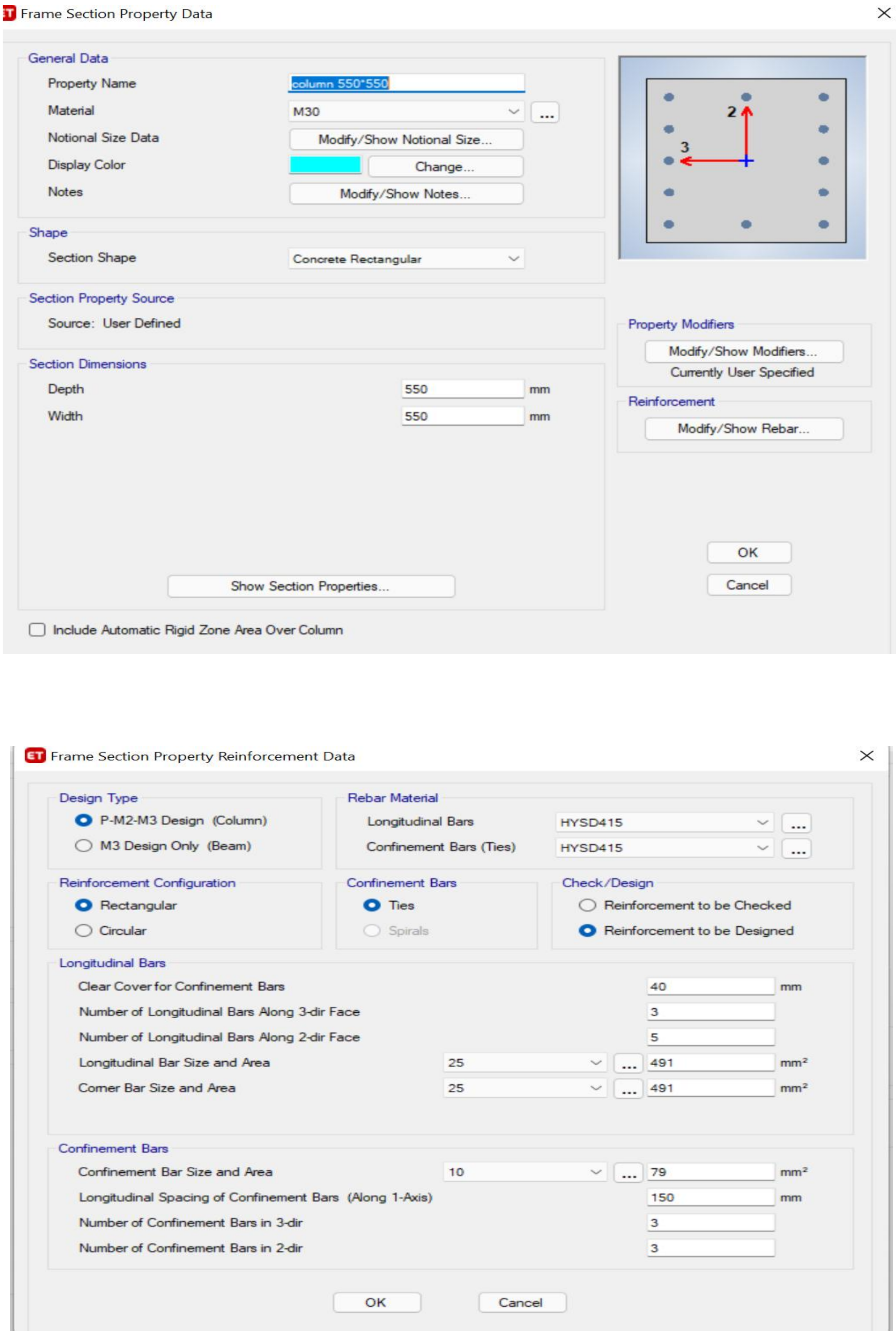

Figure 3.6 a) Column details b) Reinforcement details

3.1.3 Earthquake force calculations and Imposed loads

For the different types of loading case mentioned in the code IS 875 (part 2), design seismic force is calculated by taking the full percentage of dead load (DL) and some percentages of imposed load as mentioned in the following table:

Table 3.2 Percent of imposed load while calculating seismic weight

Floor loads (KN/m^2)	**Percent of Imposed load**
Equals to or less than $3KN/m^2$	25%
More than $3KN/m^2$	50%

3.2 Analytical Work

To analyse the progressive failure in a building, there are four methods namely,linear static analysis, non-linear static analysis, linear-dynamic analysis, non-linear dynamic analysis. Out of the above mentioned methods dynamic methods are precise

than the static methods as the dynamic analysis inherently incorporates the dynamic amplification factor. Moreover, higher modes of vibration effect in a building is calculated. The only disadvantage associated with this type of analysis is that they are complex and time consuming as it involves a number of iterations. Also the structure yield pattern is difficult to decipher. Linear methods are not very precise but ther are very simple and gives close results. Linear static methods include equivalent static method and non-linear static method include pushover analysis.

3.2.1 Material Properties

Following are the material properties used in the entire project.

Table 3.3 General properties of material

Grade of Concrete	M30
Rebar	HYSD415
Diameter of reinforcement	25mm
Modulas of Elasticity	2×10^{5}

3.2.2 Response limits

Based on GSA 2003 guidelines, following table shows the respnose limits of the linear and Non-linear procedure.

Table 3.4 Table of response limits

Linear Analysis	Non-linear Analysis
DCR$\leq$ 2	Support Rotation $\leq 6°$

a) Demand capacity ratio

Structural resistances of a building is seen by means of demand resistance ratio. A local DCR/DRR is calculated at critical sections as given under:

$$DCR/DRR=Mmax/Mr(N) \qquad (1)$$

M_{max} = Maximum B.M in a section

$M_r(N)$= combined Moment and axial resistance obtained from interaction diagram

This formula is used in case of columns since combined bending moment and axial force is taken in such a case. Here, Mmax is the maximum B.M acting on the section whereas Mr is the value of the moment obtained from the corresponding interaction diagram.

b) Support Rotation – It is the angle that the horizontal line makes with the tangent drawn at the maximum deflected shape of the structure.

c) Ductility

Ductility canbe defined as the ability of the material to deform inelastically without much loss in stiffness and strength. It can also be defined as the ‘ maximum inelastic deformation to elastic deformation’.

d) Interaction curve

Combination of the moment and axial load, this causes possible failure to the combination.The above curve is generated in accordance to the strain compatibility. Also known as the axial force/Biaxial-moment interaction surface. The plane is rotated in 3 dimensions.

3.3 Methodology

These are the four methods which are used for the analysis of the progressive collapse in the structure.

i. Linear static analysis
ii. Non-linear static analysis
iii. Linear dynamic analysis
iv. Non-linear dynamic analysis

Two methods namely linear static and non-linear static analysis are used in this study.

3.3.1 Linear Static Analysis

It is a simplest and the commonly method used for the analysis of the structure. This is achieved by using DAF i.e Dynamic amplification factor to change demand of linear static analysis into non-linear dynamic resistance. GSA 2013 guidelines makes it compulsary to follow the following loading combinations for the analysis of the progressive collapse of the RC frame structure using Linear static analysis.

$$\text{Load} = 2\times(DL + .25LL) \qquad (2)$$

Where, DL is the dead load. This load is automatically by default generated while modelling of the structure is done. And LL is the live load which is given to the structure.

Procedure

This is performed using dynamic amplification factor of 2 in combination with service loads, such as DL and LL if applied staticaally. This is the most easiest method to perform and apply. Linear static method can only be applied to simple and small structures. This method is only applied where dynamic effects can be easily predicted without much calculations. DCR gives the response of the structure, which according to General service administration (GSA) should be less than 2 and according to percentage 200%. Following are the steps followed in the analysis.

1. Building a finite element computer generated model.

2. The static load combination with the amplification factor as defined by load combination equation.

3. Linear static analysis is performed in ETABS.

4. DCR is calculated at various critical locations such as corner and middle locations.

Time of analysis is approximately 3 min, which is why it is one of the favourite methods used in engineering owing to its simplicity and ease. But this method also has certain limitations that are clearly mentioned in the GSA guidelines.

3.3.1.1 Limitations of Linear Static Method

This method is limited to structures that are 10 strories or less and which fulfill the requirements of Demand capacity ratio and irregularities. " Linear static method cannot be used for structures which are irregular and in this case also it is not important to calculate DCR. But, if the structure is irregular then it is important to calculate DCR and It should come less than 2. If the structure has irregularity and DCR comes out to be more than 2 then in that case it is not Linear static method is not valid.

3.3.1.2 Irregularity limitations

In the lateral force resisiting system and gravity load sysytem, important discontinuitoes exist. For all load bearing walls only leaving the corners, at each storey of the building or the structure the ratio of wall stiffness to strength is less than 50%. The lateral laod resisting sysytem or elements horizonatal in nature are not parallel to major orthgonal axes of the lateral load resisting system such a sdue to skewed or curved moment frames and walls which are bearing loads.

3.3.2 Non-linear Static Analysis

In this method, a non-linear static analysis inddicates a step wise increase of amplified factor of 2 until the whole structure collapses also called 'push over analysis'. In many cases pushover analysis is load controlled one,to analyse progresive collapse,performance under incremented service loads is performed.

Procedure

This method is used where dynamic effects can be easily identified. This involves various steps:

1. Build a finite element computer model.

2. Defining and assigning non linear plastic hinges to the columns.

3. Now, perform non-linear stattic analysis avalaible in Etabs.

4. Calculate the maximum controlled and various rotation and ductility values. Non-linear analysis a very complicated process which depends on various parametrs such as load-step.

3.3.2.1 Geometric Non-linearity

To improve the overall response of the structure, geometric non-linearity is incorportaed in the structure. This is because large displacements . Moreover, these large deformations can cause failure because of excessive ductility. In our project we have assumed the connections to be pinned according to GSA guidelines. At a very large support rotations in a beam-to-column connection having large tension component in thr horizontal direction is very uncertain and experimental investigation to verify its capacity. For that reason, catenary action effects has not been taken into account.

3.3.3.2 Limitations of NLS Analysis

The disadvantage of this method to not include the effects of dynamics in it and also the fact that it is an exhausting process. This method is useful for determining elastic and failure limits and can complement the non-linear dynamic procedure.

3.4 Types of Non-linearity

(i) Geometric Non-linearity

Strain displacement relations in solids have geometric non-linearity. This can occur due to enormous displacements, rotations, and so on. One more geometric non-linearity is there that is contact non-linearity. This is called geometric non-linearity beacuse of the contact area is a function of a deformation.

One more type of non-linearity also called P-delta effect which includes equilibrium compatibility relationships of a structural system loaded along its deflected configurational sysytem. The application of gravity load on the laterally displace multi -storey building structire. Story drift gets magnified with certain mecahnical behaviours while reducing deformation capacity.

(ii) Material Non-Linearity

It involves non-linear behaviour of a material on the basis of deformation or displacement, history of the deformation, temperature, pressure etc. It occurs when any material within the structure reaches strains that are past yielding of the material. When material goes beyond the yield limit the relationship between stress strain becomes non-linear as the material deforms permanently. Rubber materials can be approximated by a nonlinear, reversible (elastic) response.

(iii) Contact Non-linearity

Contact non linearity occurs when, due to the deformation of one or more parts in contact (pushing or pulling on other) produces a deformation leading to a change in the geometry of the part that translates into a change on K or on the forces (action and reaction) between the parts in contact.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Results for linear static method

This section deals with the results generated for linear static method.

4.1.1 IS1893:2016 Auto Seismic load calculation

Etabs automatically generates lateral loads for load pattern eqx. This is according to IS1893:2016.

(i) Direction and Eccentricity

Direction=X

(ii) Structural Period = Program calculated

(iii) Factors and coefficients

Table 4.1 Factors and coefficients

Seismic zone factor, Z [IS Table 3]	Z=0.36
Response Reduction factor, R[IS Table 9]	R=3
Importance factor, I [IS Table 8]	I=1
Site type [IS Table 1]	II

(iii) Seismic Response

Table 4.2 Calculation of seismic response in X-direction Seismic acceleration

Seismic Acceleration Coefficient	S_a/g= 1.36/T	S_a/g =1.943229

Seismic acceleration coefficient is response acceleration coefficient for 5% damping based on appropriate natural periods. For different damping values, different sesimic acceleration coefficient is being used.

(vi) Calculated Base shear

Base shear: Due to seismic forces, base shear gets generated which is the force of inertia generated by buildings to resist the earthquake forces.

Table 4.3 Base shear calculation

Direction	**Period used**	**W(KN)**	**V_b(KN)**
X	0.7	31391.1273	3660.0088

(vii) Applied storey forces

Storey shear forces are the lateral forces due to seismic activity at different floors. Base shear is the summation of all the shear forces acting at the different floors of the building. Applied storey forces displays generated story loads for seismic, wind and user defined lateral load cases. Available load cases are tabulated in the Applied Story Forces diagram. it also displays gravity loads defined on semirigid diaphragms with two-way slabs.

Table 4.5 Lateral forces in X-direction

Storey	**Elevation**	**X-direction (KN)**	**Y-direction (KN)**
Storey 6	18	1380.41	0
Storey 5	15	1036.2216	0
Storey 4	12	663.1818	0
Storey 3	9	373.0398	0
Storey 2	6	165.7955	0
Storey 1	3	41.3602	0
Base	0	0	0

2.For load pattern eqx+e

(i) Direction and eccentricity

Direction =X+Eccentricity Y

Eccentricity ratio= 5% for all diaphragms

(ii) Factors and coefficients

Z=0.36, R=3, I=1

(iii) Seismic response

S_a/g =1.36/T

S_a/g=2.386171

(iv) Calculated Base Shear

Table 4.6 Base shear calculation

Direction	Period used (sec)	W(KN)	V_b (KN)
X+ ecc Y	0.7	31391.1278	4494.2755

(v) Applied storey forces

Table 4.7 Lateral force in X-direction

Story	Elevation	X-direction(KN)	Y-direction (KN)
6	18	1695.0622	0
5	15	1272.4192	0
4	12	814.3483	0
3	9	458.0709	0
2	6	203.5871	0
1	3	50.7879	0
Base	0	0	0

3.For load pattern eqx-e

(i)Direction and eccentricity

Direction = Multiple

(ii)Eccentricity ratio = 5% for all diappragms

(iii)Structural period is program calculated

(iv)Factors and coefficients

Z=0.36, R=3, I=1

(v)Seismic Response

S_a/g= 1.67/T, S_a/g= 2.386071

Table 4.8 Base shear calculation

Direction	Period used	W(KN)	Va(KN)
X-eccY	0.7	31391.1278	4494.2753
Y-eccX	0.713	31391.1278	4408.5028

Storey shear forces are the lateral forces due to seismic aactivity at different floors. Base shear is the summation of all the shear forces acting at the diffrent floors of the building.

Table 4.9 Lateral force in X-direction

Story	**Elevation**	**X-direc**	**Y-direc**
6	18	1696.0	0
5	15	1272.4	0
4	12	814.34	0
3	9	458.07	0
2	6	203.58	0
1	3	50.7879	0
Base	0	0	0

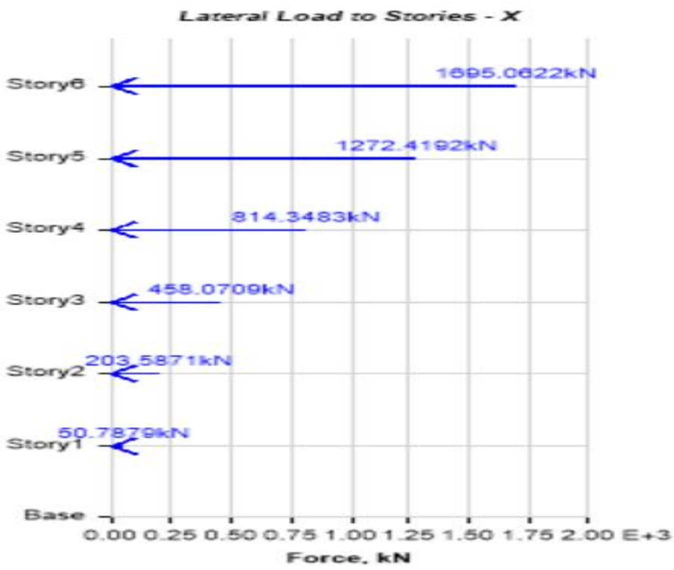

Figure 4.1 Lateral Storey shear forces in X-direction acting at different storeys

V=∑F, ∑F is the overall force acting at the base of the structure due to seismic forces .

4.1.2 Automatic load generation in Y- direction, as generated by ETABS

(i)Direction and Eccentricity

Direction=Y

(ii)Structural time period is program calculated

(iii)Factors and coefficients

Z=0.36, I=1, R=3, Site type=III

(iv)Seismic Response

Table 4.10 Base shear calculation

Direction	**Time period used**	**W(KN)**	**Vb(KN)**
Y	0.713 sec	31391.1278	4408.5027

Spectral acceleration Coefficient, $S_a/g=1.67/T$, $Sa/g=2.340$

(v)Applied storey forces

Table 4.11 Lateral force calculation in Y-direction

Storey	Elevation(m)	X-direction (KN)	Y-direction(KN)
6	18	0	1662.7121
5	15	0	1248.1352
4	12	0	798.8055
3	9	0	449.3287
2	6	0	199.7016
1	3	0	49.8186
Base	0	0	0

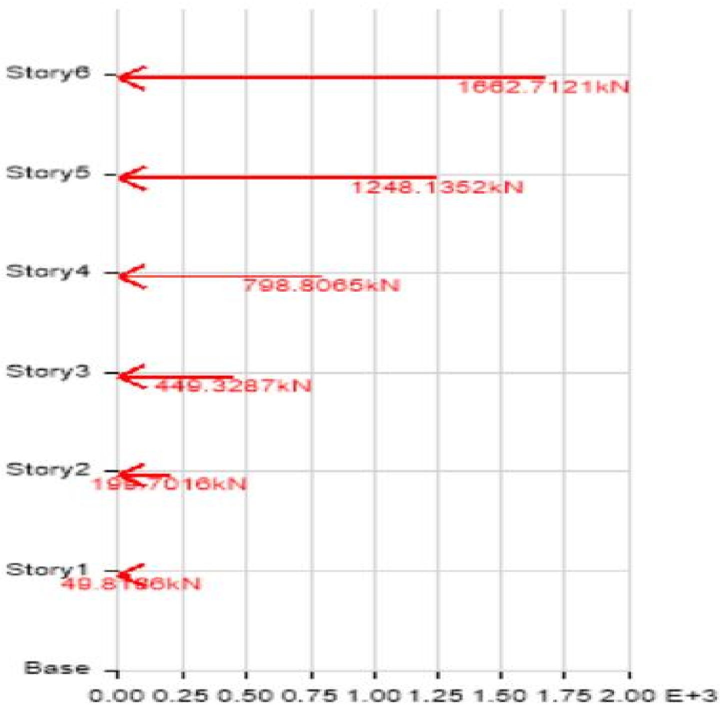

Figure 4.2 Lateral Storey shear forces in Y-direction acting at different storeys

4.1.3 Structural Results in X, Y, Z directions

The table given below presents the result of force and moment in X, Y, Z direction.

Table 4.11 Structural analysis results

Load case/combo	**F_X (KN)**	**F_Y (KN)**	**F_Z (KN)**	**M_X (KN)**	**M_Y (KN)**	**M_Z (KN)**
Dead	0	0	33706.30	337289.8	-505594.5	0
Live	0	0	5788.8	57888	-86832	0
eqx	-3660.0088	0	0	0	-52825.09	36600.96
Eqx+e	-4494.2755	0	0	0	-54866	-49707
Eqx-e	0	0	0	63628	0	40179

eqy	-4494.2755	-4408.5	0	63628	0	0
Eqy+e	0	-4408.5	0	63628	0	0
Eqy-e	0	0	0	0	0	0
L.S	-4494.2755	0	0	703523.64	-1054606	0
D+.25L	0	0	0	351761.82	-527302	0

4.1.4 Modal Results

Table 4.12 Time period for various modes

Mode	**Period**	**Freq(cycl/sec)**	**Circfreq(rad/sec)**	**Eigen value(rad^2/sec^2)**
1	0.713	1.402	8.8064	77.519
2	0.7	1.429	8.9777	80.699
3	0.658	1.519	9.5422	91.0537
4	0.227	4.406	27.863	766.037
5	0.224	4.467	28.0651	787.6502
6	0.21	4.772	29.9819	898.916

7	0.127	7.887	49.5561	2455.8113
8	0.126	7.957	49.9971	2499.7144
9	0.117	8.57	53.8434	2899.1528
10	0.085	11.791	74.086	5488.7427
11	0.084	11.862	74.543	5555.312
12	0.077	12.908	81.1006	6577.3144

Discussion : The above table depicts the results of the structure's frequency (or frequency of resonance) at which it vibrates naturally. Above given are the results of the modal analysis. Mode 1 has a time period of 0.713 seconds and then it decreases for other modes. Since, live load is also taken into account, therefore time period is slightly more than 0.6 seconds.

4.1.5 Point displacements and drift

This section deals with the displacement and drifts at the point of removed column which is also considered to be the most the critical sections. Following is the respective table:

a) Central column removed

Table 4.13 Point displacements and drift Case-1

Type of loading	**X(mm)**	**Y(mm)**	**Z(mm)**
D.L	0	-0.035	-3.876
D+.25LL	0	-0.036	-4.027

Linear-static case	0	-0.073	-8.053

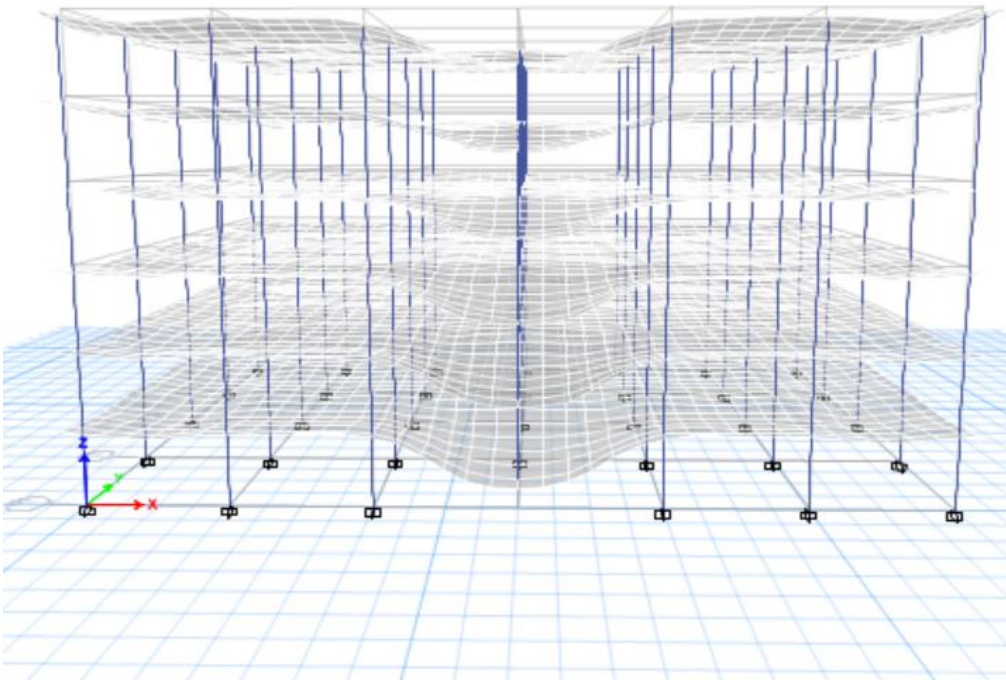

Figure 4.3 Deformed shape on removing central column

Discussion: Displacements generated above in the linear static cases is just the double of that generated in the D+0.25L cases, hence we can say that the values are up to the mark. Further the values of displacements are minor so the building is not susceptible to progressive collapse failure.

b) Corner column removed

Table 4.14 Point displacement and drift Case-2

Type of loading	X(mm)	Y(mm)	Z(mm)
Dead	-0.022	0.041	-4.863
D+.25L	-0.023	0.043	-4.803

Linear static	-0.045	0.085	-9.606

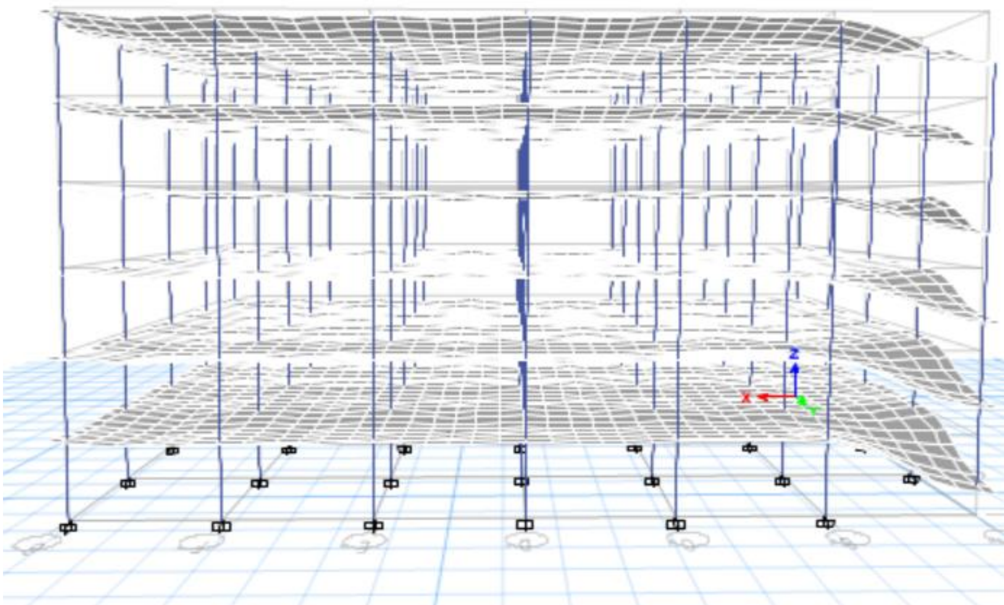

Figure 4.4 Deformed shape on removing corner column

Discussion: Displacements generated above in the linear static cases is just the double of that generated in the D+0.25L cases, hence we can say that the values are up to the mark. Further the values of displacements are minor so the building is not susceptible to progressive collapse failure.

4.1.6 Interaction curve

Following section deals with the M2 values from the interaction curve generated respectively.Interaction curve provides the reprentation of Bending moment (M_U) and axial forces (P_U) visually. The curve also reprents ultimate compression memeber capacity when subjected to axial forces and bending moments. Interaction curve for different shapes column is used to calculate moment and forces.

Table 4.15 Interaction curve data when central column is removed

Point	P(KN)	M2(KN-m)
1	5576.05	0
2	5576.0464	56.2706
3	5272.54	88.8264
4	4479.95	102.9359
5	3522.5491	120.224
6	2392.08	144.67
7	1186.90	145.944
8	6.5048	145.2381
9	-1171.7821	143.0381
10	-2294.8377	70.703
11	-2639.6126	0

Here, the maximum value of moment is taken as **145.944 KN-m.**

Discussion : Moment values from the interaction data are taken keeping in mind the optimal axial force as well as moment hence for the central column removal case 145 KN-m is taken as the maximum value.

^

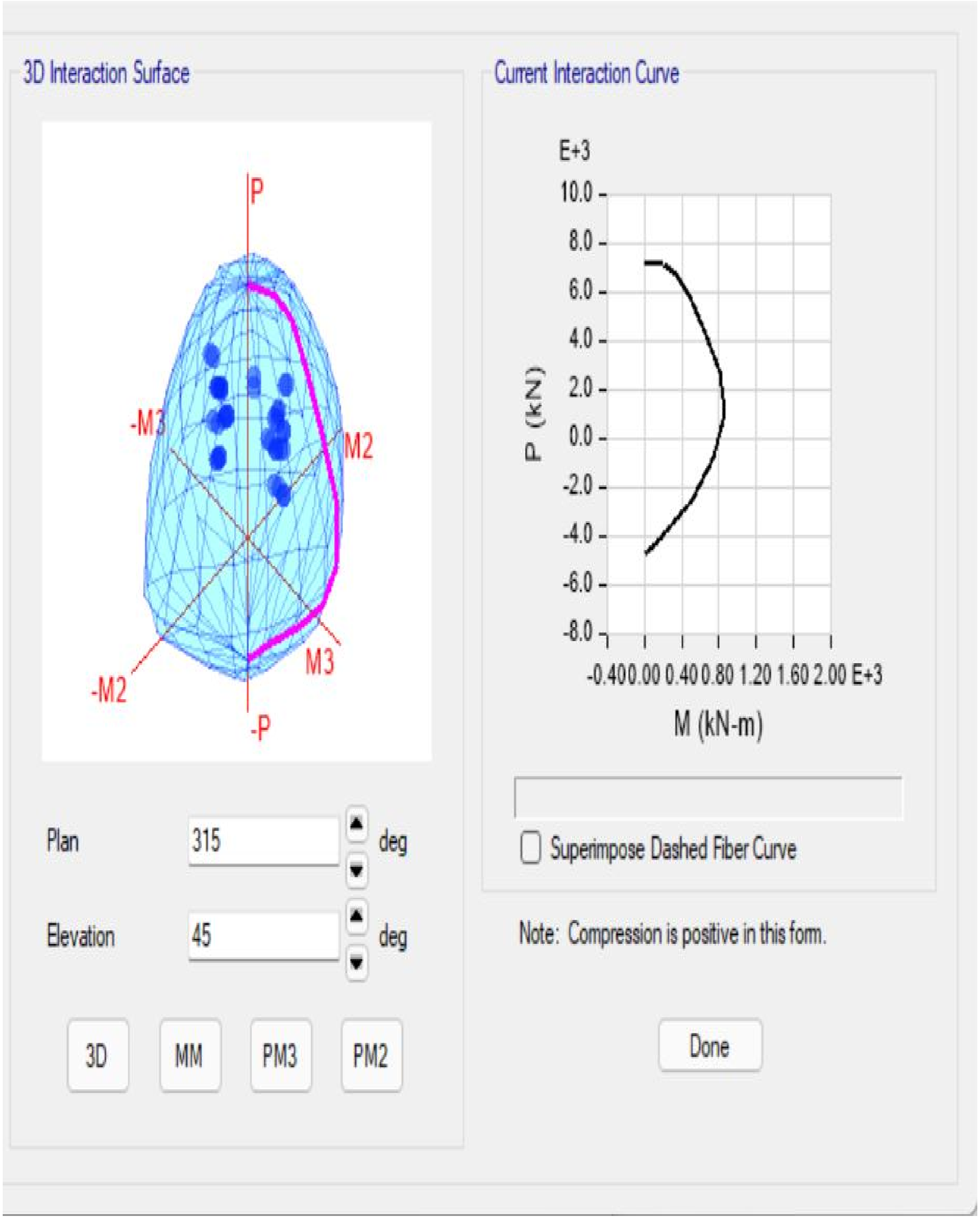

Figure 4.5 3D interaction surface and Interaction curve

Table 4.16 Interaction curve data when corner column is removed.

Point	P(KN)	M2(KN-m)
1	5665.1503	0
2	5665.1503	57.8144

3	5357.9987	90.7153
4	4552.057	105.4621
5	3573.7929	123.5417
6	2414.5446	149.1099
7	1176.3643	150.44
8	-35.8963	149.7011
9	-1246.0652	147.7011
10	-2405.069	72.9616

Discussion: Moment values from the interaction data are taken keeping in mind the optimal axial force as well as moment hence for the central column removal case 145KN-m is taken as the moment whereas for the corner column removal case 150.44 KN-m is taken as the desired value.

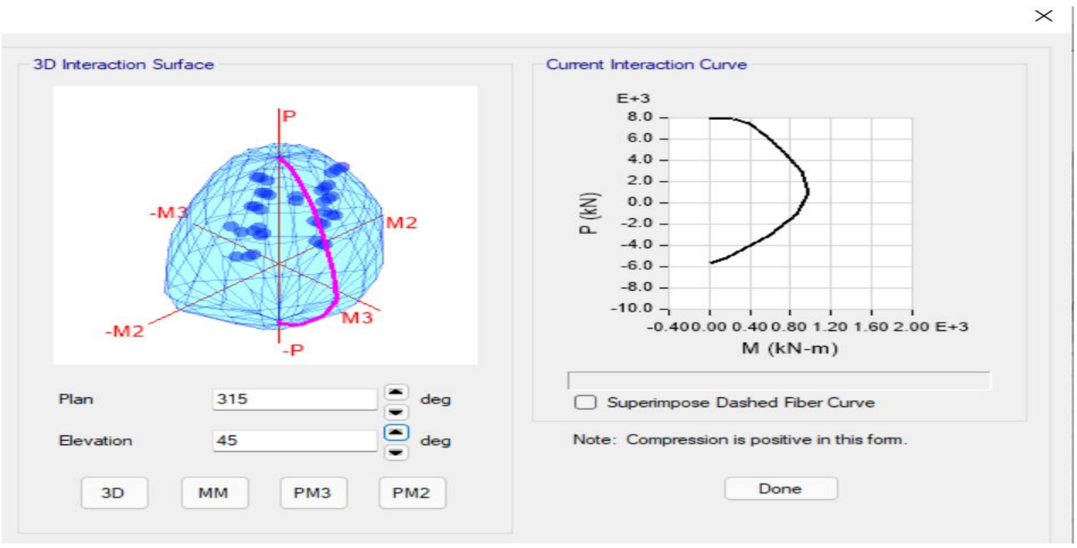

Figure 4.6 3D Interaction surface and Interaction curve

4.1.7 Demand Capacity ratio

a) Central Column Removed

Table 4.17 Moment and DCR value for central column removal

Number of storey	Moment of 1st row 4th column (KN-m)	DCR	Moment of 1st row 3rd column	DCR
6	T= -15.4721 B= 14.4191	0.106 0.099	T= 56.1310 B= -42.7518	0.386 0.294
5	T= -2.7341 B= 3.5235	0.0188 0.024	T= 32.6097 B= -34.7613	0.2245 0.239
4	T= -2.4278 B= 2.4585	0.0167 0.0168	T= 36.2049 B= -35.5632	0.249 0.244
3	T= -2.2954 B= 2.3734	0.015 0.0163	T= 34.4943 B= -33.9149	0.237 0.2335
2	T= -1.7191 B= 0	0.0118 0	T= 35.0196 B= -37.2290	0.2411 0.256
1	T=0 B=0	0 0	T= 23.8040 B= -13.0450	0.163 0.089

Discussion **:** According to (GSA 2003), for progressive collapse to occur in the structure ,DCR ratio should be less than or equal to 2 i.e DCR≤2 . Here, DCR for columns at for central column removal is coming out to be less than 2 for both the cases. Hence, the building is not vulnerable to progressive collapse.

b) Corner column removed

Table 4.18 Moment and DCR values of corner column removed

Number of storey	Moment of 1st row 6th column (KN-m)	DCR	Moment of 1st row 7th column (KN-m)	DCR
6	T= -47.1405 B =41.7975	0.313 0.278	T=34.4973 B=-26.4872	0.229 0.176
5	T=-30.6304 B=33.4909	0.204 0.222	T= 21.4239 B=-23.1532	0.142 0.153
4	T= -35.3760 B= 34.7609	0.235 0.230	T=25.7882 B= -26.9057	0.17140 .1788
3	T= -33.6853 B= 33.3261	0.222 0.221	T=26.3408 B= -24.9057	0.175 0.1655
2	T= -34.7382 B= 37.7249	0.230 0.25	T= 38.4014 B= -56.1164	0.255 0.373
1	T= -23.4354 B= 13.3626	0.155 0.088	T=0 B=0	0 0

Discussion : According to (GSA 2003), for progressive collapse to occur in the structure ,DCR ratio should be less than or equal to 2 i.e DCR≤2 . Here, DCR for corner column removal is coming out to be less than 2 for both the cases. Hence, the building is not vulnerable to progressive collapse.

4.2 Results for non-linear static method

In Non-linear static case, to avoid collapse , the support rotation should be less than 6 degree i.e $\theta \leq 6°$.

4.2.1 Support rotation and displacements a) When corner column removed

Table 4.19 Support Rotation and Displacements

Story	**Load case/combo**	**Diaphragm**	**U_X (mm)**	**U_y (mm)**	**R_z (rad**	**X m**	**Y m**	**Z m**
6	Dead	D1	-0.499	0.476	-1.4e-05	15	18	10
6	Live	D1	-0.061	0.059	-2e-06	15	18	10
6	eqx	D1	0	0	0	15	18	10
6	Eqx+e	D1	0	0	0	15	18	10
6	Eqx-e	D1	0	0	0	15	18	10
6	eqy	D1	0	0	0	15	18	10
6	Eqy+e	D1	0	0	0	15	18	10
6	Eqy-e	D1	0	0	0	15	18	10
6	NLS	D1	-1.276	1.22	-3.5e-05	15	18	10
6	D+.25L	D1	-0.499	0.476	-1.4e-05	15	18	10
5	Dead	D1	-0.403	0.384	-1.1e-05	15	18	10
5	Live	D1	-0.05	0.047	-1E-06	15	18	10
5	eqx	D1	0	0	0	15	18	10
5	Eqx+e	D1	0	0	0	15	18	10
5	Eqx-e	D1	0	0	0	15	18	10
5	eqy	D1	0	0	0	15	18	10
5	Eqy+e	D1	0	0	0	15	18	10
5	Eqy-e	D1	0	0	0	15	18	10
5	NLS	D1	-1.035	0.987	-2.8e-05	15	18	10
5	DL+.25L	D1	-.403	0.384	-1.1e-05	15	18	10
4	Dead	D1	-.31	.295	-8E-06	15	18	10

4	Live	D1	-0.038	0.035	-1E-06	15	18	10
4	eqx	D1	0	0	0	15	18	10
4	Eqx+e	D1	0	0	0	15	18	10
4	Eqx-e	D1	0	0	0	15	18	10
4	eqy	D1	0	0	0	15	18	10
4	Eqy+e	D1	0	0	0	15	18	10
4	Eqy-e	D1	0	0	0	15	18	10
4	NLS	D1	-0.798	0.759	-2.2e-05	15	18	10
4	D+.25l	D1	-0.31	.295	-8e-06	15	18	10
3	Dead	D1	-.216	.205	-6e-06	15	18	10
3	Live	D1	-0.027	0.025	-1e-05	15	18	10
3	eqx	D1	0	0	0	15	18	10
3	Eqx+e	D1	0	0	0	15	18	10
3	Eqx-e	D1	0	0	0	15	18	10
3	eqy	D1	0	0	0	15	18	10
3	Eqy+e	D1	0	0	0	15	18	10
3	Eqy-e	D1	0	0	0	15	18	10
3	NLS	D1	-0.553	.53	-1.5e-05	15	18	10
3	D+.25L	D1	-0.216	.205	-6e-06	15	18	10
2	Dead	D1	-.123	0.116	-3e-06	15	18	10
2	Live	D1	-0.015	0.015	-4.1e-07	15	18	10
2	eqx	D1	0	0	0	15	18	10
2	eqx+e	D1	0	0	0	15	18	10
2	Eqx-e	D1	0	0	0	15	18	10
2	eqy	D1	0	0	0	15	18	10
2	Eqy+e	D1	0	0	0	15	18	10

2	Eqy-e	D1	0	0	0	15	18	10
2	NLS	D1	-0.317	0.301	-8e-06	15	18	10
2	D+.25l	D1	-.123	.116	-3e-06	15	18	10
1	Dead	D1	-0.032	0.031	-1e-06	15	18	10
1	Live	D1	0.004	0.004	-1.261e-07	15	18	10
1	eqx	D1	0	0	0	15	18	10
1	Eqx+e	D1	0	0	0	15	18	10
1	Eqx-e	D1	0	0	0	15	18	10
1	eqy	D1	0	0	0	15	18	10
1	Eqy+e	D1	0	0	0	15	18	10
1	Eqy-e	D1	0	0	0	15	18	10
1	NLS	D1	-0.083	0.08	-2e-06	15	18	10
1	D+.25L	D1	-0.032	0.031	-e-06	15	18	10

Discussion : In the Non-Linear cases support rotations are to be checked as given above in the tables. For corner removal case the support rotation doesn't exceeds 6°, which is the limiting value for support rotation given by the GSA guidelines for concrete structures. Hence, we can say that building is not vulnerable to progressive collapse.

b) When central column is removed

Table 4.20 Support rotation and displacement

Story	**Diaphrag**	**Load case**	**U_X**	**U_y**	**Rz**	**X**	**Y**	**Z**

	m							
6	D1	Dead	1.004e-05	0.492	-3.095e-09	15	10	18
6	D1	Live	2.19e-06	0.067	-5.441e-10	15	10	18
6	D1	eqx	0	0	0	15	10	18
6	D1	Eqx+e	0	0	0	15	10	18
6	D1	Eqx-e	0	0	0	15	10	18
6	D1	eqy	0	0	0	15	10	18
6	D1	Eqy+e	0	0	0	15	10	18
6	D1	Eqy-e	0	0	0	15	10	18
6	D1	NLS	2.15e-05	1.038	-6.61e-09	15	10	18
5	D1	Dead	8.19e-06	0.4	-2.489e-09	15	10	18
5	D1	live	1.789e-06	0.054	-4.78e-10	15	10	18
5	D1	eqx	0	0	0	15	10	18
5	D1	Eqx+e	0	0	0	15	10	18
5	D1	Eqx-e	0	0	0	15	10	18
5	D1	eqy	0	0	0	15	10	18
5	D1	Eqy+e	0	0	0	15	10	18
5	D1	Eqy-e	0	0	0	15	10	18
5	D1	NLS	1.756e-06	0.846	-5.332e-09	15	10	18
5	D1	D+0.25L	8.196e-06	0.4	-2.456e-09	15	10	18
4	D1	Dead	6.394e-06	0.307	-1.85e-09	15	10	18
4	D1	Live	1.89e-06	0.042	-3.554e-10	15	10	18
4	D1	eqx	0	0	0	15	10	18
4	D1	Eqx+e	0	0	0	15	10	18
4	D1	Eqx-e	0	0	0	15	10	18
4	D1	eqy	0	0	0	15	10	18
4	D1	Eqy+e	0	0	0	15	10	18
4	D1	Eqy-e	0	0	0	15	10	18

4	D1	NLS	1.381e-05	0.652	-3.974e-09	15	10	18
4	D1	D+.25l	6.394e-06	0.307	-1.85e-09	15	10	18
3	D1	Dead	4.552e-06	.204	-1.236e-09	15	10	18
3	D1	live	9.847e-07	.029	-2.37e-10	15	10	18
3	D1	eqx	0	0	0	15	10	18
3	D1	Eqx+e	0	0	0	15	10	18
3	D1	Eqx-e	0	0	0	15	10	18
3	D1	eqy	0	0	0	15	10	18
3	D1	Eqy-e	0	0	0	15	10	18
3	D1	Eqy-e	0	0	0	15	10	18
3	D1	NLS	9.87e-06	.455	-2.66e-09	15	10	18
3	D1	D+.25L	4.552e-06	.214	-1.236e-09	15	10	18
2	D1	Dead	2.67e-06	.12	-6.63e-10	15	10	18
2	D1	live	5.748e-07	.017	-1.234e-10	15	10	18
2	D1	eqx	0	0	0	15	10	18
2	D1	Eqx+e	0	0	0	15	10	18
2	D1	Eqx-e	0	0	0	15	10	18
2	D1	eqy	0	0	0	15	10	18
2	D1	Eqy+e	0	0	0	15	10	18
2	D1	Eqy-e	0	0	0	15	10	18
2	D1	NLS	5.79e-06	.256	-1.47e-9	15	10	18
2	D1	D+.25L	2.67e-06	.12	-6.66e-10	15	10	18
1	D1	Dead	8.89e-07	0.032	-2e-10	15	10	18
1	D1	Live	1.883e-07	0.05	-3.8e-11	15	10	18
1	D1	eqx	0	0	0	15	10	18
1	D1	Eqx+e	0	0	0	15	10	18
1	D1	Eqx-e	0	0	0	15	10	18
1	D1	eqy	0	0	0	15	10	18

1	D1	Eqy+e	0	0	0	15	10	18
1	D1	Eqy-e	0	0	0	15	10	18
1	D1	NLS	1.92e-06	0.069	-4.316e-10	15	10	18
1	D1	D+.25L	8.89e-07	0.032	-2e-10	15	10	18

Discussion

In the Non-Linear cases support rotations are to be checked as given above in the tables. For central column removal case the support rotation doesn't exceeds 6°, which is the limiting value for support rotation given by the GSA guidelines for concrete structures. Hence, we can say that building is not vulnerable to progressive collapse.

4.2.2 Storey response plot

a) When central column is removed

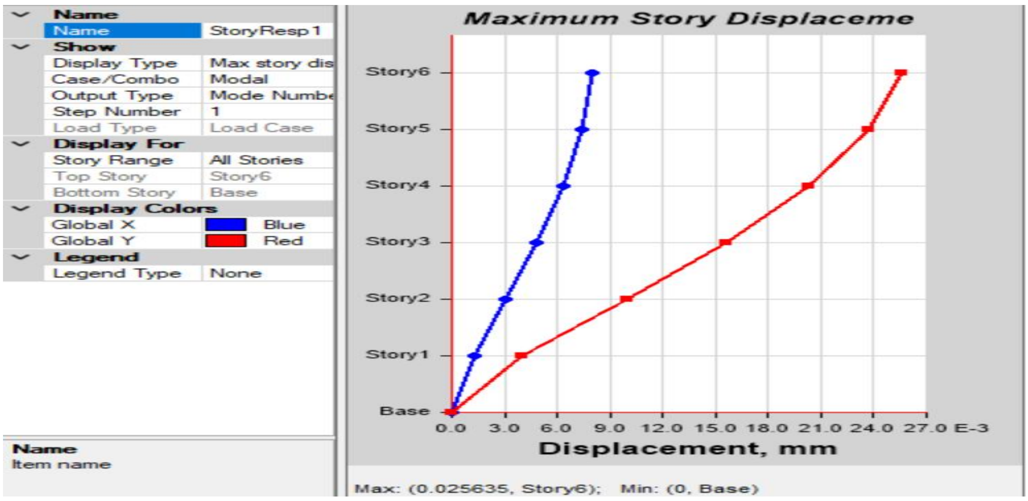

Figure 4.7 Storey response plot

Discussion : As we can see in the graph that storey drift is well below the limit as prescribed by IS1893:2016 (Criteria for Earthquake design of structures-Part I:General Provisions and Buildings ,2016), the storey drift in any storey must not exceed the 0.4% of storey height under the action of base shear with no load factor.

b) When corner column is removed

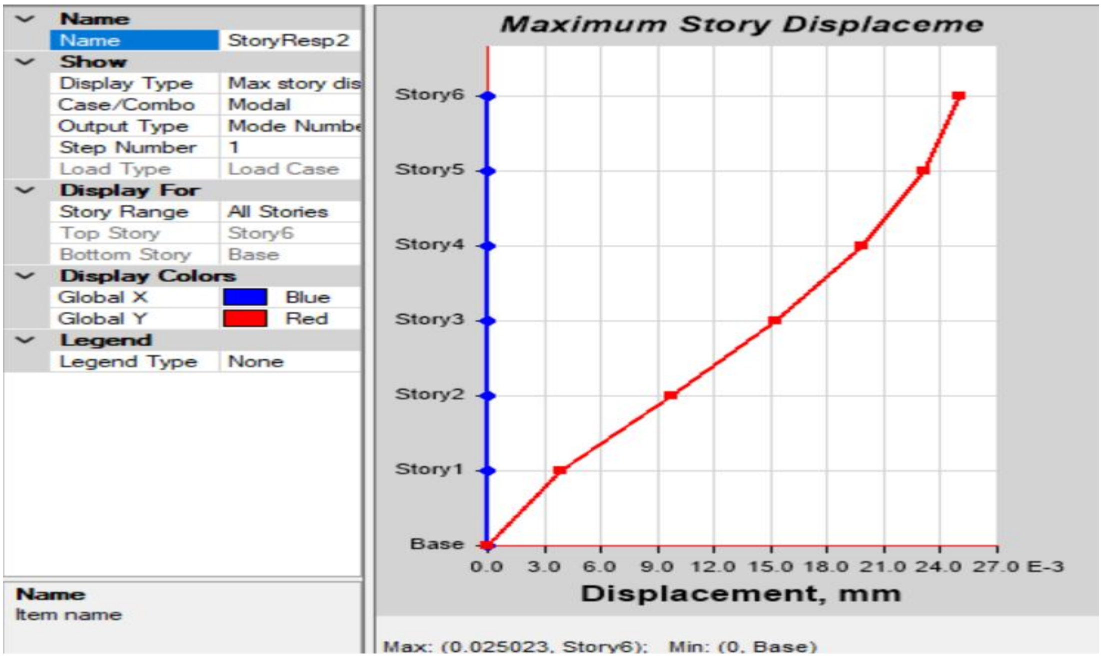

Figure 4.8 Storey response plot

Discussion : As we can see in the graph that storey drift is well below the limit as prescribed by IS1893:2016 (Criteria for Earthquake design of structures-Part I:General Provisions and Buildings ,2016), the storey drift in any storey must not exceed the 0.4% of storey height under the action of base shear with no load factor.

CHAPTER 5

CONCLUSIONS AND FUTURE SCOPE

In this project progressive collapse analysis have been done using a finite element linear static and a non linear static analysis on a 6 storey building. Though there are dynamic methods also for the analysis but static method has been used owing to its practicality and simplicity in engineering world. Demand capacity ratio of columns presents the results for linear static and support rotation at the joints represent the result for non-linear static analysis.

If in the linear static case the DCR is less than 2 then the building is said to be vulnerable to progressive collapse. Also, in the non- linear static analysis in no case the support rotation exceeds the limited value of 6 degree hence this method renders the building safe.

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