"Seismic Evaluation of Reinforced Concrete Buildings"

A Thesis submitted towards the partial fulfillment of
requirements for the award of the degree of

Master of Engineering
in
Civil Engineering
(Structures)

Submitted by

Taranpreet Singh
Roll No. 8042310

Under the Guidance of

Dr.Naveen Kwatra
(Asstt. Professor, CED)

DEPARTMENT OF CIVIL ENGINEERING
THAPAR INSTITUTE OF ENGINEERING & TECHNOLOGY,
(DEEMED UNIVERSITY),
PATIALA – 147004, INDIA
June 2006

CERTIFICATE

This is to certify that the work presented in Thesis entitled **"Seismic Evaluation of Reinforced Concrete Buildings"** submitted by **Mr. Taranpreet Singh** in partial fulfillment of requirements for the award of degree of **Masters of Engineering in Civil (Structures)** at **Thapar Institute of Engineering & Technology (Deemed University)**, **Patiala**, is an authentic record of student's own work carried out under my supervision and guidance. The matter embodied in thesis has not been submitted anywhere for award of any other degree.

Dr.Naveen Kwatra
Asstt. Professor,
Department of Civil Engineering,
Thapar Institute of Engg. & Technology,
(Deemed University)
Patiala- 147004

Prof. & Head
Civil Engineering Department
Thapar Institute of Engg. & Technology,
(Deemed University)
Patiala- 147004

Dr. T.P Singh
Dean of Academic Affairs
Thapar Institute of Engg. & Technology,
(Deemed University)
Patiala- 147004

ACKNOWLEDGEMENTS

Knowledge in itself is a continuous process. I would have never succeeded in completing my task without the cooperation, encouragement and help provided to me by various personalities.

With deep sense of gratitude I express my sincere thanks to my esteemed and worthy supervisor, Dr. Naveen Kwatra, Asstt. Professor, Department of Civil Engineering, for his invaluable guidance, wild discussions, sincere encouragement and constructive criticism during the conceptualization, of this study. The prolonged interactions with him even at odd hours of the day have really inculcated in me the spirit of an independent practicing structural engineer.

I wish to express my sincere thanks to Dr. Rafat Siddique, Professor & Head, Department of Civil Engineering, P.G. Incharge Dr. Maneek Kumar who has been a constant source of inspiration for me throughout this thesis work.

Appreciation is also extended to Dr. Pardeep Kumar, Asstt. Professor, NIT Hamirpur for his immense guidance, gratuitous and mind provoking ideas.

I am also thankful to all the staff members of Civil Engineering Department for their full cooperation and help.

The technical guidance and constant encouragement made it possible to tie over the numerous problems, which so ever came up during the study. My greatest thanks are to all who wished me success. Above all I render my gratitude to the Almighty who bestowed self-confidence, ability and strength in me to complete this work.

Taranpreet Singh

Roll No. 8042310

ABSTRACT

The Buildings, which appeared to be strong enough, may crumble like houses of cards during earthquake and deficiencies may be exposed. Experience gain from the recent earthquake of Bhuj, 2001 demonstrates that the most of buildings collapsed were found deficient to meet out the requirements of the present day codes. Due to wrong construction practices and ignorance for earthquake resistant design of buildings in our country, most of the existing buildings are vulnerable to future earthquakes. In light of these facts, it is imperative to seismically evaluate the existing building with the present day knowledge to avoid the major destruction in the future earthquakes. The Buildings found to be seismically deficient should be retrofitted/strengthened.

The present study deals with the evaluation of R.C buildings using inelastic method (Pushover Analysis). Capacity Curve, which is Load-Deformation Plot is the Output of Pushover Analysis. As, Pushover Analysis is Non-Linear Static Analysis, so the Load-Deformation Curve can be obtained from ANSYS. Finite Element Software ANSYS 5.4 is used to perform the Non-Linear Static Pushover Analysis and Cracking pattern can also be observed in ANSYS. Cracking Pattern provides the need for Strengthening required for particular Elements. Firstly, a symmetrical building is analysed using ANSYS for the procedure development as per ATC-40. Then, Seismic Evaluation is performed on unsymmetrical building (L-shape), which is designed in the first part as without considering seismic effect and in the second part, Analysis is carried out on the same building designed seismically as per I.S 1893:2002. The results have been compared for these two Analysis cases and Strengthening is suggested for the affected members.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER - 1

INTRODUCTION

1.1) General:

The Buildings, which appeared to be strong enough, may crumble like houses of cards during earthquake and deficiencies may be exposed. Experience gain from the recent earthquake of Bhuj, 2001 demonstrates that the most of buildings collapsed were found deficient to meet out the requirements of the present day codes. In last decade, four devastating earthquakes of world have been occurred in India, and low to mild intensities earthquakes are shaking our land frequently. Due to wrong construction practices and ignorance for earthquake resistant design of buildings in our country, most of the existing buildings are vulnerable to future earthquakes.

In the simplest case, seismic design can be viewed as a two-step process. The first, and usually most important one, is the conception of an effective structural system that needs to be configured with due regard to all-important seismic performance objectives, ranging from serviceability considerations to life safety and collapse prevention. This step comprises the art of seismic engineering, since no rigid rules can, or should, be imposed on the engineer's creativity to devise a system that not only fulfills seismic performance objectives, but also pays tribute to functional and economic constraints imposed by the owner, the architect, and other professionals involved in the design and construction of a building. By default, this process of creation is based on judgment, experience, and understanding of seismic behaviour, rather than rigorous mathematical formulations. Rules of thumb for strength and stiffness targets, based on the fundamental knowledge of ground motion and elastic and inelastic dynamic response characteristics, should suffice to configure and rough size an effective structural system.

This second step of the design process should involve a demand/capacity evaluation at all important performance levels, which requires identification of important capacity parameters and prescription of acceptable values of these parameters, as well as the prediction of the demands imposed by ground motions. Suitable capacity parameters and their acceptable values, as well as suitable methods for demand prediction will depend on the performance level to be evaluated.

In light of these facts, it is imperative to seismically evaluate the existing building with the present day knowledge to avoid the major destruction in the future earthquakes. The Buildings found to be seismically deficient should be retrofitted/strengthened.

Evaluation of building is required at a two stages (1) Before the retrofitting, to identify the weakness of the building to be strengthened, and (2) After the retrofitting, to estimate the adequacy and effectiveness of retrofit. Evaluation is complex process, which has to take not only the design of building but also the deterioration of the material and damage cause to the building, if any. The difficulties faced in the seismic evaluation of the building are threefold. There is no reliable method to estimate the in-situ strength of the material in components of the building. Analytical method to model the behavior of the building during earthquake is either unreliable or too complex to handle with the generally available tools. The third difficulty is the un-availability of reliable estimate of earthquake parameters, to which the buildings expected to be subjected during its residual life. (Murty, Feb., 2002)

1.2) Evaluation criteria:

The consequence of evaluation of any building should be quantitatively evaluated for its effectiveness from the viewpoints of strength, stiffness & ductility.

1.2.1) Strength/capacity:

The essence of virtually all seismic evaluation procedures is a comparison between some measures of "Demand" that earthquake take place on a structure to measure of the "Capacity" of the building to resist. Traditional design procedures characterize demand and capacity as forces. Base shear (Total Horizontal force at the lowest level of the building) is a normal parameter i.e. used for the purpose. It involves calculation of base shear demand that would be generated by given earthquake, or intensity of ground motion, and compare this to the base shear capacity of the building. The capacity of the building is an estimate of base shear that would be "acceptable". If the building subjected to a force equal to its base shear capacity, some deformation and yielding might occur in some structural elements, but the building would not collapse or reach undesirable level of damage. If the demand generated by the earthquake is less than the capacity than the design is deemed acceptable (ATC 40). More sophisticated works needs to compare the seismic demand of every structure elements with its capacity i.e. demand capacity ratios. (DCRs)

1.2.2) Stiffness:

The first formal seismic design procedure recognized that the earthquake acceleration would generate forces proportional to the weight of building. Over the years, empirical knowledge about the behaviour of real structures in earthquakes and theoretical understanding of structural dynamics advanced. The basic procedure modified to reflect the demand generated by the earthquake acceleration also a function of stiffness of the structure. It helps to recognize the inherently better behavior of some building over the others (ATC 40).

To get minimum damage and less psychological fear in the mind of peoples during the earthquake. IS 1893: 2002 permits maximum inter-story drifts as 0.004 times the story height. Inter-story drifts always depend upon the stiffness of the respective storey (IS 1893-2002). Again the abrupt changes in the stiffness along the load paths may lead to high stress concentration at some load transfer points and may create local crushing. Hence stiffness always plays vital roles and considered as an important criteria in the seismic evaluation of the building (ATC 40).

1.2.3) Ductility:

Earthquake motion often induces forces large enough to cause inelastic deformations in the structure. If the structure is brittle, sudden failure could occur. However, if the structure is to made to behave to ductile, sudden failure to sustain the earthquake effects better with some deflection larger than the yield deflection by absorption of energy. The capacity of structure to resist seismic demand is a property known as ductility. It is the ability to deform to beyond initial yielding without failing abruptly. This property is a critical component of structural integrity and required as an essential element for safety from sudden collapse during severe shocks.

1.3) Method to perform simplified nonlinear analysis (Pushover Analysis):

Two key elements of a performance based design procedure are demand and capacity. Demand is a representation of the earthquake ground motion. Capacity is a representation of the structure's ability to resist the seismic demand. The structure must have the capacity to resist the demand of the earthquake such that the performance of the structure is compatible with the objectives of the design. Simplified non-linear analysis procedures using pushover methods such as the capacity spectrum requires determination of three primary elements: Capacity, demand and performance. Each of these elements is briefly discussed as:

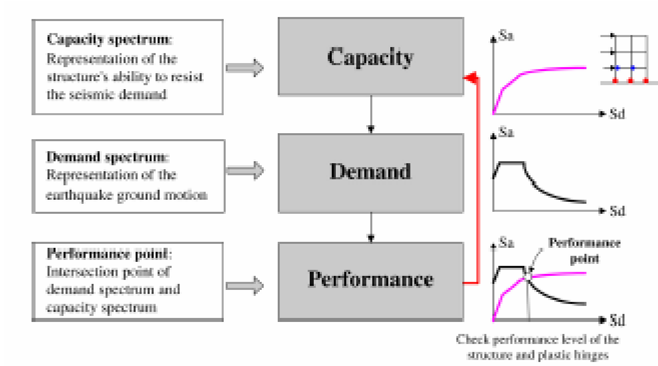


Fig.1
1.1 Nonlinear analysis procedure.

Capacity: The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. A Pushover analysis procedure uses a series of sequential elastic analysis, superimposed to approximate a force – displacement capacity diagram of the overall structure. The mathematical model of the structure is modified to account for reduced resistance of yielding components. A lateral force distribution is again applied untill a predetermined limit is reached. Pushover capacity curves approximate how structure behaves after exceeding the elastic limits.
Demand (Displacement): Ground motions during an earthquake produce complex horizontal displacement patterns in structure that may vary with time. Tracking this motion at every time step to determine structural design requirements is judged impractical. For nonlinear method it is easier and more direct to use a set of lateral displacement as a design condition for a given structure and ground motion, the displacement is an estimate of the maximum expected response of the building during ground motion.
Performance: Once a capacity curve and demand displacement are defined, a performance check can be done. A performance verifies that structural & non-structural components are not damaged beyond the acceptable limits of performance objectives for the forces and displacement implied by the displacement demand.
1.4) Objective:

In this report, the evaluation of R.C buildings using inelastic method (Pushover Analysis) is adopted. Capacity Curve, which is Load-Deformation Plot is the Output of Pushover Analysis. As, Pushover Analysis is Non-Linear Static Analysis, so the Load-Deformation Curve can be obtained from ANSYS. Finite Element Software ANSYS 5.4

is used to perform the Non-Linear Static Pushover Analysis and Cracking pattern can also be observed in ANSYS. Cracking Pattern provides the need for Strengthening required for particular Elements. Capacity Curve is obtained from ANSYS 5.4, and Response Spectra as given in I.S 1893:2002 is used. Staad.Pro 2003 has been used to provide the Reinforcement, which is required as Input parameter for ANSYS.
Firstly, a symmetrical building is analysed using ANSYS for the procedure development as per ATC-40, the details of this analysis is explained in Chapter-4. Then, in Chapter-5, Seismic Evaluation is carried out on unsymmetrical building (L-shape), which is designed in the first part as without considering seismic effect and in the second part, Analysis is carried out on the same building designed seismically as per I.S 1893:2002. The results have been compared for these two Analysis cases and Strengthening is suggested for the affected members.

CHAPTER - 2

LITERATURE REVIEW

2.1) Introduction:

To provide a detailed review of the body of literature related to seismic evaluation in its entirety would be too immense to address in this thesis. However, there are many good references that can be used as a starting point for research (ATC 40 Manual for Seismic Evaluation and Retrofitting of concrete buildings). This literature review and introduction will focus on recent contributions related to seismic evaluation and past efforts most closely related to the needs of the present work.

The goal of seismic evaluation of building is to determine how buildings will response to a design of earthquake described by the recommended spectra. In other words, the goal is to find the weak links and to identify, how their behavior will affect the response of the structural system. The location and behavior of a weak link in a load path of lateral force existing system must be evaluated. The weak links may function as a base isolator that will limit the structural response of the lateral force resisting system (NEHRP, Washington, D.C 1992).

The capacity spectrum method, which is non-linear static procedure, provides a graphical representation of the global force displacement capacity curve of the structure and compares it to the response spectra representation of the earthquakes demands. This approach includes consideration of ductility of structure on an element-by-element basis. The inelastic capacity of a building is then a measure of its ability to dissipate earthquake energy (ATC 40).

2.2) Evaluation based upon Elastic Approach

As mentioned in ATC-40, both elastic and in-elastic methods are available for the analysis of existing concrete buildings. Seismic Evaluation can be performed by Elastic procedures using DCRs (Demand-Capacity Ratios): The work carried out by (Bhardwaj, 2002) is based on elastic approach. This is a linear elastic analysis, which involves the following three stages, namely:

2.2.1) Input data stage.

i) Study of site soil conditions.

ii) Measurement of actual geometry of buildings and its component.

iii) In-situ NDT to estimate to actual strength of concrete in the building components.

iv) Test to estimate actual strength of steel reinforcement bars in the building components and the extent of corrosion, to carefully estimate their available diameters.

2.2.2) Analysis stage:

v) Preparation of 3 D model of building frame using measure geometry & material properties.

vi) Estimation of design later force on building using IS 1893:2002 for the given design response spectra.

vii) Application of design lateral force on 3D building model to determine stress resultants (i.e. axial forces, shear forces, bending moments etc.), in the frame members and determination of inter-story drifts.

viii) Determination of RC member capacities with actual cross-sectional geometry and material properties as per IS 456:2000/IS 13920:1993 and DCR of RC members at critical locations.

ix) Identification of deficient member or deficiency in lateral stiffness of the building if any.

2.2.3) Retrofit and verification stage:

x) Identification of suitable retrofitting techniques to rectify the deficiencies.

xi) Estimation of the new member sizes along with the addl. Reinforcement required, and/or the new members requires.

xii) Reanalysis of buildings to confirm the adequacy with then proposed retrofit techniques.
xiii) If strength and stiffness requirement are satisfied than the propose retrofits scheme may be adopted, else other more appropriate retrofits scheme may be identified.
The focus of these most recent efforts is to seismic evaluation using elastic procedure by calculating Demand-Capacity Ratios (DCRs). Seismic Evaluation by Non-Linear Static Analysis exposes design weaknesses that may remain hidden in an elastic approach. Such weaknesses include excessive deformation demands, strength irregularities, and overloads on potentially brittle points, such as columns and connections (Krawinkler H., Seneviratna G. 1998).
2.3) The research carried out by (Dinh T.V and Ichinose T.) suggested that the deformation demands for columns, which are critical to the safety limits of the building, are closely related to the drift in each story. This is also a case for the deformation demands for the nonstructural elements that are critical to the serviceability limit state. Thus, a designer must evaluate the story drift demand, which may vary due to uncertainties in the characteristics of future earthquake motions and the estimation of member strengths.
The FEMA-356 document (FEMA 2000) requires pushover analyses using two types of force distributions, such as modal or uniform pattern, to examine failure mechanisms and to predict story drift demands of a building. These analyses may detect two kinds of mechanisms. A modal pattern may induce a total mechanism whereas a uniform pattern may result in a story mechanism at the first story. However, story mechanisms at the second and upper stories might be overlooked because the story shear forces of these stories given by uniform pattern are smaller than those by modal pattern.
In a building with structural walls, the probability of a story mechanism decreases as the shear strength of the walls increases, as discussed by Park and Paulay (1975). In a frame building, the probability of a story mechanism decreases as the column-to-beam strength ratio increases, as discussed by Dooley and Bracci (2001).

To integrate these tendencies, a story-safety factor f_i, is defined by the following equation (Abimanyu et al. 1997):

$$f_i = \Delta V_i / V_{ui} \text{ , for } V_i = V_{si} - V_{ui}$$

where, V_{si}= strength under the forces causing a story mechanism of the *i*th story as shown in Fig. 2.1 (the sum of the shear strength of the wall and the flexural/shear strength of the columns); and V_{ui}= shear force of the *i*th story when a failure mechanism occurs under inverted triangular forces as shown in Figs. 2.2 (a and b). The difference between Vsi and Vui represents the strength margin against a story mechanism. If a building fails due to a total mechanism under static loading Fig. 2.2(a) but the value of *fi* of some of the stories are approximately zero, then it is probable that mechanisms will occur at these stories under seismic excitation. If a building fails due to a story mechanism under static loading Fig. 2.2(b), but the value of *fi* of some stories are approximately zero, then it is also probable that mechanisms will occur at these stories under seismic excitation. It should be noted that *fi* of the story where a story mechanism occurs under static loading automatically becomes zero, indicating that a story mechanism at that story is most probable under seismic excitation.

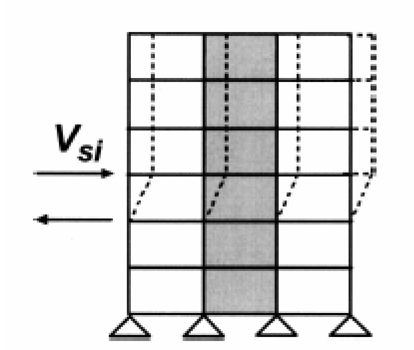
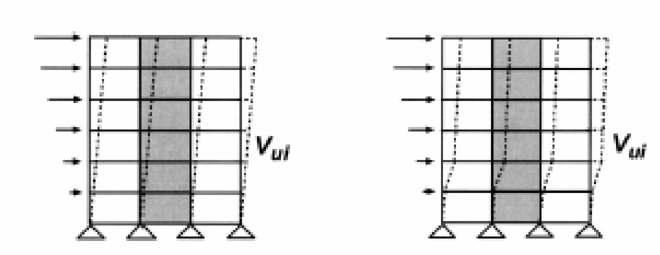


Fig.2.1 Forces causing a story mechanism



a) Total Mechanism (b) Story Mechanism

Fig.2.2. Mechanisms due to inverted triangular forces: (a) total mechanism and (b) story mechanism

The preliminary study by Ichinose and Umeno (2000) showed that the drift responses of structures deforming due to total mechanisms with weak beams Fig. 2.3(a) and weak columns Fig. 2.3(b) are almost identical if the initial periods, base shear strengths, and story-safety factors of the structures are identical even though the column-to-beam strength ratios at the beam-column joints of the left column in Fig. 2.3(a) are larger than those in Fig. 2.3(b). Thus, the story-safety factor better represents the drift response than the column-to-beam strength ratio. The story-safety factor is now used in the Japanese Standard for the seismic evaluation of existing RC buildings (Ichinose et al. 2002) to predict failure mechanisms.

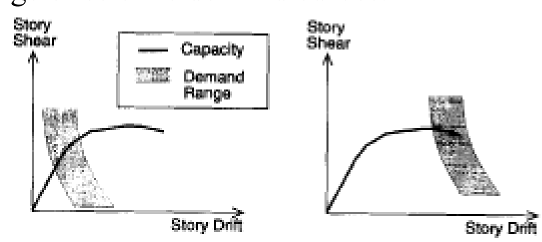
(a) Total Mechanism with Weak Beams (b) Total Mechanism with Weak Columns

(c) Story Mechanism (d) Partial Mechanism

Fig. 2.3. Failure mechanisms of frame structure: (a) total mechanism with weak beams; (b) total mechanism with weak columns; (c) story mechanism; and (d) partial mechanism.

2.4) Further (Bernal D.) investigated that, Gravity loads acting on structural deformations decrease the lateral stiffness of buildings. The reduction is typically of minor importance when elastic behavior is considered because its magnitude in realistic structures is only a small fraction of the first-order elastic stiffness. During response to severe ground motion, however, the lateral stiffness of a building decrease as a result of anticipated yielding and the reduction due to gravity can become a critical consideration. In particular, the potential for instability exists if the structure enters configurations where the effective stiffness associated with any given deformation mode becomes negative.

2.5) The research carried out by (Bracci J.M, Kunnath S.K, Reinhorn A.M) is based upon the evaluation of seismic performance and retrofit of existing low to midsize reinforced concrete (RC) buildings. The basis of the proposed method is to develop a range of site specified demand curves and compares them to a computed pushover capacities at each story level of the structure. The primary differences between the proposed scheme and capacity spectrum method are 1) the treatment of the applied lateral load during the pushover analysis: 2) the use of story level demand and capacities as opposed to overall base shear V/S top story displacement: and 3) the consideration of different levels in elasticity to generate a range of seismic demand curves.



(a) (b)

Fig.2.4 Typical Seismic Story Demand versus Capacity: (a) Safe Design; (b) Unsafe Design

Description of Original frame model Building:

Both current and past design practices for low rise RC frame buildings constructed in low to moderate seismic risk areas have generally being for only gravity loads (GLD) according to the non-seismic detailing provisions of the ACI-318-89 code. The seismic response of such structure was evaluated in a dual analytical – experimental shaking table study of 1/3rd scale model building. (Fig 2.5) The seismic deficiencies incorporated in that study, all of which violated the seismic provisions of chapter 21 in ACI-318-89 included 1) weak column, strong beam, behavior creating a structure prone to a soft story collapse mechanism: 2) In-adequate transverse reinforcement in columns and joints for shear and confinements: 3) Column loops located in potential hinge zones: 4) Discontinuous positive (bottom) beam reinforcement in beam-column joints. The imposed shaking table motions on the buildings simulated the 1952 Taft N21E earthquake component with normalized with PGAs of 0.05g, 0.20g and 0.30g. The spring motion test indicating that damaging soft story mechanism was developing at the second floor level with potential for structural failure of the entire system.

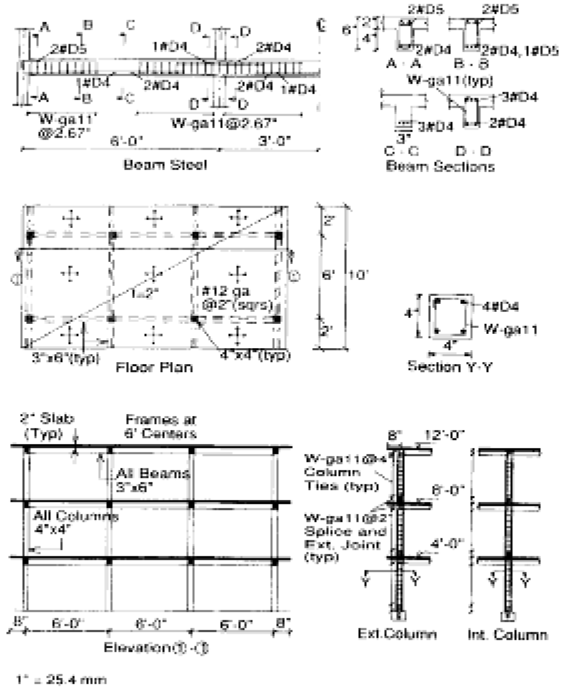
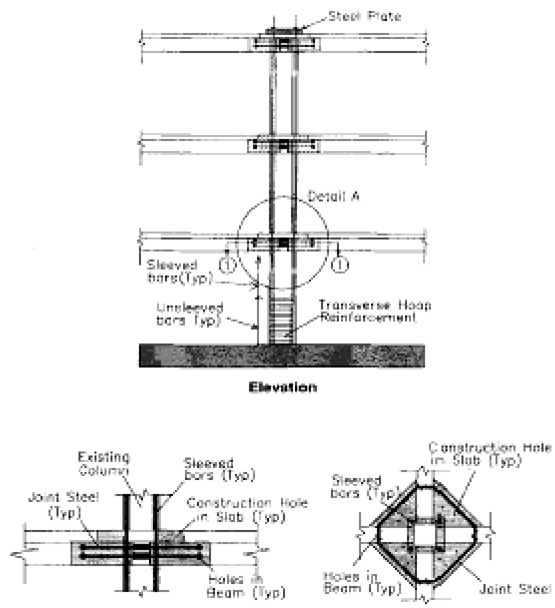


Fig.2.5. Plan, Elevation, and Reinforcing Details of Original Model

Description of Retrofitted Frame Model Building:
Following the series of damaged shaking table excitations on the original model structure, several seismic retrofit alternatives for GLD RC frame structure were investigated for improving their local and global response performance during low-to-moderate type earthquakes by comparing structural behavior from nonlinear dynamic analysis. These retrofit schemes were designed to distribute damage throughout the building with acceptable control of story deformations by averting a catastrophic soft-story failure and enforcing a more ductile beam-sideway mechanism. A minimum seismic retrofit was performed on the interior columns of the previously damaged one-third scale model building using a prestressed concrete jacketing technique (see fig 2.6) by (1) encasing selected columns in a concrete jacket with additional longitudinal and transverse reinforcement;(2) providing a reinforced concrete fillet around the unreinforced beam-column joints; and (3) post tensioning the added longitudinal column reinforcement. By strengthening only the interior columns of the model building, the retrofit scheme was considered to provide a minimum seismic resistance acceptable for low-to-moderate seismicity zones and was considered inadequate for high seismic zones.

The retrofitted model building was then experimentally tested on the shaking table with the same excitations as in the original model tests with PGAs of 0.20g and 0.30g.

When compared to preretrofit performance, the experimental results of the retrofitted model structure showed that the overall behavior and damage can be more effectively controlled using only minimum column and joint strengthening techniques, which may be suitable for structure in low-to-moderate seismic risk zones. The retrofit scheme was successful in changing the failure mode from an undesirable column sidesway mechanism to a more ductile beam sidesway mechanism.



Detail A: Reinforced Fillet   Section 1-1: Retrofitted Fillet

Fig.2.6. Prestressed Concrete Jacketing Retrofit

Nonlinear pushover analyses were performed on both the original and retrofitted model buildings. For the 0.05g PGA excitation, the demand region intersects the story capacity envelopes near the elastic portion of the response, implying elastic behavior. For the 0.20g and 0.30g PGA excitation, the demand curves intersect the capacity envelope in the region at incipient and full failure mechanism, respectively. From the standpoint of estimating the margin of safety of the structure against the imposed loading, the following observations can be made from Figs.2.7-2.8 for the original building: the structure performed in the elastic range during the 0.05g shaking: for the 0.20g intensity shaking, the structure responded in the inelastic range with likely yielding in some members, but had a sufficient margin of safety against structural collapse as seen from the strength and deformation capacity beyond the intersection of the demand end capacity lines;however,for the 0.30g shaking, the demands intersect the capacities with little strength and deformation reserve. Therefore, it can be concluded that the margin of safety against collapse for the original building was small at this intensity of shaking.

For the retrofitted model building under the same earthquake ground motions (0.20g and 0.30g), it can be observed from Figs. 2.10 and 2.11 that most of the inelastic response occurs on the first and second stories. The predicted maximum response for each story correlates well with the experimentally observed maximum behavior. From the standpoint of evaluation the adequacy of the retrofit behavior, it can be observed that the story demands intersect the capacity envelopes with sufficient strength and displacement reserves. Although inelastic response is evident during these base motions, collapse of the retrofitted structure is not imminent for these levels of ground motion excitation.

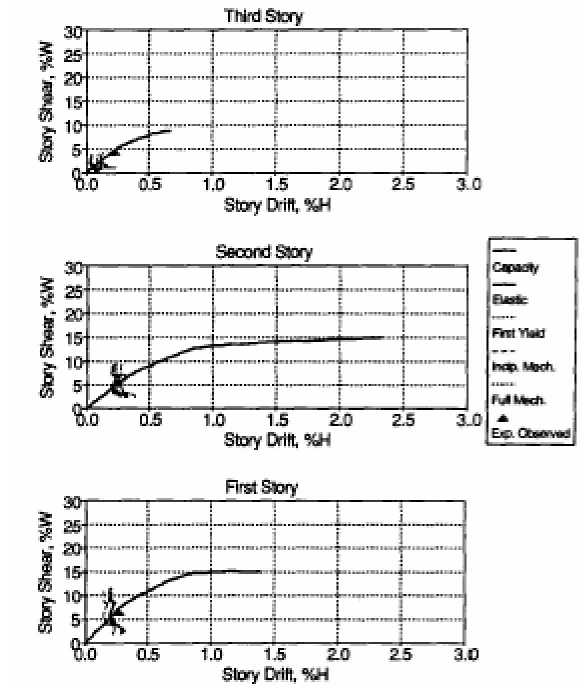


Fig.2.7 Seismic Demand versus Capacity, Original Building, PGA 0.05g.

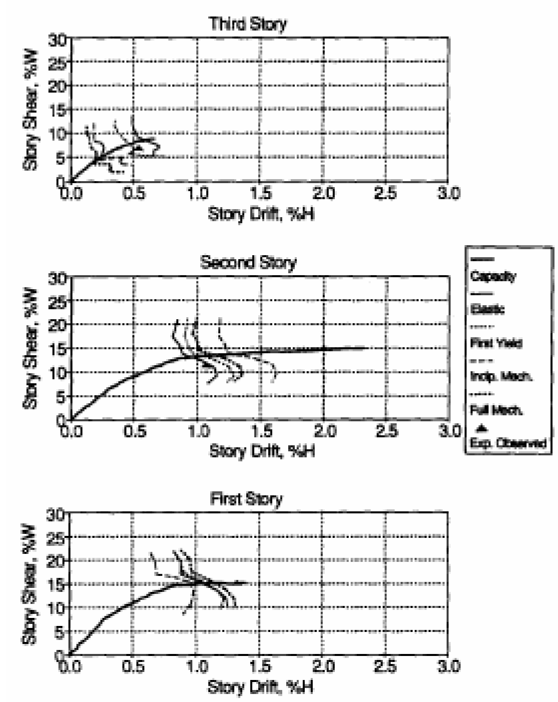


Fig.2.8 Seismic Demand versus Capacity, Original Building, PGA 0.2g

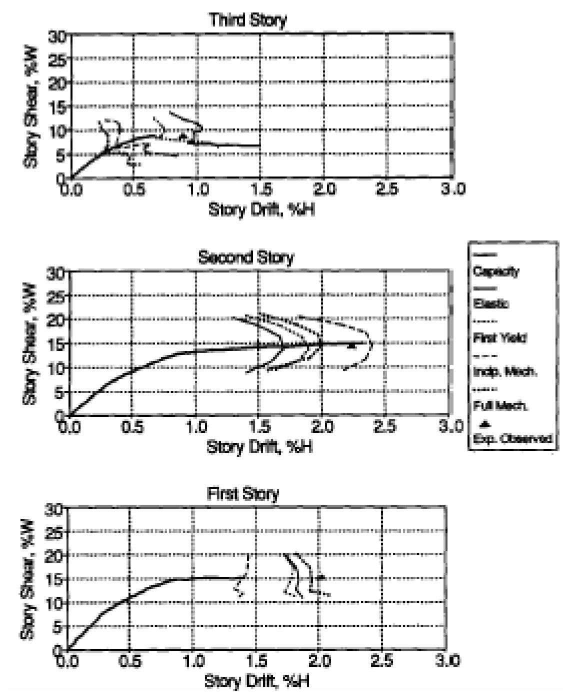


Fig.2.9 Seismic Demand versus Capacity, Original Building, PGA 0.3g

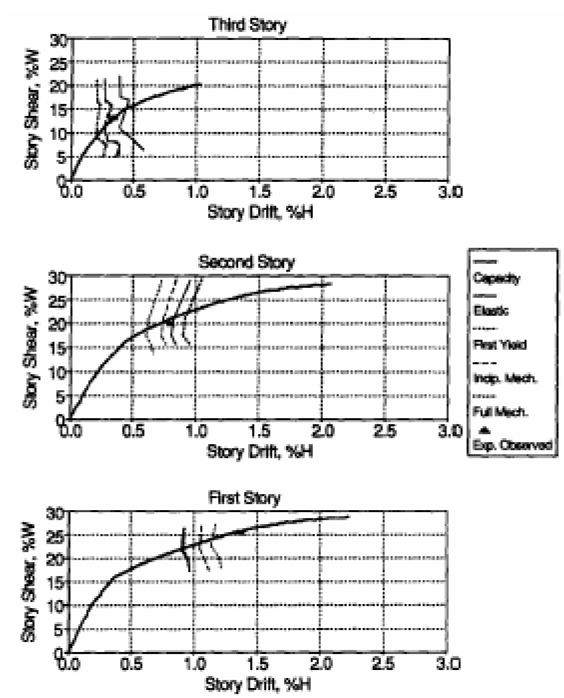


Fig.2.10 Seismic Demand versus Capacity, Retrofitted Building, PGA 0.20g

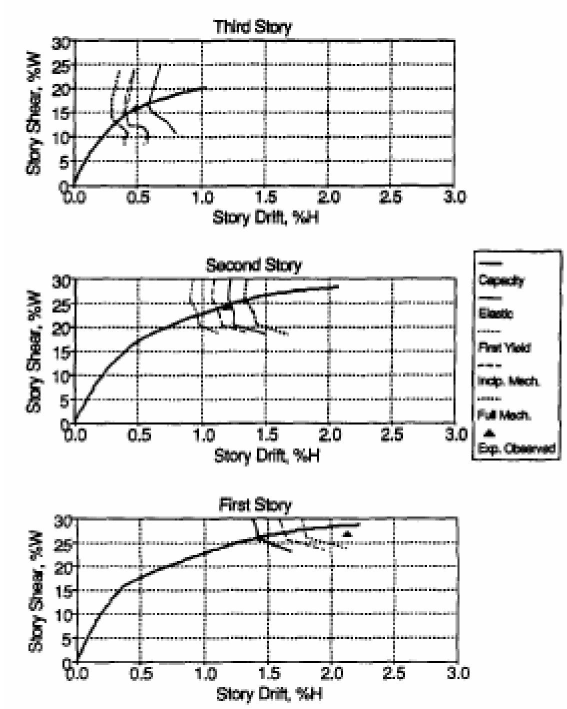


Fig.2.11 Seismic Demand versus Capacity, Retrofitted Building, PGA 0.30g

2.6) Further, the research carried out by (Kappos A.J, Manafpour A.) suggested that, along with several other seismic codes currently in force, the American UBC was based until 1994 on elastic analysis for equivalent static forces or a design spectrum. The 1997 edition of this code introduced for the first time some detailed provisions for the application of both elastic and inelastic time–history analysis, mainly with regard to the selection and scaling of ground motions. A similar development took place in the recent NEHRP Provisions [3] but in this case time–history analysis is clearly restricted to structures with seismic isolation. The seismic Eurocode, EC8 [4], recognizes that inelastic time–history analysis might be used in the design procedure, but guidance is only given with regard to the selection of input accelerograms and the way they should be scaled to match the design spectrum. No clear indication is given in EC8 as to what model(s) should be used, or which response quantities should be sought, if such an analysis is used. On the other hand, the New Zealand Code [5] is clear in specifying that the purpose of using inelastic time–history analysis might be either to calculate strength requirements in yielding members, or assess inelastic demands and/or capacity actions.

2.7) (Krawinkler H., Seneviratna G.D.P.K), suggested that a carefully performed pushover analysis will provide insight into structural aspects that control performance during severe earthquakes. For structures that vibrate primarily in the fundamental mode, the pushover analysis will very likely provide good estimates of global, as well as local inelastic, deformation demands. This analysis will also expose design weaknesses that may remain hidden in an elastic analysis. Such weaknesses include story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle elements such as columns and connections.

On the negative side, deformation estimates obtained from a pushover analysis may be very inaccurate (on the high or low side) for structures in which higher mode effects are significant and in which the story shear force vs story drift relationships are sensitive to the applied load pattern. This problem can be mitigated, but usually not eliminated, by applying more than one load pattern, including load patterns that account for elastic higher mode effects (e.g. SRSS load patterns). Perhaps most critical is the concern that the pushover analysis may detect only the first local mechanism that will form in an earthquake and may not expose other weaknesses that will be generated when the structure's dynamic characteristics change after formation of the first local mechanism.

2.8) Direction for Present Research

The literature review suggested that Seismic Evaluation of R.C Buildings using non-linear approach was indeed feasible as it exposes design weaknesses that may remain hidden in an elastic approach. Such weaknesses include excessive deformation demands, strength irregularities, and overloads on potentially brittle points, such as columns and connections (Krawinkler et al, 1998). It was decided to use ANSYS as the FE modeling package. As capacity curve is the output of Pushover analysis, Load Deformation Plot (capacity curve plot) can be obtained from ANSYS non-linear static analysis. Seismic Evaluation of pre-existing R.C buildings is carried out. Firstly, analysis was carried out on symmetrical building for procedure development as per ATC 40 guidelines. Then, analysis is performed on asymmetrical building (L-shape). In first case, evaluation is carried out on the building designed non-seismically and its results have been compared with the analysis of seismically designed building (as per IS 1893:2002). And the affected members have been suggested for strengthening.

CHAPTER-3

ATC-40 PROCEDURE FOR SEISMIC EVALUATION

This Chapter deals with the step-by-step procedure prescribed in ATC-40 for the development of Capacity Curve.

3.1) Step-by-Step Procedure to determine capacity:

The most convenient way to plot force displacement curve is by tracking the base shear and roof displacement. The capacity curve is generally constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This is generally valid for buildings with the fundamental periods of vibration upto about 1 second. For more flexible buildings with the fundamental period > 1 second, the analyst should consider addressing higher mode effects is the analysis.

1) Create a computer model of the structure following the modeling rules as per ATC-40.
2) Classify each element in the model as either primary or secondary.
3) Apply lateral storey forces to the structure in proportion to the product of the mass and fundamental mode shape. This analysis should also include gravity loads.

[As the name implies, it is the process of pushing horizontally with a prescribed loading pattern. Incrementally untill the structure reaches a limit state. There are several levels of sophistication that may be used for the pushover analysis]

i) Simply apply a single concentrated horizontal force at the top of the structure (for one story building)
ii) Apply lateral forces to each storey in proportion to the standard code procedure without the concentrated force F_t at the top
i.e. $F_x = (W_x h_x / \Sigma W_x h_x) \times V$ ……..(1)
iii) Apply lateral forces in proportion to the product of storey masses and first mode shape of the elastic model of the structure
i.e. $F_x = (W_x \Phi_x / \Sigma W_x \Phi_x) \times V$. ……..(2)
The capacity curve is generally constructed to represent the first mode response of the structure based on assumption that the fundamental mode of vibration is the predominant response of the structure.
iv) Same as level three untill first yielding. For each increment beyond yielding, adjust the forces to be consistent with changing deflected shape.
v) Similar to (iii) & (iv) above, but include the effects of the higher mode of the vibration in determining yielding in individual structural elements while plotting the capacity curve for the building in terms of first mode lateral forces and displacements. The higher mode effects may be determined by doing higher mode pushover analysis. (i.e. Loads may be progressively implied in proportion to a mode shape other than the fundamental mode shape to determine its in elastic behavior) For the higher modes the structure is being both push & pulled concurrently to maintained mode shape.

4) Calculate member forces for the required combinations of vertical and lateral load.
5) Adjust the lateral force level so that some elements (on group of elements) are stressed to with in 10% of its member strength.
6) Record the Base shear and the roof displacement. (It is also useful to record member forces & rotations because they will be needed for the performance check)
7) Revise the model using zero (or very small) stiffness for the yielding elements.
8) Apply a new increment of lateral load to the revise structure such that another element (or group of elements) yields.

[The actual forces and rotations for elements at the beginning of the increment are equal to those at the end of the previous elements. However, each application of an increment of lateral load is a separate analysis, which starts from zero initial conditions.

Thus, to determine when the next elements yields, it is necessary to add the forces from the current analysis to the some of those from the previous increments]
9) At the increment of the lateral load and the corresponding increment of roof displacement to the previous total to give the accumulated values of base shear and roof displacement.
10) Repeat steps 7,8 & 9 untill the structures reaches an ultimate limit such as: instability from P-Δ effects, distortions considerably beyond the desire performance level, an element reaching a lateral deformation level at which significant strength degradation begins.

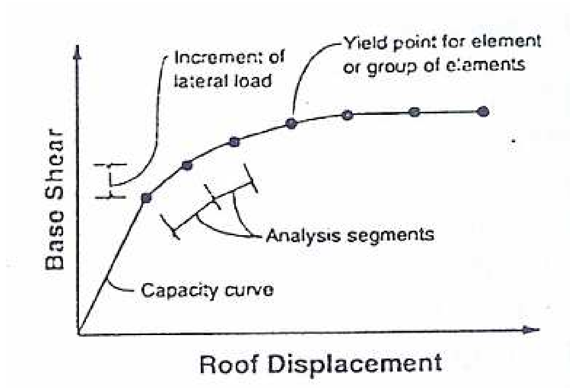


Fig.3.1 Capacity Curve

3.2) Conversion of Capacity curve to the capacity spectrum:

To use the capacity spectrum method it is necessary to convert the capacity curve, which is in terms of base shear and roof displacement to what is called a capacity spectrum, which is a representation of the capacity curve in Acceleration Displacement Response Spectra (ADRS) format i.e. (S_a vs S_d). The required equations to make the transformation are:

$$PF_1 = \{\Sigma_{i=1} (w_i \Phi_{i1})/g\} / [\Sigma_{i=1} \{w_i (\Phi_{i1})^2/g\}]$$

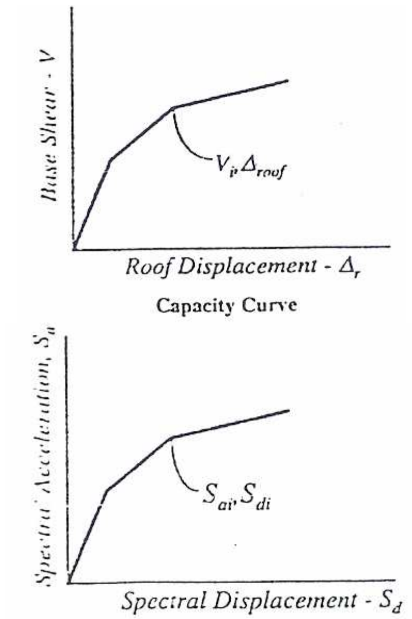
$$\alpha_1 = \{\Sigma_{i=1} (w_i \Phi_{i1})/g\}^2 / \{\Sigma_{i=1} (w_i/g\} X [\Sigma_{i=1} \{w_i (\Phi_{i1})^2/g\}]$$

$$S_a = (V/W)/\alpha_1$$

$$S_d = (\Delta_{roof}) / (PF_i \Phi_{roof.1})$$

Where, PF_i = Model participation factor for the first natural mode, α_1 = Model mass coefficient for the first natural mode,W_i /g = mass assign to level i, Φ_{il} = amplitude of mode one at level i, N = Level N, the level which is the uppermost in the main portion of the structure.

In order to develop the capacity spectrum from the capacity curve it is necessary to do a point by point conversion to first mode spectral coordinates any point V_i,Δ_{roof} on the capacity curve is converted to the corresponding point S_{ai} ,S_{di} on the capacity spectrum using the equations written above.



Capacity Spectrum

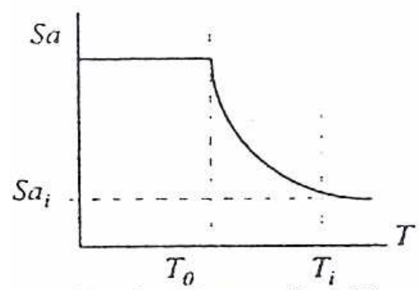
Fig.3.2 Capacity Spectrum Conversion

Every point on a response spectrum curve is associated with a unique spectral acceleration $S_{a,}$ Spectral Velocity $S_{v,}$ Spectral displacement $S_{d,}$ and period T, to convert a spectrum from a standard S_a VS T format found in a building code to ADRS format it is necessary to determine the value of S_{di} for each point on the curve $S_{ai,}$ $T_{i,}$ this can be done with equations:

$S_{di}=(T_i{}^2 \text{ x } S_{ai} \text{ x } g) / 4\pi^2$. ………(3)

Standard demand response spectra contain a range of constant spectral acceleration and second range of constant spectral velocity. Spectral acceleration and displacement at period $T_{i,}$ are given by

$S_{ai}\, g = (2\pi S) / T_i$, $\quad S_{di} = (T_i \text{ x } S_v)/ 2\pi$ ....(4)



Standard Format (Sa vs T)

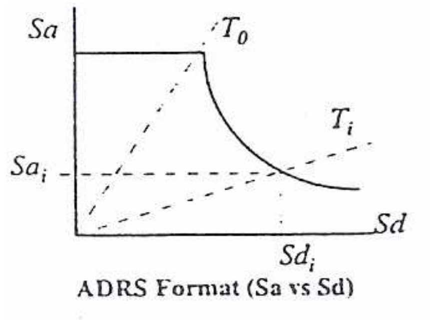


Fig.3.3 Response Spectrum Conversion

3.3) Calculating performance point:

1) A First choice of point a_{pi}, d_{pi} could be the displacement obtained using the equal displacement approximation or it might be the end point of the capacity spectrum or it might be any other point chosen on the basis of engineering judgement.
2) Develop the demand spectrum as shown in figure 3.4, draw the demand spectrum on the same plot as the capacity spectrum as shown in figure 3.6
3) Referred to figure 3.8, determine if the demand spectrum intersects the capacity spectrum at point a_{pi} d_{pi} or if the displacement at which the demand spectrum intersects the capacity spectrum d_i is with in acceptable tolerance d_{pi} as shown in figure 3.9.
4) If the demand spectrum does not intersects the capacity spectrum with in acceptable tolerance than select a new a_{pi} d_{pi} point.
5) If the demand spectrum intersects the capacity spectrum with in acceptable tolerance than the trial performance points $a_{pi,}$ d_{pi} is the performance point, a_p d_p and the displacement d_p represents the maximum structural displacement expected for the demand earthquake.

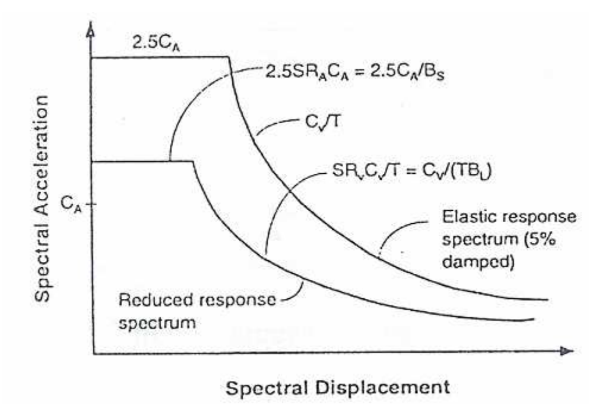


Fig.3.4 Reduced Response Spectrum

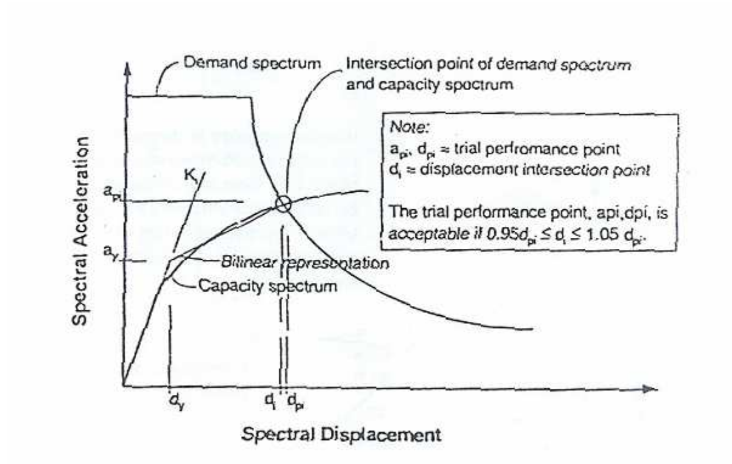


Fig.3.5 **Intersection Point of Demand and Capacity Spectrums within Acceptable Tolerance.**

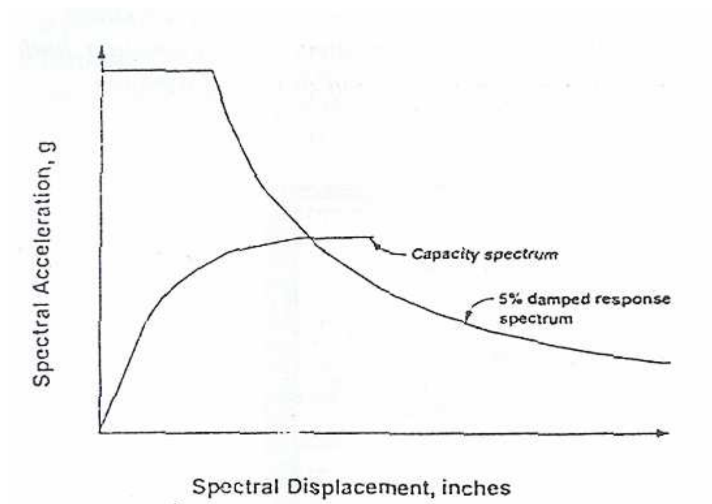


Fig.3.6 Capacity Spectrum After Step 2

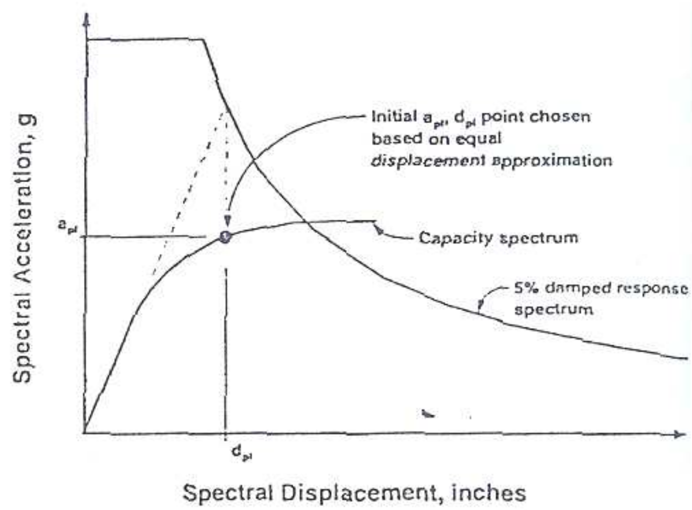


Fig.3.7 Capacity Spectrum After Step 3

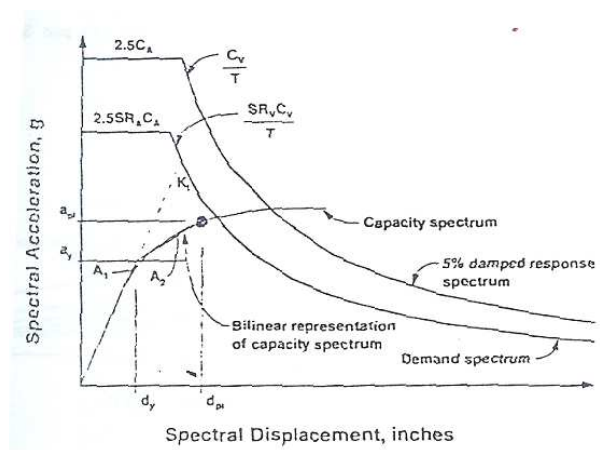


Fig.3.8 Capacity Spectrum After Step 5.

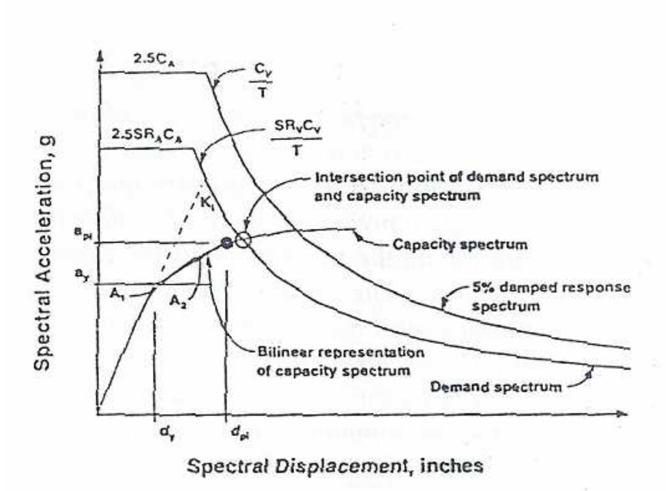


Fig.3.9 Capacity Spectrum After Step 6

CHAPTER-4

SEISMIC EVALUATION OF SYMMETRICAL BUILDING FOR PROCEDURE DEVELOPMENT

To facilitate the procedure provided in ATC 40, a 3-D finite element model (single bays along x and z-axis having height of 10.5m) was built. This Chapter deals with the Analysis of this Symmetrical Frame using Finite Element Package ANSYS 5.4. Capacity Curve, which is a Load-Deformation plot is obtained by using ANSYS. Staad Pro 2003 has been used for designing purpose and modes shapes calculations. After Viewing Cracking pattern, Strengthening of affected members is suggested.
The FEM model includes all the structural components of the building and is composed of 3 elements of: SOLID 65. Reinforcement is incorporated by Volume Ratio. Modal Analysis has been performed by using Staad Pro 2003 and Non-Linear Static Analysis is performed on the Model by using ANSYS 5.4.

4.1) Building Taken for Procedure Development

The Base of the Building has horizontal dimension of 6m x 5m.It has single bays along x-axis and z-axis. Height of the building is 10.5m with each storey height of 3.5m.It is assumed to be situated in Zone 4 with Z = 0.24.Structural details are as follows:
Slab Thickness is assumed to be 125mm.

Table: 4.1

Element	Dimensions (m)	Steel	Ties
Columns	0.3 X 0.3	4-20mm ϕ	8mm @ 150mm c/c
Beams	0.23 X 0.23	2-16mm ϕ (at centre) 2-16mm ϕ (at ends)	8mm @ 150mm c/c

The FEM model is based on the aforementioned structural components.

4.2) Loading Calculations:

Second Floor:
Slab = 0.125 x 6 x 5 x 25 = 93.75 KN
Beams = 0.23 x 0.23 x 2 (5 + 6) x 25 = 30.95 KN
Columns = 0.3 x 0.3 x 3.5/2 x 25 x 4 = 15.75 KN
Total = 147.45 KN
First Floor and Ground Floor:
Slab = 0.125 x 6x 5 x 25 = 93.75 KN
Beams = 0.23 x 0.23 x 2 (5 + 6) x 25 = 30.95 KN
Columns = 0.3 x 0.3 x 3.5 x 25 x 4 = 31.5 KN
Total = 163.2 KN
Total W = 473.85 KN

4.3) Normalized Mode Shapes Calculations: Staad Pro 2003 is used for calculating the mode shapes. Frame modeled in Staad is as shown:

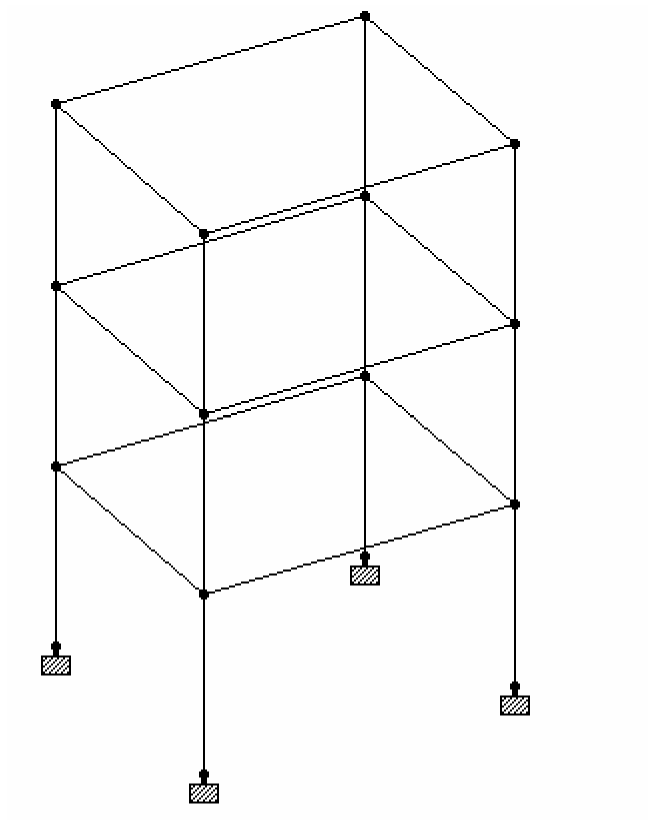


Fig.4.1 Staad Model

4.3.1) Response Spectrum Loading:

Table: 4.2

Parameters	Values Adopted
Combination Method	SRSS
Spectrum Type	Acceleration
Interpolation Type	Linear
Direction	Along x
Damping	0.05
Spectrum Pairs	Sa Vs T corresponding to Medium Soil as per I.S1893:2002

4.3.2) Loading on Beams:

Table: 4.3

Beams	Floor Level	Loading (KN/m)
6m	Roof	12
5m	Roof	10
6m	Typical	13.5
5m	Typical	11

After performing Modal Analysis, the following Dynamic Results are obtained:
First Mode Values:

Table: 4.4

Floor Level	a_{ij}
Roof	7.84
First Floor	3.8
Ground Floor	1

For getting normalized mode shapes, following Formulae has been applied:

$$\phi_{ij} = a_{ij} / (\Sigma\, m_k\, a_{kj}^{2})^{1/2} \qquad \ldots\ldots\ldots\ldots(5)$$

Where, ϕ_{ij} = Normalized i component of j modal vector.

m_k = Mass at k Level,

Corresponding Normalized Mode Shapes are:

Table: 4.5

Floor Level	ϕ_{ij}
Roof	0.00721
First Floor	0.0035
Ground Floor	0.000920

4.4) Description of the ANSYS Model

The model is a three-dimensional, linear, isotropic finite element model. The discretization of domain, geometry of the elements, material properties, boundary condition, loads and analysis procedures are described below.

Finite Element Idealization

Details of the structure are given in the previous section, "Description of the Building." The origin of the coordinate system for the model is located at the northwest corner of the basement floor. The x, y and z- axes are in the W-E, vertical and N-S directions, respectively. The model, for the most part, is composed of the structural components of the building, namely, the columns, and the beams. The foundation is not included in the model, but its behavior is reflected in the boundary conditions. Element type, which is described in the next section, is used in the model: SOLID 65 element. A total of 312 elements were generated for the model. Figure 4.2 shows the meshed finite element model.

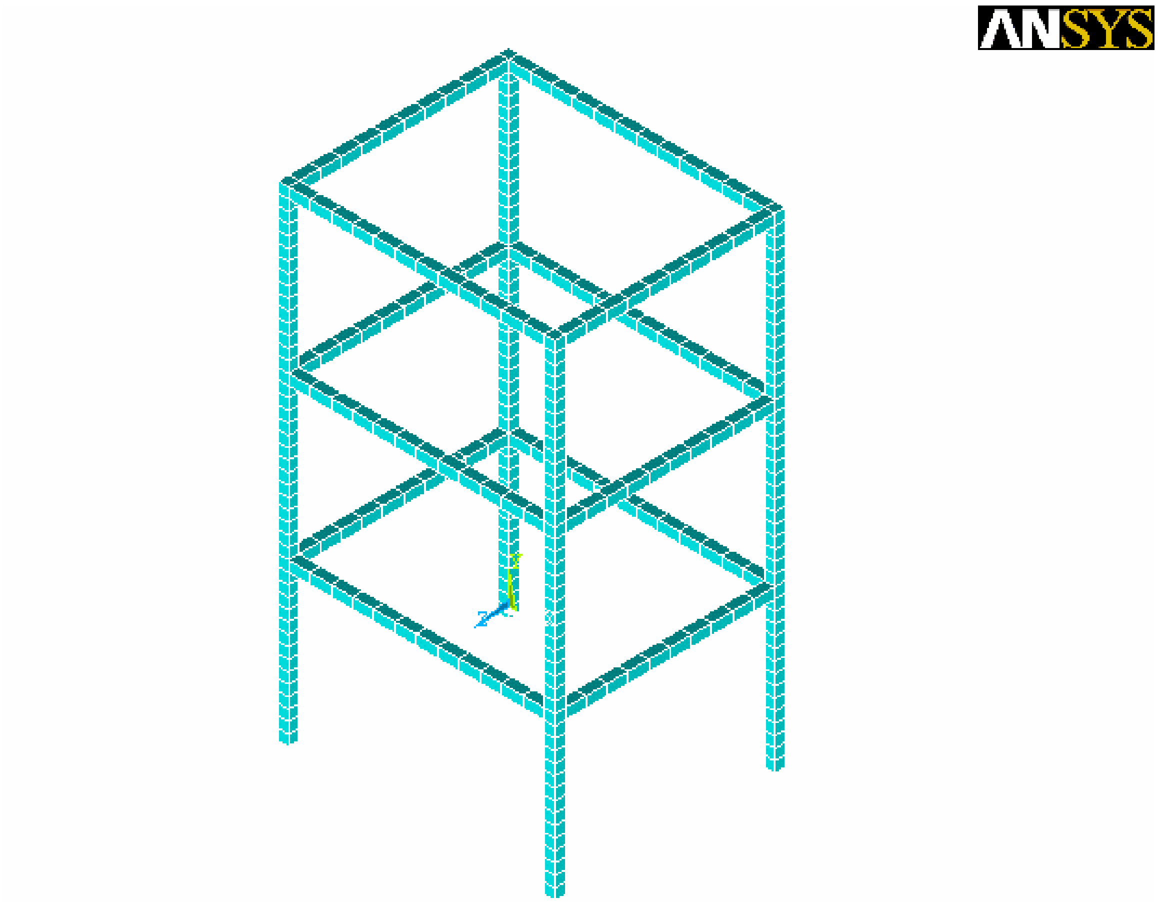


Fig.4.2 Finite Element Abstract of the Building

4.4.1) Element Types:

SOLID65 is used for the three-dimensional modeling of solids with or without reinforcing bars (rebars). The solid is capable of cracking in tension and crushing in compression. In concrete applications, for example, the solid capability of the element may be used to model the concrete while the rebar capability is available for modeling reinforcement behavior. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. Up to three different rebar specifications may be defined. The most important aspect of this element is the treatment of nonlinear material properties. The concrete is capable of cracking (in three orthogonal directions), crushing, plastic deformation, and creep. The required material property inputs for the element are Young' modulus (EX) and density (DENS). Other constant and material input options are also available. Three Solid 65 elements are used in analysis because the element constants are varying along x,y and z-directions respectively.

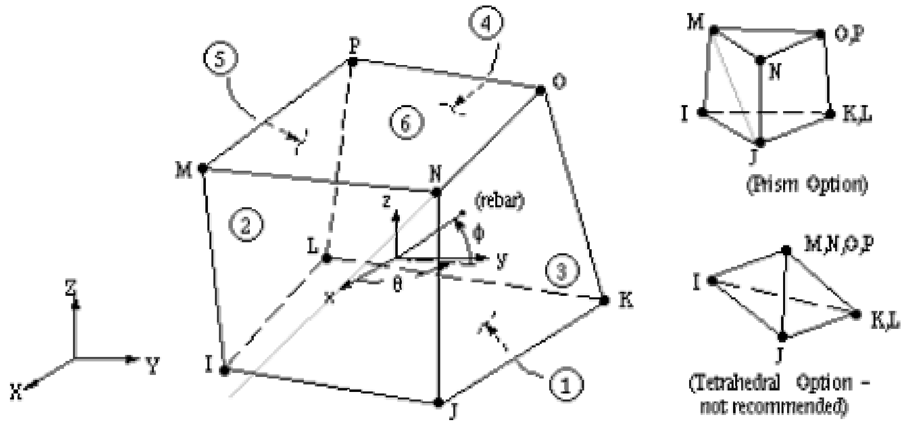


Fig.4.3 SOLID65 3-D Reinforced Concrete Solid

4.4.2) Element Constants

Nine sets of material constants were used in the model, three for each type of elements. The number of material constant sets mainly reflects the number of element thickness in the model. Tables and list the element constant sets for the beam and columns, respectively.

Table: 4.6 Element constants for column (Set 1):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.00188	0.00188	0.01396
Theta (orient.Angle)	0	0	90
Phi (orient.Angle)	90	0	0

Table: 4.7 Element constants for Beam along x-axis (Set 2):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.00246	0.00188	0.0182
Theta (orient.Angle)	90	0	0
Phi (orient.Angle)	0	0	90

Table.4.8 Element constants for column (Set 3):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.00246	0.00188	0.0182
Theta (orient.Angle)	90	0	0
Phi (orient.Angle)	0	90	0

4.4.3) Material Properties

Twelve sets of material properties are used in the model. The Young's modulus is calculated with the following formula:

$E = 5000 \sqrt{f_{ck}}$

Where, f_{ck} is the characteristic cube compressive strength of concrete in N/mm^2. The Poisson's ratio for concrete ranges from 0.15 to 0.25 [4] and a value of 0.2 is chosen for the model. Poisson's ratio for steel is taken as 0.3. The density of concrete is taken as 2500 Kg per cum was selected. It must be pointed out that in ANSYS, the density must be entered as mass per unit volume, not weight per unit volume.

Table: 4.9 Material Properties.

Material Set	**EX N/m^2**	**Density N/m^3**	**NUXY**
1	2e10	25000	0.2
2	2e11	78500	0.3

Data Table Values: Type of Data Table- Concrete

Table: 4.10

S.No.	**Parameter**	**Value Entered**
1.	Uni Tensile.Str.	$1.65e^6$
2.	Uni Comp.Str.	-1

4.4.4) Modeling:

FE Model is created by using Volume by block dimension option, that is by giving the dimensions of beams and columns as per drawing. Columns and Beams were modeled storey wise. Firstly, the columns at base level (ground storey) were modeled. Then beams were modeled into the columns at respective heights. The following model is created as shown below:

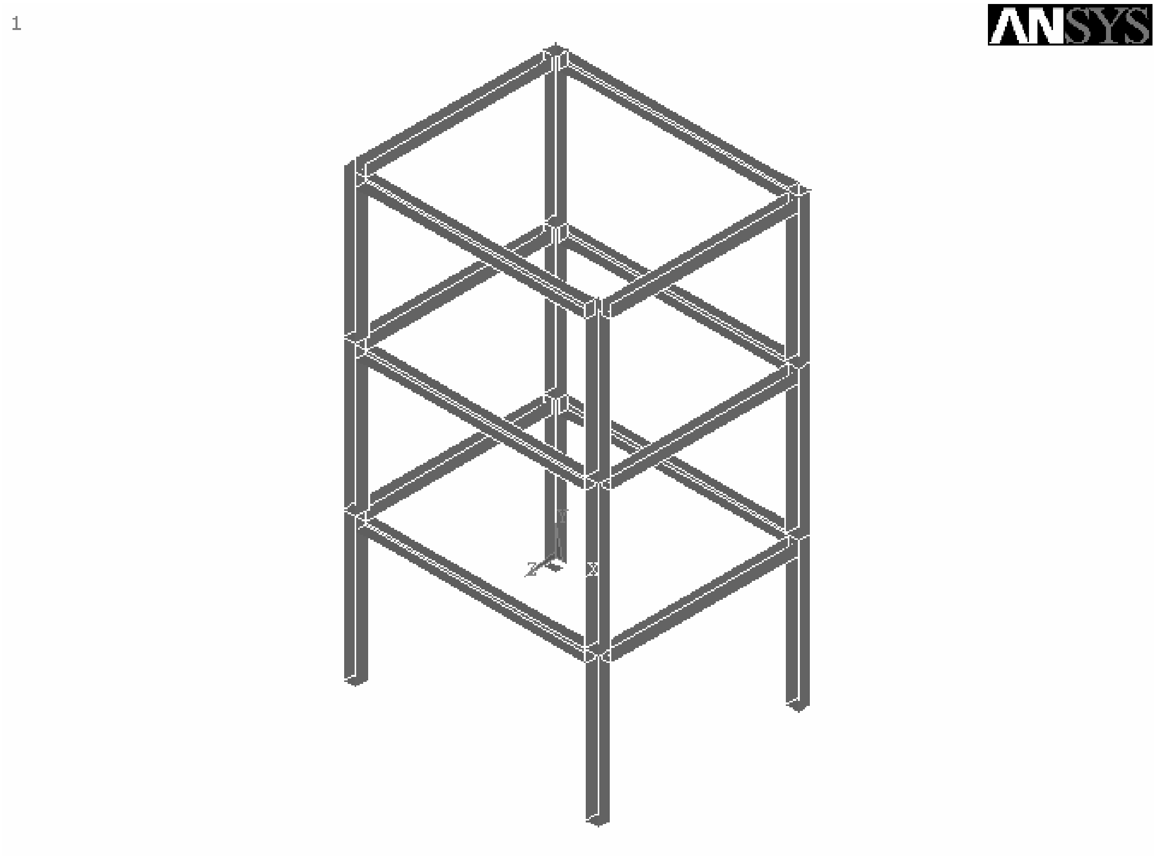


Fig: 4.4 Finite Element Model

4.4.5) Meshing:
Fine Mesh is created on this model obtained. Column lines (storey wise) have been divided into 15 parts, beams along x-axis have been divided into 12 divisions and number of divisions for beams along z-axis are 10. These divisions have been made so that every node at connection should be properly merged. Finite Mesh Model is shown on next page:

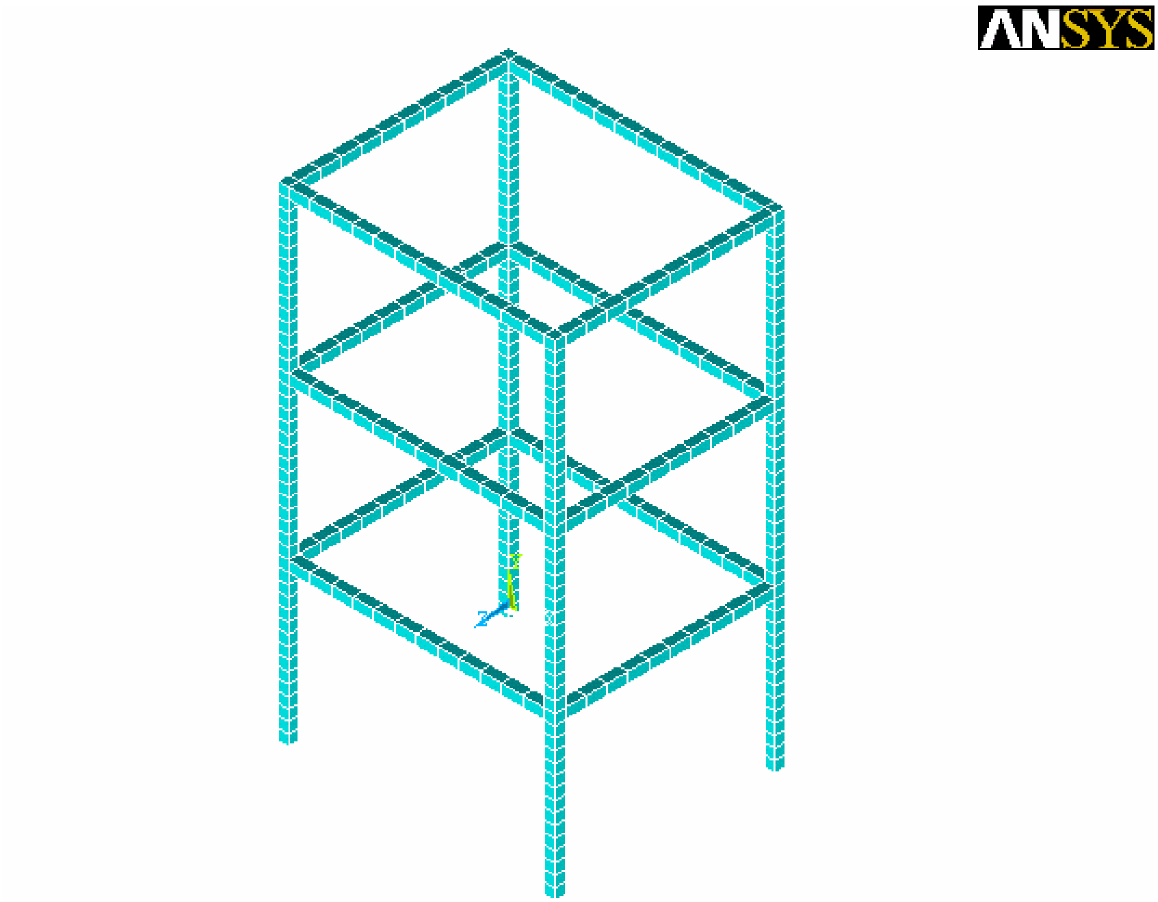


Fig: 4.5 Meshed Finite Element Model

4.4.6) Boundary Conditions and Loading:

The boundary conditions for the analysis are: a) all the nodes of the columns at the foundation level are fixed, i.e., the displacements, translational or rotational, were set to zero.

Horizontal Displacement of 0.075 m is applied at top nodes in x-direction.

4.4.7) Analysis Procedure and Options:

Non-Linear Static Analysis is done on this frame. Various options for the analysis procedure are as follows:

Load Step Options:

Output Controls:

i) Solution Printout Controls: **Table: 4.11**

Options	Value Adopted
Item for Printout Control	All Items
Print Frequency	Every Substep

ii) Controls for Database and Results File Writing: **Table: 4.12**

Options	Value Adopted
Item to be controlled	All Items
File Write Frequency	Every Substep

Time/ Frequency:

i) Time and Time Step Options: **Table: 4.13**

Options	Value Entered
Time at end of Load Step	1

i) Time and Substep Options: **Table: 4.14**

Options	Value Entered
Time at end of Load Step	1
Number of substeps	250
Maximum no. of substeps	500
Minimum no. of substeps	100

Non-Linear: **Table: 4.15**

Options	Value Adopted
Solution Control	ON

The following results are obtained after analyzing the above-described model.
As the earthquake excitation is along x-axis, all the beams and columns are deflected in x-axis. The deformed shape of the model is as shown:

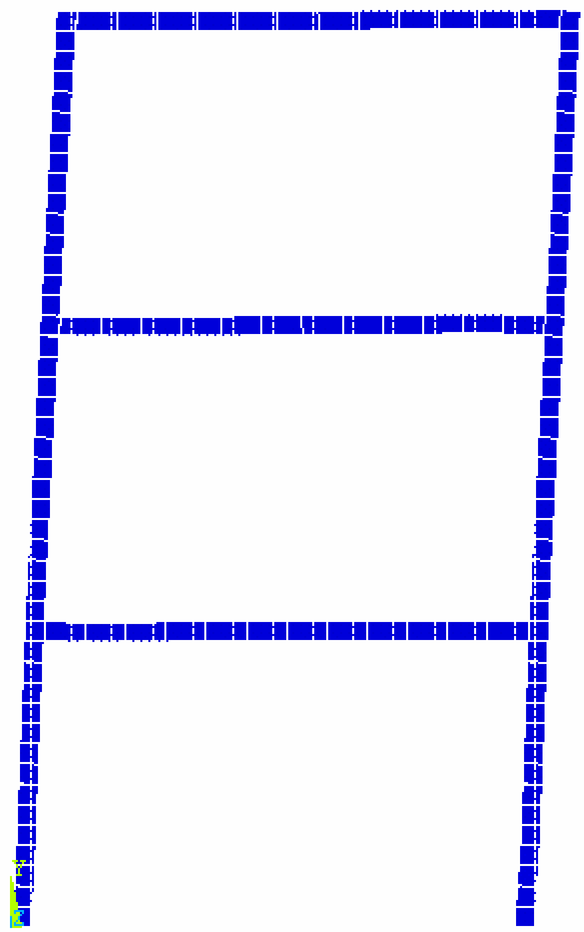


Fig.4.6 Deformed Shape

4.5) Capacity Curve Plot:

As the Capacity Curve is plot between Base Shear and Roof Displacement, so the graph is plotted between the Base Reactions and Roof Displacements obtained from Analysis

Results.

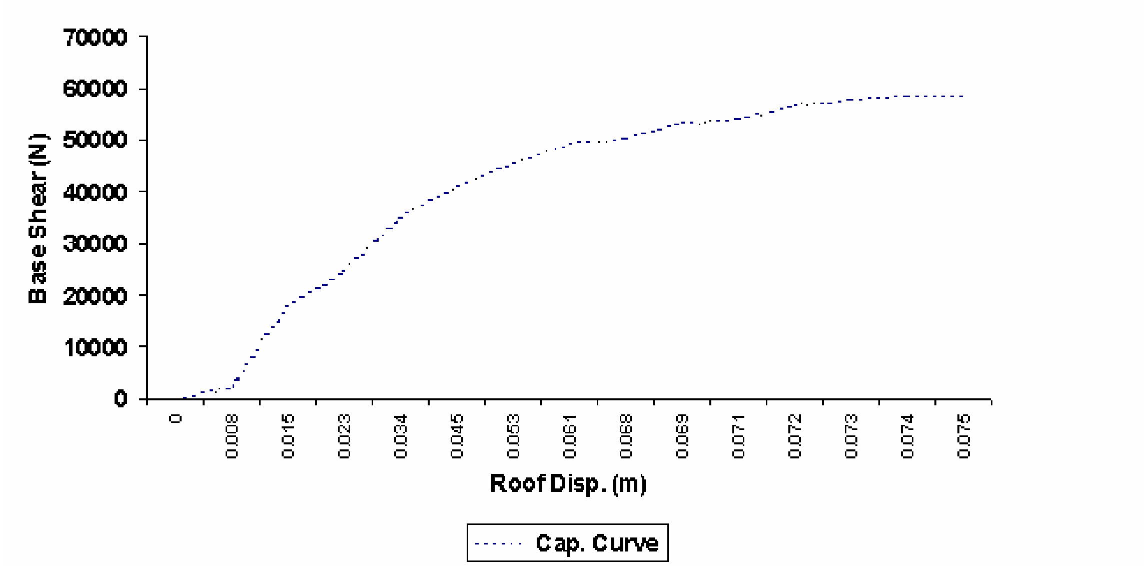


Fig.4.7 Capacity Curve

Development of Capacity Curve for an existing Building, in itself, is extremely useful and will yield insights into the building's performance characteristics as well as methods of retrofit. However, to judge acceptability for a given performance objective, either for the as-is condition or for a retrofit scheme, the probable maximum displacement for the specified ground motion must be estimated.

4.5.1) Conversion of Capacity Curve into ADRS Format:

Next Step is to convert the Capacity Curve into Capacity Spectrum as described in section-3.2.

Table: 4.16 Conversion of Capacity Curve to Capacity Spectrum

Point	V (N)	δ_R (m)	V/W	$PF_{R1}= PF_1.\phi_{roof1}$	α_1	Sa (g)	Sd (m)
A	45510	0.053	0.096	1.31	0.064	1.5	0.040
B	53277	0.069	0.11	1.26	0.063	1.75	0.055
C	57845	0.073	0.12	1.21	0.061	1.96	0.061
D	58594	0.075	0.124	1.04	0.060	2.07	0.072

Capacity Spectrum which is a Plot between Sa and Sd is as shown:

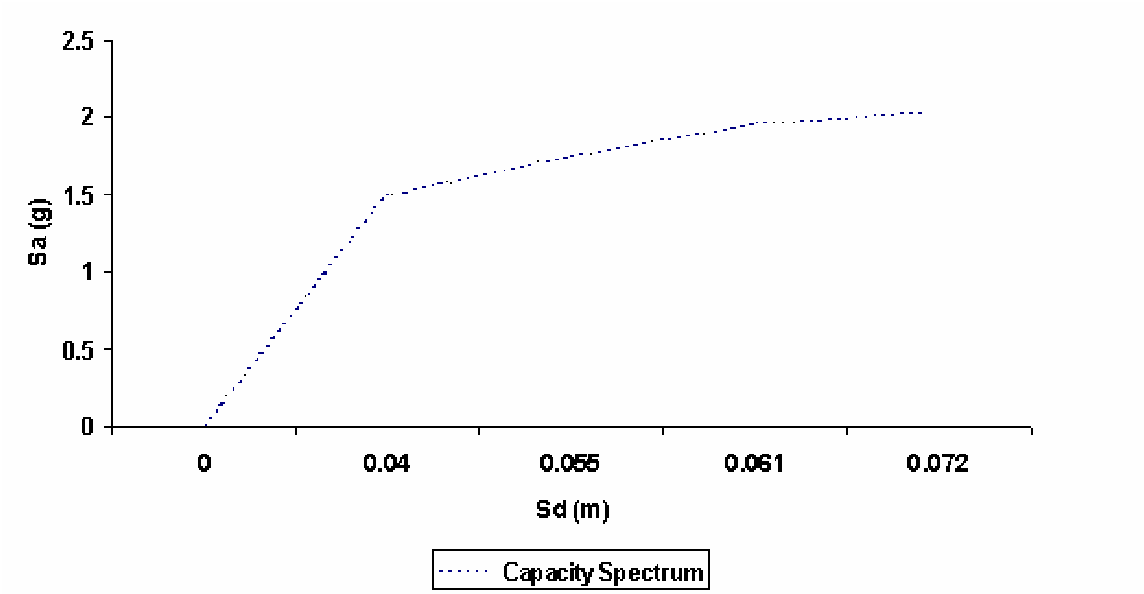


Fig.4.8 Capacity Spectrum

4.6) Response Spectrum Plot:
Response Spectra (for 5 percent Damping) is taken from I.S 1893:2002, for Type II (Medium Soil) which is a Plot between Sa/g and T, as shown:

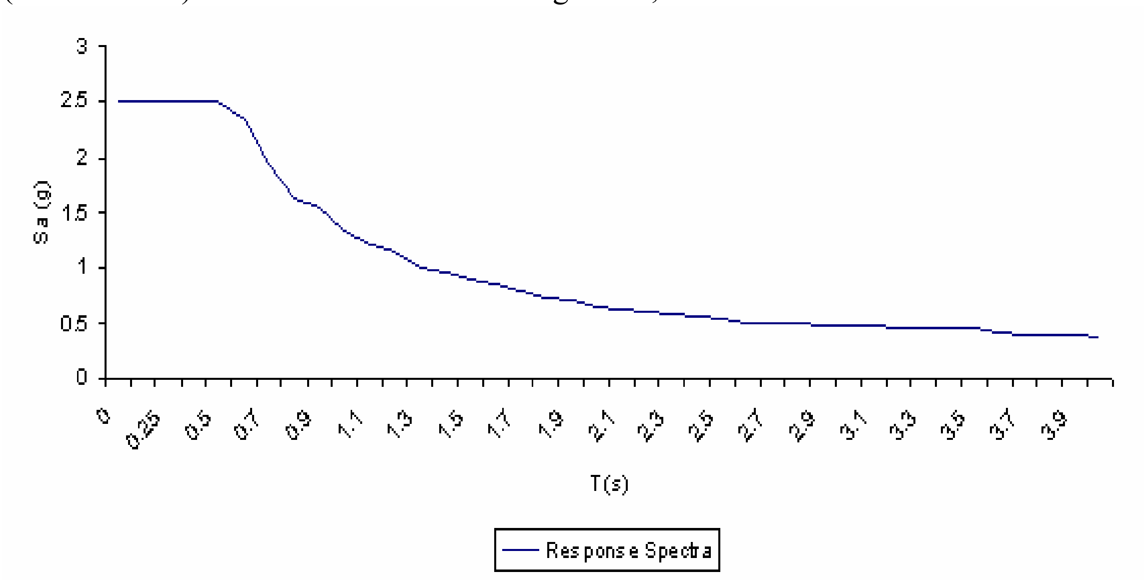


Fig: 4.9 Response Spectra for 5 percent Damping; Standard Format (Sa vs T)

Every point on a response spectrum curve has associated with it a unique spectral acceleration Sa, Spectral Displacement Sd, and period T. To convert a spectrum from the standard Sa vs T format to ADRS format, it is necessary to determine the value of Sd_i for each point on the curve, Sa_i, T_i. This can be done with the equation as described in Section-3.2. The following is Response Spectrum in ADRS Format:

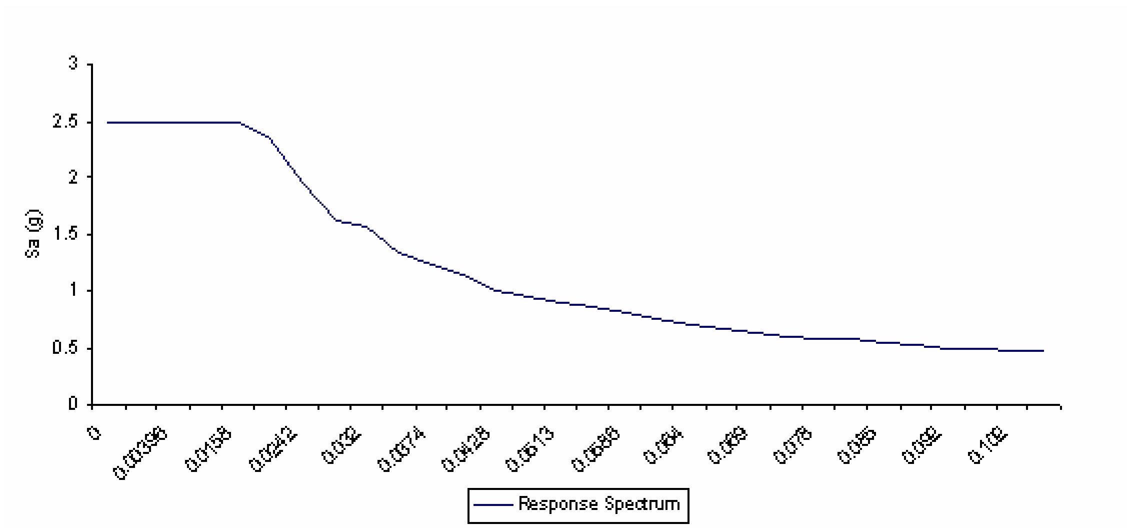


Fig.4.10 ADRS Format (Sa v Sd)

Now, the Capacity Spectrum and Response Spectrum are plotted on the same graph so as to get performance point and target displacement.

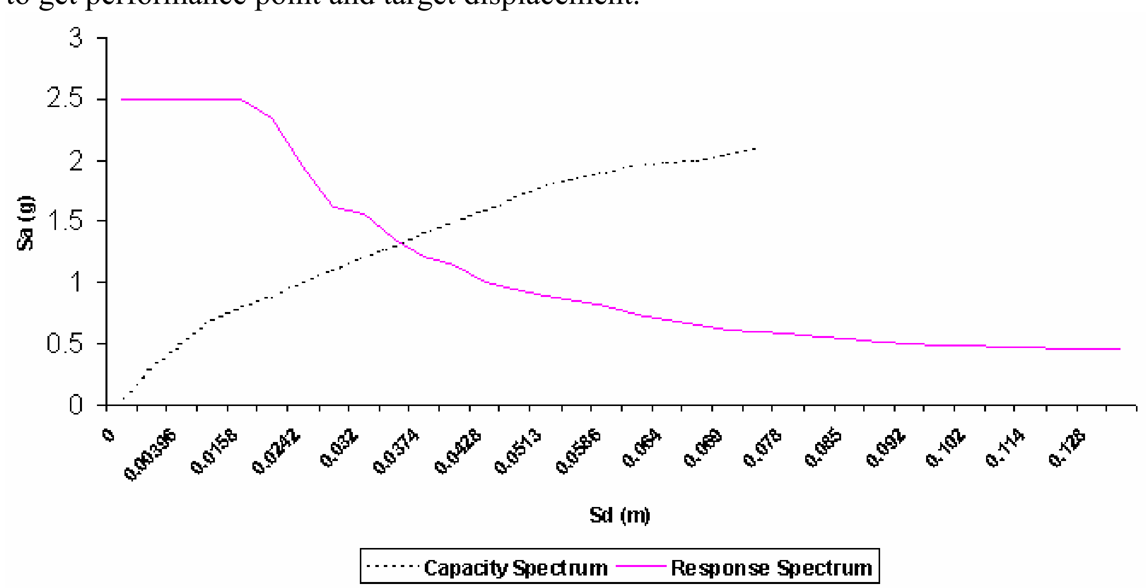


Fig: 4.11 Intersection of Response Spectrum and Capacity Spectrum

Following **Observations** are made from the above Graph:

a) Capacity Spectrum meets Response Spectrum at 0.037m (S_d), and the corresponding Base Shear V=41510 N (Table: 4.16). Base Shear (V_B) as per IS 1893:2002 is 28431 N, that is the structure can resist horizontal shear upto that value. Value of Target Displacement is 0.045m, that is the margin of safety decreases beyond that target value.

b) Response Spectrum intersects Capacity Spectrum in in-elastic range, which means that the structure is unsafe.

Now, we use the value of target displacement (45mm), to find out the cracking at that particular point in the structure. In General Post Processor in ANSYS, the displacement value of 45mm is entered to plot cracking in the Structure. The following figure shows the cracking in the structure elements.

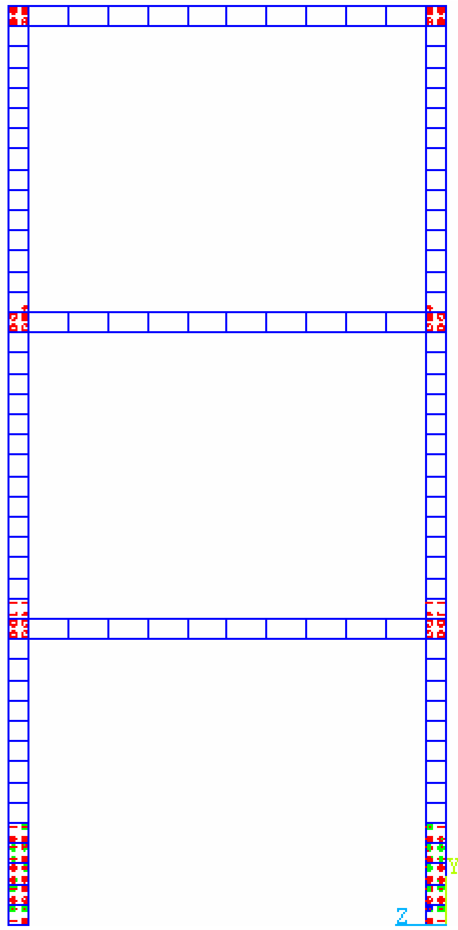


Fig: 4.12 Back View showing cracking.

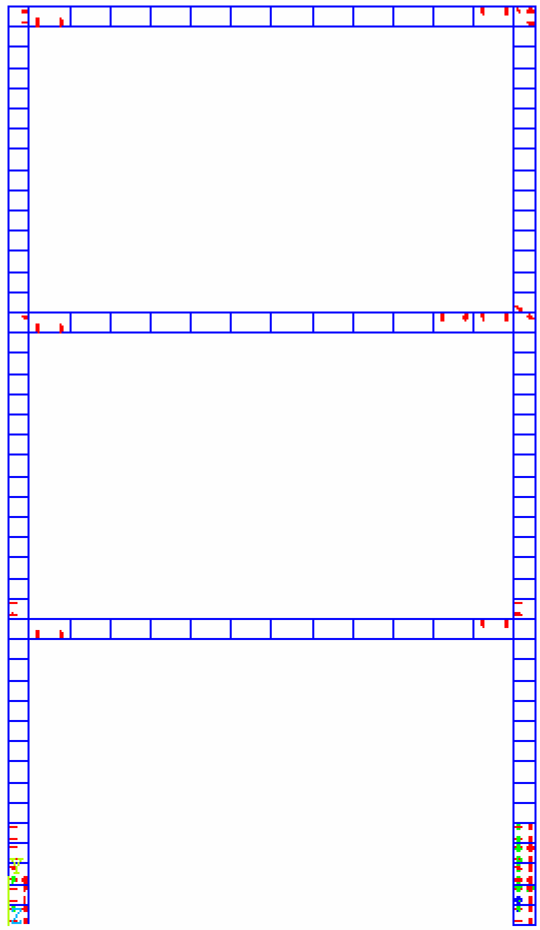


Fig: 4.13 Front View Showing Cracking

Following **Observations** are made from above Analysis:

a) It can be observed from the cracking pattern that the majority of cracking has occurred at base columns and column-beam joints at first storey. That is, the Base Columns are severely affected by the earthquake excitation.

b) First Storey Columns have also been affected, having several cracks at their bottom joints.

c) Beams along z-axis are least affected in this structure. Cracking occurred in the Beams which are along the x-axis (i.e along excitation) and these beams cracked near their ends (i.e at their junction with columns).

d) It can be concluded that the Strengthening is required in the columns at Base and First Storey Level.

CHAPTER-5

SEISMIC EVALUATION OF L-SHAPED BUILDING

In this Chapter, an Unsymmetrical building (L-shape) has been taken. First part of this Chapter deals with the Seismic Evaluation of L-shape building designed by Dead Load and Live Load only (without incorporating I.S 1893:2002 guidelines). And the second part deals with the Seismic Evaluation of L-shape building designed as per I.S 1893:2002. After Performing Analysis, results of both have been compared and the strengthening is suggested for most severely affected members.

5.1) Description of the Building:

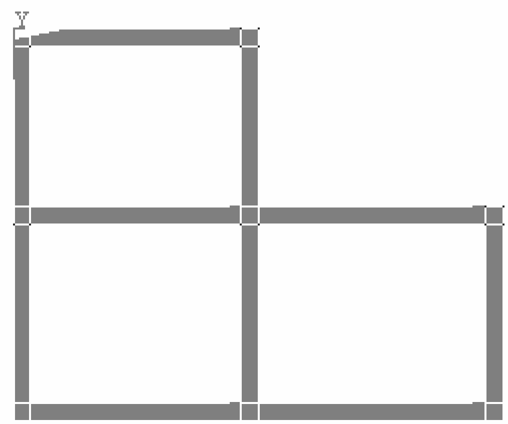


Fig: 5.1 Plan of L-shaped building

Above Figure shows the Plan of L-shape building. Along x-axis, the bay span is 5m, and along z-axis, the bay span is 4m. Height of the building is 14m with each storey height of 3.5m.

This building is designed by Staad.Pro 2003 for Dead Load and Live Load case only for getting the Reinforcement Details. Various Parameters used in Staad.Pro are:

Section Sizes: **Table: 5.1**

S.No.	Element	Dimension (m)
1.	Columns	0.345 x 0.345
2.	Beams	0.345 x 0.5

5.2) Loading Calculations:

Roof Level:

Slab = 0.125 x 25 = 3.125 KN/m^2

Mud Fuskha = 0.15 x 16 = 2.4 KN/m^2

Tiles = 0.04 x 20 = 0.80 KN/m^2

Total Load = 6.5 KN/m^2 (working)
= 9.75 KN/m^2 (factored)

Floor Level (Typical)**:**
Slab = 0.125 x 25 = 3.125 KN/m^2
Screed = 0.065 x 20 = 1.3 KN/m^2

Finish = 0.04 x 24 = 0.96 KN/m^2

Partition = 1.5 KN/m^2
Total Load = 7 KN/m^2 (working)
= 10.5 KN/m^2 (factored)

5.2.1) Loading on Beams:

Table: 5.2

Beams	Floor Level	Loading (KN/m)
5m	Roof	14
4m	Roof	10
5m	Typical	15
4m	Typical	11

Live Load: 3.5 KN/m^2

Staad Model of this building Frame is as shown:

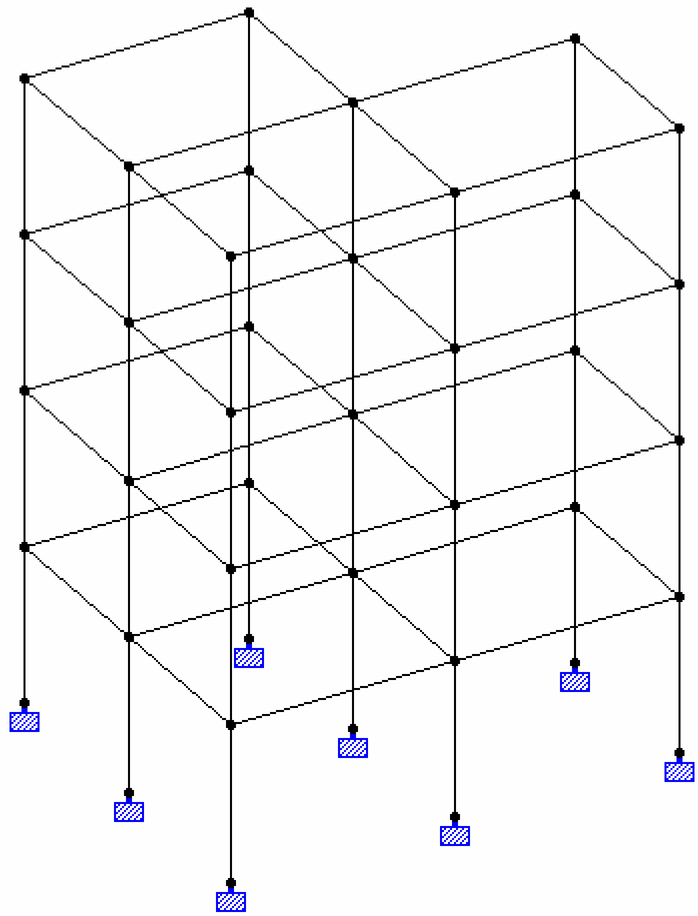


Fig: 5.2 Staad Model

Structural details (As per Analysis and Design on Staad.Pro) are as follows:

Table: 5.3

Element	Dimensions (m)	Steel	Ties
Columns	0.345 X 0.345	4-16mm ϕ	8mm @ 150mm c/c
Beams	0.345 X 0.5	3-16mm ϕ (positive steel at centre) 2-16mm ϕ (negative steel at ends)	8mm @ 150mm c/c

The FEM model is based on the aforementioned structural components.

5.2.2) Weight Calculations:

Third Floor (Roof):

Slab = 0.125 x 4 x 5 x 25 x 3 = 187.5 KN

Beams = 5 x 0.345 x 0.5 x 25 x 5 + 4 x 0.345 x 0.5 x 25 x 5 = 194 KN

Columns = 0.345 x 0.345 x 3.5/2 x 25 x 8 = 41.66 KN

Total = 381.06 KN

Floor (Typical):

Slab = 0.125 x 4 x 5 x 25 x 3 = 187.5 KN

Beams = 5 x 0.345 x 0.5 x 25 x 5 + 4 x 0.345 x 0.5 x 25 x 5 = 194 KN

Columns = 0.345 x 0.345 x 3.5 x 25 x 8 = 83.32 KN

Total = 423 KN

Total W = 1649.22 KN

5.2.3) Normalized Mode Shapes Calculations:

Response Spectrum Loading as described in Chapter 4 is used.

After performing Modal Analysis, the following Dynamic Results are obtained:

First Mode Values:

Table: 5.4

Floor Level	$\mathbf{a_{ij}}$
Roof	3.68
Second	3.18
First	2.23
Ground	1

For getting normalized mode shapes, Formulae mentioned in **Section: 4.3.2** is used, and the normalized mode shapes are:

Table: 5.5

Floor Level	ϕ_{ij}
Roof	0.0034
Second	0.0029
First	0.002
Ground	0.0009

5.3) Description of the ANSYS Model

The model is a three-dimensional, linear, isotropic finite element model. The discretization of domain, geometry of the elements, material properties, boundary condition, loads and analysis procedures are described below.

Finite Element Idealization

Details of the structure are given in the previous section, "Description of the Building." The origin of the coordinate system for the model is located at the northwest corner of the basement floor. The x, y and z- axes are in the W-E, vertical and N-S directions, respectively. The model, for the most part, is composed of the structural components of the building, namely, the columns, and the beams. The foundation is not included in the model, but its behavior is reflected in the boundary conditions. Element type, which is described in the next section, is used in the model: SOLID 65 element. A total of 664 elements were generated for the model.

5.3.1) Element Types:

SOLID65 is used for the three-dimensional modeling of the Frame. The properties of SOLED65 have already been mentioned in Chapter 4.

5.3.2) Element Constants

Nine sets of material constants were used in the model, three for each type of elements. The number of material constant sets mainly reflects the number of element thickness in the model. Tables and list the element constant sets for the beam and columns, respectively.

Table: 5.6 Element constants for column (Set 1):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.00164	0.00164	0.00676
Theta (orient.Angle)	0	0	90
Phi (orient.Angle)	90	0	0

Table: 5.7 Element constants for Beam along x-axis (Set 2):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.00164	0.0014	0.0059
Theta (orient.Angle)	90	0	0
Phi (orient.Angle)	0	0	90

Table: 5.8 Element constants for column (Set 3):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.00164	0.0014	0.0059
Theta (orient.Angle)	90	0	0
Phi (orient.Angle)	0	90	0

5.3.3) Material Properties:

The values entered for Young’s Modulus (EX), Density, Poisson’s ratio Data Table Values are same as described in Chapter 4.

5.3.4) Modeling:

FE Model is created by using Volume by block dimension option, that is by giving the dimensions of beams and columns as per drawing. Columns and Beams were modeled storey wise. Firstly, the columns at base level (ground storey) were modeled. Then beams were modeled into the columns at respective heights. The following model is created as shown below:

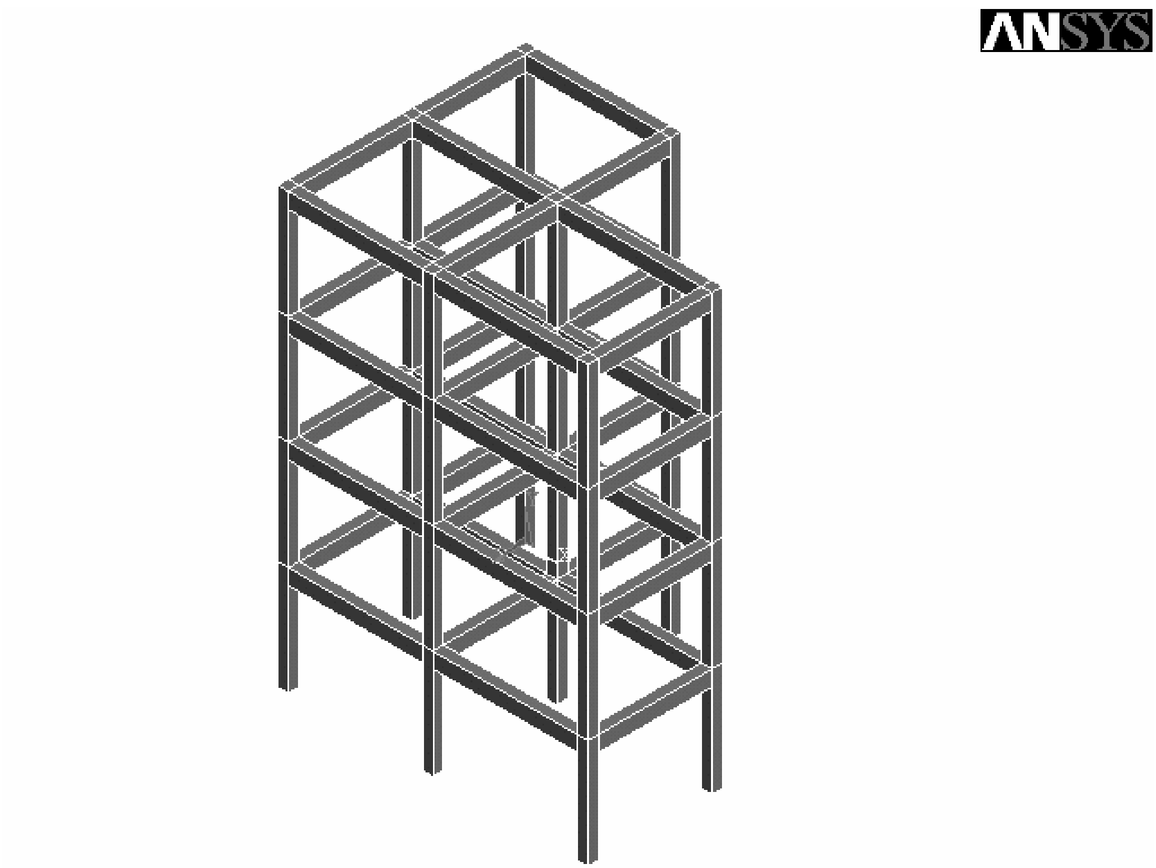


Fig: 5.3 Isometric View

5.3.5) Meshing:

Fine Mesh is created on this model obtained.

Size Controls:

All Column Elements Lines have been divided into 7 divisions, All the Beams along x-axis have been divided into 12 divisions, and the number of divisions for Beams along z-axis is 10. These divisions have been calculated by considering the fact that the mesh should be fine such that all the nodes at Beam-Column Joints should be connected. After Meshing, all the nodes have been merged with Range of Coincidence 0.001 in **Numbering Controls** Option.

Finite Mesh Model is shown below:

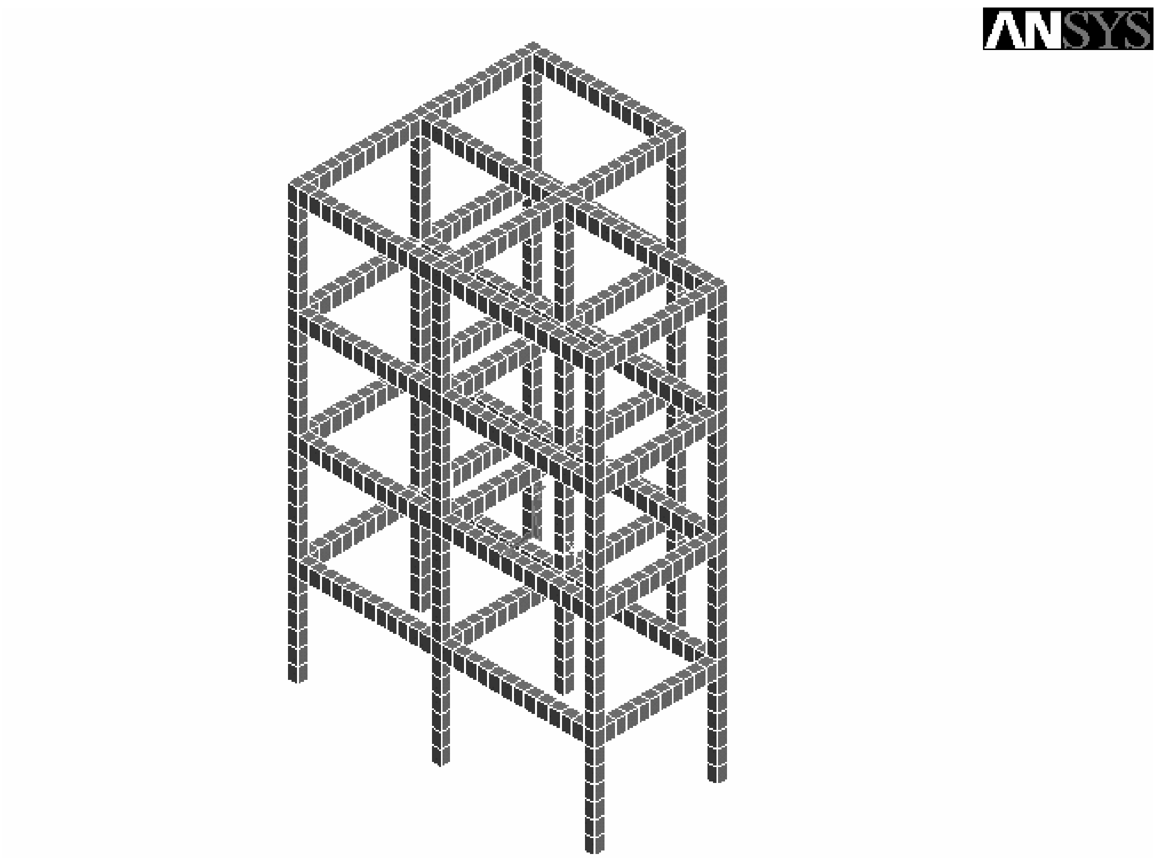


Fig: 5.4 Meshed Model

5.3.6) Boundary Conditions and Loading:

The boundary conditions for the analysis are: a) all the nodes of the columns at the foundation level are fixed, i.e., the displacements, translational or rotational, were set to zero.

Horizontal Displacement of 0.075 m is applied at top Column nodes in x-direction.

5.3.7) Analysis Procedure and Options:

Non-Linear Static Analysis is done on this frame. Various options for the analysis procedure have already been discussed in section 4.4.7**:**

The following results are obtained after analyzing the above-described model.

As the earthquake excitation is along x-axis, all the beams and columns are deflected in x-axis. The deformed shape of the model is as shown:

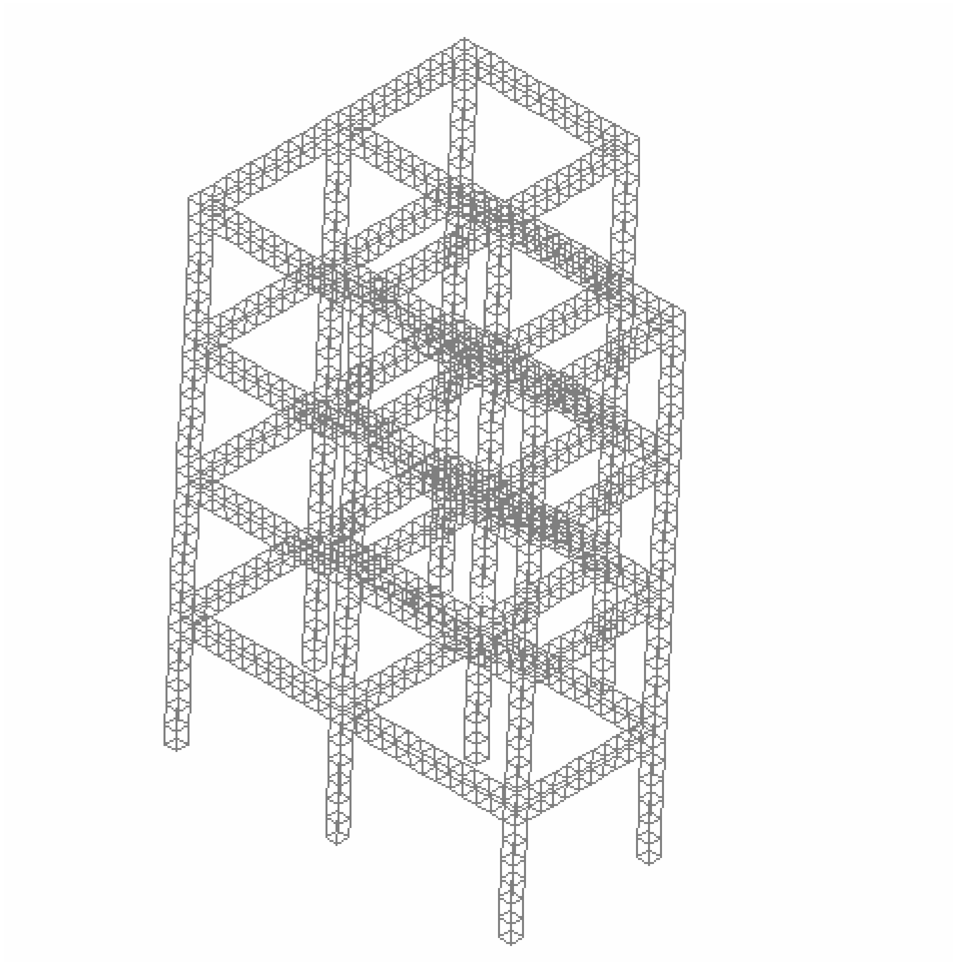


Fig: 5.5 Deformed Shape

5.4) Capacity Curve Plot:

As the Capacity Curve is plot between Base Shear and Roof Displacement, so the graph is plotted between the Base Reactions and Roof Displacements obtained from Analysis Results.

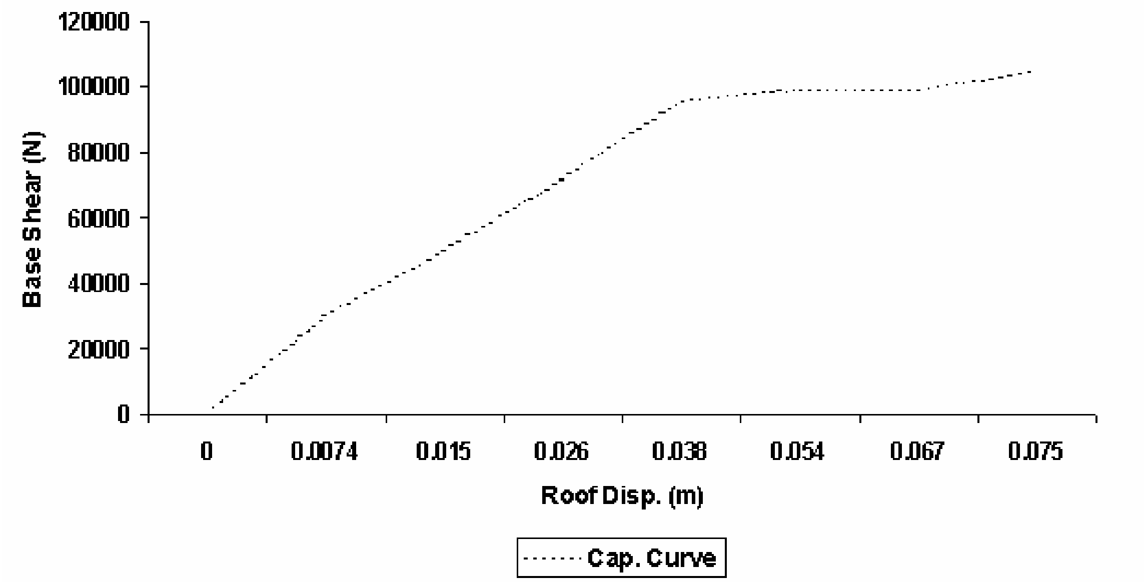


Fig: 5.6 Capacity Curve

Development of Capacity Curve for an existing Building, in itself, is extremely useful and will yield insights into the building's performance characteristics as well as methods of retrofit. However, to judge acceptability for a given performance objective, either for the as-is condition or for a retrofit scheme, the probable maximum displacement for the specified ground motion must be estimated.

5.4.1) Capacity Curve Conversion:

Next Step is to convert the Capacity Curve into Capacity Spectrum as described in Section-3.2.

Table: 5.9 Conversion of Capacity Curve to Capacity Spectrum

Point	V (N)	δ_R (m)	V/W	$PF_{R1}= PF_1.\phi_{roof1}$	α_1	Sa (g)	Sd (m)
A	71710	0.026	0.043	1.27	0.085	0.51	0.020
B	95742	0.038	0.058	1.12	0.075	0.78	0.034
C	98998	0.067	0.060	1.08	0.078	0.79	0.062
D	104942	0.075	0.0065	1.05	0.083	0.82	0.072

Capacity Spectrum which is a Plot between Sa and Sd is as shown:

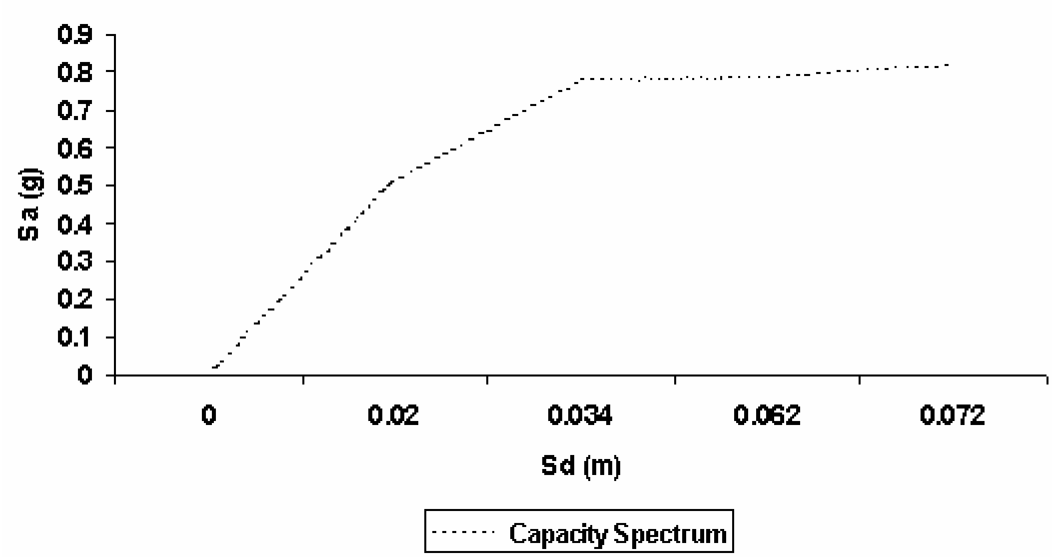


Fig: 5.7 Capacity Spectrum

5.5) Response Spectrum:

Response Spectra (for 5 percent Damping) is taken from I.S 1893:2002, for Type II (Medium Soil) which is a Plot between Sa/g and T has been shown in section 4.6. The conversion into ADRS (Acceleration displacement response spectra) has already been discussed in Section-3.2. The plot below shows the intersection of demand and capacity envelopes.

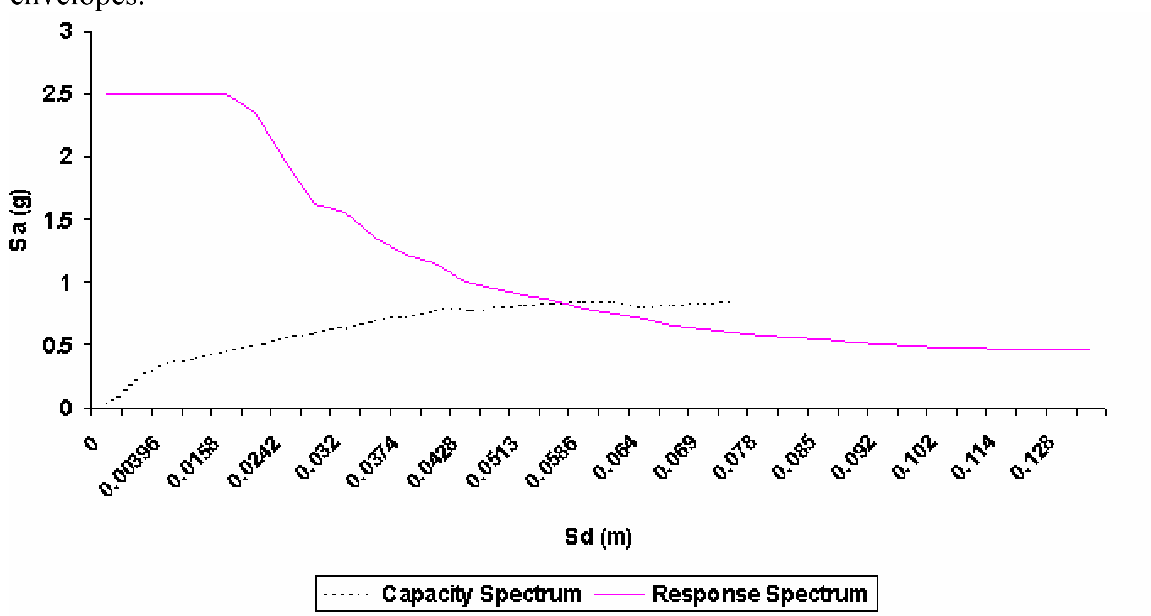


Fig: 5.8 Intersection of Capacity Spectrum and Response Spectrum

Following **Observations** are made from the above Graph:

a. Capacity Spectrum meets Response Spectrum at 0.059m (i.e Sd), and the corresponding Base Shear (V) is 98990 N (from Table: 5.9). Value of Base Shear (V_B) as per IS 1893:2002 is 98953 N, which means that the structure has been designed to resist Base Shear upto 98 KN only and after that the structure fails. Value of Target Displacement is 66mm (from Table: 5.9). Performance Point lies in the non-linear range.
b. Response Spectrum intersects Capacity Spectrum in in-elastic range, which means that the structure is unsafe.

Now, we use the value of target displacement (66mm), to find out the cracking at that particular target displacement in the structure. In General Post Processor in ANSYS, the displacement value of 66mm is entered to plot cracking in the Structure. The following figure shows the cracking in the structure elements.

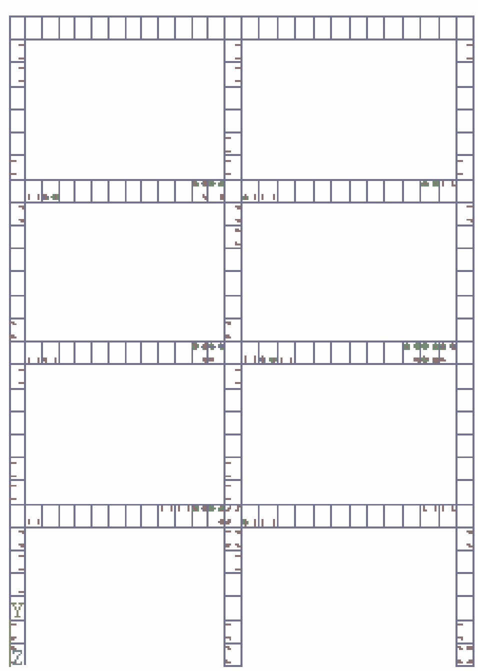


Fig: 5.9 Front View Showing cracking

5.6) Observations:

It can be observed from the Figure that:

a) Beams along x-axis shows cracking as the earthquake excitation is along x-axis.

b) Beams at Top Roof Level are least affected as compared to the beams at other Floor Levels.

c) Beams shows cracking pattern at their joints with Columns.

e) Columns at Base Level are severely affected as shown in above Figures.

f) Strengthening is required mainly at the Beam-Column joints, and at the Base of Columns at Ground Floor as these are severely affected.

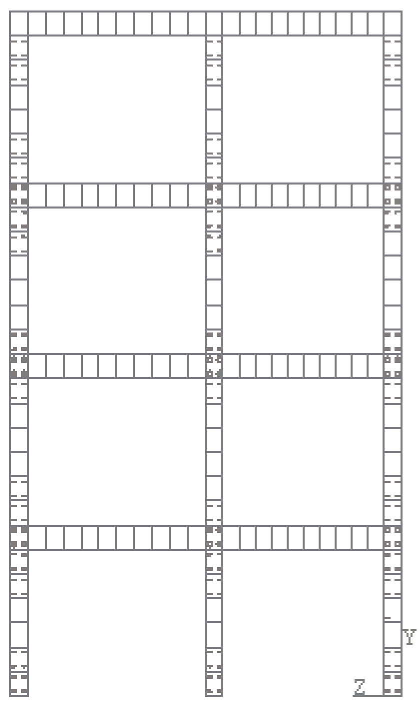


Fig: 5.10 Back View showing cracking

It can be seen that the Beams along z-axis are least affected, as the earthquake excitation is along x-axis. And the columns are mostly affected at their Base and at the joints.

5.7) Analysis of L-shape building designed as per I.S 1893:2002

In this part, the same L-shape building is designed as per I.S 1893:2002, and the analysis is performed by using ANSYS5.4.
Staad Pro 2003 is used for the seismic design of this building. Various parameters used for the design are:
The building is assumed to be situated in Zone IV (Z=0.24), Importance Factor (I=1), Response Reduction Factor (R=5), Depth below Ground Level = 1.5m,
Time Period is Calculated by the Formula, $T= 0.075\ h^{0.75}$ (h= height of building above G.L). T comes out to be 0.543sec.
Loading on Beams is taken as: **Table: 5.10**

Beams	Floor Level	Loading (KN/m)
5m	Roof	14
4m	Roof	10
5m	Typical	15
4m	Typical	11

Sections of Beams and Columns, Loading Calculations, Weight Calculations and Normalized Mode Shape Calculations have already been described in section 4.3.
Structural details (As per Analysis and Design on Staad.Pro) are as follows:

Table: 5.11

Element	Dimensions (m)	Steel	Ties
Columns	0.345 X 0.345	4-25mm ϕ	8mm @ 150mm c/c
Beams	0.345 X 0.5	4-20mm ϕ (positive steel at centre) 2-20mm ϕ (neg. steel at corners)	8mm @ 150mm c/c

The FEM model is based on the aforementioned structural components.

5.8) Description of the ANSYS Model

Value of Real Constants gets changed with the change in Reinforcement Details, rest all the procedure remains same.

5.8.1) Element Constants

Nine sets of material constants were used in the model, three for each type of elements. The number of material constant sets mainly reflects the number of element thickness in the model. Tables and list the element constant sets for the beam and columns, respectively.

Table: 5.12 Element constants for column (Set 1):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.003	0.003	0.012
Theta (orient.Angle)	0	0	90
Phi (orient.Angle)	90	0	0

Table: 5.13 Element constants for Beam along x-axis (Set 2):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.003	0.002	0.011
Theta (orient.Angle)	90	0	0
Phi (orient.Angle)	0	0	90

Table: 5.14 Element constants for column (Set 3):

Real Constants	Rebar 1	Rebar 2	Rebar 3
Volume Ratio	0.003	0.002	0.011
Theta (orient.Angle)	90	0	0
Phi (orient.Angle)	0	90	0

Same Modeling rules have been adopted. The following model is created as shown on next page:

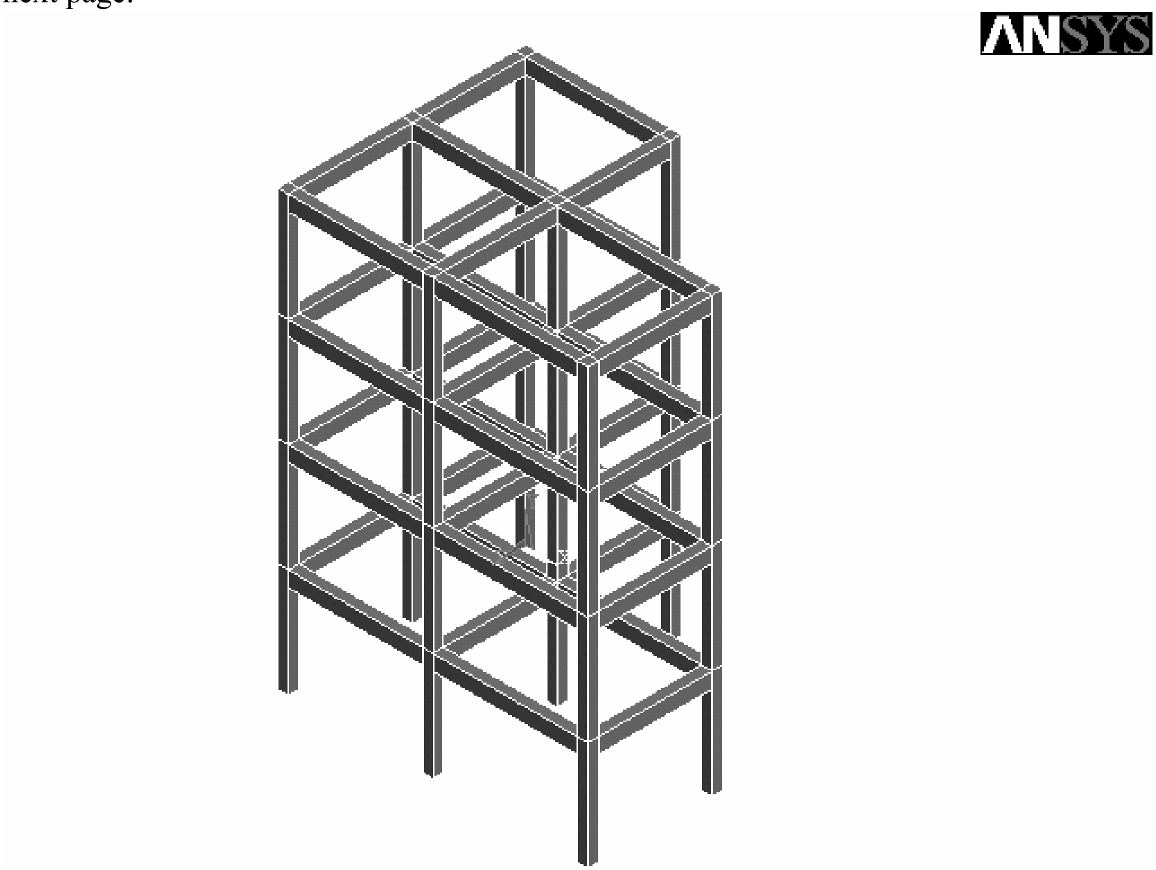


Fig: 5.11 Isometric View

5.8.2) Meshing:
Fine Mesh is created on this model obtained.

Size Controls:

All Column Elements Lines have been divided into 7 divisions, All the Beams along x-axis have been divided into 12 divisions, and the number of divisions for Beams along z-axis is 10. These divisions have been calculated by considering the fact that the mesh should be fine such that all the nodes at Beam-Column Joints should be connected. After Meshing, all the nodes have been merged with Range of Coincidence 0.001 in **Numbering Controls** Option.

Finite Mesh Model is shown below:

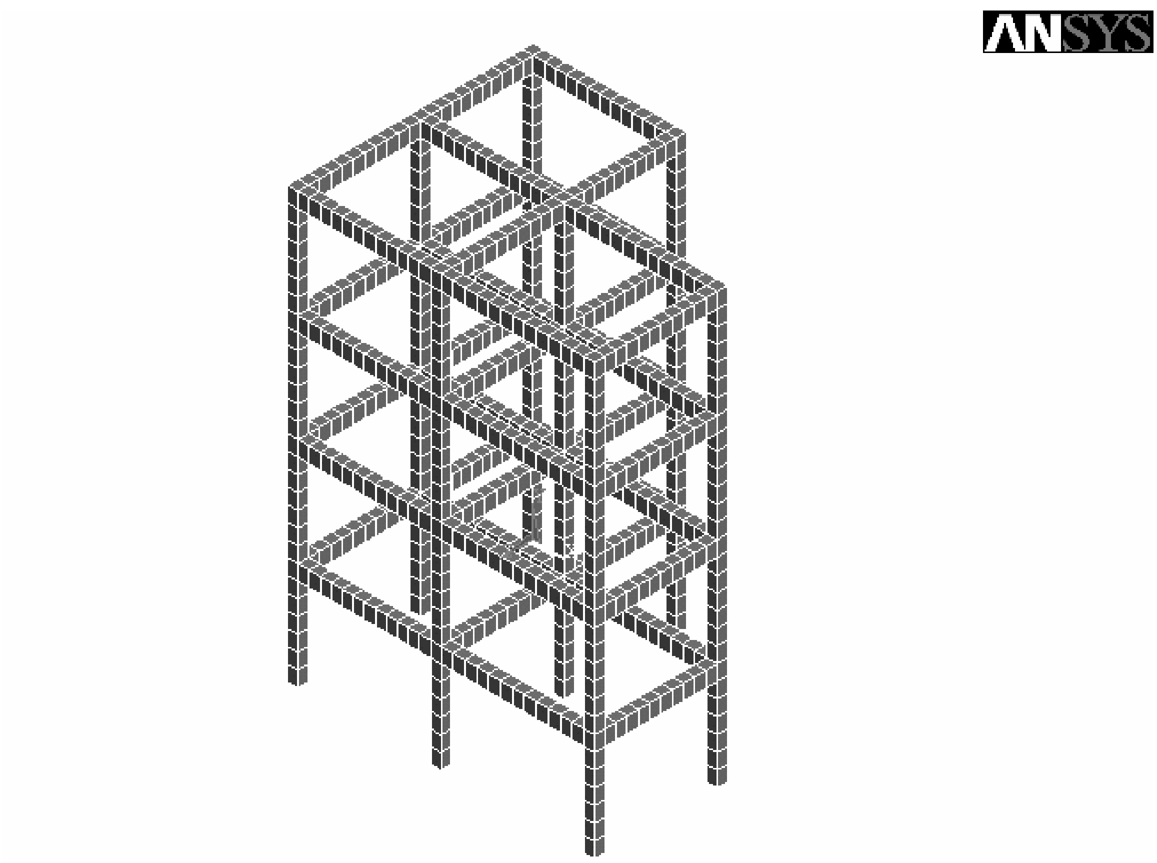


Fig: 5.12 Meshed Model

5.8.3) Boundary Conditions and Loading:

The boundary conditions for the analysis are: a) all the nodes of the columns at the foundation level are fixed, i.e., the displacements, translational or rotational, were set to zero.

Horizontal Displacement of 0.075 m is applied at top Column nodes in x-direction.

5.8.4) Analysis Procedure and Options:

Non-Linear Static Analysis is done on this frame. Various options for the analysis procedure have already been discussed in section 4.4.7:

The following results are obtained after analyzing the above-described model.

As the earthquake excitation is along x-axis, all the beams and columns are deflected in x-axis. The deformed shape of the model is as shown:

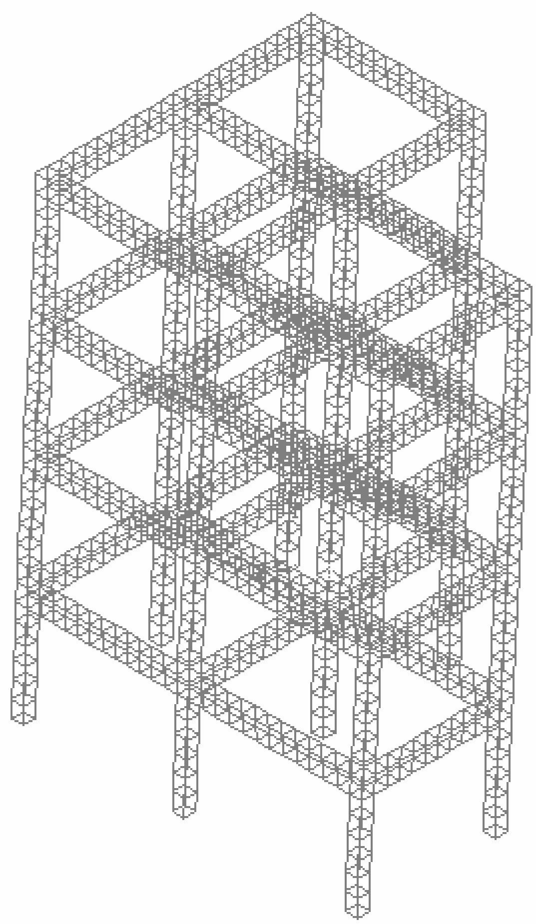


Fig: 5.13 Deformed Shape

5.9) Capacity Curve Plot:

As the Capacity Curve is plot between Base Shear and Roof Displacement, so the graph is plotted between the Base Reactions and Roof Displacements obtained from Analysis Results.

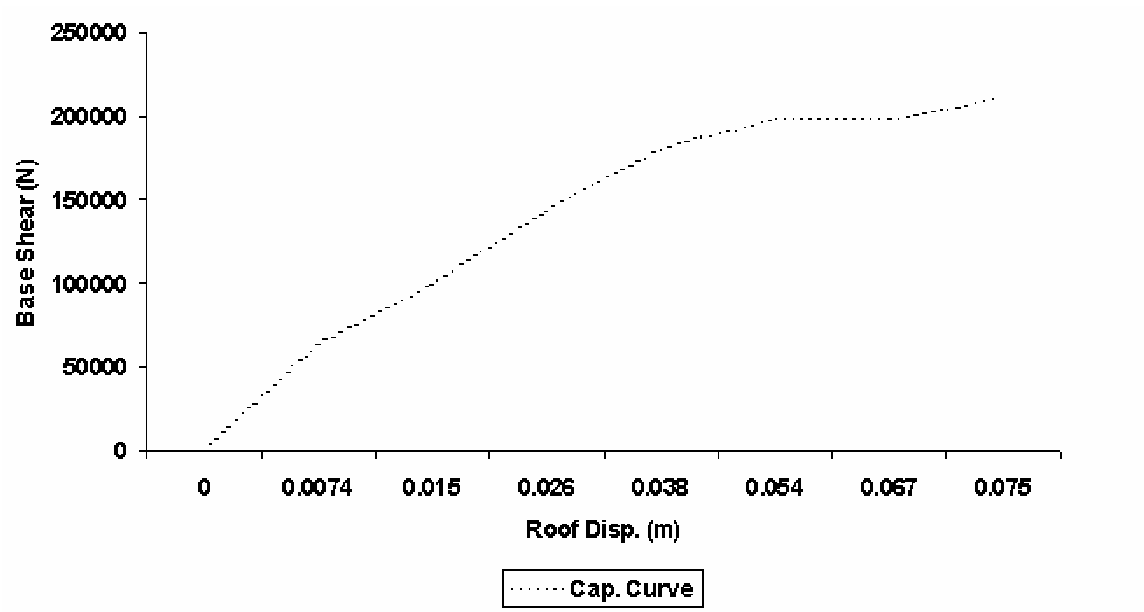


Fig: 5.14 Capacity Curve

5.9.1) Capacity Curve Conversion:

Next Step is to convert the Capacity Curve into Capacity Spectrum as described in Section-3.2

Table: 5.15 Conversion of Capacity Curve to Capacity Spectrum

Point	V (N)	δ_R (m)	V/W	$PF_{R1}= PF_1.\phi_{roof1}$	α_1	Sa (g)	Sd (m)
A	143441	0.026	0.087	1.27	0.085	1.02	0.020
B	181484	0.038	0.110	1.09	0.079	1.39	0.035
C	197996	0.067	0.120	1.05	0.083	1.44	0.064
D	209886	0.075	0.130	1.01	0.084	1.54	0.074

Capacity Spectrum which is a Plot between Sa and Sd is as shown:

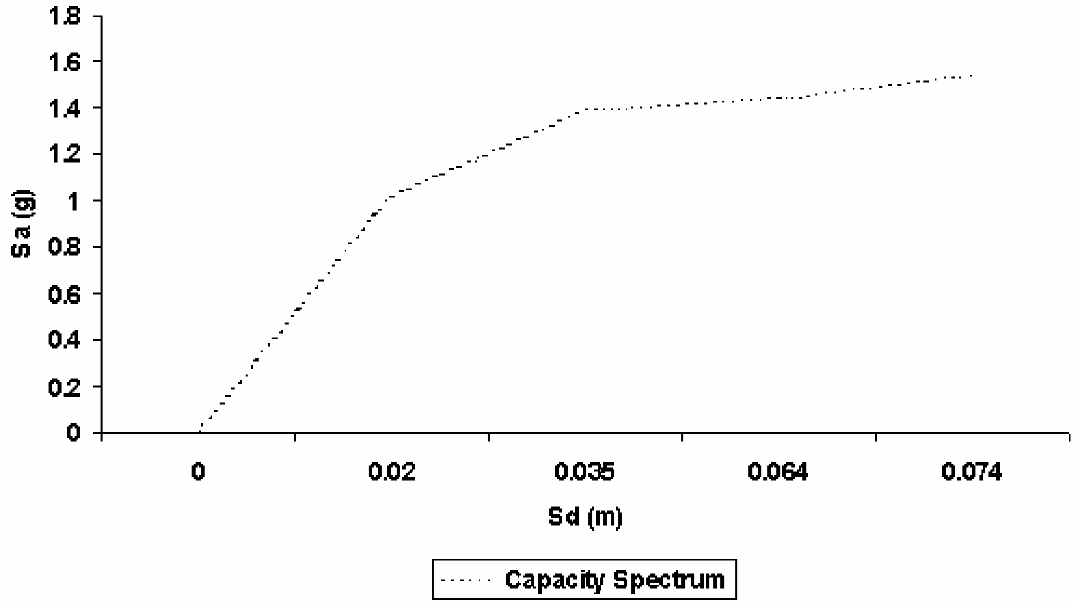


Fig: 5.15 Capacity Spectrum

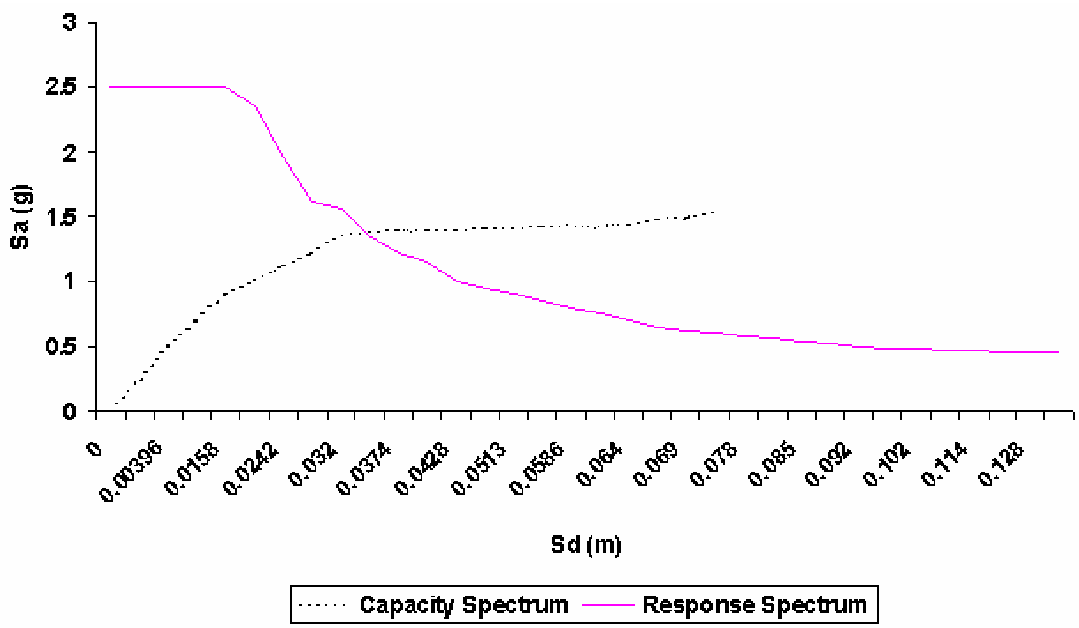


Fig: 5.16 Intersection of Capacity Spectrum and Response Spectrum

5.10) Following **Observations** are made from the above Graph:

a. Capacity Spectrum meets Response Spectrum at 0.0342m (= S_d), and the corresponding Base Shear V is 175480 N (Table: 5.15). Value of Base Shear as per IS 1893:2002 is 98953 N. Target Displacement is 38mm (Table: 5.15). It can be observed that the demand intersects the capacity envelope with sufficient strength and displacement reserves. Although inelastic response is evident during these base motions, collapse of the structure is not imminent for these levels of ground motion excitation.
b. Therefore, it can be concluded that the margin of safety against collapse for the building is sufficient at this intensity of shaking. Response Spectrum intersects Capacity Spectrum in in-elastic range. However, strengthening is required for some elements which can be clear by viewing the crack pattern obtained from ANSYS.

Now, we use the value of target displacement (38mm), to find out the cracking at that particular target displacement in the structure. In General Post Processor in ANSYS, the displacement value of 38mm is entered to plot cracking in the Structure. The following figure shows the cracking in the structure elements.

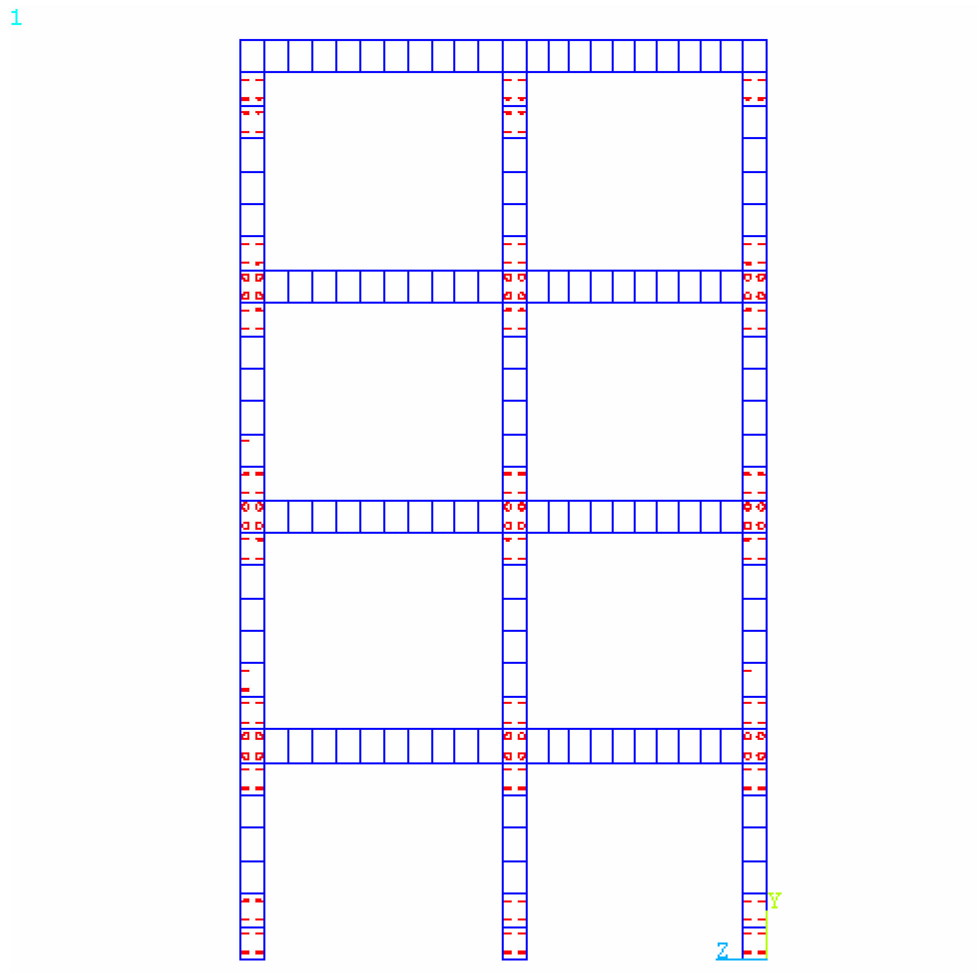


Fig: 5.17 Back View showing cracking

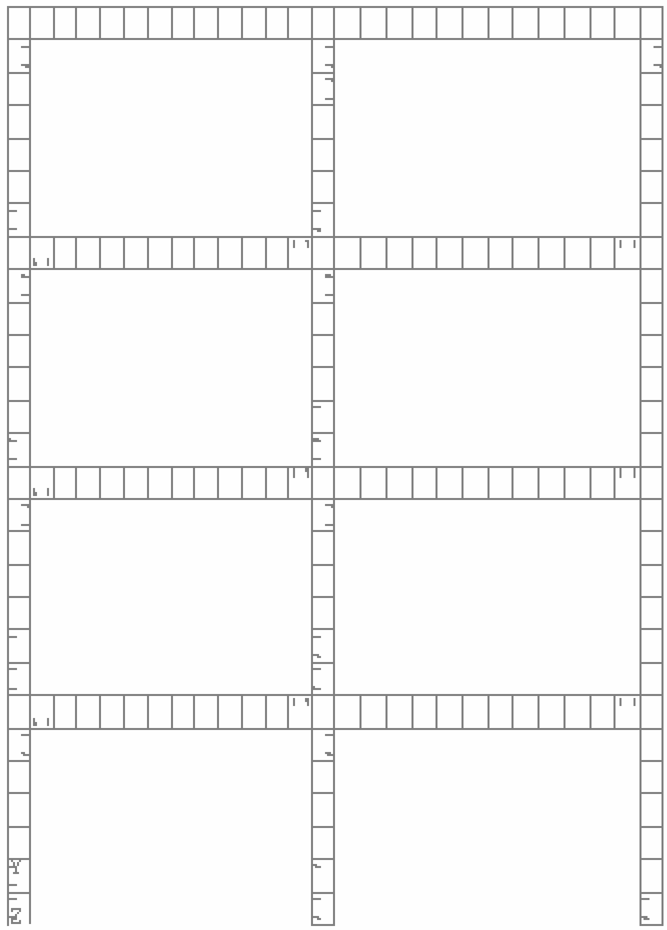


Fig: 5.18 Front View showing cracking

5.11) Observations:

It can be observed from the above Figures that:

a) Few Cracks have developed at the Base of Columns at Ground Level. As the load is applied on top left side, so the left side columns show cracking mainly at top and bottom.

b) Beams can be seen to be cracked at their connection with columns. This can be due to the fact that Ductile Detailing has not been incorporated.

c) Beams along z-axis have not been affected, as the earthquake excitation is along x-axis.

d) Strengthening is required at the Beam-Column joints.

e) However, the Cracking pattern is less severe than the previous case (that is, for the I.S 456:2000 Designed Building as discussed in part 1).

5.12) Comparison of above Analysis Results (Part 1 and Part 2) of L-shape Building:

S.No.	Parameter	Part 1 Bldg. (As per I.S 456:2000)	Part 2 Bldg. (As per I.S 1893:2002)
1.	Cracking Pattern	Shows severe cracking pattern	Beam-Column joints shows cracking.
2.	Columns at G.L (G+1) L (G+2) L (G+3) L	Severely affected Severely affected Severely affected Moderately affected	Moderately affected Moderately affected Moderately affected Least affected
3.	Beams along z-axis	Not affected (as excitation is along x-axis)	Not affected
4.	Beams along x-axis	Cracks near the adjoining area with Columns, except at Roof Lvl.	Cracking pattern is similar, but cracks are less severe.
5.	Beam-Column Joints	Severely affected	Moderately affected
6.	Strengthening Required	In Cracked Members	Beam-Column Joints

CHAPTER-6

CONCLUSION

In this report, Seismic Evaluation of R.C Buildings as per ATC-40 has been performed. Although, Seismic Evaluation can be done by various methods like elastic method (using DCRs) and In-elastic method (Pushover Analysis) as described in ATC-40 Manual. In this study, Pushover Analysis which is a Non-Linear Static Analysis is adopted to carry out evaluation. Finite Element software ANSYS 5.4 has been effectively utilized for getting the non-linear response of the structure. Capacity Curve, which is a Load-Deformation Plot (after exceeding the elastic limit) is obtained from ANSYS, which can be used further for getting the requirement of strengthening in members. Based on the study carried out, it can be concluded that:

1. ANSYS can be used as an effective tool for performing Pushover Analysis. It can be used to evaluate the seismic of both new and existing structural systems
2. If the Performance Point lies within the elastic stage, the building can said to be safe. And if Performance Point lies in in-elastic range, strengthening is required in the affected members, as can be obtained from ANSYS cracking pattern. Limiting Value of Base Shear can also be found out from the Demand and Capacity Envelopes.
3. Seismic Evaluation by Non-Linear Static Analysis exposes design weaknesses that may remain hidden in an elastic approach. Such weaknesses include excessive deformation demands, strength irregularities, and overloads on potentially brittle points, such as columns and connections.
4. The unsymmetrical Building studied shows that a lot of retrofitting is required if seismic effect is not taken into design considerations. However, in case of analysis of seismically designed building, strengthening is needed at Beam-Column Joints because ductile detailing has not been incorporated.

Scope for Further Study:

1. Ductile Detailing has not been incorporated in the present study. Further study may be undertaken, by providing reinforcement manually rather than by volume ratio in ANSYS.

2. The flexibility of Foundation is not considered in the present study. Further study may be undertaken considering soil-foundation interaction.
3. For Seismic Evaluation of Tall Buildings where higher mode effects are judged to be important, elastic and in-elastic dynamic analysis can be performed.

REFERENCES

1) ATC 40,"Seismic Evaluation and Retrofit of Concrete Buildings", Vol. 1 & 2, Applied Technology Council, Redwood City, CA, USA, Report No. SSC 96-01, Nov. 1996.
2) Bernal Dionisio," Instability of Buildings during Seismic Response", Engineering Structures, Vol. 20, Nos. 4-6, pp. 496-502, 1998.

3) Bhardwaj S.," Seismic Evaluation and Retrofitting of Multistoreyed RC Frame Building", M.E Dissertation in Earthquake Engg., IIT Roorkee, 2002.
4) Bracci J.M., Kunnath S.K., Reinhorn A.M.,"Seismic. Performance and Retrofit Evaluation of Reinforced Concrete Structures", Journal of Structural Engg., pp. 3-10, Jan 1997.
5) Chen P., Collins K.R., " Some observations on performance-based and reliability-based Seismic Design of a symmetrical building", Engineering Structures, Vol. 23, pp 1005-1010, 2001.
6) Dinh T.V., Ichinose T.,"Probabilistic Estimation of Seismic Story Drifts in Reinforced Concrete Buildings", pp 416-427, Journal of Structural Engineering, March 2005.
7) Dooley,L., and Bracci, J.M. (2001)."Seismic evaluation of column-to-beam strength ratios in reinforced concrete frames." ACI Struct. J., 98(6), pp 834-851.
8) Habibullah A., Pyle S.," Practical Three Dimensional Non-Linear Static Pushover Analysis", Published in Structure Magazine, Winter, 1998.
9) Ichinose, T., and Umeno, T. (2000)." Story shear safety factor to prevent story collapse in RC building." Proc., 12th World Conf. On Earthquake Engg.
10) Ichinose, T., Umemura, H., Kagohashi, H., and Dinh, T.V. (2002)." Shape index for seismic evaluation of RC Buildings". J. Struct. Construction Engg., 560(10), 155-160.
11) IS 1893 (Part 1): 2002," Criteria for Earthquake Resistant Design of Structures", Bureau of Indian Standards, New Delhi.
12) IS 13920: 1993, "Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces-Code of Practice", Bureau of Indian Standards, New Delhi.
13) IS 456: 2000, "Plain and Reinforced Concrete-Code of Practice", Bureau of Indian Standards, New Delhi.
14) Jan T.S., Liu M.W., Kao Y.C.,"An upper-bound pushover analysis procedure for estimating the seismic demands of high-rise buildings"., Engineering Structures 26(2004), pp 117-128.
15) Jain A.K.," Advanced Structural Analysis with Computer Applications".

16) Jor F.C., Duke L.M, Yu Z.," Finite Element Analyses of Seismic Response of Berkeley Town House"., Lawrence Berkeley National Laboratory, August 2002.
17) Krawinkler H., Seneviratna G.D.P.K.,"Pros and Cons of a pushover analysis of seismic performance evaluation", Engineering Structures, Vol. 20, Nos 4-6, pp 452-464, 1998.
18) Kunnath, S., K., Reinhorn, A., M., and Lobo, R. F.(1992)."IDARC version 3.0- a program for inelastic damage analysis of reinforced concrete structures." Tech. Rep. NCEER-92-0022, Nat. Ctr. For Earthquake Engg. Res., State Univ. of New York at Buffalo.
19) Kappos, A.,J., Manafpour, A." Seismic design of R/C buildings with the aid of advanced analytical techniques", Engineering Structures 23(2001), pp 319-332.
20) Li J.H., Su R.K.L., Chandler A.M.," Assessment of low rise building with transfer beam under seismic forces", Engineering Structures, Vol. 25, pp 1537-1549, 2003.
21) Makarios T.K.," Optimum definition of equivalent non-linear SDOF system in Pushover procedure of multistoreyed R/C Frames", Engineering Structures, Vol. 27, pp 814-825, 2005.
22) Mwafy A.M., Elhashai A.S.," Static Pushover versus Dynamic Collapse analysis of R.C Buildings", Engineering Structures, Vol. 23, pp 407-424, 2001.
23) Motlagh A.R, Saadeghvaziri M.A.," Non-Linear Seismic Response of stiffening SDOF Systems", Engineering Structures, Vol. 23, pp 1269-1280, 2001.
24) Munshi J.A., Ghosh S.K.," Analyses of Seismic performance of a code designed R.C Building", Engineering Structures, Vol. 20, Nos 7, pp 608-616, 1998.
25) Murty, C., V., R."Quantitative Approach to Seismic Strengthening of RC Frame Buildings", Seminar Proc. On Seismic Assessment and Retrofitting of Buildings, Mumbai, Feb., 2002, pp 19-27.
26) NHERP,"Hand Book for the Seismic Evaluation of Existing Buildings", Building Seismic Safety Council, Washington, D.C 1992.
27) Park, R., and Paulay, T.(1975). Reinforced concrete structures, Wiley, New York.
28) Paz Mario," Structural Dynamics Theory and Computation".

29) Tokoro K., Menum C.A.," Application of the Modal Pushover Procedure to estimate Non-Linear Response Envelopes", Research Summary 2003-04.