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Seismic Analysis on a Plan Irregular Multistorey Commercial Building Using Etabs

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ABSTRACT: The buildings are constructed mostly based on the usual standard codes considering the gravity loads consisting of the self weight of the structure and the live load. These structures are experiencing low magnitude loads in their design life that leads only to elastic response, however strong loads such as sudden earthquake will lead the structure beyond the elastic limit. The performance of reinforced concrete structures will be nonlinear under seismic loads, so the nonlinear behavior of reinforced buildings will be defined by the formation of plastic hinges and loss of considerable stiffness. In present study, push over analysis is carried out on a G+4 irregular RCC building situated in Kerala (zone lll) according to IS 1893:2002. The structure was modelled in ETABS 2018, Push over analysis and Time history analysis was performed in the same software. The push over curve, response spectrum curve, demand capacity, base reactions and performance point of building is found from the results obtained using ETABS.

Keywords: seismic, push over, demand capacity, time history

LIST OF ABBREVIATIONS

DBE	Design Basis Earthquake
MCE	Maximum Considered Earthquake
ASCE	American Society of Civil Engineers
UBC	Uniform Building Code
SEAONC	Structural Engineers Association of Northern California
SRSS	Square Root of the Sum of the Squares
CQC	Complete Quadratic Combination
MDOF	Multi Degree of Freedom
ADRS	Acceleration Displacement Response Spectrum
SDOF	Single Degree of Freedom
RCC	Reinforced Cement Concrete
ETABS	Extended Three Dimensional Analysis of Building System
SAP	Systems Applications and Products
RF	Response Reduction Factor
FEMA	Federal Emergency Management Agency
ATC	Applied Technical Council

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DCM	Displacement Coefficient Method
CSM	Capacity Spectrum Method
FNA	Fast Non linear Analysis
THA	Time History Analysis
PST	Pacific Standard Time

CHAPTER 1

INTRODUCTION

1.1 GENERAL

The sudden release of energy in the earth's crust creates seismic waves which arrive at various instance of time with different intensity levels are called as earthquake. It causes the random ground motion in all directions, radiating from epicentre, which causes structure to vibrate due to which induce inertia forces in them. Many existing structures are seismically deficient due to lack of awareness regarding seismic behaviour of structures. Due to this, there is urgent need to reverse this situation and do the seismic evaluation of existing and new structures.

1.2 SEISMIC ANALYSIS

Seismic analysis is a subset of structural analysis and is the calculation of the response of a building (or non building) structure to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent.

As seen in the figure, a building has the potential to wave back and forth during an earthquake (or even a severe wind storm). This is called the fundamental mode, and is the lowest frequency of building response. Most buildings, however, have higher modes of response, which are uniquely activated during earthquakes. The figure just shows the second mode. Nevertheless, the first and second modes tend to cause the most damage in most cases.

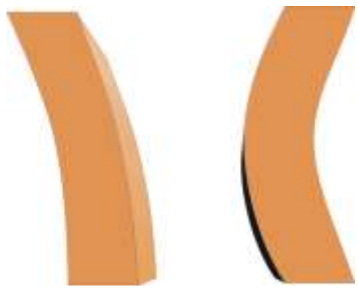

Fig.1.1 Fundamental mode

The earliest provisions for seismic resistance were the requirement to design for a lateral force equal to a proportion of the building weight (applied at each floor level). This approach was adopted in the appendix of the 1927 Uniform Building Code (UBC), which was used on the west coast of the United States. It later became clear that the dynamic properties of the structure affected the loads generated during an earthquake. In the Los Angeles Country Building Code of 1943 a provision to vary the load based on the number of floor levels was adopted (based on research carried out at Caltech in collaboration with Stanford University and the U.S. Coast and Geodetic Survey, which started in 1937). The concept of "response spectra" was developed in the 1930s, but it wasn't until 1952 that a joint committee of the San Francisco Section of the ASCE and the Structural Engineers Association of Northern California (SEAONC) proposed using the building period (the inverse of the frequency) to determine lateral forces.

Earthquake engineering has developed a lot since the early days, and some of the more complex designs now use special earthquake protective elements either just in the foundation (base isolation) or distributed throughout the structure. Analyzing these types of structures requires specialized explicit finite element computer code, which divides time into very small slices and models the actual physics, much like common video games often have physics engines. Very large and complex buildings can be modeled in this way (such as the Osaka International Convention Center).

Structural analysis methods can be divided into the following five categories.

1.2.1 Equivalent static analysis

This approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves. The response is read from a design response spectrum, given the natural frequency of the building (either calculated or defined by the building code). The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to yielding of the structure, many codes apply modification factors that reduce the design forces (e.g. force reduction factors).

1.2.2 Response spectrum analysis

This approach permits the multiple modes of response of a building to be taken into account (in the frequency domain). This is required in many building codes for all except very simple or very complex structures. The response of a structure can be defined as a combination of many special shapes (modes) that in a vibrating string correspond to the harmonics. Computer analysis can be used to determine these modes for a structure. For each mode, a response is read from the design spectrum, based on the modal frequency and the modal mass, and they are then combined to provide an estimate of the total response of the structure. In this we have to calculate the magnitude of forces in all directions i.e. X, Y & Z and then see the effects on the building. Combination methods include the following:

- absolute – peak values are added together
- square root of the sum of the squares (SRSS)
- complete quadratic combination (CQC) – a method that is an improvement on SRSS for closely spaced modes

The result of a response spectrum analysis using the response spectrum from a ground motion is typically different from that which would be calculated directly from a linear dynamic analysis using that ground motion directly, since phase information is lost in the process of generating the response spectrum.

In cases where structures are either too irregular, too tall or of significance to a community in disaster response, the response spectrum approach is no longer appropriate, and more complex analysis is often required, such as non-linear static analysis or dynamic analysis.

1.2.3 Linear dynamic analysis

Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Therefore, for tall buildings, buildings with torsional irregularities, or non-orthogonal systems, a dynamic procedure is required. In the linear dynamic procedure, the building is modelled as a multi-degree-of-freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix.

The seismic input is modelled using either modal spectral analysis or time history analysis but in both cases, the corresponding internal forces and displacements are determined using linear elastic analysis. The advantage of these linear dynamic procedures with respect to linear static procedures is that higher modes can be considered. However, they are based on linear elastic response and hence the applicability decreases with increasing nonlinear behaviour, which is approximated by global force reduction factors.

In linear dynamic analysis, the response of the structure to ground motion is calculated in the time domain, and all phase information is therefore maintained. Only linear properties are assumed. The analytical method can use modal decomposition as a means of reducing the degrees of freedom in the analysis.

1.2.4 Non linear dynamic analysis

Nonlinear dynamic analysis utilizes the combination of ground motion records with a detailed structural model, therefore is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the

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model and the modal responses are combined using schemes such as the square-root-sum-of-squares.

In non-linear dynamic analysis, the non-linear properties of the structure are considered as part of a time domain analysis. This approach is the most rigorous, and is required by some building codes for buildings of unusual configuration or of special importance. However, the calculated response can be very sensitive to the characteristics of the individual ground motion used as seismic input; therefore, several analyses are required using different ground motion records to achieve a reliable estimation of the probabilistic distribution of structural response. Since the properties of the seismic response depend on the intensity, or severity, of the seismic shaking, a comprehensive assessment calls for numerous nonlinear dynamic analyses at various levels of intensity to represent different possible earthquake scenarios. This has led to the emergence of methods like the incremental dynamic analysis.

1.2.5 Non linear static analysis

In general, linear procedures are applicable when the structure is expected to remain nearly elastic for the level of ground motion or when the design results in nearly uniform distribution of nonlinear response throughout the structure. As the performance objective of the structure implies greater inelastic demands, the uncertainty with linear procedures increases to a point that requires a high level of conservatism in demand assumptions and acceptability criteria to avoid unintended performance. Therefore, procedures incorporating inelastic analysis can reduce the uncertainty and conservatism.

This approach is also known as pushover analysis. A pattern of forces is applied to a structural model that includes non-linear properties (such as steel yield), and the total force is plotted against a reference displacement to define a capacity curve. This can then be combined with a demand curve (typically in the form of an acceleration-displacement response spectrum (ADRS)). This essentially reduces the problem to a single degree of freedom (SDOF) system.

Nonlinear static procedures use equivalent SDOF structural models and represent seismic ground motion with response spectra. Storey drifts and component actions are related subsequently to the global demand parameter by the pushover or capacity curves that are the basis of the non-linear static procedures.

1.3 SCOPE OF THE PRESENT STUDY

The scope of the present study is to familiarize with the designing and analysis software ETABS and to validate and check the possibility of implementing ETABS software. Basically seismic analysis is done to ensure life safety under Design Basis Earthquake (DBE) and collapse prevention under Maximum Considered Earthquake (MCE).

CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

Pushover analysis is a popular performance based design method, so there are many studies conducted using this method. Time history analysis is a non linear dynamic analysis to obtain the dynamic response of the structure subjected to seismic loading. Most of these studies assumed that the lateral force distribution was an inverted triangular distribution, according to recommendation of codes only flexural plastic hinges were considered. It was also studied that mode shapes and the lateral distribution of base shear gives the same results. The following are some studies in brief:

Chatali Patel, Payal Patel, Grishma Thaker (2018) Pushover analysis of High rise RCC buildings with vertical irregularities: The seismic behavior of multi-storied building frame during an earthquake motion depends upon the distribution of strength, mass and stiffness in both horizontal and vertical planes. Pushover analysis is one of the analysis method which is adopted for the present study. Present study aims towards doing Nonlinear Static Pushover Analysis of G +20 high rise RCC residential building. This work shows that the comparison seismic performance and behavior of building frame with and without vertical irregularity in terms of parameter like storey shear, storey displacement, and storey drift. Also comparison of seismic response of the structure in terms of base shear and displacement along with the location of the plastic hinges at the performance point of all the models are considered. All building frames are analyzed by using design and analysis software ETABS and design as per IS 456:2000 and IS 1893:2002.

P.Lestuzzi, H. Harif, (2018), Nonlinear Time-History Analysis for Validation of the Displacement-Based Seismic Assessment of the RC Upper Bridge of a Dam: The paper focuses on the nonlinear time-history analyses which were achieved in order to check the accuracy of the results obtained using the displacement-based method. The structural characteristics of the reinforced concrete upper bridge are similar to those of conventional bridges. However, the piers were built with very little reinforcement and consequently they will exhibit a rocking behavior in case of earthquake loading. Rocking is rather a favorable failure mechanism and is related to a certain amount of displacement capacity. However, this behavior is not linked to plastic energy dissipation which may significantly increase the related displacement demand. In order to determine the real displacement demand, nonlinear time-history analyses were achieved with SDOF systems defined by an “S” shape hysteretic model. Spectrum compatible stationary synthetic accelerograms and slightly modified recorded earthquakes were both used for acceleration time-histories. The results showed that the displacement demand corresponds well with the one determined by usual push-over analysis. The results show a very favorable seismic situation, related to a relatively stiff structure associated to rock soil conditions with an A class soil. The seismic safety of the upper bridge is already satisfactory for the current state (without retrofitting). Consequently, the proposed costly reinforcement for the upper bridge could be significantly reduced.

S.M Patil, Y.M Pudale, V.V Nair (2018) Study of pushover analysis of vertical irregular structures: The seismic performance of building frame changes with the variation or the discontinuity in stiffness, strength and mass of the building. This causes the irregularity of the building. So that, pushover analysis is one of the method to study the seismic behavior of vertical irregular structure when the structure is subjected to earthquake forces. The vertical irregularity that is irregularity in elevation is considered for present study. Five G+7 RCC building frames having different percentage of irregularity are considered for the present study and it is designed and analyzed by using design and analysis software ETABS v9.5.0.All the building frames are designed as per the IS 456:2000 and IS1893:2002.The purpose of this concerned work is to compare the pushover result obtained in terms of parameter storey drift, storey displacement, storey shear, Base shear, spectral displacement and spectral acceleration of different vertical irregular structure and to study the effect of increase in vertical irregularity.

Namani Saikiran, T.Parimala (2017) Study of irregular RC frame buildings under seismic: Buildings may be considered as asymmetric in plan or in elevation based on the distribution of mass and stiffness along each storey, throughout the height of the buildings. Most of the hilly regions of India are highly seismic. To study the effect of varying height of columns in ground storey due to sloping ground, the plan layout is kept similar for both buildings on plane and sloping ground. The models have been conducted and analyzed in the ETABS pro program by using equivalent linear static method and response spectrum method for comparing and investigating the changes in structural behavior and the irregularity effect in plan and elevation on sloping ground. The result of the analysis for displacement and storey drift have been studied and compared with reference to the serviceability and the time period, storey shear, storey moment and storey torsion, have been studied and compared for different configurations structure models and it was presenting in graphical and tabular form.

Anju Nayas, Minu Antony (2017) Push over analysis of plan irregular RC buildings with special columns : Irregular buildings constitute a major portion of the modern urban infrastructure. The group of people involved in constructing the building facilities, including owner, architect, structural engineer, contractor and local authorities, come up with the overall planning, selection of structural system, and its configuration. This may lead to building structures with irregularities in their mass, stiffness and strength along the height of building. The objective of this study is to carry out nonlinear static analysis of irregular RC frame using special shaped columns with plan irregularity. This study also finds out which plan irregular building is the most effective in resisting lateral loads. The software used for modelling and analysis is ETABS 2015.

A.M Mwafy, S.Khalifa (2017) Impacts of vertical irregularity of seismic design of high rise buildings: Many tall buildings are practically irregular, as a perfect regular high-rise building rarely exists. The structural irregularity increases the uncertainty related to the capacity of the building to meet the design objectives. There is a pressing need to systematically assess the impacts of vertical irregularity on the seismic design of tall buildings, particularly the extreme irregularity types. This study is thus devoted to evaluate the seismic design coefficients of the modern tall buildings with different vertical irregularity features. The comprehensive results obtained from a large number of inelastic pushover and incremental dynamic analyses provide insights into the local and global seismic response of the reference structures and confirm the unsatisfactory response of tall buildings with severe vertical irregularities. The study also concluded that although the design coefficients of regular tall

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structures and buildings with insignificant irregularities are adequately conservative, they can be revised to arrive at a more cost-effective design of tall buildings.

A.S Patil, P.D Kumbhar (2013) Time History analysis of multi storied building under different seismic intensities : In the present paper study of nonlinear dynamic analysis of Ten storied RCC building considering different seismic intensities is carried out and seismic responses of such building are studied. The building under consideration is modeled with the help of SAP2000-15 software. Five different time histories have been used considering seismic intensities V, VI, VII, VIII, IX and X on Modified Mercalli's Intensity scale (MMI) for establishment of relationship between seismic intensities and seismic responses. The results of the study shows similar variations pattern in Seismic responses such as base shear and storey displacements with intensities V to X. From the study it is recommended that analysis of multistoried RCC building using Time History method becomes necessary to ensure safety against earthquake force.

2.2 SUMMARY

The seismic behavior of multi-storied building frame during an earthquake motion depends upon the distribution of strength, mass and stiffness in both horizontal and vertical planes. All models are analyzed by using design and analysis software ETABS or SAP and designed as per IS 456:2000 and IS 1893:2002. Push over analysis is a non linear static analysis had been used to obtain the inelastic deformation capability of frame. Only non-linear dynamic analysis is more accurate than pushover analysis; where non-linear dynamic analysis is time taking to perform. In order to obtain dynamic response of the structure, Time history analysis is carried out. So we can conclude that pushover analysis is the appropriate method to use for performance based design to get the response of the structures.

CHAPTER 3

OBJECTIVES

The objective of the present study is

- To model a G+4 storey building in ETABS as per IS-1893
- To perform pushover analysis to get the seismic response of the structure.
- To check whether the building designed meet the demand capacity obtained from the software
- To perform time history analysis to obtain dynamic response of the structure

CHAPTER 4

METHODOLOGY

- Studying the literature reviews for understanding the concept
- Choosing the software and its validation.
- Identifying the building plan and material properties.
- Modelling the plan in ETABS.
- Analysis of the building using non linear static pushover
- Analysis of the building using non linear dynamic time history
- Design and analysis of building by using ETABS software.
- Observation of results and discussions.

CHAPTER 5

VALIDATION

5.1 GENERAL

The validation object enables to evaluate the quality of mapping across source and target meshes. It provides quantitative measures that help in identifying regions on the target where the mapping failed to provide an accurate estimate of the source data.

5.2 ANALYTICAL METHOD

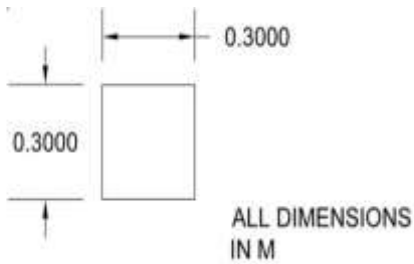

Fig.5.1 Dimensions of beam

By analytical method the total deflection of a fixed beam is given by the equation

Deflection = $wl^4/384$ EI

Eqn (1)

Table.5.1 Dimensions and specifications of beam section

Length l	4000 mm
Breadth b	300 mm
Depth d	300 mm

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Modulus of elasticity E	15811.388N/mm^2
Moment of Inertia I	675x10^6 mm^4

From the formula, value of deflection is = 0.210 mm

5.3 SOFTWARE METHOD
In order to analyze the total deflection, a model is designed with same dimensions.

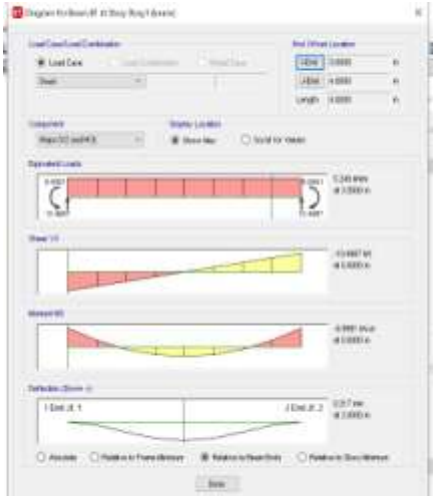


Fig.5.2 Deflection from software

The total deflection on the model as given by the software is 0.217 mm

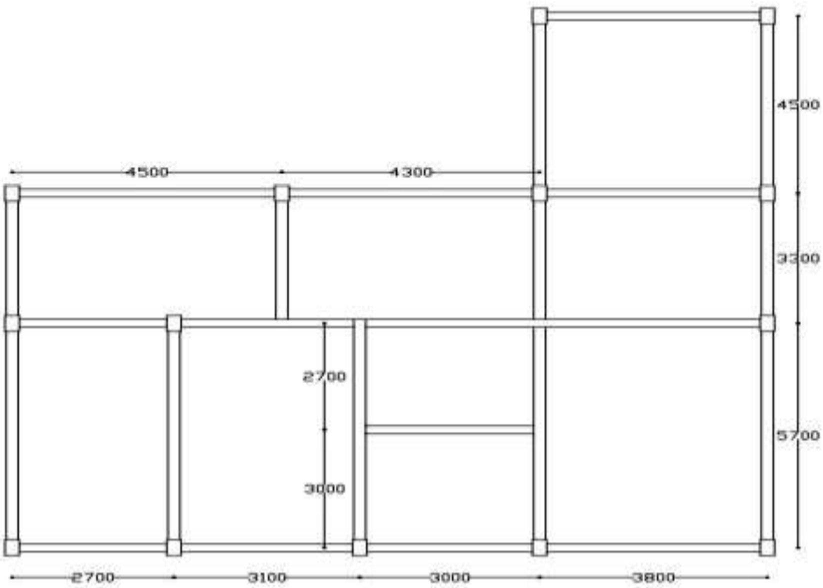
5.4 VALIDATION SUMMARY
Table 3.2 shows the result obtained from numerical and analytical method. Comparing manual and software results, almost same results were obtained but software value 3.22% higher than analytical result.

Table.5.2 Comparison of results

Result obtained analytically	Result obtained from software	Variation in %
0.210 mm	0.217 mm	3.22%

As per the literature of ETABS, the maximum percentage variation is limited to 5%. Here the variation obtained is 3.22% which is within the permissible limit. So the software was validated.

CHAPTER 6
BUILDING DETAILS
6.1 PLAN DETAILS
The building is a 5 storied structure for commercial purpose. It is defined as G+4 building having a p-storey and identical 5 stories. The first floor to fifth floor are typical floors .It also consist of a stair cum machine room and a lift.
The building as a whole, covers an area of 652.5 m^2. The typical floor area is 130.5 m^2.



(All dimensions in mm)

Fig.6.1 Plan of the building

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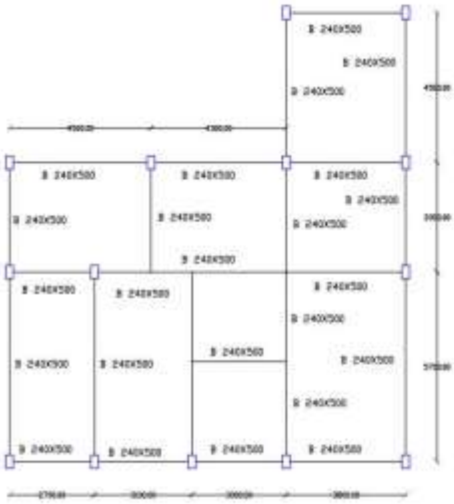
6.2 STRUCTURAL DETAILS

6.2.1 Beam layout

Beams of size 240 x 500 mm were provided and plinth beam of size 300 x 450mm were provided on the p-storey i.e below first floor. All beams were notated as B 240 x 500 in the layout.



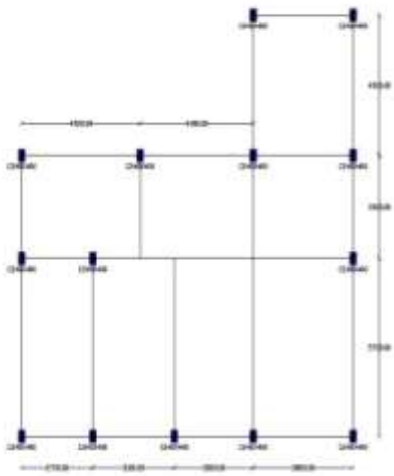


Fig.6.2 Beam layout
Fig.6.3 Column layout

6.2.2 Column layout

There are 14 no.s of columns in each floor, with a dimension of 240 x 400mm. All columns were notated as C 240 x 400 in the layout.

6.2.3 Material properties

The materials used in the designing were M25 grade concrete for beams and slabs, M30 grade concrete for columns, and Fe 415 grade steel reinforcements.

Table.6.1 Material properties

Parameters	Concrete	Steel
Young's Modulus (E)	21718500 kN/m^2	2×10^3 kN/m^2
Poison's Ratio (nu)	0.17	0.3
Density	23.5616 kN/m^2	76.8195 kN/m^2

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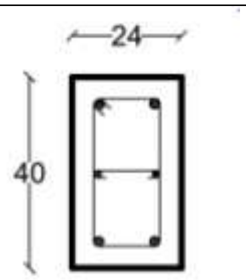
Critical Damping Ratio	0.05	0.03

Table.6.2 Column details

Column	C
Column details	
Column size	24 x 40 cm
Main steel	4 No.s 16 # + 2 No.s 12 #.
Ties details	8# @ 18 cm @ H/4 from top and bottom and 8# @ 20 cm @ midspan

6.2.4 Foundation details

Table.6.3 Foundation details

Footing	F1
Concrete mix	M 30
Steel	Fe 415
Size of P.C.C	140 x 160 x 15 cm
Size of R.C.C	120 x 140 x 20 cm
Reinforcement details	12 # @ 15 cm c/c both directions

CHAPTER 7

BUILDING MODELLING

The complete structure is framed structure. Thus, to analyse the columns and beams, software used is ETABS.

7.1 STEPS IN MODELLING

7.1.1 Storey and grid data

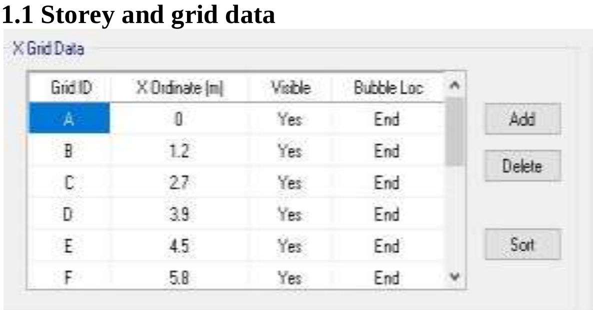
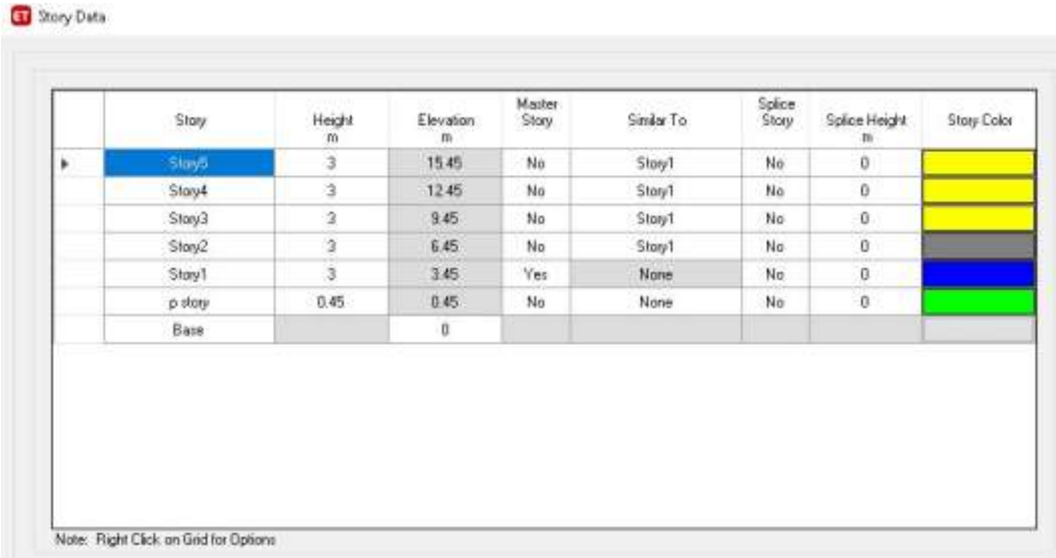


Fig.7.1 X grid data

Fig.7.2 Y grid data



Story Data

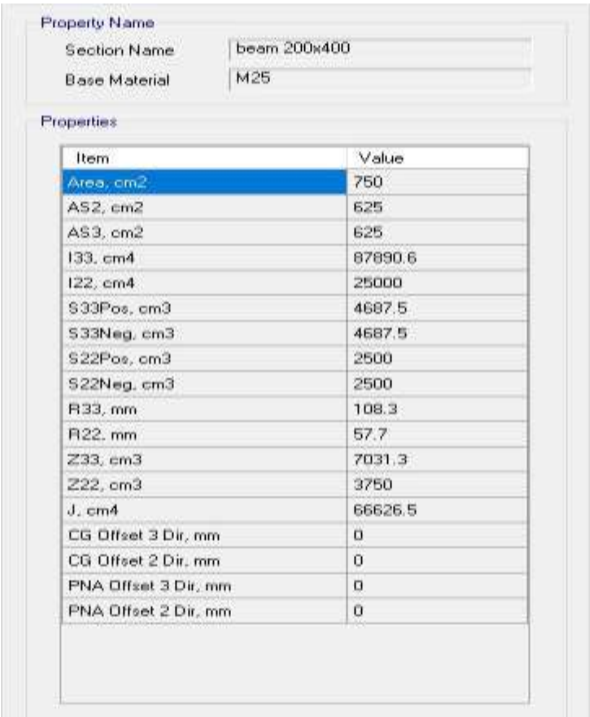
	Story	Height m	Elevation m	Master Story	Similar To	Splice Story	Splice Height m	Story Color
▸	Story5	3	15.45	No	Story1	No	0	
	Story4	3	12.45	No	Story1	No	0	
	Story3	3	9.45	No	Story1	No	0	
	Story2	3	6.45	No	Story1	No	0	
	Story1	3	3.45	Yes	None	No	0	
	p story	0.45	0.45	No	None	No	0	
	Base		0					

Note: Right Click on Grid for Options

Fig.7.3 Storey data

7.1.2 Defining and assigning beams

- Beam size 240 x 500 mm
- Concrete M25
- Property modifiers
- Torsional constant: 0.01
- I about both axes: 0.35



Property Name

Section Name: beam 200x400

Base Material: M25

Properties

Item	Value
Area, cm2	750
AS2, cm2	625
AS3, cm2	625
I33, cm4	87890.6
I22, cm4	25000
S33Pos, cm3	4687.5
S33Neg, cm3	4687.5
S22Pos, cm3	2500
S22Neg, cm3	2500
R33, mm	108.3
R22, mm	57.7
Z33, cm3	7031.3
Z22, cm3	3750
J, cm4	66626.5
CG Offset 3 Dir, mm	0
CG Offset 2 Dir, mm	0
PNA Offset 3 Dir, mm	0
PNA Offset 2 Dir, mm	0

Fig.7.4 Beam properties

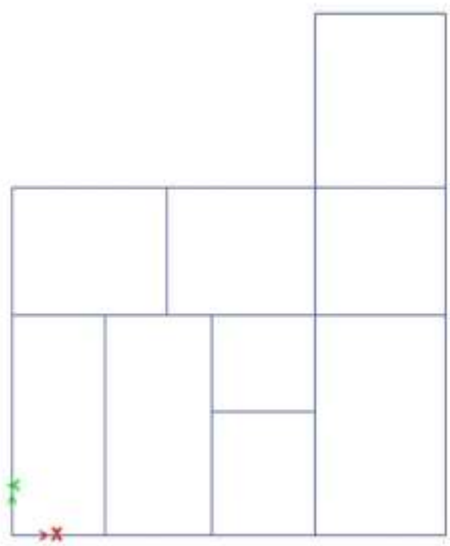


Fig.7.5 Assigned beams

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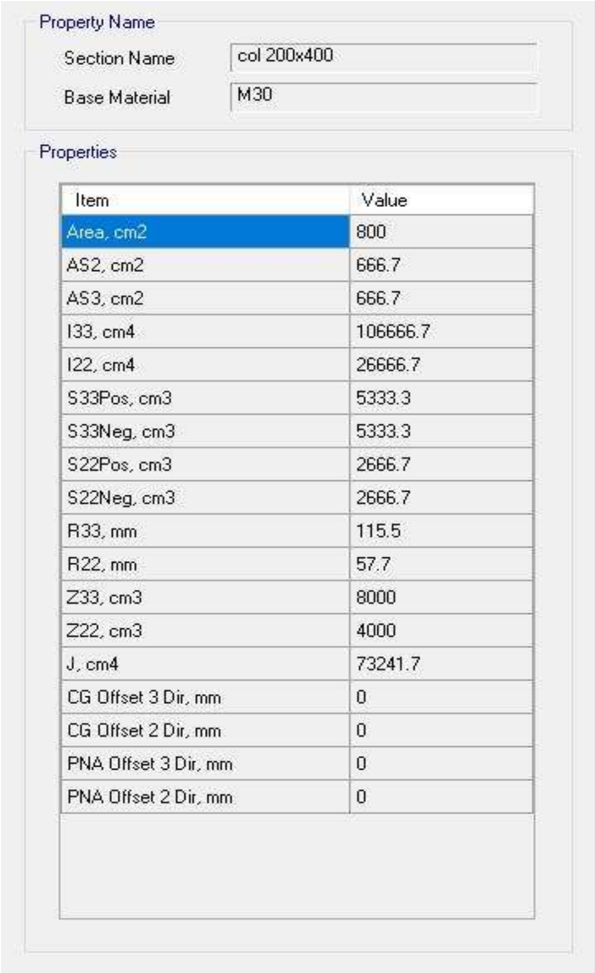
7.1.3 Defining and assigning columns

- Column size 240 x 400 mm
- Concrete M30
- Property modifiers
- Torsional constant: 0.01
- I about both axes: 0.7


Property Name

Section Name: col 200x400

Base Material: M30

Properties

Item	Value
Area, cm2	800
AS2, cm2	666.7
AS3, cm2	666.7
I33, cm4	106666.7
I22, cm4	26666.7
S33Pos, cm3	5333.3
S33Neg, cm3	5333.3
S22Pos, cm3	2666.7
S22Neg, cm3	2666.7
R33, mm	115.5
R22, mm	57.7
Z33, cm3	8000
Z22, cm3	4000
J, cm4	73241.7
CG Offset 3 Dir, mm	0
CG Offset 2 Dir, mm	0
PNA Offset 3 Dir, mm	0
PNA Offset 2 Dir, mm	0

Fig.7.6 Column properties

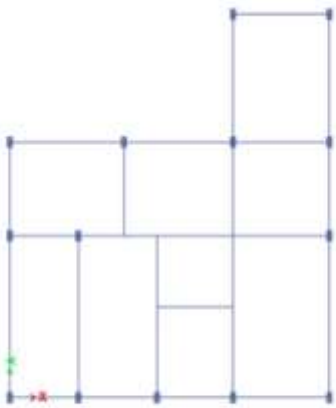


Fig.7.7 Assigned columns

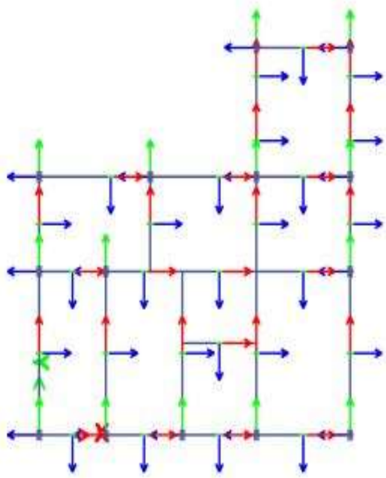


Fig.7.8 Orientation of columns

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7.1.4 Defining and assigning slabs

Table.7.1 Slab details

Floor slab	Thickness 140 mm	Modelling type - membrane
Stair slab	Thickness 200 mm One way distribution	Modelling type – membrane
Concrete	M25	

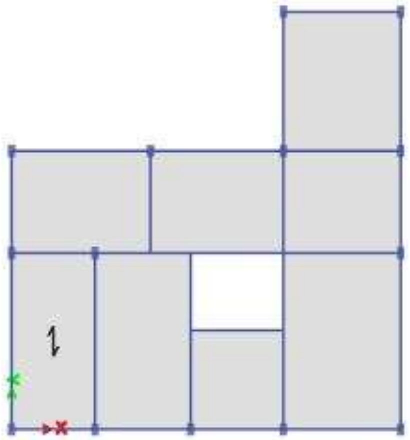

Fig.7.9 Assigned slabs

7.1.5 Defining retaining wall

- Modelling type – membrane
- Thickness – 200 mm
- Concrete M30

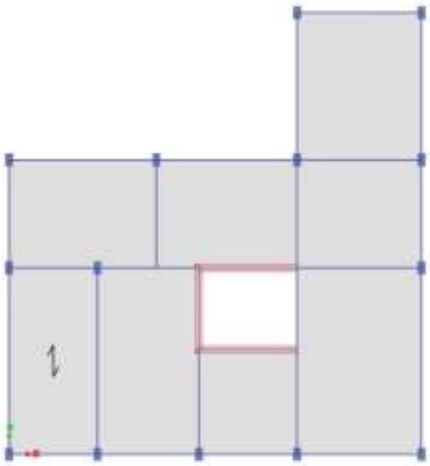

Fig.7.10 Shear wall with beams

Fig.7.11 Shear wall without beams

With the above steps, the modelling in ETABS has been completed.

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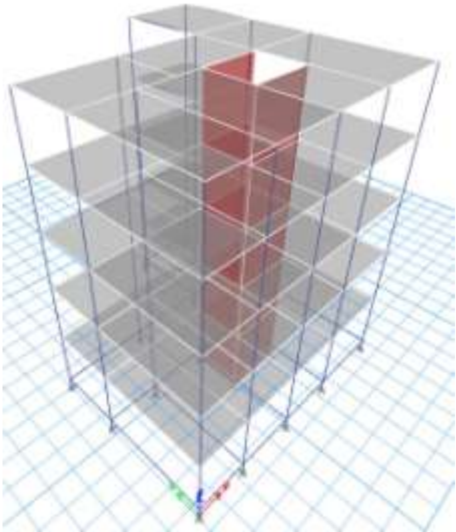

Fig.7.12 3-D view of structure

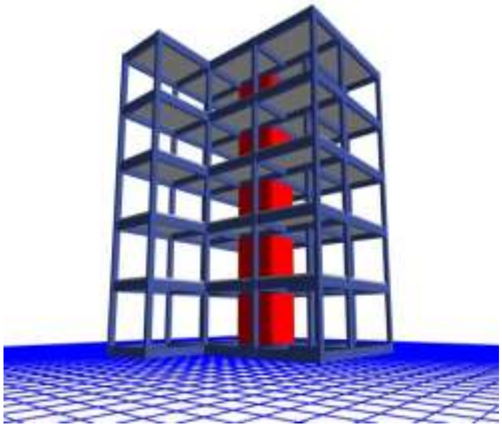

Fig.7.13 3-D rendered view

CHAPTER 8

LOAD CALCULATIONS

As per IS 1893:2002 the following seismic parameters were used to calculate the seismic forces.

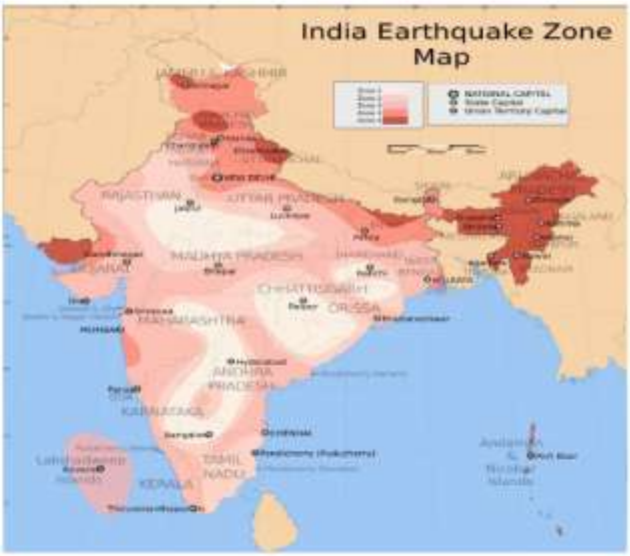

Fig.8.1 The India earthquake zone map

Zone Factor (Kerala) Z= 0.16 (Zone III)
Importance Factor I= 1.0
Response reduction factor (RF) = 3 (Ordinary RC Moment Resisting Frame)
Soil type = Medium Soil

$$V_B = \frac{ZI}{2R}\frac{Sa}{g} x W$$

Eqn (2)
V_B=Base shear
Z=Zone factor, for Maximum Considered Earthquake (MCE)
Z/2 is used to reduce the MCE to Design Basis Earthquake (DBE)
I=The Importance Factor depending upon the functional use of structures characterized by hazardous consequences of its failure
R=Response reduction factor depending on the perceived seismic damage performance of the structure
Sa/g is the average response acceleration coefficient

Table.8.1 Section dimensions and other details

Beam size	240x500mm
Column size	240x450mm
Slab thickness –Floor -Stair	140mm 200 mm
Concrete Grade	M25, M30
Shear wall thickness	120mm
Brick masonry unit weight	20kN/m
Unit weight of RCC	25 kN/m^3
Live Load	3 kN/m^2

8.1 SEISMIC LOAD

The structure is analysed and designed for live load, seismic load as per IS-1893-2002 and dead load consisting of self-weight of beams, columns and slabs.

Total floor area = 113.4 m^2

D.L of beams = 25 x 0.24 x 0.5 x 95300 = 2860 kN

D.L of columns = 25 x 0.24 x 0.4 x 3 x 14 = 100.8 kN

D.L of slabs = 25 x (0.14 x 89.91 + 0.2 x 15.39) = 410.8725 kN

D.L of wall = 20 x 0.12 x 3 x 8.7 =62.64 kN

Total D.L = 3434.1 KN

Upto 3kN, L.L is taken as 25% of L.L [IS 1893(Part 1):2002, clause 7.3.1]

Total L.L = 0.25 x 3 x 105.3 = 78.975 kN/m^2

Total load = 3513.28 kN

Total factored load = 5300 kN

Weight of floors 1,2,3 and 4 is 5300KN

Weight of 5^{th} floor = 2860+410.8725+0.5(100.8+62.64) =1.5 x 3352.59 = 5030 kN

Total seismic load = W=5300 x 4 + 5030 =26230 kN

8.2 SEISMIC BASE SHEAR

The design of base shear is the sum of lateral forces applied at all levels that are finally transferred to the ground.

$$V_B = Ah \times W \qquad \text{Eqn (3)}$$

$$Ah = (ZI/2R)\ (Sa/\ g) \qquad \text{Eqn (4)}$$

The fundamental natural period for buildings are in IS 1893(part 1) 2002 Class 7.6.

$$Ta = 0.09h/\sqrt{d} \qquad \text{Eqn (5)}$$

Z = 0.16

I = 1

R = 3

Sa/g = 2.5

Ah = (ZI/2R) (Sa/ g)

Vb= Ah x W

The fundamental periods on both x and y directions are respectively given below:

Tx=0.38 Sec, Ty=0.36 Sec.

From figure 2 of IS 1893, Sa/g is found to be 2.5 for both the periods, so the design horizontal seismic coefficient (Ah) will be same which will result in producing the same amount of base shear in both the directions.

Ah = (ZI/2R) (Sa/g) = (0.16x1/2x3) x (2.5) = 0.067

Vb = Ah x W = 0.067 x 26230kN = 1575kN

So the base shear for 5 storey building is found to be equal to 1575kN

CHAPTER 9

Static Non –Linear Pushover Analysis

Pushover analysis is a performance based design which is recommended by Euro code and FEMA 273 and FEMA 356. This method considers the nonlinear behaviour of the structure which increases the load taking capacity of the building. It also focuses on ductility of the structure by providing plastic hinges. Pushover analysis is applicable to new and existing structures which can be a good method for retrofitting of structures after its design life is over. It considers target displacement and defining objectives whenever the performance meet the objectives then the damage at that performance level is acceptable.

9.1 GENERAL

The use of the non linear static pushover analysis came into practice in 1970's but the potential of pushover analysis has been recognized for last 10 to 15 years. This procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake. This procedure can be used for checking the adequacy of the new structural design. Push over analysis is defined as an analysis wearing a mathematical model directly incorporating the normal load deformation characteristics of individual components and elements of the building shall be

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subjected to monotonically interesting lateral loads representing inertia forces in an earthquake.

9.1.1 Steps involved

- Evaluation of Capacity of building i.e. Representation of the structure's ability to resist forces
- Evaluation of Demand curve i.e. Representation of earthquake ground motion.
- Determination of Performance point i.e. Intersection point of demand curve and capacity.

The performance of a building is depended upon the performance of the structural and the non-structural components. After obtaining the performance point, the performance of the structures is checked against these performance levels.

9.1.2 Description

Federal Emergency Management Agency (FEMA) and Applied Technical Council (ATC) are the two agencies which formulated and suggested the non linear static analysis or push over analysis under seismic rehabilitation programs and guidelines. This included the documents FEMA-356, FEMA-273 and ATC-40.

(i) Introduction to FEMA-356

The primary purpose of FEMA-356 document is to provide technically sound and nationally acceptable guidelines for the seismic rehabilitation of the buildings. The guide lines for the seismic rehabilitation of the buildings are intended to serve as a ready tool for design professional for carrying out the design and analysis of the buildings, a reference document for the building regulatory officials and a foundation for future development and implementation of the building code provisions and standards.

(ii) Introduction to ATC-40

Seismic evaluation and retrofit of concrete buildings commonly referred to as ATC-40 was developed by Applied Technical Council (ATC) with funding with California safety Commission. Although the procedures recommended in this document are for concrete buildings, they are applicable to most building types.

9.2 TYPES OF PUSHOVER ANALYSIS

Presently there are two non linear static analysis procedures available, one termed as the Displacement Coefficient Method (DCM), documented FEMA-356 and other is the Capacity Spectrum Method (CSM) documented in ATC-40. Both the methods depend on the lateral load deformation variation obtained by non linear static analysis under the gravity loading and idealized lateral loading due to the seismic action. This analysis is called Push Over Analysis.

9.2.1 Capacity Spectrum Method

CSM is a non linear static analysis procedure which provides a graphical representation of the expected seismic performance of the structure by intersecting the structure's capacity spectrum with the response spectrum of the earthquake. The intersection point is called the performance point. The displacement coordinate of the performance point is the estimated displacement demand on the structure for the specified level of seismic hazard.

9.2.2 Displacement Coefficient Method

DCM is a non linear static analysis procedure which provides a numerical process for estimating the displacement demand on the structure, by using a bilinear representation of the capacity curve and a series of modification factors or coefficients to calculate a target displacement. The point on the capacity curve at the target displacement is equivalent of the performance point in the capacity spectrum method.

9.3 PROCEDURE

Pushover analysis includes the application of increasing lateral loads or deformations to a nonlinear mathematical model of a structure. The nonlinear load-deformation behaviour of each section of the structure is modelled in separate way. In a force-controlled push, the loads are applied monotonically until either the total load reaches a target value or the building has a collapse mechanism. In a displacement-controlled push, the displacements are increased monotonically until either the displacement of a predefined control node in the building exceeds a target value or the building has a collapse mechanism. For convenience, the control node can be taken at the design centre of mass of the roof of the building. The target displacement is the maximum considered displacement that is approximated and predefined initially.

First of all the structure to be designed for gravity loads in any design software and then the pushover analysis to be performed. The lateral load as per is 1893 is applied in increasing manner or the first fundamental mode shape is used to take the seismic demand force from the dynamic characteristics. It is very important to determine the displacement control point and the direction of the first fundamental mode. The plastic hinges to be defined for each beam and column at both ends.

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There are two possibilities the first possibility is that the load may reach its target value and the building at that value of load is safe, where the second case it can reach collapse mechanism. Even in the collapse mechanism the hinges should be carefully studied and the performance point maybe observed if the performance point exists and the failure at that level is acceptable then the overall performance of the structure at that level is acceptable.

- Capacity**:** The capacity of the structure in general depends on the displacement each individual member can take or we can say that the capacity of structure depends on the capacities of individual components deformation. Considering this phenomenon the critical sections are determined and the mathematical model of the structure is enhanced and the response is calculated again until the demand is satisfied.
- Demand: As we know the earthquake yields in complex horizontal displacements for any structure. The maximum target displacement is the displacement assumed to be from the potential earthquake. Basically this target displacement is the demand. Once the maximum forces applied to the building laterally could not result in the displacement beyond the target displacement then it is concluded that the building performed well.
- Performance level: The Performance level of the building is defined in terms of the collapse state of the building. Buildings which yields to more plastic hinges is said to have performed badly against certain earthquake. When there are less number of plastic hinges then it's said to be performing well.

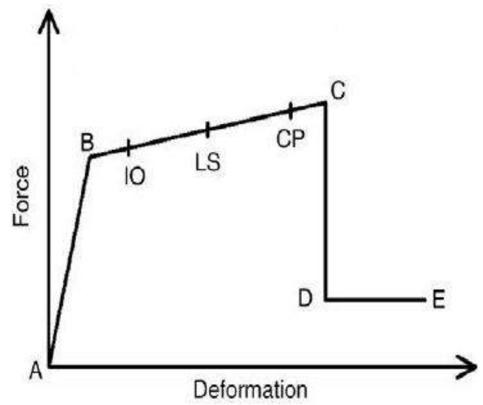

Fig.9.1 Force V/s deformation curve

The performance levels (IO, LS, and CP) of a structural element are represented in the load versus deformation curve as shown below,
1. A to B – Elastic state,
i) Point 'A' corresponds to the unloaded condition.
ii) Point 'B' corresponds to the onset of yielding.
2. B to IO- below immediate occupancy,
3. IO to LS – between immediate occupancy and life safety,
4. LS to CP- between life safety to collapse prevention,
5. CP to C – between collapse prevention and ultimate capacity,
i) Point 'C' corresponds to the ultimate strength
6. C to D- between C and residual strength,
i) Point 'D' corresponds to the residual strength
7. D to E- between D and collapse
i) Point 'E' corresponds to the collapse.

CHAPTER 10
PUSH OVER IN ETABS

The following steps are done in ETABS for push over analysis.

10.1 DEFINE LOAD PATTERNS

In addition to DL and LL, earthquake loads push x and push y is defined.

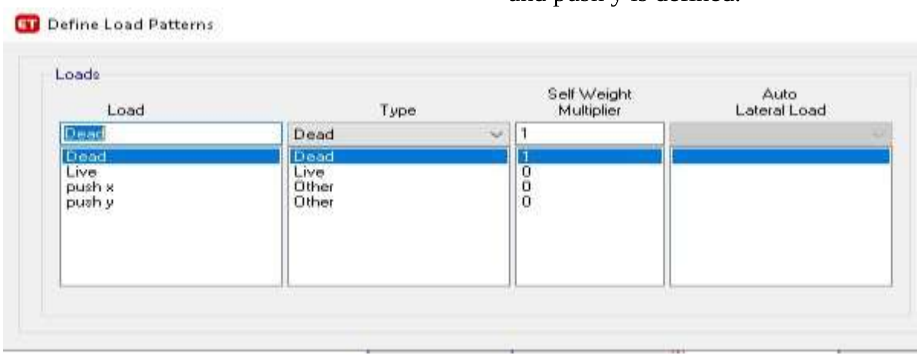

Fig.10.1 Define load patterns

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10.2 DEFINE LOAD CASES PUSH X AND PUSH Y

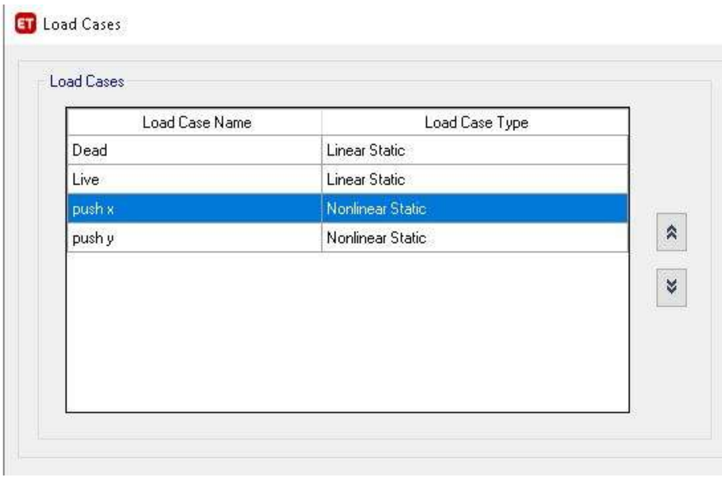

Fig.10.2 Load cases

(i) Push X

- Load case type – non linear static
- Mass source – MsSrc1
- Load type – acceleration
- Load application – displacement control
- Result saved – multiple states
- Load name – UX
- Scale factor – 1

(ii) Push Y

- Load case type – non linear static
- Mass source – MsSrc1
- Load type – acceleration
- Load application – displacement control
- Result saved – multiple states
- Load name – UY
- Scale factor - 1

10.3 ASSIGN AUTO PLASTIC HINGES TO BEAMS

- Hinge property – auto
- Auto hinge assignment data
  - Type – From tables in ASCE 41-17
  - Table – Table 10-7 (Concrete beams- Flexure) item i
  - DOF- M3

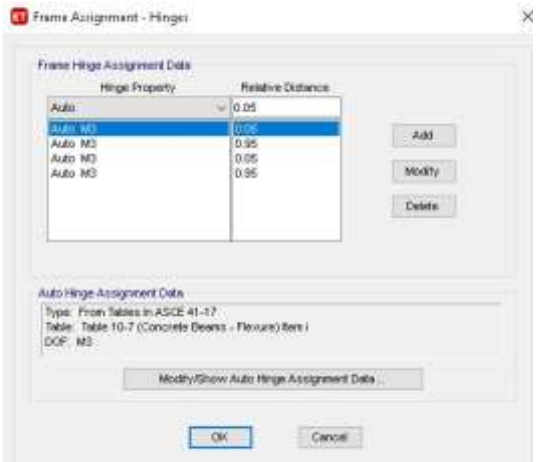

Fig.10.3 Auto hinge assignment in beams

Fig.10.4 Assigned auto hinges in beam

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10.4 ASSIGN AUTO PLASTIC HINGES TO COLUMNS

- Hinge property – auto
- Auto hinge assignment data
  - Type – From tables in ASCE 41-17
  - Table – Table 10-8 and 10-9 (Concrete columns)
  - DOF- P2-M2-M3

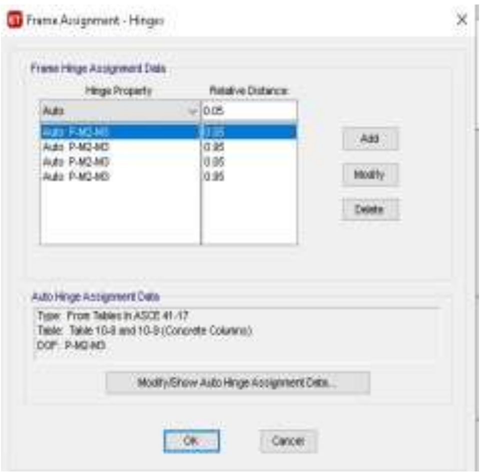

Fig.10.5 Auto hinge assignment in columns

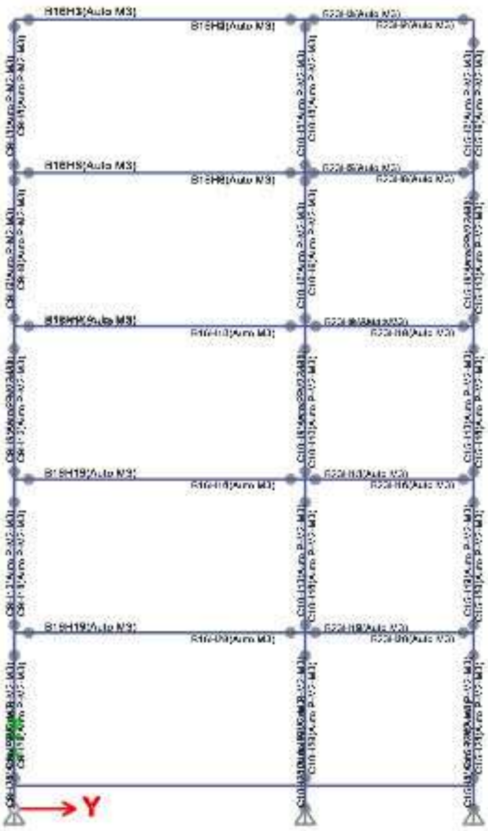

Fig.10.6 Assigned auto hinges in columns

10.5 RUN ANALYSIS

Run analysis for push X, push Y, Dead Load and Live load cases

Fig.10.7 Run analysis

CHAPTER 11
TIME HISTORY ANALYSIS
11.1 GENERAL

Structural dynamics is a type of structural analysis which covers the behavior of a structure subjected to dynamic (actions having high acceleration) loading. Dynamic loads include people, wind, waves, traffic, earthquakes, and blasts. Any structure can be subjected to dynamic loading. Dynamic analysis can be used to find dynamic displacements, time history, and modal analysis. In time history analysis, the structural response is computed at a number of subsequent time intervals. In other words, time histories of the structural response to a given input are obtained and a result.

11.2 FAST NON LINEAR ANALYSIS

Fast Nonlinear Analysis (FNA) is a modal analysis method useful for the static or dynamic evaluation of linear or nonlinear structural systems. Because of its computationally efficient formulation, FNA is well-suited for time-history analysis, and often recommended over direct-integration applications. During dynamic-nonlinear FNA application, analytical models should:

- Be primarily linear-elastic.

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- Have a limited number of predefined nonlinear members.
- Lump nonlinear behavior within link objects.

In addition to nonlinear material force-deformation relationships, these link objects may simulate concentrated damping devices, isolators, and other energy-dissipating technologies. If fuse mechanisms are not integral to the design intention, an initial elastic analysis may reveal locations where inelasticity is likely to occur. However, it is always best to predefine inelastic mechanisms such that their design may provide for sufficient ductility, while elastic systems are ensured sufficient strength. Capacity Design provides for a more reliable model and a better-performing structure.

The efficiency of FNA formulation is largely due to the separation of the nonlinear-object force vector $R_{NL}(t)$ from the elastic stiffness matrix and the damped equations of motion, as seen in the fundamental equilibrium equation of FNA, expressed as:

$$M\ \ddot{u}(t) + C\ \dot{u}(t) + K\ u(t) + R_{NL}(t) = R(t) \qquad \text{Eqn(6)}$$

Stiffness- and mass-orthogonal Load-Dependent Ritz Vectors represent the equilibrium relationships within the elastic structural system. At each time increment, the uncoupled modal equations are solved exactly, while forces within the predefined nonlinear DOF, indexed within $R_{NL}(t)$, are resolved through an iterative process which converges to satisfy equilibrium. Following this procedure, FNA is an efficient and accurate dynamic-nonlinear application which satisfies equilibrium, force-deformation, and compatibility relationships.

11.3 TIME HISTORY FUNCTION

- Linear cases always start from zero, therefore the corresponding time function must also start from zero.
- Nonlinear cases may either start from zero or may continue from a previous case. When starting from zero, the time function is simply defined to start with a zero value. When analysis continues from a previous case, it is assumed that the time function also continues relative to its starting value. A long record may be broken into multiple sequential analyses which use a single function with arrival times. This prevents the need to create multiple modified functions.

In this study, non linear time function is considered and the program files are extracted from the El Centro Earthquake.

11.3.1 El Centro earthquake

The 1940 El Centro earthquake (or 1940 Imperial Valley earthquake) occurred at 21:35 Pacific Standard Time on May 18 (05:35 UTC on May 19) in the Imperial Valley in Southern California near the international border of the United States and Mexico. It had a moment magnitude of 6.9 and a maximum perceived intensity of X (Extreme) on the Mercalli intensity scale. It was the first major earthquake to be recorded by a strong-motion seismograph located next to a fault rupture. The earthquake was characterized as a typical moderate-sized destructive event with a complex energy release signature. It was the strongest recorded earthquake to hit the Imperial Valley, and caused widespread damage to irrigation systems and led to the deaths of nine people.

Table.11.1 Earthquake details

Title	El Centro earthquake
Date	May 18, 1940
Time	21:35 PST
Magnitude	6.9 M_w
Depth	16 Km
Epi center	32.733°N 115.5°WCo ordinates
Type	Strike-slip
Affected area	United states, Mexico
Total damage	$6 Million
Max intensity	X, Extreme

CHAPTER 12
TIME HISTORY ANALYSIS IN ETABS

The following steps are done in ETABS for time history analysis.

12.1 DEFINE TIME HISTORY FUNCTION

Time history function is defined by extracting the time history data of El Centro Earthquake.

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Fig.12.1 Time history function definition

12.2 DEFINE LOAD CASES

In addition to DL, LL, earthquake loads in x and y direction, Time history loads in x and y direction are defined. Both are defined as fast non linear.

Load case data is provided with following details:

- Load case type : Time history
- Load case subtype : Non linear Modal
- Initial conditions : Zero initial condition
- Load type : Acceleration
- No. of output time steps : 200
- Output time step size : 0.1 sec

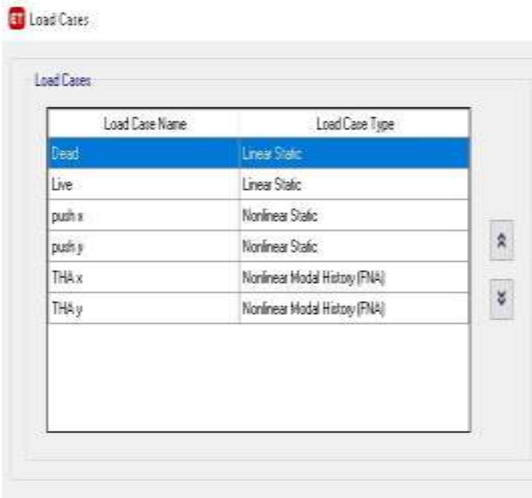

Fig.12.2 Load cases

Fig.12.3 Load case data

12.3 RUN ANALYSIS

The load cases of dead load, live load earthquake load in x and y direction, time history load in x and y direction are set to run.

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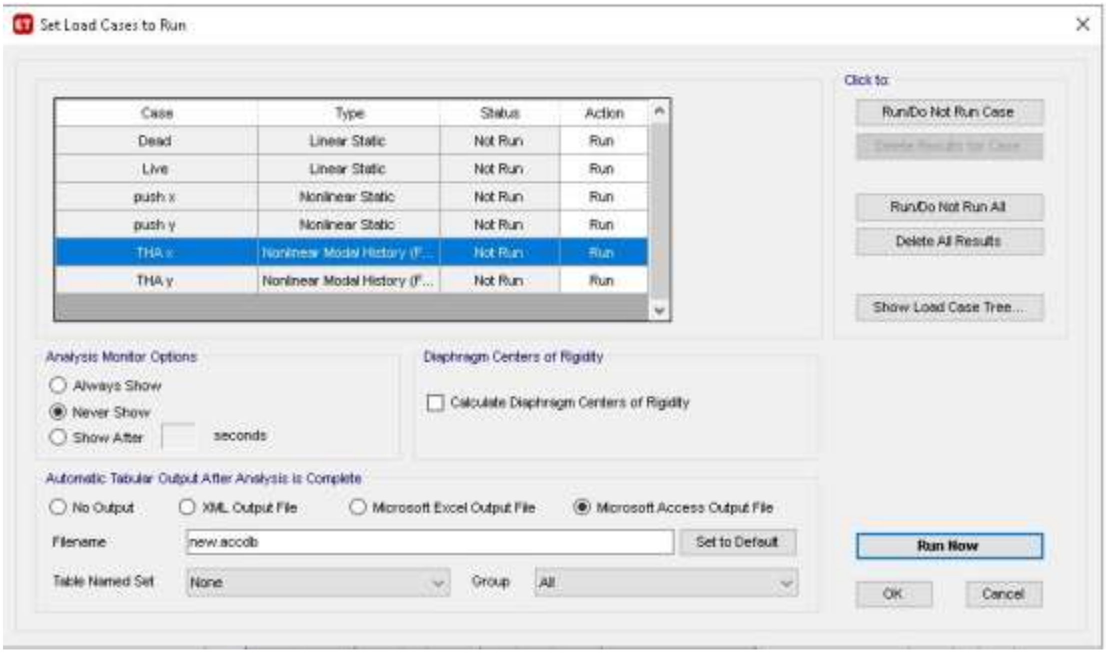

Fig.12.4 Run Analysis

CHAPTER 13
RESULTS AND DISCUSSION
13.1 PUSHOVER ANAYSIS RESULTS

Pushover Analysis was carried out over the designed G+4 storey building using ETABS 2018. The members were assigned with their self-weight of the building considering beams, columns slabs. And the analysis was carried out for combinations of loads as per IS 1893-2002. The building is pushed in lateral directions until the collapse mechanism is reached. The various curves resulting from the analysis are briefly discussed below.

13.1.1 Pushover analysis of G+4 RC building

The figure shows the Pushover curve base shear vs lateral displacement. The unit for Base Reaction is kN and Displacement is meter. The maximum node displacement is equal to -168.18 m. The Pushover Curve shows that the building has objectively high Base Shear Capacity than the Design Base Shear. The Design base shear (V_B) was found to be 1575 kN in chapter 8 and the capacity is 7637 kN which is much higher, hence the building is safe for this level of earthquake.

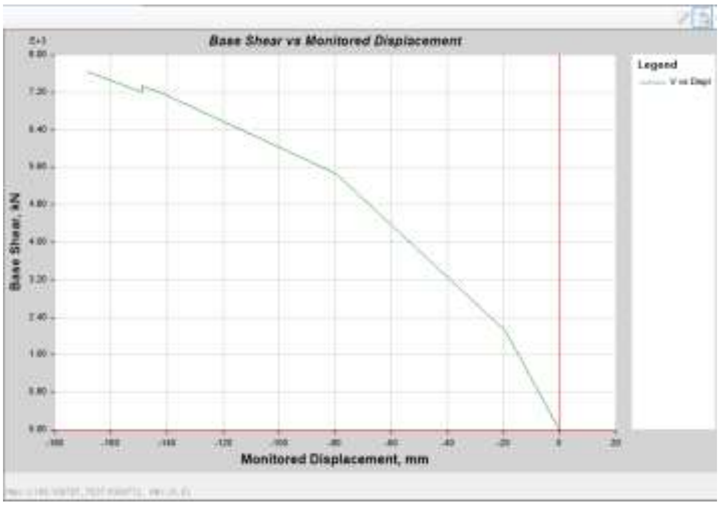

Fig.13.1 Push over curve

13.1.2 Hinge formation

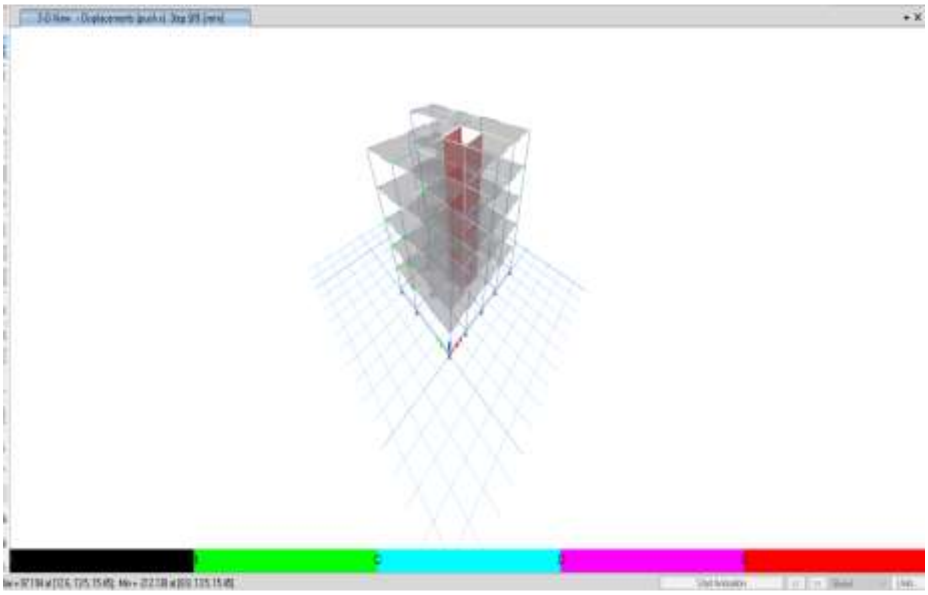

Fig.13.2 Hinge formed due to push X

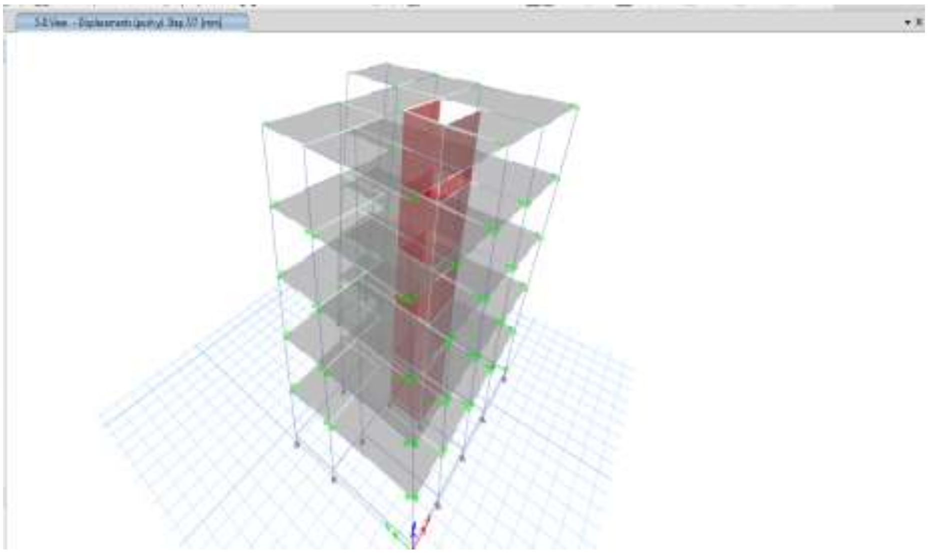

Fig.13.3 Hinge formed due to push Y

13.1.3 Displacement under loads

(i) Dead load

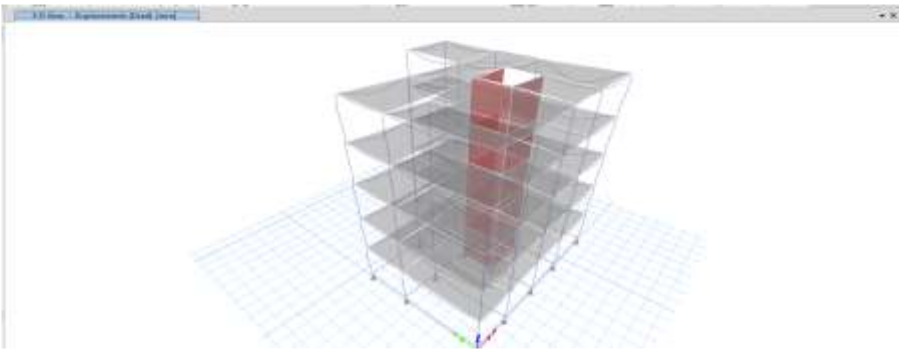

Fig.13.4 Displacement due to Dead Load

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(ii) Live load

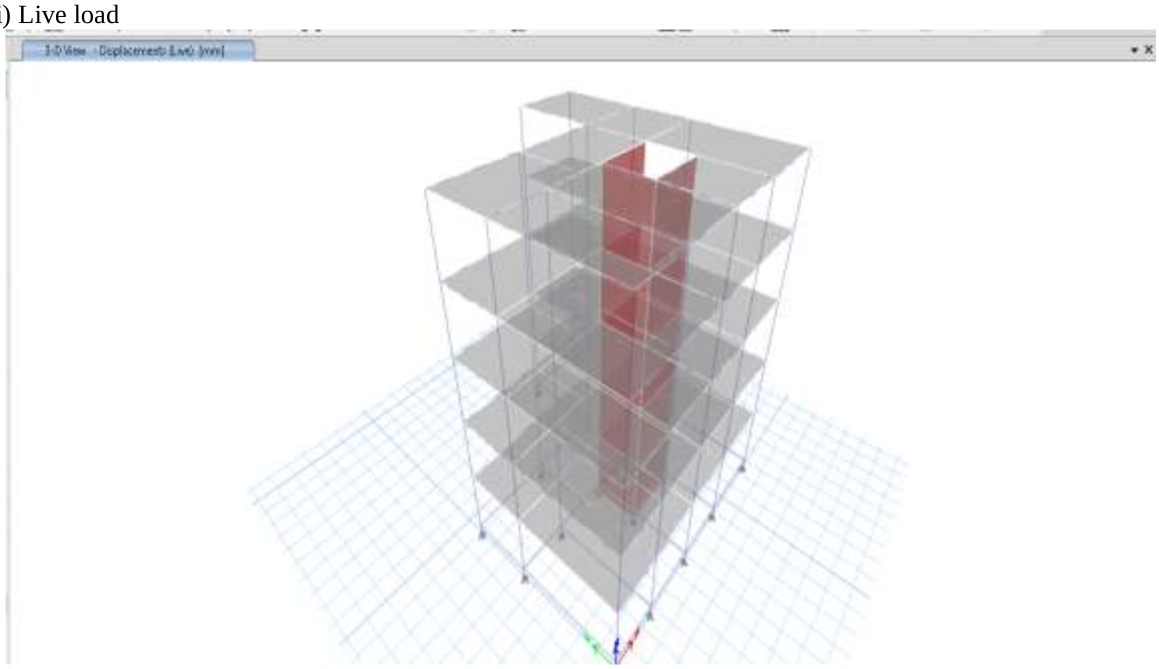

Fig.13.5 Displacement due to Live load

(iii) Push X (Step 9)

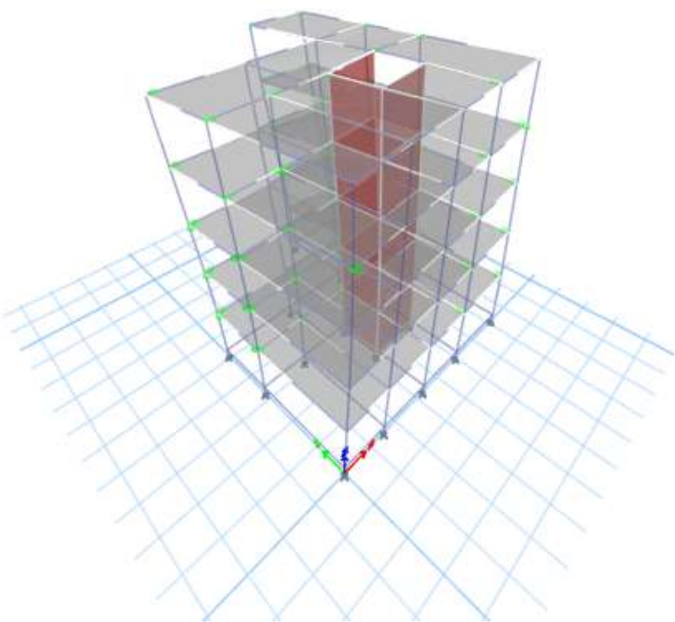

Fig.13.6 Displacement due to push X

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(iv) Push Y (Step 7)

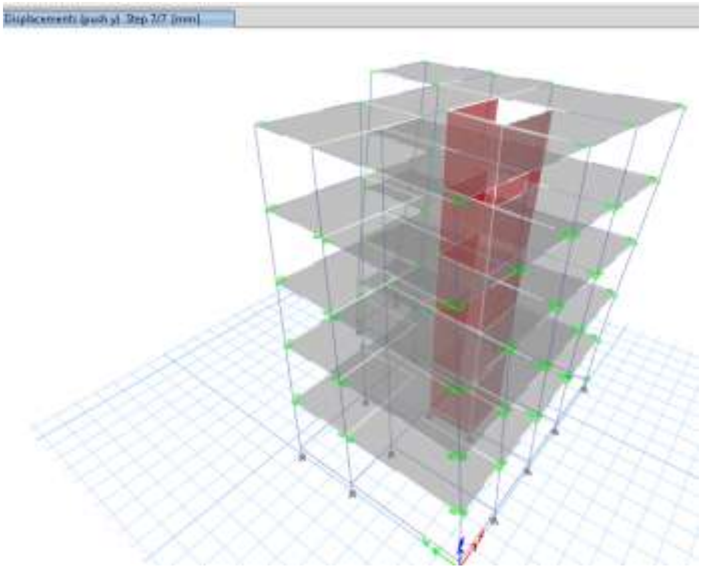

Fig.13.7 Displacement due to push Y

13.1.4 Performance Point

The capacity curve is also called as pushover curve which is a plot of base shear vs displacement where the capacity spectrum is the plot between base acceleration and the roof displacement. The Response spectrum is the plot between spectral acceleration and the monitored displacement. The Performance point is the intersection of the demand and capacity curves. During pushover analysis if a building is having performance point and the damage at the same point is acceptable then the structure is said to be satisfying the target performance level. At performance point, the effective time period is found to be 0.334 sec.

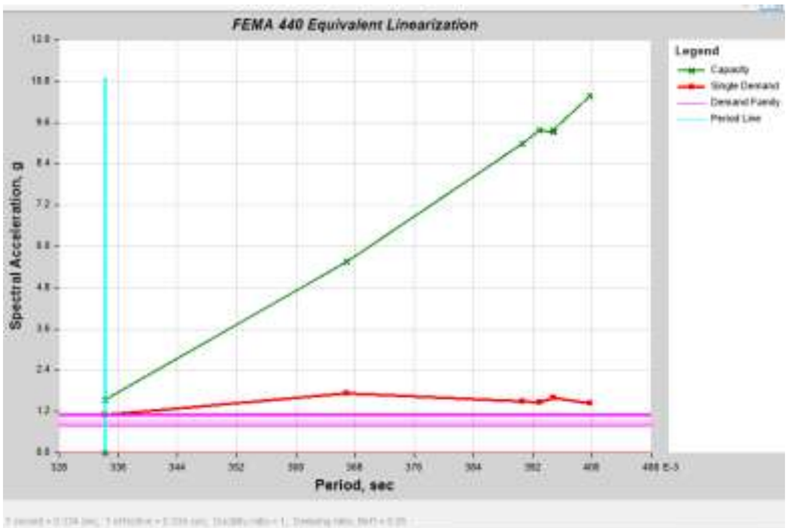

Fig.13.8 Performance point

Maximum base force is found to be 9847.23 kN at step 8 of push X and minimum is found to be zero

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Output Case	Case Type	Step Type	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	X m	Y m	Z M
push x	NonStatic	Max	9847.2338	9.948E-06	0	0	84120.0087	0	0	0	0
push x	NonStatic	Min	0	0	-1.828E-06	-0.0001	0	-51740.0992	0	0	0
push y	NonStatic	Max	0	7611.5826	0	0	0	50207.9448	0	0	0
push y	NonStatic	Min	-8.834E-07	0	0	-65021.9554	-2.068E-05	0	0	0	0

Fig.13.9 Base force

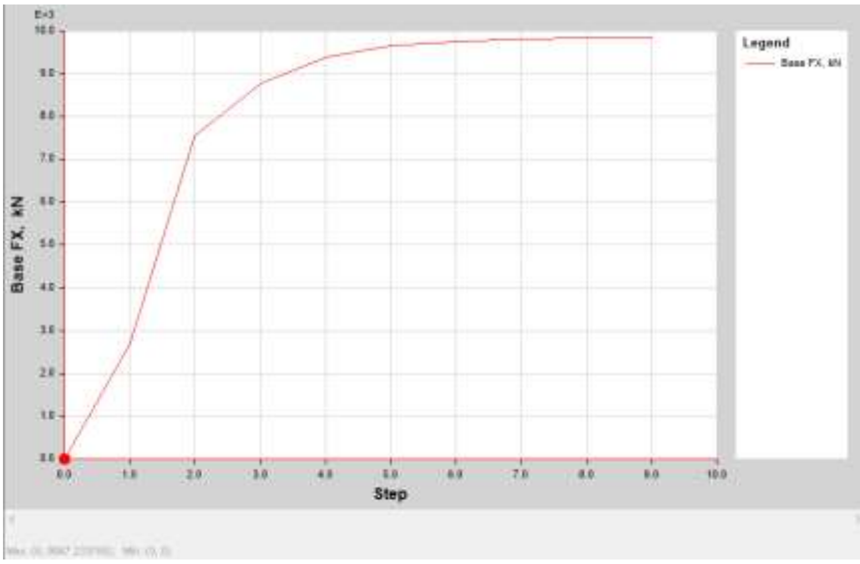

Fig.13.10 Base force v/s step curve

13.2 TIME HISTORY ANALYSIS RESULTS
13.2.1 Modal Results

Case	Mode	Period sec	Frequency cyc/sec	CircFreq rad/sec	Eigenvalue rad2/sec2
Modal	1	0.825	1.212	7.6153	57.9921
Modal	2	0.368	2.72	17.0898	292.0604
Modal	3	0.339	2.946	18.5094	342.5967
Modal	4	0.239	4.176	26.2404	688.5578
Modal	5	0.122	8.214	51.607	2663.2825
Modal	6	0.087	11.49	72.1918	5211.6569
Modal	7	0.081	12.307	77.3241	5979.0138
Modal	8	0.074	13.565	85.2284	7263.8868
Modal	9	0.065	15.386	96.675	9346.0467
Modal	10	0.057	17.56	110.3316	12173.0635
Modal	11	0.052	19.201	120.6456	14555.3704
Modal	12	0.048	20.834	130.9066	17136.5407

Fig.13.11 Modal periods and frequencies

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Case	Mode	Period sec	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX
Modal	1	0.825	0.0138	0.083	0	0.0138	0.083	0	0.0314	0.0051	0.6413	0.0314
Modal	2	0.368	0.6155	0.0372	0	0.6294	0.1201	0	0.0183	0.3246	0.0017	0.0496
Modal	3	0.339	0.0248	0.5593	0	0.6541	0.6795	0	0.2727	0.0153	0.1026	0.3223
Modal	4	0.239	0.0023	0.0207	0	0.6564	0.7002	0	0.0243	0.0072	0.1013	0.3466
Modal	5	0.122	0.001	0.0004	0	0.6574	0.7006	0	0.0002	0.0018	0.0458	0.3469
Modal	6	0.087	0.0015	0.1671	0	0.6589	0.8677	0	0.2951	0.0023	0.0146	0.642
Modal	7	0.081	0.0001	0.0059	0	0.659	0.8736	0	0.0108	0.0004	0.0108	0.6528
Modal	8	0.074	0.1914	0.0004	0	0.8503	0.874	0	0.0007	0.259	0.0048	0.6535
Modal	9	0.065	0.0001	0.0004	0	0.8504	0.8744	0	0.0007	0.0001	0.0027	0.6542
Modal	10	0.057	4.018E-05	0.0026	0	0.8504	0.877	0	0.0079	0.0001	0.0002	0.6621
Modal	11	0.052	0.0005	0.0012	0	0.851	0.8783	0	0.0037	0.0016	0.0007	0.6658
Modal	12	0.048	0.0039	0.0009	0	0.8549	0.8792	0	0.0026	0.0119	0.0032	0.6684

Fig.13.12 Modal Participating Mass Ratios

Case	Mode	Period sec	UX	UY	UZ	RZ
Modal	1	0.825	0.019	0.114	0	0.867
Modal	2	0.368	0.941	0.055	0	0.004
Modal	3	0.339	0.04	0.83	0	0.129
Modal	4	0.239	0.019	0.091	0	0.89
Modal	5	0.122	0.017	0.068	0	0.915
Modal	6	0.087	0.008	0.868	0	0.124
Modal	7	0.081	0.014	0.049	0	0.937
Modal	8	0.074	0.974	0.002	0	0.024
Modal	9	0.065	0.012	0.02	0	0.967
Modal	10	0.057	0.012	0.713	0	0.275
Modal	11	0.052	0.011	0.021	0	0.967
Modal	12	0.048	0.026	0.005	0	0.969

Fig.13.13 Modal Direction factors

13.2.2 Structure Results

Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

Base Reactions

Output Case	Case Type	Step Type	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	X m
push x	NonStatic	Max	9849.5488	5.666E-06	0	0	84139.7844	0	0
push x	NonStatic	Min	0	0	-1.829E-06	-0.0001	0	-51752.2628	0
push y	NonStatic	Max	2.176E-06	6112.9833	0	0	1.173E-05	40322.8008	0
push y	NonStatic	Min	0	0	0	-52220.1687	0	0	0
THA x	NonModHist	Max	2.9546	0.2416	0	2.2952	23.1727	5.8817	0
THA x	NonModHist	Min	-1.5345	-0.2009	0	-2.7337	-15.3625	-14.9503	0
THA y	NonModHist	Max	0.2416	2.832	0	12.9878	2.7425	17.3101	0
THA y	NonModHist	Min	-0.2009	-1.4881	0	-23.6159	-2.288	-6.3888	0

Fig.13.14 Base reactions

By increasing the scale factor, new base reactions were obtained which satisfies the condition that Time history load in X direction i.e THA x is equal to 85% Earthquake load in X direction i.e Push x. The base reaction in push x was found to be 9849.5488 kN, and the base reaction of time history load case THA x was found to be 8372.3662 kN.

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Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted
Filter:

Base Reactions

Output Case	Case Type	Step Type	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m	X m
push x	NonStatic	Max	9849.5488	5.666E-06	0	0	84139.7844	0	0
push x	NonStatic	Min	0	0	-1.829E-06	-0.0001	0	-51752.2628	0
push y	NonStatic	Max	2.176E-06	6112.9833	0	0	1.173E-05	40322.8008	0
push y	NonStatic	Min	0	0	0	-52220.1687	0	0	0
THA x	NonModHist	Max	8372.3662	685.4364	1.095E-06	6381.8395	65977.5083	16376.6034	0
THA x	NonModHist	Min	-4335.2017	-559.0406	-1.514E-06	-7734.0224	-43650.0013	-42391.6157	0
THA y	NonModHist	Max	680.7241	8044.9463	1.08E-06	36608.267	7740.7583	49252.7858	0
THA y	NonModHist	Min	-559.6492	-4192.965	-9.308E-07	-67373.0105	-6406.9207	-17978.1563	0

Fig.13.15 Modified base reaction

13.2.3 Response spectrum curve

Response spectra are curves plotted between maximum response of SDOF system subjected to specified earthquake ground motion and its time period (or frequency). Here, maximum value occurs at zero damping.

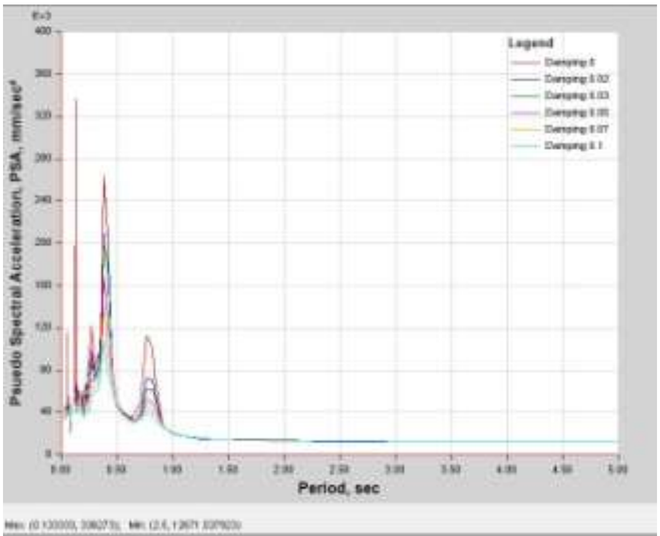

Fig.13.16 Response spectrum curve

13.3 SUMMARY

A G+4 storied reinforced concrete building was taken to analysis. The structure was subjected to design earthquake forces as specified in the IS code for zone III. Pushover curves for the building is generated that shows the behavior of the structure in terms of its stiffness and ductility. The design base shear was found out to be 1575 kN and the capacity of the performance point is found to be 7637 kN. It clearly indicates that the capacity is higher than the design base shear. Hence the performance at this point is acceptable.

The same building was taken for time history analysis. The time history data of El Centro earthquake was considered for the study. Modal periods and frequencies, Modal direction factors were obtained. Modal participating mass ratios of the final modes were approximately equal to 1. By increasing the scale factor, the base reaction of the time history load (8372.3662 kN) in x direction was equal to 85% of the base reaction of the earthquake load in x direction (9849.5488kN). Response spectrum curves were plotted between pseudo spectral acceleration and time period. Maximum occurs at zero damping condition.

CHAPTER 14
CONCLUSION

After studying the curves and tables a conclusion was made that the pushover analysis result and time history analysis shows that the building was able to achieve the performance point within its elastic range.

Further it can be conclude that, Pushover analysis the simplest way to get the response of existing or new structures. Considering this project, it was concluded if the buildings are designed with proper sections and reinforcement details as per standard codes will perform better under seismic forces. Hinges developed from the lower floors, indicating that the lower floor columns shall have more reinforcement area than upper floor columns. The performance of the pushover analysis mostly depends on the material used in the structure. The time history data of El Centro earthquake was considered for the study. By increasing the scale factor, the base reaction of the time history load in x direction was equal to 85% of the base reaction of the earthquake load in x direction. Hence the building is safe against dynamic loading condition.

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