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A Comparative Study on Seismic Performance Evaluation between Reinforced Concrete Structures and Post Tensioned Concrete Structures 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. Seismic design is a vital process of structural analysis while designing a building which is subjected to earthquake motions. Thus seismic analysis is a tool to estimate the structural response of the structure while designing earthquake resistant structures or vulnerably existing structures. In present study, seismic performance evaluation 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 results are then compared with the same building modelled with post tensioned slabs and carried out the same seismic evaluation methods using ETABS.

Keywords: seismic, push over, time history, post tension

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 Types of Structures

1.2.1 Reinforced Concrete Structures

The reinforced concrete structure refers to the members, such as beams, columns, roof trusses, consisting of concrete and steel bars. Reinforced concrete structures can be classified based on different parameters. On the basis of regularity, they are classified as regular and irregular structures. Irregularity can be further sub classified as mass irregular stiffness irregular and geometric irregular. Geometric irregularity is again divided as horizontally irregular (plan) and vertically irregular buildings.

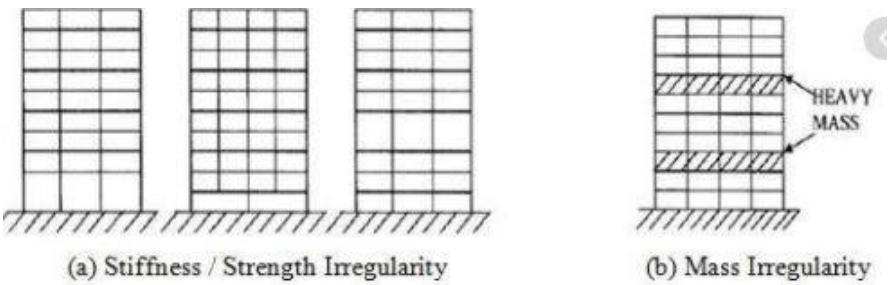


Fig.1.1 Irregular buildings

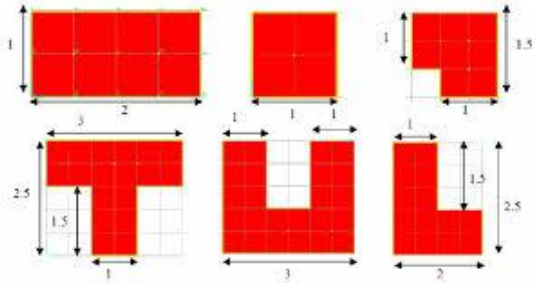


Fig.1.2 Plan regular and plan irregular building

1.2.2 Post tensioned structures

Post-tensioning is simply a method of producing prestressed concrete, masonry, and other structural elements. When the steel is tensioned before concrete placement, the process is called pretensioning. When the steel is tensioned after concrete placement, the process is called post-tensioning. A post tension slab is a concrete slab that has steel cables running through it that have been placed under 33,000 +/- pounds of tension. This tension makes the concrete slab and foundation much stronger than concrete without reinforcement and helps reduce cracking.

1.3 Seismic Analysis

Seismic analysis is a tool for the estimation of structural response in the process of designing earthquake resistant structures and vulnerable existing structures. It is part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent.

Seismic analysis methods can be divided into the following categories:

1.3.1 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.

1.3.2 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.

1.3.3 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).

1.3.4 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.

1.3.5 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 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.

1.4 Scope of the Present Study

Comparative study between seismic performance of R.C structures and post tensioned structures. There are different seismic analysis methods, but this project is limited to push over analysis and time history analysis. Irregularity in structures can be in different manners, but in this study, plan irregularity is considered.

Literature Review

2.1 General

In today's world the exploding population creating the disasters like land scarcity which leads us to the bringing some new construction technology and commercial structures. A normal building structure has number of beams in it. But while taking flat slabs no beams are casted separately. A structure is said to be more stable

when it satisfy the stability. Two approximate methods are adopted by the codes for the design and analyze the flat slab or say flat plate. These methods can be used provided the limitations specified therein are satisfied. The two design methods are i) The direct design method, ii) The equivalent frame method. In a developing country like India the benefits of pre-stressing and particularly of post-tensioning are yet to be recognized. The inherent hurdle is undoubtedly the higher initial investment that is required from the clients. This has to be overlooked considering the significant benefits of post-tensioning and the high benefit-aspect ratio that can be advantageously procured. In the present study an attempt is made to be compare the design and cost effectiveness of post-tensioned flat slab with respect to the reinforced concrete flat slab system. 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 structures and buildings with insignificant irregularities are adequately conservative, they can be revised to arrive at a more cost-effective design of tall buildings.

Jnanesh Reddy RK (2017) Comparative Study of Post Tensioned and RCC Flat Slab in Multi-Storey Commercial Building: In the present study the attempt is made to comparing the cost effectiveness of Post-Tensioned flat slab systems with respect to RF concrete flat slab system. Both the systems are analyzed using RAPT and ETABS respectively which is based on the design methodology. There are many other benefits of using PT slab. As the thickness of the slab is much lesser than the R.C.C flat slab, aesthetic look of the building may get enhanced leading to a clear height for a longer distance. Hence, using a PT Slab is more advisable for a commercial building than using a R.C.C Flat Slab. Construction of a structure using Post-Tensioned Slab also leads to a lighter structure as the Dead Load gets reduced.

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.

Boskey Bahoria (2010), Comparative Design of RCC & Post-tensioned flat slabs: In the present study the design of the post-tensioned flat slab is done by using two methods, load balancing method and equivalent frame method. The technical study of the post-tensioned flat slab by varying the span by 0.5 m interval is done and results of the different parameters such as thickness of slab, grade of concrete, loss due to stress, normal reinforcement, reinforcement for the shear, Member of tendons, stressing force per tendons and deflection etc. are presented in the graphical form. A design of post tensioned beam is also done. For the study of post tensioned flat slab and beams a case study of multistory office building (G+4floor system) is taken and it is designed by four cases, the post tensioned flat slab, post-tensioned beams and the RCC flat slab and the RCC slab and beams. After the design of these four cases the comparative study with respect to economy is carried

out. The analysis, design and the estimation of the office building of the four floors systems is done. The study shows the variation of the rate per square meter for these four different cases.

U. Prawatwong (2008) Seismic performance of post–tensioned interior slab column connections with and without drop panel: This paper presents an technical study on the seismic activity of two three fifth scale post-tensioned interior slab column connection models, one without drop panel and another on with drop panel. The model without drop panelled was designed and constructed to represents a typical connection between interior column and post-tensioned flat plate with bonded tendons are found in Thailand. The another model was leads to represent an improved design of typical post-tensioned slab-column connections by using drop panel. Both models were tested under a constant same gravitational load.

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. Boskey Bahoria gives the idea about the post tensioned flat slab building structure having four cases depending upon by varying the span length by 0.5 m interval and discuss the comparative study of four cases with respect to economy.U. Prawatwong makes a two models one with drop panel shows the connections between slab-column and another is without drop panel shows connection between interior columns with PT flat plate and bonded tendons having seismic performance on two three fifth scale pattern under constant gravity load to investigate the seismic performance. Jnanesh Reddy RK compares the cost effectiveness of the post-tensioned flat slab with respect to RCC flat slab by using RAPT and ETABS softwares giving the final statement that PT flat slab is more advisable than RCC flat slab because it reduced the dead load by reducing thickness of slab.

Objectives

The objectives of the present study is

- To model a G+4 storey RC irregular building in ETABS 2018 as per IS codes. [Plan irregular building – L shaped]
- To perform pushover analysis to get the seismic response of the structure.
- To perform time history analysis to obtain dynamic response of the structure
- To compare the results with the same building modelled with post tensioned slab under same seismic evaluation methods.

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
- Modelling of post tensioned slabs and analysis of building using ETABS software
- Observation of results and discussions.

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

Table.5.1 Reference journal details

Title	A Proposed Draft for IS : 1893 Provisions on Seismic Design of Buildings – Part II : Commentary and Examples
Name of journal	Journal of Structural Engineering
Author	Sudhir K. Jain
Year	1995, July

Table.5.2 Building details

Sl. No.	Parameter	Value
1	No. of storeys	4
2	Zone	III
3	Live load	3 kN/m^2
4	Columns	450 x 450 mm
5	Beams	250 x 400 mm
6	Slab thickness	150 mm
7	Wall thickness	120 mm
8	Importance factor	1.0
9	Structure type	OMRF

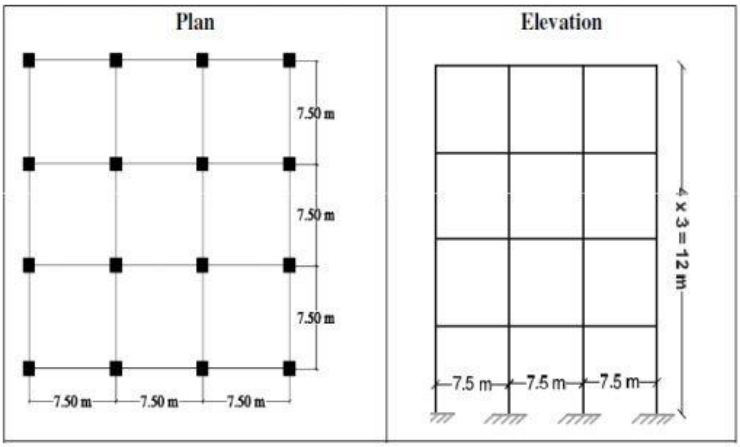


Fig.5.1 Plan and elevation of the building

5.2.1 Calculations

[All values as per IS 1893:2002 Part l]

- Fundamental period
  - Natural time period $T_a = 0.09\ (h/\sqrt{d})$ => $0.09(12/\sqrt{22.5}) = 0.2277$s
- Spectral acceleration
  - For medium type soil & $T_a = 0.2277$s => $S_a/g = 2.5$
  - Zone factor Z = 0.16 (Zone III)
  - Importance factor I =1.0
  - Response reduction factor R = 3.0 (OMRF)
  - Horizontal acceleration coefficient $A_h = Z/2 \times S_a/g \times I/R = 0.0667$
- Computation of seismic weights

- Slab
  - DL due to weight of slab = (22.5 x 22.5 x 0.15) x 25 = 1898 kN
- Beams
  - Self weight of beam per unit length = 0.25 x 0.4 x 25 = 2.5 kN/m
  - Total length = 4 x 22.5 x 2 = 180 m
  - DL due to weight of beams = 2.5 x 180 = 450 kN
- Columns
  - Self weight of beam per unit length = 0.45 x 0.4 x 25 = 5.0625 kN/m
  - DL due to weight of columns = 16 x 5.0625 x 3= 243 kN
- Walls
  - Self weight of wall per unit length = 0.12 x 3 x 20 = 7.2 kN/m
  - Total length = 4 x 22.5 x 2 = 180 m
  - Self weight of wall = 7.2 x 180 = 1296 kN

- Total load on each floor = 1898 + 450 + 243 + 1296 = 3890 kN

5.2.2 Seismic weight computation

- Live load = 3 x 22.5 x 22.5 = 1518.75 kN
- Storey forces : Storey 1 = Storey 2 = Storey 3 = 3890 + 25%(1518.75) = 4270 kN
- Storey forces : Storey 4 = 450 + (243/2) + 1898 + (1296/2) = 3118 kN
- Seismic weight W = 3 x 4270 + 3118 = 15938 kN

Design base shear $V_B = W \times A_h$ = 15928 x 0.066 = 917.88 Kn

5.4 Numerical Method

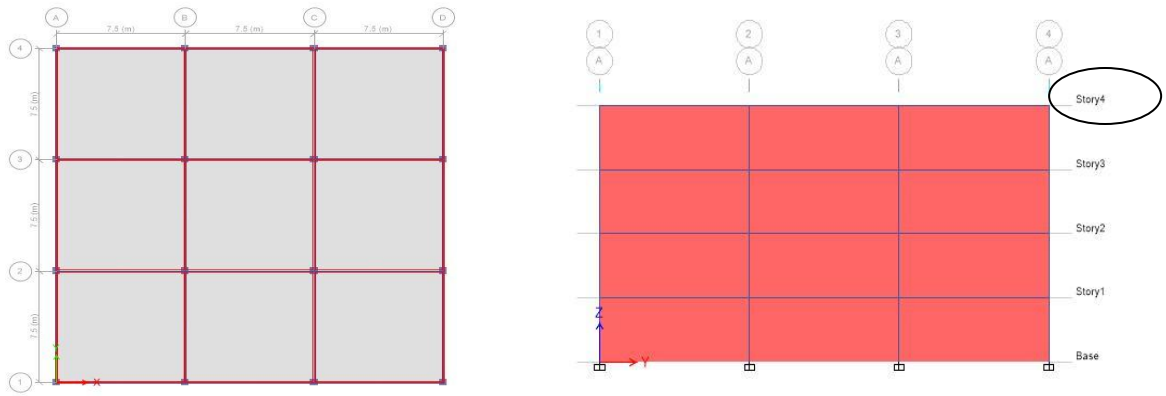


Fig.5.2 Plan and elevation from software

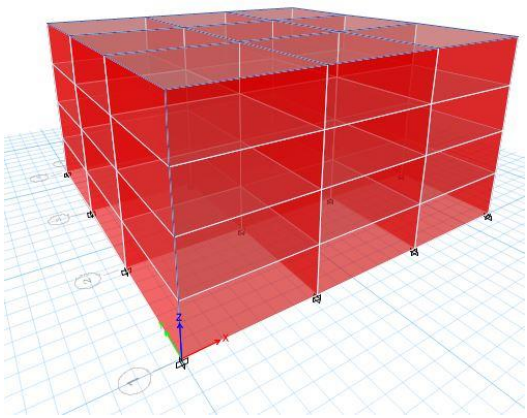


Fig.5.3 3-D View

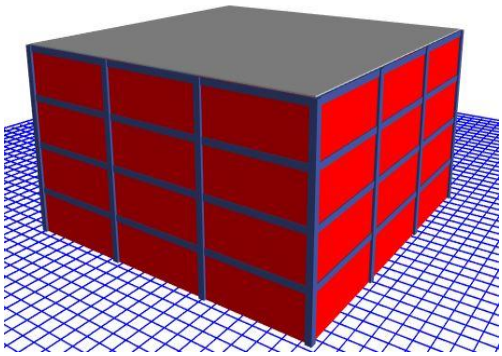


Fig.5.4 Rendered View

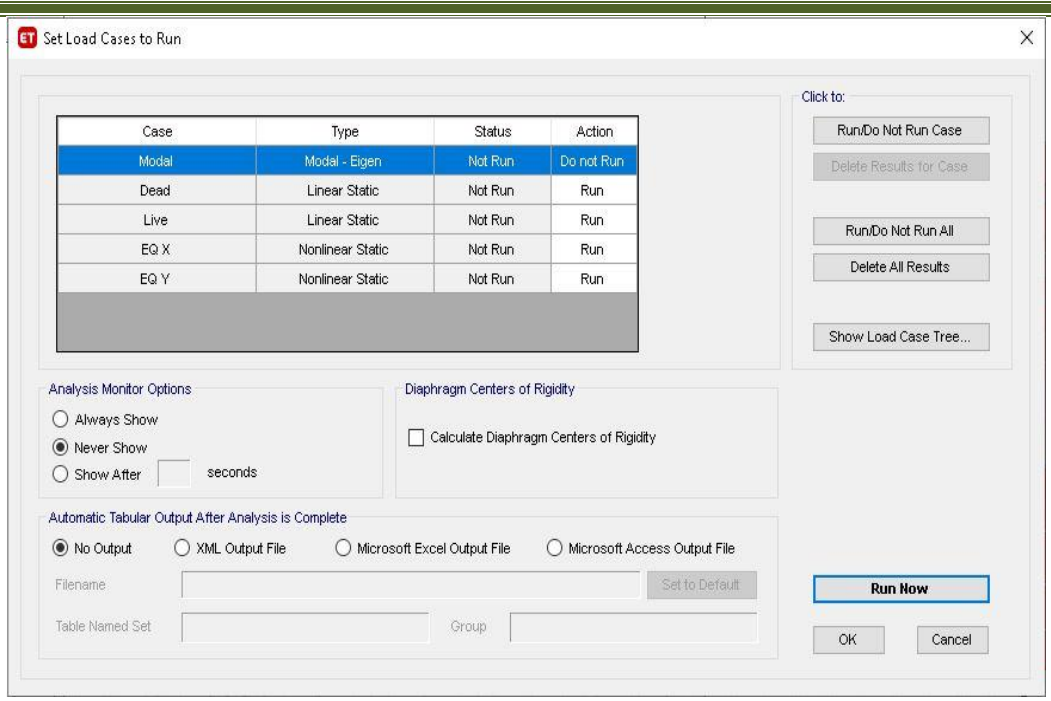
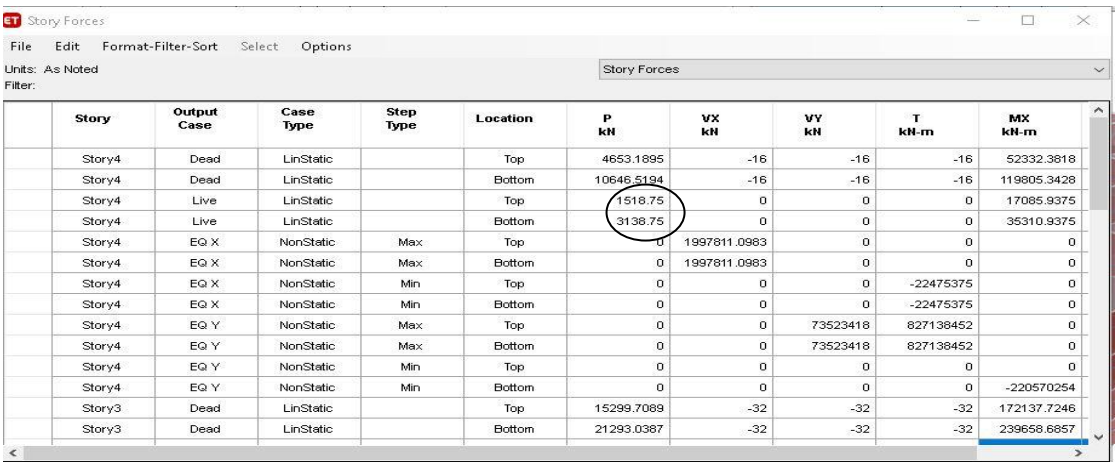


Fig.5.5 Run analysis



Story	Output Case	Case Type	Step Type	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m
Story4	Dead	LinStatic		Top	4653.1895	-16	-16	-16	52332.3818
Story4	Dead	LinStatic		Bottom	10646.5194	-16	-16	-16	119805.3428
Story4	Live	LinStatic		Top	1518.75	0	0	0	17085.9375
Story4	Live	LinStatic		Bottom	3138.75	0	0	0	35310.9375
Story4	EQ X	NonStatic	Max	Top	0	1997811.0983	0	0	0
Story4	EQ X	NonStatic	Max	Bottom	0	1997811.0983	0	0	0
Story4	EQ X	NonStatic	Min	Top	0	0	0	-22475375	0
Story4	EQ X	NonStatic	Min	Bottom	0	0	0	-22475375	0
Story4	EQ Y	NonStatic	Max	Top	0	0	73523418	827138452	0
Story4	EQ Y	NonStatic	Max	Bottom	0	0	73523418	827138452	0
Story4	EQ Y	NonStatic	Min	Top	0	0	0	0	0
Story4	EQ Y	NonStatic	Min	Bottom	0	0	0	0	-220570254
Story3	Dead	LinStatic		Top	15299.7089	-32	-32	-32	172137.7246
Story3	Dead	LinStatic		Bottom	21293.0387	-32	-32	-32	239658.6857

Fig.5.6 Results from numerical analysis

Analysis Results 1/9/2021

Table 3.7 - Story Forces

Story	Output Case	Case Type	Step Type	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story4	Dead	LinStatic		Top	4653.1895	-16	-16	-16	52332.3818	-52364.3818
Story4	Dead	LinStatic		Bottom	10646.5194	-16	-16	-16	119805.3428	-119837.3428
Story4	Live	LinStatic		Top	1518.75	0	0	0	17085.9375	-17085.9375
Story4	Live	LinStatic		Bottom	3138.75	0	0	0	35310.9375	-35310.9375

Fig.5.7 Storey forces

5.5 Validation Summary

Table 5.3 shows the result obtained from numerical and analytical method. Comparing analytical and numerical results, almost same results were obtained with a % variation of 0.66% .

Table.5.3 Comparison of results

Result obtained analytically	Result obtained from numerically	Variation in %
3118 kN	3138.75 kN	0.66%

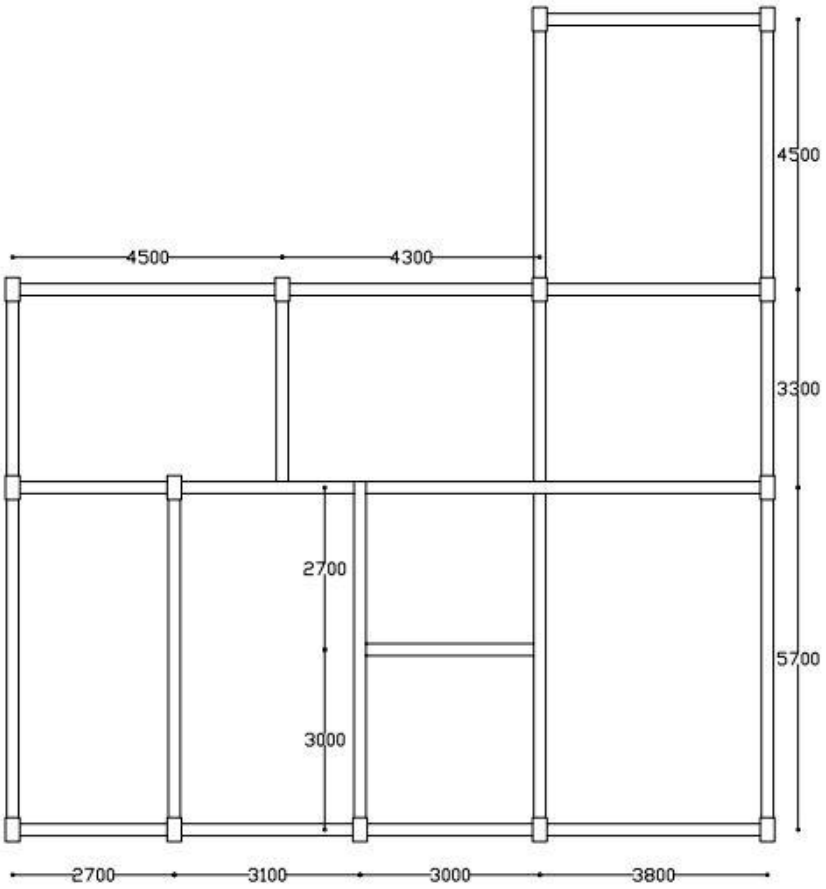
As per the literature of ETABS, the maximum percentage variation is limited to 5%. Here the variation obtained is 0.66% which is within the permissible limit. So the software was validated.

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

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.

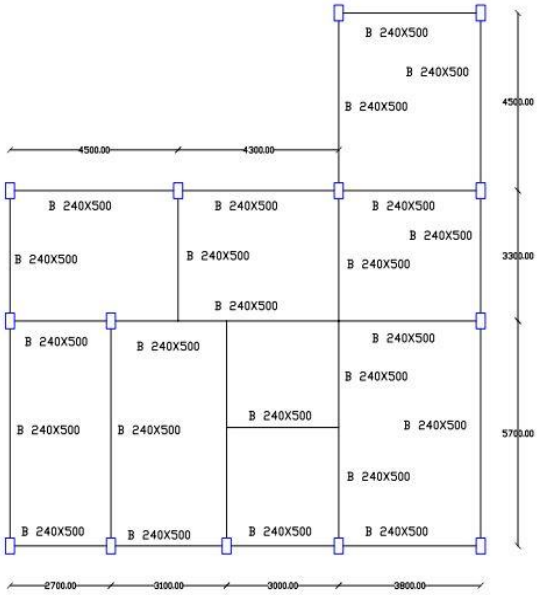


Fig.6.2 Beam layout

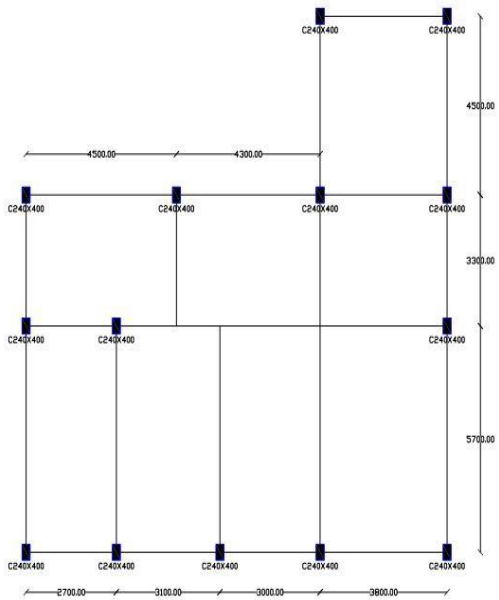


Fig.6.3 Column layout

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.

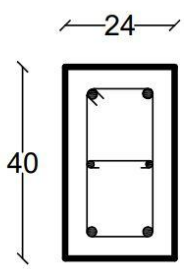
6.2.3 Material properties

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
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

Building Modelling

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

7.1 Reinforced Concrete Building

7.1.1 Storey and grid data

X Grid Data

Grid ID	X Ordinate (m)	Visible	Bubble Loc
A	0	Yes	End
B	1.2	Yes	End
C	2.7	Yes	End
D	3.9	Yes	End
E	4.5	Yes	End
F	5.8	Yes	End

Add Delete Sort

Fig.7.1 X grid data

Y Grid Data

Grid ID	Y Ordinate (m)	Visible	Bubble Loc
1	0	Yes	Start
2	5.7	Yes	Start
3	9	Yes	Start
4	11.5	Yes	Start
5	13.5	Yes	Start

Add Delete Sort

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					

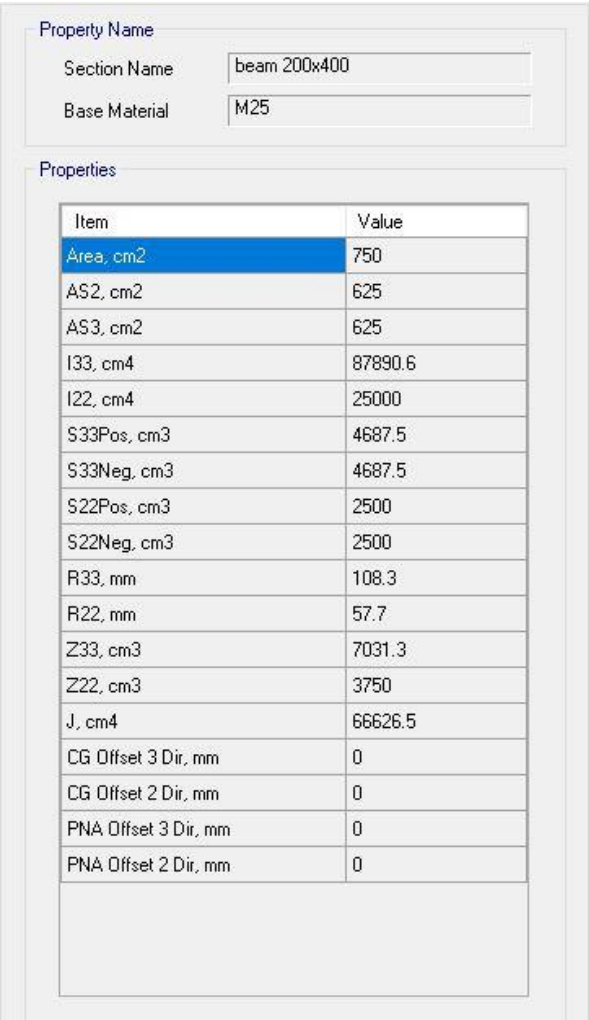
Note: Right Click on Grid for Options

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

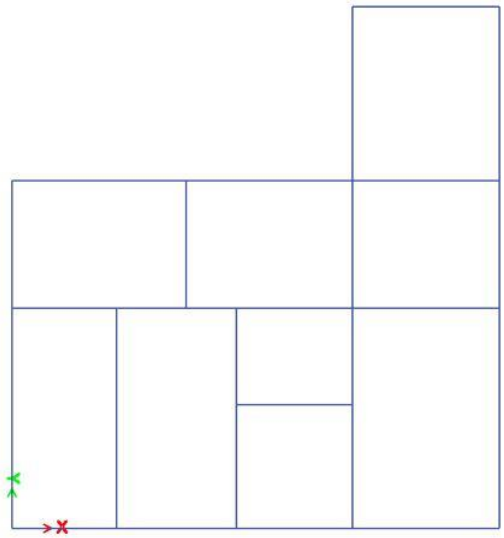
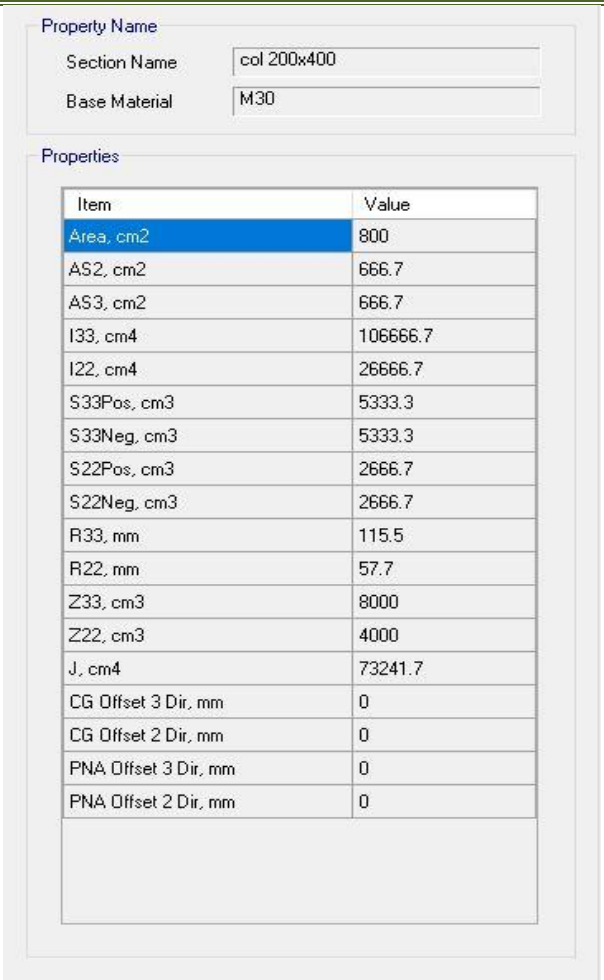


Fig.7.5 Assigned beams

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

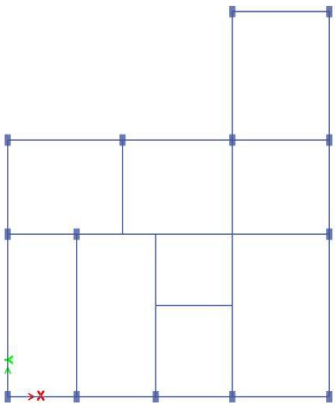


Fig.7.7 Assigned columns

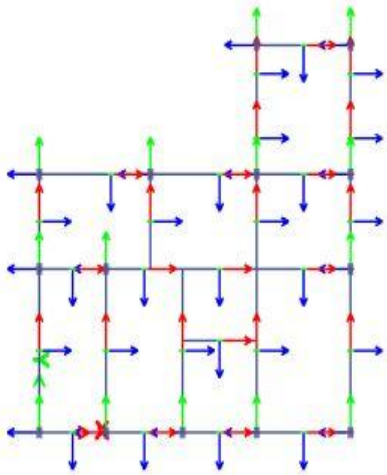


Fig.7.8 Orientation of columns

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	

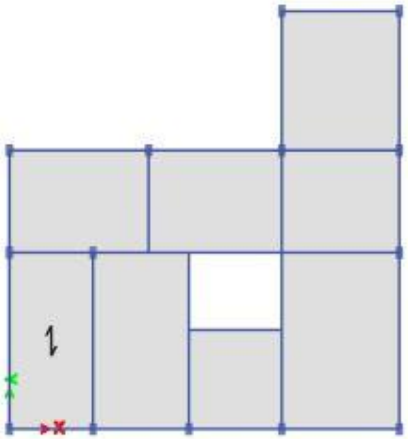


Fig.7.9 Assigned slabs

7.1.5 Defining retaining wall

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

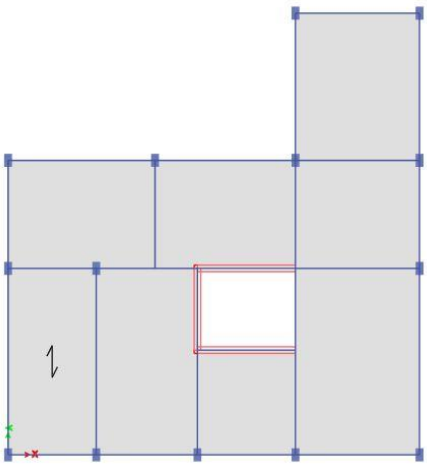


Fig.7.10 Shear wall with beams

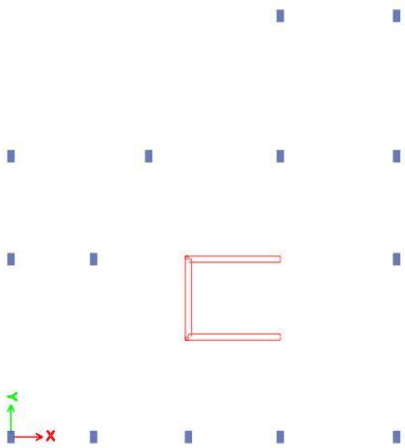


Fig.7.11 Shear wall without beams

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

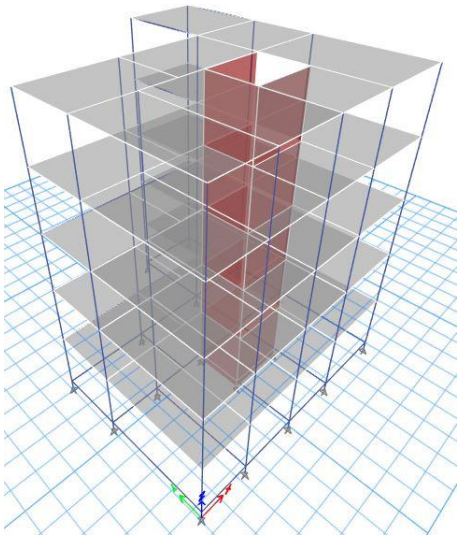


Fig.7.12 3-D view of structure

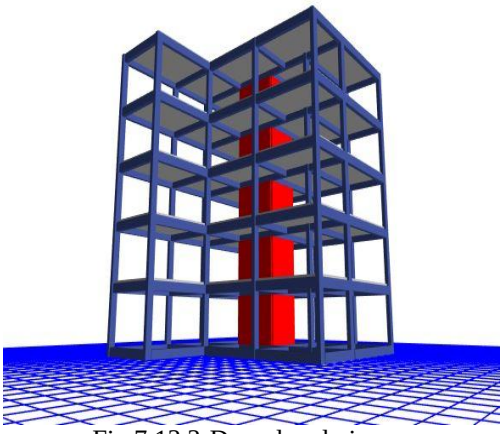


Fig.7.13 3-D rendered view

7.2 Post Tensioned Building

Tendons

- Banded tendons in X direction
  - Group of tendons placed together as a narrow strip and is laid along the column line
- Distributed tendons in Y direction
  - Bundle of tendons laid perpendicular to banded tendons

In addition to the steps in modelling of reinforced concrete building, the following steps have to be done.

7.2.1 Define materials

- Material name - A416Gr270
- Material type – Tendon
- Symmetry - Uniaxial

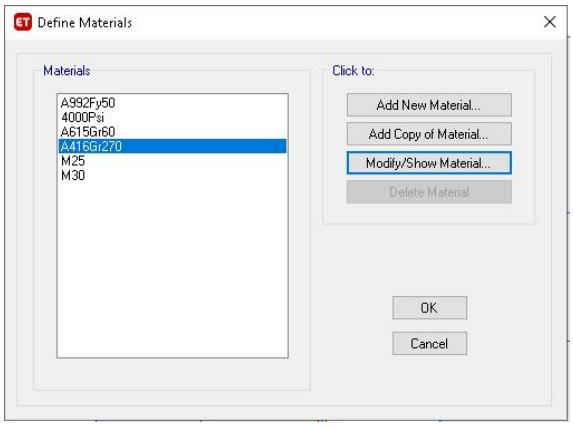


Fig.7.14 Define materials

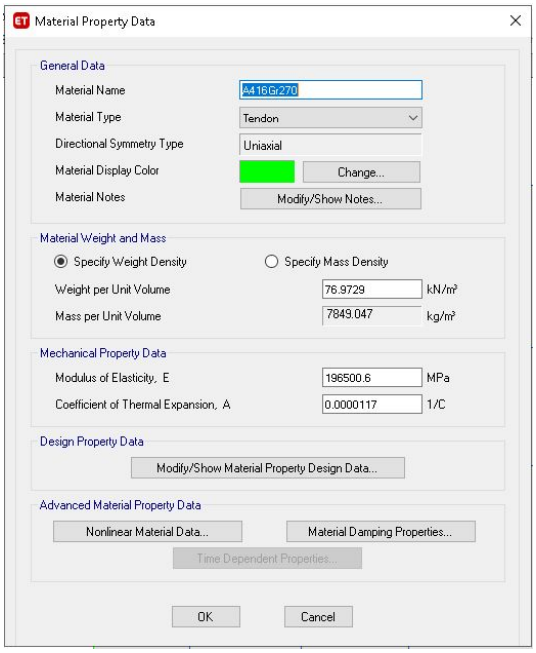


Fig.7.15 Material property data

7.2.2 Tendon properties

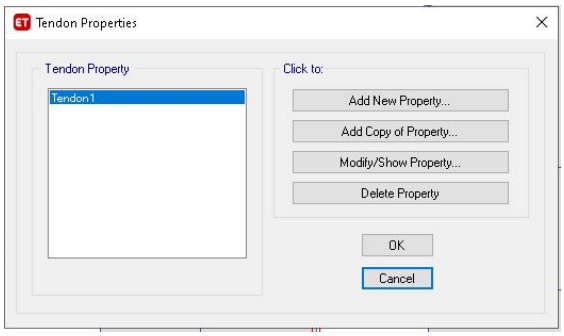


Fig.7.16 Tendon properties

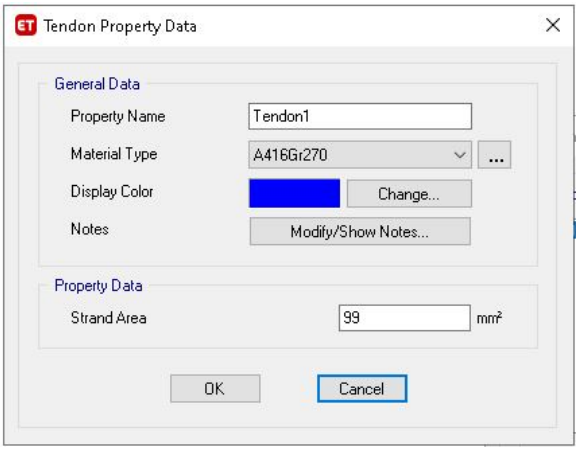


Fig.7.17 Tendon property data

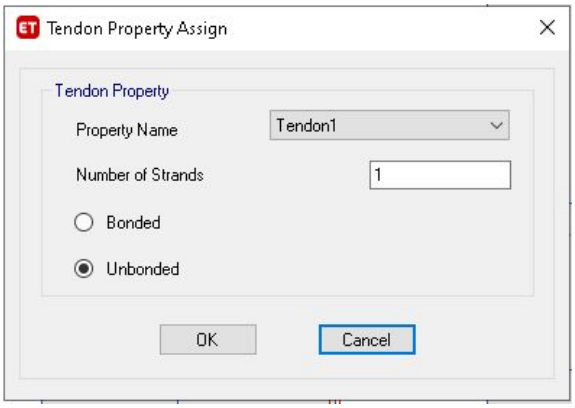


Fig .7.18 Tendon property assign

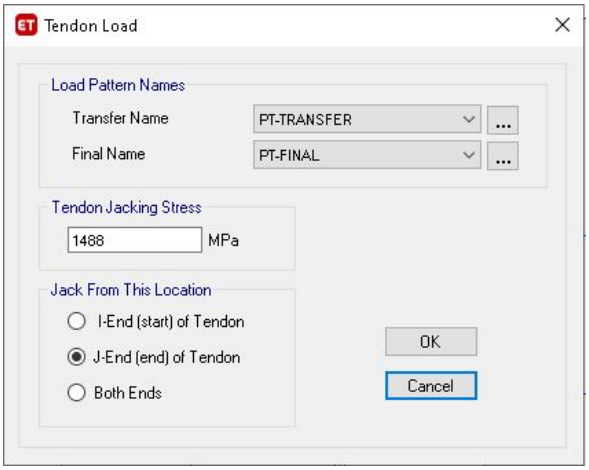


Fig.7.19 Tendon load

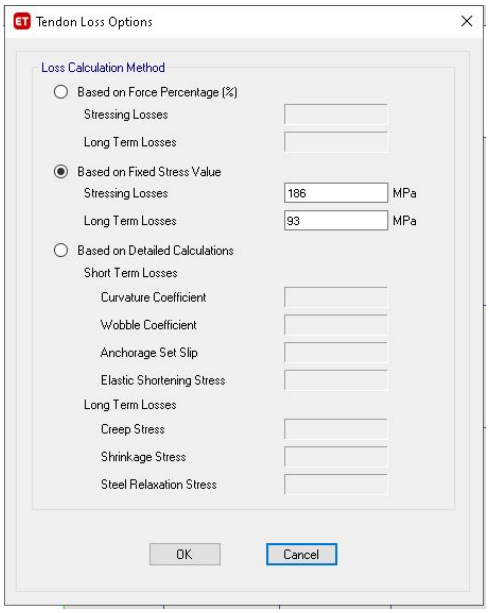


Fig.7.20 Tendon loss option

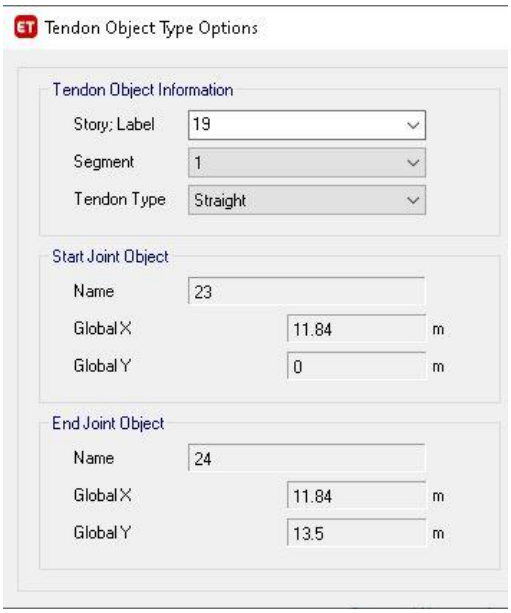


Fig.7.21 Tendon object type

7.2.3 Add Design strips

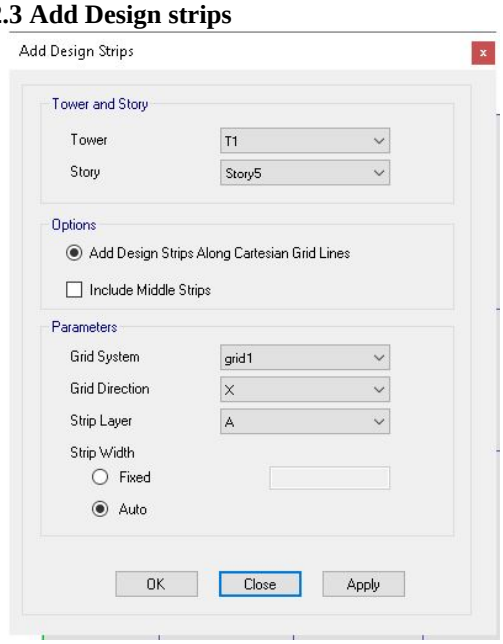


Fig.7.22 Design strips - X

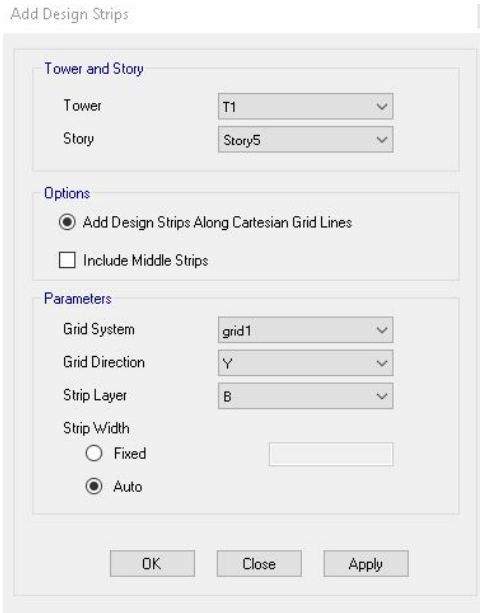


Fig.7.23 Design strips - Y

7.2.4 Tendon layout

(a) Banded tendon

- Vertical profile - Parabola

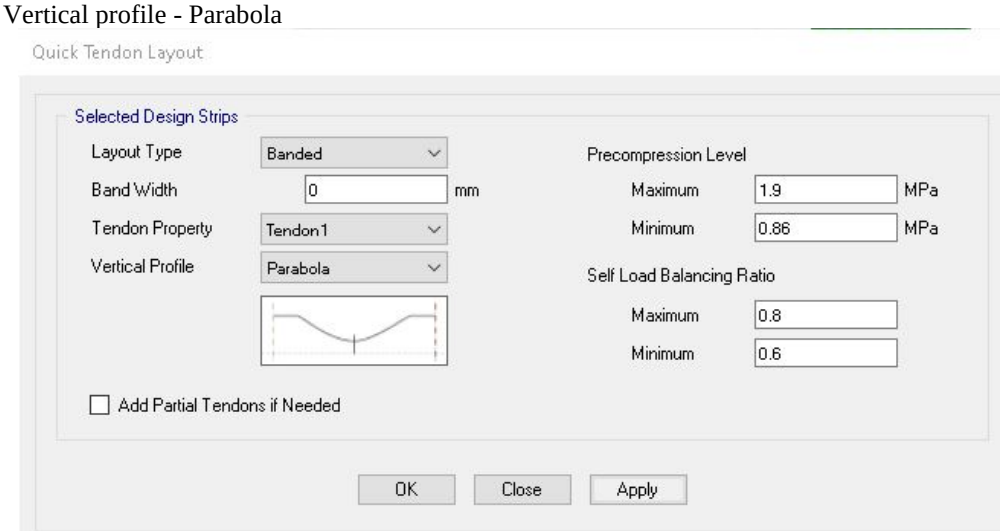
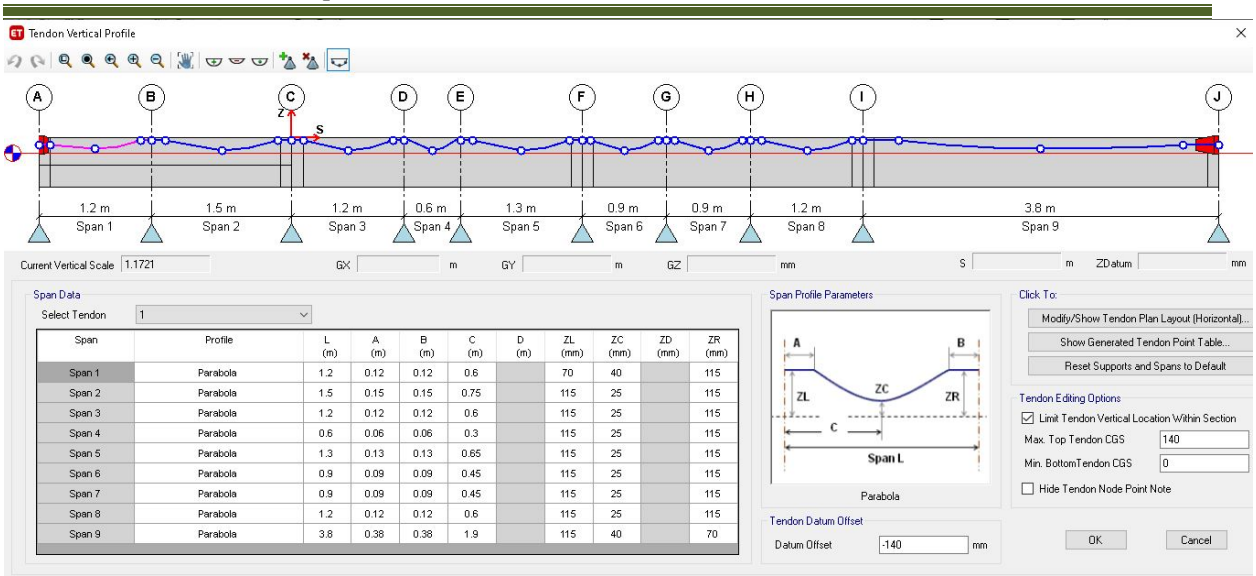


Fig.7.24 Banded tendon layout



Span	Profile	L (m)	A (m)	B (m)	C (m)	D (m)	ZL (mm)	ZC (mm)	ZD (mm)	ZR (mm)
Span 1	Parabola	1.2	0.12	0.12	0.6		70	40		115
Span 2	Parabola	1.5	0.15	0.15	0.75		115	25		115
Span 3	Parabola	1.2	0.12	0.12	0.6		115	25		115
Span 4	Parabola	0.6	0.06	0.06	0.3		115	25		115
Span 5	Parabola	1.3	0.13	0.13	0.65		115	25		115
Span 6	Parabola	0.9	0.09	0.09	0.45		115	25		115
Span 7	Parabola	0.9	0.09	0.09	0.45		115	25		115
Span 8	Parabola	1.2	0.12	0.12	0.6		115	25		115
Span 9	Parabola	3.8	0.38	0.38	1.9		115	40		70

Fig.7.25 Banded tendon vertical profile

(b) Distributed tendon

- Vertical profile – Reverse parabola

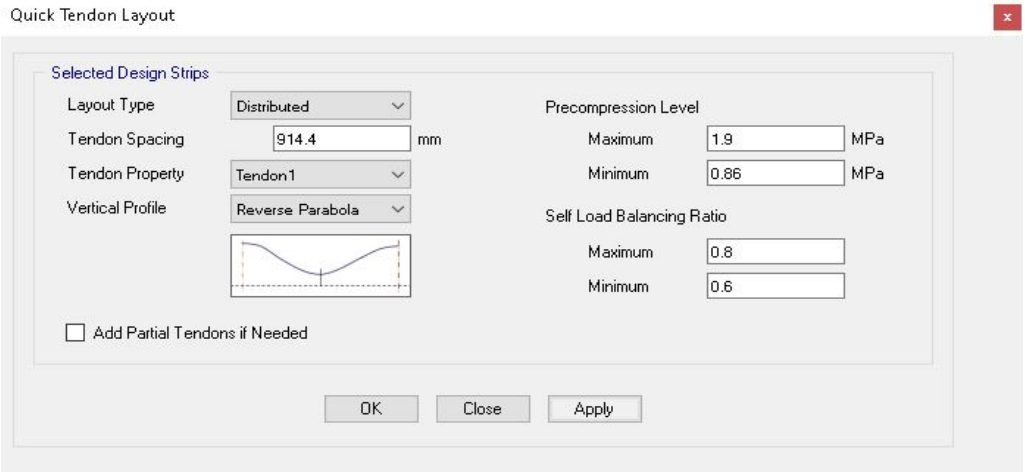
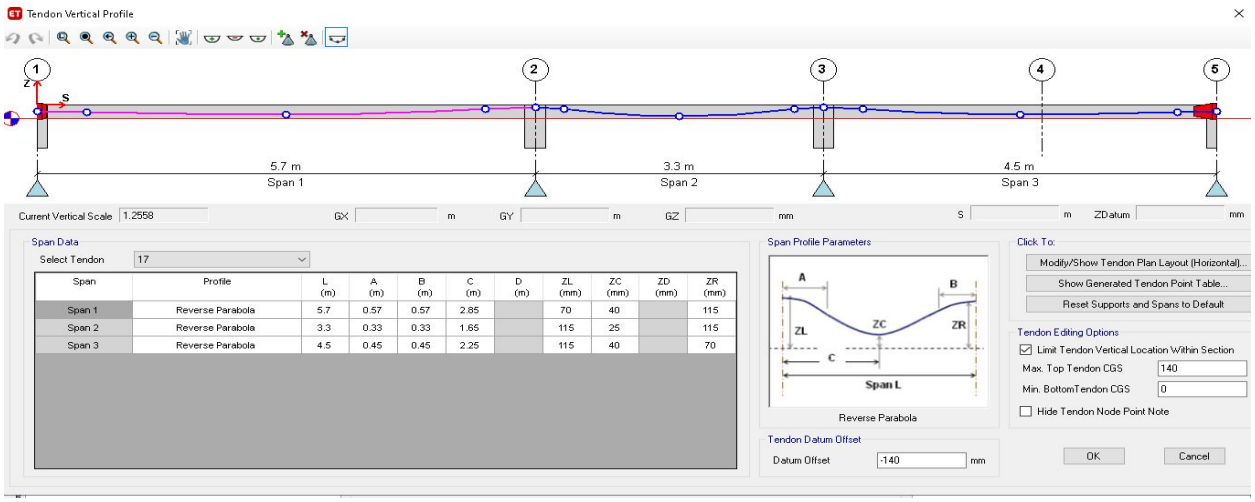


Fig.7.26 Distributed tendon layout



Span	Profile	L (m)	A (m)	B (m)	C (m)	D (m)	ZL (mm)	ZC (mm)	ZD (mm)	ZR (mm)
Span 1	Reverse Parabola	5.7	0.57	0.57	2.85		70	40		115
Span 2	Reverse Parabola	3.3	0.33	0.33	1.65		115	25		115
Span 3	Reverse Parabola	4.5	0.45	0.45	2.25		115	40		70

Fig.7.27 Distributed tendon vertical profile

7.2.5 Design of post tensioned slabs

- Design preferences as per IS 456:2000
- Concrete strength ratio at transfer = 0.8

Fig.7.28 Concrete slab design preferences

Display type is of two types as follows:
(a) Flexural stress check long term

- Design basis – strip based
- Stress type – compressive
- Scale factor – 1
- Plot type – stress diagrams

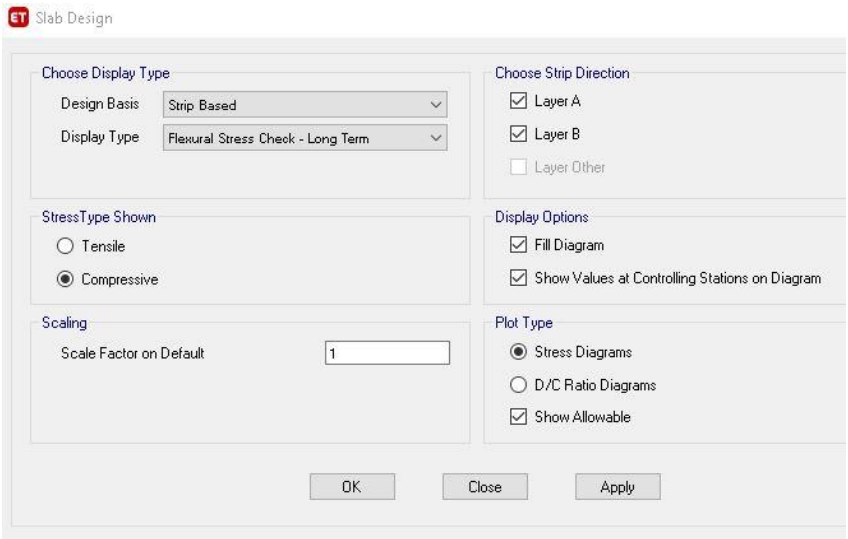


Fig.7.29 Flexural stress check – long term

(b) Enveloping flexural reinforcement

- Design basis – Strip based
- Reinforcing display type – Show rebar intensity
- Scale factor - 1

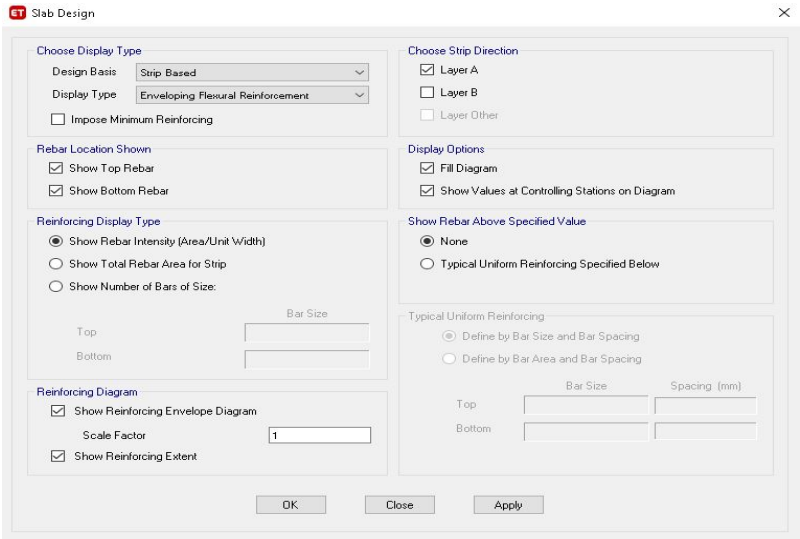


Fig.7.30 Enveloping flexural reinforcement

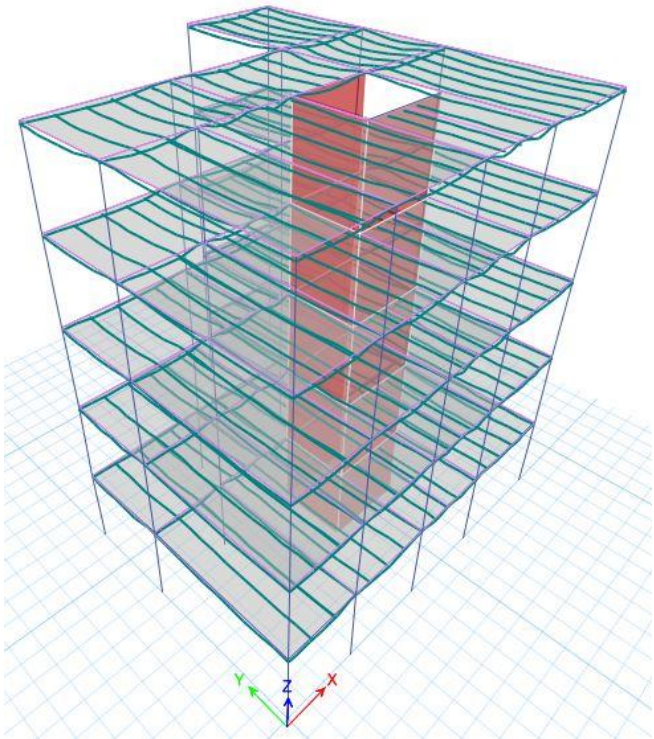


Fig.7.31 3-D View of post tensioned building

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.

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

8.1.1 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.

8.2 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. 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.

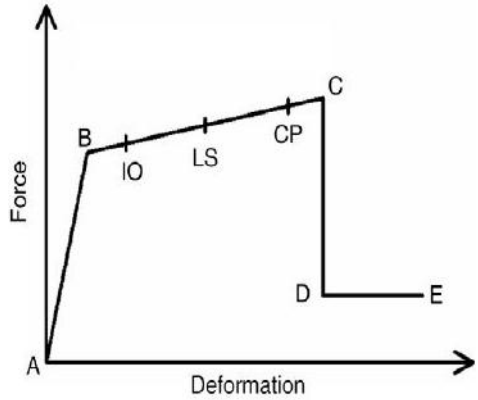


Fig.8.1 Force V/s deformation curve

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.

Push Over In Etabs

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

9.1 Define Load Patterns

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

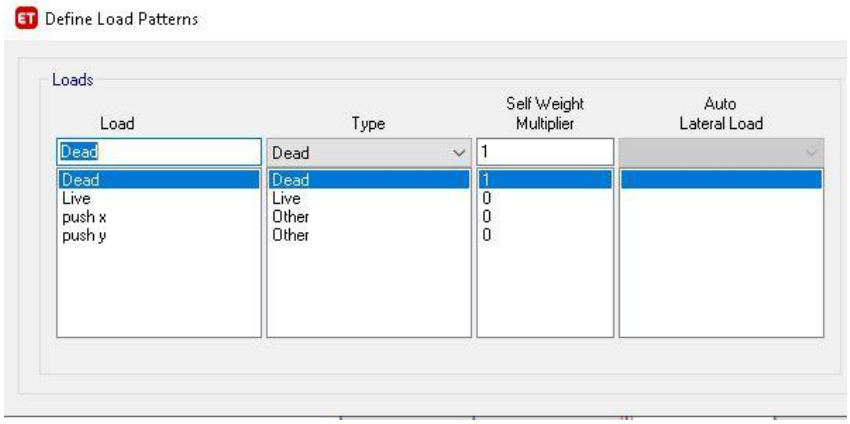


Fig.9.1 Define load patterns – R.C. Building

For post tensioned structure, an extra load i.e super dead load is also defined in addition to the above.

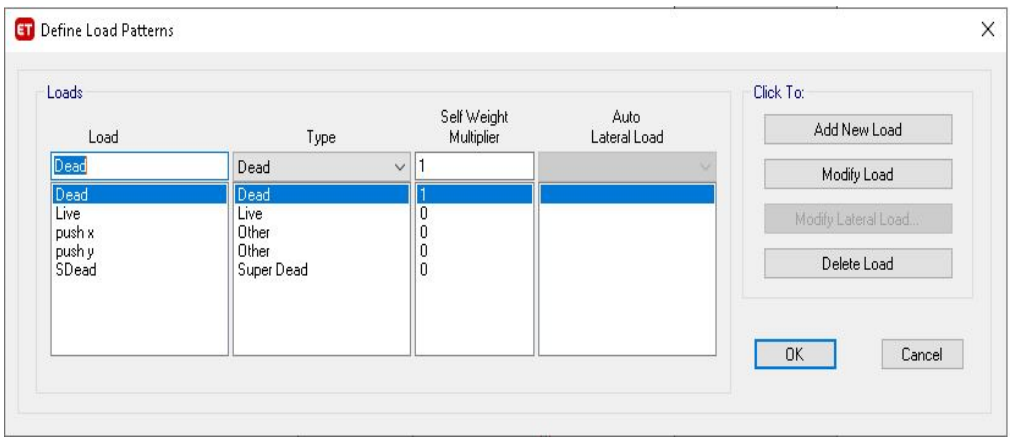


Fig.9.2 Define load patterns – P.T. Building

9.2 Define Load Cases Push X And Push Y

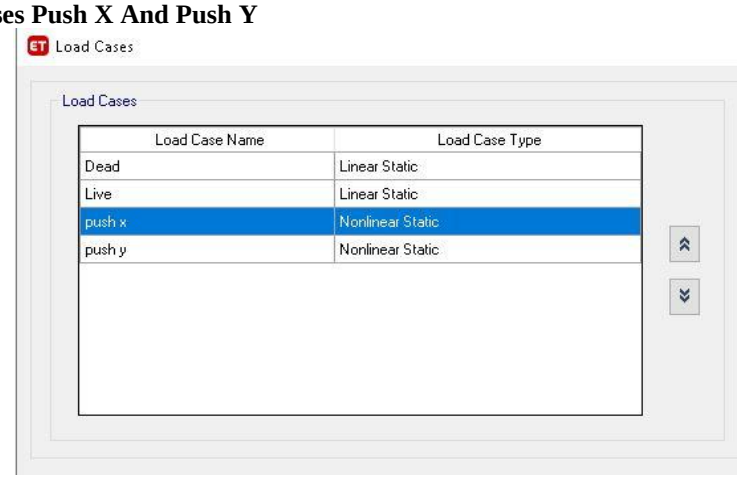


Fig.9.3 Load cases

(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

9.3 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

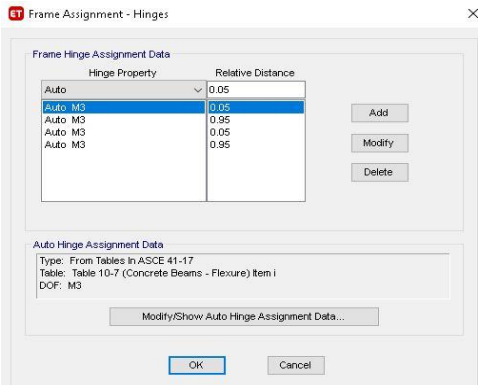


Fig.9.4 Auto hinge assignment in beams

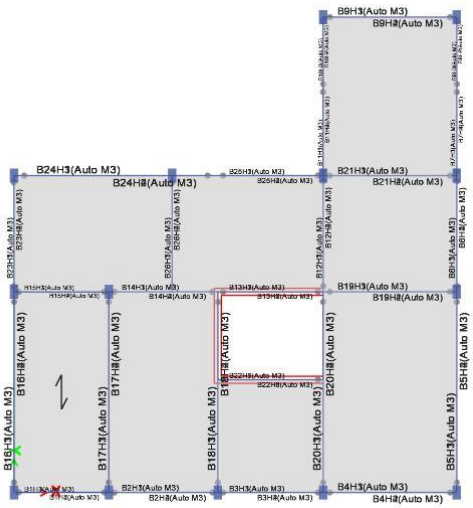


Fig.9.5 Assigned auto hinges in beam

9.4 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

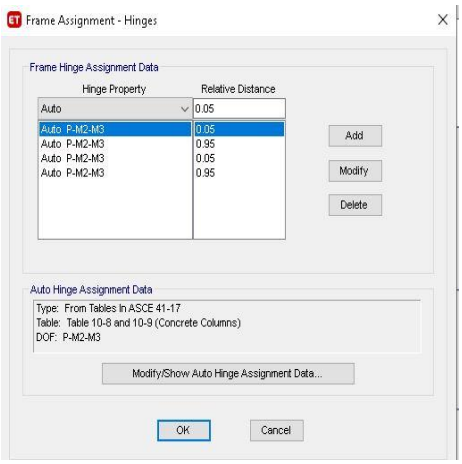


Figure.9.6 Auto hinge assignment in columns

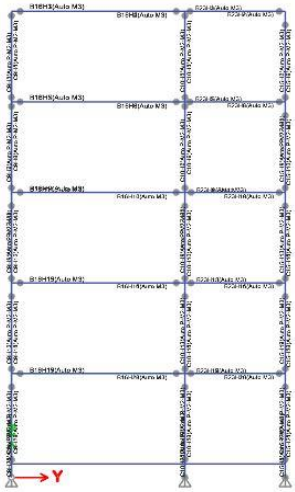


Figure.9.7 Assigned auto hinges in columns

10.5 Run Analysis

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

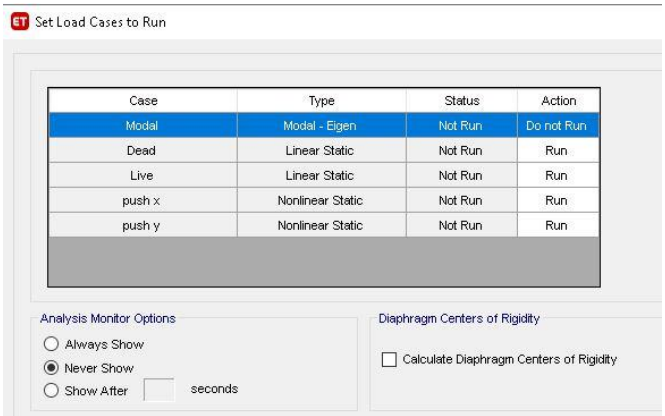


Fig.9.8 Run analysis

Time History Analysis

10.1 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.

10.2 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.
- 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.

10.3 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.

10.3.1 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.10.1 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°WCoordinates
Type	Strike-slip
Affected area	United states, Mexico
Total damage	$6 Million
Max intensity	X, Extreme

Time History Analysis In Etabs

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

11.1 Define Time History Function

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

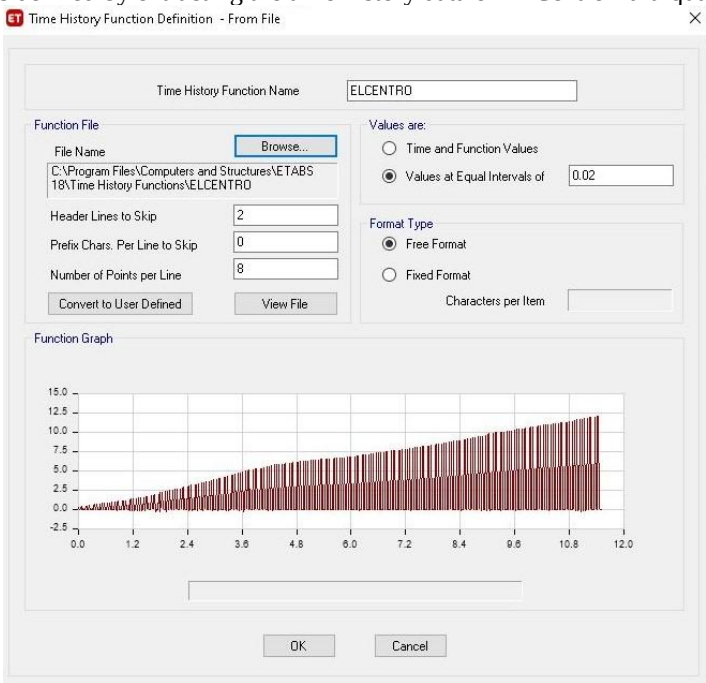


Fig.11.1 Time history function definition

11.2 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

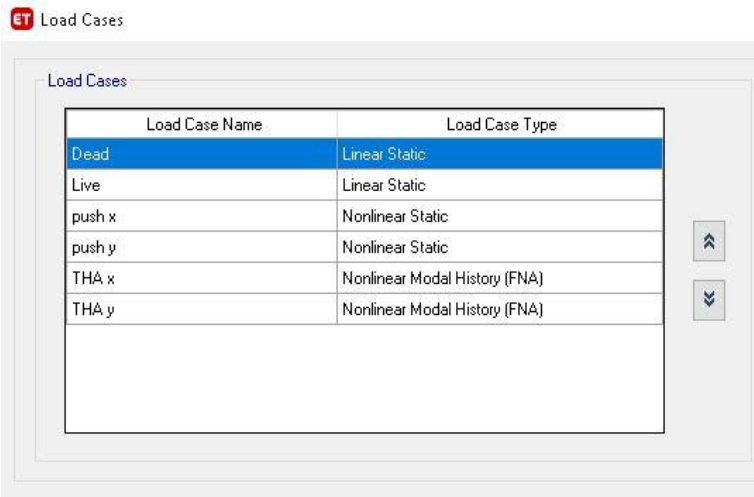


Fig.11.2 Load cases

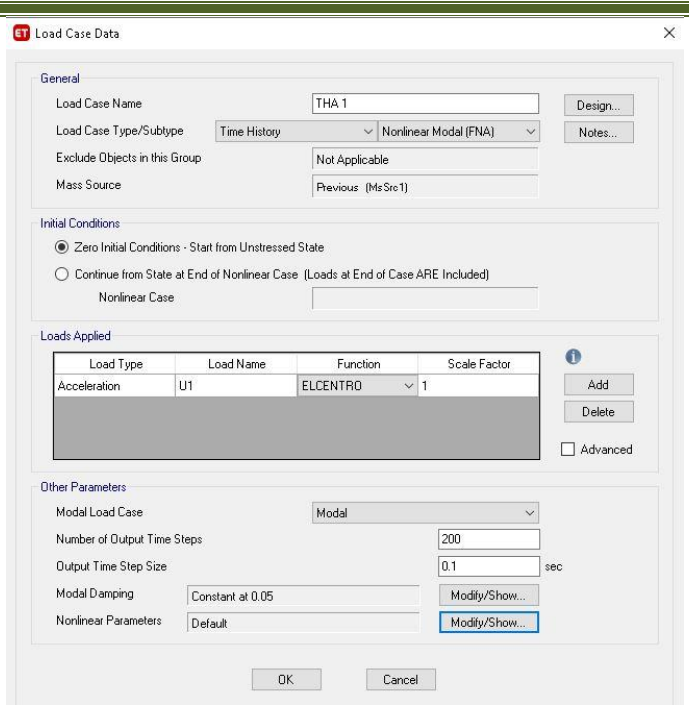


Fig.11.3 Load case data

11.3 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 for R.C. Building.

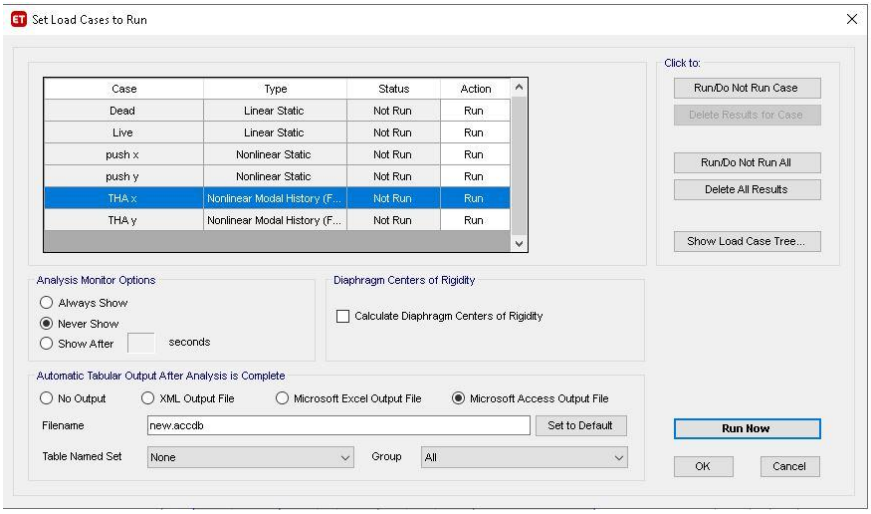


Fig.11.4 Run Analysis – R.C. Structure

In post tensioned structure, in addition to DL, LL, push x, push y, THA x, THA y, Sdead (super dead load), PT- FINAL, PT –TRANSFER and PT – FINAL -HP are set to run.

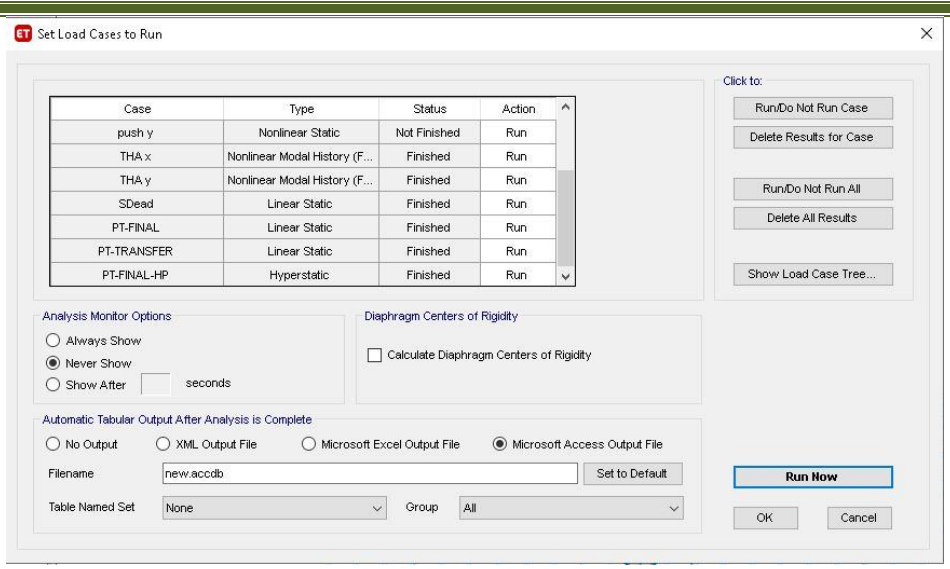


Fig.11.5 Run analysis – P.T. Structure

Results and Discussion

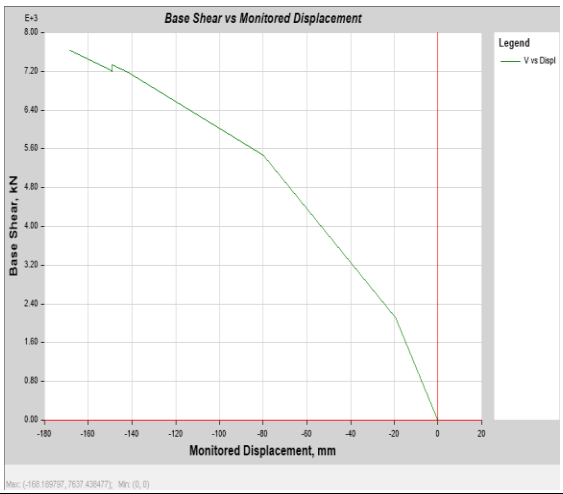
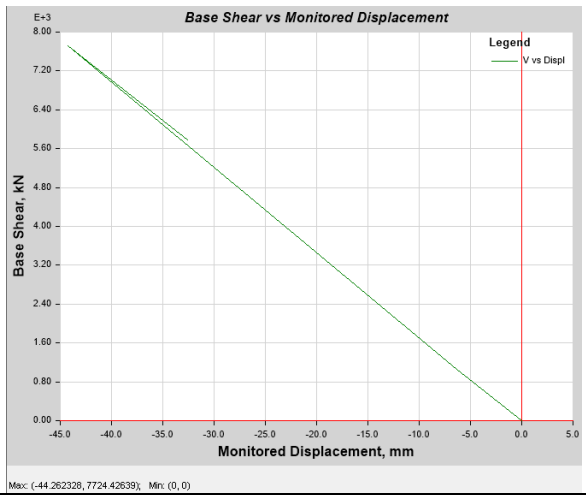
12.1 Pushover Anaysis Results

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.

12.1.1 Pushover curve

It is the plot between base shear (kN) and monitored displacement(mm)

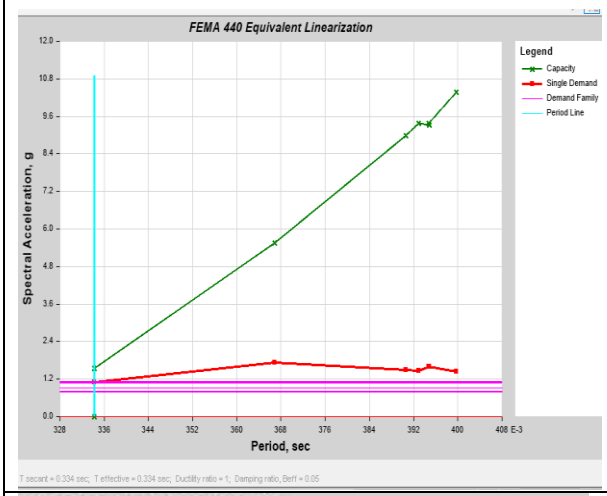
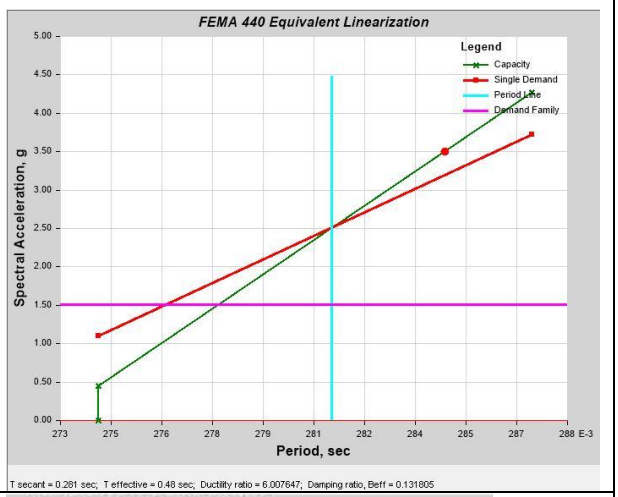
Table.12.1 Displacements

R.C. Building	P.T. Building
	
Monitored displacement = 168.189 mm	Monitored displacement = 44.26 mm

12.1.2 Performance point

The Performance point is the intersection of the demand and capacity curves

Table.12.2 Performance point

R.C. Building	P.T. Building
	
Performance Point Point Found: No	**Performance Point** Point Found: Yes

12.1.3 Effective time period

Shorter time period indicates greater probabilities of collapse [Ref : WCEE2012_5015]

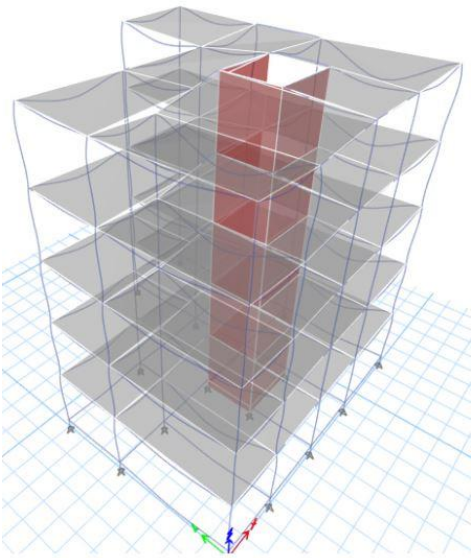
Table.12.3 Effective time period

Building type	Effective time period from FEMA 440 Equivalent linearization	Time period (s)
R.C.	T secant = 0.334 sec; T effective = 0.334 sec; Ductility ratio = 1; Damping ratio, Beff = 0.05	0.334
P.T.	T secant = 0.281 sec; T effective = 0.48 sec; Ductility ratio = 6.007647; Damping ratio, Beff = 0.131805	0.48

12.1.4 Deformed shape under loads

(a) Dead load

R.C. Building



P.T. Building

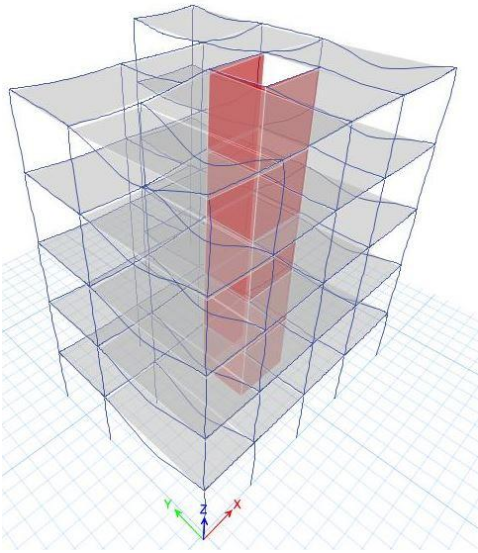


Fig.12.1 Deformed shape under dead loads

(b) Live load

R.C. Building P.T. Building

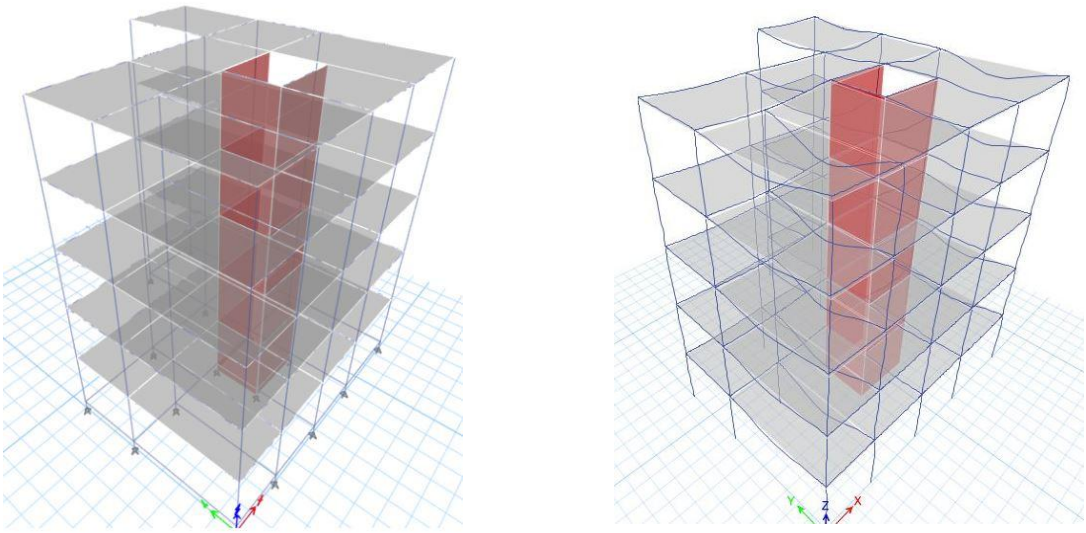


Fig.12.2 Deformed shape under live loads

(c) Push X

R.C. Building P.T. Building

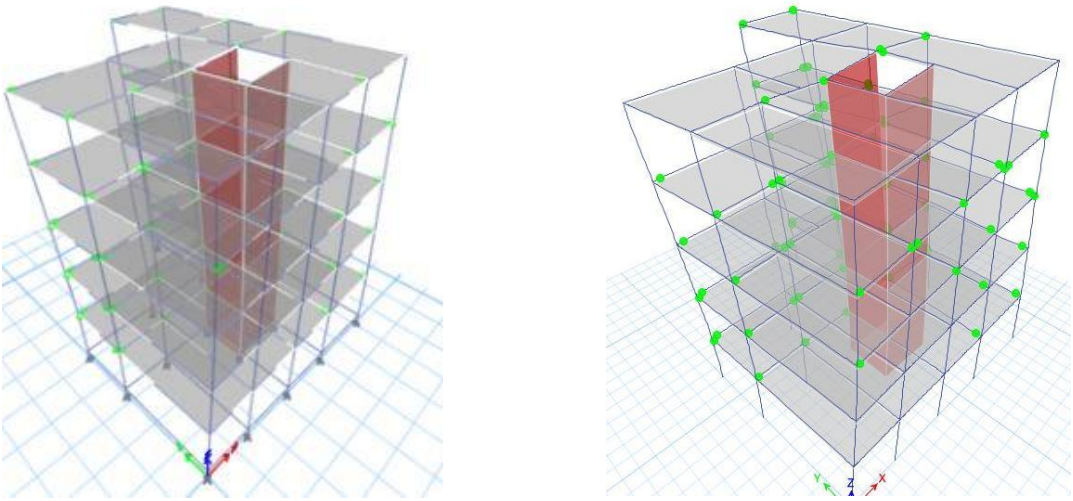


Fig.12.3 Deformed shape under push X loads

(d) Push Y

R.C. Building P.T. Building

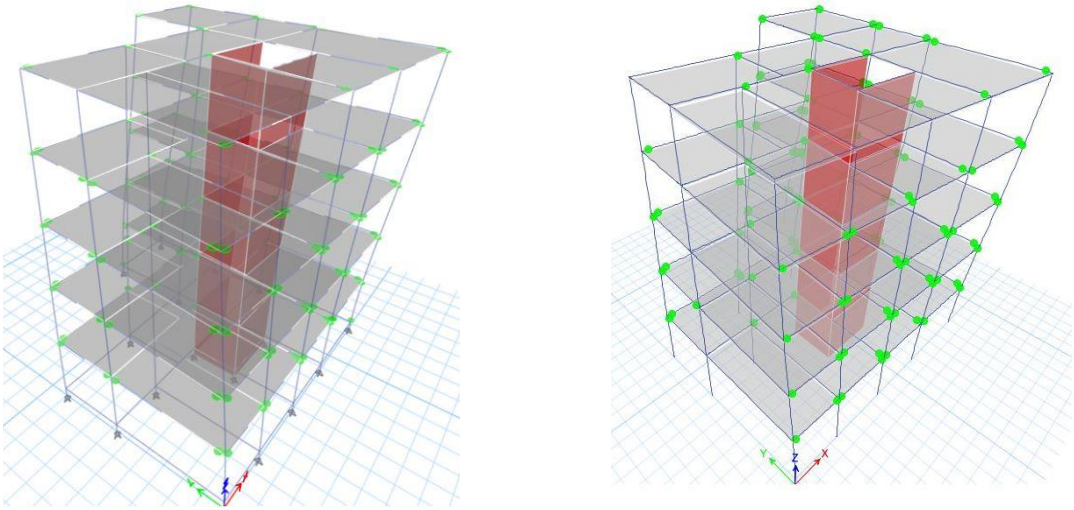


Fig.12.4 Deformed shaped under push Y loads

(e) Tendon loads (P.T. Building)

PT-TRANSFER PT-FINAL

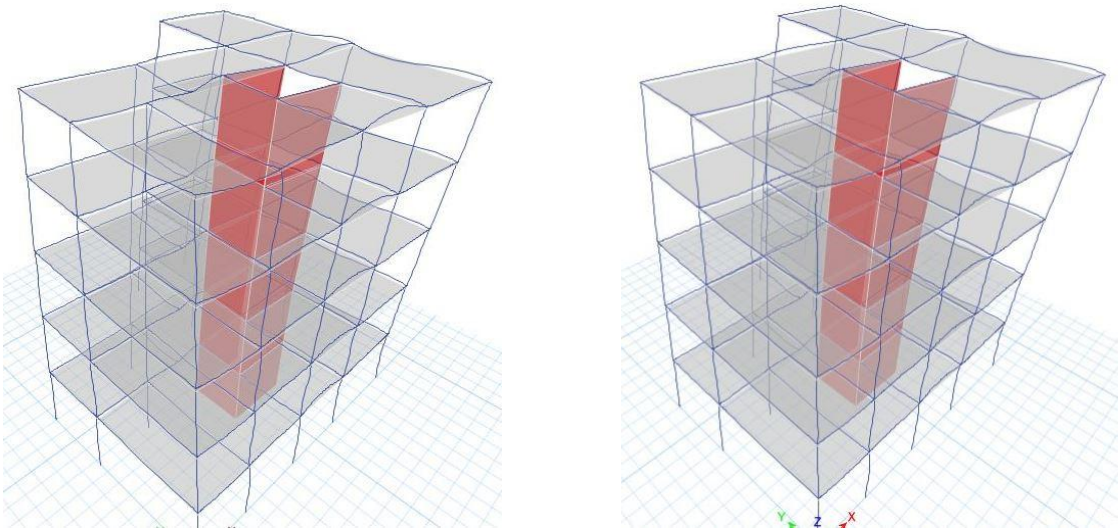


Fig.12.5 Deformed shaped under tendon loads

(f) Super imposed deadload (SDead)

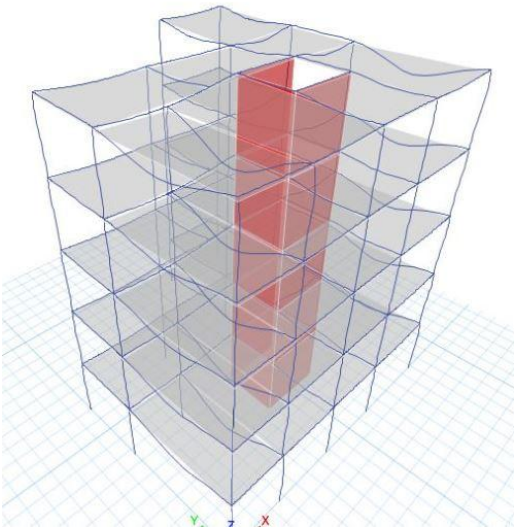


Fig.12.6 Deformed shape under SDead loads

12.1.5 Storey forces

Storey forces are the amount of lateral load acting per storey.

Table.12.4 Storey forces

R.C. Building

Table 5.4 - Story Forces

Story	Output Case	Case Type	Step Type	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story5	push x	NonStatic	Max	Top	0	1621.3492	2.873E-06	0	1.717E-06	0
Story5	push x	NonStatic	Max	Bottom	0	1622.3994	2.876E-06	0	0	4865.3867
Story5	push x	NonStatic	Min	Top	0	0	0	-8543.2071	0	-3.007E-06
Story5	push x	NonStatic	Min	Bottom	0	0	0	-8549.216	-7.188E-06	0
Story5	push y	NonStatic	Max	Top	0	0	1253.2487	8217.0032	0	0
Story5	push y	NonStatic	Max	Bottom	0	0	1254.0605	8222.3669	0	0
Story5	push y	NonStatic	Min	Top	0	-1.953E-06	0	0	0	0
Story5	push y	NonStatic	Min	Bottom	0	-1.957E-06	0	0	-3760.7813	-5.527E-06

P.T. Building

Table 3.3 - Story Forces

Story	Output Case	Case Type	Step Type	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story5	push x	NonStatic	Max	Top	0	1267.4588	7.123E-06	0	8.27E-07	0
Story5	push x	NonStatic	Max	Bottom	0	1268.2801	7.123E-06	0	0	3803.4236
Story5	push x	NonStatic	Min	Top	0	0	0	-6678.4654	0	-1.947E-06
Story5	push x	NonStatic	Min	Bottom	0	0	0	-6683.1641	-2.057E-05	0
Story5	push y	NonStatic	Max	Top	0	0	1005.2047	6590.8693	1.696E-06	0
Story5	push y	NonStatic	Max	Bottom	0	0	1005.856	6595.1725	0	0
Story5	push y	NonStatic	Min	Top	0	-3.462E-06	0	0	0	-2.014E-06
Story5	push y	NonStatic	Min	Bottom	0	-3.461E-06	0	0	-3016.4446	-1.21E-05

12.2 Time History Analysis Results

12.2.1 Modal Periods and frequencies

Table.12.5 Modal periods and frequencies

R.C. Building

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

P.T. Building

Case	Mode	Period sec	Frequency cyc/sec	CircFreq rad/sec	Eigenvalue rad2/sec2
Modal	1	0.706	1.416	8.8986	79.1857
Modal	2	0.348	2.87	18.0329	325.1837
Modal	3	0.324	3.083	19.3718	375.2666
Modal	4	0.215	4.656	29.2559	855.9093
Modal	5	0.115	8.695	54.6314	2984.5947
Modal	6	0.086	11.613	72.9653	5323.9358
Modal	7	0.079	12.614	79.2555	6281.437
Modal	8	0.073	13.724	86.2315	7435.8781
Modal	9	0.065	15.403	96.783	9366.9554
Modal	10	0.057	17.566	110.3724	12182.0658
Modal	11	0.052	19.203	120.6588	14558.5522
Modal	12	0.048	20.837	130.9256	17141.5108

Higher Eigen value represents the load magnitude subject to a constraint on structural weight.

12.2.2 Modal directional factors

Modal directional factors identify the predominance of excitation directions. Each factor is a % which relates its component to the total displacement over the stories

Table.12.6 Modal directional factors

R.C. Building

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

P.T. Building

Case	Mode	Period sec	UX	UY	UZ	RZ
Modal	1	0.706	0.016	0.106	0	0.878
Modal	2	0.348	0.894	0.105	0	0.001
Modal	3	0.324	0.089	0.791	0	0.12
Modal	4	0.215	0.018	0.085	0	0.897
Modal	5	0.115	0.016	0.066	0	0.918
Modal	6	0.086	0.009	0.877	0	0.114
Modal	7	0.079	0.015	0.04	0	0.946
Modal	8	0.073	0.976	0.003	0	0.021
Modal	9	0.065	0.013	0.018	0	0.969
Modal	10	0.057	0.012	0.717	0	0.271
Modal	11	0.052	0.011	0.021	0	0.968
Modal	12	0.048	0.026	0.007	0	0.967

12.2.3 Structure Results

Base reactions are the sum of the vertical force acting at the supports.

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.12.7 Base reactions – R.C. Building

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.

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	686.7241	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.12.8 Modified base reaction – R.C. Building

Base Reactions

File Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

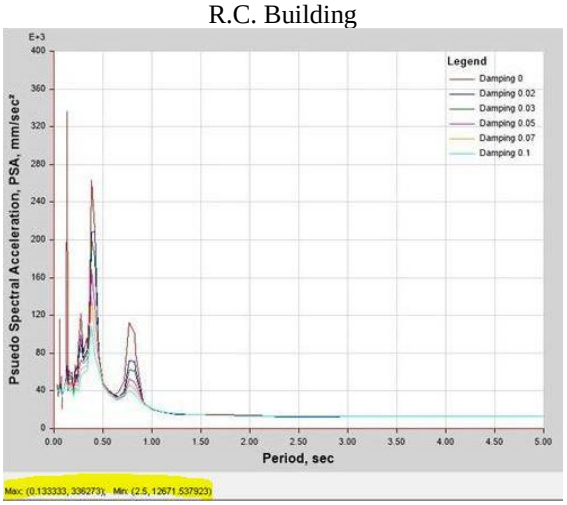
Base Reactions

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	7698.2701	1.678E-05	0	0	65760.8651	0	0
push x	NonStatic	Min	0	0	-3.033E-06	-0.0002	0	-40448.7015	0
push y	NonStatic	Max	0	6105.3954	8.97E-07	0	0	40273.7155	0
push y	NonStatic	Min	-1.301E-05	0	0	-52154.0661	-0.0001	0	0
THA x	NonModHist	Max	8046.8813	709.7414	7.806E-07	7269.8997	67502.1267	15926.6539	0
THA x	NonModHist	Min	-3965.3637	-626.087	-1.564E-06	-8025.2847	-39641.9634	-41940.4939	0
THA y	NonModHist	Max	759.5999	8036.7667	5.907E-07	34020.8131	8980.7514	51616.2949	0
THA y	NonModHist	Min	-698.0137	-4750.9218	-7.266E-07	-68890.7061	-8377.2267	-16930.2179	0
SDead	LinStatic		0	0	891	4696.6952	-6105.5424	-0.0007	0
PT-FINAL	LinStatic		0	0	0	-4.6705	-5.7716	0.1076	0
PT-TRANSFER	LinStatic		0	0	0	-5.0298	-6.2156	0.1159	0

Fig.12.9 Modified Base reactions – P.T. Building

12.2.4 Response spectrum curve

R.C. Building



P.T. Building

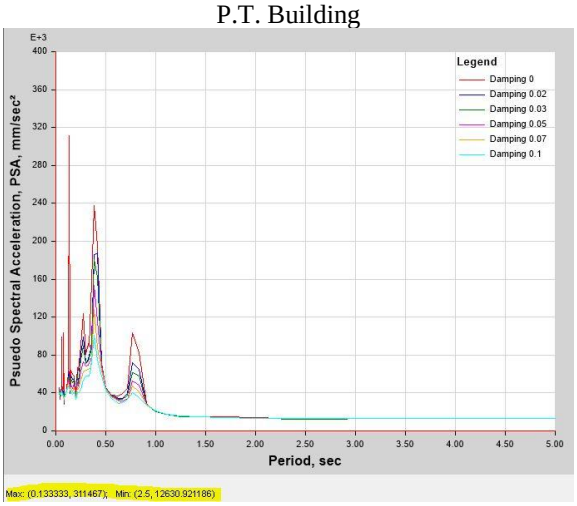


Fig.12.10 Response spectrum curve

From response spectrum curves,
Spectral acceleration - that describes the maximum acceleration in an earthquake on an object (mm/sec^2)

Table.12.7 Spectral acceleration

Building		R.C.	P.T.
Spectral acceleration (mm/sec^2)	Maximum	336273	311467
	Minimum	12671.54	12630.92

12.3 Summary

Table.12.8 Summary

Sl. No.	Parameters	R.C. Building	P.T. Building	% variation	+ / -
1	Monitored displacement	168.189 mm	44.26 mm	73.68	-
2	Performance point found	No	Yes	-	
3	Effective time period	0.336 s	0.48 s	30	+
4	Storey forces	1621.3492 kN	1267.4588 kN	21.8	-
5	Eigen value	57.9921 rad^2/sec^2	79.1857 rad^2/sec^2	26.76	+
6	Modal directional factor	0.019	0.016	15.78	-
7	Base reaction – push x	9849.4588 kN	7698.2701 kN	21.84	-
8	Base reaction – THA x	8372.3662 kN	8046.8813 kN	3.8	-
9	Spectral acceleration max	336273 mm/sec^2	311467 mm/sec^2	7.37	-
10	Spectral acceleration minimum	12671.5379 mm/sec^2	12630.92 mm/sec^2	0.03	-

Conclusion

After studying the curves and tables a conclusion was made that the Post tensioned structures exhibit superior performance than Reinforced concrete structure under the effect of earthquake forces.

As summarized above post tensioned structures has considerable % variation in most of the parameters taken for the study. Push over analysis and time history analysis was conducted on the two models. 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. Pushover curve, response spectrum curve modal results and structure results were obtained from the software.

Monitored displacement obtained from the pushover curve is comparatively much lesser than that of reinforced concrete building. Similarly storey forces, base reactions, spectral acceleration was found to be much

less for P.T. building, proving the superior performance. Effective time period was found to be 30% higher for P.T. Building indicating lesser probability for collapse than R.C. Building.

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