

Lateral load analysis of Post tensioned and RCC building using Pushover analysis

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Abstract - The post-tensioned columns can give higher recentering ability and uncracked section after earthquake vibration effects. In current study, the earthquake behavior of building with and without post-tensioned columns was evaluated by doing pushover analysis. Pushover analysis is the nonlinear static analysis which is becoming a popular tool for seismic performance evaluation of existing and new structures. In this analysis, performance parameters of building were evaluated. Vertical bonded tendons were equally distributed along four sides of the columns. The shopping complex building situated in Pattambi was selected for analysis which lies in seismic zone III. Here corner columns were forced by giving extra vertical PT force up to 50% of its axial load capacity with maximum reinforcement steel up to 4% of the area. The results of the analysis showed that increases in Performance point, maximum base shear, response reduction factor, ductility factor and overstrength factor can be achieved by increasing PT steel and PT force in tendons.

Key Words: pushover analysis, plastic hinges, modified response reduction factor, overstrength factor, ductility factor

1. INTRODUCTION

Recent earthquakes in which many concrete structures have been severely damaged or collapsed, have indicated the need for evaluating the seismic adequacy of existing buildings. About 60% of the land area of our country is susceptible to damaging levels of seismic hazard. We can't avoid future earthquakes, but preparedness and safe building construction practices can certainly reduce the extent of damage and loss. In order to strengthen and resist the buildings for future earthquakes, some procedures have to be adopted. One of the procedures is the static pushover analysis which is becoming a popular tool for seismic performance evaluation of existing and new structures. In particular, the seismic rehabilitation of older concrete structures in high seismic areas is a matter of growing concern, since structures vulnerable to damage must be

identified and an acceptable level of safety must be determined. To make such assessment, simplified linear-elastic methods are not adequate.

Pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements may yield sequentially. Using a pushover analysis, a characteristic nonlinear force displacement relationship can be determined.

The post-tensioning method is now a days increasing widely, due to its superior performance. By using post-tensioning method one can design the most economic and the safe design. Vertical post-tensioning in columns are generally used for precast concrete columns. It is also used for bridge columns to get higher re-centering ability and to get uncracked section after vibration. Sometimes it is used for slender column which undergo higher overturning moments. It is also used for shear walls in tall building to get less rebar quantity and less congestions. This method is applied to general building columns.

1.1 Advantages of post tensioning systems

1. By comparison with reinforced concrete, a considerable saving in concrete and steel since, due to the working of the entire concrete cross-section more slender designs are possible.
2. Smaller deflections compared to with steel and reinforced concrete structures.
3. Good crack behaviour and therefore permanent protection of the steel against corrosion.

2. SCOPE& OBJECTIVE

A. Scope of the Study

Performance parameters such as overstrength factor, response reduction factor was not explained or evaluated in detail in the past studies. The importance of these parameters has to be studied. So it is necessary to evaluate and compare the performance of post tensioned and RCC building using nonlinear static analysis to assess the damage experienced by the structure and to determine the crack pattern, yielding behavior of the structures for different values of prestressing force and prestressing reinforcement.

B.Objectives of the Study

- 1.To study and understand pushover analysis.
- 2.To analyse an RCC and a post tensioned framed structure under pushover technique.
- 3.To compare the values of pushover analysis for RCC and post tensioned framed structure in terms of base shear, number of hinges, ductility factor,over strength factor and response reduction factor.
4. To determine the area of PT reinforcement required for better performance of selected building.
5. To determine the optimum amount of prestressing force required for better performance of selected building.
6. To suggest the effective model having superior seismic performance under pushover analysis.

III.METHODOLOGY

The different static nonlinear procedures used are as follows:

- a. ATC 40 - Capacity Spectrum Method(CSM)
- b. FEMA 356- Displacement Coefficient Method (DCM)
- c. FEMA 440 - Equivalent Linearization - Modified CSM
- d. FEMA 440 - Displacement Modification- Improvement for DCM

a. ATC 40 - Capacity Spectrum Method (CSM) -This method is based on the equivalent linearization of a nonlinear system. The important assumption here is that inelastic displacement of a nonlinear SDF system will be approximately equal to the maximum elastic displacement of linear SDF system with natural time period and damping values greater than the initial values for those in nonlinear system. ATC 40 describes three procedures (A, B and C) for the CSM and the second one is used in this study.

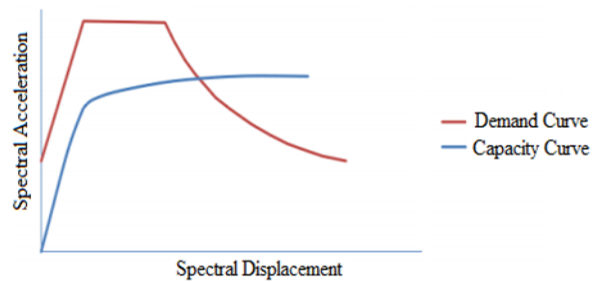


Fig- 1:Combination of Demand and Capacity curve

The capacity spectrum method is a nonlinear static method to predict the structural damage that a building of a particular capacity will be developing under a particular earthquake loading. In order to estimate the performance, both the capacity and the demand curve must be converted from their own domain to a common domain. i.e, Spectral acceleration and spectral displacement domain(S_a - S_d).

3.1 Conversion of demand curve (response spectrum) to

S_a - S_d

Every point of the demand curve in the format spectral acceleration -time period (S_a - T) should be converted to acceleration displacement demand spectrum (ADRS) by means of following equation.

$$S_d = S_a \times g / \left(\frac{2\pi}{T}\right)^2 \quad \text{Where } g$$

is the acceleration due to gravity, T is the time period in seconds, S_a is the spectral acceleration and S_d is the spectral displacement.

3.2 Conversion of capacity curve (pushover curve) to S_a - S_d

In order to produce the capacity spectrum from the capacity curve, conversion should be done to first mode spectral coordinates. It is converted based on the following equation.

$$S_{ai} = \left(\frac{V_i}{W}\right) / \alpha_1$$

$$S_{di} = \delta_t / (PF_1 \times \phi_{1,t})$$

Where,

S_{ai} =Spectral acceleration at the i^{th} point

S_{di} =Spectral displacement at the i^{th} point

α_1 =Modal mass coefficient for the first natural mode of the structure.

PF_1 =Participation factor for the first natural mode of the structure.

$\phi_{1,t}$ = Roof level amplitude for the first mode

V_i =Base shear at the i^{th} point

δ_i =Roof displacement at the i^{th} point

3.3 Stages in Pushover Analysis

The pushover analysis consists of the several stages listed below.

1. Defining non-linear hinges in a structure calculation model.
2. Assigning non-linear properties to the hinges (force-displacement or moment-rotation diagrams).
3. Performing structure modal analysis to determine one mode.
4. Defining a set of lateral forces (dependent on the mass matrix type used in the modal analysis).
5. Defining a control node as well as direction and value of the allowable displacement.
6. Determining parameters of the non-linear analysis.
7. Starting the non-linear analysis. The analysis result is the equilibrium state curve $V = V(D)$ where the shear forces are defined as the sum of reactions for a given direction caused by the appropriate set of lateral forces.
8. Determining the capacity curve $S_a^{cap} = S_a^{cap}(S_d)$, where S_a^{cap} is spectral acceleration and S_d is spectral displacement.
9. Smoothing of the capacity curve.
10. Determining the performance point.

IV. MODELING AND ANALYSIS OF THE STRUCTURE

M30 grade of concrete and Fe 415 grade of reinforcing steel are used for all members of the frame structures.

Two structures representing post tensioned and reinforced concrete framed buildings are considered in this study. For the present study, an existing 8 storied post tensioned building located in Pattambi is chosen. These structures are designed according to Indian Standards.

The properties of the considered building configurations in the present study are summarized below:

1. Size of building = 30 m X 38 m (plan)
2. Floor to floor height = 3.0m.
3. Thickness of slab = 0.25 m.
4. Dead load= 4kN/m²

5. Live load= 4kN/m²

6. Modulus of Elasticity (E_c)
 $= 5000\sqrt{f_{ck}} = 5000 \times \sqrt{30} = 27386128 \text{ kN/m}^2$

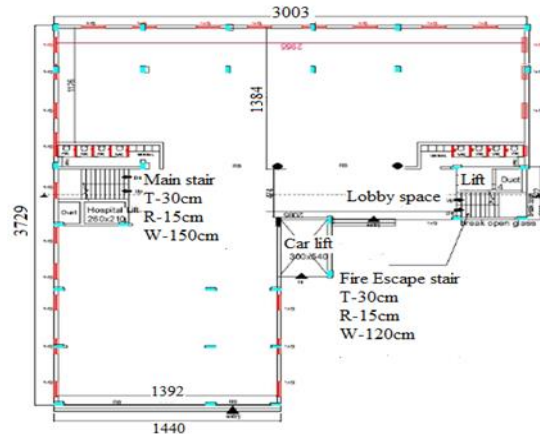


Fig-2: Plan of building model

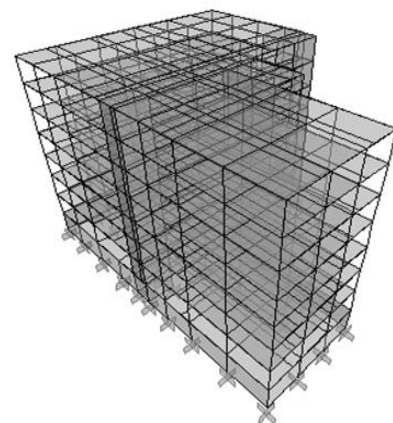


Fig -3: Isometric view of RCC model

The general finite element package SAP 2000 (Version.14) has been used for the analyses. A three dimensional model of each structure has been created to undertake the nonlinear analysis. Beams and columns are modelled as nonlinear frame elements with lumped plasticity at the start and the end of each element. SAP 2000 provides default hinge properties and recommends M3 hinges for columns and M3 hinges for beams as described in FEMA 356.

Total sections of columns provided are given below.

1. Rectangular column of size 800mm×240mm
2. Rectangular column of size 600mm×240mm
3. Circular column of diameter 600mm

Edge beam of size 450mm×250mm was provided along the perimeter of the structure. Reinforcement in pile, pile cap, beam and column sections for the structure are calculated using both gravity load and earthquake loads with M30 concrete and Fe415 steel reinforcement.

For the post tensioned building, tendons with different area and different force are provided. In this project, bonded tendons as elements are provided. PT loss parameters are assigned as per IS 1343. Here maximum 4% reinforcement steel is provided in columns. So axial capacity becomes 4876 kN as per IS 456:2000 design formula.

$$P_u = 0.45 \times f_{ck} \times A_c + 0.67 \times f_y \times A_{SC}$$

Where P_u =Axial load on member, f_{ck} =Characteristic compressive strength of concrete, A_c =Area of concrete, f_y = Characteristic strength of compressive reinforcement, A_{SC} =Area of longitudinal reinforcement for columns. So half of column capacity becomes 2438 kN and extra PT force can be applied upto this force limit. Here maximum allowable tensile stress in tendon is 1720 N/mm².

In this analysis to determine the effect of PT on columns 0.4%, 0.8%, 1.2% and 1.6% PT reinforcement is provided of column cross sectional area, so the area becomes 810, 1620, 2430 and 3240mm² respectively. Area of each tendon 202.5, 405, 607.5 and 810 mm² considering four tendons in each column. Typically, a case name PT 0.4 0 indicates a column with 4 PT tendons having 0.4% of cross sectional area as PT tendons stressed upto 0% of their capacity. Similarly, PT 0.8 30 indicates column having 4 PT tendons with total 0.8% PT reinforcement stretched to 30% of their capacity. Different cases for pushover analysis are given in table 1 and table 2. Figure 2 and 4 shows the typical plan and geometric configuration of frame with bonded PT tendons.

Table -1:Different cases for pushover analysis

Case name	% of force to its ultimate capacity of tendon	Force in each tendon	Total force by tendon	% of extra compressive force on column
	%	kN	kN	%
No PT(RCC)	0	0	0	0
PT 0.4 0	0	0	0	0
PT 0.4 10	10	33.02	132.09	2.70
PT 0.4 20	20	66.04	264.19	5.41
PT 0.4 30	30	99.07	396.28	8.12
PT 0.4 40	40	132.09	528.38	10.83
PT 0.4 50	50	165.12	660.48	13.54
PT 0.4 60	60	198.14	792.576	16.25
PT 0.4 70	70	231.16	924.67	18.96
PT 0.4 80	80	264.19	1056.76	21.67
PT 0.4 90	90	297.21	1188.86	24.38
PT 0.4 100	100	330.24	1320.96	27.09
PT 0.8 0	0	0	0	0
PT 0.8 10	10	66.04	264.19	5.41
PT 0.8 20	20	132.09	528.38	10.83
PT 0.8 30	30	198.14	792.57	16.25
PT 0.8 40	40	264.19	1056.76	21.67
PT 0.8 50	50	330.24	1320.96	27.09
PT 0.8 60	60	396.28	1585.15	32.50
PT 0.8 70	70	462.33	1849.34	37.92
PT 0.8 80	80	528.38	2113.53	43.34
PT 0.9 90	90	594.43	2377.72	48.76

V.RESULTS

The selected building was analyzed using pushover analysis. Pushover analysis was performed first by considering response spectrum analysis for defining gravity load case and then a lateral pushover analysis was performed in a displacement control manner.

From the following curves, the performance point i.e. the point at which capacity curve and demand curve intersected is near to the event point B. In fig.5 and fig.6, green colour curve shows the capacity curve and red colour curve shows demand capacity curve.

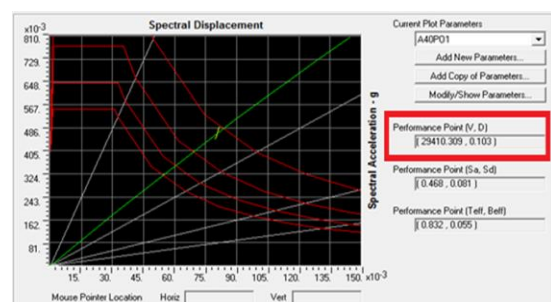


Fig -4: Demand capacity curve for RCC building

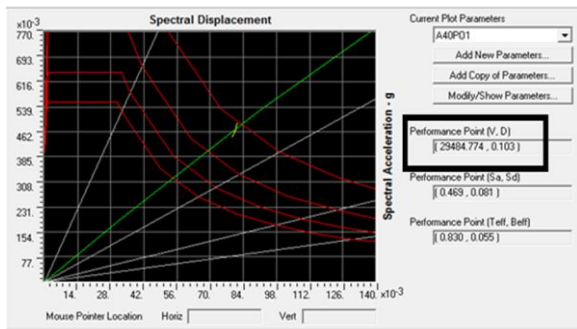


Fig -5: Demand capacity curve for PT IV 0.4 30%

5.1 Plastic Hinge Mechanism

Plastic hinge formations for the RCC building and post tensioned building mechanisms have been obtained at different displacements levels.

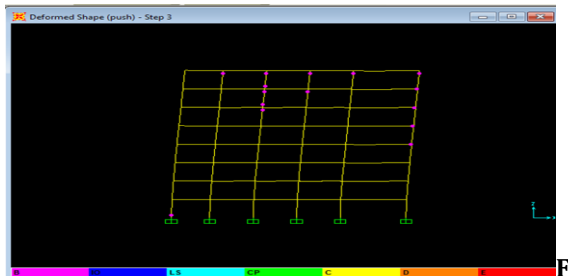


Fig-6: Hinge conditions at performance point for RCC building

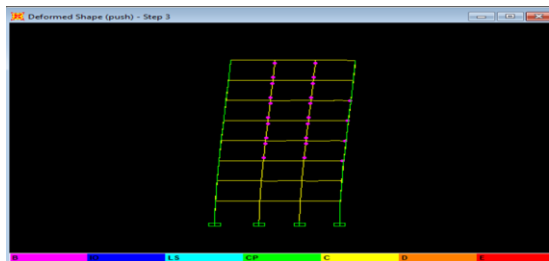


Fig-7: Hinge conditions at performance point for PT IV 0.4 30%

Table- 2:Variation of base shear, no. of hinges, response reduction factor, ductility factor and overstrength factor for RCC and Post tensioning buildings (PT 0.4 30%)

Case Name	Base shear at performance point	Number of hinges at performance point	Response reduction factor	Ductility factor	Overstrength factor
RCC	29410	761	2.931	2.817	1.041
PT	29767	722	9.640	7.857	1.227

Table-3:Variation of base shear, no. of hinges,response reduction factor, ductility factor and over strength factor for PT 0.4

Case Name	Base shear at performance point	Number of hinges at performance point	Response reduction factor	Ductility factor	Overstrength factor
PT 0.4 0	29484	761	7.574	6.743	1.123
PT 0.4 10	29484	761	7.574	6.743	1.123
PT 0.4 20	29484	761	7.574	6.743	1.123
PT 0.4 30	29484	761	7.574	6.743	1.123
PT 0.4 40	29484	761	7.574	6.743	1.123
PT 0.4 50	29484	761	7.574	6.743	1.123
PT 0.4 60	29484	761	7.574	6.743	1.123
PT 0.4 70	29484	761	7.574	6.743	1.123
PT 0.4 80	29484	761	7.574	6.743	1.123
PT 0.4 90	29484	761	7.574	6.743	1.123
PT 0.4 100	29484	761	7.574	6.743	1.123

Table-4:Variation of base shear, no. of hinges,response reduction factor, ductility factor and over strength factor for PT 0.8

Case Name	Base shear at performance point	Number of hinges at performance point	Response reduction factor	Ductility factor	Overstrength factor
PT 0.8 0	29468	786	7.495	7.495	1.052
PT 0.8 10	29504	748	7.461	7.461	1.060
PT 0.8 20	29489	754	7.427	7.427	1.069
PT 0.8 30	29540	786	7.394	7.394	1.078
PT 0.8 40	29594	792	7.361	7.361	1.086
PT 0.8 50	29612	811	7.328	7.328	1.095
PT 0.8 60	29656	881	7.295	7.295	1.104
PT 0.8 70	29677	919	7.262	7.262	1.113
PT 0.8 80	29711	815	7.229	7.229	1.122

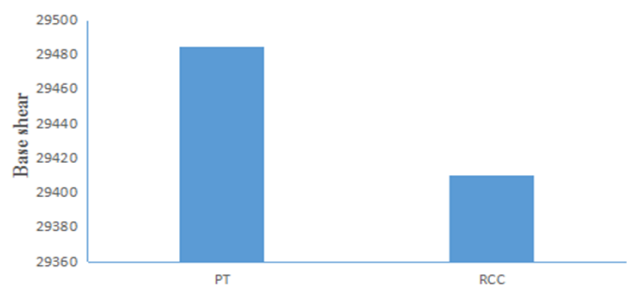


Fig -8:Base shear variation at performance point

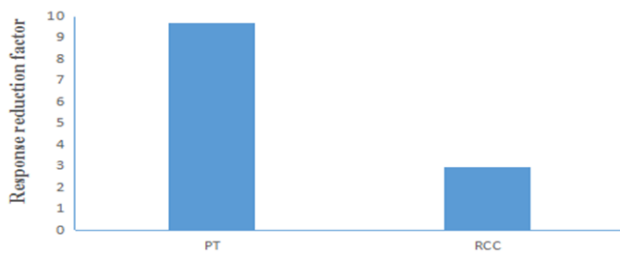


Fig-9:Response reduction factor variation at performance point

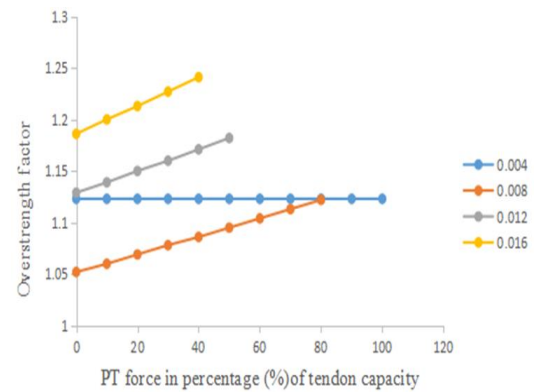


Fig-13 : Overstrength factor

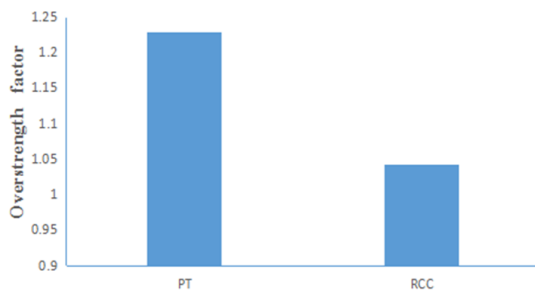


Fig-10:Over strength factor variation at performance point

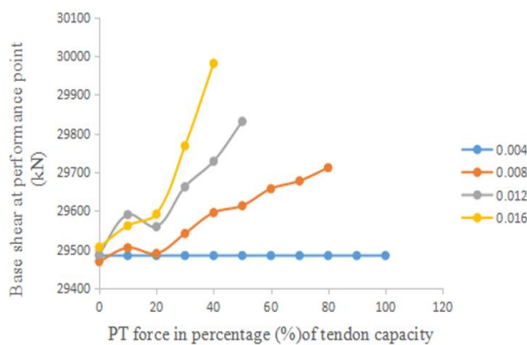


Fig-11: Base shear at performance point

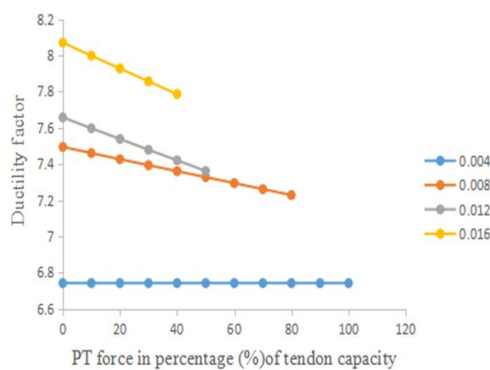


Fig-12: Ductility factor

From fig.16,fig.17 Modified response reduction factor increases as the amount of post tensioning reinforcement increases. But as the stretching of tendons increases, the response reduction factor decreases. The ductility factor (R_{μ}) becomes almost 2 times on introducing post tensioning tendons to that of RCC building. Ductility factor and over strength factor increases as the amount of PT reinforcement increases. 0.4% PT reinforcement in column has no change in R value, ductility factor and over strength factor with increase in PT force.

VI.CONCLUSIONS

1. Provision of a post tensioning tendons in corner columns of a structure influences the seismic performance of the structure with reference to base shear and number of hinges. Structure with post tensioning columns at corners performs better and the base shear increased by 1.2% when compared to that of the RCC frame. The number of hinges formed in the immediate occupancy level decreased in the post tensioned frame to that of RCC building by 5.4%.
2. The over strength factor increased by 1.17% in post tensioned building compared to RCC building. The ductility factor increased by 2.78% in post tensioned building compared to RCC building. The modified response reduction factor for post tensioned frame is 3.28 times that of RCC frame.
3. We can determine that number of hinges will decrease initially at performance point and later increases with increase in PT force. When PT force is applied beyond 30% of tendon stretching there is no significance.
4. It was determined that response reduction factor increases with increase in the amount of PT steel and PT force to column. But as the stretching of tendons increases, the ductility factor decreases.
5. The ductility factor (R_{μ}) becomes almost 2 times on introducing PT tendons as compared to that with no PT

tendons. The overstrength factor and ductility factor was increasing with increase of post tensioning area of tendons.

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