STUDY OF BUILDING WITH VERTICAL POST TENSIONED COLUMNS UNDER EARTHQUAKE LOAD BY PUSHOVER ANALYSIS

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Abstract

Vertical post tensioned columns are widely used for bridges in the USA and other countries. These columns can give higher re-centering ability and uncracked section after earthquake vibration effects. This method is applied to general building columns. In current stud, their earthquake behavior is evaluated by doing pushover analysis. In this analysis, performance parameters of building and modified response reduction factor are evaluated. Vertical bonded tendons are equally distributed along four sides of the columns. The amounts of PT (pre-stressed) steel provided are 0.4%, 0.8%, 1.2% and 1.6% of the column cross section area. RC (reinforced concrete) space frame building with aspect ratio 4 is selected for study. Only corner columns are pre-stressed. The results of the analysis show that increases in Performance point, maximum base shear (V max) and response reduction factor (R) can be achieved by increasing PT steel and PT force in tendons. Number of Hinges develop in frame will decrease in initial cases and increase in later part as increase in PT forces. Here columns are punished by giving extra vertical PT force up to 50% of its axial load capacity with maximum reinforcement steel up to 4% of the area.

Keywords: *Post Tensioned Column, Pushover Analysis, Performance Point, Response Reduction Factor and Bonded Tendon*

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1. INTRODUCTION AND MOTIVATION

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. In 1987 this method was used in sunshine skyway bridge piers to take only static loads [1]. In 1999, a viaduct for US highway 183 in Austin, Texas was constructed with post tensioned precast piers [2]. Research was conducted by Hewes and Priestly [3] in 2002 for segmental unbounded post tensioned columns. Single tendon was used in center of column with no longitudinal reinforcements. They concluded that up to 20%of axial compression by PT tendon gives good results. Jeong et.al.[4] in 2008 worked for partially pre-stressed column. They used combined PT tendon in center with longitudinal reinforcement. This arrangement gives impressive re centering ability, tendons remain elastic, and with increase in PT force the section remains uncracked at large drift. M Saidi [5] in 2012 worked on vertical post tensioned columns. They tested two unbounded cast in situ bridge columns and developed analytical model on SAP2000 [6]. They concluded that with lower PT force and higher longitudinal steel higher ductility, higher energy dissipation and higher re centering ability can be achieved. Precast post tensioned bridge columns were constructed for highway bridge in Indiana USA in 2012 [7].

Above all work and practical application of post tensioned columns was done as single degree of freedom system. This method is applied for multi degree of freedom system in precast beam column building. So from all above results and applications it was proposed to use this method to general building with cast in situ post tensioned columns. In this paper G+11 building with aspect ratio 4 is studied. 4 corner columns are provided with vertical post tensioned bonded tendons. Pushover analysis is carried with different percentage of tendon reinforcement with different pre-stressing forces. According to literature survey, columns are punished up to 50% of their axial capacity through stretching tendons.

2. MODELING OF BUILDING

A G+11 story building with size of beam is 350mm x 650mm and size of columns as 450mm x 450mm is studied. Tendons with different area and different forces are provided. The building is having 3m bays of 3m each in both the lateral directions giving an overall plan dimension of 9m x 9m. The story height is also 3m giving and overall height as 36m. Thus the height to width ratio is 4 for the building. The grade of concrete in all beams and in columns is taken as M30 (30 N/mm2) and grade of steel as Fe415 (415 N/mm2). In this research bonded tendons as elements are provided. Running from top to bottom for 36min the four corner columns shown by figure 1a. 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 [8] design formula.

$P_U = 0.45 \text{ x } f_{CK} \text{ x } A_C + 0.67 \text{ x } f_Y \text{ x } A_{SC}$

Where Pu = axial load on member
Fck = characteristic compressive strength of concrete
Ac = area of concrete
Fy = characteristic strength of compressive reinforcement
Asc = area of longitudinal reinforcement for columns

So half of columns capacity becomes 2438 kN and extra PT force can be applied up to this force limit. Here maximum allowable tensile stress in tendon is 1720 N/mm2. Numbers of cases which are taken for analysis with different properties are given in table 1. Figure 1a and 1b shows the geometric configuration of frame with bonded PT tendons. 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 3240 mm^2 respectively. Area of each tendon 202.5, 405, 607.5 and $810mm^2$ 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 up to 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.

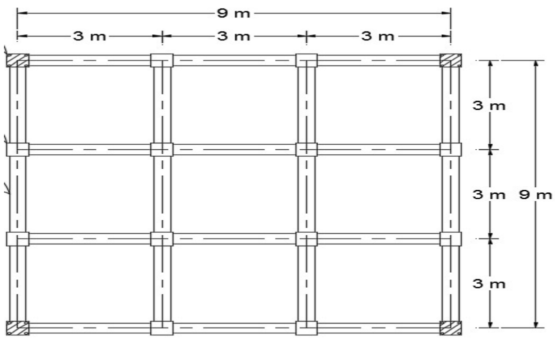


Fig-1a: Typical plan view of building

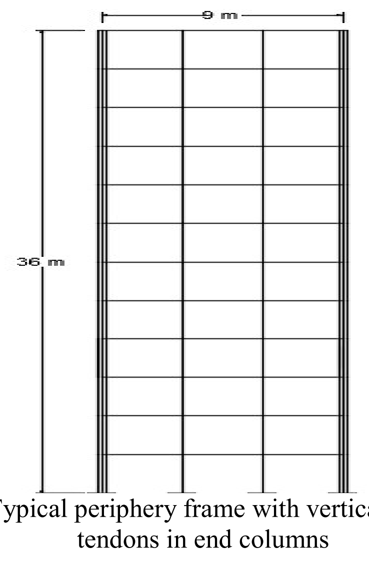


Fig-1b: Typical periphery frame with vertical bonded PT tendons in end columns

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	0	0	0	0
PT 0.4 0	0	0	0	0
PT 0.4 10	10	34.83	139.32	2.85
PT 0.4 20	20	69.66	278.64	5.71
PT 0.4 30	30	104.49	417.96	8.57
PT 0.4 40	40	139.32	557.28	11.43
PT 0.4 50	50	174.15	696.6	14.29
PT 0.4 60	60	208.98	835.92	17.14
PT 0.4 70	70	243.81	975.24	20.00
PT 0.4 80	80	278.64	1114.5	22.86
PT 0.4 90	90	313.47	1253.8	25.72
PT 0.4 100	100	348.3	1393.2	28.57
PT 0.8 0	0	0	0	0
PT 0.8 10	10	69.66	278.64	5.71
PT 0.8 20	20	139.32	557.28	11.43
PT 0.8 30	30	208.98	835.92	17.14
PT 0.8 40	40	278.64	1114.56	22.86
PT 0.8 50	50	348.3	1393.2	28.57
PT 0.8 60	60	417.96	1671.84	34.29
PT 0.8 70	70	487.62	1950.48	40.00
PT 0.8 80	80	557.28	2229.12	45.72
PT 1.2 0	0	0	0	0
PT 1.2 10	10	104.49	417.96	8.57
PT 1.2 20	20	208.98	835.92	17.14
PT 1.2 30	30	313.47	1253.88	25.72
PT 1.2 40	40	417.96	1671.84	34.29
PT 1.2 50	50	522.45	2089.8	42.86
PT 1.6 0	0	0	0	0
PT 1.6 10	10	139.32	557.28	11.43
PT 1.6 20	20	278.64	1114.56	22.86
PT 1.6 30	30	417.96	1671.84	34.29
PT 1.6 40	40	557.28	2229.12	45.72

3. PUSHOVER ANALYSIS

Displacement controlled pushover analysis are performed (SAP 2000) by subjecting structures to a monotonically increasing pattern of earthquake load through elastic and inelastic behavior until an ultimate point, representing the inertial forces which can be sustained by the structure when subjected to ground shaking. Under cumulative increasing loads various structural elements will yield progressively. At each step, the structure experiences a loss in stiffness. Material nonlinearity is assigned to discrete hinge locations where plastic rotation would occur according to FEMA-356 [9] and FEMA-440[10]. Hinge properties are defined in the form of force-deformation curvature with five points labeled A, B, C, D, and E (Figure 2). The value of these points determined from the moment curvature relationship of an element depends on the type of geometry, longitudinal reinforcement, material property, shear reinforcement and load subjected to a particular member. The method of unloading entire structure is employed for redistributing loads of the plastic hinges. Using pushover analysis, characteristic non-linear force displacement relationships are obtained. Figure 3 graphically indicates the modified force displacement relationship in reinforced concrete where FU, FY and FR represent ultimate, yield and residual strength while DY, DU, DL and DR correspond to displacements for point of yield, ultimate, ductile and residual strength. Dx is the displacement at collapse or ultimate failure. For pushover analysis 100% dead load and 25% live loads are considered as initial loads. Auto hinges with hinge type M3 and P-M2-M3 hinges are assigned to beams and columns respectively. Fig 5 shows the base shear vs. displacement curve. Fig 4s shows acceleration displacement response spectra (ADRS) per FEMA 356 for particular case. The demand parameters in this ADRS curve generation are taken as zone 5 and soil type considered as hard soil as per IS 1893:2002 [11]. At performance point the hinge state remains in immediate occupancy (IO) stage only. Fig. 6 shows hinge condition at performance point. Table 2 shows colors of different hinge stage. The performance point and hinge condition at this point is given in following table 3.

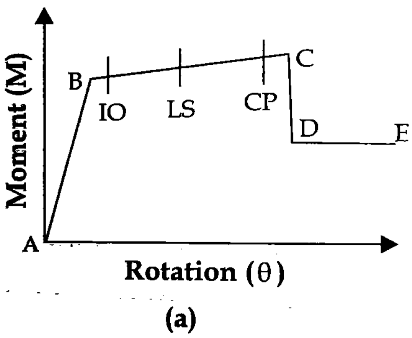


Fig 2: Curve for moment vs rotation

Table 2: Colors of different hinge stage

Hinge Conditions	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E
Colors							

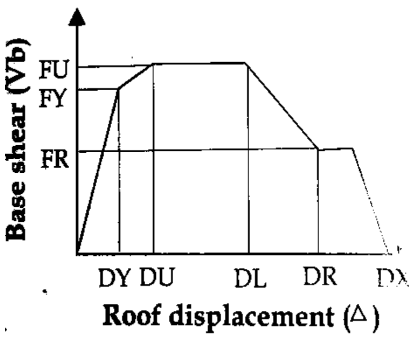


Fig 3: Idealized backbone curve

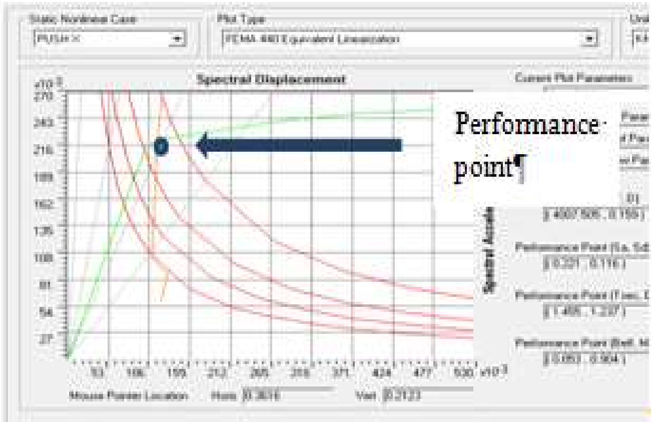


Fig 4: FEMA 440 curve (ADRS form) for PT IV 0.4 30%

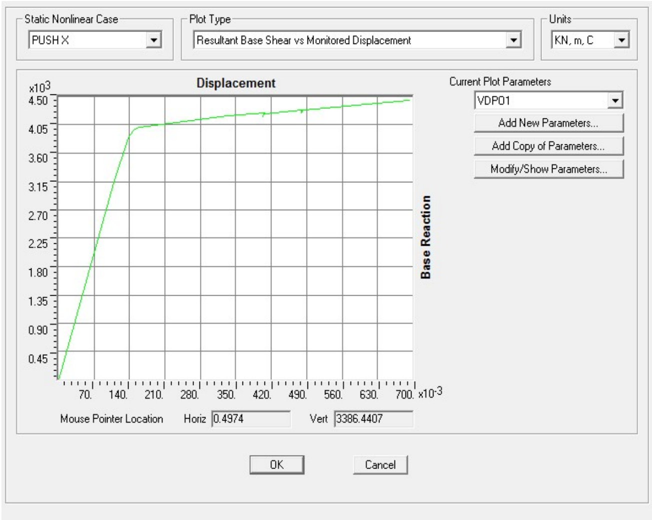


Fig 5: Capacity curve for case PT IV 0.4 30%

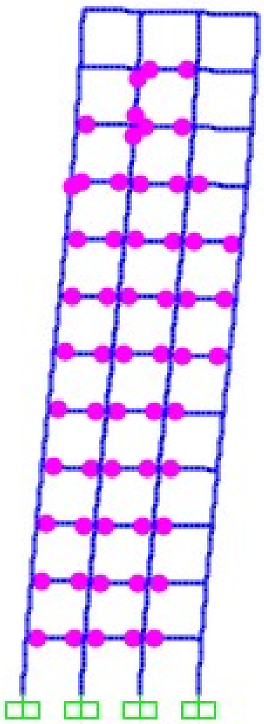


Fig 6: Hinge conditions at performance point

Table 3: Performance point for different cases

Case Name	**Performance Point (P – Point)**		**Hinge condition at P - Point**
	V (base shear)	D (displacement)	IO
	kN	m	No of hinges
NO PT	4000.62	0.159	240
PT 0.4 0	4007.705	0.159	240
PT 0.4 10	4007.705	0.159	240
PT 0.4 20	4007.705	0.159	240
PT 0.4 30	4007.705	0.159	240
PT 0.4 40	4007.705	0.159	240
PT 0.4 50	4007.705	0.159	240
PT 0.4 60	4007.705	0.159	240
PT 0.4 70	4007.705	0.159	240
PT 0.4 80	4007.705	0.159	240
PT 0.4 90	4007.705	0.159	240
PT 0.4 100	4007.705	0.159	240
PT 0.8 0	4005.590	0.159	248
PT 0.8 10	4010.420	0.159	236
PT 0.8 20	4008.420	0.159	238
PT 0.8 30	4015.450	0.159	248
PT 0.8 40	4022.780	0.159	250
PT 0.8 50	4025.220	0.159	256
PT 0.8 60	4031.177	0.159	278
PT 0.8 70	4033.954	0.159	290
PT 0.8 80	4038.580	0.159	294
PT 1.2 0	4008.080	0.158	236
PT 1.2 10	4022.080	0.158	225
PT 1.2 20	4017.860	0.158	227
PT 1.2 30	4031.860	0.159	236
PT 1.2 40	4040.860	0.159	238
PT 1.2 50	4054.860	0.160	244
PT 1.6 0	4010.640	0.158	228
PT 1.6 10	4018.170	0.157	217
PT 1.6 20	4022.257	0.157	218
PT 1.6 30	4046.257	0.160	228
PT 1.6 40	4075.257	0.158	230

4. MODIFICATION OF RESPONSE REDUCTION FACTOR

Generally, the response reduction factor is measured in terms of over-strength, ductility, redundancy and damping of the structure. Mathematically, it can be written as:

$$R = R_S \text{ x } R_\mu \text{ x } R_\xi \text{ x } R_R$$

Where R_S is strength factor, R_μ is ductility factor, R_ξ is damping factor and R_R is redundancy factor. The maximum lateral strength of the building (V_u) will generally exceed the design lateral strength (V_d) of building because the members or elements are designed with capacities commonly greater than design actions and material strength also exceed specified nominal strengths. Thus the strength factor (R_S) or over-strength factor is defined as the ratio of ultimate base shear to yield base shear.

$$R_S = \frac{V_u}{V_Y}$$

The ductility factor (R_μ) is an indication of global nonlinear response of framing systems. It depends on ductility (μ) of building, which can be calculated as the ratio of ultimate or maximum displacement (D_{max}) to yield displacement (D_y).

$$\mu = \frac{D_{max}}{D_Y}$$

Many studies have been carried out to determine the ductility factor of structure. Among these, work done by Krawlinker and Nassar [12], Newmark and Hall [13], T. Paulay and M. J. N. Priestley [14] are significant. In the present study, the relationship between R_μ and ductility level (μ) developed by T. Paulay and M. J. N. Priestley is used. As per T. Paulay and M. J. N. Priestley, the relationship is given by,

$$R\mu = 1 + \frac{(\mu - 1)\,T}{0.7}$$

The redundancy factor, (R_r) is a measure of redundancy in a lateral load resisting system. This depends on the structural system adopted. As per ASCE7 [15], the redundancy factor is taken as 1 when the structure has geometric configuration of parallel frame system. Following this guideline of ASCE7, for present study the value of redundancy factor is taken as 1. The damping factor (R_ξ) depends upon external damping of the structure. For structure which is not provided with any external damping, it is taken as 1. In this study, external damping is not provided in system. Thus, R_ξ is equal to 1. After determination of push curve, their data was used in calculation of R value. This modified value of response reduction factor is given in table 4.

Table 4: Modified response reduction factor

Case Name	**Dmax**	**Vmax**	**Dy**	**Vy**	**R_s**	**u**	**R_μ**	**R**
	m	kN	m	kN				
NO PT	0.268	4119.500	0.138	3958.480	1.041	1.942	2.817	2.931
PT 0.4 0	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 10	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 20	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 30	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 40	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 50	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 60	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 70	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 80	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 90	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.4 100	0.545	4435.320	0.137	3948.900	1.123	3.978	6.743	7.574
PT 0.8 0	0.594	4153.670	0.136	3948.780	1.052	4.368	7.495	7.884
PT 0.8 10	0.592	4191.053	0.136	3952.334	1.060	4.350	7.461	7.912
PT 0.8 20	0.589	4228.773	0.136	3955.891	1.069	4.333	7.427	7.940
PT 0.8 30	0.587	4266.831	0.136	3959.451	1.078	4.315	7.394	7.968
PT 0.8 40	0.585	4305.233	0.136	3963.015	1.086	4.298	7.361	7.996
PT 0.8 50	0.582	4343.980	0.136	3966.582	1.095	4.281	7.328	8.025
PT 0.8 60	0.580	4383.076	0.136	3970.151	1.104	4.264	7.295	8.053
PT 0.8 70	0.578	4422.524	0.136	3973.725	1.113	4.247	7.262	8.082
PT 0.8 80	0.575	4462.326	0.136	3977.301	1.122	4.230	7.229	8.111
PT 1.2 0	0.610	4457.940	0.137	3949.070	1.129	4.453	7.658	8.645
PT 1.2 10	0.606	4502.519	0.137	3952.229	1.139	4.421	7.598	8.656
PT 1.2 20	0.601	4547.545	0.137	3955.391	1.150	4.390	7.539	8.667
PT 1.2 30	0.597	4593.020	0.137	3958.555	1.160	4.360	7.479	8.678
PT 1.2 40	0.593	4638.950	0.137	3961.722	1.171	4.329	7.421	8.689
PT 1.2 50	0.589	4685.340	0.137	3964.892	1.182	4.299	7.362	8.700
PT 1.6 0	0.630	4689.340	0.135	3952.890	1.186	4.667	8.071	9.575
PT 1.6 10	0.625	4745.612	0.135	3955.657	1.200	4.629	7.999	9.597
PT 1.6 20	0.620	4802.559	0.135	3958.426	1.213	4.592	7.928	9.619
PT 1.6 30	0.615	4860.190	0.135	3961.197	1.227	4.556	7.857	9.640
PT 1.6 40	0.610	4918.512	0.135	3963.970	1.241	4.519	7.787	9.662

5. DESIGN OUTPUT

Before doing pushover analysis, the structural design of building is done as per IS 456:2000 for dead, live, EQ X, EQ Y and Pre-stress load case and their combinations. Load combinations as per IS 875 part 3 [16] are taken and prestress load case is added to all of them. After the design, the quantity of material is determined for all cases which are mentioned in table 5.

Table 5: Quantity of material for different cases

Case Name	Quantity of concrete	Quantity of steel in Beams	Quantity of steel in columns	Quantity of PT steel	Total quantity of steel
	m3	ton	ton	ton	ton
NO PT	313.200	14.875	5.654	0.000	20.529
PT 0.4 0	313.200	13.811	5.529	0.916	20.255
PT 0.4 10	313.200	13.811	5.519	0.916	20.245
PT 0.4 20	313.200	13.811	5.506	0.916	20.232
PT 0.4 30	313.200	13.811	5.492	0.916	20.218
PT 0.4 40	313.200	13.811	5.478	0.916	20.204
PT 0.4 50	313.200	13.811	5.464	0.916	20.190
PT 0.4 60	313.200	13.811	5.449	0.916	20.176
PT 0.4 70	313.200	13.811	5.435	0.916	20.161
PT 0.4 80	313.200	13.811	5.422	0.916	20.148
PT 0.4 90	313.200	13.811	5.407	0.916	20.133
PT 0.4 100	313.200	13.811	5.388	0.916	20.114
PT 0.8 0	313.200	13.820	5.524	1.831	21.175
PT 0.8 10	313.200	13.825	5.504	1.831	21.160
PT 0.8 20	313.200	13.825	5.476	1.831	21.132
PT 0.8 30	313.200	13.825	5.448	1.831	21.104
PT 0.8 40	313.200	13.825	5.420	1.831	21.076
PT 0.8 50	313.200	13.825	5.389	1.831	21.045
PT 0.8 60	313.200	13.825	5.354	1.831	21.010
PT 0.8 70	313.200	13.825	5.317	1.831	20.973
PT 0.8 80	313.200	13.825	5.283	1.831	20.939
PT 1.2 0	313.200	13.849	5.519	2.747	22.114
PT 1.2 10	313.200	13.849	5.489	2.747	22.085
PT 1.2 20	313.200	13.849	5.447	2.747	22.043
PT 1.2 30	313.200	13.849	5.406	2.747	22.001
PT 1.2 40	313.200	13.849	5.353	2.747	21.949
PT 1.2 50	313.200	13.849	5.301	2.747	21.897
PT 1.6 0	313.200	13.864	5.517	3.662	23.043
PT 1.6 10	313.200	13.864	5.474	3.662	23.000
PT 1.6 20	313.200	13.864	5.419	3.662	22.945
PT 1.6 30	313.200	13.864	5.354	3.662	22.880
PT 1.6 40	313.200	13.864	5.286	3.662	22.813

6. DISCUSSION AND CONCLUSION

Chart 1:- steel quantity vs PT force

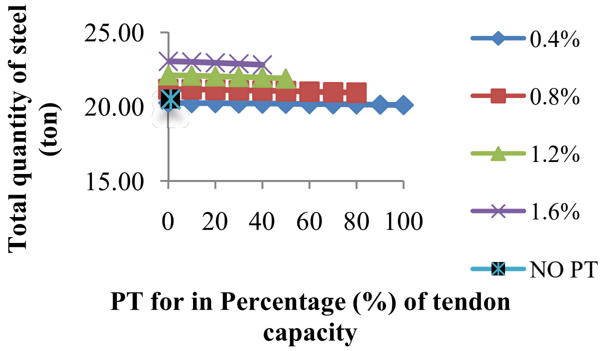
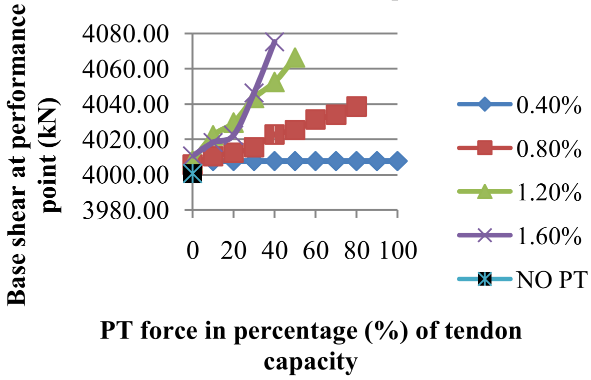
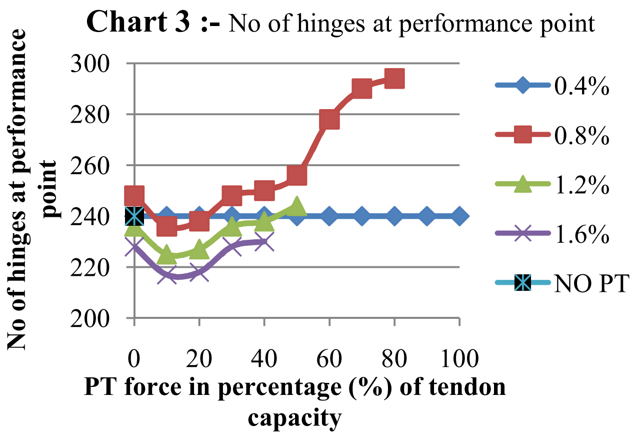
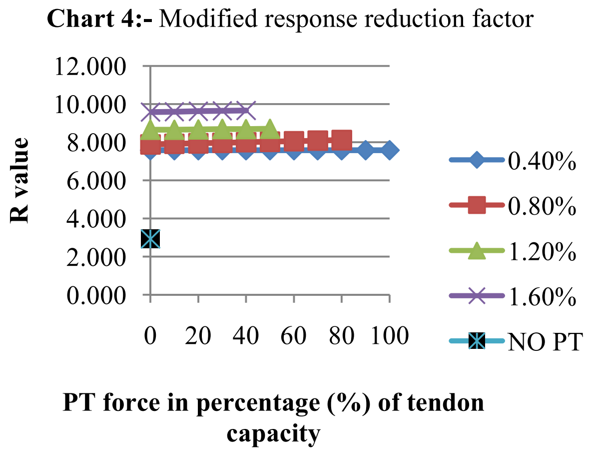


Chart 2:- Performance point







1. Quantity of reinforced steel decreases in column with increase in PT forces which can be seen in chart 1. There is no change in beam steel as increase in PT force. Compered to normal RC building with no PT, more steel will be consumed in columns of building with PT.
2. From the graph 2, it is clear that the performance point is not affected by increasing the PT force in frame having 0.4% PT reinforcement is not affected.
3. From chart 3, we can determine that number of hinges will decrease initially at performance point and later increases with increase in PT force. This indicates that when PT force is applied beyond 30% of tendon stretching there is more damage in building.
4. From chart 4, we can determine that Response reduction factor increases with increase in the amount of PT steel and PT force to column. 0.4% PT reinforcement in column has no affect on response reduction factor.
5. With increase in higher percentage of PT steel, the number of hinges developed at performance point decreases.
6. Referring to table 4, it can be observe that the ductility factor (μ) becomes almost 2 times on introducing PT tendons as compared to that with no PT tendons, so providing PT reinforcement we are increasing the ductility of building but there will be slight change in ductility as increase in PT reinforcement and PT forces.
7. The general nature of capacity curve under pushover analysis remains almost same as shown in figure 4 for all cases, indicating semi ductile performance under seismic forces.

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