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Comparative Study on Behaviour of RC and Post-Tensioned Building by Pushover Analysis

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Abstract— The Buildings which appeared to be strong enough, may crumble like houses of cards during earthquake and deficiencies may be exposed. In this fast-paced and competitive world, building sector is at the apex of the growth of any country. High-rise buildings are admired by every human being. Traditionally the construction of a building is done by RCC but in present world, construction of high rise buildings is done by Post-Tensioning to achieve unobstructed spaces which leads to slender structure. So, here an attempt has been made to study the seismic performances of reinforced-concrete buildings and post tension building by pushover analysis. A widely used computer program SAP is used to perform static nonlinear analysis for different storied buildings. The results obtained from analysis will compare in form of time period, drift, displacement, maximum base shear at Performance point, ductility factor, response reduction factor, and over strength factor.

Keywords— Reinforced Concrete, Post-Tensioning, Multi-storied, pushover analysis, plastic hinges, SAP-2000

I. INTRODUCTION

As the floor system plays an important role in the overall cost of a building, a post-tensioned floor system is invented which reduces the time for the construction and overall cost. In some countries, including the U.S., Australia, South Africa, Thailand and India, a great number of large buildings have been successfully constructed using post-tensioned (PT) floors. Post-tensioned concrete has been used in seismic resistance building structure. Hence the structure will be safe from earthquake. 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. Hence Post tensioned concrete floor has become quite popular now-a-days because of its distinct advantages such as low cost due to ease of construction, low floor-to-floor height because of shallow beams known as "fat" beams, and flexible use of space due to large span., Since the PT flat floor systems provide improved crack and deflection control, and allow relatively large span-to-thickness ratios and also very efficient in particular PT flat floor systems.

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 seismically 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 is a static-nonlinear analysis method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which continuously increases through elastic and inelastic behaviour until an ultimate condition is reached. Lateral load may represent the range of base shear induced by earthquake loading, and its configuration may be proportional to the distribution of mass along building height, mode shapes, or another practical means.

II. MODELLING ASPECTS

The present study compares the seismic performance of a typical framed structure of an overall plan dimension of 28m x 28m having panel size of 7m x 7m with the following three variations in beam size in its frame modelling as All RC beams of size 300mm × 600mm deep, Perimeter RC beams of same size and interior PT beams of 500mm × 375mm deep and All PT beams of size 300mm × 600mm deep. The column size for the G+5 storey building are 500mm X 500mm, G+9 storey building are 700mmX 700mm (up to G+3 floor), 600mm X 600mm (4 to 7 floor) and 500mm X 500mm (above 7 floor), G+12 storey building are 900mmX 900mm (up to G+3 floor), 700mm X 700mm (4 to 7 floor) and 600mm X 600mm (above 7 floor).

For all the models, slab is modelled as a shell element accounting for diaphragm action to be considered for seismic analysis. The columns are considered to be fixed at the foundation level. In case of PT beams the tendons are modelled as per the selected tendon profile. The column height is considered as 3m for each floor. Static Nonlinear Pushover analysis is carried out for all the models generated using SAP2000 software and compare the result in terms of time period, drift, displacement, base shear, ductility factor, response reduction factor and over strength factor.

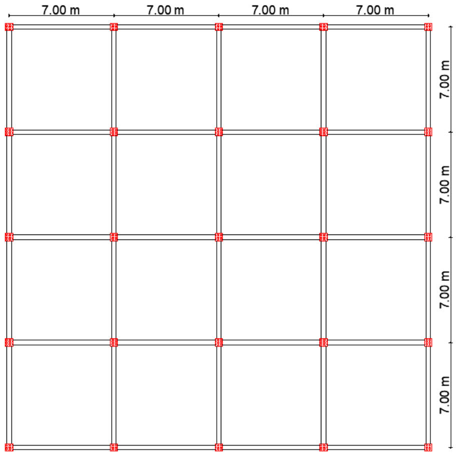

Fig. 1 Typical Floor Plan

TABLE I
DESIGN PARAMETERS FOR PT BEAMS

Sr. No.	Designation of PT Beams	Section Size of PT Beams in mm	Jacking Forced in KN	No. of Cables
1	Periphery Beams at Typical Floor	500 x 375	734.30	5
2	Internal Beams at Typical Floor	500 x 375	881.16	6

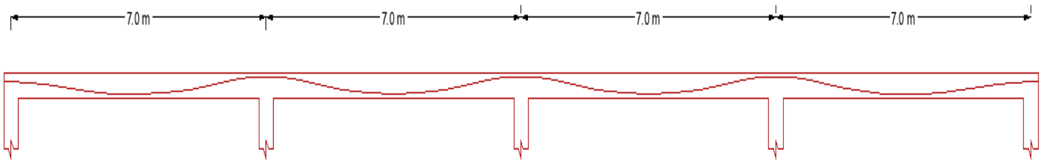

Fig. 2 Tendon Profile

III. LOAD DEFINITIONS

Each of the models is subjected to a floor finished load of 1.5 kN/m2 as super dead load and 2 kN/m2 as live load on all the typical floors. All external periphery beams are subjected to a uniformly distributed 230mm thick wall load of 11.04 kN/m on typical floors. A pre-stress load is defined in the analysis models pertaining to the transfer of axial pre-compression due to post tensioned cables. This load case is in the form of jacking forces applied at the end of all PT beams calculated by separate software ADAPT. These forces will balance the gravity loads only. Table 1 represents the design and analytical data of PT beams used in mathematical models in SAP2000 software. The cable profile used is a reverse parabola as shown in Fig. 2, which generally gives maximum advantage of load balancing. The pre-stressing steel considered for post-tensioning is a strand composed of 7-wires, low relaxation steel wires, twisted in a helical pattern around 1 centre wire. The strand used is as per the strand designation No. 13 of ASTM-A416M.

IV. PUSHOVER ANALYSIS

Flexural (M3) and Shear (V2) are defined at 0.1L and 0.9L for all beam elements and The Combined Axial and Flexural (PMM) and Shear (V2) type of hinges are defined at 0.1L and 0.9L for all the column elements, where L is the length of the beam element. The 0.1L flexural hinge in beams is typically defined to capture the effects due to maximum sagging moment developed at mid span of beams during the push in the gravity direction. The static analysis is carried out for the given dead, live and earthquake loads. Typically, the following two push over analysis cases are defined for each of the buildings. PUSH1 is the case in which the gravity loads are applied up to their total force magnitude. It may be noted here that the jacking force applied at the ends of the PT cables as per Table 1 is already in effect simultaneously. PUSH2 is defined as the push in the lateral X-direction, and it starts from the end of PUSH1. The X-displacement of the roof level node is monitored up to the magnitude of 4 percent of the building height, when push is given as per the earthquake force profile in the X-direction. Once the displacement is noted down at performance point, which is much less than 4 percent of the height of the building for all cases, one more cycle of push over analysis is carried out by modifying the target displacement of roof level node to the displacement obtained at performance point. This is typically done to get the relevant data like number and state of hinges at the performance point as one stops pushing the structure beyond performance point in the second cycle of push over analysis.

V. RESPONSE MODIFICATION FACTOR

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

$$R = R_S \times R\mu \times R_R$$

Where R_S is strength factor, R_μ is ductility factor and R_R is redundancy factor. The real strength of the structure may be higher than its design strength because of overall design simplifications. Modern computer- aided tools allow us to model & design structures that closely match those that are actually built. The ratio of maximum base shear coefficient to design base shear is termed as over strength factor R_s.

$$R_s = V_0 / V_d$$

Ductility factors are used to evaluate translation ductility ratios. The relationship between maximum elastic loads & maximum inelastic loads can define as the ductility factor for the same structural building under inelastic behaviour. The calculation of the ductility factor is carried out with the equations derived by Newmark & Hall, which can be calculated as the ratio of ultimate or maximum displacement (Δ_{max}) to yield displacement (Δ_y).

$$\mu = \Delta_{max}/ \Delta_y$$

Redundancy is commonly defined as “beyond what is essential or naturally excessive”. In general, redundancy in a structural system is active under an earthquake resistant design. ATC (1995) explains that the redundancy factor value is considered based on the line of vertical seismic framing. The redundancy factor is taken as 1 when the structure has geometric configuration of parallel frame system.

VI. RESULTS OF THE ANALYSIS

The results of the analysis for the three types of models considered are represented in the form of deformed shapes with numbers of hinges developed when the model is pushed up to the performance point. The corresponding demand/capacity curves for the models under PUSH-X (lateral X–direction push).once the analysis has been done we extract the results like performance point results, time period, displacement, drift, over strength factor, ductility factor and response modification factor.

TABLE II
RESULTS OBTAINED FOR DIFFERENT MODALS AT PERFORMANCE POINT

Storey	**Base Shear V in kN**	**Displacement D in mm**	**Spectral Acceleration S_a**	**Spectral Acceleration S_d**	**Effective Time Period $T_{eff.}$**	**Effective Damping $B_{eff.}$**
G+5 RC	8013.269	158.430	0.136	123.242	1.895	0.202
G+5 RCPT	8186.645	170.323	0.140	128.121	1.906	0.184
G+5 PT	7872.870	183.910	0.134	136.341	2.012	0.181
G+9 RC	11930.577	229.573	0.128	170.314	2.295	0.142
G+9 RCPT	11717.061	253.082	0.124	181.923	2.410	0.135
G+9 PT	11522.334	270.374	0.123	189.858	2.480	0.129
G+12 RC	13051.257	286.519	0.107	211.435	2.788	0.134
G+12 RCPT	12776.283	319.826	0.104	226.751	2.932	0.127
G+12 PT	12402.358	343.936	0.101	238.416	3.053	0.122

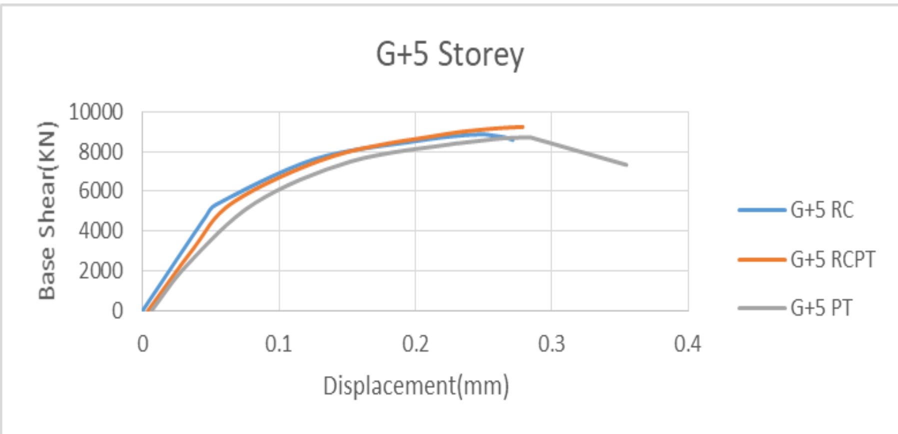

Fig. 3 Pushover Curve for G+5 Storey Building

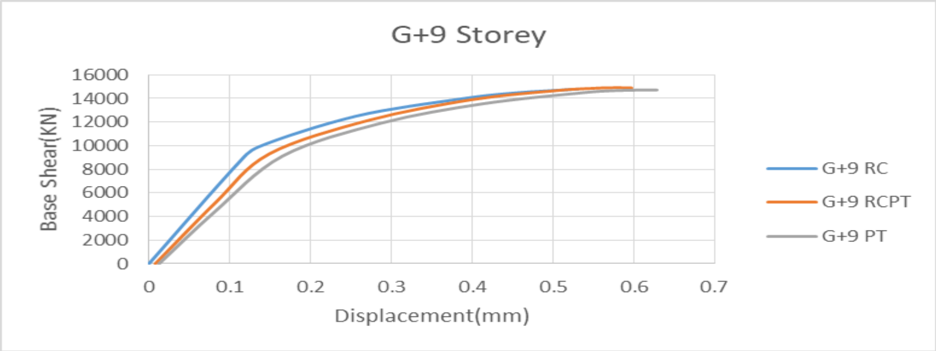

Fig. 4 Pushover Curve for G+9 Storey Building

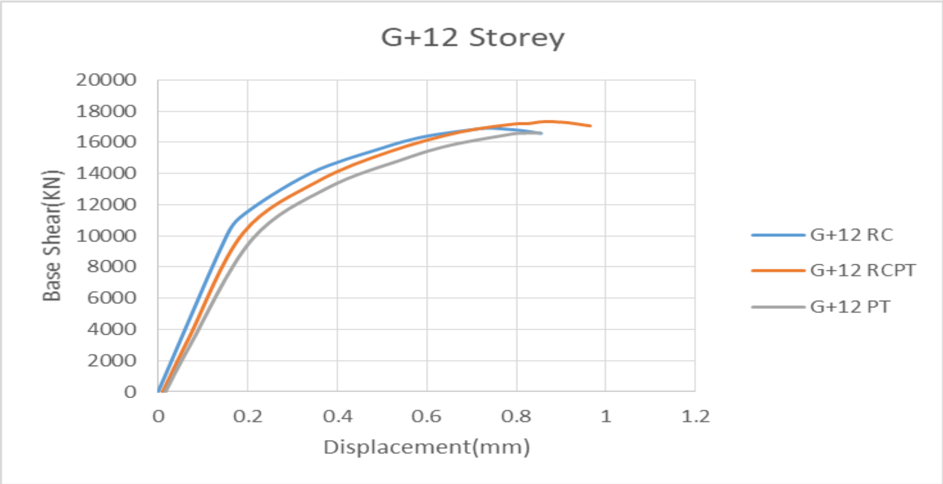

Fig. 5 Pushover Curve for G+12 Storey Building

TABLE III
NUMBER OF HINGES DEVELOPED AT DIFFERENT STRESS LEVEL AT PERFORMANCE POINT

Storey	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
G+5 RC	1103	121	213	54	0	69	0	0	1560
G+5 RCPT	1100	101	242	53	0	64	0	0	1560
G+5 PT	1108	98	263	38	0	53	0	0	1560
G+9 RC	1886	154	331	114	0	115	0	0	2600
G+9 RCPT	1857	130	382	74	0	157	0	0	2600
G+9 PT	1850	117	365	94	0	174	0	0	2600
G+12 RC	2556	146	219	160	0	299	0	0	3380
G+12 RCPT	2540	137	186	90	0	427	0	0	3380
G+12 PT	2494	94	296	156	0	340	0	0	3380

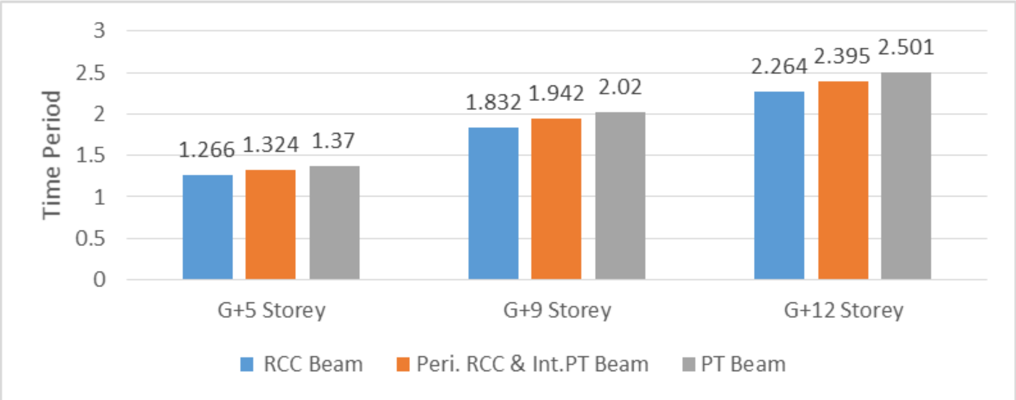

Fig. 6 Modal Time Period

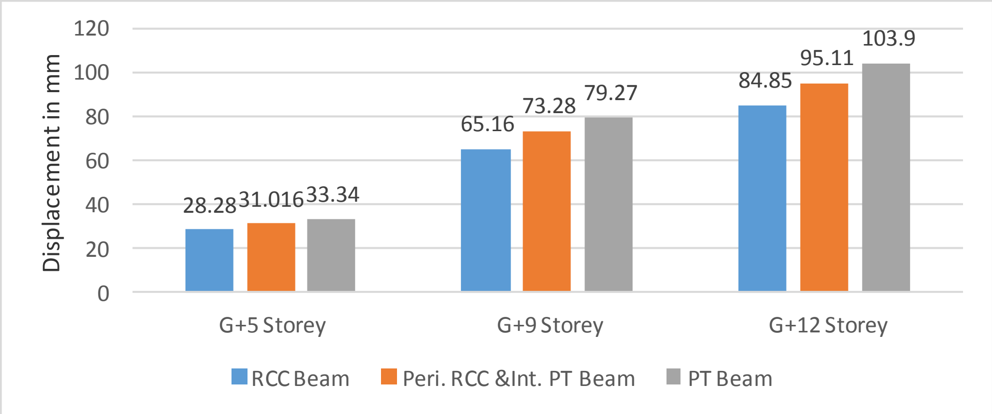

Fig. 7 Modal Displacement Max.

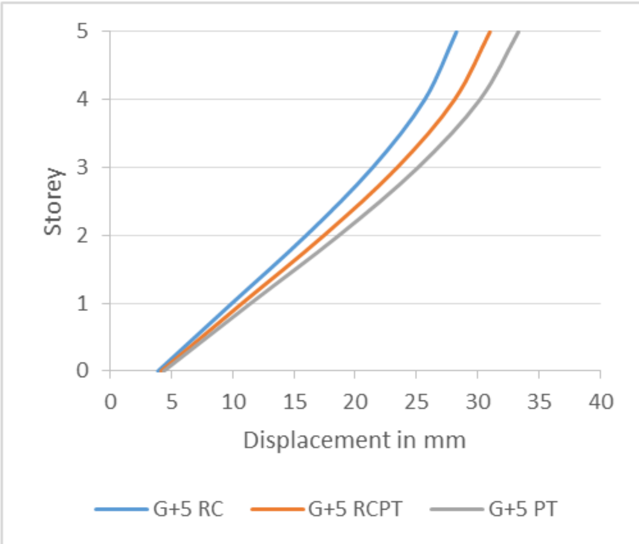

Fig. 8 Storey vs. Displacement for G+5 Storey Building

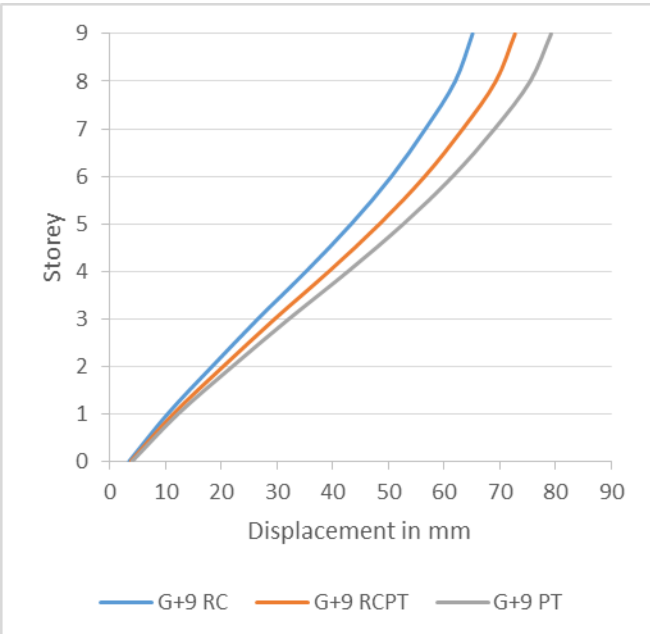

Fig. 9 Storey vs. Displacement for G+9 Storey Building

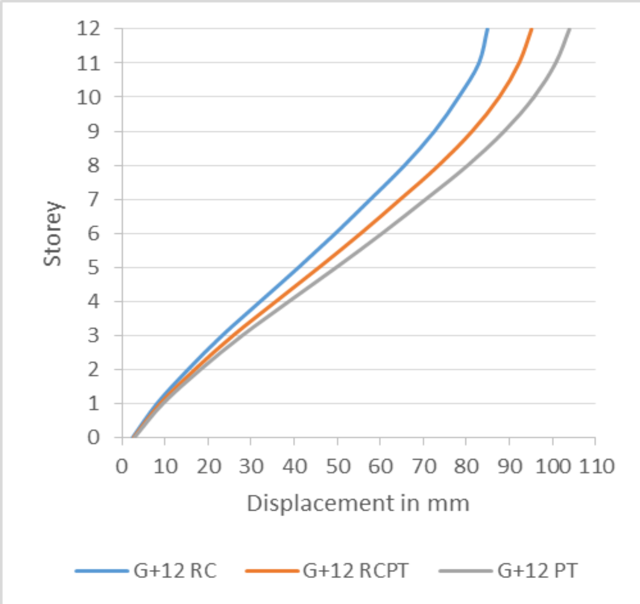

Fig. 10 Storey vs. Displacement for G+12 Storey Building

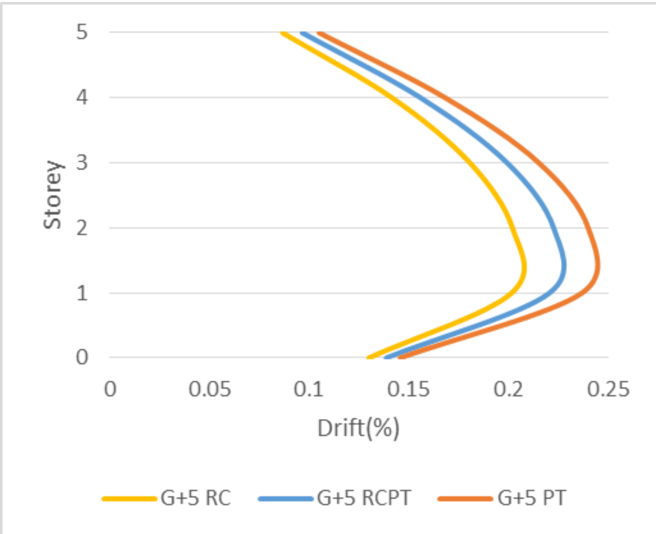

Fig. 11 Storey Drift for G+5 Story Building

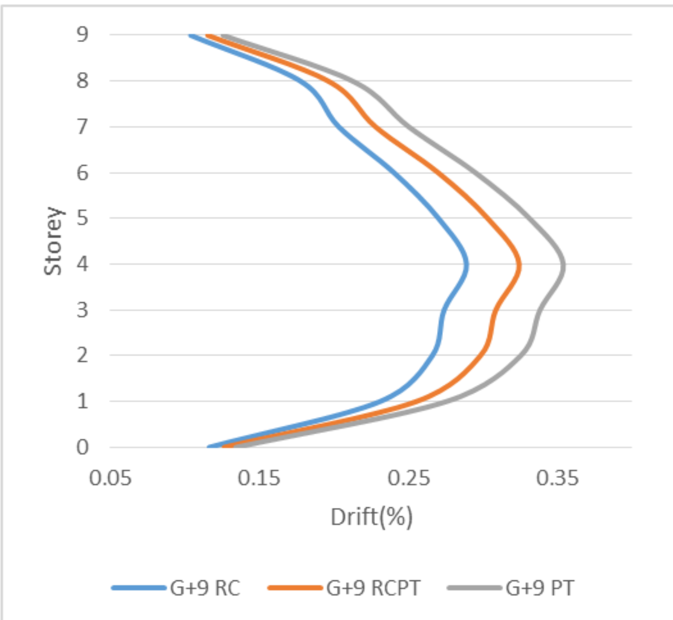

Fig. 12 Storey Drift for G+9 Storey Building

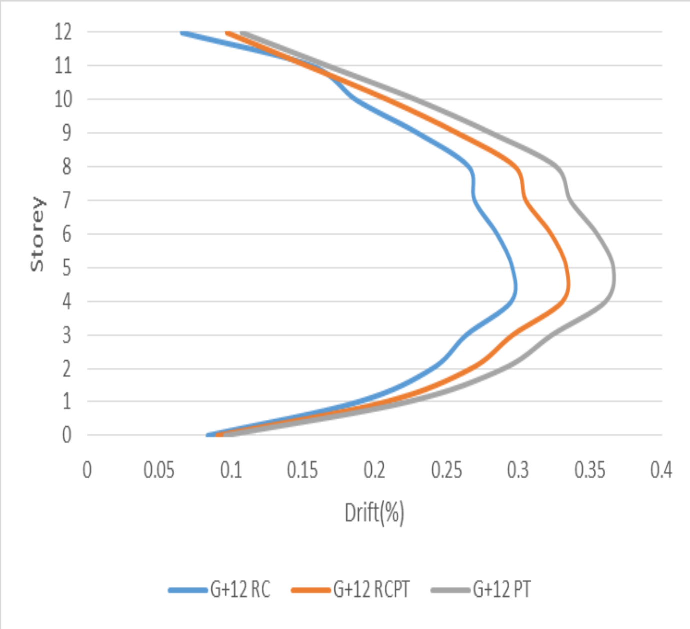

Fig. 13 Storey Drift for G+12 Storey Building

TABLE IV
RESPONSE MODIFICATION FACTOR

Storey	**Max. Base Shear V in KN(Vo)**	**Design Base Shear in kN(V_d)**	**Maximum Drift Capacity in mm (Δ_{max})**	**Yield Drift Capacity in mm (Δ_y)**	**Over Strength Factor R_S**	**Redundancy Factor R_R**	**Ductility Factor R_μ**	**Response Modification Factor R**
G+5 RC	8013.269	1818.876	72.000	54.410	4.406	1	1.323	5.829138
G+5 RCPT	8186.645	1826.44	72.000	36.760	4.482	1	1.959	8.780238
G+5 PT	7872.870	1831.48	72.000	43.130	4.299	1	1.669	7.175031
G+9 RC	11930.577	3062.93	120.000	107.350	3.895	1	1.118	4.35461
G+9 RCPT	11717.061	3075.53	120.000	63.230	3.81	1	1.898	7.23138
G+9 PT	11522.334	3083.93	120.000	84.310	3.736	1	1.423	5.316328
G+12 RC	13051.257	4026.11	156.000	134.800	3.242	1	1.157	3.750994
G+12 RCPT	12776.283	4042.49	156.000	107.840	3.16	1	1.447	4.57252
G+12 PT	12402.358	4053.41	156.000	115.190	3.06	1	1.354	4.14324

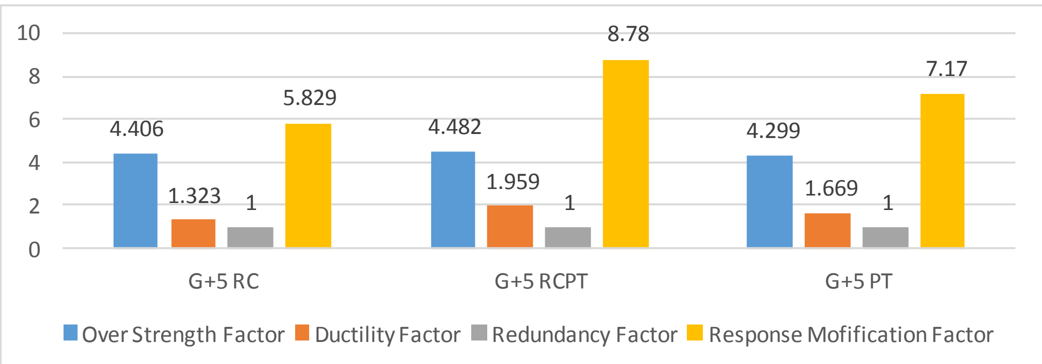

Fig. 14 Response Modification factor for G+5 Storey Building

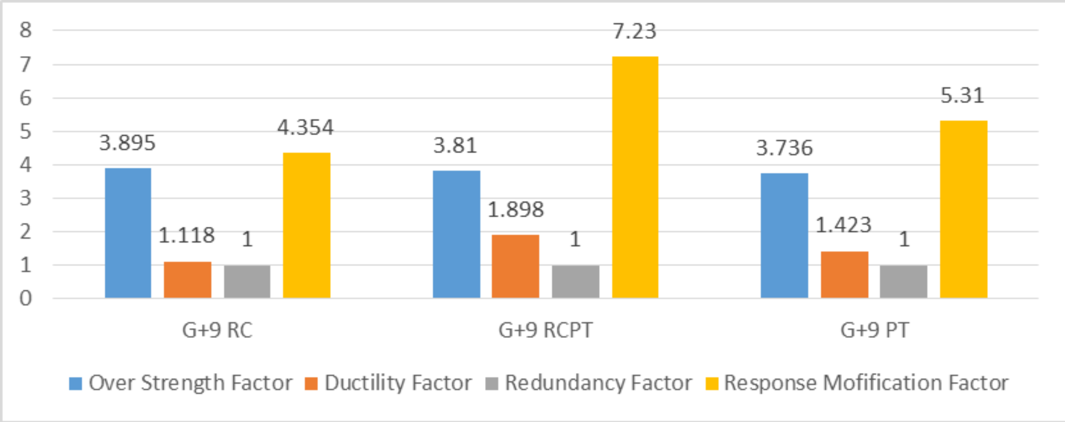

Fig. 15 Response Modification factor for G+9 Storey Building

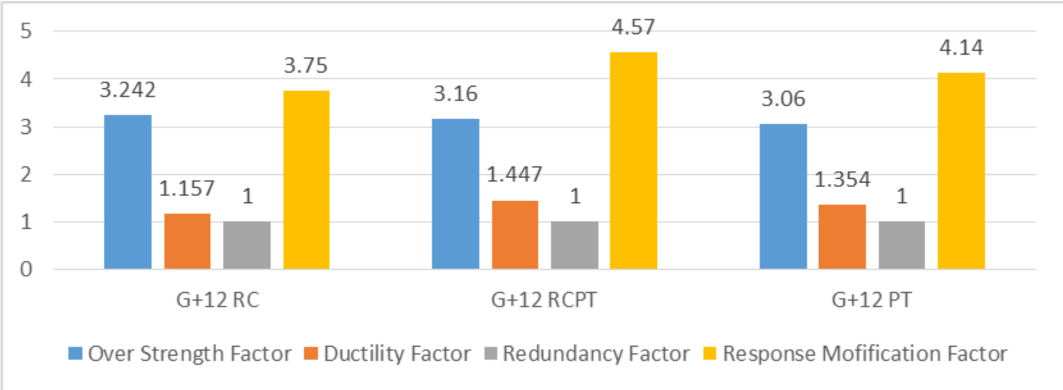

Fig. 16 Response Modification factor for G+12 Storey Building

VII. Discussion of Results

The comparison is made between a conventional RC frame and a building frame consisting of PT beams, the base shear increase by 2.16% in G+5 RCPT building and decrease by 1.75% in G+5 PT building as compare to G+5 RC building. The base shear decrease by 1.78% in G+9 RCPT building and decrease by 3.42% in G+9 PT building as compare to G+9 RC building. . The base shear decrease by 2.10% in G+12 RCPT building and decrease by 4.97% in G+12 PT building as compare to G+12 RC building.

The over strength factor increase by 1.72% in G+5 RCPT building and decrease by 2.42% in G+5 PT building as compare to G+5 RC building. The over strength factor decrease by 2.18% in G+9 RCPT building and decrease by 4.08% in G+9 PT building as compare to G+9 RC building. The over strength factor decrease by 2.52% in G+12 RCPT building and decrease by 5.61% in G+12 PT building as compare to G+12 RC building.

Ductility factor increase by 42.32% in G+5 RCPT building and increase by 26.15% in G+5 PT building as compare to G+5 RC building. Ductility factor increase by 69.76% in G+9 RCPT building and increase by 27.28% in G+9 PT building as compare to G+9 RC building. Ductility factor increase by 25.06% in G+12 RCPT building and increase by 17.02% in G+12 PT building as compare to G+12 RC building.

Response Modification factor increase by 44.62% in G+5 RCPT building and increase by 23.0% in G+5 PT building as compare to G+5 RC building. Response Modification factor increase by 66.05% in G+9 RCPT building and increase by 22.09% in G+9 PT building as compare to G+9 RC building. Response Modification factor increase by 21.92% in G+12 RCPT building and increase by 10.48% in G+12 PT building as compare to G+12 RC building.

VIII. Conclusions

It can be concluded from the results obtained from the analysis the seismic performance of RC framed structures having conventional RC beams on the periphery of the building and PT beams in the interior grids of the structure is the best for up to G+12 storey structures. The results further indicate that the over strength factor and ductility factor increase with increase of post-tensioning area of tendons in RCPT building thus Response Modification Factor also within acceptable level.

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