CHAPTER 12
SEISMIC RESPONSE OF FRAMES WITH POST TENSIONED BEAMS

12.1 THE CONCEPT OF POST TENSIONED CONCRETE SYSTEM

Post-tensioning is a technique of pre-loading the concrete in a manner which eliminates, or reduces, the tensile stresses that are induced by the dead and live loads. High strength steel ropes, called *strands*, are arranged to pass through the concrete floor. When the concrete gets hardened, each set of strands is gripped in the jaws of *a hydraulic jack* and stretched to a pre-determined force. Then the strand is locked in a device, called an *anchorage*, which has been cast in the concrete; this induces a compressive stress in the concrete. The strand is thereafter held permanently by the anchorage. The non-jacking end of the strand may be bonded in concrete, or it may be fitted with a pre-locked anchorage which has also been cast in the concrete. To allow the strand to stretch in the hardened concrete under the load applied by the jack, bond between the strand and concrete is prevented by a tube through which the strand passes. The tube is termed *a duct* or *sheathing*. If extruded, the strand is injected with rust-inhibiting grease. After stressing, the sheathing, if not of the extruded kind, is grouted with cement mortar using a mechanical pump. **Fig. 12.1** is a diagrammatic representation of the process.

In prestressing, a permanent external axial force, of predetermined magnitude, is applied to the concrete member, which induces a compressive stress in the concrete section. When the service load is applied, the generated tensile stress has to overcome the compressive prestress before the concrete is driven into any tension. The tensile strength of concrete is, therefore, effectively enhanced. The prestressing force does not significantly change with the load within the serviceability limit. The principle is illustrated in **Fig. 12.2**.

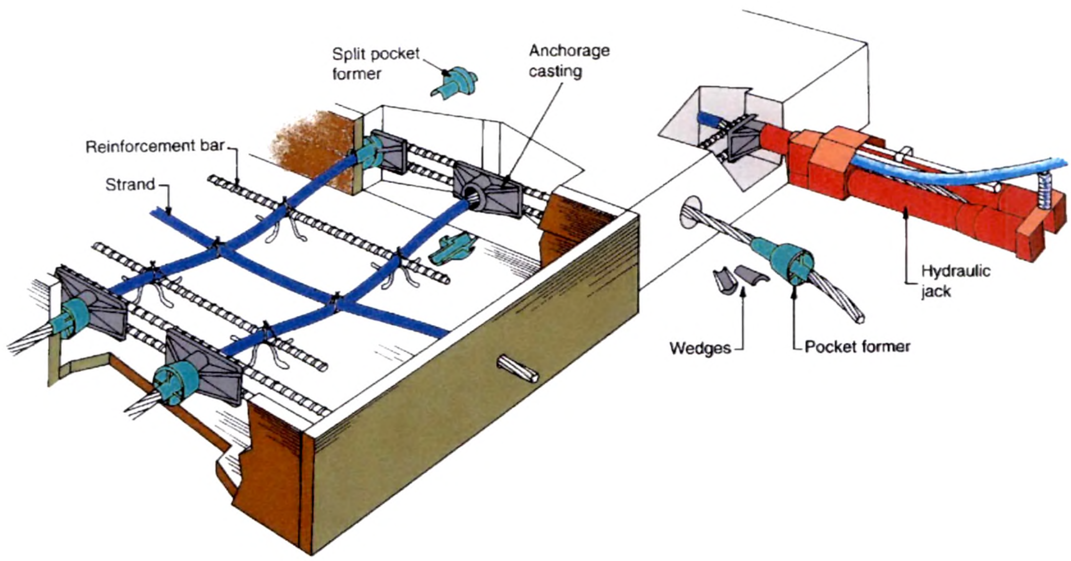


Fig. 12.1 Post Tensioning Systems and Devices

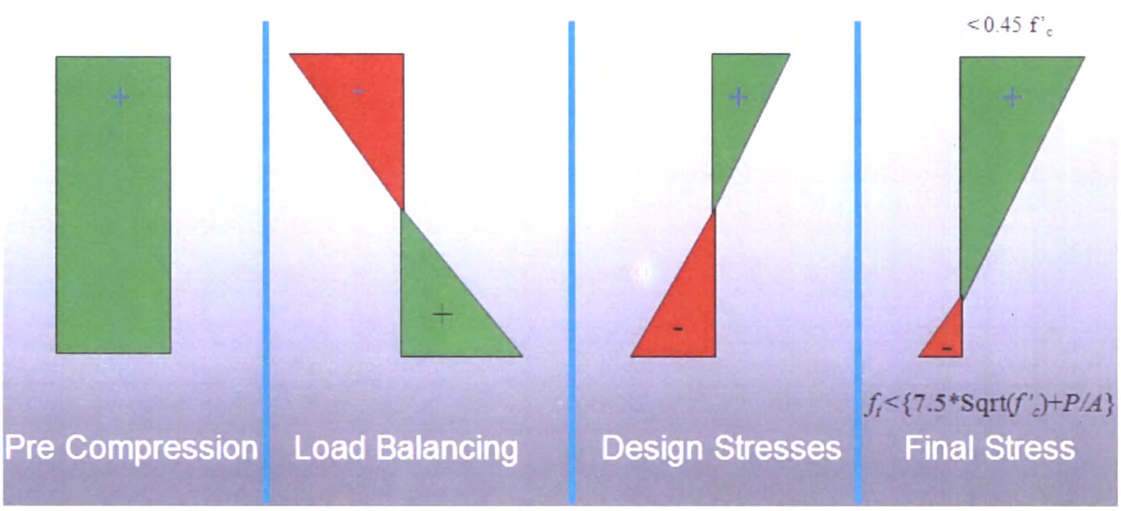


Fig. 12.2 Stress Sequence in Pre Stressed Systems

The tendons can be stressed either before casting the concrete or after the concrete has been cast and has gained some strength. In *pre-*

tensioning the wires or strands are stressed against external anchor points (or sometimes against the mould) and concrete is then cast in direct contact with the tendons, thus allowing bond to develop. In *post-tensioning*, concrete is not allowed to come in contact with the tendons. The tendons are placed in ducts, or sheaths, which prevent bond, and concrete is cast so that the duct itself is bonded but the tendon inside remains free to move.

12.2 REINFORCED AND POST TENSIONED CONCRETE FLOORS

Reinforced concrete technology is widely available and is well understood. Post-tensioning is an advancement in reinforced concrete technology and it is often discussed in the context of reinforced concrete. For post-tensioning, it is important to consider availability of the hardware and the technical expertise required. Excepting very special design objectives, post-tensioning is unlikely to be economical for short spans as shown in **Fig. 12.3**. Often a combination of post-tensioning and another form of construction offers a good solution.

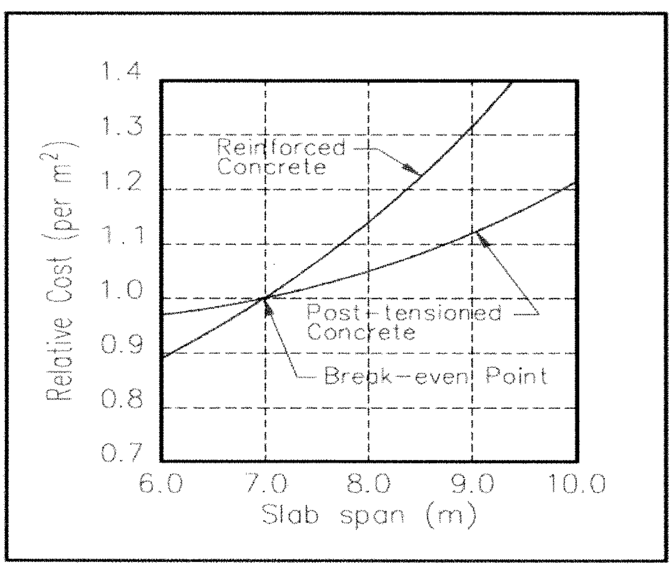


Fig. 12.3 Comparison between Reinforced and PT Concrete

For reinforced concrete, only the ultimate strength calculations are normally carried out and deflection in the serviceability state is deemed to be satisfied by confining the span-to-depth ratio within limits prescribed in the national standards. Deflection is not a major issue and crack control is usually governed by specifying the reinforcement spacing.

In post-tensioned concrete design, serviceability calculations are carried out for the initial and final loading conditions, for deflection and cracking, and the ultimate strength is checked after this. Structural design of prestressed concrete, therefore, requires more effort. The shallow depth of a post-tensioned floor is a particular advantage in multistorey buildings; in some cases it has been possible to add an extra floor where there was a restriction on building height. Even where there is no such restriction, the reduced building volume generates savings in the cost of services. The post tensioned concrete technology is often referred to as PT for short.

The function of post-tensioning is to place the concrete structure under compression in those regions where load causes tensile stress. Tension caused by the load will first have to cancel the compression induced by the post-tensioning before it can crack the concrete. **Figure 12.4(a)** shows a plainly reinforced concrete single-span beam and a cantilever beam cracked under applied load. **Figure 12.4(b)** shows the same unloaded beams with post-tensioning forces applied by stressing high strength tendons. By placing the post-tensioning low in the simple-span beam and high in the cantilever beam, compression is induced in the tension zones; creating upward camber.

Figure 12.4(c) shows the two post-tensioning beams after loads have been applied. The loads cause both the simple-span beam and cantilever beam to deflect down, creating tensile stresses in the bottom of the simple-span beam and top of the cantilever beam. Effect of post-tensioning balances the effects of load in such a way that tension from the loading is compensated by compression induced by the post-tensioning. Tension is eliminated under the combination of the two and tension cracks are prevented. Also, construction materials are used more efficiently; optimizing materials, construction effort and cost.

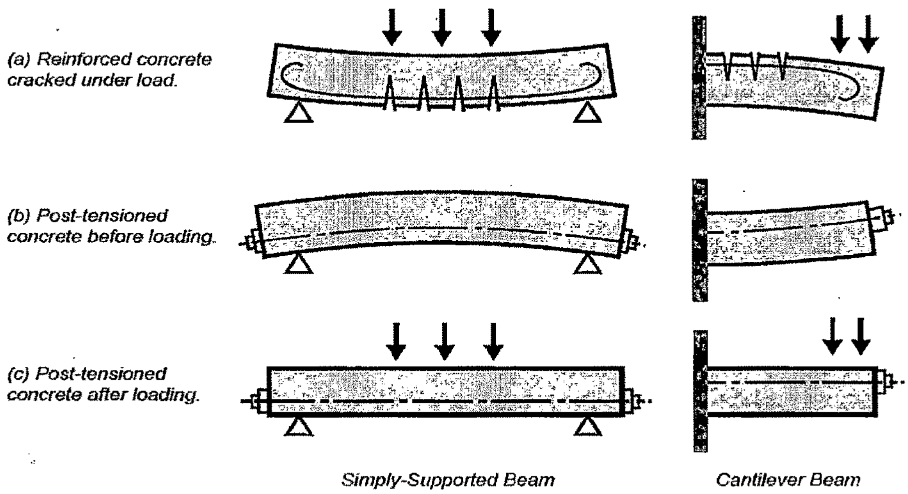


Fig. 12.4 Reinforced and Prestressed Concrete Beam Comparison

12.3 THE EFFECT OF PT FLOORS ON SEISMIC PERFORMANCE

The earthquake resistance of modern structures is based on their capacity to safely dissipate the vibration energy imparted to them by the ground shaking. As they are designed to remain elastic under only a small fraction, 1/R, of the design seismic action (R denote response reduction factor of modem seismic design standards), most of the input seismic energy has to be dissipated through the inelastic behaviour of the structural members. For given ground motion the total input seismic energy depends mainly on the magnitude and distribution of the mass and the elastic stiffness of the structure, and much less on the extent and distribution of the inelastic response. Accordingly, the main aim of modern earthquake resistant design is to direct the energy absorption only to the members and regions capable of dissipating inelastically significant amounts of energy without loss of their (gravity) load-bearing capacity, and to spread the total energy dissipation demand to these latter regions as evenly as possible, in order to avoid failure of regions with excessively large share in the energy dissipation. Capacity design is the tool that modern seismic design codes use to spread uniformly the inelastic energy dissipation to the regions capable of it.

The PT technology although quiet widely accepted the world over is relatively a new entrant in our country. The literature studied points out the need to use a separate lateral load resisting system in the form of shear walls or bracings in high seismic zones. The present study aims at evaluating the performance of typical all RC framed, RC and PT framed and all PT framed building against lateral earthquake forces. The building has to be designed with conventional limit state design. To understand the nonlinear behaviour of building as a whole, static nonlinear (pushover) analysis is carried out.

Further, the parameters like sway potential, capacity of the building in terms of base shear, maximum roof displacement, performance point, are to be quantified and compared for the building designed by both the approaches. Thus, the main objective of study is to explore and evaluate the seismic performance of newly emerging approach of post tensioned floors over the conventional RC one and hence to maintain the proper strength hierarchy and ductility in structural elements of buildings to safeguard them for life safety and collapse prevention conditions.

12.4 THE HYBRID CONCEPT APPLIED TO FRAMES WITH PT BEAMS

It is observed from the previous chapters that the hybrid frame is the one in which all the internal beams have a semi rigid ends, whereas the external beams have a fully rigid connection with the columns. This type of assumed frame behaves as a flexible core inside bounded by a stiff outer shell. This concept is often used for design of high rise buildings where lateral forces are predominant. It is even logical to think about the hybid frame as a structure with only outer frame which is contributing the most in resisting the lateral loads. Thus, it is seen in the previous chapter that even if one considers the flexural stiffness of all the internal beams as zero, the seismic capacity is only slightly affected. This particular fact is used in the current study of frames with PT beams. The distinct advantage of PT lies in the fact that the floor to floor height reduces due to wide shallow beams often known as "fat" beams and flexible use of space due to large spans. In particular, Post-Tensioned (PT) flat floor systems are very efficient, since they provide improved crack and deflection control, and allow relatively large span-to-thickness ratios of the order of 35 to 45. PT floor systems are commonly used to resist only gravity loads in high seismic regions (Seismic Design Category D or E, IBC-2006) **[76]**; however, they may be utilized as intermediate moment frames (ACI 318-2005, Section 21.12.6) **[77]** in areas with moderate seismic demands. Given the broad potential applications, a detailed study of flat floor system behavior subjected to lateral forces and/or displacements is important.

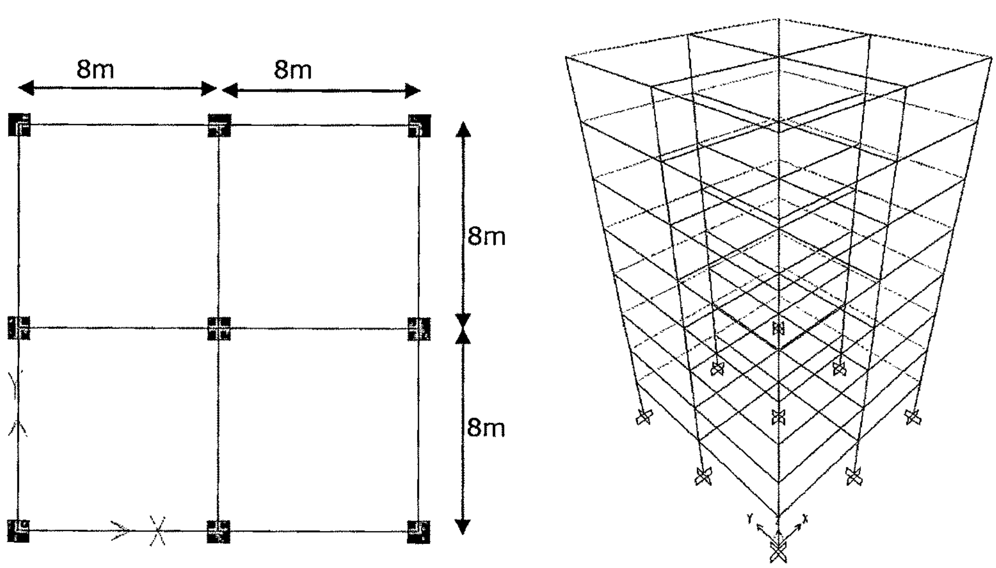
The use of shallow wide beams are envisaged by structural engineers as having less flexural rigidity compared to conventional RC beam in the major direction. This fact can be seen as a PT beam having low flexural rigidity can be compared to an RC frame with low rigidity at the ends of all the internal beams. Thus, a regular, fully rigid jointed RC frame in the periphery with all internal beams as PT fat beams can be considered as a hybrid frame defined in the earlier chapter. Hence, it is proposed to compare the performance of a regular RC frame having all beam column joints as rigid with that of the same structure having internal beams as PT beams representing a hybrid frame. It is also worthwhile to study the performance of the same frame considering all beams including the peripheral beams as PT beams.

12.5 MODELS DEVELOPED TO STUDY THE EFFECT OF PT BEAMS

12.5.1 Geometry of the Models

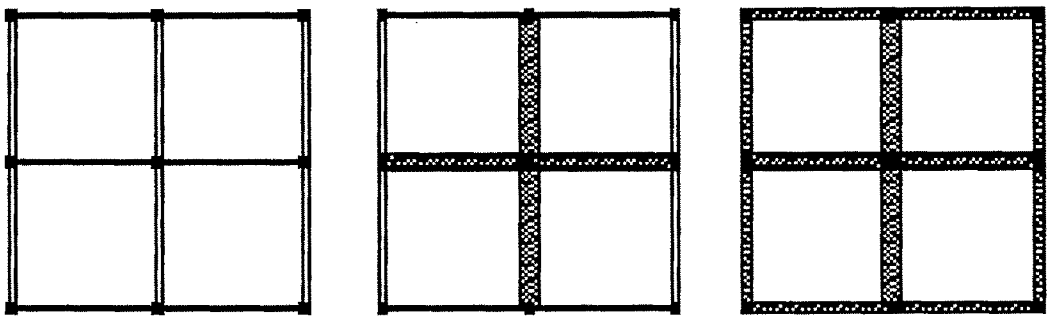
1. Overall plan dimensions are 16m x 16m having 8m x 8m panel size with the following three variations in its frame modelling,
   a). All beams are RC of size 300mm x 600mm deep.
   b). Perimeter RC beams of size 300mm x 600mm deep and internal Post tensioned (PT) beams of size 1000mm x 350mm deep.
   c). All beams are PT beams having perimeter beam size as 500mm x 350mm deep and internal beam size as 1000mm x 350mm deep.

   The typical plan and an isometric view are shown in **Fig. 12.5** and the typical beam arrangements as per a, b and c above are shown in **Fig. 12.6**.



Typical Floor Plan **Isometric View of a G+7 Frame**

Fig. 12.5 Geometry Considered for the Models



a) All RC beams b) Peripheral RC internal PT c) All PT beams

Fig. 12.6 Typical Floor Plans of the Mathematical Models

2. Storeys considered are G+3, G+4, G+5, G+6 and G+7
3. The height of the columns in the global Z-direction is considered as 3.5m for each storey. The columns are considered to be fixed at the foundation level.
4. Square columns are considered at all grid intersections. The size of columns is considered as 450mm x 450mm for all models on all stories. The column sizes are increased to 550mm x 550mm in bottom one storey for the G+6 and G+7 structures.
5. Materials used are concrete of M30 grade and steel of Fe415 grade.

12.5.2 Post Tension Beam Parameters

The PT strands are considered to be of Grade 1860 MPa (270 ksi) low relaxation, seven-wire strand, twisted in a helical pattern around 1 center wire conforming to the requirements of ASTM A 416. The strand used is as per the strand designation No. 13 of ASTM-A416M (2002) **[78]**. The other parameters such as losses and coefficients are used from the technical note of ADAPT Corporation (2004) **[79]**.

A separate load case called pre-stress is defined in the analysis models pertaining to the transfer of axial pre-compression and load balancing 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 balance the gravity loads only. **Table 12.1** 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. 12.7**, which generally gives maximum advantage of load balancing.

Table 12.1 Design Parameters for PT Beams

Designation of PT beams	Section size of PT beams in mm	Jacking force in kN	No. of cables	Axial stress in N/mm^2	Dead load balancing in %
Perimeter beams at typical floor	500 x 350	977	8	1.24	52
Perimeter beams at terrace floor	500 x 350	735	6	0.93	48
Internal beams on all floors	1000 x 350	1352	11	0.86	52

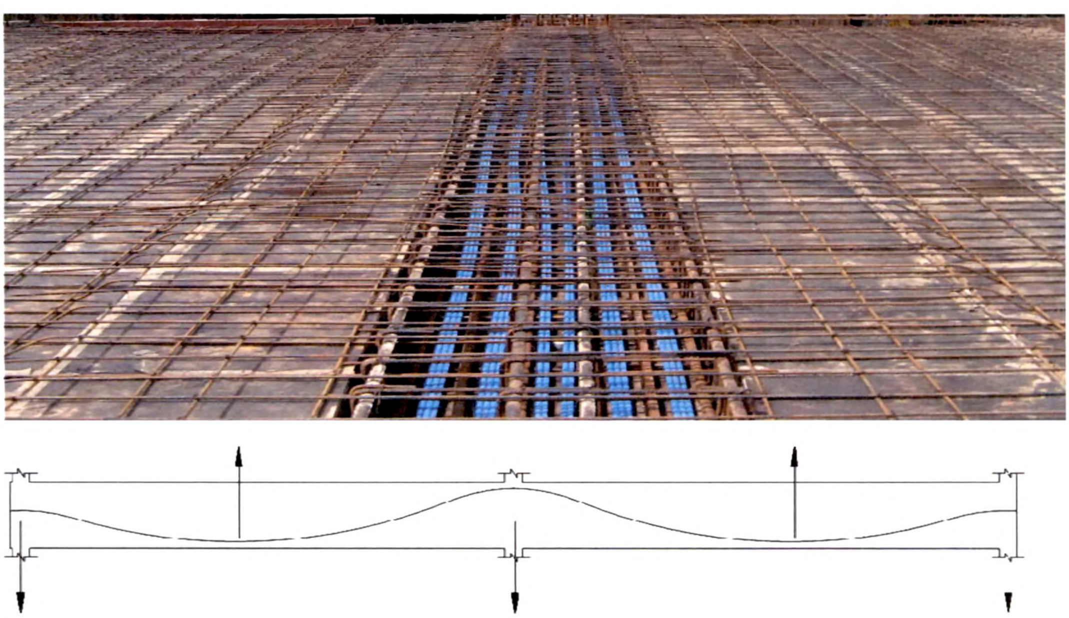


Fig. 12.7 The Reverse Parabola Cable Profile for the PT Beams

12.5.3 Push over Analysis Parameters

The combined axial and flexural (PMM) type of hinges are defined at 0.05L and 0.95L for all the column elements and Flexural (M3) hinges are defined at 0.05L, 0.5L and 0.95L for all beam elements where L is the length of the beam element. The 0.5L 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. The slab is modelled as a shell element and a rigid diaphragm action is considered for the seismic analysis.

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 12.1** 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.

Other Push over analysis parameters considered in SAP2000 software are:

1. Local redistribution method for hinge unloading is used.

2. Displacement controlled nonlinear static analysis is considered.
3. P-Delta type geometric nonlinearity is considered.
4. To evaluate seismic performance, considered seismic coefficients are $C_A = 0.312$ and $C_V = 0.456$ considering seismic zone IV and medium soil as per IS 1893, Part 1, 2002. (Refer **Tables 10.2** and **10.4**).

12.5.4 Loads Considered on the Models

Slab thickness = 0.175 m

Imposed loads considered at terrace level = 1.5 kN/m^2

At typical floor level = 3.0 kN/m^2

Dead load on terrace floor including floor finish = 7.5 kN/m^2

On typical floor level = 6.5 kN/m^2

Dead load due to walls on peripheral beams considered as

For terrace floor level = 6 kN/m

For typical floor level = 14.5 kN/m

Earthquake load parameters considered as per IS 1893:Part 1, 2002,

Seismic Zone factor Z = 0.24, for zone IV

Type of soil = Medium,

Importance factor = 1, and

Response reduction factor = 5 considering ductile detailing.

12.6 THE RESULTS OF PUSH OVER ANALYSIS

Push over analysis is carried out for the 15 mathematical models developed as per the parameters defined for G+3 storey space frame to G+7 storey structure. The results obtained at performance point for all the models are presented in **Table 12.2**. The various parameters noted include base shear and roof displacement for the models. The number of hinges in different categories developed at performance point for the same is presented in **Table 12.3**.

Table 12.2 Results Obtained at Performance Point for PUSH2

Storey	Parameter	All RCC Beams	RC + PT Beams	All PT Beams
G+3	Base Shear V in kN	2600	4081	3641
	Roof Displacement D in mm	106	110	138
	Sa/g	0.224	0.35	0.307
	Sd in mm	95	87	110
	Teff in sec	1.304	1.003	1.2
	Beff (%)	20.5	12.7	10.8
G+4	Base Shear V in kN	3552	3869	3643
	Roof Displacement D in mm	123	148	176
	Sa/g	0.243	0.255	0.246
	Sd in mm	115	120	141
	Teff in sec	1.381	1.372	1.517
	Beff (%)	14.4	12.1	10.3
G+5	Base Shear V in kN	3886	4159	3312
	Roof Displacement D in mm	145	170	178
	Sa/g	0.232	0.239	0.256
	Sd in mm	131	137	143
	Teff in sec	1.509	1.517	1.498
	Beff (%)	12.7	11.4	9.5
G+6	Base Shear V in kN	3930	4204	3998
	Roof Displacement D in mm	169	195	241
	Sa/g	0.201	0.207	0.201
	Sd in mm	149	156	191
	Teff in sec	1.728	1.741	1.956
	Beff (%)	13.1	11.5	8.7
G+7	Base Shear V in kN	3918	4204	3892
	Roof Displacement D in m	194	221	269
	Sa/g	0.174	0.18	0.172
	Sd in m	167	176	215
	Teff in sec	1.968	1.984	2.247
	Beff (%)	13.7	11.9	9.3

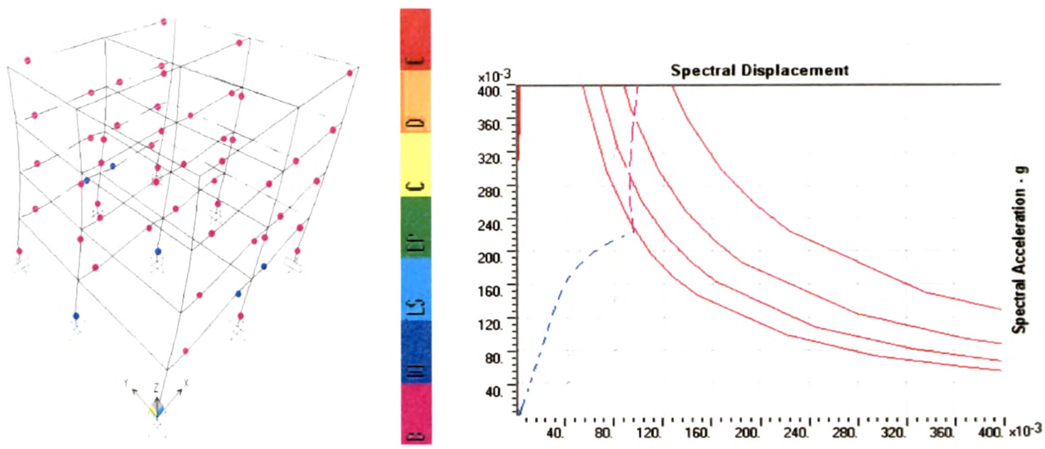
Table 12.3 Hinges Developed at Performance Point for PUSH2

Storey	Frame Type	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	> E	Total
G+3	All RC	**159**	**51**	**6**	0	0	0	0	0	**216**
	RC+PT	**177**	**39**	**0**	0	0	0	0	0	**216**
	All PT	**175**	**39**	**2**	0	0	0	0	0	**216**
G+4	All RC	**233**	**37**	0	0	0	0	0	0	**270**
	RC+PT	**234**	**36**	0	0	0	0	0	0	**270**
	All PT	**229**	**41**	0	0	0	0	0	0	**270**
G+5	All RC	**287**	**37**	0	0	0	0	0	0	**324**
	RC+PT	**283**	**41**	0	0	0	0	0	0	**324**
	All PT	**274**	**50**	0	0	0	0	0	0	**324**
G+6	All RC	**339**	**39**	0	0	0	0	0	0	**378**
	RC+PT	**336**	**42**	0	0	0	0	0	0	**378**
	All PT	**340**	**38**	0	0	0	0	0	0	**378**
G+7	All RC	**394**	**38**	0	0	0	0	0	0	**432**
	RC+PT	**386**	**46**	0	0	0	0	0	0	**432**
	All PT	**388**	**44**	0	0	0	0	0	0	**432**

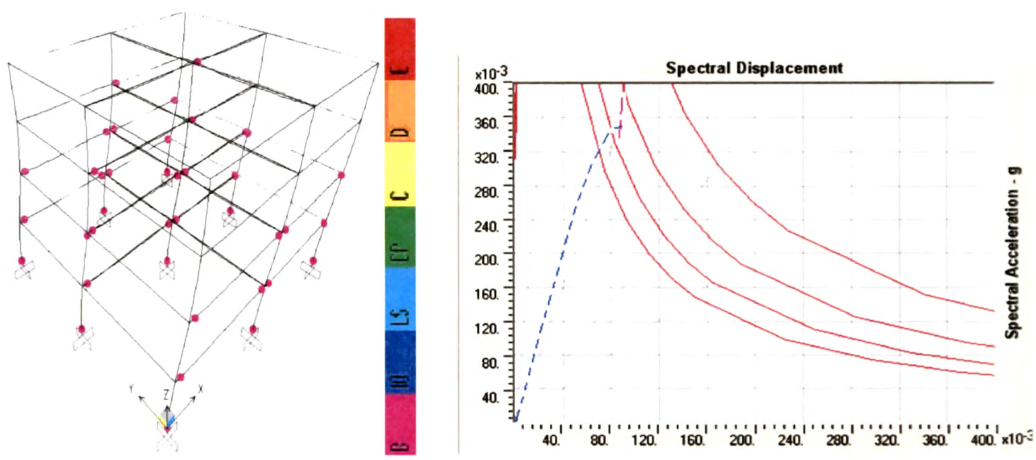
The results of the analysis for the three types of models considered are represented in the form of deformed shapes in **Figs. 12.8** to **12.12** with colour coded hinges developed when the model is pushed up to the performance point. The corresponding demand/capacity curves for the models under PUSH-X (lateral X-direction push) are shown side by side. A typical demand/capacity curve represents the family of demand spectra for 5, 10, 15 and 20 percent damping shown by the solid red lines and the capacity curve, shown in the ADRS format is represented by a broken blue line in the figures. It also plots single demand spectra with variable damping shown as dash dot convention in magenta colour and constant period lines (in the radial directions shown in grey) for time periods of 0.5, 1.0, 1.5 and 2.0 seconds.

Figure 12.13 represents the base shear resisted by each of the models at performance point segregated according to the three basic types of model for G+3 to G+7 buildings whereas **Fig. 12.14** shows the roof displacement for all the buildings at performance point for three basic variations in the framing. The variation in the base shear at performance point is also studied for all the 15 models by plotting the values on a single plot. This variation is shown in **Fig. 12.15**. For the same set of models, the variation in roof displacement is also plotted in **Fig. 12.16**. The effective damping at performance point which is a measure of damage suffered by the building under lateral force is plotted in **Fig. 12.17** for the models considered for analysis.

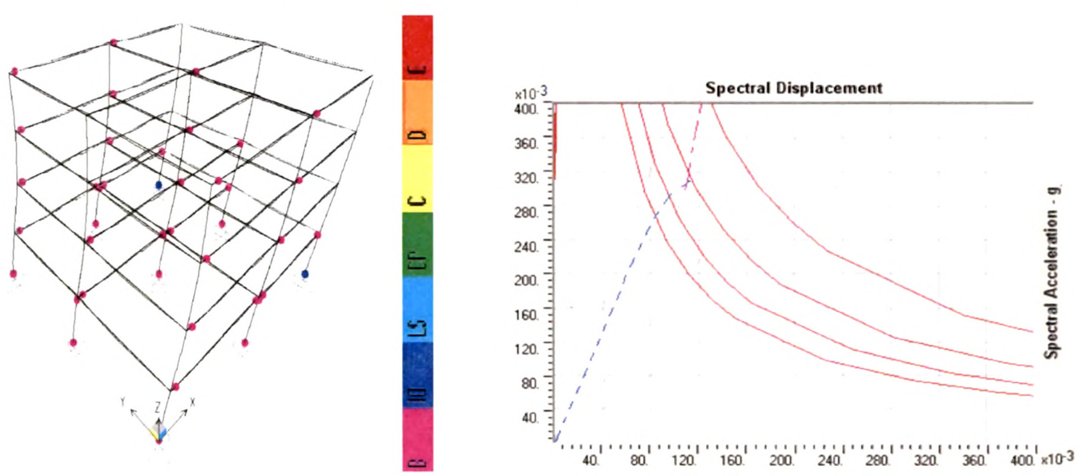
The number and severity of plastic hinges developed at performance point give an insight into the seismic performance of a building. Thus, it is one of the important parameters which needs to be studied. The plastic hinges developed at performance point shown in **Table 12.3** are plotted for comparison purpose in **Fig. 12.18**. It may be noted that the severity of the plastic hinges do not exceed the life safety stage and hence only three categories of hinges are plotted. The A-B category is the elastic range, whereas B-IO is the category of hinges developed up to the immediate occupancy stage. The IO-LS signifies the plastic hinges developed beyond the immediate occupancy stage but less than the life safety stage.



(a) G+3 Frame with all RC beams

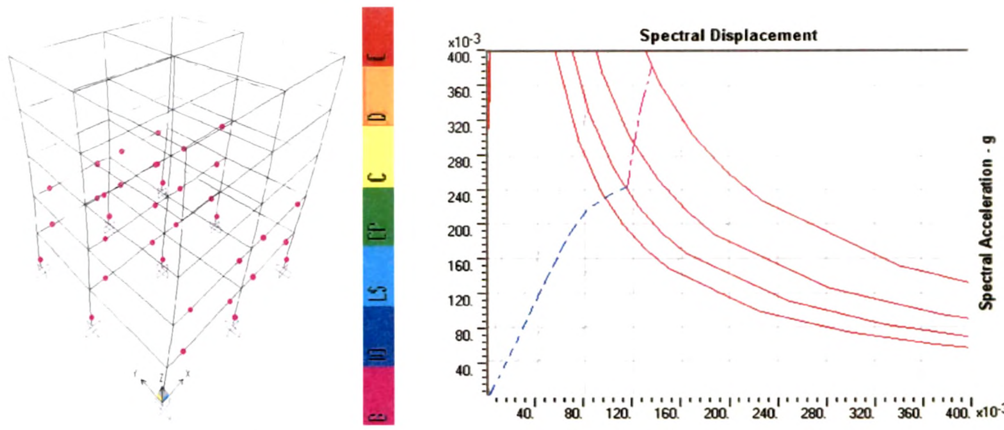


(b) G+3 Frame with PT and RC beams

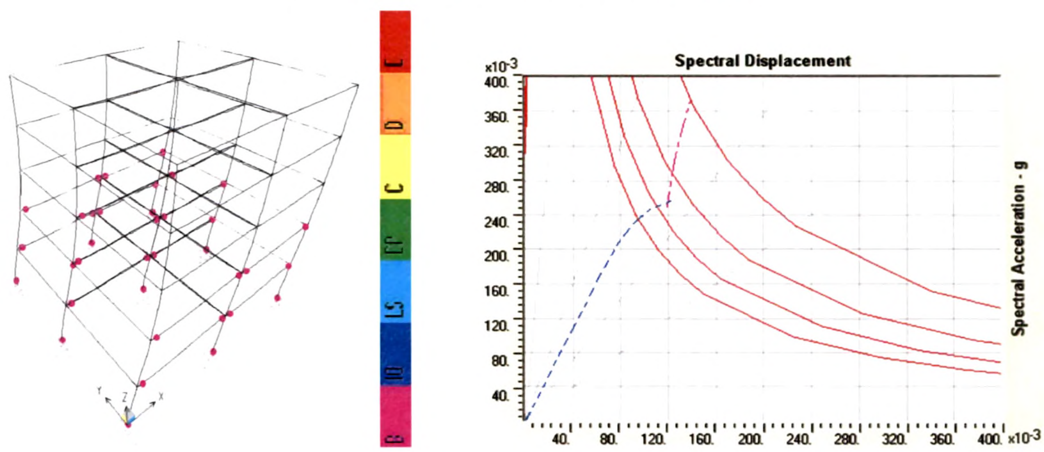


(c) G+3 Frame with all PT beams

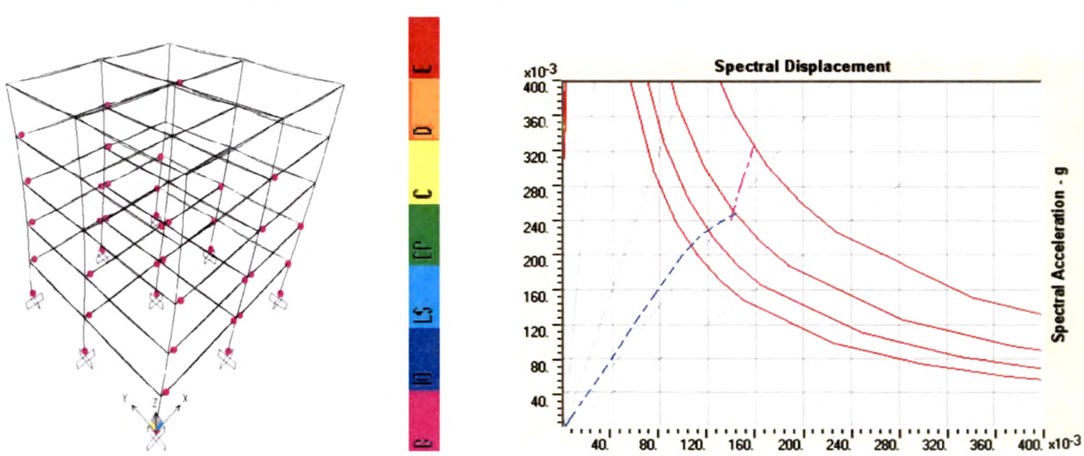
Fig. 12.8 Deformed Shapes at Performance Point for G+3 Frames



(a) G+4 Frame with all RC beams

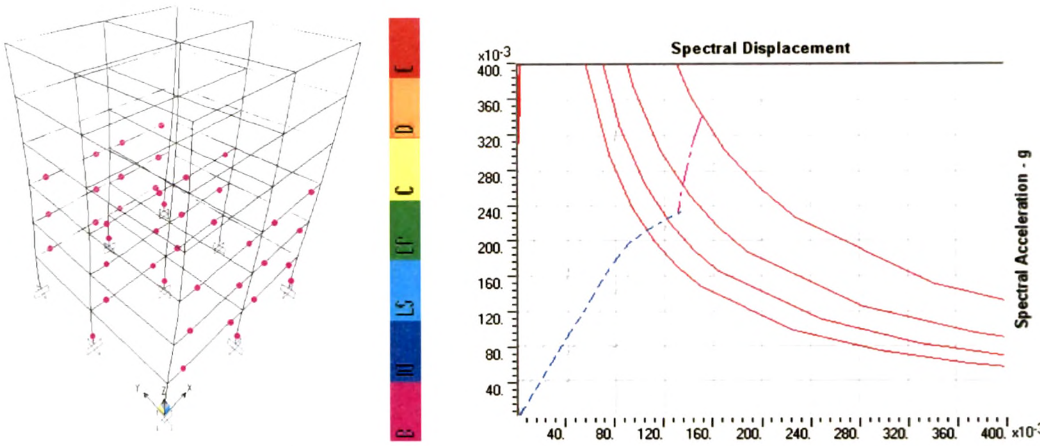


(b) G+4 Frame with PT and RC beams

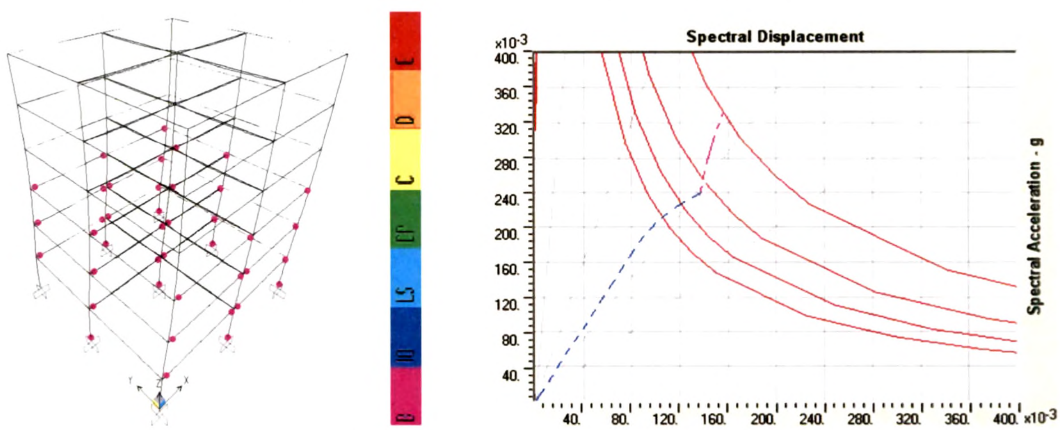


(c) G+4 Frame with all PT beams

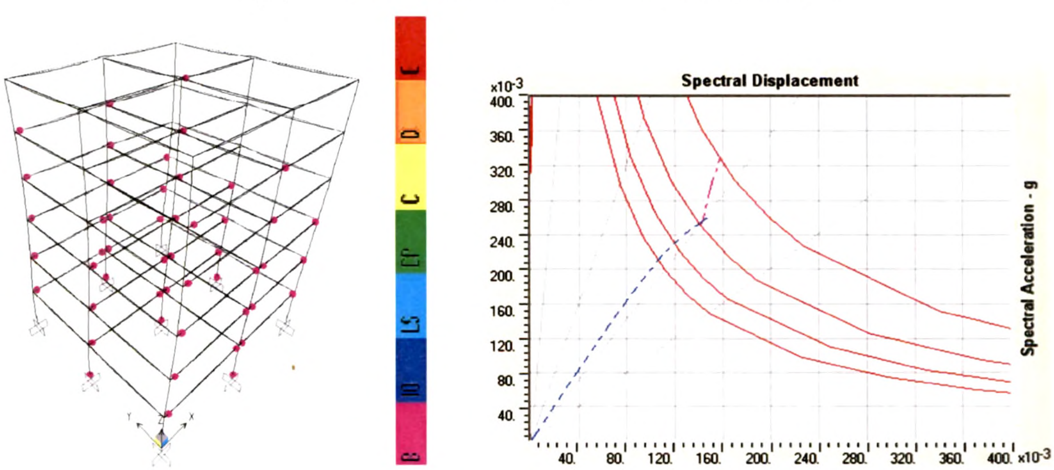
Fig. 12.9 Deformed Shapes at Performance Point for G+4 Frames



(a) G+5 Frame with all RC beams

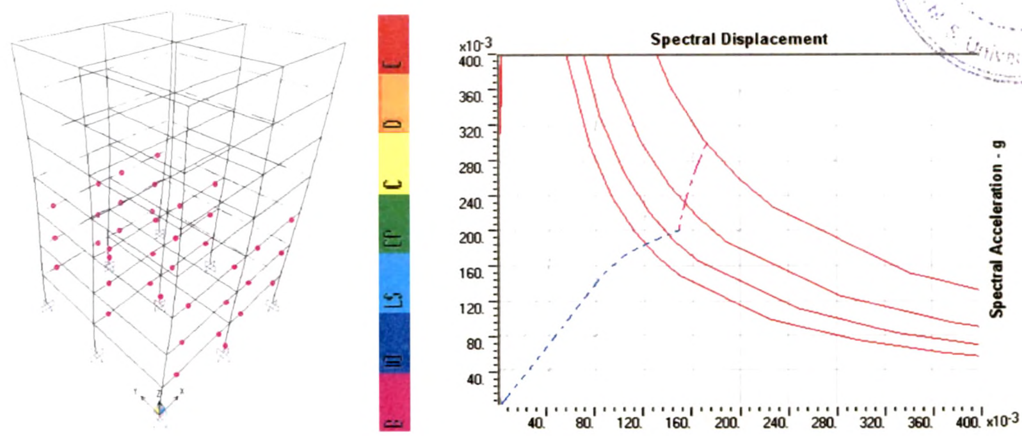


(b) G+5 Frame with PT and RC beams

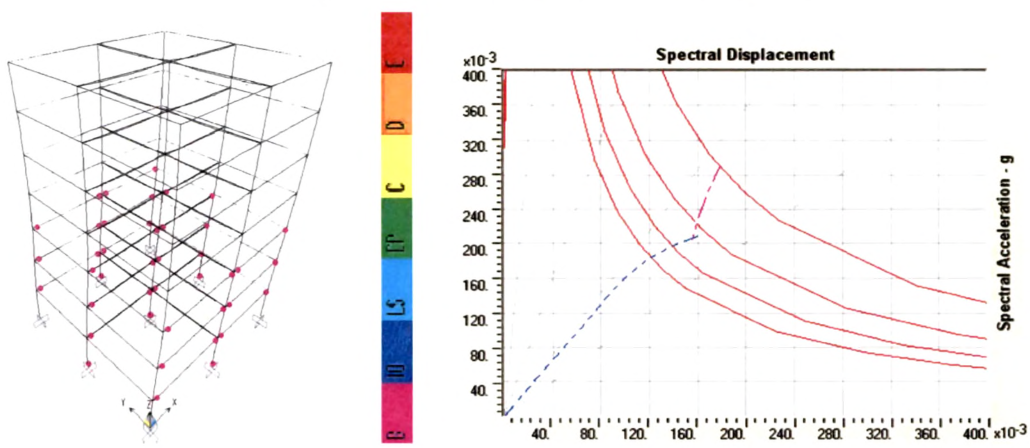


(c) G+5 Frame with all PT beams

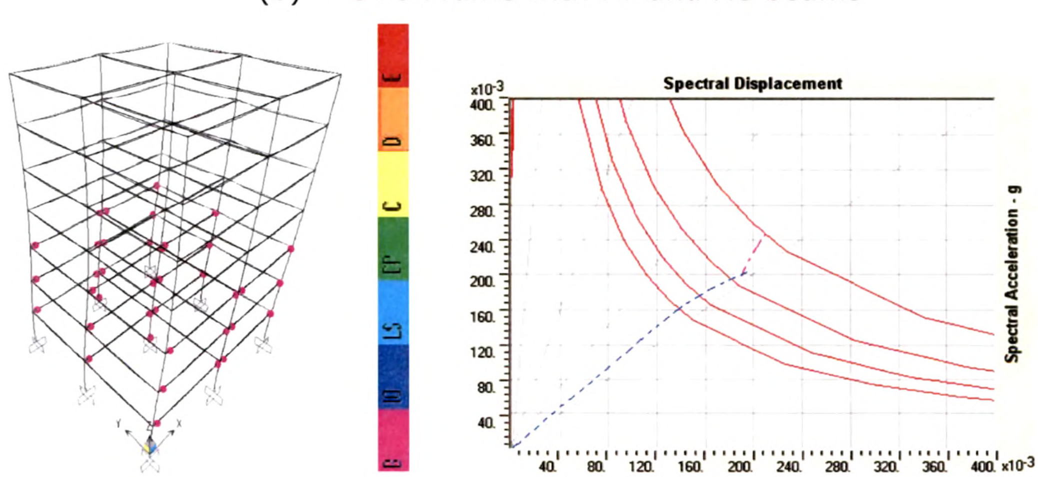
Fig. 12.10 Deformed Shapes at Performance Point for G+5 Frames



(a) G+6 Frame with all RC beams

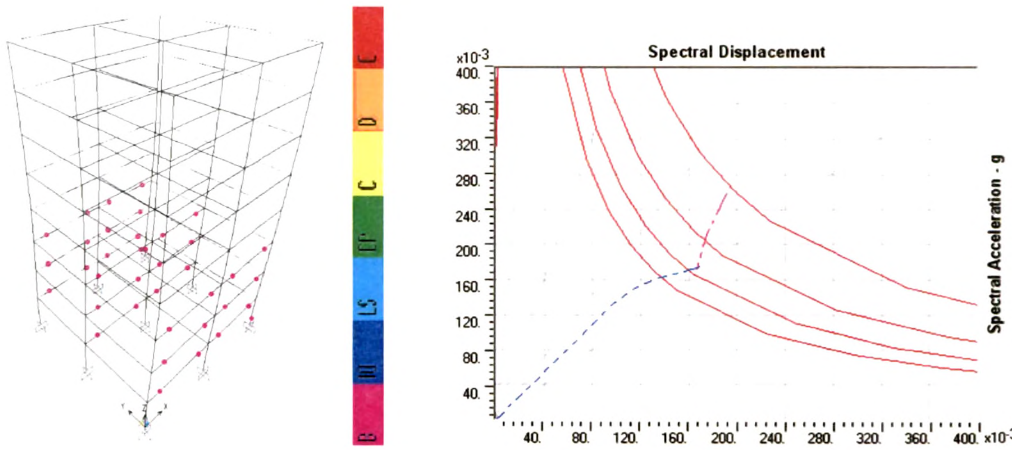


(b) G+6 Frame with PT and RC beams

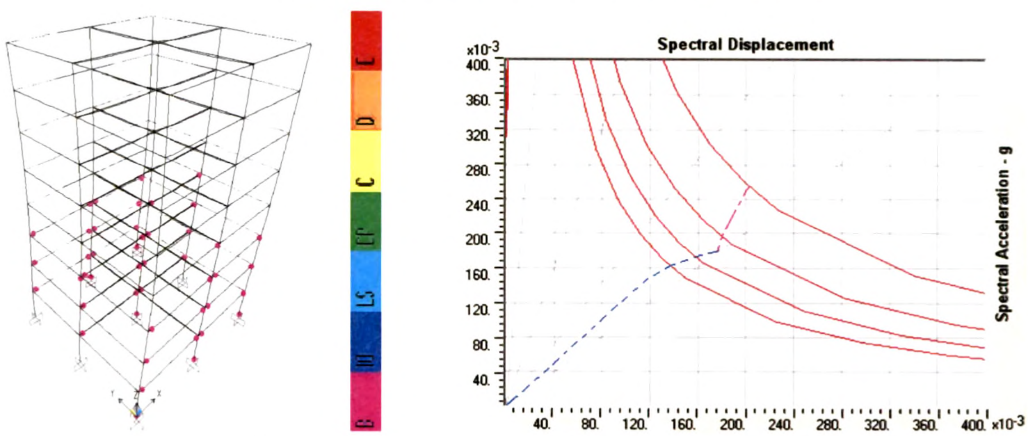


(c) G+6 Frame with all PT beams

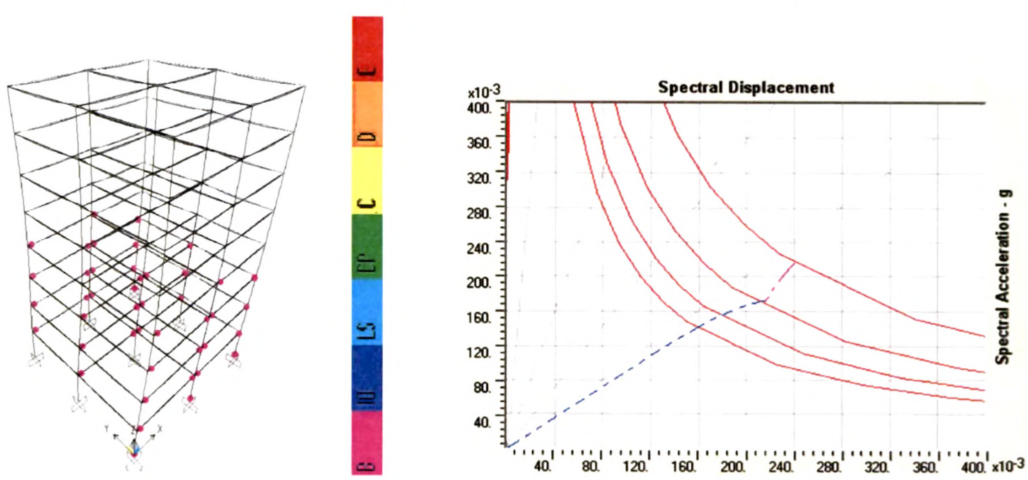
Fig. 12.11 Deformed Shapes at Performance Point for G+6 Frames



(a) G+7 Frame with all RC beams

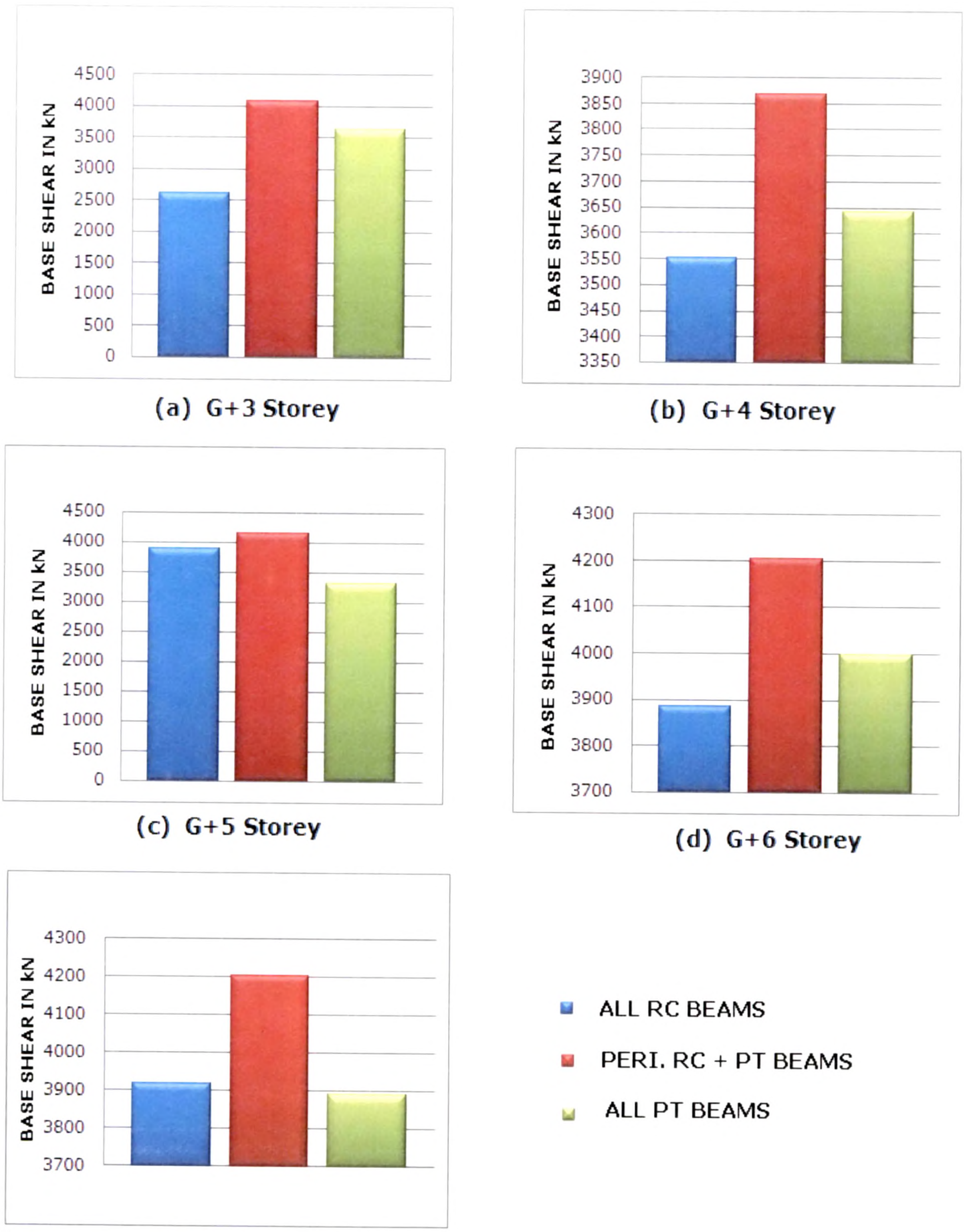


(b) G+7 Frame with PT and RC beams



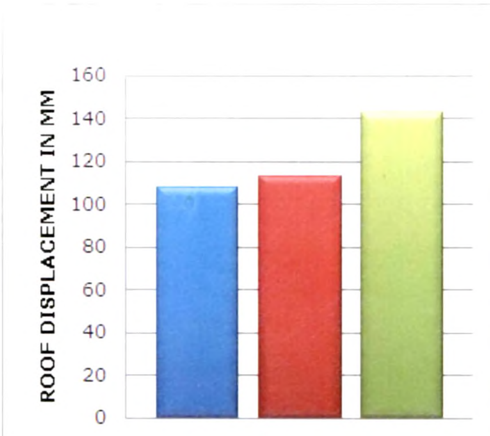
(c) G+7 Frame with all PT beams

Fig. 12.12 Deformed Shapes at Performance Point for G+7 Frames

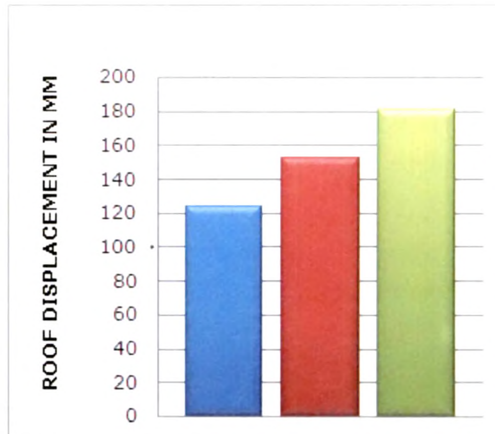


(e) G+7 Storey

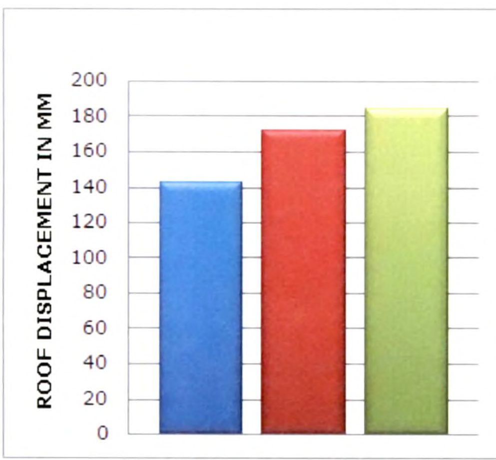
Fig. 12.13 Base Shear Variation at Performance Point



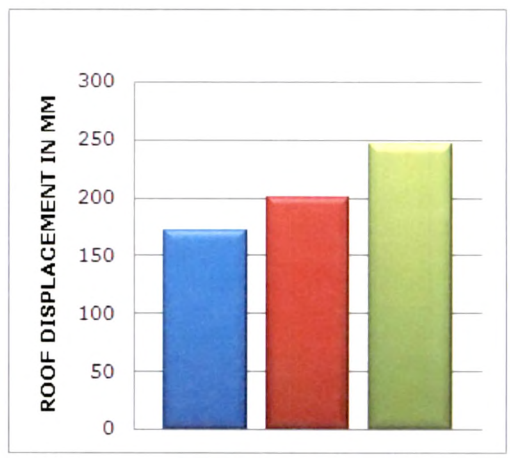
(a) G+3 Storey



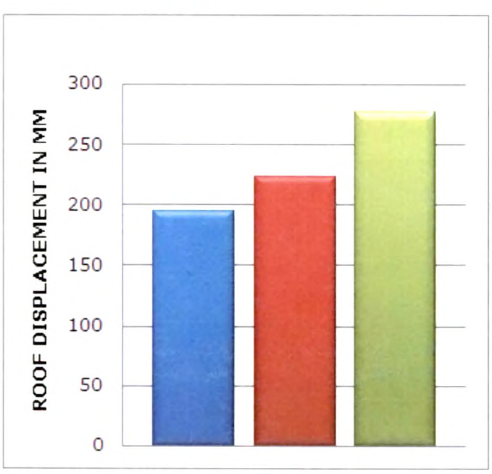
(b) G+4 Storey



(c) G+5 Storey



(d) G+6 Storey



(e) G+7 Storey

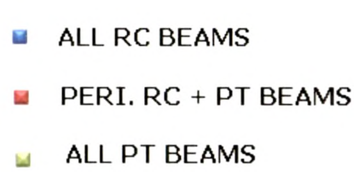


Fig. 12.14 Roof Displacement Variation at Performance Point

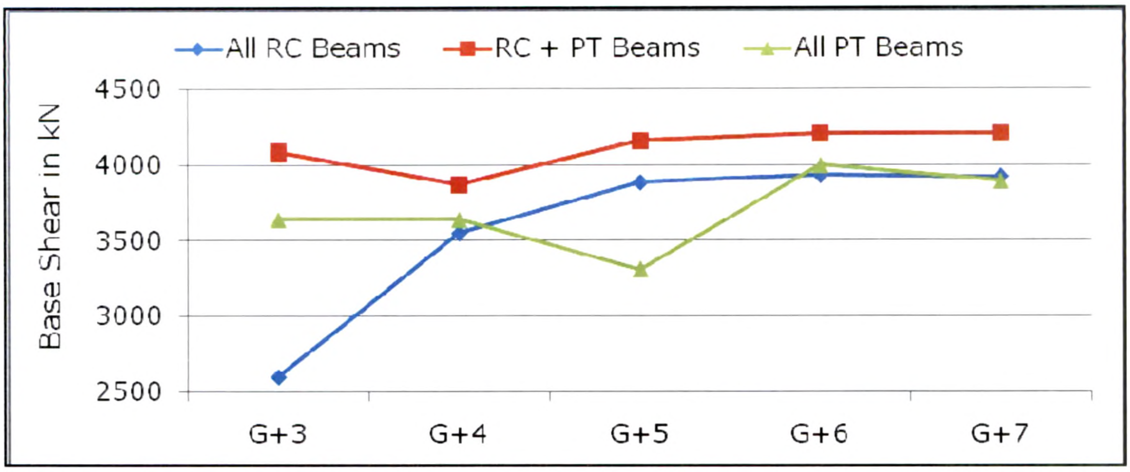


Fig. 12.15 Base Shear Variation at Performance Point

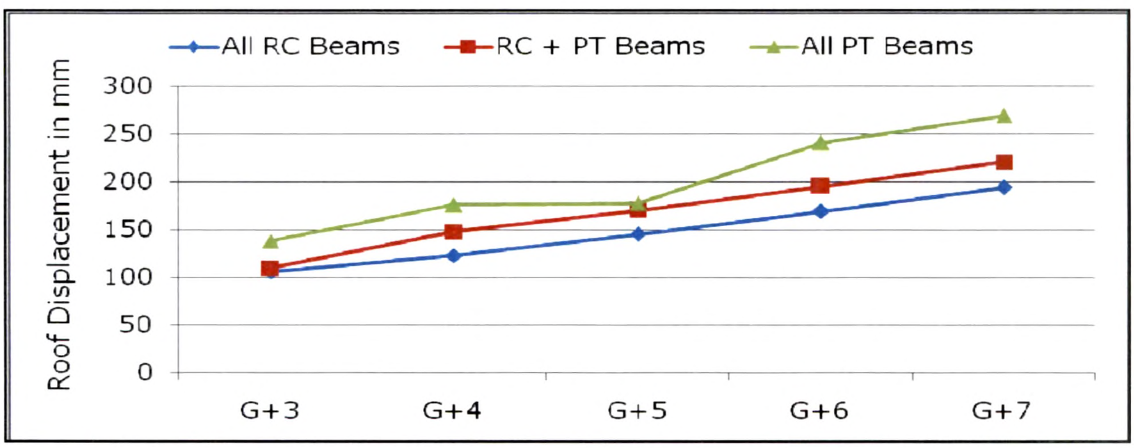


Fig. 12.16 Roof Displacement Variation at Performance Point

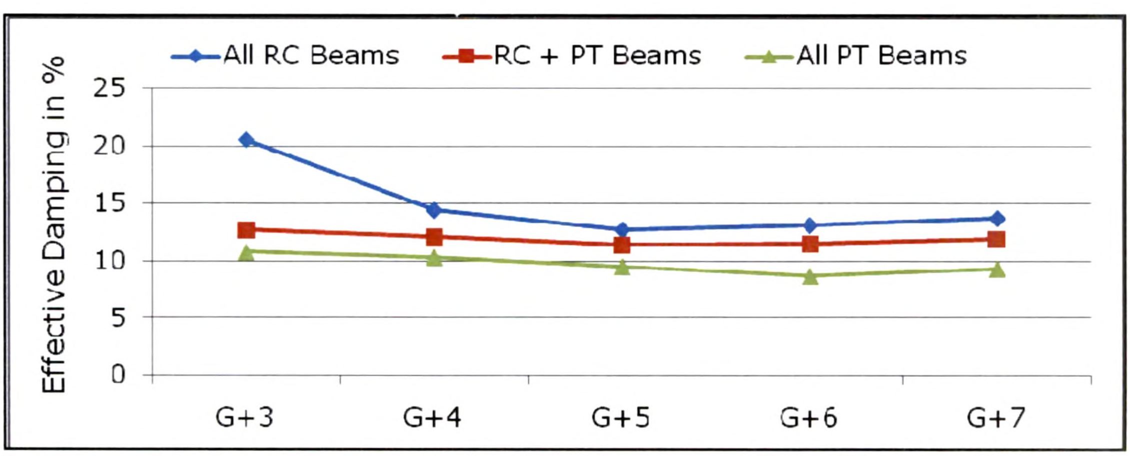
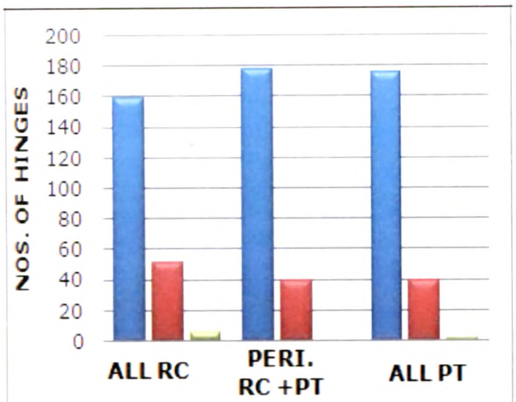
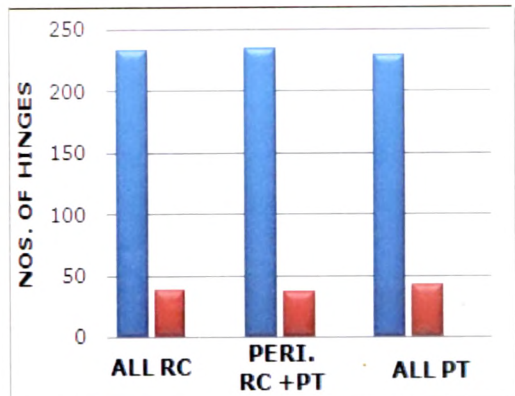


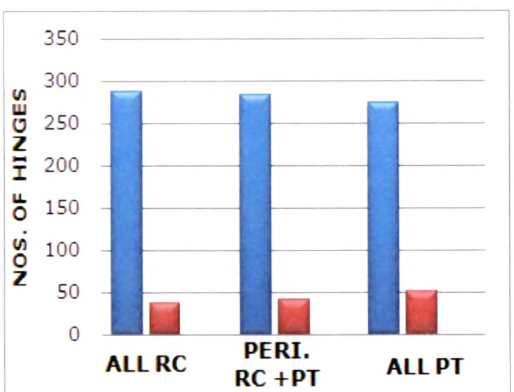
Fig. 12.17 Variation in Effective Damping at Performance Point



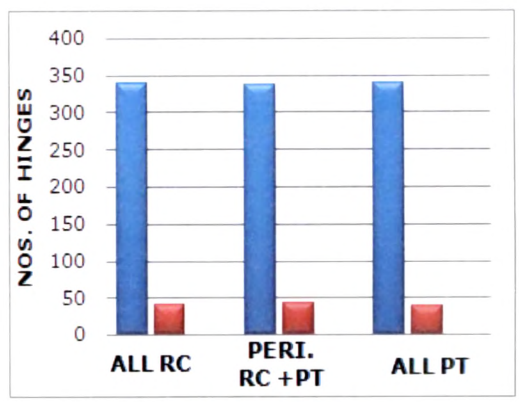
(a) G+3 Storey



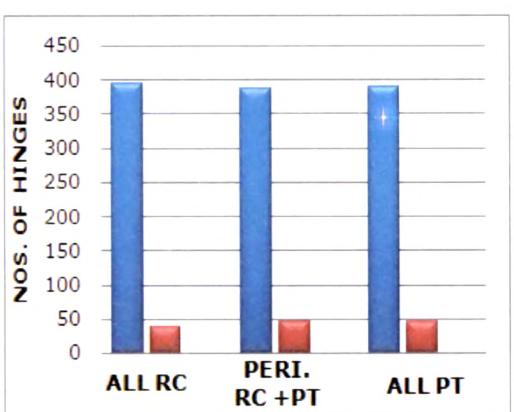
(b) G+4 Storey

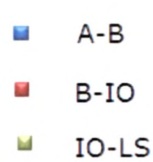


(c) G+5 Storey



(d) G+6 Storey





(e) G+7 Storey

Fig. 12.18 Number of Hinges Developed at Performance Point

12.7 THE COMPARISON OF RESULTS

It is clear from the results that when a comparison is made between a conventional RC frame and a building frame consisting of PT beams, there is no marked difference in the seismic performance for buildings up to G+7 storey. In fact, if a comparison of base shear is made, as shown in **Figs. 12.13** and **12.15**, it can be seen that a frame having peripheral RC beams and internal PT beams resist maximum base shear at performance point. Although the roof displacement in all the models ranging from G+3 to G+7 having internal PT beams is slightly higher than that having all the beams as conventional RC beams (**Figs. 12.14** and **12.16**), the seismic performance is quite good. This fact is well supported by the seismic performance of a hybrid frame as observed in chapters 10 and 11, where the internal beam's beam-column stiffness does not contribute much to the seismic resistance of the building frame. Thus, an RC frame with external conventional beams behaves like an external shell which resists major part of the seismic forces and the internal PT beams, which do not contribute much to the stiffness (as they have a shallow depth), are not forming a part of lateral force resisting system.

It is clear from the **Table 12.2** and **Fig. 12.17** that the effective damping at performance point for the models with peripheral RC beams and internal PT beams range from 11.4% to 12.7% for G+3 to G+7 storey frames. The value for effective damping for frames having all beams as conventional RC beams is as high as 20.5% for a G+3 structure indicating a higher stress value in the plastic hinges defined. This fact is also observed from **Table 12.3** and the corresponding **Fig. 12.18** which represents the number of hinges at various stress levels developed at performance point. **Table 12.2** indicates that the effective time period for the frames having all the beams as conventional RC beams and that having peripheral RC beams and internal PT beams for G+3 to G+7 storey structures are almost similar at performance point. It is also observed from both the tables that the building frames with peripheral RC beams and internal PT beams show a consistently good performance as compared to the other two types of frames having either all conventional beams or all PT beams.

12.8 OUTCOME OF THE STUDY

1. 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 G+3 to G+7 storey structures
2. The stress value in the plastic hinges in case of frames with peripheral RC beams and internal PT beams is observed to be within immediate occupancy stage for all defined hinges in G+3 to G+7 storey building frames. This indicates a consistently better seismic performance.
3. As the number of storey increases, the base shear at performance point remains almost constant for frames with perimeter RC beams and internal PT beams. Moreover, it is the highest compared to the other two types of frames, regardless of the number of storey.
4. The roof displacement at performance point is constantly increasing with the increase in number of storey. As seen in **Fig. 12.16** the graph line for roof displacement in case of all RC beams and RC + PT beams are almost parallel going from G+3 to G+7 storey frames. This shows a consistent seismic performance.
5. The effective damping at performance point is almost constant for all frames with a combination of PT and RC beams as seen in **Fig. 12.17** indicating one more parameter showing consistency.