PERFORMANCE BASED DESIGN OF REINFORCED CONCRETE PLANE FRAMES

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ABSTRACT :

The seismic analysis and design of reinforced concrete (RC) frames is still an unresolved issue due to its complex behaviour. Seismic design codes are traditionally based on the force-based approach wherein structures are designed with a certain minimum lateral strength. However, it has been observed that such an approach, which relates to the elastic response, does not produce consistent inelastic response in terms of the amount and distribution of damage in structural elements. In views of the above, the displacement based approach, also known as performance based design (PBD) approach, has been explored in recent times. In this approach, the prime response quantities of interest are the inter-storey drifts and the design process directly attempts to limit these drifts to an acceptable value. However, PBD is still not ready for codification as several issues need to be understood. These include the influence of parameters such as the strength, stiffness, energy dissipation capacity and detailing such as percentage of reinforcement and amount of confining steel on the local and overall damage. In this study, a small portion of the overall problem is studied extensively to arrive at meaningful conclusions which can be used to develop a suitable design approach. Over 700 RC regular frames of two, four and eight storeys, designed and detailed as per the Indian seismic codal provisions, are analyzed by varying the time period, response reduction factor and percentage of longitudinal reinforcement. Non-linear time history analyses for six different earthquake accelerograms are carried out using the Pivot hysteretic model. The response parameters considered are local and global ductility demand, local damage index and inter-storey drift index. Variation in the response parameters with time period, percentage of reinforcement and response reduction factor are presented in graphical form. It is shown that the percentage of reinforcement plays a major role in the seismic performance. Based on the study, a simple design procedure to implement performance based design is suggested.

KEYWORDS: Performance Based Design, RC frames, Non-linear time-history analysis, ductility demands, damage index, response reduction factors, storey drift

1. INTRODUCTION

Seismic design codes are traditionally based on the force-based approach wherein structures are designed with a certain minimum lateral strength. It is well known that seismic response is a function of not only of the strength but also of the initial stiffness, ductility and hysteretic energy dissipation capacity. The initial stiffness is taken into account by calculating the seismic force as a function of the time period using a response spectrum while the ductility is accounted for by considering a response reduction factor. However, it has been observed that such an approach, which relates to the elastic response, does not produce consistent inelastic response in terms of the amount and distribution of damage in structural elements. When the performance of non-structural elements is also considered, the force-based approach gives widely different results (Priestley 2000).

In view of the above, the displacement based approach, also known as performance based design approach, has been explored in recent years and found to give better results. In this approach, the prime response quantities of interest are the inter-storey drifts and the design process directly attempts to limit these drifts to an acceptable value. The only difficulty in adopting has been the complexity of the approach for practical design office use and so several 'simplified methods' have been suggested (Zou and Chan 2005). While these methods yield better results, they are still not ready for codification as several issues need to be understood. The issues which need clarification include the influence of parameters such as the strength, stiffness, energy dissipation capacity and detailing parameters such as the ratios of beam-to-column strength and stiffness, percentage of reinforcement and amount of confining steel on the local and overall damage. While the effect of each individual parameter on the response can be studied in isolation, understanding the effect of their combinations will require considerable study.

In this study, a small portion of the overall problem is studied extensively to arrive at meaningful conclusions which can be used to develop a suitable design approach. First the variation in the response of structures, designed by the force-based approach, is studied and the additional parameters which need to be considered are identified. By carrying out non-linear time-history analyses, the relationships between the design and performance parameters are clarified. It is shown that the percentage of reinforcement has a significant effect on the performance of the frame.

2. LITERATURE REVIEW

Panagiotakos and Fardis (1999) developed a procedure for estimating member inelastic deformations and storey drifts based on hundreds of nonlinear dynamic analyses on three, four and twelve storeyed RC plane frames. The frames were designed as per capacity design principles but with varying levels of ductility and were subjected to design spectrum compatible ground motions. Based on the results, it was reconfirmed that the well known equal displacement rule for SDOF systems gave good results. The cracked elastic time period itself was a function of the elastic stiffness, which has been recognized to be dependent not only on the cross-sectional dimensions but also on the percentage of longitudinal reinforcement (Priestley 2000). However, the ratios of inelastic to elastic chord rotations and drifts were obtained as an average for all the analyses and recommended as the multiplying factors for elastic values, to obtain the corresponding inelastic values. Thus, the design method proposed will require apriori knowledge about the member longitudinal reinforcement. As an extension to the above study, Panagiotakos and Fardis (2001) proposed a displacement based design procedure for RC frames and applied it to a four storey frame along with the usual procedure as per Eurocode 8. By testing frames designed by the two procedures, they showed that significant saving in steel can be obtained by using the proposed procedure in liew of the Eurocode procedure which follows prescriptive detailing rules similar to IS 13920 (1993).

Chandler and Mendis (2000) presented a case study for RC moment resisting frames, designed and detailed according to European and Australian earthquake code provisions. The authors used elastic perfectly plastic hysteretic model for non-linear time history analysis and a single earthquake accelerogram. The overall ductility demands have been computed for the force-based analyses, conducted on the typical design frame and also the performance of the case study frames has been re-evaluated in the light of displacement based principles. The authors concluded that both the displacement based and the force-based approaches give similar results.

Zou and Chan (2005) developed a computer-based technique that incorporates pushover analysis, together with numerical optimization procedure, to automate the performance based design of RC buildings. They considered percentage of reinforcement as a parameter during the design optimization process. However, it is difficult to provide the reinforcement ratio that has been suggested by them in practice. Cruz and Lopez (2004) also presented an iterative design method based on repeated pushover analyses which gives a desired damage level.

Mwafy and Elnashai (2001) assessed the validity and the applicability of the static pushover analysis to RC frames by comparing with results obtained from incremental dynamic analyses. They concluded that the two methods gave similar results for regular frames and the effect of higher modes was insignificant. In a similar study conducted on steel and concrete frames, Kunnath and Kalkan (2004) found that the pushover analysis results were sensitive to the load pattern used and generally tend to underestimate the demands in upper storeys due to higher mode effects.

The effect of hysteretic model on the dynamic response has been studied for a long time. In a more recent study, Lee *et al* (1999) used bilinear, strength degrading, stiffness degrading and pinching hysteretic models and over forty earthquakes and concluded that lower response reduction factors are obtained with increasing degradation and pinching. This underlies the importance of using a proper hysteretic model in evaluating the seismic performance.

3. MODELLING AND ANALYSIS OF RC FRAMES

Nonlinear time-history were carried out on RC frames to evaluate their seismic performance. Mass of each floor was calculated and equally applied at the two nodes for every floor. SAP2000 NL V9.1.4 was used for the nonlinear time-history analysis of RC frames. Geometric and material nonlinearities have been considered in the nonlinear analyses of RC frames. The modeling and analysis procedure was verified with results in published literature.

Regular two, four and eight storey plane frames were designed and detailed as per IS 1893(Part1):2002 and IS 13920:1993 for this study. Bay width and storey height for the frames are assumed as 4m and 3.5m, respectively. For calculating the dead and imposed loads, the frames are assumed to be spaced 3m apart and supporting reinforced concrete slab of thickness 120mm. A brick wall of 120mm thickness is assumed for calculating dead loads on the frame but the in-plane stiffness of the wall is not considered in the analysis. Dead loads and imposed loads on all the frames were calculated as per IS 875:1987. The floor masses were obtained as 99 kN except for the top floor in which case it was 74 kN for all the frames. The frames are assumed to be located in zone V, on rocky strata. A damping ratio of 5% is assumed. Equivalent lateral loads on frames were calculated as per IS 1893 (Part1):2002. The frames were designed to satisfy the load combinations stipulated in IS 1893 (Part1):2002.

For developing the moment versus rotation curve of a hinge, the stress-strain model for concrete subjected to uniaxial compression and confined by transverse reinforcement, as proposed by Mander *et al* (1988) and modified by Panagiotakos and Fardis (2001), was used. Beams and Columns were modelled with concentrated plastic hinges at the ends. Beams have both moment (M_3) and shear (V_2) hinges whereas, columns have axial load plus moment (P-M_3) hinges and shear hinges. The plastic hinge rotation and moment values corresponding to yield and ultimate states were calculated for each section and used to define the hinge properties. The monotonic moment rotation curve considered for the analysis is shown in Fig. 1.

For the nonlinear time-history analysis, the columns and beams were modelled as nonlinear plastic link elements (NLLink). The Pivot hysteretic model (Dowell *et al* 1998), which simulates stiffness degradation and pinching as shown in Fig. 2, was used to model the cyclic behaviour of the plastic hinges. The seismic performance of two, four and eight storey RC frames under six different earthquakes were obtained. The details of the earthquake accelerograms used are given in Table. 3.1. All the earthquakes were normalised to a peak ground acceleration of 0.36g, where, ‘g’ is the acceleration due to gravity. All the hinge properties are calculated according to FEMA 273 (1997) guidelines.

4. PARAMETRIC STUDY OF RC FRAMES

The force-based method considers the seismic performance of frames as a function of the strength (or response reduction factor R) and stiffness (or time period) and the properties of the ground motion, characterized by the spectral displacement. In addition to these, the percentage of reinforcement has been recognized recently as an important parameter in controlling the performance as observed in the literature. This parameter not only controls the stiffness but also determines the yield displacement and hence affects the ductility. In view of this, there have been suggestions to consider absolute drifts and deformations in place of ductility (Priestley 2000). However, since ductility has been given prime importance in traditional design, it is better to study the influence of the percentage of reinforcement on the ductility demand and this is one of the prime objectives of the present study. In view of these considerations, the design parameters considered are limited to strength (yield and ultimate), stiffness (or time period) and percentage of reinforcement while the response parameters considered are local and global ductility, roof and inter-storey drifts and damage indices. For parametric study, different values of response reduction factor R, (3, 5 and 7) and various percentage of reinforcement p_t (0.8%, 1.6% and 2.4%) were considered. The time periods T, considered were 0.3, 0.45, 0.6, 0.75 and 0.9 seconds, 0.5, 0.8, 1.1 and 1.4 seconds and 0.8, 1.3, 1.8 to 2.3 seconds for two, four and eight storey single-bay plane reinforced concrete frames respectively. Each frame is subjected to 3×3×6×T (3 R values, 3 percentages of reinforcement, 6 earthquake ground motions and Time periods). So a total of 3×3×6×5 = 270 two storey frames, 3×3×6×4= 216 four storey frames and 3×3×6×4 = 216 eight storey frames, giving a grand total of 270+216+216 = 702 frames, were analyzed.

In the present study, local rotational ductility demand (μ_L) is defined as the ratio of maximum rotation to yield rotation at any plastic hinge located at member ends. Global displacement ductility demand (μ_G) is defined as the ratio of maximum roof displacement to the roof displacement at first yield anywhere in the structure. The inter storey drift index (IDI) is defined as the difference in displacement of two consecutive floors divided by the storey height and is expressed as a percentage.

Seismic structural damage at the plastic hinge (local) can be expressed in terms of the damage index which is a linear combination of the maximum deformation and the hysteretic energy. The modified Park and Ang (1985) damage index is used on account of its simplicity and extensive calibration with experimentally observed seismic damage in reinforced concrete structures. The index is given by

$$\text{Local damage index}\,(DI_L) = (1-\beta)\left(\frac{\theta_{\max}-\theta_y}{\theta_{ult}-\theta_y}\right) + \beta\left(\frac{\int dE}{M_y\theta_y}\right) \tag{4.1}$$

Where, β = Coefficient for cyclic loading effect (function of structural parameters) (0.05 for RCC), dE = Incrementa absorbed hysteretic energy and M_y = Yield moment.

5. RESULTS AND DISCUSSIONS

The results are presented as variations of the response parameters as functions of the system parameters. All the graphs are drawn for mean plus standard deviation of six earthquakes. For want of space, only the time history analysis results for the 4-storey frames are presented. The results for two and eight storey frames have shown similar trends.

5.1 Variation of Local Ductility Demand (μ_L)

The local ductility demands need to be limited to the ductility capacity of the cross-section used. While the effect of response reduction factor (or strength) and the time period as obtained from the elastic stiffness are well known, the reinforcement percentage also plays an important role. This is primarily because the three parameters are related in practical design as a reduction in strength invariably involves a reduction in stiffness and consequent change in the percentage of reinforcement. For a given cross section, the change in percentage of reinforcement also results in a change in the yield displacement. Also, the ductility capacity of reinforced concrete sections depend to a large extent on the percentage of reinforcement and higher percentages produce a drastic reduction in ductility.

In general, the seismic demands increase with increase in R factor as design seismic base shear capacity is reduced. The flexibility of the frame is increased with increase in p_t as geometrical cross section decrease with increase in p_t for a given strength. The seismic demand on any frame also depends on time period of the frame. In general, the displacement related demands increase with increase in time periods. Time period can be varied independently by simply varying the mass.

The variation of local ductility demand with increase in time period at various values of response reduction factor R and percentage of reinforcement p_t is shown in Fig. 5.1. Local ductility demands decrease with increase in percentage of reinforcement as the stiffness to strength ratio is low at high percentage of reinforcement.

5.2 Variation of Local Damage Index (DI_L)

The variation of local damage indices with increase in time period at various values of response reduction factor R and percentage of reinforcement p_t is shown in Fig. 5.2. It can be observed that the damage indices increase with increase in percentage of reinforcement as ultimate rotation capacity of the member decreases with increase in percentage of reinforcement.

5.3 Variation of Global Ductility Demand (μ_G)

Variation of global ductility demand μ_G with increase in time period at various values of R factor for the four storey frame is shown in Fig. 5.3. It can be observed that the global ductility demands linearly increase with increase in time period for various percentages of reinforcement. Frames with higher percentage of reinforcement undergo relatively higher yield rotations when compared with frames having lower percentage of reinforcement, for a given strength.

5.4 Variation of Dynamic Base Shear Factor (DBF)

The Dynamic Base shear Factor DBF is defined as the ratio of maximum inelastic base shear attracted by the frame under a given earthquake excitation to that of corresponding design seismic base shear (i.e. $V_B = A_h W$). Thus it is similar to the overstrength factor.

The dynamic base shear was obtained for the frames by adopting non-linear time-history analysis under six different earthquakes for different values of R factor and p_t. Fig. 5.4 shows the variation of DBF with time period for various values of R factor for the four storey frame. It can be observed that the DBF increases linearly with increase in time period of the frame and the R factor. This is because in the force based method, frames with higher time periods are designed with higher R factors.

5.5 Variation of Inter-Storey Drift Index (IDI)

The maximum drift is used to define the state of distress to the structure. Maximum inter storey drift for each storey need not occur at the same instance during the seismic loading. The inter storey drift depends on stiffness of moment resisting frame followed by plastic deformation and the mode shape of vibration. Variation of inter-storey drift index (IDI) with time period at various percentages of reinforcement is shown in Fig. 5.5 for the four storey frame. It is observed that the inter-storey drift index increases with increase in R factor and/or percentage of reinforcement. It is also observed that the inter-storey drift index increases linearly with increase in time period.

6. DESIGN RECOMMENDATIONS

From the above results, it may be concluded that the performance of code-compliant frame is not consistent for various values of time periods due to the use of constant R factor. Though the performance based design is accepted, adequate design guide lines are not available for seismic design of reinforced concrete frames. The performance of the frames can be improved by choosing smaller strength reduction factors for larger periods and by choosing more appropriate percentage of longitudinal steel. However, consistent performance may not be possible over the range of time periods due to limitations on the minimum strength required and the percentage of longitudinal steel.

7. CONCLUSIONS

1. Frames designed and detailed as per current codal procedures give widely different performance. This underscores the need to revise the design procedure.
2. The percentage of longitudinal reinforcement has considerable effect on the seismic performance and so must be considered in performance based design.
3. The relationships obtained between the response parameters namely ductility, drifts and damage indices with system parameters such as time period, response reduction factor and percentage of longitudinal reinforcement, can be used to achieve desired performance.

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Table 3.1 List of earthquakes used for analysis

S.No	Designation of Earthquake	Date	Name of Earthquake	Location	Magnitude	Peak Ground Acceleration (PGA) * (g)
1	EQ1	September 20,1999	Chi-Chi	Chi-Chi, Taiwan	7.6	0.364
2	EQ2	**October 17, 1989**	Loma Preita	California, USA	6.9	0.400
3	EQ3	January 16, 1995	Kobe	Kobe, Japan	6.9	0.821
4	EQ4	January 17, 1994	Northridge	Los Angeles, USA	6.7	0.568
5	EQ5	January 26, 2001	Bhuj	Ahmedabad, India	7.9	0.11
6	EQ6	May 18, 1940	Imperial Valley	El Centro, California, USA	7.1	0.319

***** All the earthquakes are normalised to a PGA of 0.36g.

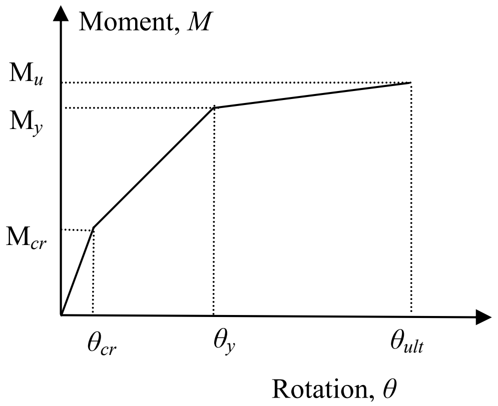


Fig. 1 Moment Rotation Curve

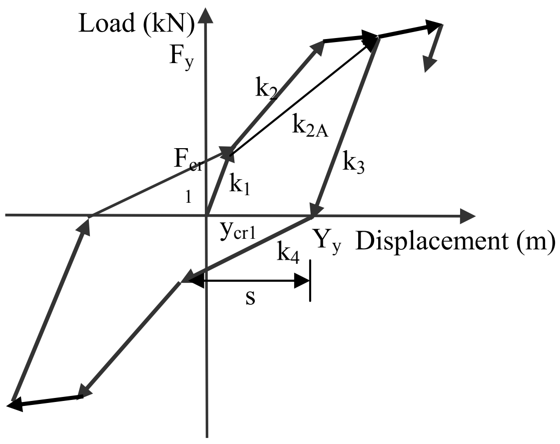
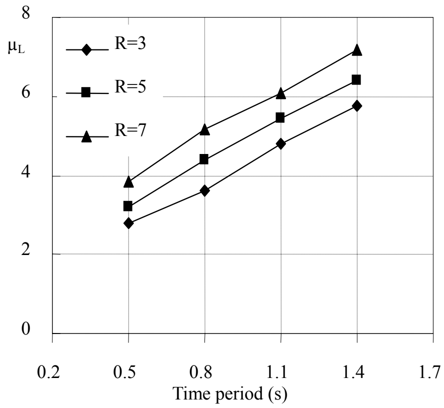
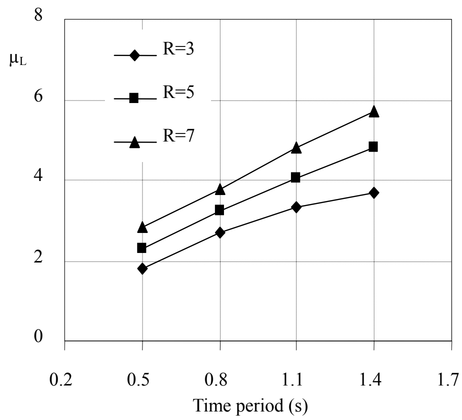


Fig. 2 Multi-Linear Hysteretic Model

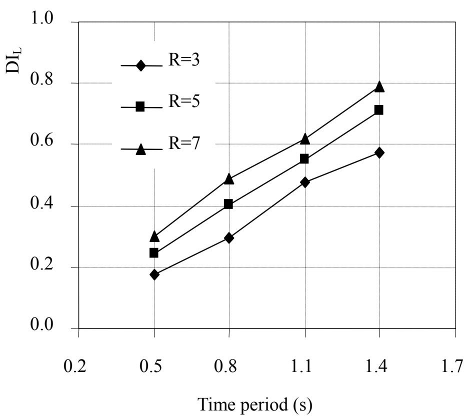


(a)

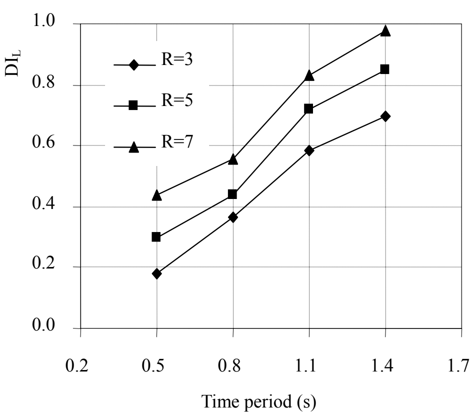


(b)

Fig. 5.1 Variation of local ductility demand *(a)* p_t=0.8% (b) p_t=2.4%

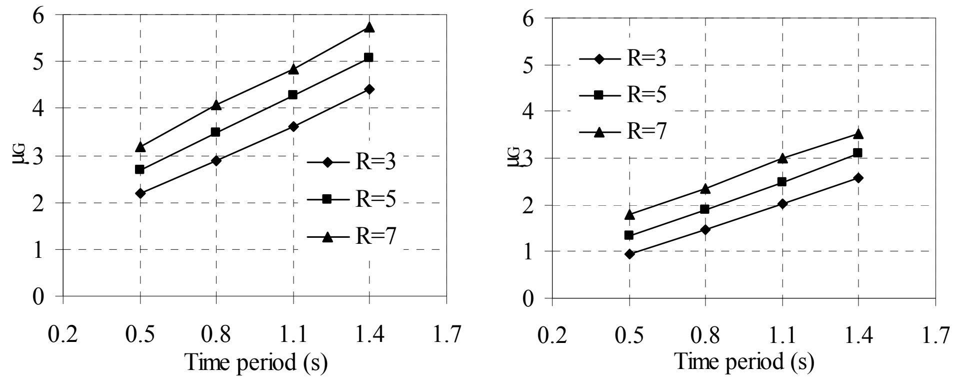


(a)



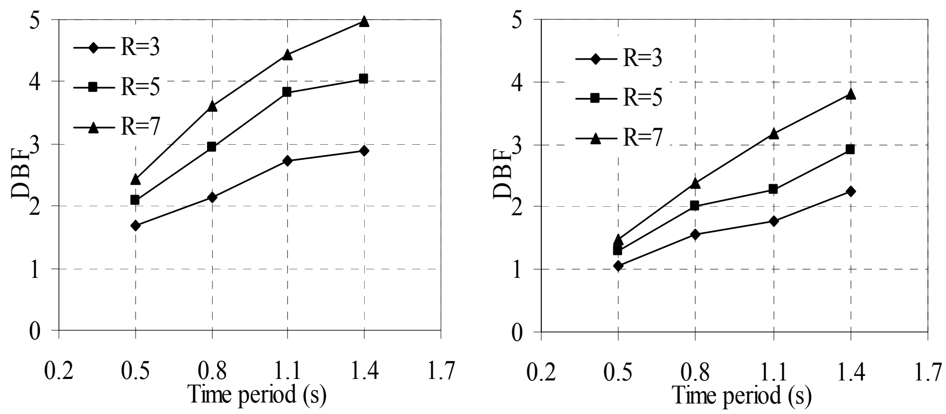
(b)

Fig. 5.2 Variation of Local Damage Index *(a)* p_t=0.8% (b) p_t=2.4%



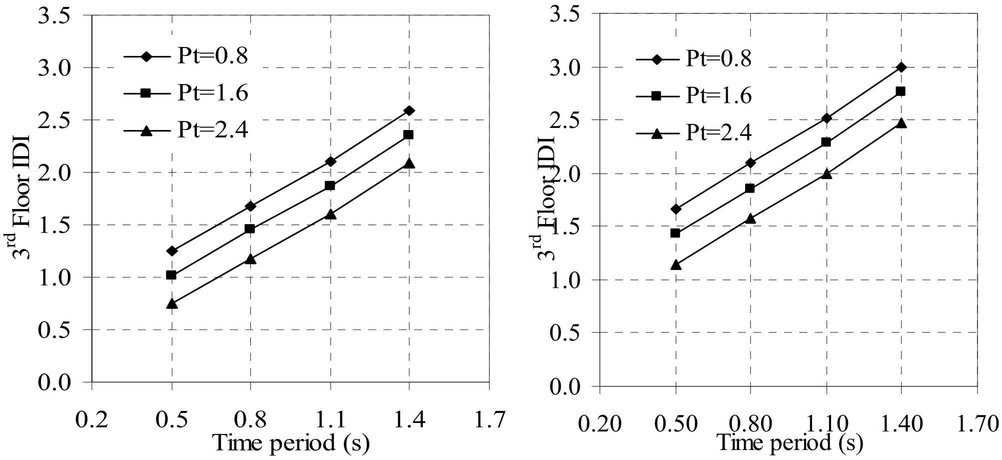
(*a*) (*b*)

Fig. 5.3 Variation of Global Ductility Demand (*a*) p_t=0.8% (*b*) p_t=2.4%



(*a*) (*b*)

Fig. 5.4 Variation of Dynamic Base shear Factor (*a*) p_t=0.8% (b) p_t=2.4%



(*a*) (*b*)

Fig. 5.5 Variation of Inter-storey Drift Index (*a*) R = 3 (*b*) R = 7