A Study on Dynamic Shear Amplification in RC Frame Structures

Uday Sinha*and Satyabrata Choudhury**
*PG scholar, Department of Civil Engineering, NIT Silchar;
Email: udaysinha58@gmail.com
**Professor, Department of Civil Engineering, NIT Silchar;
Email: scnitsilchar@gmail.com (corresponding author)

Abstract: In this paper a study has been carried to know the dynamic shear amplification in the structures. Unified Performance-Based Seismic Design (UPBD) has been used to design the buildings where both target drift of buildings is considered along with the performance level. Two different buiding with different plan is used. The performance objectives of the buildings are selected in terms of interstorey drift ratio (IDR) and member performance level. The target performance objectives for the two buildings considered are 2% IDR with Life Safety (LS) performance level, and 3% IDR with Collapse Prevention (CP) performance level. The buildings have been modeled using SAP2000 V 14.0.0 software. The performance parameters have been evaluated by performing nonlinear analysis.

Keywords: Unified Performance-Based Design (UPBD), Interstorey Drift Ratio (IDR), Reinforced concrete frames, Building performance level, Dynamic shear amplification.

Introduction

With rise in population, the importance of tall structures come into effect to accommodate many number of people with lesser land use. Therefore, it is important to design the tall structural adequately keeping in mind the natural hazard to which it is surrounded and one such hazard are the earthquakes. Seismic design of buildings has been traditionally force based. In the force-based codal method of design, the base shear is computed based on perceived seismic hazard level, importance of the building and probable reduction in demand due to nonlinear hysteresis effects. The computed base shear is distributed at floor levels with some prescribed or estimated distribution pattern. Through force based method of design, an engineer cannot deliberately design structure for an intended performance level. The alternative approaches are displacement-based design (DBD) and performance-based design (PBD) which are gradually becoming popular in recent times. In these methods the design is done for intended displacements or, some intended performance objectives under a perceived hazard level. The DDBD (Direct Displacement Based Design) method for frame buildings by Pettinga and Priestley is evolved out of DBD method. In this method, the building is converted to an equivalent single degree of freedom (ESDOF) system and the design is aimed at satisfying some specified interstorey drift limit. If the required interstorey drift is not satisfied, the trial interstorey drift is reduced and redesign is done so that finally the target interstorey drift may be achieved. In this method, however, the member performance levels are not considered. Choudhury and Singh in 2013 reported a method which is an improvement towards the DDBD method for frame buildings, which tried to satisfy the target drift and performance level, and christened it as unified PBD (UPBD) approach. In this paper buildings are designed with UPBD method for desired target performance objectives and the dynamic behaviour of the buildings is observed from which dynamic amplification of shear force is studied.

Literature studies

Priestley and Kowalsky (2000) presented direct displacement based seismic design of concrete buildings in which design was done to achieve a specified damage level under specified earthquake. This method computed the required base shear to achieve specified damage level state.

Pettinga and Priestley (2005) applied direct displacement-based design method to six RC frame structures and evaluated the interstorey drift ratio which exceeded assumed drift limit through nonlinear time history analysis. They proposed a revised form of the modified modal superposition to account for higher-mode amplification of column shear forces, while a simple intensity dependent scaling factor to be applied in the capacity design process was developed for column bending moments.

Nilupa Herath, Priyan Mendis, Tuan Ngo, Nicholas Haritos (2010) presented a study on the seismic performance of super tall buildings. In the study the effect of higher modes on the performance of super tall buildings is discussed and current methods of analysis of super tall buildings are reviewed in the study.

Priestley (2000) presented performance based seismic design to develop an alternative approach to achieve an expected strain or drift performance level under an expected seismic hazard level.
Choudhury and Singh (2013) has improved Pettinga and Priestley (2005) method to incorporate both member performance level and drift limit simultaneously in displacement-based design and reported a new UPBD method.

Methodology

Design Philosophy

The Unified Performance Based Design (UPBD) method as stated earlier is an improvement over the DDBD method in which both the target drift and performance level of the building is taken care of. In this method the design of the structure can be done for a target performance objectives in terms of interstorey drift ratio (IDR) and member performance level. The member performance level is expressed in terms of plastic rotation allowed in beams corresponding to the desired damage state. Weak-beam strong-column concept is followed thereby allowing plastic hinges to develop only in beams and not in columns and hence columns remain elastic up to the performance point
Target objectives in the form of design drift and member performance level is achieved by estimating a design beam depth by the following considerations:
The frame yield rotation θ_{yF} is given by Eq. (1) in which ε_y is yield strain of rebar, l_b is length of beam and h_b is depth of beam.

$$\theta_{yF} = 0.5\varepsilon_y l_b / h_b \tag{1}$$

Eq. (1) gives the maximum elastic angular drift of a frame building. This includes the contribution of beam rotation, joint rotation, column end displacement and shear deformation. The design angular drift θ_d of the system is the sum of frame yield rotation and plastic rotation in the system (θ_p), as expressed by Eq. (2) and explained in Fig. 1. In Fig. 1, H_e effective height of ESDOF system. With capacity design, only beam is allowed to yield and plastic rotation comes from beams (that is, $\theta_p = \theta_{pb}$), where θ_{pb} is the plastic rotation in beams.

$$\theta_d = \theta_{yF} + \theta_{pb} \tag{2}$$

Substituting Eq. (1) in Eq. (2) and rearranging, Eq. (3) is obtained.

$$h_b = \frac{0.5\varepsilon_y l_b}{\theta_d - \theta_{pb}} \tag{3}$$

Eq. (3) gives a beam depth that shall satisfy the interstorey drift and target performance level of the building corresponding to plastic rotation allowed in beams. Here, θ_{pb} is average plastic rotation in beams corresponding to the performance level desired, and can be obtained from FEMA-356

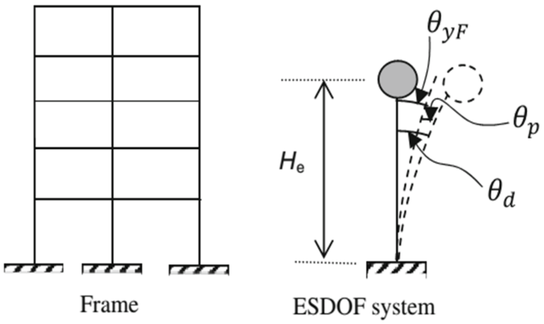


Figure 1. Equivalent SDOF system (Source: Choudhury and Singh)

UPBD method suggests that IO performance level can be combined with design drift from 1 to 1.5 %. It has been further assumed that the LS performance level can be combined with drift from 1.5 to 2.5 % and that the CP performance level can be combined with drift beyond 2.5 %. Eq. (3) can be used to express ratio of beam depth to beam length (h_b/l_b) for various drifts and performance levels (i.e. beam plastic rotations)
The design objectives include interstorey drift limit and member performance level. After choosing the of design objectives, suitable beam depth is obtained. The column sizes are chosen by trial such that the column steel from demand imposed lies approximately from 3 to 4 % of gross section area of column. The other steps of design are vastly after DDBD method of Pettinga and Priestley. The following steps are taken in designing as furnished below:

1. The target design drift and performance level of the building are decided. The beam depths (from h_b/l_b ratio) are found out from Eq. (3). The beam width has been kept from one-third to half of beam depth as per general design practice

2. The column sizes are preliminarily adopted from experience. In final design stage the column sizes are so adjusted that the column steel from demand imposed is restricted from 3 to 4 % of column sectional area.
3. The ESDOF system properties are determined,

$$\Delta_d = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i}, m_e = \frac{\sum m_i \Delta_i}{\Delta_d}, H_e = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i} \quad (4)$$

Here m_i, h_i and Δ_i are respectively the mass, height from base and displacement for *i*-th storey, Δ_d is target (spectral) displacement, m_e is equivalent mass, He is the effective height of the ESDOF system

4. The displacement spectra corresponding to design acceleration spectra are generated for various damping. This is the specified hazard level for design. Here, EC-8 design spectra of 0.45g level is used for type B soil.
5. The damping in the system is computed from ductility as given below. The yield displacement (Δ_y) of ESDOF system, frame ductility (μ) and equivalent effective damping (ξ) in the system is obtained from the following equations,

$$\Delta_y = \theta_{yF} H_e, \mu = \frac{\Delta_d}{\Delta_y}, \xi = 5 + 120\left(\frac{1-\mu^{-0.5}}{\pi}\right)\% \quad (5)$$

6. The design base shear is computed as detailed below. The effective time period (Te) is obtained from displacement spectra corresponding to the curve for equivalent damping ξ and the value of target displacement Δ_d. Effective stiffness for ESDOF (K_e) system and base shear (V_b) is given as follows

$$K_e = 4\pi^2 \frac{m_e}{T_e^2}, V_b = K_e \Delta_d \quad (6)$$

7. The base shear is now distributed at floor levels as per the equation given below. If the building is more than 10-storey high, then to take into account the effect of higher modes, F_t is typically 10 % of base shear put at roof level.

$$F_i = V_b \frac{\Delta_i m_i}{\sum \Delta_i m_i}, \; F_i = F_t + V_b \frac{\Delta_i m_i}{\sum \Delta_i m_i} \quad (7)$$

8. The design is done with expected (mean) strengths of materials. As per FEMA-356 provisions the expected strength of concrete is 1.5 times of the 28-days characteristic strength and, that for steel is 1.25 times the yield strength of rebar. The load combinations are as below:

   $D+L$, $D+L\pm F_x$, $D+L\pm F_y$

   here, D is the dead load, L is the live load, Fx and Fy are storey forces in mutually perpendicular directions.
9. After designing, the performance of the building is checked through Non-linear analysis.

Modelling Aspects

Two reinforced concrete frame buildings have been modeled using SAP2000 v.14.0.0. The building frames have been assumed to stand on fixed supports. The floor slabs have been modelled as rigid diaphragms. As the stiffness is proportional to strength, the effective stiffness values of the members is evaluated after design stage and incorporated in the model for nonlinear analyses. The yield moments for beam sections are obtained from SAP2000 corresponding to design steel. The effective beam flexural rigidity is given below, in which E is modulus of elasticity of concrete, I_{eff},beam is effective moment of inertia of beam, M_{yb} is beam yield moment and ϕ_{by} is yield curvature of beam. Effective flexural rigidity of column is given below, in which I_{eff},column is effective moment of inertia of column, M_{cy} is column moment capacity. M_{cy} is read from interaction diagram of column constructed at expected strength level, corresponding to gravity axial load in the column. Yield curvature of column ϕ_{yc} is given below, in which h_c is column depth in the direction of earthquake under consideration.

$EI_{eff,beam} = \frac{M_{by}}{\phi_{by}}$, $\phi_{by} = 1.7\frac{\varepsilon_y}{h_b}$, $EI_{eff,column} = \frac{M_{cy}}{\phi_{cy}}$, $\phi_{cy} = 2.1\frac{\varepsilon_y}{h_c}$

The default hinge properties available in SAP2000 v.14.0.0 has been used for both column and beam members. Consequently the post-elastic force deformation behaviour for the members has been adopted in the modelling as per FEMA-356 (2000).

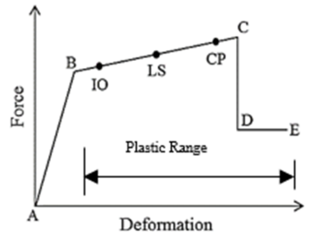


Figure 2. Typical force-deformation behaviour as per FEMA-356 (2000)

Building description

The RC frame buildings have been designed as per UPBD method. Two different type of plans are used.P1 denotes plan 1 and P2 denotes plan 2. The grade of concrete is taken as M25 and grade of steel as Fe500. The target objectives, member size considered and design parameters are given in the following tables.

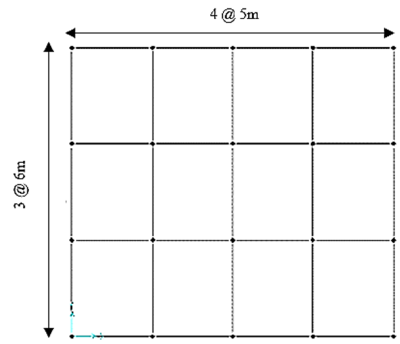


Figure 3. Plan 1

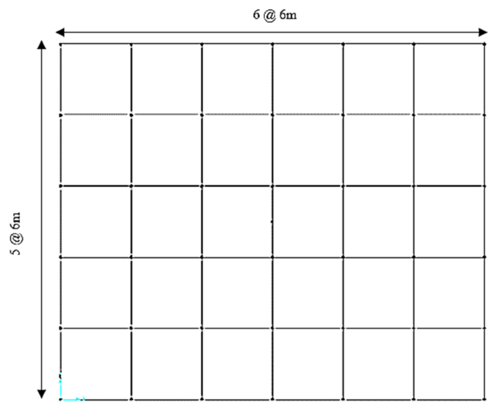


Figure 4. Plan 2

Table 1. Building name and design considerations

Building name	No. of storeys	Target performance objectives	
		θ_d	Performance level
P1_10_CP	10	3%	CP
P2_12_LS	12	2%	LS

Table 2. Member sizes of the building considered

Building name	Column size (mm x mm)	h_b/l_b		Beam size (mm x mm)	
		Upper bound	Lower bound	Long Direction	Short Direction
P1_10_CP	575 X 575 to 625 x625	0.13	0.04	450 x 250	540 x 300
P2_12_LS	675 x 675 to 925 x 925	0.14	0.06	750 x 400	750 x 400

Table 3. Design parameters for the building

Design parameters	Building considerd	
	P1_10_CP	**P2_12_LS**
Θd (%)	0.03	0.02
Δd (m)	0.5475	0.4316
Me (kg)	4259.69	16012.2
He (m)	21.94	25.99
μ	1.79	1.66
ξ (%)	14.7	13.55
Te (sec)	4.35	3.48
Ke	8887.08	52197.9
Vb (KN)	4865.85	22527.7

The base shear (Vb) obtained is then distributed through the storeys and the storey shear forces Fx and Fy are obtained and the building is designed. After designing performance check is done through non-linear analysis.

Analysis and results

Non linear static analysis (Pushover case)

Nonlinear static analysis is performed to check the performance of the designed buildings. Pushover analysis with lateral load patterns as mode proportional and uniform load have been used where the performance of the structure has been decided by the maximum value of response out of ATC 40 (1996), Capacity spectrum method, FEMA 356 (2000) DCM, FEMA 440 (2004) Equivalent linearization method and FEMA 440 (2004) Displacement modification method.

The perpormance point (PP) denotes the performance level of the building. The curve shown is for CP level buiding and after performing Non Linear static analysis, it is seen that the building is adequaltely designed to perform in CP level.Similarly, the other building has been designed to perform in the LS level.

Non linear dynamic analysis (Time history analysis)

Nonlinear time history analysis (NLTHA) has been carried out to determine the performances of the buildings. Time history method shall calculate building response at discrete time steps using discretized recorded or synthetic time histories as base motion.The NLTHA has been carried out with five spectrum compatible ground motions (SCGM) after EC-8 design spectrum at 0.45g level of seismicity. The arificial earthquales were generated using Kumar (2004) software.

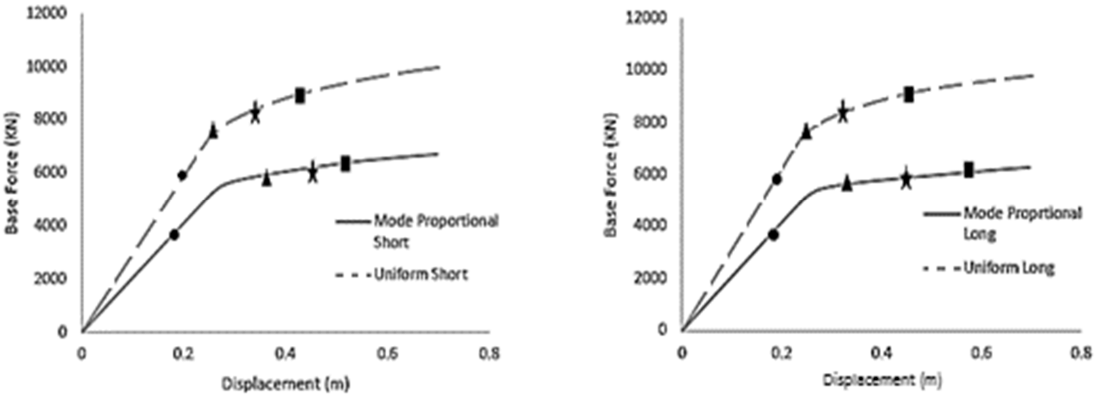


Figure 5. Typical pushover curves for the building **P1_10_CP** is shown for (a) short direction and (b) long direction

Table 4. Ground motions considered

Sl. No	Name	Background Earthquake	Direction	PGA	Duration (sec)
1	SCGM1	Duzce 1999	Duzce, 270 (ERD)	1.37	20.2
2	SCGM2	EL Centro 1940	N-S Component	1.18	31.8
3	SCGM3	Gazli 1976	Karakyr, 90	1.05	16.3
4	SCGM4	Kocaeli 1999	Sakarya, 90(ERD)	1.14	45
5	SCGM5	N.Palm Spring 1986	0920, USGS station 5070	1.64	20

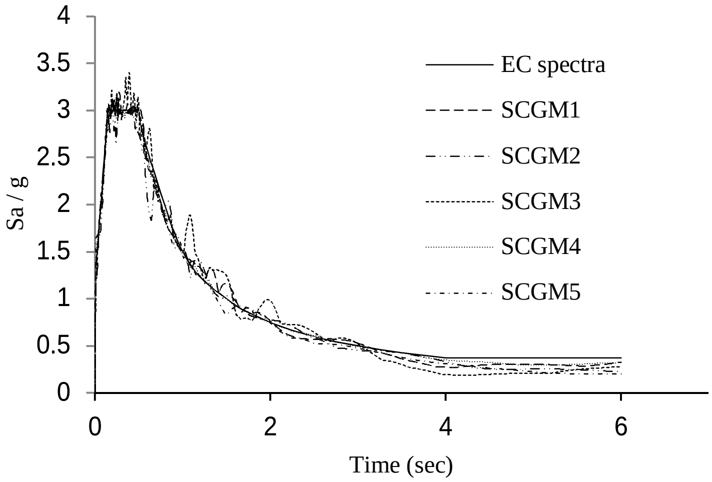


Figure 6. Match of EC 8 design spectrum with response spectra of SCGMs used for Non-Linear time history analysis

SAP 2000 v 14.0.0 software, the formation of plastic hinge in structural elements is shown as such that the pink color indicates Immediate Occupancy (IO) hinge, blue color represents LS hinge and cyan color represents CP hinge. At the end of time history analysis, structure P1_10_CP revealed formation cyan hinges which suggests it is in CP performance level. Similarly, for P2_12_LS blue hinges were developed making it a LS performing building.

The inter-story drifts (IDR) have been obtained under five SCGMs and the maximum drift has been reported. The dotted line represents the design drift for the building.

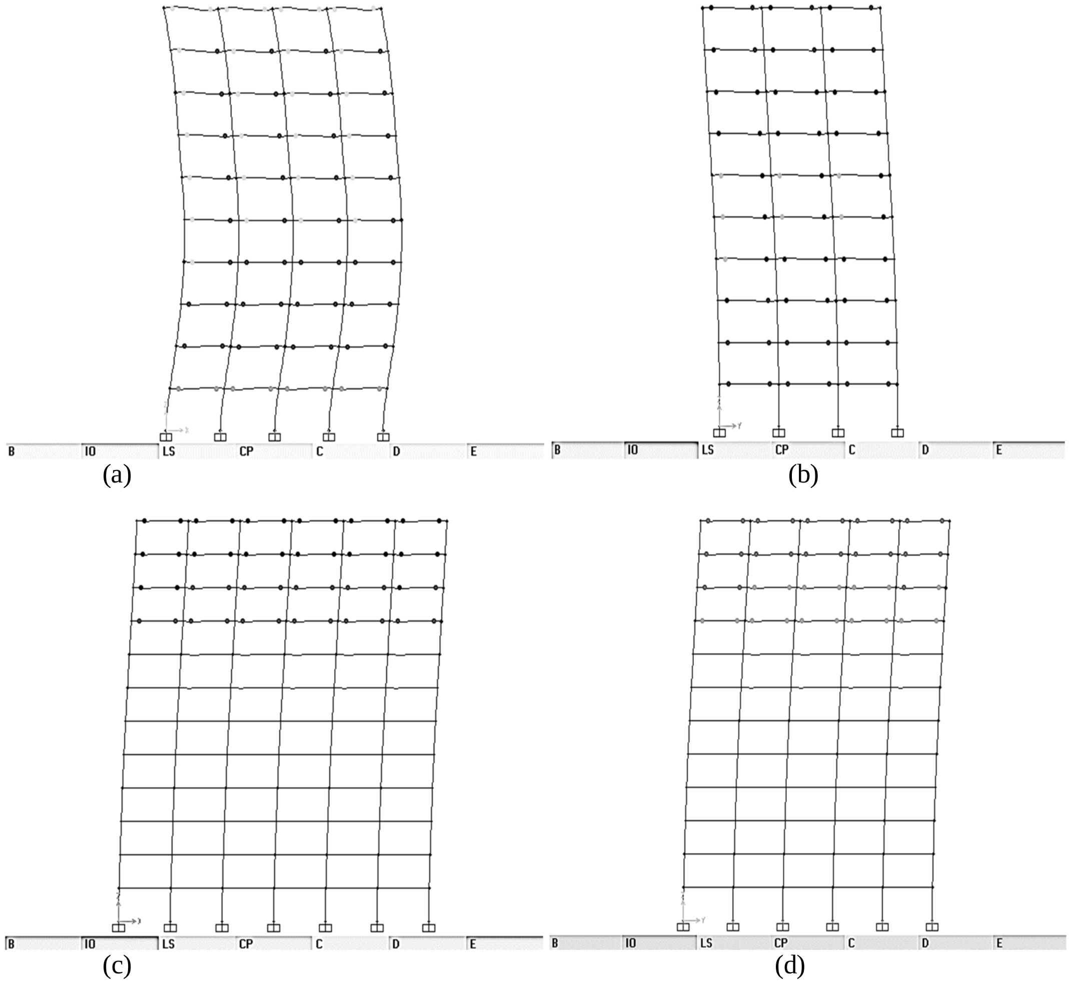
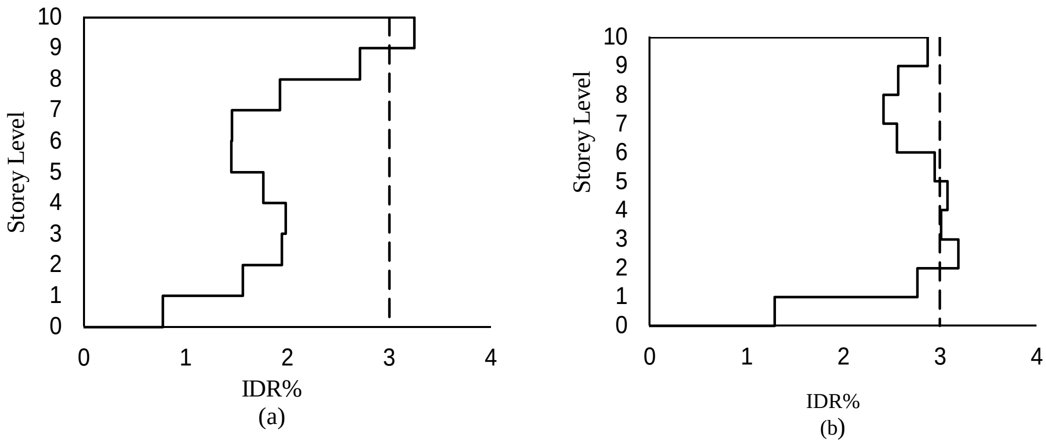


Figure 7. Typical hinge results for the buildings after time history analysis
(a) P1_10_CP_long, (b) P1_10_CP_short, (c) P2_12_LS_long, (d) P2_12_LS_short



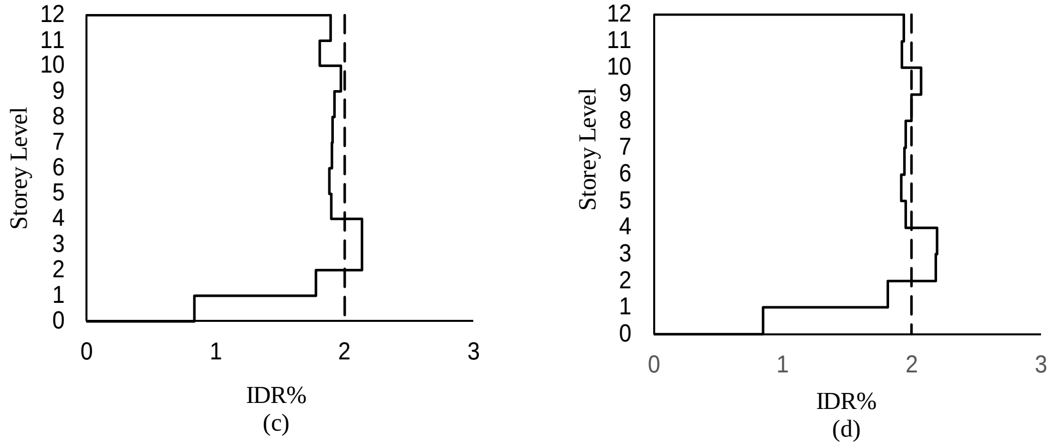
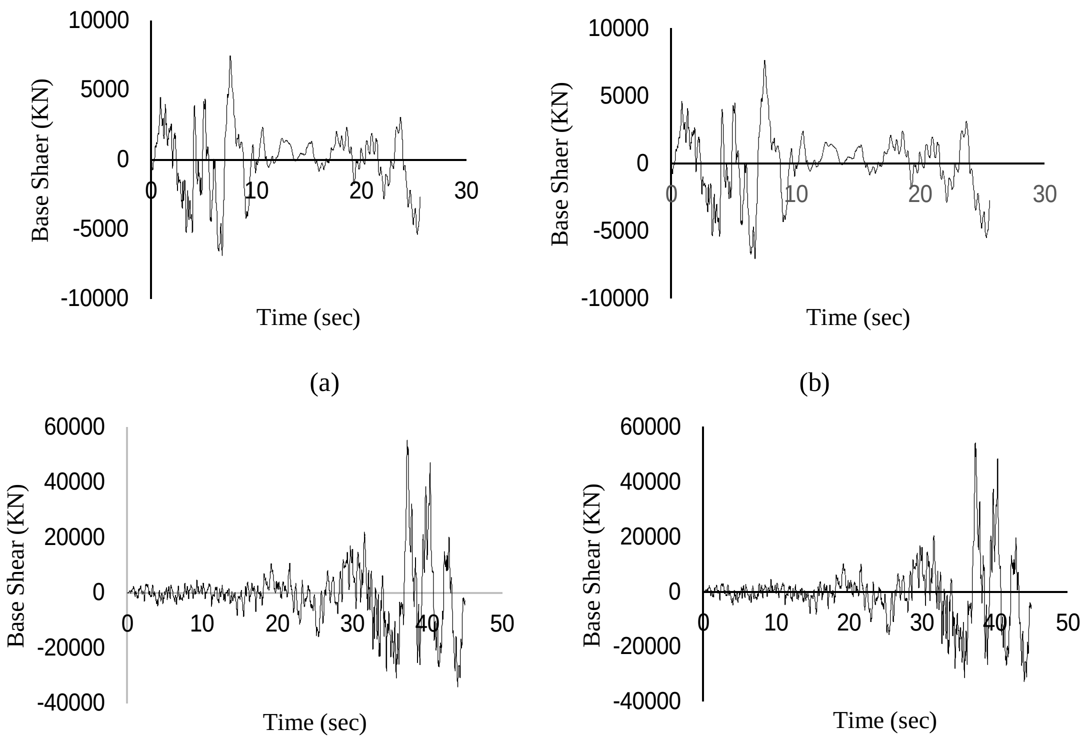


Figure 8. Typical inter storey drift ratio of the buildings after time history analysis (a) **P1_10_CP_long**, (b) P1_10_CP_short, (c) P2_12_LS_long, (d) P2_12_LS_short

The interstorey drift results have been shown above. The figures indicates that the building have achieved the target drift for which it is designed.

The base shear variation of the buildings under the SCGMs is reported below. Out of the five SCGMs considered, base shear variation with maximum peak amplitude is shown below.



(c) (d)

Figure 9. Base shear variation of the buildings after time history analysis (a) P1_10_CP_long, (b) P1_10_CP_short, (c) P2_12_LS_long, (d) P2_12_LS_short

From the base shear variation maximum amplitude of base shear is noted and the amplification of the storey shear forces from the design shear forces is found out. The difference between the shear forces between the design and time history results is shown below.

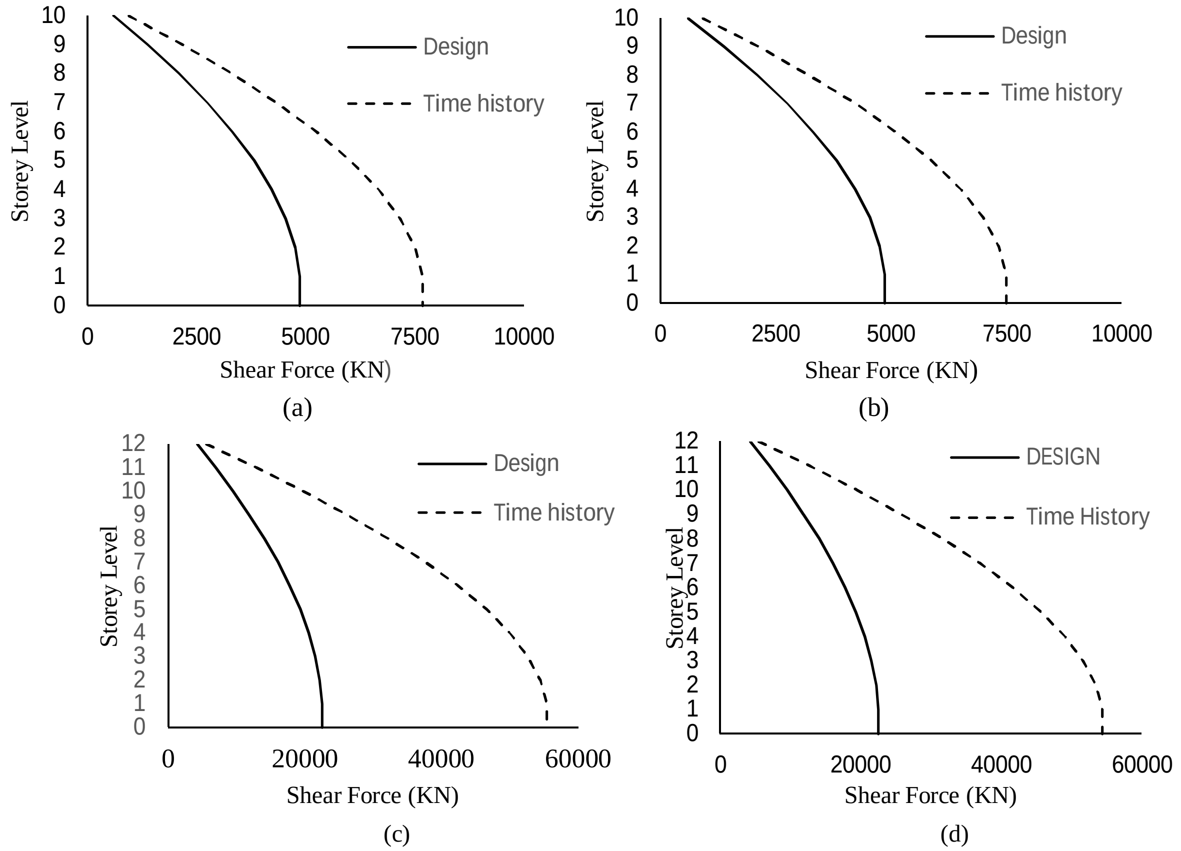


Figure 10. Difference in storey shear forces between design and time history analysis results (a) P1_10_CP_long, (b) P1_10_CP_short, (c) P2_12_LS_long, (d) P2_12_LS_short

Conclusion

The buildings as stated earlier has been designed with UPBD method to get the desired target performance objectives which is not possible with forced based approach i.e., Codal method. The main purpose of designing through UPBD method is to know the actual behaviour of the designed building which is checked by performing Non- linear analysis. The two buildings as mentioned earlier has been designed to their respective performance level i.e., the buildings have met the desired target performance objectives which is clear from Fig. 5, 7 and 8. The main study carried out here is to know the dynamic shear force amplification that occurs in the structure under the action of earthquakes. The maximum base shear variation of the structures under the five SCGM mentioned is noted. Storey shear forces are distributed with base shear results obtain after time history analysis. A comparison is made between the design and time history results. Fig.10 clearly depicts that there is significant amplification in shear force from the designed shear force. Proper amplification factors needs to be developed to accurately predict the amount of amplification by carrying out the study to many number of buildings.

References

[1] M. J. N. Priestley and M. J. Kowalasky, “Direct Displacement-Based Design of Concrete Buildings”, Bulletin of New Zealand National Society of Earthquake Engineering, Vol.33, No.4, 2000, pp. 421-444.

[2] J. D. Pettinga and M. J. N. Priestley, “Dynamic Behavior of Reinforced Concrete Frames Designed with Direct Displacement-Based Design”, Journal of Earthquake Engineering,2005,pp. 309-330

[3] M. J. N. Priestley, “Performance-Based Seismic Design”, Bulletin of the New Zealand Society for Earthquake Engineering, Vol.33, No.3,2000, 2000, pp.325-346.

[4] M. J. N. Priestley, “Myths and Fallacies of Earthquake Engineering”, The Mallet Milne Lecture, IUSS Press, Pavia, Italy, 2003.

[5] S. Choudhury, S. M. Singh, "A Unified Approach to Performance-Based Design of RC Frame Buildings", Journal of the Institution of Engineers (India)": Series ~ DOI I0.1007 /s40030- 013-0037-8, 2013.
[6] B.R. Rad and P. Adebar, "Dynamic Shear Amplification in High-Rise Concrete Walls: Effect of Multiple Flexural Hinges and Shear Cracking", The 14th World Conference on Earthquake Engineering, October 12-17, 2008, Beijing, China.
[7] "ATC-40: Seismic Evaluation and Retrofit of Concrete Buildings", Applied Technology Council, Redwood City, California, 1996.
[8] "FEMA-356: Prestandard and commentary on the seismic rehabilitation of Buildings", Federal Emergency Management Agency, Washington, D.C., 2000.
[9] "FEMA-440: Improvement in Nonlinear Static Seismic Analysis Procedures", Federal Emergency Management Agency, Washington, D.C., 2000.