SESIMIC DESIGN OF BUILDINGS WITH IRREGULAR CONFIGURATIONS

A DISSERTATION

Submitted in the partial fulfilment of the

requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With specialization in Structural Dynamics)

by

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CANDIDATE'S DECLARATION

I hereby, declare that the work which is being presented in this dissertation entitled, **"SESIMIC DESIGN OF BUILDINGS WITH IRREGULAR CONFIGURATIONS"**, being submitted in partial fulfilment of the requirements for the award of degree of "Master of Technology" in "Earthquake Engineering" with specialization in Structural Dynamics, to the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, under the supervision of Dr. Pankaj Agrawal, Professor & Head, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out during the period of June 2018 to June 2019.

I declare that I have not submitted the material embodied in this dissertation for the award of any other degree or diploma.

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CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge and belief.

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ABSTRACT

To perform well in earthquake, a building should possess four main attributes, namely simple and regular configuration, and adequate lateral strength, stiffness and ductility. With the increase in demand of high rise building and new architectural design, the irregularities in buildings has grown up and in this dissertation the comparative performance of floating column irregularity with normal conventional building has been discussed, and the two design techniques i.e. FBD (IS code method) and DDBD is used to enhance the performance of floating column building. It is found that this vertical irregularity is critical due to non-definite path in load transfer and show large stress concentration in columns adjacent to the floating columns. The storey displacement as well as interstorey drift ratio of floating column building is very high with respect to normal conventional building. It is found in the study that the overhanging portion do not contribute to lateral stiffness of building instead it shows more response in Z direction, thus making it to fail in vertical earthquake. The design from the two methods enhance the performance of floating column building and the study shows, the DDBD design performs better than FBD, though it still lags in performance with conventional building, over size section is needed for taking account of reinforcement and increase the capacity of the structure.

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First appreciation goes to my parents, without whose beliefs, pursuing post-graduation would still have been a dream.

Completion of this dissertation would not have been possible without the expertise of my supervisor **Dr. Pankaj Agarwal**, Professor & Head, Department of Earthquake Engineering, Indian institute of Technology Roorkee. Frequent discussions throughout the dissertation work were extremely fruitful and helped me to overcome hurdles and problems I faced during this work.

I take this opportunity to thank my friends for their valuable support they have extended to me. I would like to thank all those people who have helped me directly or indirectly in creating right atmosphere for doing this work.

Place: Roorkee Sateesh Kumar Puri

Date:

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CHAPTER 1 INTRODUCTION

Earthquake resistant design of reinforced concrete buildings is a continuing area of research since the earthquake engineering has started not only in India but in other developed countries also. The buildings still damage due to some one or the other reason during earthquakes. The building configuration has been described as regular or irregular in term of size and shape of the building, arrangement of structural elements and mass. Regular building configuration are almost symmetrical (in plan and elevation) about the axis and have uniform distribution of lateral force-resisting structure such that, it provides a continuous load path for both gravity and lateral loads. A building that lacks of symmetry and has discontinuity in geometry, mass, or load resisting element is called irregular. These irregularities may cause interruption of force flow and stress concentrations. Asymmetrical arrangements of mass and stiffness of elements may cause a large torsional force where the center of mass does not coincide with the center of rigidity.(Murty et al. 2012)

The section 7 of IS 1893 (Part 1): 2016 enlists the irregularity in building configuration system.

These buildings have both in plane and out of plane irregularities in strength and stiffness and hence are seismically vulnerable.

1.1 OBJECTIVE

The aim of the present study is the comparative analysis of conventional G+9 storey building and a building with similar configuration having floating column i.e. vertical irregularity and to suggest design techniques as well as retrofitting techniques, suitable for such buildings. The objective of the study is:

1. To determine the static, dynamic and performance of conventional as well as floating column building.
2. To differentiate the performance of floating columns building designed by Direct Displacement Based Method(DDBD) and by Force Based Method(FBD).
3. To increase the capacity of building by changing size of column and beam at predefined location.

CHAPTER 2 DYNAMIC ANALYSIS OF G+9 STOREY RC FRAME BUILDING WITH AND WITHOUT FLOATING COLUMN

2.1 GENRAL

The proposed study is the comparative performance analysis of conventional G+9 storey building designed for both gravity and seismic to a building with similar configuration having floating as a vertical irregularity, designed for gravity. Thus a conventional building is designed using IS 456:2000 and IS 1893:2016 in SAP 2000 and then irregularity is introduced and designed for gravity. In this chapter the PM interaction diagram of column has been plotted against the demand of floating column building and compared for its failure. Then storey displacement under equivalent lateral force is discussed and nonlinear static push over in X and Y is ran and its performance is discussed. After analyzing floating column building for its failure mechanism, it is then designed by two proposed methods i.e. DDBD and FBD. These two method’s performance is again compared and the conclusion is drawn with respect to most efficient. Physical verification of the building to see any verification could not be carried out. The building as such, has been considered as-built considered according to the design.

2.2 GENERAL DESCRIPTION OF BUILDING

The building is a normal conventional building having G+9 storeys. The plan of the building is shown in figure. The building is unsymmetrical ordinary moment resisting frame building with columns having equal spacing. The details of elevation in X and Y direction has been shown in figure. The floating column is introduced by removing the exterior columns in ground storeys in Y-direction and then redesigning it for gravity loads and checking its performance. The floor to floor height is 3m. The sizes of beam taken is (400x600) mm in mid span, (500x600) mm in end span of exterior frame and (600x600) mm in end span of interior frame over the entire height of the building. The columns size is of uniform height throughout the height of the building and the size is 600x600 mm. The grade of concrete used is M40. The grade of steel is Fe415.Importance factor is taken as 1, Response reduction factor is 3 (ordinary RC frame), Zone is V (Z = 0.36), soil type is II i.e. medium or stiff soil. (IS 1893(part 1):2016). The configuration of building is shown in Fig. 2.1-2.2-2.3-2.4.

2.2.1 EXISTING CONFIGURATION

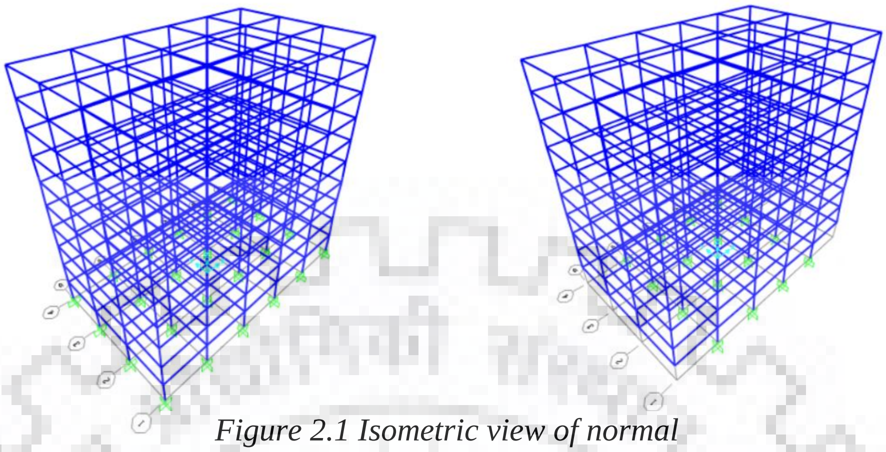


Figure 2.1 Isometric view of normal conventional building and floating column building

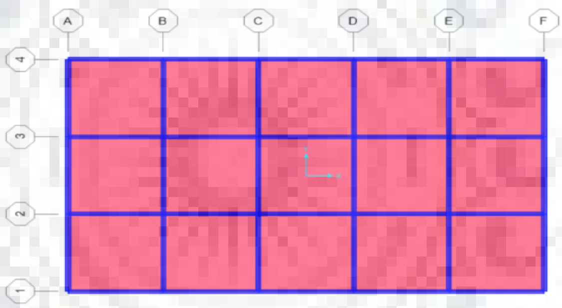


Figure 2.2 Plan of building

Figure 2.3 Elevation of building in X-Z direction

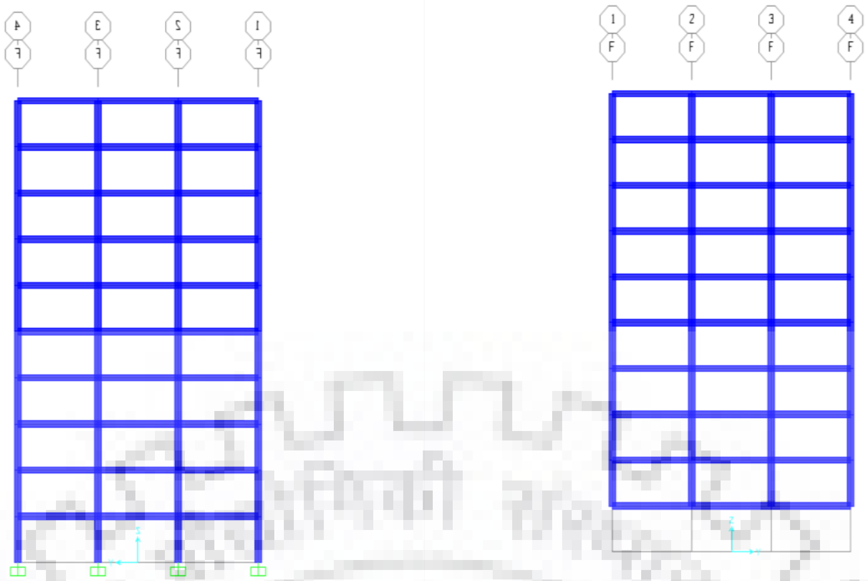


Figure 2.4 Elevation of building in

Y-Z direction

The storey height of the building is 3.0m for each floor. The bay width for first & third bay is 6.0m and that is for second bay is 6.0m along X- direction, while the bay width along Y- direction is 6.0m. The Grade of concrete and reinforcement used is M40 & Fe415 respectively. The self-weight of the members is automatically assigned using a load case in SAP2000. The dead load due to slab weight, floor finish IS 875(Part 1):1987 was assigned using a separate load case. The live load on the floors (3.0 KN/m^2) and on roof (1.5 KN/m^2) are assigned as per IS 875(Part 2): 1987. The seismic weight of the building was considered as Dead load plus 25% of live load as per Table 10 of IS 1893 (Part 1):2016. The effective stiffness of the beams & columns was taken as per ASCE 41.

The nodes at each floor level is assigned rigid diaphragm constraint so as to simulate the effect of slabs at each floor levels (SAP2000) This constraint enables the nodes at each floor level to move together as a planar diaphragm

2.3 METHODS OF ANALYSIS

There are two methods of dynamic analysis available which is time history and response spectrum analysis. Time history analysis determines the responses of the structure to a known ground motion at predetermined time steps while response spectrum method consists in determining response of few mode of vibration and then combining total response by suitable combination rules. As the peak of maximum response do not occur at the same time thus modal combination rules such as square root of sum of squares (SQRSS) or complete quadratic combination are used to get complete response. The equivalent static analysis, response spectrum method of dynamic analysis and nonlinear static analysis i.e. push over analysis is used in this study.(Chopra and Goel 2001)

2.4 MODELLING FOR DYNAMIC ANALYSIS

The building is idealized as three dimensional linear space frame model. Bare frame analysis is pursued, which neglects the effect of the stiffness of infill walls on the structural response, using SAP 2000 software. The masses are lumped at each floor level. For gravity loads, the floor weights of respective tributary areas are distributed triangularly over beams. The weight of the infill wall in any storey is equally distributed to the floors above and below the storey. For response spectrum analysis the seismic weight at any floor level is considered as its full dead load plus 25% of live load. (IS 1893(part 1):2016)

Assumption in dynamic analysis

The following assumptions are made in dynamic analysis: -

1) The bases of the frame are assumed as fixed with respect to rotational and translational movements.
2) Soil Structure interaction effects are neglected.
3) Neglecting the effects of stiffness of infill bare frame is analyzed.
4) The damping of the structure is assumed to be constant as 5% of critical, actually damping also changes with strength and stiffness.
5) The members are assumed to be homogeneous and isotropic.
6) Elastic modulus in tension and compression in assumed to be same.
7) The cracked section of beam and column is taken
8) Flexural rigidity EI is calculated on basis of gross concrete section, not transformed section.(Agarwal and Shrikhande 2007)

2.5 LOAD COMBINATION

As per IS 1893(part1):2016

Table 2.1 Load combination for assessment of existing building

LOAD CASE TYPE	DEAD LOAD	LIVE LOAD	EARTHQUAKE LOAD	
			EQx	EQy
DL	1.50	0.00	0.00	0.00
DL+LL	1.50	1.50	0.00	0.00
DL+LL+EQx	1.20	1.20	(+/-)1.20	0.00
DL+LL+EQy	1.20	1.20	0.00	(+/-)1.20
DL+EQx	1.50	0.00	(+/-)1.50	0.00
DL+EQy	1.50	0.00	0.00	(+/-)1.50
DL+EQx	0.90	0.00	(+/-)1.50	0.00
DL+EQy	0.90	0.00	0.00	(+/-)1.50

2.6 DESIGN FORCES IN NORMAL CONVENTIONAL BUILDING

For the outer frame beam in first floor of first span i.e. beam AB as shown in Fig. 2.5, force resultants for various load cases and load combinations have been obtained from computer analysis and are summarised in Table 2.2 and Table 2.3 which show force resultants for different load combinations; with the maximum values to be used for design being underlined. As the beam under consideration is parallel to X direction, earthquake loads in X direction are predominant and hence the 13 load combinations (IS 1893(part1):2016) of Table 2.1 reduce to 7 as shown in Table 2.3. Similarly, all the beams and columns have been designed and check for normal conventional building as well as for floating column building. Sample calculation for a beam A-B has been shown

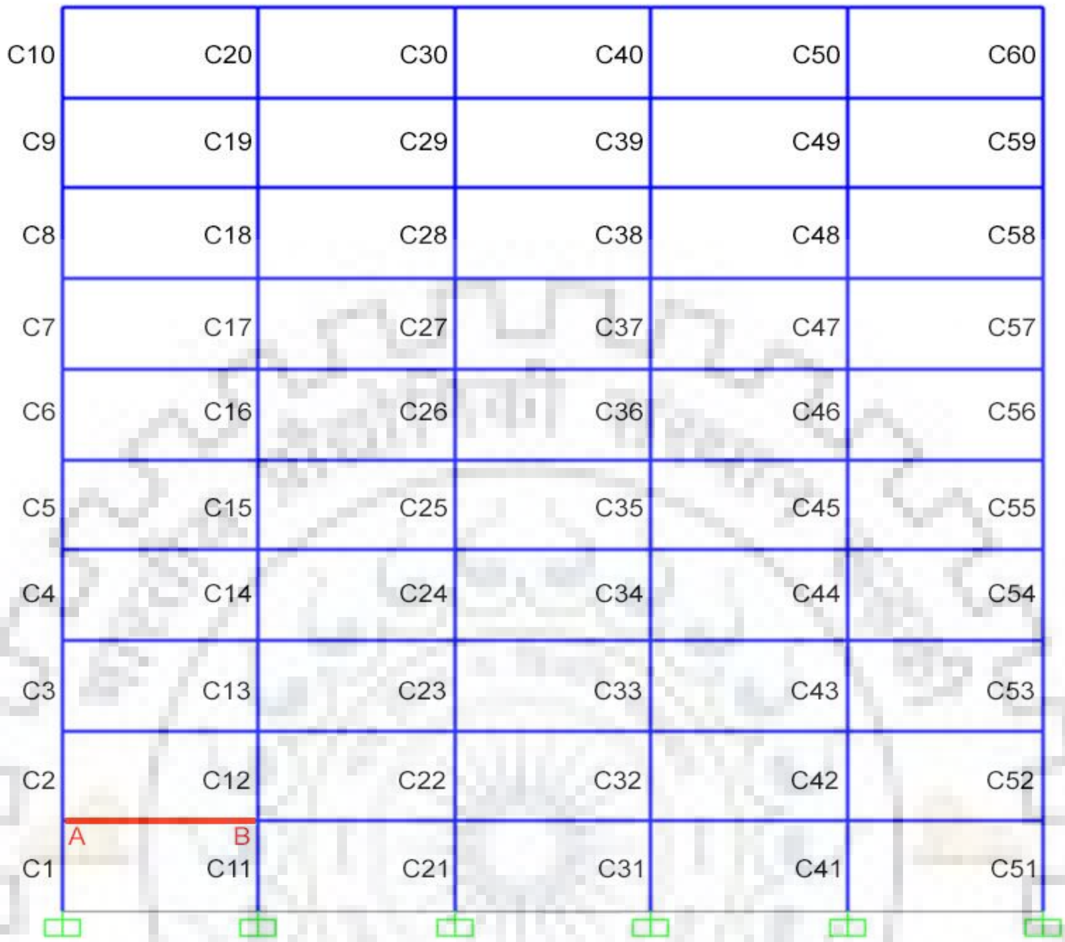


Figure 2.5 Beam A-B of normal conventional building

Table 2.2 Force resultant in beam for various load case

Load Case	Left End		Center		Right End	
	Shear(kN)	Moment (kN-m)	Shear(kN)	Moment (kN-m)	Shear(kN)	Moment (kN-m)
DL	-23	-21	0	13	23	-23
LL	-14	-17	0	11	14	-17
EQ X	70	215	70	6	70	-204

Note: The results are rounded off to the next higher integer value.

Table 2.3 Force resultants in a beam for different load combinations

S.No.	Load Combination	Left End		Center		Right End	
		Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)	Shear (kN)	Moment (kN-m)
1	1.5DL+1.5LL	56	57	0	**36**	56	60
2	1.2(DL+LL*+EQX)	52	228	84	22	116	-278
3	1.2(DL+LL*-EQX)	**-116**	-288	-84	12	-52	212
4	1.5(DL+EQX)	71	291	**105**	29	**140**	**-341**
5	1.5(DL-EQX)	-140	**-354**	-105	11	-71	272
6	0.9DL+1.5 EQX	85	**304**	105	21	126	-327
7	0.9DL-1.5 EQX	-126	-342	-105	3	-85	**285**

* Appropriate fraction of live load has been taken

And for maximum combination the beam elements have been designed as per IS 456:2000 and IS 1893(part 1):2016(Equivalent Static method and response spectrum method) for normal conventional building. The demand for reinforcement is presented in Table 2.4 for normal conventional building as well as for floating column building.

2.6.1 DESIGN FOR FLEXURE

2.6.1.1 DESIGN FOR HOGGING AND SAGGING MOMENT

For left end calculation has been shown and similarly for right end as well as for every beam, the reinforcement spreadsheet has been prepared. B=500 mm, D=600 mm, d= 560 mm.

Mu=354 kN-m

$A_{st\ min}$ = (0.85/fy) *Bd=1159.04 mm^2

As $Mu_{limiting}$ of cross section of beam taken is 993.34 kN-m which is greater than 354kN-m thus A_{st} provided according to Eq (2.1)

$$Mu = 0.87\, fy * Ast\left(1 - \frac{Ast * fy}{fck * b * d}\right) \tag{2.1}$$

A_{st} at top = 1738.73 mm^2 i.e.0.58 %(> minimum reinforcement and < maximum reinforcement i.e.4%)

Half of Ast at top is provided at the bottom i.e. A_{sc} = 870 mm^2

Similarly, for sagging moment = 304 kN-m

A_{st} provided at bottom = 1479 mm^2 i.e.0.5 %(> minimum reinforcement and < maximum reinforcement i.e.4%)

A_{sc} provided at top = 1479/2 = 740 mm^2

2.6.1.2 REQUIRED REINFORCEMENT

Top reinforcement required is larger of 1738.73 mm^2 and 756 mm^2. Hence, provide 1738.73 mm^2. Bottom reinforcement required is larger of 870 mm^2 and 1479 mm^2. Hence, provide 1479 mm^2.

2.6.2 CHECK FOR SHEAR

Tensile steel provided at left end = 0.58%

Permissible design shear stress of concrete,

τ_c = 0.55 MPa (IS 456:2000 Table 19)

Design shear strength of concrete

= τ_c b d

= 0.55 x 500 x 560 /1,000

= 154 kN (> 116 thus minimum shear reinforcement is provided i.e. 4 legged 8 mm bars)

Minimum shear reinforcement as per Clause 26.5.1.6 of IS 456:2000

Sv = Asv x 0.87 fy /(0.4 b)

= 4 x 50 x 0.87 x 415 / (500 x 0.4)

= 300 mm.

< 560 x 0.75 = 420 mm

Hence, ok.

Similarly, design shear strength of concrete at center and right end is evaluated as 74 kN and 154 kN, respectively.

2.6.3 CHECK FOR DEFLECTION

The deflection of reinforced concrete beams is not directly calculated and the serviceability of the beam is measured by comparing the calculated limiting basic span/effective depth ratio L/d, modification factor for tension and compression is taken as 1. Thus satisfying Eq. (2.2).

$$\frac{Span}{Depth} = \frac{6000}{560} = 10.72 < 26 \tag{2.2}$$

Hence, ok.

Capacity design of normal conventional building has been done as per IS13920-2016 i.e. strong column weak beam failure mechanism.

Percentage reinforcement in normal conventional building and floating column building design is in Table 2.4-2.5-2.6-2.7.

Table 2.4 Design details for interior frame beams

Storey, i	Interior frame beams (% reinforcement)									
	Span 1		Span 2		Span 3		Span 4		Span 5	
	Conv	Float	Conv	Float	Conv	Float	Conv	Float	Conv	Float
10	0.96	1.31	0.92	0.92	0.92	0.92	0.92	0.92	0.96	1.31
9	0.87	2.18	0.92	0.92	0.92	0.92	0.92	0.92	0.87	2.18
8	1.13	2.18	2.09	0.92	2.09	0.92	2.09	0.92	1.13	2.18
7	1.22	2.18	1.57	0.92	1.57	0.92	1.57	0.92	1.22	2.18
6	1.57	2.18	1.70	0.92	1.70	0.92	1.70	0.92	1.57	2.18
5	1.77	2.18	2.25	1.02	2.25	1.02	2.25	1.02	1.77	2.18
4	1.77	2.18	2.04	1.02	2.04	1.02	2.04	1.02	1.77	2.18
3	1.77	2.18	2.04	1.02	2.04	1.02	2.04	1.02	1.77	2.18
2	1.77	2.18	2.04	1.02	2.04	1.02	2.04	1.02	1.77	2.18
1	1.64	2.18	1.64	1.02	1.64	1.02	1.64	1.02	1.64	2.18

Table 2.5 Design details for interior frame columns

Storey, i	Interior frame columns (% reinforcement)											
	Col. 1		Col. 2		Col. 3		Col. 4		Col. 5		Col. 6	
	Conv	Float	Conv	Float	Conv	Float	Conv	Float	Conv	Float	Conv	Float
10	2.18	2.18	1.64	1.64	1.05	1.05	1.05	1.05	1.64	1.64	2.18	2.18
9	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
8	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
7	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
6	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
5	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
4	1.05	1.05	1.64	1.64	1.05	1.05	1.05	1.05	1.64	1.64	1.05	1.05
3	1.05	1.05	1.64	1.74	1.05	1.05	1.05	1.05	1.64	1.74	1.05	1.05
2	2.73	2.73	1.42	2.73	1.05	1.05	1.05	1.05	1.42	2.73	2.73	2.73
1	2.73	0	3.27	3.50	3.27	3.30	3.27	3.30	3.27	3.50	2.73	0

Table 2.6 Design details for exterior frame beams

Storey , i	Exterior frame beams (% reinforcement)									
	Span 1		Span 2		Span 3		Span 4		Span 5	
	Conv	Float	Conv	Float	Conv	Float	Conv	Float	Conv	Float
10	0.84	1.05	0.92	0.61	0.92	0.61	0.92	0.61	0.84	1.05
9	0.94	2.00	0.92	0.61	0.92	0.61	0.92	0.61	0.94	2.00
8	1.15	2.00	1.18	0.61	1.18	0.61	1.18	0.61	1.15	2.00
7	1.15	2.00	1.44	0.61	1.44	0.61	1.44	0.61	1.15	2.00
6	1.36	2.00	1.44	0.61	1.44	0.61	1.44	0.61	1.36	2.00
5	1.47	2.00	1.43	0.68	1.43	0.68	1.43	0.68	1.47	2.00
4	1.47	2.00	1.64	0.68	1.64	0.68	1.64	0.68	1.47	2.00
3	1.47	2.00	1.64	0.68	1.64	0.68	1.64	0.68	1.47	2.00
2	1.47	2.00	1.64	0.68	1.64	0.68	1.64	0.68	1.47	2.00
1	1.31	2.00	1.23	0.68	1.23	0.68	1.23	0.68	1.31	2.00

Table 2.7 Design details for exterior frame columns

Storey, i	Exterior frame columns (% reinforcement)											
	Col. 1		Col. 2		Col. 3		Col. 4		Col. 5		Col. 6	
	Conv	Float	Conv	Float	Conv	Float	Conv	Float	Conv	Float	Conv	Float
10	1.64	1.64	1.64	1.64	1.05	1.05	1.05	1.05	1.64	1.64	1.64	1.64
9	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
8	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
7	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
6	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
5	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
4	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
3	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
2	2.18	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	2.18	1.05
1	2.18	0.00	2.18	2.50	2.18	2.50	2.18	2.50	2.18	2.50	2.18	0.00

Conventional building is designed for both gravity as well as seismic while floating column building is designed only for gravity thus showing less reinforcement in floating building but the overhanging beams has much more reinforcement than conventional due to its high moment demand in gravity. Columns has almost same reinforcement in conventional as well as floating (designed for gravity only) but in seismic, almost every element except for cantilever beams is failing i.e. their demand to capacity ratio is exceeding 1. Also from column reinforcement details it can be seen that only ground storey column reinforcement has been increased i.e. upper columns are not affected by this vertical irregularity. (Murty et al. 2012)

CHAPTER 3 DESIGN OF FLOATING COLUMN BUILDING BY DIRECT DISPLACEMENT BASED METHOD

3.1 INTRODUCTION

Over the last years, as the importance of displacements, rather than forces, has become better appreciated, a growing interested appeared for methods based on displacements, in particular for what regards RC structures. Several contributions were made towards the development of Displacement Based Design (DBD) approaches, but it was only in the 1990's that formal proposals were made to implement the emerging ideas into formalized design procedures. One of these new design procedures is the Direct Displacement Based Design, which was developed on the base of Priestley works. The central idea of the Direct Displacement Based Design (DDBD) procedure is to design structures in order to achieve displacements corresponding to a given seismic hazard level.(Muljati et al. 2015)

The objective of this study is to apply the Direct Displacement Based Design to a simple case of study, a reinforced concrete frame building and to assess the applicability of the method and the needed of develop an automatic design tool.(Priestley et al. 2007)

3.2 DIRECT DISPLACEMENT BASED DESIGN METHOD FOR REINFORCED CONCRETE FRAMES

The step by step DDBD procedure is listed in the following:(Massena et al. 2012)

Step 1. Definition of the target displacement shape and amplitude of the MDOF structure on the base of performance level considerations (material strain or drift limits) and then derive from there the design displacement Δd of the substitute SDOF structure of the MDOF.Fig.3.1 presents a simplified model of a multi-storey frame building, where shown the required variables in DDBD procedure.

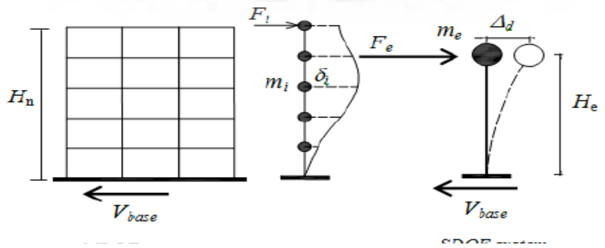


Figure 3.1 Simplified model of a multi-storey building by Priestley et al.(2007).

where, δ_i is the normalized inelastic mode shape, mi are the masses at each significant storey i , me is the equivalent mass of the SDOF, H_i is the height of each storey, H_n are the total height of the building, He is the equivalent height and Δd is the equivalent SDOF design displacement.

Displacement Shape The normalized inelastic mode shape δ_i of the frame MDOF structure is defined in Priestley et al. (2007) and should be obtained according to the number of stories, n, as

$$\text{for } n \leq 4 : \delta_i = \frac{H_i}{H_n} \tag{3.1}$$

$$\text{for } n \geq 4 : \delta_i = \frac{4}{3}\left(\frac{H_i}{H_n}\right)\left(1-\frac{H_i}{4H_n}\right) \tag{3.2}$$

The design storey displacements Δ_i are found using the shape vector δ_i, defined from Eq. (3.1) or Eq. (3.2), scaled with respect to the critical storey displacement Δ_c and to the corresponding mode shape at the critical storey level δ_c. According to (Calvi et al. 2008), the design storey displacements for frame buildings will normally be governed by drift limits in the lower storey of the building (i.e. in general Δ_c=Δ1 and δ_c=δ1). Knowing the displacement of the critical storey (Δ_c) and the critical normalised inelastic mode shape (δ_c), the design storey displacements of the individual masses are obtained from:

$$\Delta_i = w_\theta.\delta_i.\left(\frac{\delta_c}{\Delta_c}\right) \tag{3.3}$$

where, ω_θ is a drift reduction factor to take into account the higher mode effects and is given by, $\omega_\theta = 1.15 - 0.0034H_n \leq 1.0$ (H_n in m)

Design Displacement of the equivalent SDOF structure

The equivalent design displacement can be evaluated as:

$$\Delta_d = \sum_{i=1}^{n}\left(m_i\Delta_i^2\right) / \sum_{i=1}^{n}\left(m_i\Delta_i\right) \tag{3.4}$$

Equivalent Mass of the SDOF structure

The mass of the substitute structure is given by the following Eq. (3.5)

$$m_e = \sum_{i=1}^{n} m_i \left(\frac{\Delta_i}{\Delta_d} \right) = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} \tag{3.5}$$

Equivalent Height of the SDOF structure

The equivalent height (see Fig.3.1) of the SDOF substitute structure is given by

$$H_e = \sum_{i=1}^{n} \left(m_i \Delta_i H_i \right) / \sum_{i=1}^{n} \left(m_i \Delta_i \right) \tag{3.6}$$

Step 2. Estimation of the level of equivalent viscous damping ξ. To obtain the equivalent viscous damping the displacement ductility μ must be known. The displacement ductility is the ratio between the equivalent design displacement and the equivalent yield displacement Δy (see Fig.3.2). The equivalent yield displacement is estimated according to the considered properties of the structural elements, for example through the use of approximated equations proposed in Priestley et al. (2007), and based on the yield curvature.

Displacement ductility of the SDOF structure:

The SDOF design displacement ductility (see Fig.3.2) is given by Eq. (3.7) and is related to the equivalent yield displacement Δy:

$$\mu = \frac{\Delta_d}{\Delta_y} \tag{3.7}$$

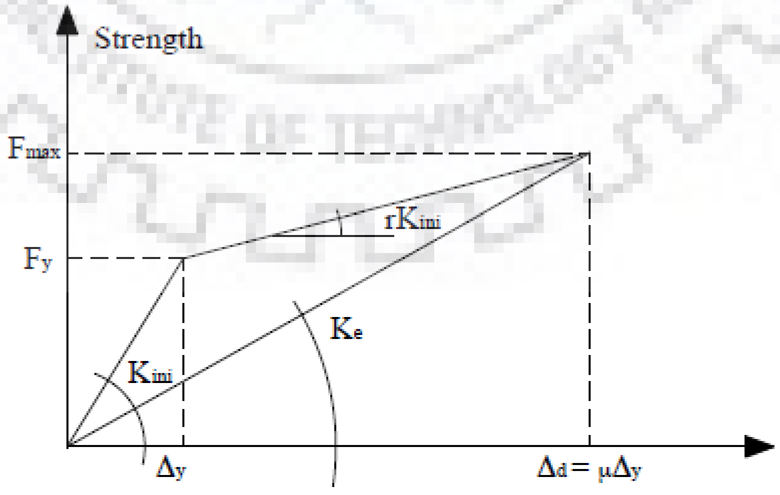


Figure 3.2 Constitutive law of the equivalent SDOF system by Priestley et al.(2007).

Equivalent yield displacement

The equivalent yield displacement is given by the following equation:

$$\Delta_y = \theta_y H_e \tag{3.8}$$

where θ*y* is the yield drift and for reinforced concrete frames is given by:

$$\theta_y = 0.5 \in_y L_{j-1} / h_b \tag{3.9}$$

Equivalent viscous damping

To take into account the inelastic behavior of the real structure, hysteretic damping (ξhyst) is combined with elastic damping (ξ_0). Usually, for reinforced concrete structures the elastic damping is taken equal to 0.05, related to critical damping. The equivalent viscous damping of the substitute structure for frames could be defined according to Priestley et al. (2007) by the following equation:

$$\xi = \xi_0 + 0.565\left(\frac{\mu - 1}{\mu\pi}\right) \tag{3.10}$$

Step 3. Determination of the effective period Te of the SDOF structure. The effective period of the SDOF structure at peak displacement response is found from the design displacement spectrum for the equivalent viscous damping ξ, i.e. entering the design displacement of the substitute SDOF structure Δd and determining the effective period Te (see Fig.3.3).

The displacement spectra for other different levels of ξ than 5% can be found from the formulation defined in Eurocode 8, as:

$$S_{D,\xi} = S_{D,5\%}\left(\frac{10}{5+\xi}\right)^{1/2} \tag{3.11}$$

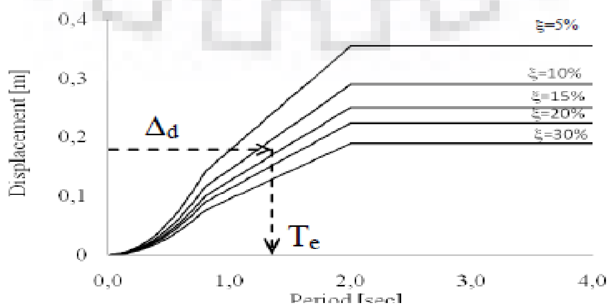


Figure 3.3 Design Displacement Spectrum by Priestley et al.(2007).

Step 4. Derivation of the effective stiffness ke of the substitute structure from its effective mass and effective period. Then it is possible to obtain the design base shear as the product of the effective stiffness by the design displacement of the substitute SDOF structure.

Effective stiffness of the substitute SDOF structure:

$$k_e = \frac{4\pi^2 m_e}{T_e^3} \tag{3.12}$$

Design base shear force

$$V_{base} = k_e \Delta_d \tag{3.13}$$

P-Δ effects in Direct Displacement-Based Design

As suggested in Wei et al. (2011), for reinforced concrete structures P-Δ effects should be considered if the stability index θΔ is greater than 0.10, with a maximum value of 0.33. The stability index compares the magnitude of the P-Δ effect at expected maximum displacement (Δmax) to the design base moment capacity of the structure (MD). The structural stability index is given by:

$$\theta_\Delta = \frac{P\Delta_{\max}}{M_D} \tag{3.14}$$

Substituting in Eq. (3.14) MD = OTM and Δmax=Δd, where OTM is the overturning moment at the base given by Eq. (3.19) and Δd is the design displacement of the substitute SDOF structure. P is the axial force due to gravity loads.

The design base shear force Vbase to take into account the P-Δ effects is given by:

$$V_{base} = k_e \Delta_d + C\frac{P\Delta_d}{H_e} \tag{3.15}$$

where, ke is the effective stiffness and He is the equivalent height of the SDOF substitute structure. The C parameter shall be taken as 0.5 for reinforced concrete buildings.

Therefore, the required base moment capacity is(Wei et al. 2011):

$$M_B = K_e \Delta_d H_e + CP\Delta_d \tag{3.16}$$

After the determination/actualization of the design base shear force, this is distributed between the mass elements of the MDOF structure as inertia forces.

Step 5. Distribution of the design base shear force V_{base} to the locations of storey mass of the building (MDOF structure).

The design base shear force is distributed to the storey levels as:

For n < 10 $$F_i = V_{base} \frac{(m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)}$$

(3.17)

For n ≥ 10 $$F_i = F_t + 0.9 V_{base} \frac{(m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)} \tag{3.18}$$

where, Ft = 0.1Vbase at roof level, and Ft = 0 at all other storey levels.

Step 6. Evaluation of design moments at potential hinge locations. To this purpose the method of analysis used is a simplified method based on equilibrium considerations (statically admissible distribution of internal forces).

Beam Moments

The lateral seismic forces Fi obtained with Eq. (3.17) or Eq. (3.18) produce in each of the columns axial forces (compression or tension) and column-base moments (Mcj). The seismic axial forces induced in each of the columns (T for tension or C for compression) by the seismic beams shears are the sum of seismic beam shears in each vertical alignment (ΣVBi). In Fig.3.4 is shown a typical distribution of seismic lateral forces Fi and the corresponding internal forces induced in a frame building. Considering the equilibrium at base level, the total overturning moment is given by:

$$OTM = \sum_{i=1}^{n} F_i H_i \tag{3.19}$$

Knowing that equilibrium should be assured between internal and external forces, the total overturning moment at the base of the structure, hence:

$$OTM = \sum_{i=1}^{m} M_{cj} + \sum_{i=1}^{n} \left[\left(\sum_{i=1}^{n} V_{B_{j-1},i} \right) \times L_{j-1} \right] \tag{3.20}$$

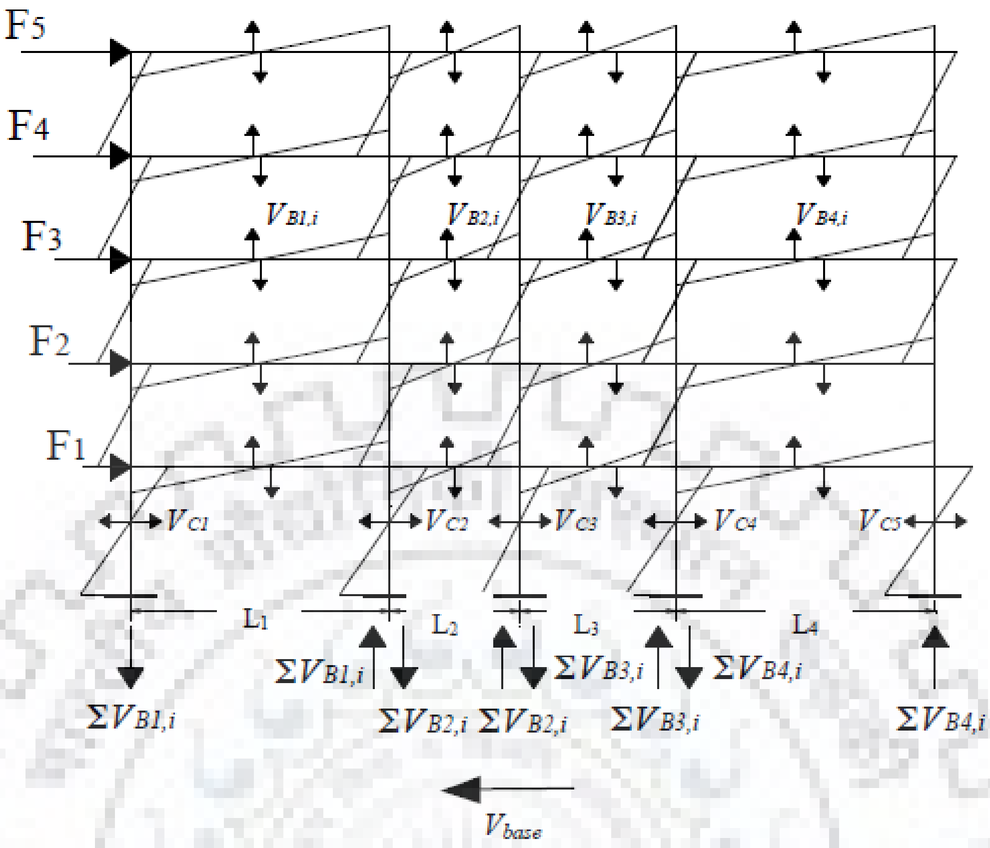


Figure 3.4 Seismic Moments from DDBD(by Priestley et al.(2007).)

where, Mcj are the column-base moments (m is the number of columns) and Lj-1 is the length of each span.

From Fig.3.5 and considering only a parcel of OTM regarding the seismic axial forces (OTM*), the corresponding overturning moment is given by:

$$OTM^{*} = \sum_{j=2}^{m}\left(\sum_{i=1}^{n} V_{B_{j-1},i}\right)\times L_{j-1} = \sum_{i=1}^{n} V_{B1,j}\times L_{1} + \sum_{i=1}^{n} V_{B2,j}\times L_{2} + \sum_{i=1}^{n} V_{B3,j}\times L_{3} + \sum_{i=1}^{n} V_{B4,j}\times L_{4}$$

(3.21)

where, $V_{B1,\,i}$, $V_{B2,\,i}$, $V_{B3,\,i}$ and $V_{B4,\,i}$ are the seismic beam shears at level i for bay 1 to 4, respectively. The seismic beam shears for each span is constant, thus, $V_{Bj-1,\,i} = 2M_{Bj-1,\,i}/L_{j-1}$, where $M_{Bj-1,\,i}$ is beam moment of each span at the storey i.

Replacing the seismic beam shears in Eq. (3.21), the overturning moment OTM* will be:

$$OTM^{*} = 2\sum_{j=2}^{m}\left(\sum_{i=1}^{n} M_{B_{j-1},i}\right) \qquad (3.22)$$

According to the example, OTM* is thus:

$$OTM^* = 2\sum_{i=1}^{m}\left(M_{B1,j} + M_{B2,j} + M_{B3,j} + M_{B4,j}\right) \tag{3.23}$$

Considering a relationship between beam moments as $M_{B2,\,i} = \alpha M_{B1,\,i}$, $MB_{3,\,i} = \beta M_{B1,\,i}$, and $M_{B4,\,i} = \chi M_{B1,\,i}$ and replacing in turn in Eq. (3.23), the beam moments corresponding to the first span L1 are given by:

$$\sum_{i=1}^{n} M_{B1,j} = \frac{OTM^*}{2(1+\alpha+\beta+\chi)} \tag{3.24}$$

If α, β and χ are replaced in Eq. (3.24) and then $\sum_{i=1}^{n} M_{B1,i}$ the seismic beam shears for the first span is:

$$\sum_{i=1}^{n} V_{B1,i} = \frac{2\sum_{i=1}^{n} M_{B1,i}}{L_1} = \frac{OTM^*}{L_1}\left(\frac{\sum_{i=1}^{n} M_{B1,i}}{\sum_{i=1}^{n} M_{B1,i} + \sum_{i=1}^{n} M_{B2,i} + \sum_{i=1}^{n} M_{B3,i} + \sum_{i=1}^{n} M_{B4,i}}\right) \tag{3.25}$$

Therefore, for each span the seismic beam shears due to OTM* are given by:

$$\sum_{i=1}^{n} V_{B_{j-1},i} = \frac{\sum_{i=1}^{n} M_{B_{j-1},i}}{\sum_{j=2}^{m}\sum_{i=1}^{n} M_{B_{j-1},i}} \frac{OTM^*}{L_{j-1}} \tag{3.26}$$

Combining Eq. (3.19) and Eq. (3.20) and replacing the parcel of seismic axial forces due to OTM* given by Eq. (3.26), the total sum of seismic axial forces is defined as:

$$\sum_{i=1}^{n} V_{B_{j-1},i} = \frac{\sum_{i=1}^{n} M_{B_{j-1},i}}{\sum_{j=2}^{m}\sum_{i=1}^{n} M_{B_{j-1},i}} \frac{\left(\sum_{i=1}^{n} F_i H_i - \sum_{j=1}^{m} M_{cj}\right)}{L_{j-1}} \tag{3.27}$$

All distribution of the total required beam shear that assures Eq. (3.27) will result in a statically admissible equilibrium solution and can be chosen on the base of engineering judgment. However, in Priestley et al. (2007) it is suggested that the distribution of the total beam shear force could be done in proportion to the storey shears in the level below the beam under consideration as depicted in Fig.3.6.

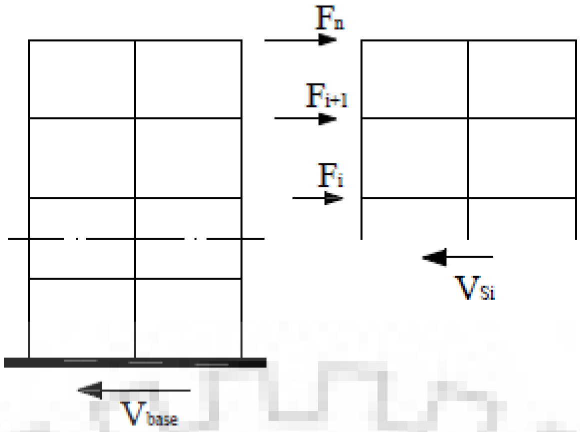


Figure 3.5 Storey Shear Forces

The distribution of the total beam shear force is thus:

$$V_{B_{j-1},i} = \sum_{i=1}^{n} V_{B_{j-1},i} \cdot \frac{V_{s,i}}{\sum_{i=1}^{n} V_{s,i}} \tag{3.28}$$

where the storey shear forces at level i, $V_{s,i}$ are given by:

$$V_{s,i} = \sum_{k=1}^{n} F_k \tag{3.29}$$

After the individual beam shear forces have been calculated, the beam design moments at the column centerlines are obtained by the following equation:

$$M_{Bl_{j-1},i} + M_{Br_{j-1},i} = V_{B_{j-1},i} . L_{j-1} \tag{3.30}$$

where, $M_{Bl\,j-1,i}$ and $M_{Br\,j-1,i}$ are the beam moments at the column centerlines at the left and right end of the beam, respectively.

Column Moments

Knowing that structural analysis based on equilibrium considerations is actually an approximation of the real distribution, the designer gets some freedom in choosing distribution of the total storey shear force between the columns and the design moment at the column-base of first storey, provided the equilibrium is maintained between internal and external forces. The total storey shear force given by Eq. (3.29) is shared between the columns. This could be done according to the following ratio: 1 for external columns and 2 for internal columns, as suggested in Priestley et al. (2007); from the shear forces at the base of each column VC, it is then possible to obtain the column-base moments at the base and top of the columns between the ground storey and 1st storey. According to Calvi et al. (2008), for one-way frames the contra-flexure point for the 1st storey columns-base moment $M_{C01,b}$ could be considered around 60% of the height of the column H1 (see Fig.3.6). Therefore, the column-base moments at the bottom and top of the 1st storey are given by:

$$M_{C01,b} = 0.6V_{C01}.H_1 \tag{3.31}$$

$$M_{C01,t} = 0.4V_{C01}.H_1 \tag{3.32}$$

Once known the column-base moments of the first storey and the beam moments at each node is determined, it is then possible to obtain the column moments distribution in height, considering the equilibrium from the 1st storey nodes and successively until the top level is reached, as illustrated in Fig.3.6.

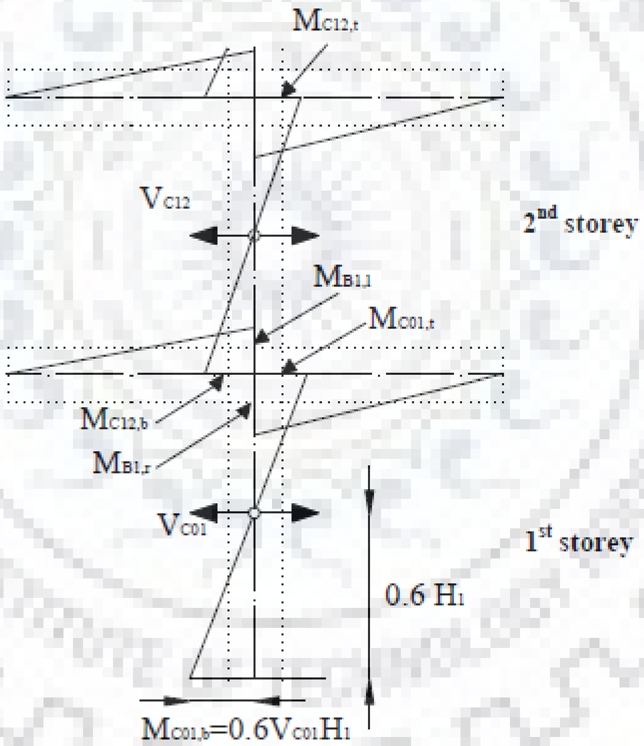


Figure 3.6 Determination of Column Moments from Considerations of Joint Equilibrium by Priestley et al.(2007).

Step 7. Capacity Design Requirements for Frames.

Capacity design rules must then be implemented to ensure that plastic hinges cannot develop at unintended locations and, that shear failure cannot occur for the desired mechanism (see Fig.3.7).

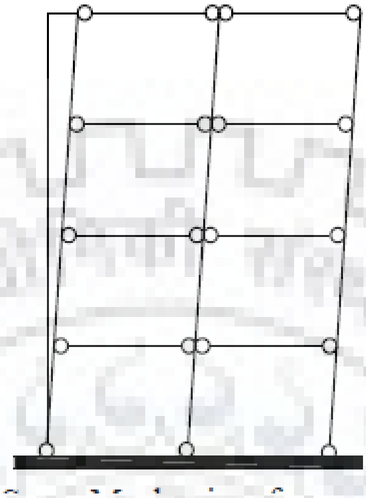


Figure 3.7 Beam Sway Mechanism for Frame

For this purpose, column flexural strengths at locations other than at the base or top and shear strengths at plastic hinge location must be amplified all through the structure. The relationship between design strength and basic strengths are given by the following equation:

$$\phi_s S_D \geq \phi^o w_f S_E \tag{3.33}$$

where, SD is the design strength defined according to the capacity design rules, φ_S is a strength reduction factor relating dependable and design strengths of the action ($\varphi_S = 1$ should be adopted for flexural design of plastic hinges and $\varphi_S < 1$ for other actions and locations), SE is the basic strength, i.e. the value corresponding to the design lateral force distribution determined from the DDBD method, φ^0 is an overstrength factor to account for the overcapacity at the plastic hinges and ω_f is the amplification due to higher mode effects. To apply capacity design rules an approximate method, as proposed in Priestley et al. (2008) is used.

Beam Flexure:

According to the desired inelastic mode, depicted in Fig.7, the plastic hinges should form at beam ends. For these regions the flexural design of plastic hinges is based on the larger of the moments due to factored gravity loads or corresponding to the design lateral forces from DDBD procedure (seismic moments). For the regions between the beam plastic hinges, design moments are found from the combination of reduced gravity loads applicable for the seismic design combination, and overstrength moment capacity at the beam hinges. Therefore, at a distance x from the left support, the total moment is given by:

$$M_x = M_{E,l}^o + (M_{E,r}^o - M_{E,l}^o) \times \frac{x}{L} + \frac{w_G^o L}{2} x - \frac{w_G^o x^2}{2} \tag{3.34}$$

where L is the beam span, $M^0_{E,l}$ ($=\varphi^0(x)M_{Bi,l}$) and $M^0_{E,r}$ ($=\varphi^0(x)M_{Bi,l}$) are the moments at left and right of column centerlines, respectively, and w_G^0 is the gravity load (dead and live) constant along the beam and amplified of 30% of seismic gravity moments are considered to account for elastic vertical response of the beam to vertical ground accelerations. Eq. (3.34) is defined taken into account that the beam moments cannot exceed M^0_E, the overstrength values at the beam plastic hinges; thus the design moments are defined by adding the gravity moments for a simple supported beam to the seismic moments.

Beam Shears:

The seismic beam shears corresponding to the plastic hinges locations are constant along the beam. As recommended in Priestley et al. (2008) the design shear force along the beam, should considerer the effects of beam vertical response (combined seismic shears with reduced gravity shears applicable for seismic load combinations), therefore:

$$V_x = \frac{(M_{E,r}^o - M_{E,l}^o)}{L_{j-1}} + \frac{w_G^o L_{j-1}}{2} - w_G^o x \tag{3.35}$$

Column Flexure:

Column end moments, other than at the base or top, and shears forces are amplified for both potential overstrength capacity at beam plastic hinges (material strengths exceed the design values) and dynamic amplification resulting for higher mode effects, which are not considered in the structural analysis.

The required column flexural strength according to DDBD capacity design rules is given by:

$$\phi_f M_N \geq \phi^o w_f M_E \tag{3.36}$$

where,

φ_f is the strength reduction factor;

MN is the design column moments;

φ^0 is the overstrength factor;

ω_f is the dynamic amplification factor, defined in the following;

M_E is the column moments resulting from lateral seismic forces (see Fig.3.5).

The overstrength factor φ^0 is the ratio of overstrength moment capacity to required capacity of the plastic hinges, as referred previously and could be obtained by moment-curvature analysis or using a default value. The effort to obtain overstrength factors by moment-curvature analysis maybe excessive for some structures and as suggested in Priestley et al.(2007) default value should be considered. It is possible to adopt two values for different situations, if the design is based on a strain-hardening model for the flexural reinforcement φ^0 is taken as 1.25, if not, it is recommended a value of 1.60.

The dynamic moment amplification factor ω_f is height and ductility dependent, as shown in Fig.3.13. From the first storey until ¾ of the total height, for one-way frames $\omega_{f,c}$ is given by:

$$w_{f,c} = 1.15 + 0.13(\mu^o - 1) \tag{3.37}$$

where,

$$\mu_D^o = \frac{\mu}{\sqrt{2}.\phi^o} \geq 1 \tag{3.38}$$

where, μ^0 is the reduced ductility corresponding to the average overstrength capacity of the beam hinges. The value at the base of the bottom storey and at the top should be taken as $\omega_{f,t} = 1.0$ (see Fig.3.8), where hinging at the column is acceptable, according with the desirable inelastic mode referred previously.(Priestley et al. 2007)

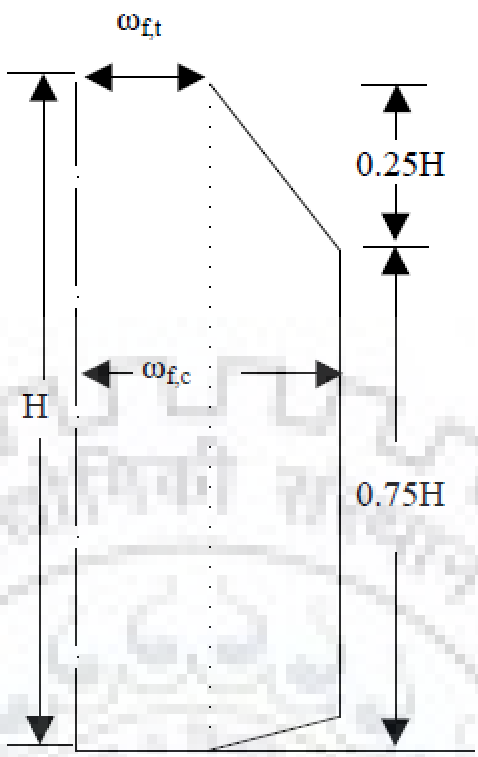


Figure 3.8 Dynamic amplification of frame column moments by Priestley et al.(2007).

Column Design Shear Forces:

According to Priestley et al. (2007) it has been stated that the dynamic amplification factor for column shear should be obtained by a constant offset of shear demand above design-force envelope with height, given by:

$$\phi_S V_N \geq \phi^o V_E + 0.1\mu V_{E,base} \leq \frac{M^o_{Ci,t} + M^o_{Ci,b}}{H_{Ci}} \tag{3.39}$$

where,

V_E is the shear demands from lateral seismic forces;

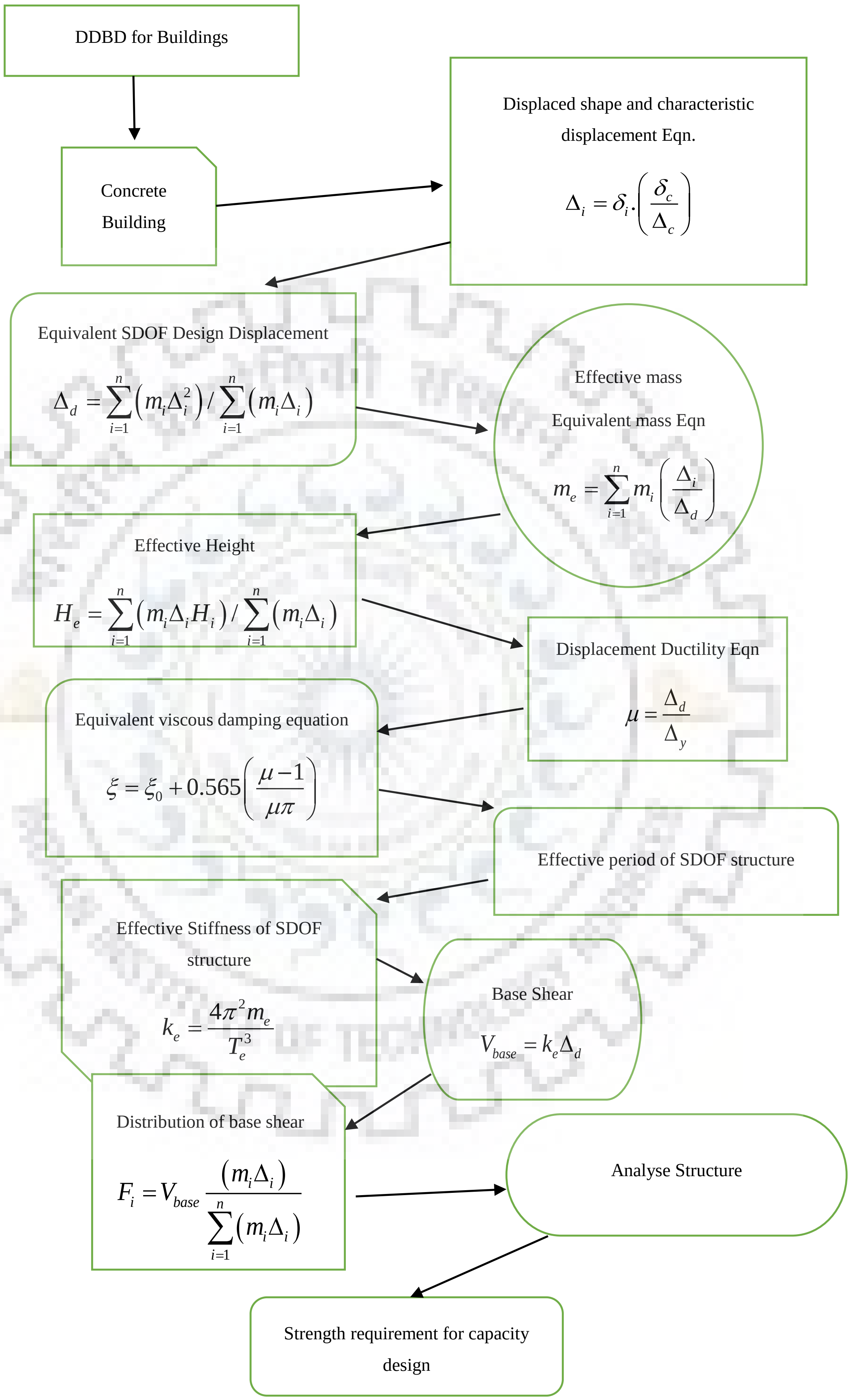
$V_{E,\,base}$ is the V_E value at the base of the column;

μ is the displacement ductility;

$M_{Ci,t}{}^{o}$ and $M_{Ci,b}{}^{o}$ are the moments at the top and bottom of the column, respectively, corresponding to development of plastic hinging;

H_{ci} is the clear height of the column.

Flow chart for DDBD design method is presented on next page.(Priestley et al. 2008)


DDBD for Buildings
Concrete Building
Displaced shape and characteristic displacement Eqn.
$\Delta_i = \delta_i . \left(\frac{\delta_c}{\Delta_c} \right)$
Equivalent SDOF Design Displacement
$\Delta_d = \sum_{i=1}^{n} (m_i \Delta_i^2) / \sum_{i=1}^{n} (m_i \Delta_i)$
Effective mass
Equivalent mass Eqn
$m_e = \sum_{i=1}^{n} m_i \left(\frac{\Delta_i}{\Delta_d} \right)$
Effective Height
$H_e = \sum_{i=1}^{n} (m_i \Delta_i H_i) / \sum_{i=1}^{n} (m_i \Delta_i)$
Displacement Ductility Eqn
$\mu = \frac{\Delta_d}{\Delta_y}$
Equivalent viscous damping equation
$\xi = \xi_0 + 0.565 \left(\frac{\mu - 1}{\mu \pi} \right)$
Effective period of SDOF structure
Effective Stiffness of SDOF structure
$k_e = \frac{4\pi^2 m_e}{T_e^3}$
Base Shear
$V_{base} = k_e \Delta_d$
Distribution of base shear
$F_i = V_{base} \frac{(m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)}$
Analyse Structure
Strength requirement for capacity design

3.3 DIRECT DISPLACEMENT BASED DESIGN FOR A PEAK GROUND ACCELERATION OF 0.36g

In this section is presented the frame design according to DDBD procedure for inner frame at Y=3 and for ag equal to 0.36g. The step-by-step procedure defined in above section is followed for design of floating column building.

Step 1. Definition of the design storey displacement, design displacement of the SDOF structure, equivalent mass and equivalent height

The normalized inelastic mode shape of the MDOF frame structure for this case of study is given by Eq. (1), with n=10. According to Priestley et al. (2007), for frame buildings the design displacement of the substitute SDOF structure will usually be governed by a specified drift limit in the lower storeys of the building. This shape implies that the maximum drift occurs between the ground and first storey. For design purpose and according Priestley et al. (2007) the drift limit was considered as 2.5 %. The critical design storey displacement for the first storey (H_1= 3 m) is thus Δ_c = 0.025 x 3 = 0.075 m and the critical normalized inelastic mode shape $\delta_c = \delta_1$= 0.13.

The design storey displacement profile is found from Eq. (3.3), reproduced herein by convenience:

$$\Delta_i = \omega_\theta \left(\frac{\Delta_c}{\delta_c} \right) = 1.0 \times \frac{0.075}{0.13} \delta_i = 0.577 \delta_i \qquad (3.40)$$

where, ω_θ is taken as 1.0.

Table 3.1 Calculations to obtain design displacement of the SDOF structure

Storey level i	Height Hi (m)	Mass Mi(kg)	δi	Δi	MiΔi	MiΔi^2	MiΔiHi
10	30	145790	1	0.577	84109.61	48524.77	2523288.462
9	27	161500	0.93	0.536	86650.96	46491.57	2339575.962
8	24	161500	0.853	0.492	79507.69	39142.25	1908184.615
7	21	161500	0.77	0.444	71743.27	31870.56	1506608.654
6	18	161500	0.68	0.392	63357.69	24855.71	1140438.462
5	15	161500	0.583	0.336	54350.96	18291.18	815264.4231
4	12	161500	0.48	0.277	44723.07	12384.85	536676.9231
3	9	161500	0.37	0.213	34474.04	7358.88	310266.3462
2	6	161500	0.253	0.146	23603.84	3449.79	141623.0769
1	3	158740	0.13	0.075	11905.5	892.91	35716.5
	0		0	0			
	Sum	1596530			554426.65	233262.50	11257643.42

In Fig.3.9 is depicted the design storey displacements profile according to the selected target drift limit, where the top target displacement Δ_{target} (roof displacement) is equal to 0.577m.

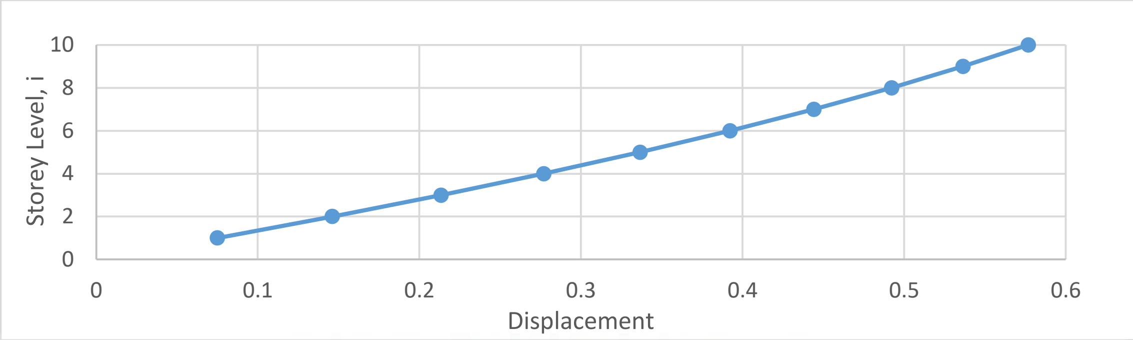


Figure 3.9 Design storey displacements

From Eq. (3.4) to Eq. (3.6) and from the values presented in Table 3.1 it is possible to derive the design displacement Δ_d, the equivalent mass me and equivalent height He of the SDOF structure. Therefore, the design displacement of the SDOF structure is 0.39m, the equivalent mass is 1317781.071 kg and the equivalent height is 20.305m (67.68% of building height).

Step 2. Estimation of the level of equivalent viscous damping

The design displacement ductility is given by Eq. (3.7), reproduced herein by convenience:

$$\mu = \frac{\Delta_d}{\Delta_y} \tag{3.41}$$

The equivalent yield displacement is the product between the yield rotation (see Eq. (9)) and the equivalent height of the SDOF structure. In this case of study, with beam depths for spans 1 and 2 hb1 = hb2 = 600 mm, the yield rotation θ_y is given by:

$$\theta_y = 0.5 \in_y L_{j-1} / h_b \tag{3.42}$$

$$\in_y = f_y / E_s = 1.1 \times 415 / 200000 = 0.00228 \tag{3.43}$$

where the design yield strength of steel is f_{ye} = 1.1 f_y, according to the recommendations in Priestley et al. (2007)for design material strengths for plastic hinge regions.

The equivalent yield displacement is given by:

$$\Delta_y = \frac{M_{1,i}\theta_{y1,i} + M_{2,i}\theta_{y2,i}}{M_{1,i} + M_{2,i}}.H_e = \frac{0.0114 + 0.0114}{2} \times 20.305 = 0.39\,\text{m} \tag{3.44}$$

The $M_{1,i}$ and $M_{2,i}$ are the contribution from both bays, and the considered relationship between them is $M_{1,i}/\ M_{2,i}$ = 1. Replacing in Eq. (3.41) the design displacement of the

SDOF structure and the equivalent yield displacement, the SDOF system design displacement ductility is μ=1.82. The equivalent viscous damping of the SDOF structure was obtained by Eq. (3.10), reproduced herein by convenience:

$$\xi = \xi_0 + 0.565\left(\frac{\mu - 1}{\mu\pi}\right) = 9.8\ \% \tag{3.45}$$

Step 3. Determination of the effective period

The effective period at peak displacement response is found from the design displacement spectrum defined for the equivalent viscous damping of ξ=9.8 % through Eq. (3.11) and Eq. (3.39), entering the design displacement of the equivalent SDOF structure Δd and determining the effective period Te (see Fig.3.11).(Calvi and Kowalsky 2007) The effective period of the SDOF structure is T_e = 4 sec

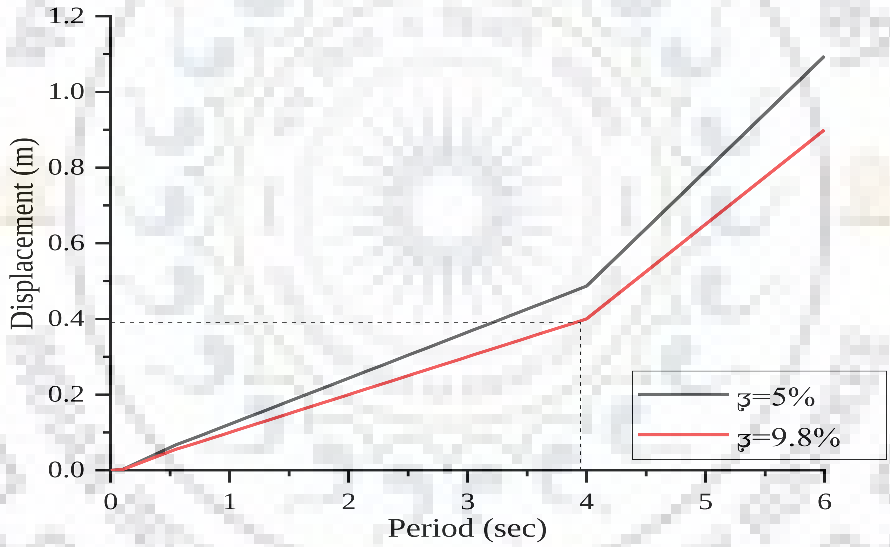


Figure 3.10 Design displacement spectrum

Step 4. Derivation of the effective stiffness and design base shear force

Knowing the effective period, it is possible to derive the effective stiffness and the design base shear force of the SDOF structure from Eq. (3.12) and Eq. (3.13), respectively. The effective stiffness of the SDOF structure is ke = 3251.48kN /m and the design base shear force is V_{base} = 1368 kN Table 3.2 presents a summary of the results obtained previously. Taken into the account of P-Δ effect as per Eq. (3.14) and Eq. (3.15)

Table 3.2 Results of DDBD in terms of displacement, equivalent yield displacement, ductility, effective mass, effective period and design base shear force

Δ_{target} (m)	Δ_d (m)	Δ_y (m)	μ	m_e (kg)	ξ (%)	Te (s)	V_{base} (kN)
0.577	0.390	0.231	1.82	1317781.071	9.8	4	1368

Step 5. Distribution of the design base shear force

The next step of the DDBD procedure involves the distribution of the design base shear force obtained for the SDOF structure in the real structure (MDOF structure), in a variation of the equivalent lateral force based. The distribution of the design base shear through the real structure was obtained by Eq. (3.17) and the values presented in Table 3.3.

Step 6. Design actions for MDOF Structure

The real structure is then analyzed under these forces (defined in step 5) and then the design moments are obtained.

<u>Beam Moments</u>

Table 3 shows the calculations to obtain the distribution of the design base shear through the real structures, the value of column shear forces in each alignment (shared between the exterior and interior columns in proportion 1:2 as suggested in Priestley et al. (2007)). Storey shear forces V_{Si} obtained from Eq. (3.29) are defined by summing the storey shear forces above the storey (see Fig.3.10). The last column of Table 3.6 presents the overturning moment OTM given by Eq. (3.20).

Table 3.3 Calculations for overturning moment

Stry lvl i	Ht (m)	MiΔi	Fi(kN)	Vci,2 (kN)	Vci,3 (kN)	Vci,4 (kN)	Vci,5 (kN)	Vsi (kN)	OTM (kN-m)
10	30	84109.6	362.0	60.3	120.7	120.7	60.3	362.0	0.0
9	27	86651.0	215.2	35.9	71.7	71.7	35.9	577.2	1085.9
8	24	79507.7	197.5	32.9	65.8	65.8	32.9	774.7	2817.5
7	21	71743.3	178.2	29.7	59.4	59.4	29.7	952.9	5141.6
6	18	63357.7	157.4	26.2	52.5	52.5	26.2	1110.3	8000.4
5	15	54351.0	135.0	22.5	45.0	45.0	22.5	1245.3	11331.3
4	12	44723.1	111.1	18.5	37.0	37.0	18.5	1356.4	15067.2
3	9	34474.0	85.6	14.3	28.5	28.5	14.3	1442.0	19136.4
2	6	23603.8	58.6	9.8	19.5	19.5	9.8	1500.7	23462.6
1	3	11905.5	29.6	4.9	9.9	9.9	4.9	1530.3	27964.6
									32555.4
Sum		554426.7	1530.3	255.0	510.1	510.1	255.0	10851.8	

P-Δ effects

According to Eq. (3.14) the stability index θ_Δ for this example is 0.15, therefore there is need to consider P-Δ effects, because $\theta_\Delta > 0.10$. Thus, the value of the design base shear force V_{base} to use in DDBD procedure is 1530.25 kN according to Eq.15(Wei et al. 2011)

$$V_{base} = k_e \Delta_d + C \frac{P\Delta_d}{H_e} \tag{3.46}$$

Based on Eq. (3.31) the total resisting moment provided at the column base is thus:

$$M_{C01,b} = 0.6V_{C01}.H_1 = 0.6 * 3 * 1530.25 = 2462.38 \text{ kN-m} (\approx 7.5\% \text{ OTM}) \quad (3.47)$$

According to Eq. (27), beam seismic shears corresponding to design lateral forces, admitting a relationship between beam moments $M_{B1,i}/M_{B2,i}=1$, for all span are given by:

$$\sum_{i=1}^{n} V_{B1,i} = \frac{1}{3}\left(32555.4 - 2462.38\right)/6 = 1671.83 \text{ kN} \quad (3.48)$$

These forces are distributed to the beams in proportion to the storey shears directly below the beams considered according to Eq. (3.28) and Eq. (3.29).

$$V_{B1,i} = 1671.83 \times V_{S,i} / 10851.8 = 0.154V_{S,i} \quad (3.49)$$

The resulting seismic beam shears for each span is presented Table 3.4.

Table 3.4 Calculations for seismic beam shears

Storey level	$V_{b,i}$ (kN)
10	55.76
9	88.92
8	119.35
7	146.81
6	171.05
5	191.85
4	208.97
3	222.16
2	231.20
1	235.75

The beam design moments at the column centerlines are given by Eq. (3.30) and at column faces by:

$$M_{B_{j-1},i} = V_{B_{j-1},i}(L_{j-1} - h_c)/2 \tag{3.50}$$

In Tables 3.5 and 3.6 are presented the values of the seismic design beam moments at the centerline and at the column face, respectively.

Table 3.5 Beam seismic moments at the centerline (ignoring gravity loads)

	SPAN = 6m	
Storey level, i	$M_{b,i,l}$ (kN-m)	$M_{b,i,r}$ (kN-m)
10	167.29	167.29
9	266.77	266.77
8	358.05	358.05
7	440.42	440.42
6	513.16	513.16
5	575.56	575.56
4	626.91	626.91
3	666.49	666.49
2	693.59	693.59
1	707.25	707.25

Table 3.6 Beam seismic moments at the face of the column (ignoring gravity loads)

	SPAN = 6m	
Storey level, i	$M_{b,i,l}$ (kN-m)	$M_{b,i,r}$ (kN-m)
10	150.56	-150.56
9	240.10	-240.10
8	322.25	-322.25
7	396.38	-396.38
6	461.85	-461.85
5	518.01	-518.01
4	564.22	-564.22
3	599.84	-599.84
2	624.23	-624.23
1	636.53	-636.53

According to Calvi and Kowalsky (2007) the flexural design of the beam plastic hinges is based on moments due to factored gravity loads or seismic moments corresponding to the design lateral forces (seismic case). Both values should be compared and the larger should be adopted for the design. Therefore, it is presented the calculations for these two cases. The factored gravity moments were obtained considered three load cases:1) the dead and live loads applied to both spans at the same time, 2) and 3) considering alternate live loads acting in the spans.

In Table 3.7 is presented the larger beam factored gravity moments for the three combinations. In Table 3.8 is shown the design beam moments for plastic hinges locations.

Table 3.7 Beam moments due to factored-gravity loads

Storey level	SPAN 1(kN-m)			SPAN 2(kN-m)			SPAN 3(kN-m)			SPAN 4(kN-m)			SPAN 5(kN-m)		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
10	629.41	89.97	-899.43	-163.51	59.50	-132.53	-130.07	75.13	-129.14	-133.47	59.58	-162.41	-896.95	90.05	627.10
9	794.01	104.65	-1124.66	-153.73	85.71	-178.27	-158.89	91.41	-157.77	-179.31	85.74	-152.62	-1121.81	104.68	791.21
8	772.35	104.73	-1102.86	-161.46	84.04	-174.14	-158.62	91.66	-157.54	-175.16	84.08	-160.36	-1100.12	104.77	769.69
7	792.48	104.67	-1123.12	-162.99	84.16	-172.35	-158.64	91.64	-157.55	-173.38	84.20	-161.89	-1120.38	104.70	789.82
6	811.94	104.55	-1142.79	-166.53	83.92	-169.33	-158.56	91.72	-157.48	-170.36	83.96	-165.42	-1140.06	104.59	809.28
5	837.66	104.35	-1168.92	-170.79	83.69	-165.57	-158.49	91.79	-157.41	-166.62	83.73	-169.67	-1166.19	104.39	835.00
4	870.02	104.50	-1200.99	-176.09	83.38	-160.95	-158.39	91.88	-157.32	-162.01	83.42	-174.95	-1198.22	104.54	867.33
3	901.39	102.67	-1236.01	-182.17	83.07	-155.58	-158.26	92.00	-157.21	-156.64	83.11	-181.03	-1233.18	102.72	898.66
2	976.87	110.54	-1295.77	-189.36	82.64	-149.19	-158.17	92.09	-157.12	-150.26	82.68	-188.21	-1292.73	110.53	973.82
1	841.61	69.91	-1241.76	-202.38	81.21	-139.94	-157.64	92.45	-156.94	-140.72	81.27	-201.49	-1238.30	70.19	838.72

Table 3.8 Design beam moments

Storey level i	SPAN 1(kN-m)		SPAN 2(kN-m)		SPAN 3(kN-m)		SPAN 4(kN-m)		SPAN 5(kN-m)	
	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
10	567.45	-842.86	163.51	150.56	150.56	150.56	150.56	150.56	-842.86	567.45
9	711.16	-1040.46	240.10	240.10	240.10	240.10	240.10	240.10	-1040.46	711.16
8	692.94	-1023.98	322.25	322.25	322.25	322.25	322.25	322.25	-1023.98	692.94
7	712.00	-1042.90	396.38	396.38	396.38	396.38	396.38	396.38	-1042.90	712.00
6	730.95	-1062.38	461.85	461.85	461.85	461.85	461.85	461.85	-1062.38	730.95
5	755.90	-1087.98	518.01	518.01	518.01	518.01	518.01	518.01	-1087.98	755.90
4	787.27	-1119.43	564.22	564.22	564.22	564.22	564.22	564.22	-1119.43	787.27
3	818.02	-1154.01	599.84	599.84	599.84	599.84	599.84	599.84	-1154.01	818.02
2	889.74	-1211.49	624.23	624.23	624.23	624.23	624.23	624.23	-1211.49	889.74
1	768.04	-1166.45	636.53	636.53	636.53	636.53	636.53	636.53	-1166.45	768.04

Column moments

The column moments presented in Table 3.9 corresponds to the design lateral forces and they were obtained by equilibrium considerations as described

Table 3.9 Column moments

Storey ,i		Col 1 (kN-m)	Col 2 (kN-m)	Col 3 (kN-m)	Col 4 (kN-m)	Col 5 (kN-m)	Col 6 (kN-m)
10	Top	91.28	180.24	360.48	360.48	180.24	91.28
	Bottom	-91.28	-361.22	-722.44	-722.44	-361.22	-91.28
9	Top	82.37	172.33	344.66	344.66	172.33	82.37
	Bottom	-82.37	-460.93	-921.86	-921.86	-460.93	-82.37
8	Top	72.74	255.18	510.36	510.36	255.18	72.74
	Bottom	-72.74	-642.53	-1285.06	-1285.06	-642.53	-72.74
7	Top	62.40	238.31	476.62	476.62	238.31	62.40
	Bottom	-62.40	-714.77	-1429.54	-1429.54	-714.77	-62.40
6	Top	51.35	311.55	623.10	623.10	311.55	51.35
	Bottom	-51.35	-866.70	-1733.41	-1733.41	-866.70	-51.35
5	Top	39.58	284.42	568.84	568.84	284.42	39.58
	Bottom	-39.58	-907.08	-1814.15	-1814.15	-907.08	-39.58
4	Top	27.10	346.74	693.48	693.48	346.74	27.10
	Bottom	-27.10	-1024.94	-2049.89	-2049.89	-1024.94	-27.10
3	Top	13.67	308.03	616.06	616.06	308.03	13.67
	Bottom	-13.67	-1029.05	-2058.10	-2058.10	-1029.05	-13.67
2	Top	13.67	358.12	716.24	716.24	358.12	13.67
	Bottom	-13.67	-1108.46	-2216.92	-2216.92	-1108.46	-13.67
1	Top	0.00	306.05	612.10	612.10	306.05	0.00
	Bottom	0.00	-459.08	-918.15	-918.15	-459.08	0.00

Step 7. Capacity design requirements for frames

In the following it is presented the application of the capacity design rules.

Beam Flexure

In DDBD procedure recommendations (J. N. Priestley et al. 2008) the material design strengths for design locations of intended plastic hinges, for concrete and reinforcement should be $f'_{ce}=1.3f'_c$ and $f_{ye}=1.1fy$, respectively. Where, f'_c is the specified (28 days) concrete compression strength, f'_{ce} is the expected compression strength of DDBD, f'_y is the specified minimum characteristic yield strength of steel and fye is the expected yield strength of steel for DDBD.

The required longitudinal reinforcement for beams ends is shown in Table 3.10. The longitudinal reinforcement was obtained for simple flexure; the values of Table 3.10 are reproduced for convenience.

$$Ast\ min = 0.85/fy = 0.85*100/415 = 0.45\ \% \tag{3.51}$$

$$Ast\ max = 4\ \% \tag{3.52}$$

Table 3.10 Design Details of beam 600x600 section and 400x600 section

	Beam 600x600									Check	
	Storey level i	Location	Ast (mm^2)	Øst (mm)	Location	Ast (mm^2)	Øst (mm)	Ast provided(mm^2)	ρ(%)	Ast>Astmin	Ast<Astmax(4%)
		Left End Mdes (kN-m)			Right End Mdes (kN-m)						
SPAN 1/5	10	567.45	2854.34	11ø20	842.86	4465.81	16ø20	16ø20	1.31	ok	ok
	9	711.16	3671.50	14ø20	1040.46	5759.32	21ø20	21ø20	1.66	ok	ok
	8	692.94	3565.24	13ø20	1023.98	5645.98	20ø20	20ø20	1.57	ok	ok
	7	712.00	3676.40	14ø20	1042.90	5776.23	21ø20	21ø20	1.66	ok	ok
	6	730.95	3787.88	14ø20	1062.38	5911.83	21ø20	21ø20	1.66	ok	ok
	5	755.90	3936.06	10ø25	1087.98	6092.56	14ø25	14ø25	1.77	ok	ok
	4	787.27	4124.73	10ø25	1119.43	6318.68	16ø25	16ø25	1.77	ok	ok
	3	818.02	4312.28	10ø25	1154.01	6572.71	17ø25	17ø25	1.91	ok	ok
	2	889.74	4760.69	11ø25	1211.49	7593.21	18ø25	18ø25	2.18	ok	ok
	1	768.04	4008.78	10ø25	1166.45	6665.59	17ø25	17ø25	1.91	ok	ok

	Beam 400x600									Check	
	Storey level i	Location	Ast (mm^2)	Øst (mm)	Location	Ast (mm^2)	Øst (mm)	Ast provided(mm^2)	ρ(%)	Ast>Astmin	Ast<Astmax(4%)
		Left End Mdes (kN-m)			Right End Mdes (kN-m)						
	10	163.51	781.18	4ø20	150.56	717.26	4ø20	4ø20	0.52	ok	ok
SPAN 2/3/4	9	240.10	1167.22	4ø20	240.10	1167.22	4ø20	4ø20	0.52	ok	ok
	8	322.25	1597.94	6ø20	322.25	1597.94	6ø20	6ø20	0.79	ok	ok
	7	396.38	2003.23	7ø20	396.38	2003.23	7ø20	7ø20	0.92	ok	ok
	6	461.85	2376.00	8ø20	461.85	2376.00	8ø20	8ø20	1.05	ok	ok
	5	518.01	2708.27	9ø20	518.01	2708.27	9ø20	9ø20	1.18	ok	ok
	4	564.22	2991.33	10ø20	564.22	2991.33	10ø20	10ø20	1.31	ok	ok
	3	599.84	3216.08	12ø20	599.84	3216.08	12ø20	12ø20	1.44	ok	ok
	2	624.23	3373.51	12ø20	624.23	3373.51	12ø20	12ø20	1.44	ok	ok
	1	636.53	3454.07	12ø20	636.53	3454.07	12ø20	12ø20	1.44	ok	ok

The reinforcement values for beams were obtained considering the requests specified in IS 456:2000. For this case study, the reinforcement of the compression zone will be equal to the reinforcement of the tension zone.

The design beam moments at mid span due to seismic loads and the correspondent beam longitudinal reinforcement at Table 3.11. These are obtained from the combination of reduced gravity loads applicable for the design seismic combination, and overstrenght moment capacity at beam hinge location, according to Eq. (3.34). The overstrenght factor φ_0 is considered equal to 1.25. The design material strengths used are the characteristic material strengths, without amplification. Shear reinforcement of 4 legged 8mm bars are provided and these are satisfying the beam shear force, given in Eq. (3.35).

Table 3.11 Design beam moments at mid span and longitudinal reinforcement details

	Beam 600x600					Check	
	Storey Level, i	Mom. (kN-m)	Ast (mm^2)	Øst (mm)	ρ(%)	Ast>Astmin	Ast<Astmax (4%)
Span 1/5	10	278.89	1339.10	6ø20	0.44	ok	ok
	9	308.94	1490.15	6ø20	0.44	ok	ok
	8	308.94	1490.15	6ø20	0.44	ok	ok
	7	308.94	1490.15	6ø20	0.44	ok	ok
	6	308.94	1490.15	6ø20	0.44	ok	ok
	5	308.94	1490.15	6ø20	0.44	ok	ok
	4	308.94	1490.15	6ø20	0.44	ok	ok
	3	308.94	1490.15	6ø20	0.44	ok	ok
	2	308.94	1490.15	6ø20	0.44	ok	ok
	1	303.66	1463.51	6ø20	0.44	ok	ok

	Beam 400x600					Check	
	Storey Level i	Mom. (kN-m)	Ast (mm2)	Øst (mm)	ρ(%)	Ast>Astmin	Ast<Astmax (4%)
Span 2/3/4	10	278.89	1339.10	6ø20	0.65	ok	ok
	9	308.94	1490.15	6ø20	0.65	ok	ok
	8	308.94	1490.15	6ø20	0.65	ok	ok
	7	308.94	1490.15	6ø20	0.65	ok	ok
	6	308.94	1490.15	6ø20	0.65	ok	ok
	5	308.94	1490.15	6ø20	0.65	ok	ok
	4	308.94	1490.15	6ø20	0.65	ok	ok
	3	308.94	1490.15	6ø20	0.65	ok	ok
	2	308.94	1490.15	6ø20	0.65	ok	ok
	1	303.66	1463.51	6ø20	0.65	ok	ok

Column Flexure

The required column flexural strength according to DDBD capacity design rules is given by Eq. (3.36), reproduced herein by convenience.

$$\phi_f M_N \geq \phi^o w_f M_E \tag{3.53}$$

φ^0 is the overstrenght factor considered as 1.25;

ω_f is the dynamic amplification factor - Eq. (37);

M_E is the column moments resulting from design forces (given in Table 9);

φ_f is the strength reduction factor considered as 0.9.

The design column moments and axial forces are shown in Table 3.12 and 3.13, respectively. As column 1 and 6 are not taking part in lateral resisting system thus provided reinforcement as gravity load only. And axial forces in column 1 and 6 is very less or close to 0(Zero) as compared to other columns.

Table 3.12 Design column moments

Storey level i		w_f	Col2 (kN-m)	Col3 (kN-m)	Col4 (kN-m)	Col5 (kN-m)
10	Top	1.00	250.33	500.66	500.66	250.33
	Bottom	1.00	-501.69	-1003.38	-1003.38	-501.69
9	Top	1.21	289.32	578.65	578.65	289.32
	Bottom	1.21	-773.86	-1547.73	-1547.73	-773.86
8	Top	1.21	428.42	856.85	856.85	428.42
	Bottom	1.21	-1078.76	-2157.52	-2157.52	-1078.76
7	Top	1.21	400.11	800.21	800.21	400.11
	Bottom	1.21	-1200.04	-2400.09	-2400.09	-1200.04
6	Top	1.21	523.07	1046.14	1046.14	523.07
	Bottom	1.21	-1455.12	-2910.25	-2910.25	-1455.12
5	Top	1.21	477.52	955.03	955.03	477.52
	Bottom	1.21	-1522.91	-3045.81	-3045.81	-1522.91
4	Top	1.21	582.15	1164.29	1164.29	582.15
	Bottom	1.21	-1720.79	-3441.59	-3441.59	-1720.79
3	Top	1.21	517.16	1034.31	1034.31	517.16
	Bottom	1.21	-1727.69	-3455.39	-3455.39	-1727.69
2	Top	1.21	601.25	1202.50	1202.50	601.25
	Bottom	1.21	-1861.01	-3722.02	-3722.02	-1861.01
1	Top	1.00	382.56	765.13	765.13	382.56
	Bottom	1.00	-573.84	-1147.69	-1147.69	-573.84

Table 3.13 Axial Forces in Columns

Storey level i		Column 2 Axial(kN)	Column 3 Axial(kN)	Column 4 Axial(kN)	Column 5 Axial(kN)
10	Top	-472.13	-324.36	-324.41	-471.35
	Bottom	-472.13	-324.36	-324.41	-471.35
9	Top	-1022.93	-701.27	-701.27	-1022.93
	Bottom	-1022.93	-701.27	-701.27	-1022.93
8	Top	-1575.19	-1076.80	-1076.80	-1575.19
	Bottom	-1575.19	-1076.80	-1076.80	-1575.19
7	Top	-2135.82	-1451.01	-1451.01	-2135.82
	Bottom	-2135.82	-1451.01	-1451.01	-2135.82
6	Top	-2707.68	-1823.56	-1823.56	-2707.68
	Bottom	-2707.68	-1823.56	-1823.56	-2707.68
5	Top	-3293.13	-2193.70	-2193.70	-3293.13
	Bottom	-3293.13	-2193.70	-2193.70	-3293.13
4	Top	-3896.05	-2560.94	-2560.94	-3896.05
	Bottom	-3896.05	-2560.94	-2560.94	-3896.05
3	Top	-4518.50	-2924.80	-2924.80	-4518.50
	Bottom	-4518.50	-2924.80	-2924.80	-4518.50
2	Top	-5171.94	-3283.82	-3283.82	-5171.94
	Bottom	-5171.94	-3283.82	-3283.82	-5171.94
1	Top	-5807.71	-3637.85	-3637.85	-5807.71
	Bottom	-5807.71	-3637.85	-3637.85	-5807.71

The required longitudinal reinforcement for the rectangular column sections was obtained considering composed bending and it is presented in Table 3.14. The required longitudinal reinforcement was obtained using SP 16 aid to IS 456 :2000.

The design material design strengths used are the characteristic ones, without amplification, except for the column base, where it is expected the formation of plastic hinges (beam-sway mechanism).

Table 3.14 Longitudinal reinforcement bars /face

Storey,i	Col.1		Col. 2		Col. 3		Col. 4		Col. 5		Col. 6	
	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars
10	3600	8ø25	5760	8ø32	11520	15ø32	11520	15ø32	5760	8ø32	3600	8ø25
9	3600	8ø25	11520	15ø32	20160	26ø32	20160	26ø32	11520	15ø32	3600	8ø25
8	3600	8ø25	11520	15ø32	20880	26ø32	20880	26ø32	11520	15ø32	3600	8ø25
7	3600	8ø25	17280	22ø32	17280	22ø32	17280	22ø32	17280	22ø32	3600	8ø25
6	3600	8ø25	18000	23ø32	18720	24ø32	18720	24ø32	18000	23ø32	3600	8ø25
5	3600	8ø25	18720	24ø32	20160	26ø32	20160	26ø32	18720	24ø32	3600	8ø25
4	3600	8ø25	20160	26ø32	20880	26ø32	20880	26ø32	20160	26ø32	3600	8ø25
3	3600	8ø25	20160	26ø32	20160	26ø32	20160	26ø32	20160	26ø32	3600	8ø25
2	3600	8ø25	20160	26ø32	20160	26ø32	20160	26ø32	20160	26ø32	3600	8ø25
1	3600	8ø25	14400	18ø32	14400	18ø32	14400	18ø32	14400	18ø32	3600	8ø25

Table 3.15 Reinforcement ratios (ρ (%))

Storey, i	Col. 2 Ac(0.36 m^2)	Col. 3 Ac(0.36 m^2)	Col. 4 Ac(0.36 m^2)	Col. 5 Ac(0.36 m^2)
10	1.6	3.2	3.2	1.6
9	3.2	5.6	5.6	3.2
8	3.2	5.8	5.8	3.2
7	4.8	4.8	4.8	4.8
6	5	5.2	5.2	5
5	5.2	5.6	5.6	5.2
4	5.6	5.8	5.8	5.6
3	5.6	5.6	5.6	5.6
2	5.6	5.6	5.6	5.6
1	4	4	4	4

Column 1 and 6 are provided with 1% reinforcement ratio. According to IS 456 (2000) for a structure the longitudinal reinforcement ratio for columns should be greater than 0.008 (0.8%) and not less of 0.06 (6%). From Table 3.15 it can be stated that these requirements are fulfilled.

Similarly, the exterior frame of floating column building designed as per DDBD and the reinforcement details of beam as well as columns has been obtained and presented below.

Δ_{target} (m)	Δ_d (m)	Δ_y (m)	μ	m_e (kg)	ξ (%)	Te (s)	V_{base} (kN)
0.577	0.390	0.231	1.82	798582.30	9.81	4	768.46

After Considering P Delta effect (Wei et al. 2011), the base shear changes to V_{base} = 866.78 kN

Table 3.16 Reinforcement in beam in exterior frame

	Beam 500x600									Check	
		Location			Location						
	Storey level i	Left End Mdes (kN-m)	Ast (mm^2)	Øst (mm)	Right End Mdes (kN-m)	Ast (mm^2)	Øst (mm)	Ast prov. (mm^2)	ρ(%)	Ast>Astmin	Ast<Astmax
SPAN 1/5	10	441.05	2204.04	8ø20	604.98	3132.1	10ø20	10ø20	1.05	ok	ok
	9	518.36	2632.62	9ø20	711.32	3777.2	13ø20	13ø20	1.36	ok	ok
	8	505.46	2560.07	9ø20	698.6	3698.1	12ø20	12ø20	1.26	ok	ok
	7	514.29	2609.72	9ø20	707.41	3752.8	12ø20	12ø20	1.26	ok	ok
	6	521.48	2650.28	9ø20	715.05	3800.6	13ø20	13ø20	1.36	ok	ok
	5	531.23	2705.53	6ø25	725.31	3865.1	8ø25	8ø25	0.84	ok	ok
	4	543.72	2776.66	6ø25	737.91	3944.7	9ø25	9ø25	0.94	ok	ok
	3	555.12	2841.97	6ø25	750.98	4028	9ø25	9ø25	0.94	ok	ok
	2	587.09	3027.19	7ø25	775.93	4188.9	9ø25	9ø25	0.94	ok	ok
	1	526.93	2681.11	6ø25	749.67	4019.6	9ø25	9ø25	0.94	ok	ok

	Beam 400x600									Check	
	Storey level, i	Location	Ast (mm^2)	Øst (mm)	Location	Ast (mm^2)	Øst (mm)	Ast prov. (mm^2)	ρ(%)	Ast>Astmin	Ast<Astmax
		Left End Mdes (kN-m)			Right End Mdes (kN-m)						
SPAN 2/3/4	10	105.3	496.73	3ø20	84.14	395.17	3ø20	3ø20	0.39	ok	ok
	9	135.12	641.52	3ø20	135.12	641.52	3ø20	3ø20	0.39	ok	ok
	8	181.89	872.56	3ø20	181.89	872.56	3ø20	3ø20	0.39	ok	ok
	7	224.1	1085.4	4ø20	224.1	1085.4	4ø20	4ø20	0.52	ok	ok
	6	261.37	1277.03	5ø20	261.37	1277	5ø20	5ø20	0.65	ok	ok
	5	293.35	1444.31	5ø20	293.35	1444.3	5ø20	5ø20	0.65	ok	ok
	4	319.66	1584.05	6ø20	319.66	1584.1	6ø20	6ø20	0.79	ok	ok
	3	339.94	1693.13	6ø20	339.94	1693.1	6ø20	6ø20	0.79	ok	ok
	2	353.82	1768.5	7ø20	353.82	1768.5	7ø20	7ø20	0.79	ok	ok
	1	360.75	1806.32	7ø20	360.75	1806.3	7ø20	7ø20	0.79	ok	ok

Table 3.17 Design beam moments at mid span and longitudinal reinforcement details

	Beam 500x600					Check	
	Storey level, i	Mom. (kN-m)	Ast (mm^2)	Øst (mm)	ρ(%)	Ast>Astmin	Ast<Astmax (4%)
Span 1/5	10	164.70	781.43	4ø20	0.42	ok	ok
	9	187.81	894.67	4ø20	0.42	ok	ok
	8	187.81	894.67	4ø20	0.42	ok	ok
	7	187.81	894.67	4ø20	0.42	ok	ok
	6	187.81	894.67	4ø20	0.42	ok	ok
	5	187.81	894.67	4ø20	0.42	ok	ok
	4	187.81	894.67	4ø20	0.42	ok	ok
	3	187.81	894.67	4ø20	0.42	ok	ok
	2	187.81	894.67	4ø20	0.42	ok	ok
	1	182.53	868.72	4ø20	0.42	ok	ok
	Beam 400x600					Check	
	Storey level i	Mom. (kN-m)	Ast (mm2)	Øst (mm)	ρ(%)	Ast>Astmin	Ast<Astmax (4%)
Span 2/3/4	10	164.70	781.43	4ø20	0.52	ok	ok
	9	187.81	894.67	4ø20	0.52	ok	ok
	8	187.81	894.67	4ø20	0.52	ok	ok
	7	187.81	894.67	4ø20	0.52	ok	ok
	6	187.81	894.67	4ø20	0.52	ok	ok
	5	187.81	894.67	4ø20	0.52	ok	ok
	4	187.81	894.67	4ø20	0.52	ok	ok
	3	187.81	894.67	4ø20	0.52	ok	ok
	2	187.81	894.67	4ø20	0.52	ok	ok
	1	182.53	868.72	4ø20	0.52	ok	ok

Table 3.18 Longitudinal reinforcement bars /face

Storey, i	Col.1		Col. 2		Col. 3		Col. 4		Col. 5		Col. 6	
	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars
10	3600	8ø25	3600	8ø25	3600	8ø25	3600	8ø25	3600	8ø25	3600	8ø25
9	3600	8ø25	3600	8ø25	5760	12ø25	5760	12ø25	3600	8ø25	3600	8ø25
8	3600	8ø25	7200	16ø25	11520	24ø25	11520	24ø25	7200	16ø25	3600	8ø25
7	3600	8ø25	8640	18ø25	8640	18ø25	8640	18ø25	8640	18ø25	3600	8ø25
6	3600	8ø25	8640	18ø25	11520	24ø25	11520	24ø25	8640	18ø25	3600	8ø25
5	3600	8ø25	10080	22ø25	10080	22ø25	10080	22ø25	10080	22ø25	3600	8ø25
4	3600	8ø25	12960	28ø25	10080	22ø25	10080	22ø25	12960	28ø25	3600	8ø25
3	3600	8ø25	12960	28ø25	11520	24ø25	11520	24ø25	12960	28ø25	3600	8ø25
2	3600	8ø25	8640	18ø25	8640	18ø25	8640	18ø25	8640	18ø25	3600	8ø25
1	0	0	3600	8ø25	3600	8ø25	3600	8ø25	3600	8ø25	0	0

Table 3.19 Reinforcement ratios (ρ (%))

Storey, i	Col. 2 Ac(0.36 m^2)	Col. 3 Ac(0.36 m^2)	Col. 4 Ac(0.36 m^2)	Col. 5 Ac(0.36 m^2)
10	1	1	1	1
9	1	1.6	1.6	1
8	2	3.2	3.2	2
7	2.4	2.4	2.4	2.4
6	2.4	3.2	3.2	2.4
5	2.8	2.8	2.8	2.8
4	3.6	2.8	2.8	3.6
3	3.6	3.2	3.2	3.6
2	2.4	2.4	2.4	2.4
1	1	1	1	1

Beam reinforcement for exterior frame is given in Table 3.16-3.17. Column 1 and 6 are provided with 1% reinforcement ratio. According to IS 456 (2000) for a structure the longitudinal reinforcement ratio for columns should be greater than 0.008 (0.8%) and not less of 0.06 (6%). From Table 3.18-3.19 it can be stated that these requirements are fulfilled.

Accidental eccentricity is envisioned to provide consideration of uncertainty in calculations of mass and stiffness distributions. Typically, this is effected in force-based design by considering two alternate positions of the centre of mass, separated by ±0.05Z* from the calculated location of Cm, where Z is the building plan dimension perpendicular to the direction of seismic force considered. However, as it has been pointed out by Paulay this appears to be unreliable with other aspects of seismic design, where larger uncertainties may exist. In particular, calculation of the location of the centre of mass is likely to be one of the most reliable of the calculated parameters in seismic design. Also, design for accidental eccentricity is likely to be incompetent, since it involves increasing the strength of all elements which will not reduce the apparent torsional strength eccentricity. In fact, it can be argued that it will exacerbate the problem, since the overall base shear capacity will be increased in proportion to the strength increase of the individual elements, without reducing the strength eccentricity. Hence torsional moments are likely to increase. As a consequence of these considerations we do not recommend to take accidental eccentricity to be taken into account.(Calvi and Kowalsky 2007)

CHAPTER 4 DESIGN OF FLOATING COLUMN BUILDING BY FORCE BASED METHOD

Current code uses the FBD procedure for seismic design. In this procedure, the earthquake excitation is an elastic response spectrum computed for 5% damping for most types of building. The response spectrum is divided by a reduction factor R depending on the type of structure. Modal analysis is used to compute the main periods of the structure and their contribution to seismic response. The element/member forces obtained from this analysis or from a simplified procedure called the Equivalent Lateral Load procedure are the seismic demands used for the design and the designer for capacity design shall provide member capacities greater than the demands. Since the members are designed for lower forces than the earthquake demand, the members will yield. Special detailing shall be provided to ensure that the members can accommodate the displacement demands imposed by the earthquake. A check of the deflections is performed against allowable limits. If comes greater than the prescribed limit then iterates the whole process again and do the checks(Muljati et al. 2015) A more detailed presentation of the FBD is presented below:

a) A preliminary design based on structure geometry, gravity loads and experience is used to define the member sizes such as column heights, inertia masses and design spectrum, this will be the first estimate of the member sizes.
b) Estimate member initial stiffness i.e. based on the size estimates, obtained in step one. Reduced stiffness may be used for some elements to account for member softening and cracking.
c) Modal analysis is performed to obtain significant periods and participations factors to apply response spectrum method.
d) Using the response spectrum method, participating accelerations will be obtained as shown in Fig.4.1 as per IS 1893(part 1):2016 and will be accounted for force calculation.
e) Elastic base shear is obtained, and this shear is reduced by a factor R that depends on the type of structure as well as to account for inelastic behavior of structure.
f) Compute inertial forces based on the base shear obtained above or use the Equivalent Lateral Load Procedure Analyze the structure using this inertial forces to obtain the member forces.

g) Design the members to ensure that the axial and bending capacities are greater than the demands. Capacity design principles are used making sure that yielding occurs in the beam and not in the columns (strong column-weak beam principle) and also for shear design of the elements.

h) Comply with allowable code displacement limits, if the displacements are larger than the allowable, the structure shall be modified and reanalyzed until compliance.

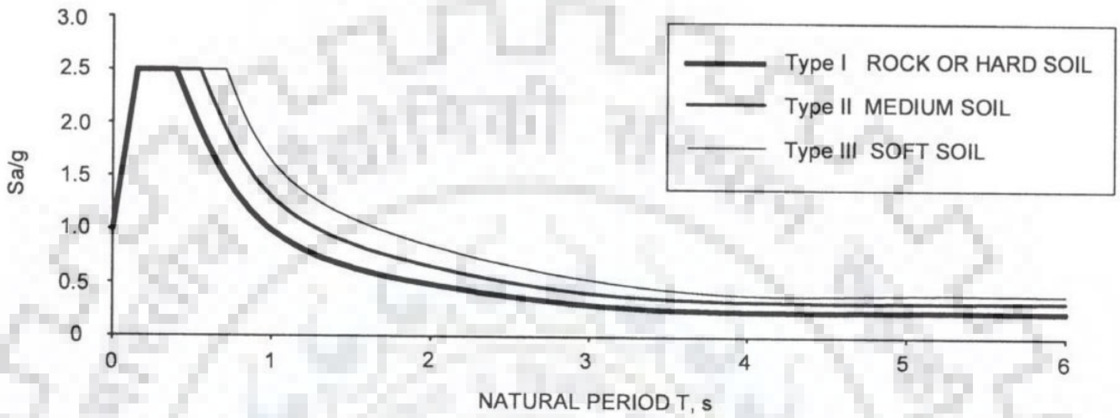


Figure 4.1 Force Base Design 5% Acceleration Spectrum

4.1 FORCE BASED DESIGN OF FLOATING COLUMN BUILDING

The floating column building (see Fig.4.1) is analyzed for gravity and seismic as per IS 456 (2000) and IS 1893(part 1):2016. Loads, time period and base shear are manually calculated as per the code (from time period provided by IS 1893(part 1):2016) as SAP2000 takes actual time period to calculate lateral forces) for equivalent static method and response spectrum method and put on the structure then analyzed in structure program software and reinforcement details are shown below

Interior frame is 2-2 and 3-3 section. V_{base}=Ah*W= 2008.17 kN (4.1)

$$\text{Time period} = 0.09h / \sqrt{d} = 0.64 \text{ sec} \tag{4.2}$$

Exterior frame is 1-1 and 4-4 section. V_{base}=Ah*W= 1216.28 kN (4.3)

$$\text{Time period} = 0.09h / \sqrt{d} = 0.64 \text{ sec} \tag{4.4}$$

Torsion Irregularity is checked and found to be in limits, thus it is torsional irregular and accidental torsion of 5% is taken into account to calculate base shear. Also the capacity design is done as per IS 13920:2016 i.e. strong column weak beam (beam side sway failure mechanism)

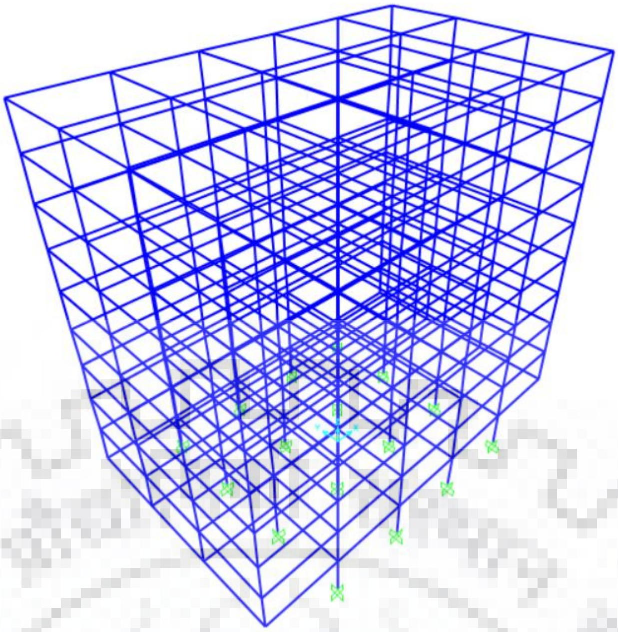


Fig. 4.1 Floating column building elements

Design base shear is distributed in floor levels according to mass storey given in IS 1893(part 1):2016

$$F_i = V_{base} \frac{(m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)} \quad (4.5)$$

Table 4.1 Lateral Load Distribution with Height for interior frame

Storey, i	Wi(kg)	Hi(m)	wiHi2	Qi(kN)
10	145790	30	131211000	483.0924
9	161500	27	117733500	433.4709
8	161500	24	93024000	342.4956
7	161500	21	71221500	262.2232
6	161500	18	52326000	192.6537
5	161500	15	36337500	133.7873
4	161500	12	23256000	85.62389
3	161500	9	13081500	48.16344
2	161500	6	5814000	21.40597
1	158740	3	1428660	5.260037
		sum	545433660	2008.176

Table 4.2Lateral load Distribution with Height for exterior frame

Storey, i	Wi(kg)	Hi(m)	wiHi2	Qi(kN)
10	86100	30	77490000	285.3025
9	98180	27	71573220	263.5181
8	98180	24	56551680	208.2118
7	98180	21	43297380	159.4122
6	98180	18	31810320	117.1192
5	98180	15	22090500	81.33275
4	98180	12	14137920	52.05296
3	98180	9	7952580	29.27979
2	98180	6	3534480	13.01324
1	95420	3	858780	3.161854
		sum	329296860	1212.404

Reinforcement details of beam and columns in floating building are designed as per loads given by IS Code 875(part 1 and 2) 2015 and the combination given in Table 2.1 is taken to calculated demand on the building and capacity design is provided as per IS 13920:2016. P-Δ effects are also taken into consideration by SAP2000 to calculate base shear. The reinforcement details for beam is given in Table 4.3.

Table 4.3 Reinforcement Details of beam of interior frame 2-2 and 3-3 by FBD

	Beam 600x600			Check		
	Storey, i	Ast(mm^2)	Øst	ρ (%)	Ast>Astmin	Ast<Astmax
Span 1/5	10	6696	22ø20	1.92	ok	ok
	9	8676	28ø20	2.44	ok	ok
	8	8676	28ø20	2.44	ok	ok
	7	8676	28ø20	2.44	ok	ok
	6	8676	28ø20	2.44	ok	ok
	5	8676	18ø25	2.45	ok	ok
	4	8676	18ø25	2.45	ok	ok
	3	9576	20ø25	2.73	ok	ok
	2	9576	20ø25	2.73	ok	ok
	1	9576	20ø25	2.73	ok	ok

	Beam 400x600			Check		
	Storey, i	Ast(mm^2)	Øst	ρ (%)	Ast>Astmin	Ast<Astmax
Span 2/3/4	10	2016	7ø20	0.92	ok	ok
	9	2016	7ø20	0.92	ok	ok
	8	2016	7ø20	0.92	ok	ok
	7	2400	8ø20	1.05	ok	ok
	6	2400	8ø20	1.05	ok	ok
	5	2400	5ø25	1.02	ok	ok
	4	2904	6ø25	1.23	ok	ok
	3	2904	6ø25	1.23	ok	ok
	2	2904	6ø25	1.23	ok	ok
	1	2904	6ø25	1.23	ok	ok

Columns are also designed as per code IS 456:2000 and their reinforcement tabular data is presented in Table 4.4-4.5.

Table 4.4 Longitudinal reinforcement details of column of interior frame 2-2 and 3-3 by FBD

Storey, i	Col.1		Col. 2		Col. 3		Col. 4		Col. 5		Col. 6	
	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars
10	7848	16ø25	7848	16ø25	7848	16ø25	7848	16ø25	7848	16ø25	7848	16ø25
9	3924	8ø25	3600	8ø25	3924	8ø25	3924	8ø25	3924	8ø25	3924	8ø25
8	3924	8ø25	7200	15ø25	3924	8ø25	3924	8ø25	3924	8ø25	3924	8ø25
7	5904	13ø25	8640	18ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
6	5904	13ø25	8640	18ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
5	5904	13ø25	10080	21ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
4	5904	13ø25	12960	27ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
3	5904	13ø25	12960	27ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
2	9828	21ø25	8640	18ø25	9828	21ø25	9828	21ø25	9828	21ø25	9828	21ø25
1	0	0	3600	8ø25	13752	29ø25	13752	29ø25	13752	29ø25	0	0

Table 4.5 Reinforcement percentage ratio for interior frame 2-2 and 3-3 by FBD

Storey, i	Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
	Ac ($0.36\ m^2$)	Ac ($0.36\ m^2$)	Ac ($0.36\ m^2$)	Ac ($0.36\ m^2$)	Ac ($0.36\ m^2$)	Ac ($0.36\ m^2$)
10	2.18	2.18	2.18	2.18	2.18	2.18
9	1.09	1.09	1.09	1.09	1.09	1.09
8	1.09	1.09	1.09	1.09	1.09	1.09
7	1.64	1.64	1.64	1.64	1.64	1.64
6	1.64	1.64	1.64	1.64	1.64	1.64
5	1.64	1.64	1.64	1.64	1.64	1.64
4	1.64	1.64	1.64	1.64	1.64	1.64
3	1.64	1.64	1.64	1.64	1.64	1.64
2	2.73	2.73	2.73	2.73	2.73	2.73
1	0	3.82	3.82	3.82	3.82	0

Exterior frame 1-1 and 4-4 reinforcement (see Fig 4.1) details in Table 4.6-4.7-4.8.

Table 4.6 Reinforcement Details of beam of exterior frame 1-1 and 4-4 by FBD

	Beam 500x600			Check		
	Storey, i	Ast(mm2)	Øst	ρ (%)	Ast>Astmin	Ast<Astmax
Span 1/5	10	3132.123	10ø20	0.87	ok	ok
	9	3777.235	13ø20	1.13	ok	ok
	8	3698.05	12ø20	1.05	ok	ok
	7	3752.84	12ø20	1.05	ok	ok
	6	3800.588	13ø20	1.13	ok	ok
	5	3865.046	8ø25	1.09	ok	ok
	4	3944.697	9ø25	1.23	ok	ok
	3	4028.038	9ø25	1.23	ok	ok
	2	4188.945	9ø25	1.23	ok	ok
	1	4019.626	9ø25	1.23	ok	ok

	Beam 400x600			Check		
	Storey, i	Ast(mm2)	Øst	ρ (%)	Ast>Astmin	Ast<Astmax
Span 2/3/4	10	1536	5ø20	0.65	ok	ok
	9	2016	7ø20	0.92	ok	ok
	8	2016	7ø20	0.92	ok	ok
	7	2016	7ø20	0.92	ok	ok
	6	2016	7ø20	0.92	ok	ok
	5	2016	5ø25	1.02	ok	ok
	4	2016	5ø25	1.02	ok	ok
	3	2016	5ø25	1.02	ok	ok
	2	2016	5ø25	1.02	ok	ok
	1	2016	5ø25	1.02	ok	ok

Table 4.7 Reinforcement percentage ratio for exterior frame 1-1 and 4-4 by FBD

Storey, i	Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
	Ac (0.36 m^2)	Ac (0.36 m^2)	Ac (0.36 m^2)	Ac (0.36 m^2)	Ac (0.36 m^2)	Ac (0.36 m^2)
10	2.18	1.64	1.05	1.05	1.64	1.64
9	1.1	1.1	1.1	1.1	1.1	1.1
8	1.1	1.1	1.1	1.1	1.1	1.1
7	1.64	1.64	1.64	1.64	1.64	1.64
6	1.64	1.64	1.64	1.64	1.64	1.64
5	1.64	1.64	1.64	1.64	1.64	1.64
4	1.64	1.64	1.64	1.64	1.64	1.64
3	1.64	1.64	1.64	1.64	1.64	1.64
2	1.64	1.64	1.64	1.64	1.64	1.64
1	0	2.73	2.73	2.73	2.73	0

Table 4.8 Longitudinal reinforcement details of column of exterior frame 1-1 and 4-4 by FBD

Storey, i	Col.1		Col. 2		Col. 3		Col. 4		Col. 5		Col. 6	
	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars	Ast (mm^2)	Bars
10	7848	16ø25	5904	13ø25	3780	8ø25	3780	8ø25	5904	13ø25	7848	16ø25
9	3960	9ø25	3600	8ø25	3960	9ø25	3960	9ø25	3960	9ø25	3960	9ø25
8	3960	9ø25	7200	15ø25	3960	9ø25	3960	9ø25	3960	9ø25	3960	9ø25
7	5904	13ø25	8640	18ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
6	5904	13ø25	8640	18ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
5	5904	13ø25	10080	21ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
4	5904	13ø25	12960	27ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
3	5904	13ø25	12960	27ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
2	5904	13ø25	8640	18ø25	5904	13ø25	5904	13ø25	5904	13ø25	5904	13ø25
1	0	0	3600	8ø25	9828	21ø25	9828	21ø25	9828	21ø25	0	0

CHAPTER 5 RESULTS AND DISCUSSION

5.1 GENERAL

Performance comparison of normal conventional building (designed for both gravity and seismic) and floating column building (designed only for gravity) is carried out. And after analysis the floating column building is designed by two methods i.e. by Force Based Method (IS Code method) and by Direct Displacement Based method(DDBD).and then again the design performance of the two method is compared and conclusion is drawn.

5.2 TIME PERIOD AND MASS PARTICIPATION

The time period obtained from dynamic analysis of building is 1.47 sec and the time period of building with floating column is 1.77 sec.

The fundamental time period has also been work out by using empirical expression given in IS 1893(part 1):2016 as follows: -

$$Ta=.09h/(\sqrt{d}) \tag{5.1}$$

where, h= height of the building = 30 m

d=base dimension of the building at plinth level in meters along considered direction of earthquake force = 18 m

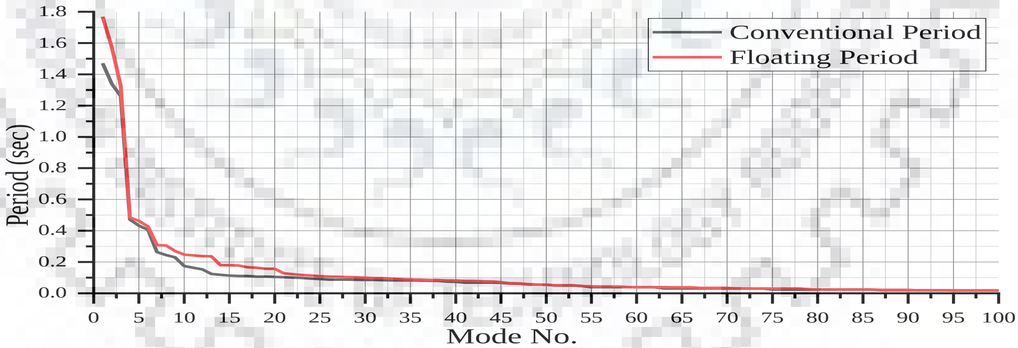


Figure 5.1 Modal period of conventional and floating column building

This gives the fundamental time period as 0.493 sec and 0.64 sec in X and Y direction respectively for conventional building while for floating column it is 0.64 sec in both direction X and Y. These values are much smaller than those obtained from dynamic analysis. Therefore, the frame represents more flexibility through dynamic analysis and floating column configuration increases the flexibility of conventional building.

5.3 DESIGN BASE SHEAR

The values of base shear obtained from dynamic analysis of building in X and Y direction through SAP2000 as well as has also been work out using empirical formula given in IS 1893(part 1):2016 and found out to be same.

$$V_b = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g} \tag{58}$$

where,

Z=zone factor taken Zone V (0.36)

I=importance factor (1)

R=response reduction factor (3)

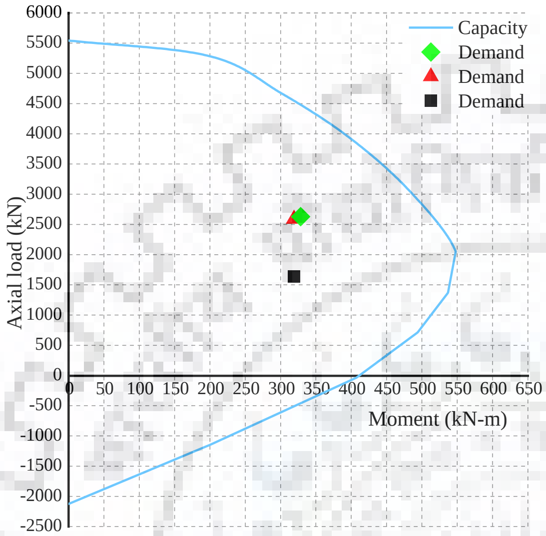
Sa/g=the spectral acceleration coefficient corresponding to time period Ta, obtained from response spectra for 5% damping (taken fundamental time period) and

W= seismic weight of the building =17874.72KN for conventional building and 17658.78 KN for building with floating column.

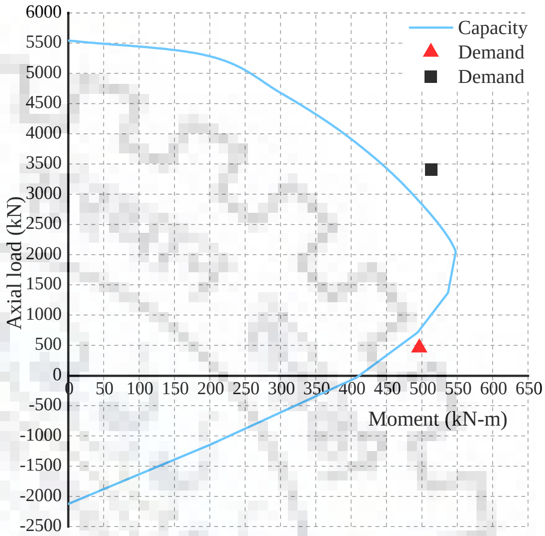
This gives design base shear for conventional building in X direction Vb= 3069.46 kN and in Y direction Vb= 2795.04 kN, for building with floating column design base shear in X direction is 2319.78 kN and in Y direction is 2609.01 kN.

5.4 P-M INTERACTION COMPARISON OF COLUMNS

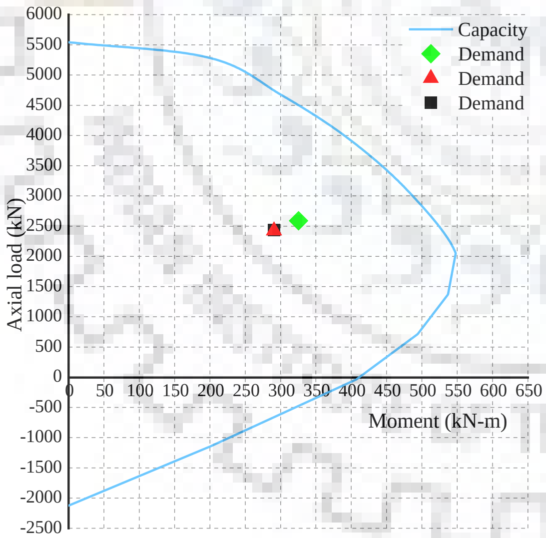
The results obtained from linear elastic and dynamic analysis of the bare frame model of the building and the building with floating column are discussed and presented in the Fig. 5.2, where diamond represents 1F columns, triangle represents 1E columns and square represents 1D columns (see Fig.4.1): -



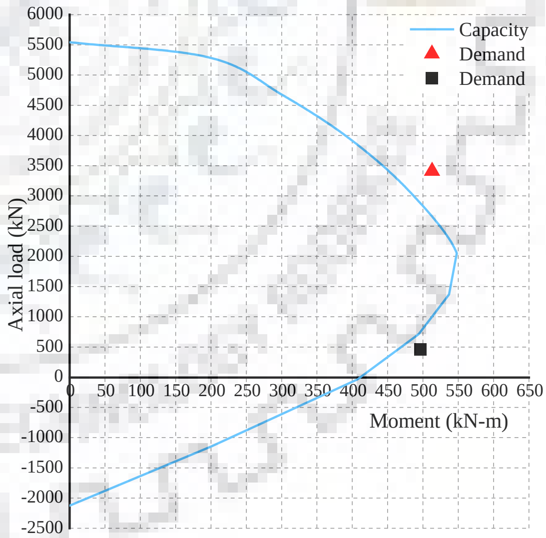
(a)



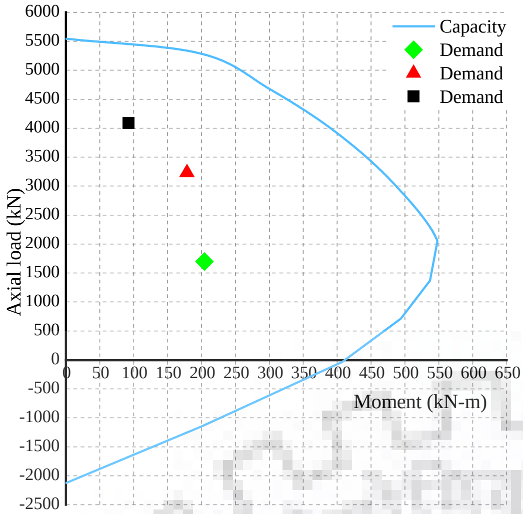
(b)



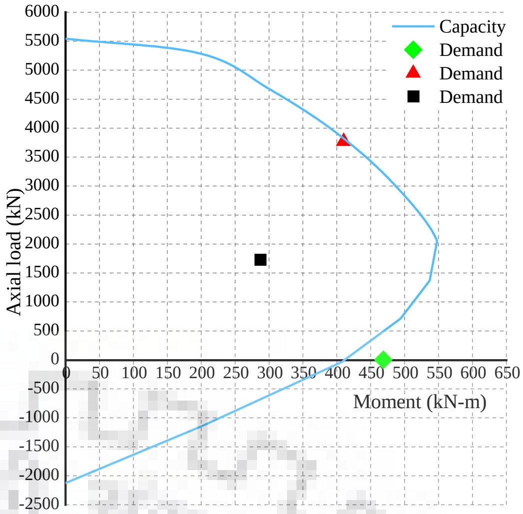
(c)



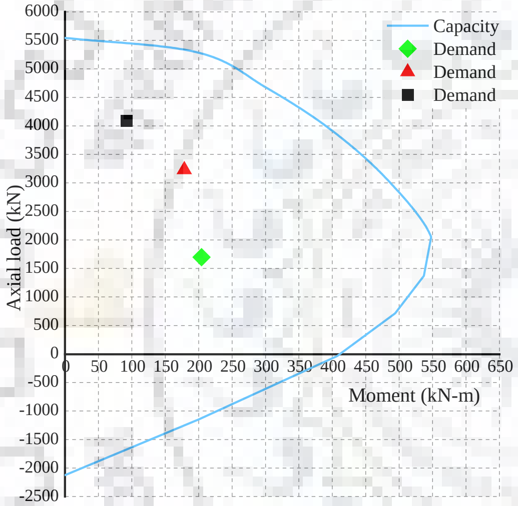
(d)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

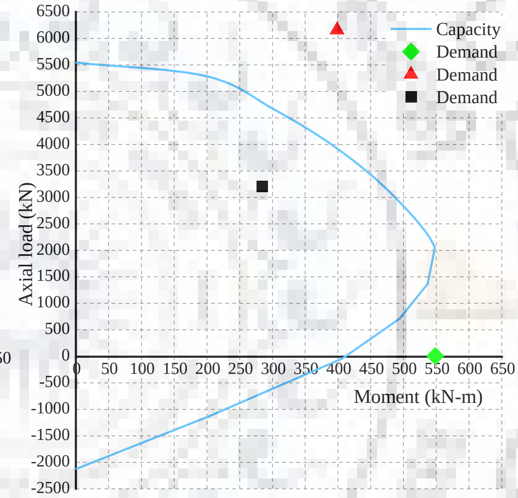
(e)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

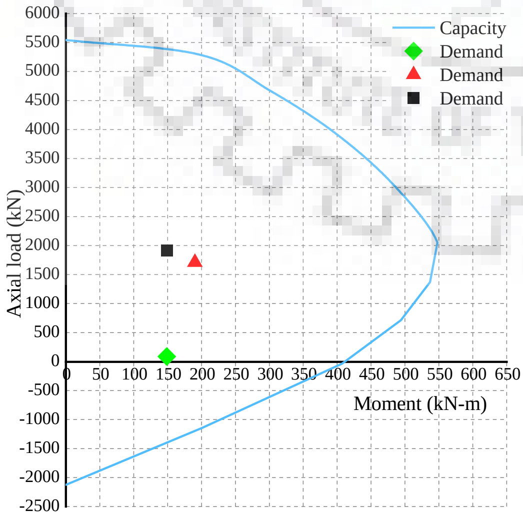
(f)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

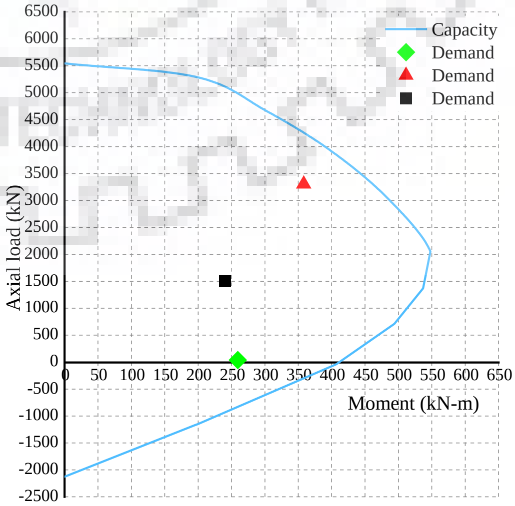
(g)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

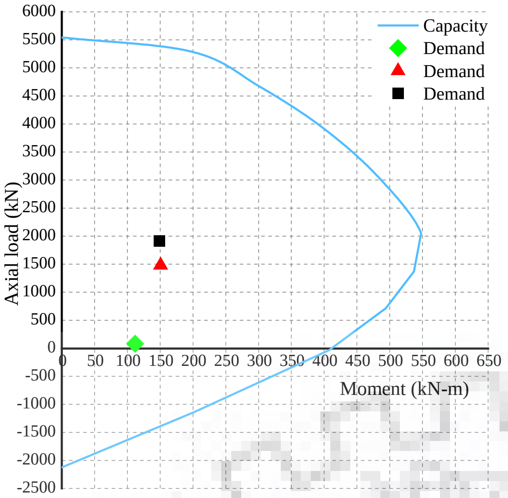
(h)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

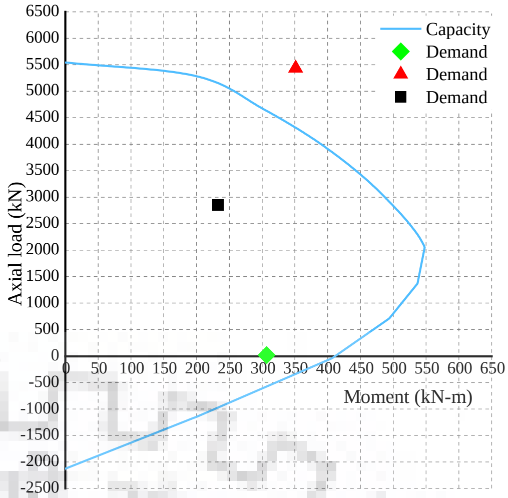
(i)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

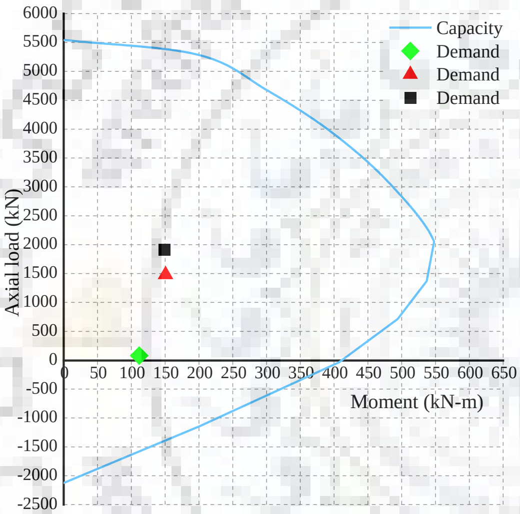
(j)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

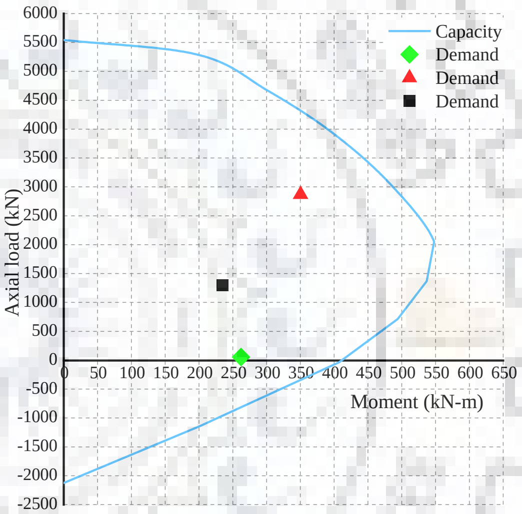
(k)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

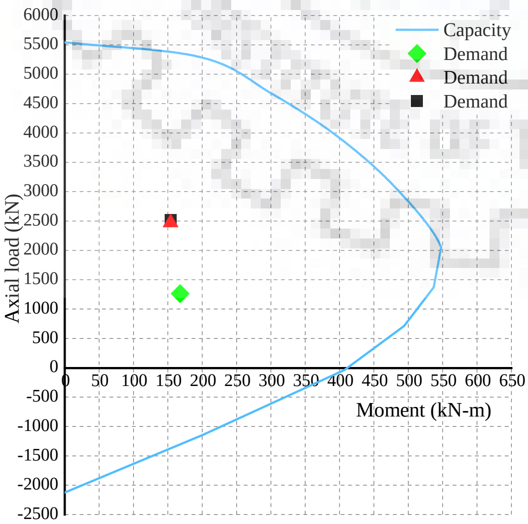
(l)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

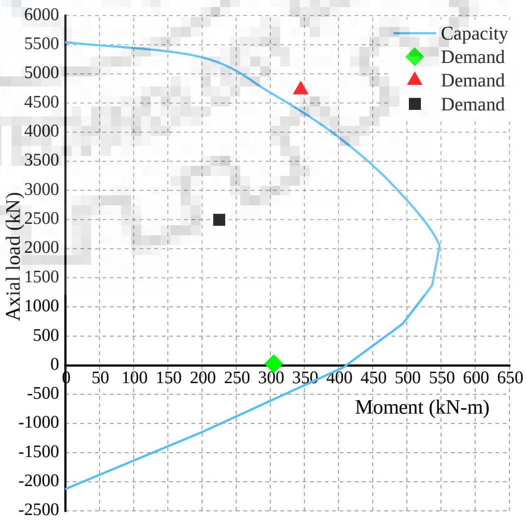
(m)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

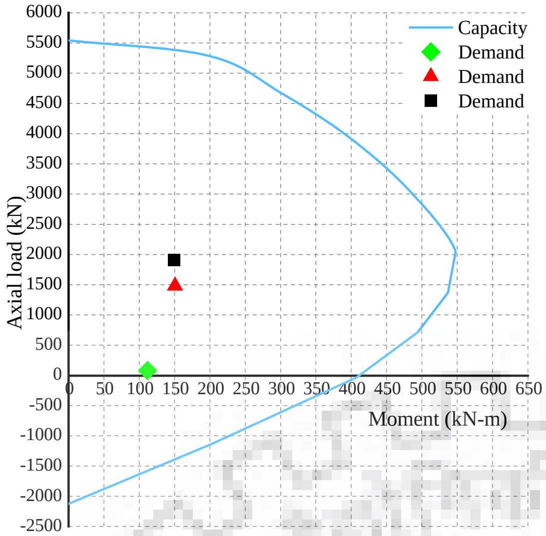
(n)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

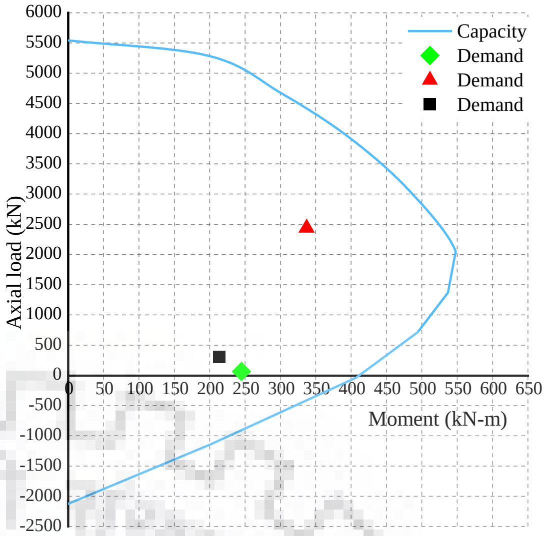
(o)


Capacity
Demand
Demand
Demand
Axial load (kN)
Moment (kN-m)

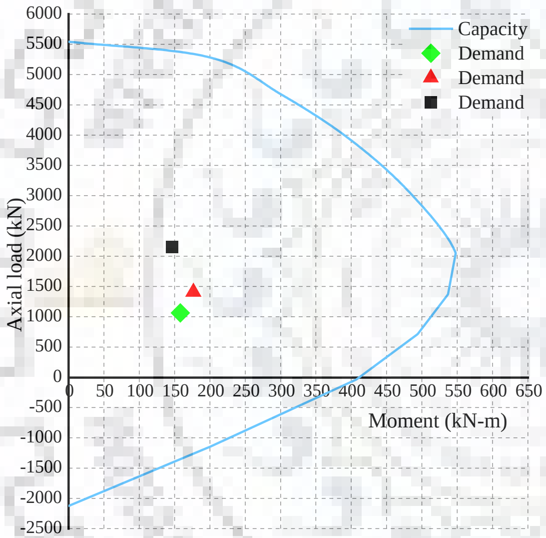
(p)



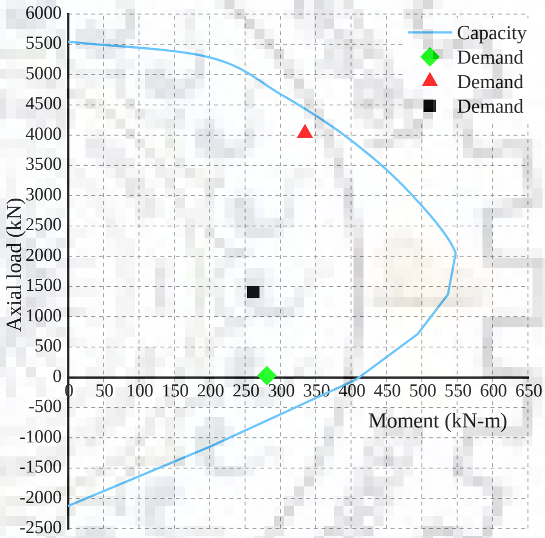
(q)



(r)



(s)



(t)

Figure 5.2 (a) Ground Storey Exterior Columns for Conventional Building

(b) Ground Storey Exterior Columns for Floating Column Building

(c) Ground Storey Interior Columns for Conventional Building

(d) Ground Storey Interior Columns for Floating Column Building

(e) First Storey Exterior Columns for Conventional Building

(f) First Storey Exterior Columns for Floating Column Building

(g) First Storey Interior Columns for Conventional Building

(h) First Storey Interior Columns for Floating Column Building

(i) Second Storey Exterior Columns for Conventional Building

(j) Second Storey Exterior Columns for Floating Column Building

(k) Second Storey Interior Columns for Conventional Building

(l) Second Storey Interior Columns for Floating Column Building

(m) Third Storey Exterior Columns for Conventional Building.

(n) Third Storey Exterior Columns for Floating Column Building.

(o) Third Storey Interior Columns for Conventional Building.

(p) Third Storey Interior Columns for Floating Column Building.

(q) Fourth Storey Exterior Columns for Conventional Building.

(r) Fourth Storey Exterior Columns for Floating Column Building

(s) Fourth Storey Interior Columns for Conventional Building.

(t) Fourth Storey Interior Columns for Floating Column Building.

Here demand represents the existing moment and axial force of column on one storey.

Now as we move up the storeys there are less variation in demand of columns as stated by saint venant's principle thus demand is coinciding while going up. As this vertical irregularity is less affecting the upper columns

5.5 MOMENT CURVATURE RELATIONSHIP OF BEAM

Following moment curvature is plotted for cantilever beam as they are the critical members in the floating columns and is showing large demand with respect to conventional building. IB is Interior beam while EB is Exterior beam and 1-10 represents floor number.(Sahai 1987)

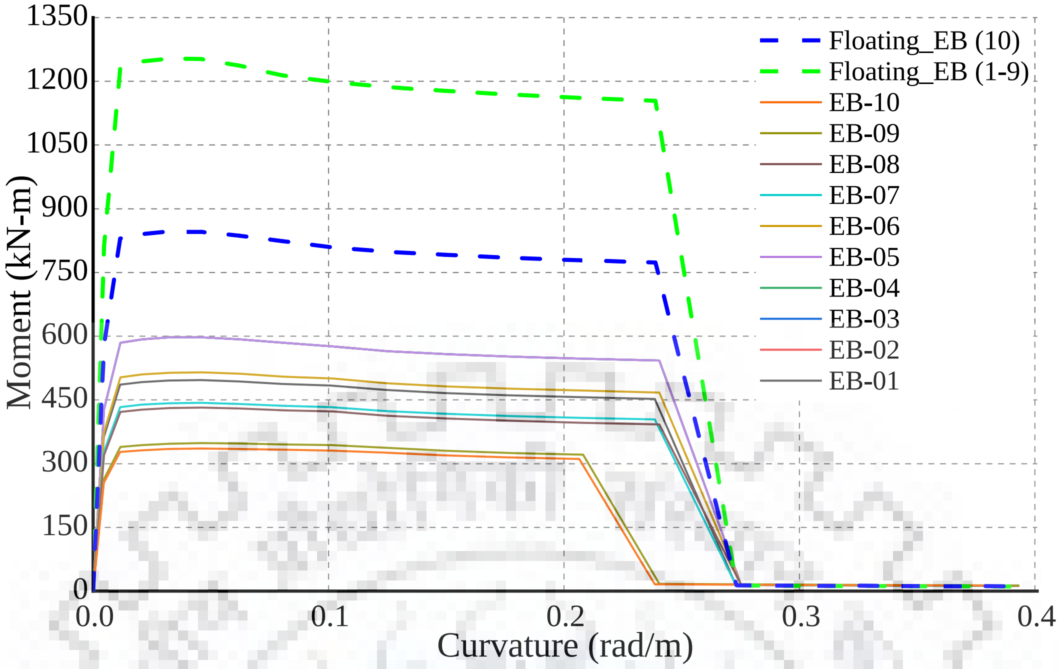


Figure 5.3 Moment curvature diagram of span 1 beams in exterior frame

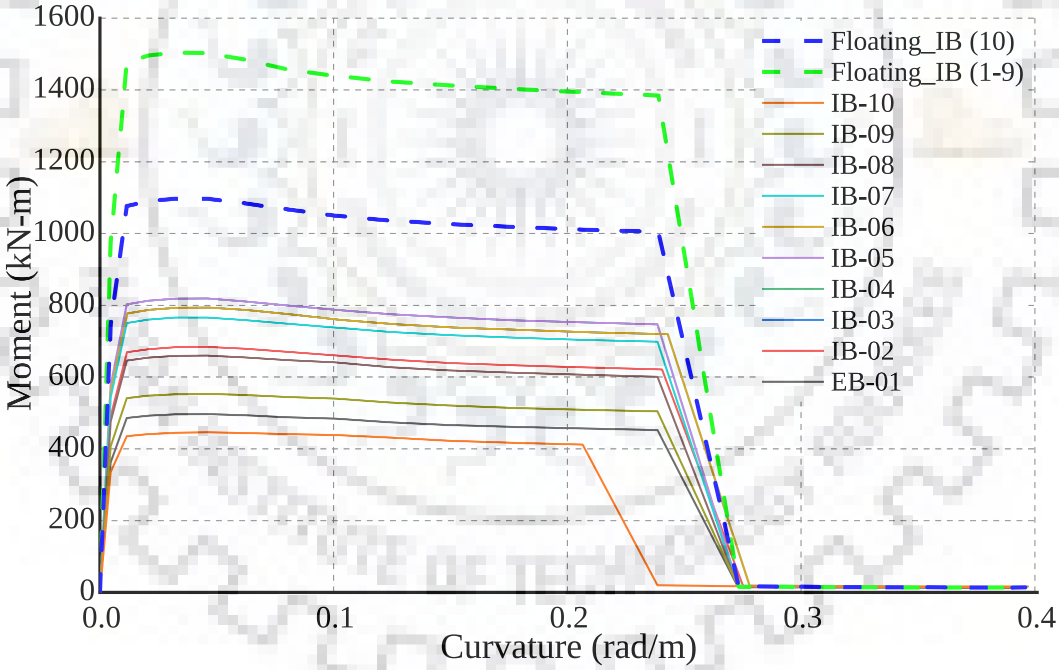
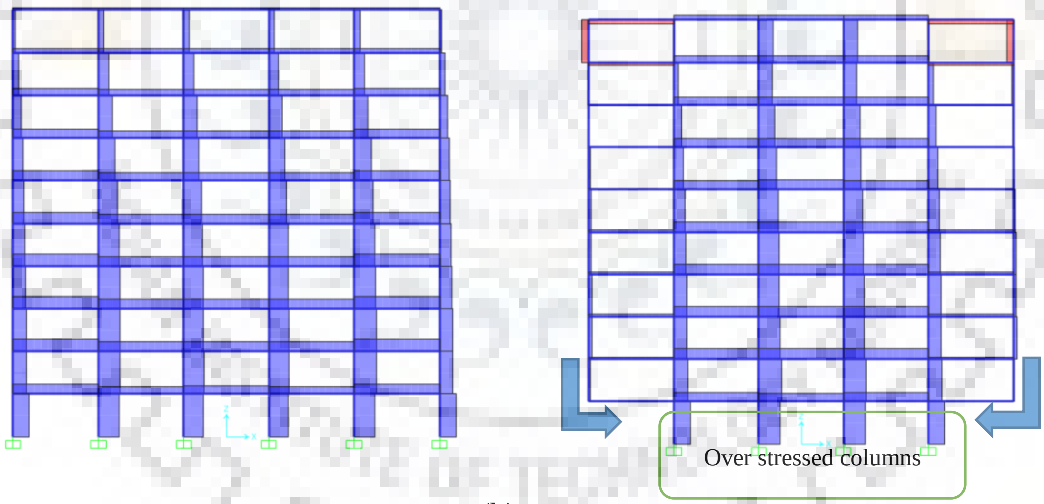


Figure 5.4 Moment curvature diagram of span 1 beams in interior frame

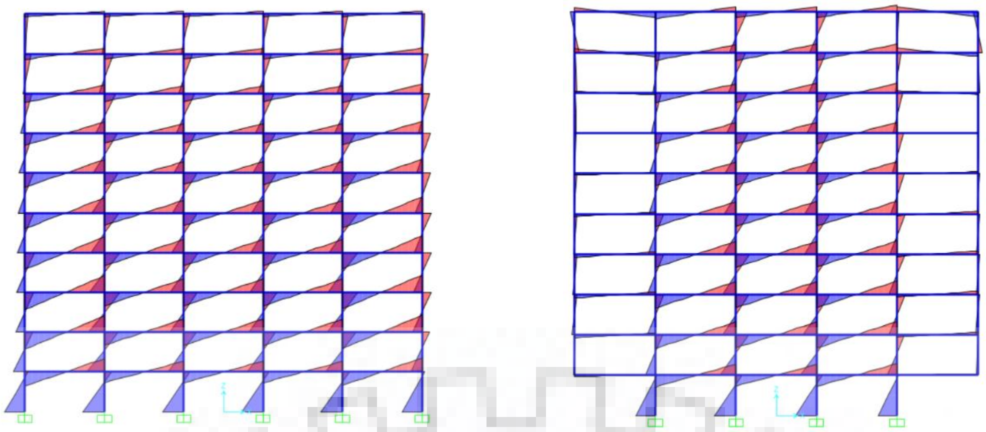
As I have taken the cross section of beam same in both conventional and floating column building and only the percentage of reinforcement is changing thus the slope of moment curvature in elastic state is not differing much (see Fog.5.4-5.5). But the moment capacity of cantilever beam in floating column is much higher with respect to conventional building as from Table 2.4 and 2.6 it can be seen that the percentage reinforcement is almost 3-4 times in floating building thus giving higher moment capacity to it. Rest of the interior beams have same size as well as almost same reinforcement (Table 2.4 and 2.6) thus M-C diagram will be same for both conventional and floating column building.

5.6 LATERAL LOAD RESISTING SYSTEM IN FLOATING VS CONVENTIONAL BUILDING

(a)



(b)



(c)

Figure 5.5 (a), (b), (c) are axial force, shear force and bending moment variation in conventional vs floating column building

As we can see from the shear force diagram (see Fig.5.5) of normal conventional building that a common form of discontinuity in load path arises in moment frames with a floating column, i.e., when a column coming from top of the building is discontinued at a lower level, normally at the ground storey. In such cases, loads from the overhanging or cantilever portions take a different load path transfer and travel to the nearest column that is continuous till the foundation. Thus, leads to increased demand on the columns in the ground storey and causes failure of these columns during strong earthquake shaking.

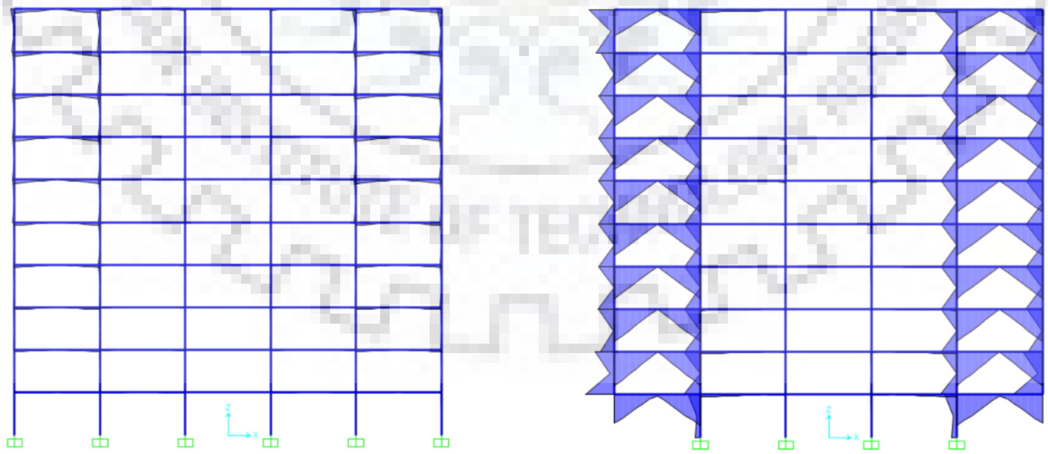


Figure 5.6 Response of conventional and floating column building in Z direction

These overhanging portion of the building do not take part in lateral resistance part of the system but these vertical irregularity takes a very predominant part in earthquake in Z direction, which we can see Fig 5.5 (Murty et al. 2012)

Reinforcement details is more in conventional building on their interior frames as due to design for both seismic and gravity while floating column building is only designed for gravity as in old state areas. But reinforcement for overhanging portion is more due to greater gravity demands in cantilever part.

5.7 STOREY DISPLACEMENT OF CONVENTIONAL BUILDING AND FLOATING COLUMN BUILDING

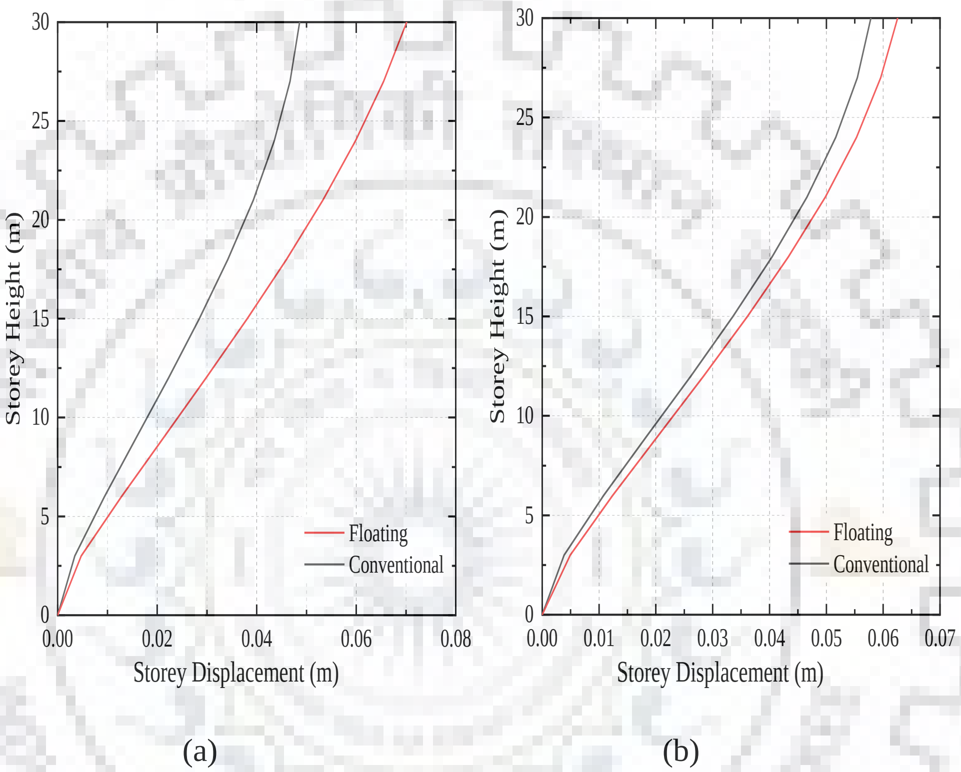


Figure 5.7 Storey displacement due earthquake in (a) X direc. (b) in Y direc.

Storey Displacement obtained from Equivalent Static method (see Fig.5.7) shows that the stiffness reduces much in X direction i.e. almost 1.5 times in almost every floor while in Y direction its almost comparable and thus giving critical response in X direction. This is due to reduction of stiffness is more in X direction than in Y direction. And this storey displacement profile represents the direction for design of building in seismic. And thus we carried DDBD design and FBD design in floating column building in X direction.(Agarwal and Shrikhande. 2007)

5.8 PUSH OVER ANALYSIS OF CONVENTIONAL AND FLOATING COLUMN BUILDING

A pushover analysis is performed through subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic non linear force displacement relationship can be determined. Pushover analysis of the existing structure has been performed. The analysis was done for both the direction of the building. The pushover analysis stated that the building fails by forming hinges in the beam first and then in the columns of the ground storey. Most of the hinges formed in the beams and columns were in the collapse prevention limit.(Chopra and Goel 2001) The peak displacement and base shear force of the existing building was found to be 0.526 m and 11932.96 kN respectively along X- direction. In Y-direction, the peak displacement and base shear force of the building was found to be 0.58 m and 10831.78 kN respectively. As well as the peak displacement and base shear force of the building with floating column was found to be 1.2m and 3151.34 kN respectively along X-direction. In Y-direction, the peak displacement and base shear force of the floating column building was found to be 0.65m and 5239.52 kN respectively.(see Table 5.8-5.9)

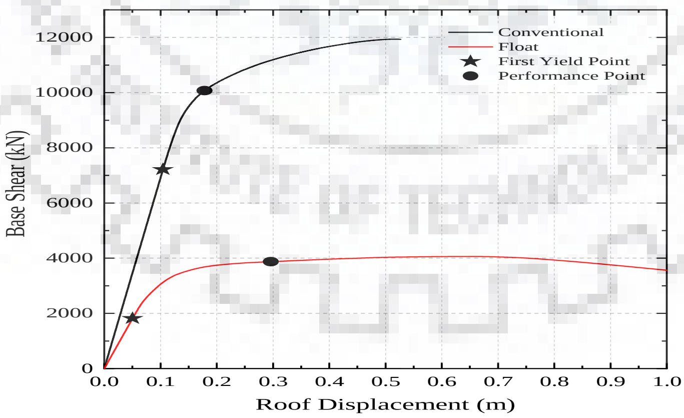


Figure 5.8 Push Over in X direction

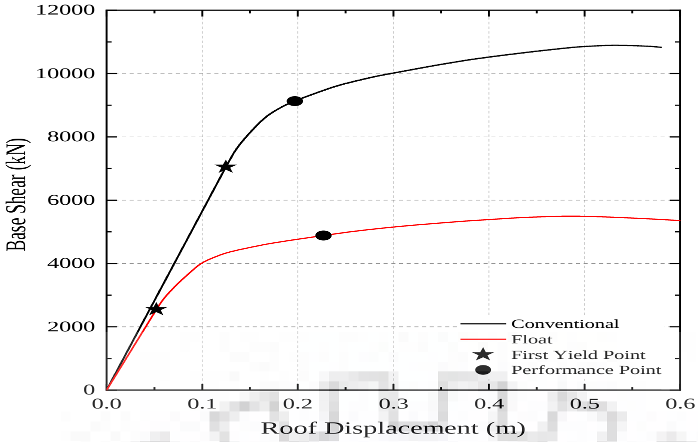


Figure 5.9 Push Over in Y direction

Starting of Hinges formation and yielding is in first floor exterior beam for conventional building while for floating column building it is forming at second floor exterior beam.(see Fig.5.10)

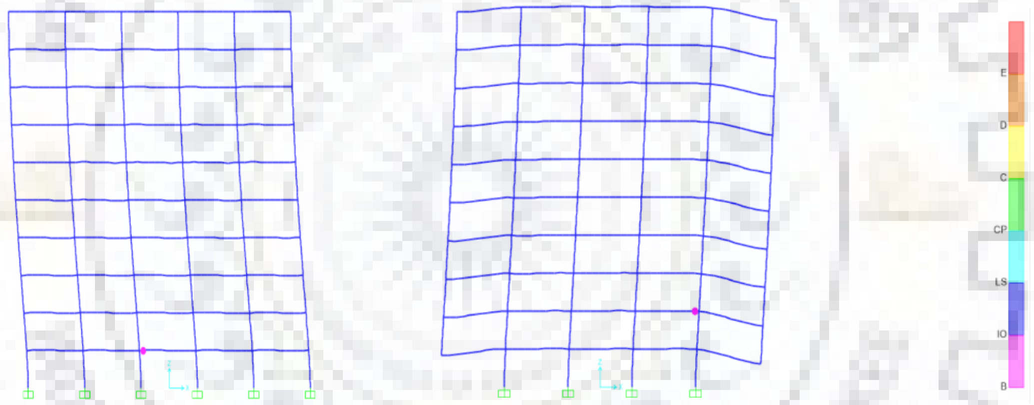


Figure 5.10 Hinge formation In X direction for floating and conventional building

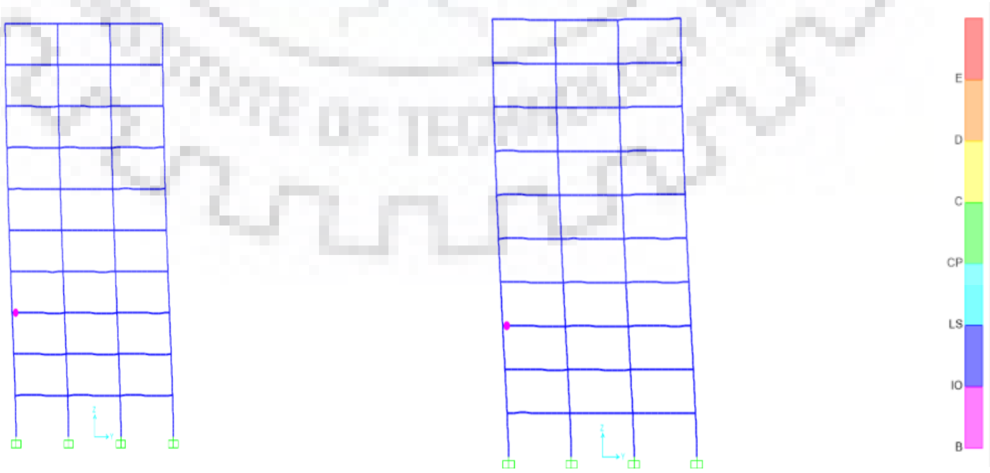


Figure 5.11 Hinge formation in Y direction for floating and conventional building

In Y direction both the building has hinge formation starting from third floor. (see Fig.5.11)

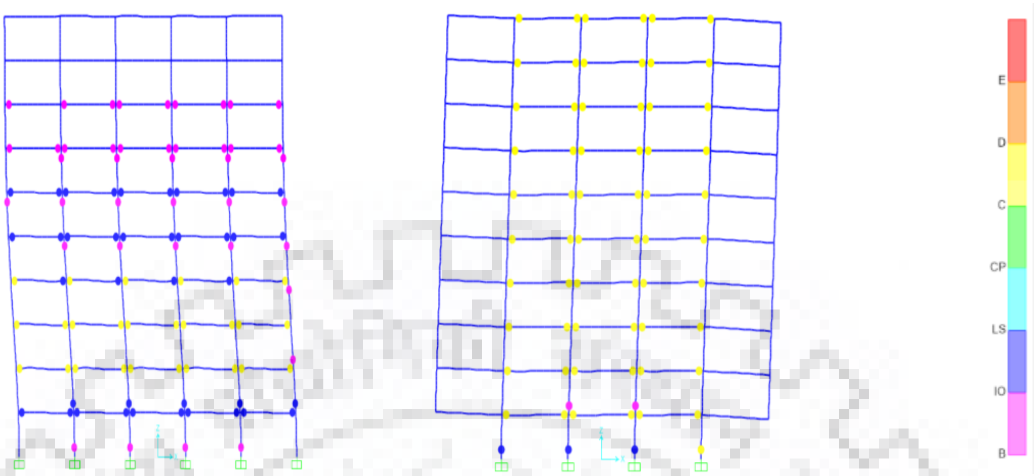


Figure 5.12 Hinge formation at last stage of Push Over conventional vs floating building in EQ X

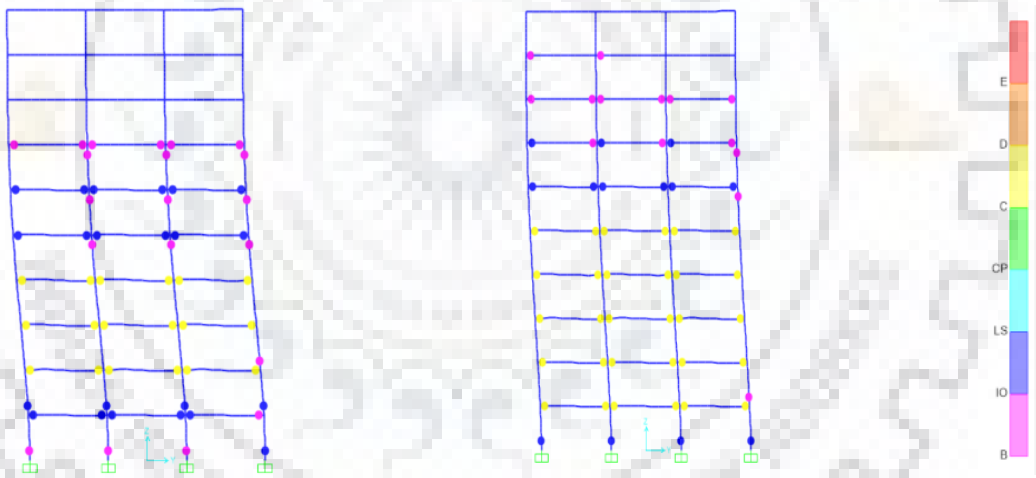


Figure 5.13 Hinge formation at last stage of Push Over conventional vs floating building in EQ Y

The hinge formation shows various column failures in conventional building while in floating building, ground storey column develops hinges first and this leads to its failure.(see Fig.5.12-5.13) The same can be observe in EQ Y direction.

Table 5.1 Conventional Building Push X

Design Base Shear (Vd)	3069.46 kN		
Base Shear at Yield (Vy)	9256.5 kN	Yield Displacement (Δy)	0.13 m
Base Shear at Failure	11933kN	Maximum Displacement (Δmax)	0.18 m
Overstrength Factor (V_y/ V_d)	3.02	Displacement Ductility Ratio (μ)	1.38

Table 5.2 Conventional building Push Y

Design Base Shear (Vd)	2795.04 kN		
Base Shear at Yield (Vy)	8316.18 kN	Yield Displacement (Δy)	0.148 m
Base Shear at Failure	10831.78 kN	Maximum Displacement (Δmax)	0.197 m
Overstrength Factor (V_y/ V_d)	2.98	Displacement Ductility Ratio (μ)	1.33

Table 5.3 Floating Building Push X

Design Base Shear (Vd)	2319.78 kN		
Base Shear at Yield (Vy)	3372.45 kN	Yield Displacement (Δy)	0.094 m
Base Shear at Failure	3151.34 kN	Maximum Displacement (Δmax)	0.296 m
Overstrength Factor (V_y/ V_d)	1.45	Displacement Ductility Ratio (μ)	3.15

Table 5.4 Floating Building Push Y

Design Base Shear (Vd)	2609 kN		
Base Shear at Yield (Vy)	3979.3 kN	Yield Displacement (Δy)	0.085 m
Base Shear at Failure	5239.52 kN	Maximum Displacement (Δmax)	0.227 m
Overstrength Factor (V_y/ V_d)	1.32	Displacement Ductility Ratio (μ)	2.67

Thus overstrength factor decreases due to development of floating column, making it less stiff while ductility increases due to early development of first yield in both directions.(see Table 5.1-5.1-5.3-5.4)

5.9 PERCENTAGE VARIATION OF REINFORCEMENT IN DDBD VS FBD DESIGN

Due to non-participation of overhanging beams and column in lateral resistance during earthquake it is only designed for gravity thus has same reinforcement as shown in Table 5.5 but elsewhere FBD has more reinforcement due to more demand as per combination 3 given in Table 2.1 i.e. D.L.+L.L.+EQX/EQY.(Muljati et al. 2015)

Table 5.5 Reinforcement variation in interior frame beam FBD vs DDBD

Storey, i	Interior frame beams (% reinforcement)									
	Span 1		Span 2		Span 3		Span 4		Span 5	
	FB	DDB	FB	DDB	FB	DDB	FB	DDB	FB	DDB
10	1.31	1.31	0.92	0.52	0.92	0.52	0.92	0.52	1.31	1.31
9	1.66	1.66	0.92	0.52	0.92	0.52	0.92	0.52	1.66	1.66
8	1.57	1.57	0.92	0.79	0.92	0.79	0.92	0.79	1.57	1.57
7	1.66	1.66	1.05	0.92	1.05	0.92	1.05	0.92	1.66	1.66
6	1.66	1.66	1.05	1.05	1.05	1.05	1.05	1.05	1.66	1.66
5	1.77	1.77	1.02	1.18	1.02	1.18	1.02	1.18	1.77	1.77
4	1.77	1.77	1.23	1.31	1.23	1.31	1.23	1.31	1.77	1.77
3	1.91	1.91	1.23	1.44	1.23	1.44	1.23	1.44	1.91	1.91
2	2.18	2.18	1.23	1.44	1.23	1.44	1.23	1.44	2.18	2.18
1	1.91	1.91	1.23	1.44	1.23	1.44	1.23	1.44	1.91	1.91

Table 5.6 Reinforcement variation in interior frame column FBD vs DDBD

Storey,i	Interior frame columns (% reinforcement)											
	Col. 1		Col. 2		Col. 3		Col. 4		Col. 5		Col. 6	
	FB	DDB	FB	DDB	FB	DDB	FB	DDB	FB	DDB	FB	DDB
10	2.18	1.00	2.18	1.6	2.18	3.2	2.18	3.2	2.18	1.6	2.18	1.00
9	1.09	1.00	1.09	3.2	1.09	5.6	1.09	5.6	1.09	3.2	1.09	1.00
8	1.09	1.00	1.09	3.2	1.09	5.8	1.09	5.8	1.09	3.2	1.09	1.00
7	1.64	1.00	1.64	4.8	1.64	4.8	1.64	4.8	1.64	4.8	1.64	1.00
6	1.64	1.00	1.64	5	1.64	5.2	1.64	5.2	1.64	5	1.64	1.00
5	1.64	1.00	1.64	5.2	1.64	5.6	1.64	5.6	1.64	5.2	1.64	1.00
4	1.64	1.00	1.64	5.6	1.64	5.8	1.64	5.8	1.64	5.6	1.64	1.00
3	1.64	1.00	1.64	5.6	1.64	5.6	1.64	5.6	1.64	5.6	1.64	1.00
2	2.73	1.00	2.73	5.6	2.73	5.6	2.73	5.6	2.73	5.6	2.73	1.00
1	0	0	3.82	4	3.82	4	3.82	4	3.82	4	0	0

Table 5.7 Reinforcement variation in exterior frame FBD vs DDBD

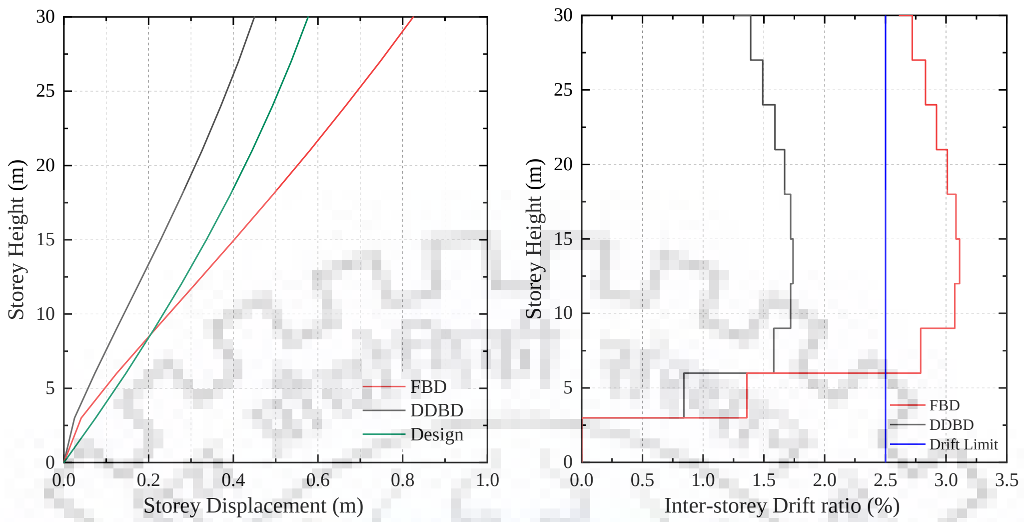
Storey, i	Exterior frame beams (% reinforcement)									
	Span 1		Span 2		Span 3		Span 4		Span 5	
	FB	DDB	FB	DDB	FB	DDB	FB	DDB	FB	DDB
10	1.05	1.05	0.65	0.39	0.65	0.39	0.65	0.39	1.05	1.05
9	1.36	1.36	0.92	0.39	0.92	0.39	0.92	0.39	1.36	1.36
8	1.26	1.26	0.92	0.39	0.92	0.39	0.92	0.39	1.26	1.26
7	1.26	1.26	0.92	0.52	0.92	0.52	0.92	0.52	1.26	1.26
6	1.36	1.36	0.92	0.65	0.92	0.65	0.92	0.65	1.36	1.36
5	1.31	1.31	1.02	0.65	1.02	0.65	1.02	0.65	1.31	1.31
4	1.47	1.47	1.02	0.79	1.02	0.79	1.02	0.79	1.47	1.47
3	1.47	1.47	1.02	0.79	1.02	0.79	1.02	0.79	1.47	1.47
2	1.47	1.47	1.02	0.79	1.02	0.79	1.02	0.79	1.47	1.47
1	1.47	1.47	1.02	0.79	1.02	0.79	1.02	0.79	1.47	1.47

Table 5.8 Reinforcement variation in column for exterior frame FBD vs DDBD

Storey, i	Exterior frame columns (% reinforcement)											
	Col. 1		Col. 2		Col. 3		Col. 4		Col. 5		Col. 6	
	FB	DDB	FB	DDB	FB	DDB	FB	DDB	FB	DDB	FB	DDB
10	2.18	1.00	1.64	1	1.05	1	1.05	1	1.64	1	1.64	1.00
9	1.10	1.00	1.10	1	1.10	1.6	1.10	1.6	1.10	1	1.1	1.00
8	1.10	1.00	1.10	2	1.10	3.2	1.10	3.2	1.10	2	1.1	1.00
7	1.64	1.00	1.64	2.4	1.64	2.4	1.64	2.4	1.64	2.4	1.64	1.00
6	1.64	1.00	1.64	2.4	1.64	3.2	1.64	3.2	1.64	2.4	1.64	1.00
5	1.64	1.00	1.64	2.8	1.64	2.8	1.64	2.8	1.64	2.8	1.64	1.00
4	1.64	1.00	1.64	3.6	1.64	2.8	1.64	2.8	1.64	3.6	1.64	1.00
3	1.64	1.00	1.64	3.6	1.64	3.2	1.64	3.2	1.64	3.6	1.64	1.00
2	1.64	1.00	1.64	2.4	1.64	2.4	1.64	2.4	1.64	2.4	1.64	1.00
1	0	0	2.73	1	2.73	1	2.73	1	2.73	1	0	0

As we can see from the reinforcement details that the reinforcement required by FBD technique is more than that by DDBD technique thus needs more steel and is less economical than DDBD.(see Table 5.6-5.7-5.8)

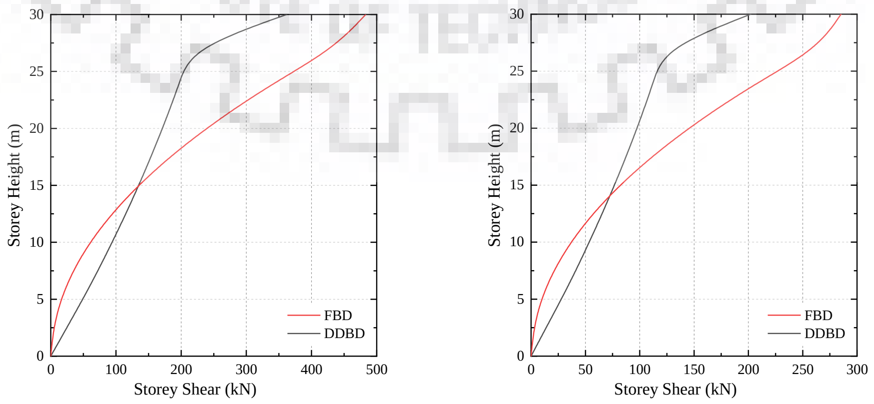
5.10 STOREY DISPLACEMENT AND IDR IN DDBD VS FBD DESIGN



(a) (b)

Figure 5.14 (a) Storey displacement by FBD and DDBD (b) IDR obtained by FBD and DDBD

The floating column structure is provided with reinforcement details as per calculated by DDBD and FBD and then analyzed in SAP2000 with nonlinear static push over curve. Thus we can see from the Fig 5.14 (a) and Fig 5.14 (b) that the vertical irregularity produces a large inter storey drift ratio in both the method but it has more effect on FBD designed structure as in that its even exceeding the damage control limit of 2.5 (Fig. 5.14 (b)). Thus not meeting up the design criteria, and also in storey displacement its even surpassing the design storey displacement set up by DDBD.(Muljati et al. 2015) Storey shears by two method is shown in Fig.5.15



(a) (b)

Figure 5.15 Lateral force variation in (a) internal frame (b) external frame

5.11 PUSH OVER ANALYSIS OF FBD VS DDBD DESIGN

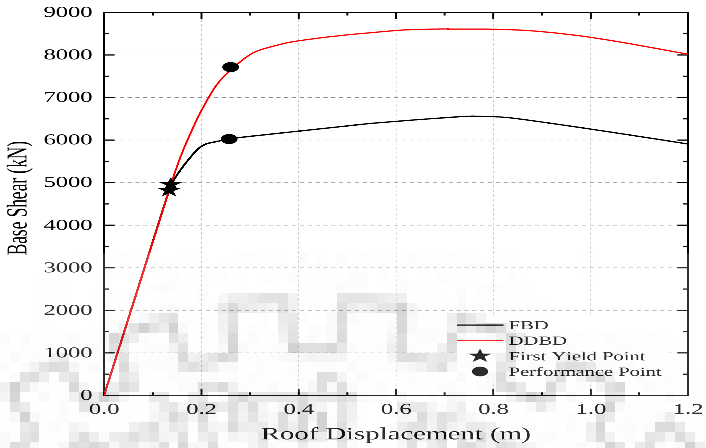
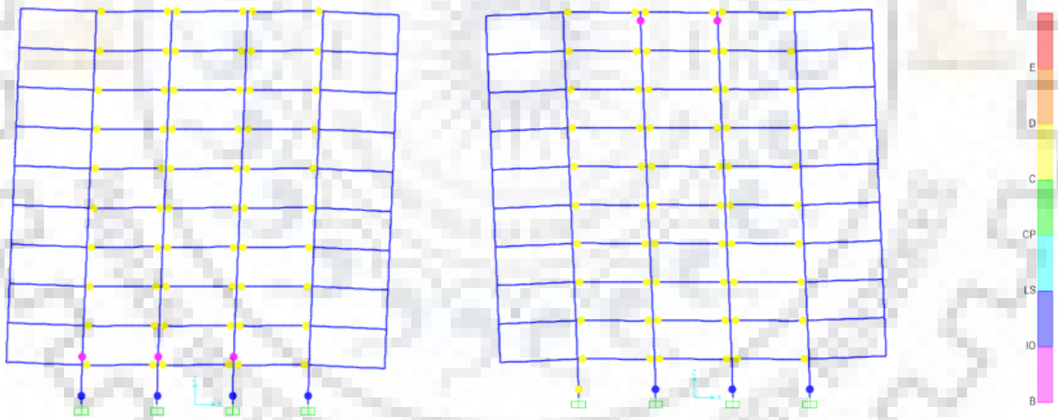


Figure 5.16 Capacity diagram of building by FBD and DDBD

From the Fig5.16it can be seen that the capacity of structure by DDBD is much more than FBD, though first yield of structure is almost same, but performance by DDBD is enhanced much more than FBD with same ductility.

Failure mechanism in DDBD vs FBD (see Fig.5.17)



(a) (b)

Figure 5.17 Hinge formation at last step of push over in (a) FBD (b) DDBD

The above figure shows the plastic hinge formation at the worst case scenario resulted by nonlinear static push over analysis. The structure designed by the two methods gives acceptable result to meet the beam side-sway-mechanism as expected which only allows plastic hinges formation at three locations, i.e. all beams end, top columns at top story, and base columns. In FBD, column hinge formation starts at first storey while in DDBD it starts from tenth storey.

5.12 ENHANCED DDBD PUSH OVER

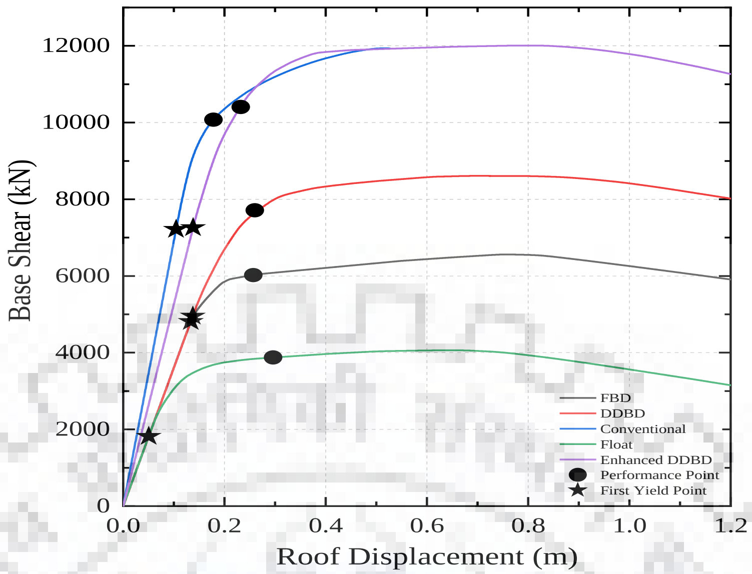


Figure 5.18 Comparative push over analysis by all methods

Even though after designing the floating column building by FBD and DDBD, the capacity of normal conventional building is still very high (see Fig.5.18), so in order to meet that capacity, change in cross section of beams and columns is needed. Thus column size has been changed from (600x600) mm to (800x800) mm on all sides and in first floor beam from (400x600) mm to (500x600) mm. The capacity curve which we can see gives comparable result, the performance point and yield point gets enhanced, with little increase in ductility.

CHAPTER 6 CONCLUSION

6.1 SUMMARY

Floating column building are highly unstable and unsafe for medium and high hazard seismic areas. The load path transfer from beam to column gets disrupted at the point of floating irregularity and in such cases, loads from the overhanging portions take a detour and travel to the nearest column that is continuous till the foundation thus gives very high stress to the column adjacent to it and cause its failure. It decreases the capacity and stiffness of the structure to a remarkable value and increases the flexibility of the structure. These floating structures gets much more damage in vertical earthquake than horizontal one. The structure should contain a continuous load path for transferring the lateral load, which develops due to acceleration of individual elements to the ground. The load path transfer must be complete and well defined. The general load path is as follows: earthquake force which originate in all the elements of the building are transmitted through structural connections to horizontal diaphragm, the diaphragm distributes the forces to vertical resisting component and vertical element transfer the forces into the foundation, and then to soil.

Thus in order to increase its performance and yield, this floating structure is designed by Force Based Design method (FBD) i.e. IS code method and by Direct Displacement Based Design method (DDBD) and its performance has been checked, compared and explained.

6.2 CONCLUSION

On the basis of present study following conclusions can be drawn

1) Floating column building is highly unstable in torsional mode, but due to the symmetric configuration of the structure in geometry as well as in lateral stiffness, the time period of the structure in 1st and 2nd mode is coming in X and Y direction respectively, while the torsional mode is in 3rd mode, thus making it torsionally stable.
2) Through the PM interaction curves it is seen that the demand on the column adjacent to the introduction of this irregularity increases suddenly, thus leading to its failure in seismic and the effect of this irregularity decreases towards the upper storey.

3) The moment capacity of overhanging beam is much more than the normal conventional beam; this is due to high curvature demand in gravity loading.
4) From the study it is found that the overhanging portion of floating column building do not take part in lateral resistance during earthquake in X and Y direction, they just move freely sideways, though in Z direction these beam shows higher stress concentration and deformation, that’s why it is not suitable to provide this vertical irregularity in structure.
5) Due stiffness reduction in X direction, the floating column building show storey displacement as twice as that of conventional building, while in Y direction it is almost equal, this is due to absence of floating column effect.
6) It is seen that the reinforcement requirement by DDBD method is less than that of FBD method in beam of exterior as well as interior frame, though the reinforcement in column increases from FBD to DDBD leading to more capacity and oversized section.
7) It is seen that DDBD designed structure perform better than FBD design structure as the storey displacement and inter storey drift ratio (IDR) came out be more in FBD designed structure. This is because the DDBD method deliberately designs the structure to achieve a given performance limit state, while FBD needs several iterations.
8) In order to meet up the performance with normal conventional building, it is needed to change the cross section of elements of structure, so as to increase the stiffness of the structure and this is achieved by increasing the column size, and this will lead to highly uneconomical structure.
9) If floating column structure is to be provided, then it should be fully analyzed and then the highly stressed elements should be provided with bracings or dampers.

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