Comparative study on Conventional Design and Capacity Based Design approaches for RC Building using Pushover Analysis

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***Abstract* – With increasing earthquake frequencies across the globe causing catastrophic failure and ample of loss to life and property, there is an utmost need of earthquake resisting structure. One of the major cause is brittle failure, to overcome the above limitation, codal provisions have incorporated the ductility aspect. This paper aims to have a comparative study of conventional and capacity based design of reinforced concrete with the help of pushover analysis-nonlinear static analysis method. The essence of capacity based design procedure is to set a strength hierarchy within each of the structural members, and then in the structure as a whole. Comparative evaluation of both the designs have been performed on the grounds of column to beam capacity ratio, hinge formation pattern, storey drift ratio, sway potential index etc. Outcome of this study could benefit academic community, industry practitioners and researchers in enhancing the design of earthquake resisting structure.**

***Index Terms* – Capacity based design, Conventional design, Ductility, Pushover analysis.**

I. INTRODUCTION

The main cause of failure of multi-storey, multi-bay reinforced concrete frames during seismic motion is the soft storey sway mechanism or column sway mechanism. The deficiency in structures is more generally a consequence of lack of ductility.
If the frame is designed on the basis of strong column weak beam concept the possibilities of collapse due to sway mechanisms can be completely eliminated. It can be achieved by allowing the plastic hinges to form in a predetermined sequence at the ends of the beams while the columns remain essentially in elastic state and by avoiding shear mode of failures in columns and beams. This procedure for design is known as Capacity Based Design which would be the future design philosophy for earthquake resistant design of multi storey, multi bay reinforced concrete frames.

Many researchers have come across important conclusions regarding the studies based on pushover analysis, such as [1] concluded the ductility requirement of all the storeys of the structure is different and to the procedure to identify the soft storey mechanism [2] found that the behavior of a multi storey framed building during strong earthquake motions depends upon the distribution of mass, stiffness and strength in both horizontal and vertical directions. Similarly, some important conclusions were also made on capacity based design which study concludes that the application of strong column weak beam concept in the design of the structure the possibilities of collapse due to sway mechanisms can be completely eliminated. [3] derived a simplified approach to calculate the moment capacity ratio for a structure.

II. NONLINEAR STATIC PUSHOVER ANALYSIS

Nonlinear static pushover analysis is performed by applying an assumed distribution of lateral loads over the height of the structure, increasing gradually from zero to the ultimate level corresponding to the collapse of the structure, where the gravity load remains constant during the analysis. The capacity of structure is represented by pushover curve.

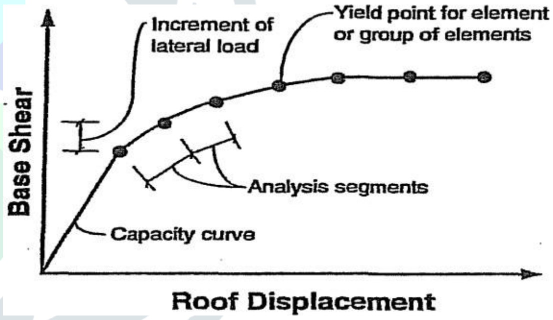

Fig. 1 Typical pushover curve [4]

A. Acceleration-Displacement Response Spectra Method (ADRS)

It is one of the methods used to determine the performance point. The ADRS Method requires that both the capacity curve and the demand curve be represented in response spectral ordinates. Conversion of the capacity curve to the capacity spectrum and demand curve to ADRS format can be done as per ATC 40.

B. Performance Point

The intersection of the pushover capacity and demand spectrum curves defines the "performance point." At the performance point, the resulting responses of the building should then be checked using certain acceptability criteria.

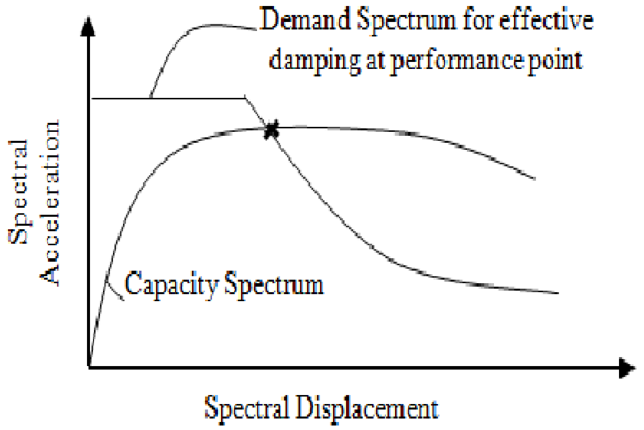

Fig. 2 Performance point [4]

III. CAPACITY BASED DESIGN

CBD method is an approach towards spreading of inelastic deformation demands throughout the structures in such a way that formation of plastic hinges takes place at predetermined positions and sequences. The essence of this procedure is to first set a strength hierarchy within each of the structural members, and then in the structure as a whole. In addition, it relies heavily on ductility at selected sections, and then, in selected members.

For RC building structures, capacity design criteria comprise of two kinds of provisions:

- Local resistance criteria

Local capacity design rules apply at the member level, where the load-transfer and strength capacities in flexure, in shear and in bond/anchorage are in series and early exhaustion of one of them precludes full development of the others

- “Global” or "system-type" resistance criteria.

“Global” type capacity design rules for multi-storey RC buildings aiming at making sure that, with the exception of their base region above the connection to the foundation, columns and structural walls will remain elastic, i.e. nearly straight in comparison to the large inelastic deformations expected to develop in the other parts of the structure, namely the beams and the base of these vertical elements.

The design criteria influencing the design action effects are the following, in detail:

1. All critical regions of the structure must exhibit resistance higher than action effects developed under earthquake loading.
2. Brittle or other undesirable failure modes must be excluded.
3. Concentration of plastic hinges in any single storey and particularly at both ends of a number of columns in the same storey should be avoided. Plastic hinges should be developed only in beams and not on columns, except for the unavoidable formation of plastic hinges at the base of the building.

The assumptions here are that columns do not form plastic hinges (due to SCWB design) and the beams do not carry axial forces.

A. *Failure Mechanism: SCWB Concept*

Strong Column Weak Beam (SCWB) concept is to ensure that damage to columns is eliminated by eliminating hinges in the column, because columns are required to transfer loads (largely the gravity loads) even after an earthquake.

It must be recognized that even with a SCWB concept, column plastic hinges must form at the base of the column.

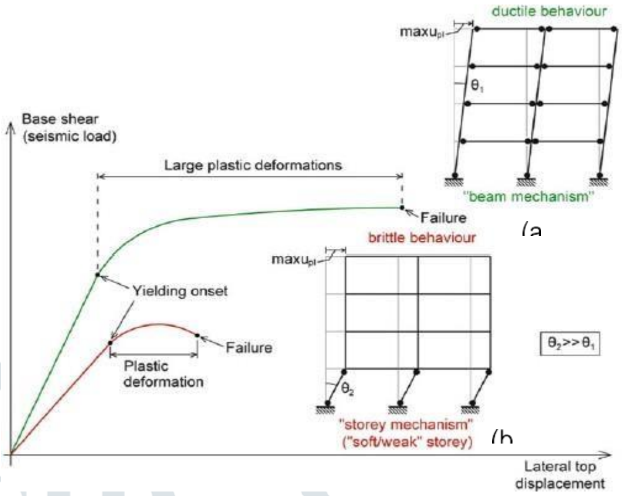

Fig. 3 Moment frames showing storey mechanism and beam mechanism

B. *Capacity Design For RC Frames*

The moment capacities of the columns are checked for the sum of the moment capacities of beams at joint with an over-strength factor of 1.4 as stated in IS 13920:2016, clause 7.2.1, clause 7.2.1.2 and clause 7.21.3 [5].

C. *Capacity Design For Beam*

The shear force of the beam is calculated as per the formulas given in the IS 13920:2016 code, clause 6.3.3.[5]

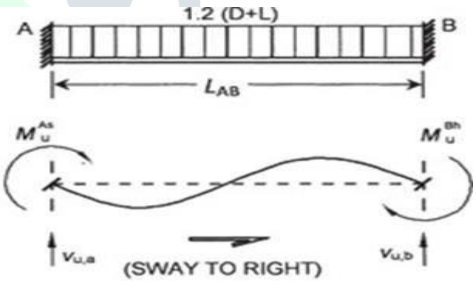

Fig. 4 Sway mechanism of beam

D. *Capacity Design For Column*

For the design of columns, formulas given in the IS 13920:2016 code, clause 7.5 is used [5].

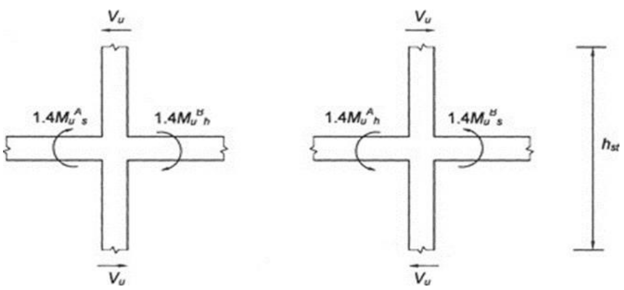

Fig. 5 Column sway mechanism

IV. PROBLEM FORMULATION

RC building models of 8 storeys and 26m height modelled with seismic force resisting structural systems have been designed in detail and analyzed. First model consists of RC frames which are designed by capacity-based methodology (**MRFCBD**). Second model consists of RC frames which are designed by conventional Indian methodology practiced by the structural engineers (**MRFCD**).

Table 1 Geometric & Material Data

Building type and location	Residential building (G+8) in Surat, Gujarat
Ground Storey	Open parking
Plan dimensions	30m x 25m
Building height	26m
Typical storey height	3m
Ground storey height	5m
Grade of concrete & steel	M25 & Fe 500

Table 2 Loading Data

1) Dead load (DL)	
Thickness of slab assumed	120 mm
Self-weight of the slab	3 kN/m^2
floor finish load assumed	1 kN/m^2
Roof finish load assumed	2 kN/m^2
Approximate Partition wall load	1.5 kN/m^2
Parapet wall load (façade load)	3 kN/m
Outer wall load (façade load)	10 kN/m
2) Live load (LL)	
live load on all floors except terrace	3 kN/m^2
live load on terrace	1 kN/m^2
3) Seismic load	
Earthquake zone and zone factor	Zone III & Factor 0.16
Importance factor	1
Response reduction factor	5.0 (SMRF)
time period calculated	0.8636 sec

The plan area of the building is 30m x 25 m with 6m bay spacing in X direction and 5m bay spacing in Y direction. The buildings are analyzed by code based linear dynamic procedure i.e. Response spectra analysis method given in IS 1893:2016 and then designed according to IS 456: 2000 [6] and IS 13920: 2016 [5]. Performance assessment of buildings designed by both the approaches is carried out using nonlinear static analysis for design basis earthquake level and different parameters are quantified for comparison. All the buildings have been modelled in ETABs 2016 version 16.0.2 software.

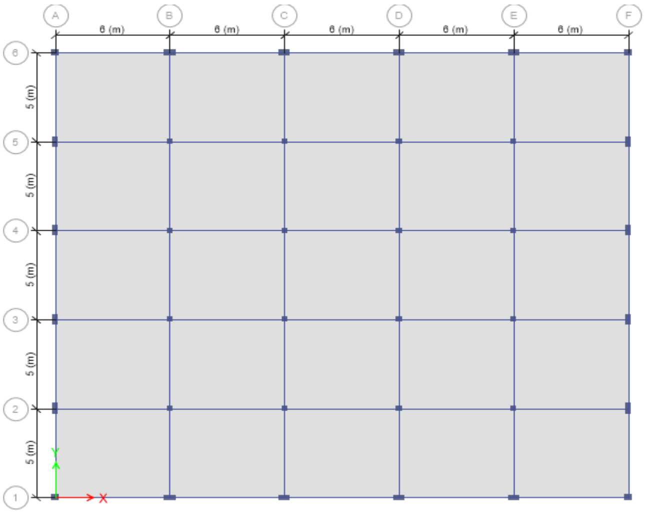

Fig. 6 Plan of the RC frame building

A. *Modelling Assumptions*

The beam and column elements are modelled as line elements and the slabs are modelled as membrane elements and a rigid diaphragm action is considered at each storey.

B. *Lateral Load Resisting System: RC Frames*

In MRFCBD building designed by capacity-based approach, the perimeter frames A, F and 1, 6 are chosen to resist entire seismic forces acting on the structure in X and Y direction respectively. Rest of the frames are designed for gravity loading only as per the clause 1.1.3 of IS 13920:2016 [5]. The beam column joints are then checked for column by beam capacity ratio (CBC) and the columns are checked for capacity by demand ratio (CDR). Also, the inter-storey drift ratios were within permissible limits as given by IS 1893:2016. The sections are finalized after satisfying these checks. Now the interior frames are modelled separately and are designed to resist gravity loads and seismic displacements only as per clause 3.6, IS 13920: 2016 code [5].

V. ANALYSIS AND DESIGN OF RC BUILDING

A. Design Criteria

After analyzing the building models, they are designed for all the combinations given in IS 456:2000 [6]. For the MRFCBD building, the concept of capacity design is implemented, where beams are designed similar to normal procedure for the calculated forces. The design forces of columns are not completely based on linear elastic analysis, but they depend upon the flexural capacities of the beams framing into the same joint such that plastic hinges may not form at the base of the column above and at the top of the column below joint.

Table 3 Comparision of column dimension CBD and CD

Storey no.	**C1**		**C2**		**C3**	
	CBD	**CD**	**CBD**	**CD**	**CBD**	**CD**
1	525X525	450X450	300X900	300X750	450X450	450X450
2	525X525	450X450	300X825	300X600	450X450	450X450
3	525X525	375X375	300X825	300X525	450X450	375X375
4	525X525	375X375	300X750	300X525	450X450	375X375
5	525X525	375X375	300X750	300X450	375X375	375X375

Storey no.	C1		C2		C3	
	CBD	CD	CBD	CD	CBD	CD
6	450X450	300X300	300X675	300X450	375X375	300X300
7	450X450	300X300	300X675	300X450	300X300	300X300
8	450X450	300X300	300X675	300X450	300X300	300X300

A. Column By Beam Flexural Capacity Ratio Results

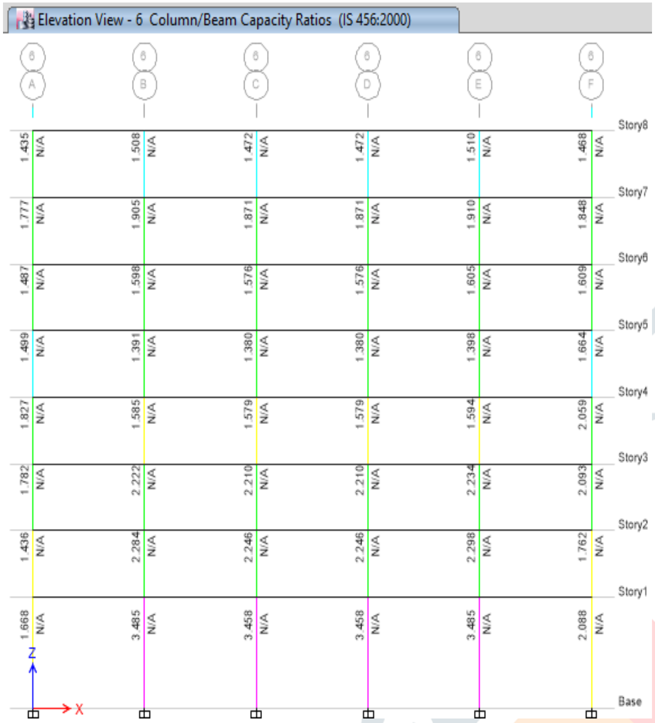

Fig. 7 Column by beam capacity ratio in X direction for CBD

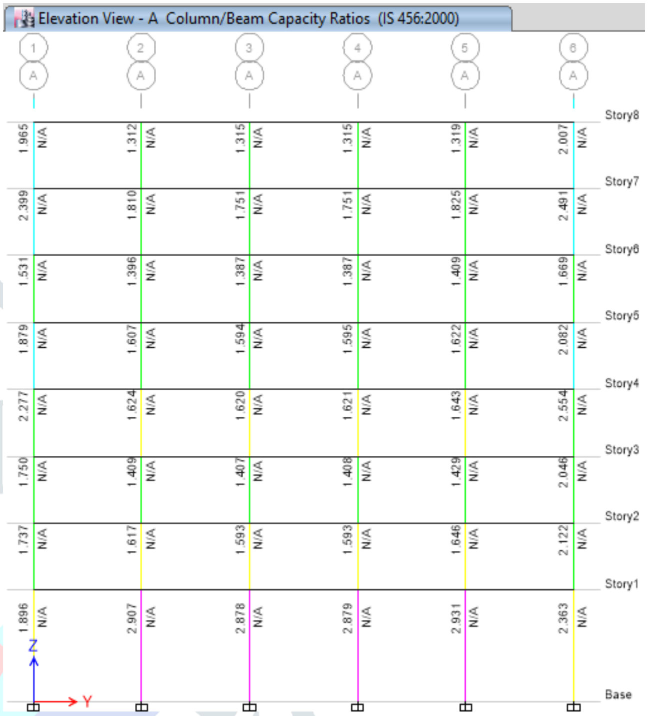

Fig. 9 Column by beam capacity ratio in Y direction for CBD

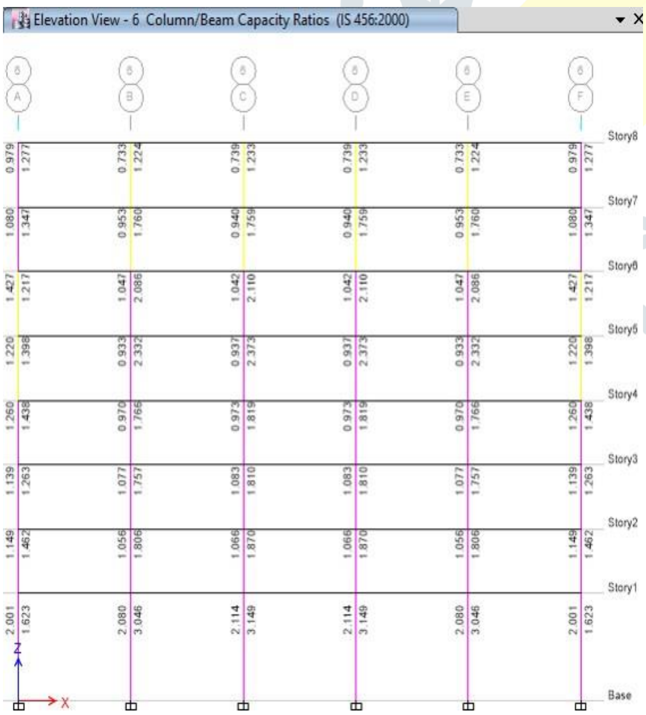

Fig. 8 Column by beam capacity ratio in X direction for CD

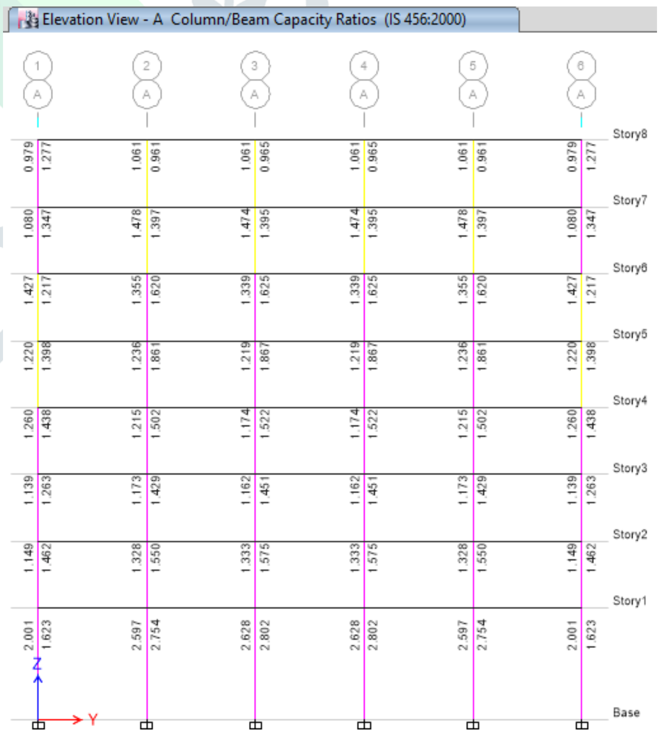

Fig. 10 Column by beam capacity ratio in Y direction for CD

VI. RESULTS AND COMPARISON

For detailed performance assessment it is necessary to assess the building during its nonlinear behavior and for which linear or elastic analyses techniques may not be used. Thus, it is necessary to perform pushover analysis for assessing its nonlinear behavior. There are many parameters to evaluate the performance of the building against lateral earthquake forces which are explained in brief.

A. Hinge Formation

For obtaining the hinge pattern formation in the MRFCBD & MRFCD frames, hinges have been allocated in the beams and columns at both the ends. Software analysis on MRFCBD frame revealed the formation of hinges first in the beams (due to flexure yielding of beam) and later at the column base. With increasing lateral load the hinges formed shifted from B to IO to LS to CP to C, thus moving to higher stage of hinge property. However, in MRFCD frame the hinges formation was sporadic due to improper strength hierarchy.

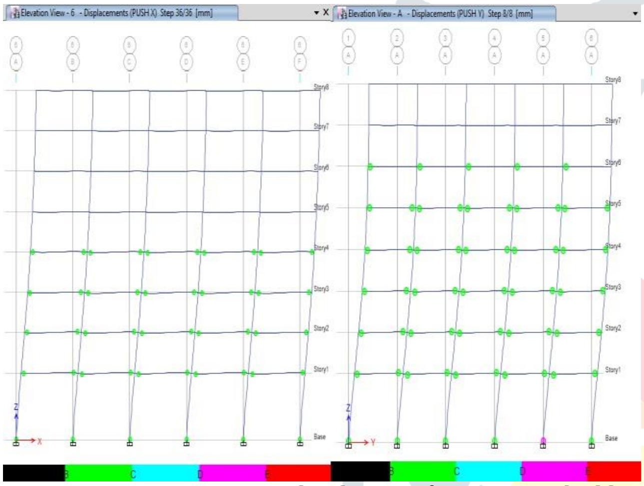

Fig. 11 Hinge pattern in X and Y direction for MRFCBD building

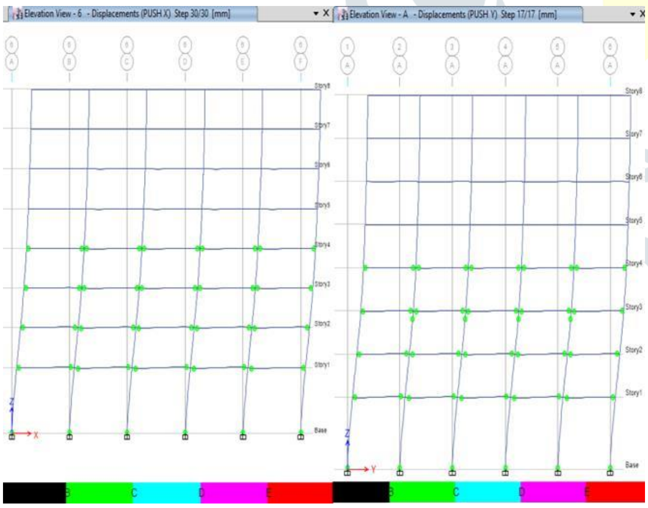

Fig. 12 Hinge pattern in X and Y direction for MRFCD building

B. Pushover Curve

Pushover analysis is performed in two independent orthogonal directions X and Y and the initial target displacement of the building was taken to be 0.4% of the total height which has been increased to 2.5% of the total height of the structure. The resulting base shear v/s displacement for X and Y direction are show in fig. It is observed that they are linear up to some extent and then start to deviate from original slope and become more flat (due to beams and columns undergo inelastic actions). Thus, building designed by capacity method behaves more in elastic range compared to conventional one for both X and Y direction.

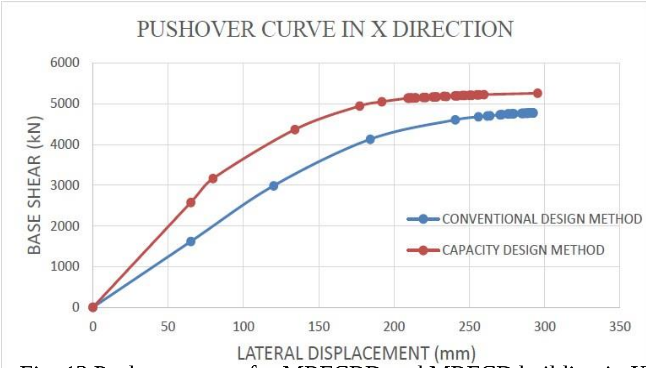

Fig. 13 Pushover curve for MRFCBD and MRFCD building in X direction

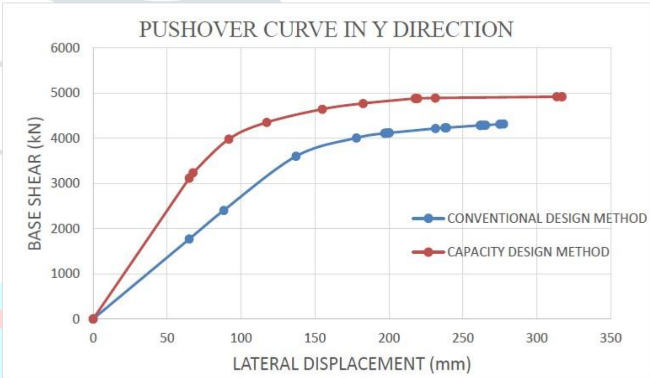

Fig. 14 Pushover curve for MRFCBD and MRFCD building in Y direction

C. Performance Point

The capacity curve obtained from the above step can be converted to Capacity Spectrum Curve in the form of Acceleration Displacement Response Spectrum format (ADRS) with reference to ATC 40. Demand spectrum curve can be plotted considering the soil condition and earthquake zone which is III in this case. Performance point can be obtained from the intersection of the Response Spectrum Curve and the Demand curve. At performance point, the base shear obtained is the design shear for the given demand spectra and the roof displacement is the actual displacement of the building for that particular hazard level.

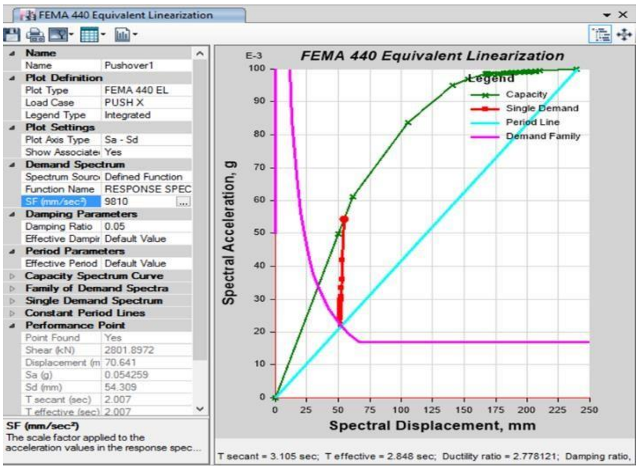

Fig. 15 Performance point for PUSH X load case for MRFCBD

Table 4 Performance point of MRFCBD and MRFCD building in X and Y direction

Performance Point		Capacity Method (MRFCBD)	Conventional Method (MRFCD)	% increase or decrease w.r.t MRFCD
Push X	Base shear (kN)	2801.897	2207.209	26.94
	Roof displacement (mm)	70.641	88.660	-20.32
Push Y	Base shear (kN)	3098.371	2331.073	32.92
	Roof displacement (mm)	64.567	85.581	-24.55

D. Inter Storey Drift Ratio

Inter-storey drift ratio measures the relative displacement of a storey with immediate storey above or below it. It is observed that inter-storey drift is maximum at storey 4 (0.00265) for MRFCBD and is maximum at storey 6 (0.00441) in MRFCD building. The maximum value attained is 0.00265 which is well within the range of 0.004 recommended by IS 1893:2016 [7]. The maximum storey drift value of MRFCD building is 1.66 times more than that of MRFCBD building. Further in building with CBD smooth curve is obtained and for each storey their values are lower than that of conventional design which may be due to the larger cracked section stiffness and less pronounced inelasticity of the more heavily reinforced concrete columns. Thus, chances of formation of column sway mechanism is higher in MRFCD building.

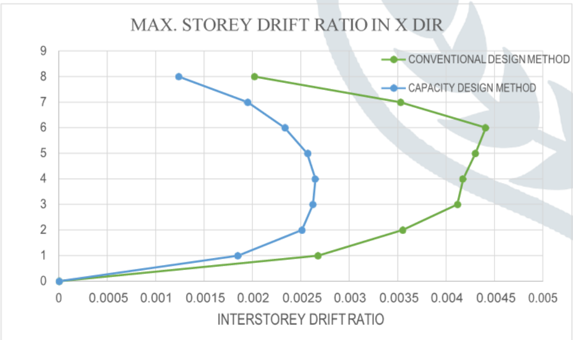

Fig. 16 Inter-storey drift ratio comparison of MRFCBD and MRFCD building in X direction

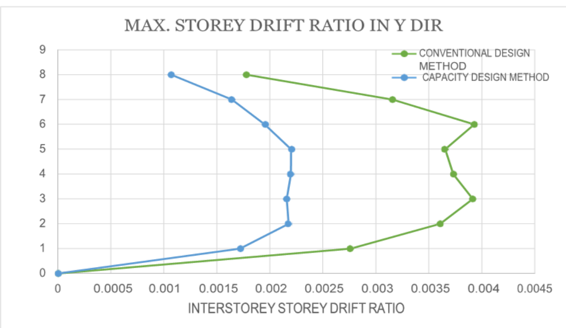

Fig. 17 Inter-storey drift ratio comparison of MRFCBD and MRFCD building in Y direction

E. Global Ductility Ratio

Global ductility ratio defines the overall ductility of the structure i.e. the non-linear behavior ranges of the structure. It is the ratio of the ultimate displacement upon the yield displacement of the building. Hence, building designed by capacity method can dissipate more energy as compared to conventional one. Higher the ductility higher is the ability of the structure to deform without getting collapsed.

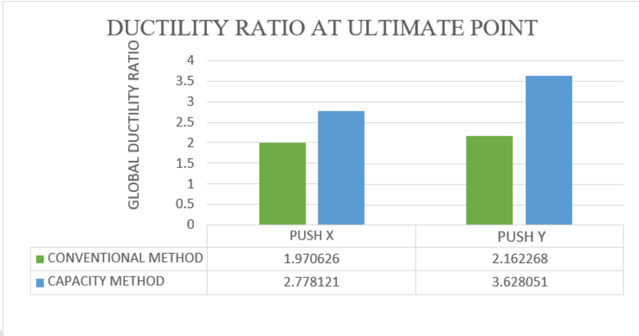

Fig. 18 Global ductility ratio at ultimate point for Push X and Push Y load case

F. Damping Ratio

It is observed that the damping ratio obtained for all the cases is much more than 0.05 as recommended by IS 1893: 2016 [7]. It is observed that MRFCBD capacity design building has 66.17% and 90.82% higher damping value than the MRFCD conventional design building which indicates that capacity design method will be effective in reducing the seismic force effect on the structure to a huge extent by dampening the effects of seismic forces induced in the structure.

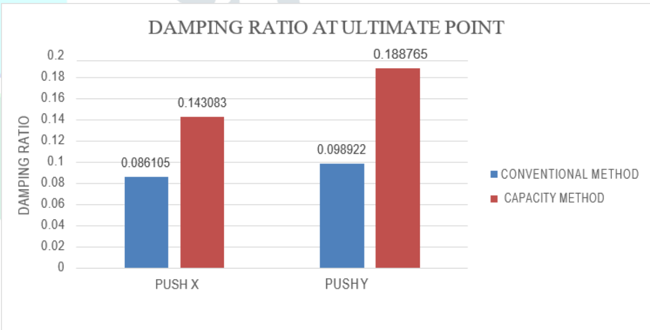

Fig. 19 Damping ratio for PUSH X and PUSH Y load case

G. Effective Time Period

Effective time period shows the flexibility of the building during ground motions. From fig., it can be said that the conventionally designed building has 6.5% lower time period at ultimate point in Y direction as compared to capacity design one. But both the buildings have almost same time period at ultimate point in X direction. Thus, MRFCBD capacity design building is more flexible in Y direction as compared to the MRFCD conventional design building.

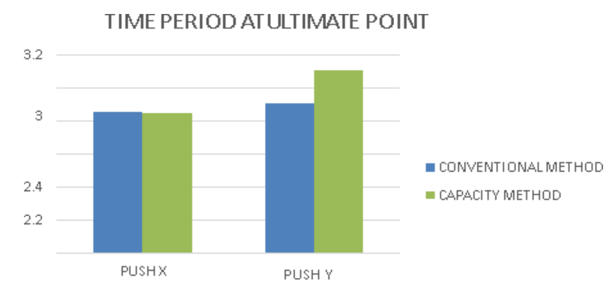

Fig. 20 Effective time period at ultimate point for PUSH X and PUSH Y load case

H. Response Reduction Factor

Fig. 21 shows that the value of R factor is 46.29% and 81.89% higher for MRFCBD building as compared to the MRFCD building in X & Y direction respectively. This shows that the seismic demand i.e. base shear value is less for CD building than the CBD. Hence it has to be designed for lower seismic forces.

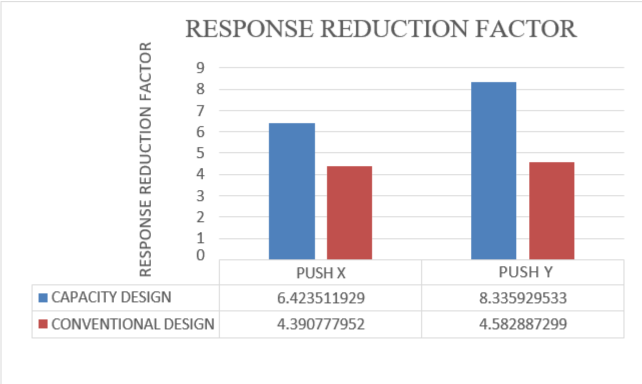

Fig. 21 Response reduction factor comparison of MRFCBD and MRFCD building

I. Cost Comparison

From the dimensions obtained from the CD and CBD, the volume of concrete and weight of steel was evaluated using spreadsheet. Considering only the material cost, M25 grade concrete costed Rs.4750/m^3 and steel of grade Fe 500 at Rs.45/kg. It was observed from the cost calculation that, the % increase in cost for CBD was 16.6% higher w.r.t concrete and 18.9% higher with respect to steel.

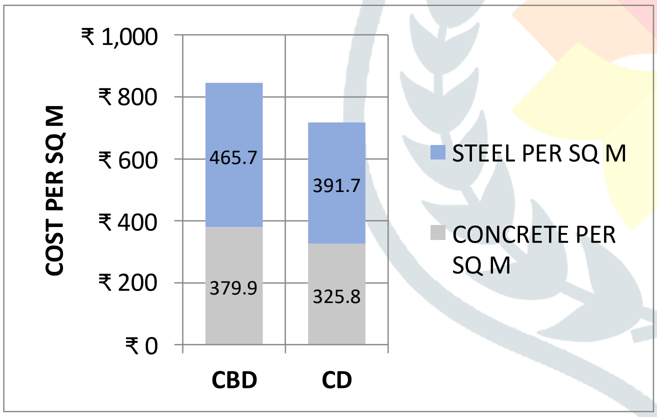

Fig. 22 Cost comparison of CBD and CD frame

In our study, we have assumed that in CBD the lateral forces are to be resisted by peripheral frame only, so during earthquake only peripheral members will get damaged. Hence, at the time of retrofitting, only peripheral members are to be retrofitted with high level of certainty to ensure the structural safety of building, in contrast to conventional design where every/most of the structural member may require retrofitting. Hence, it can be inferred that though the CBD has higher initial cost than CD, it might offset the future maintenance/retrofitting cost.

VII. CONCLUSION

The main objective of the study was to explore the demerits of conventional method of designing RC structures and brought out the distinct features of Capacity Design Methodology. Based on analysis, design and performance of building by CD and CBD, the following remarks can be made:

- In CD approach, beams and columns are designed to satisfy design gravity and lateral force demands irrespective of relevance with each other. While in case of CBD approach, specifically columns are designed in relevance with the beam capacities meeting at a joint with the column. Further beams and columns are detailed appropriately so as to form plastic hinges at predetermined locations.
- The column by beam flexural capacity ratio is more than 1.4 in seismic resisting frames of CBD building whereas the ratio varies from 0.7 to 1.6 in the model designed by CD.
- The ultimate base shear capacity and the roof displacement values obtained for CBD building are more than that of CD building.
- Inter-storey drift observed for the CBD building designed by is well within the prescribed limit given by IS code but the drift exceeds this limit for few stories in case of CD building. Also, the drift ratio obtained from CBD approach is lesser than CD approach at every storey level.
- CBD approach provides higher ductility as compare to CD. Higher the ductility higher is the ability of the structure to deform without getting collapsed.
- Also, the CBD building has 66.17% and 90.82% higher damping value than the CD building which indicates that CBD method will be effective in reducing the seismic force effect on the structure to a huge extent by dampening the effects of seismic forces induced in the structure.
- It is observed that CBD model dissipates more energy in non-linear range i.e. 37.5% in X direction and 35.6% in Y direction whereas CD building dissipates more energy in linear range (85.2%) in X direction and 72% in Y direction.

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