COMPARISION OF DIRECT DISPLACEMENT SEISMIC DESIGN WITH FORCE BASED DESIGN USING INDIAN STANDARDS – A CASE STUDY

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Abstract

Development of Force-based seismic design is historical, and every earthquake has thrown new challenges for already existing assumptions and principles, which helped in the evolution of force-based design. Although many of the assumptions were answered but some remained unaltered and unanswered. Hence, this paved way for developing new theories and philosophies and one such theory is Direct Displacement-Based seismic Design (DDBD). Indian Standard codes, IS 1893 and IS 13920, underwent a major revision in the year 2016, considering the latest developments in Force based design principles.

Current work is an attempt to understand the performance comparison between latest IS 1893 & IS 13920 with DDBD method. Initially, three buildings i.e., 3, 6 and 9 storey buildings were designed using DDBD at inter-storey drift of 4% with a performance objective to achieve "No collapse". The spectrum developed using IS code equations does not represent ideal displacement spectrum shape and hence PGA, T and C_A dependent equations are used for DDBD methodology of design. PGA of 0.36g is used in the current study which represents the highest seismic zone in India. Later these buildings were subjected to lateral displacement using displacement-based pushover analysis method. The inelastic strength along with drifts achieved by DDBD are compared with the IS code compliant buildings. From the comparison, it was observed that inelastic strength achieved by 6,9 storey buildings designed according to FBD are greater than the buildings designed according to DDBD at "No Collapse". For 3 storey structure, the strength achieved by FBD is less than DDBD. If zone is reduced from Zone V to IV or PGA is increased from 0.36g, then inelastic strength achieved by DDBD with performance objective "No Collapse" would be more than FBD requiring revision of response reduction values for different categories of buildings or PGA values in respective seismic zones. The inelastic drifts achieved by both the methods are comparable only for 3 storey structure.

Keywords: FBD; DDBD; performance objective; capacity design; PGA, Inelastic Strength

1. Introduction

Over the past few decades, there is a surge in new design methodologies and philosophies, and it is needless to mention how Performance-Based Design (PBD) changed the way in which earthquake engineering is perceived. The main aim of PBD lies in achieving a targeted performance. Different performance levels can be targeted starting from the onset of cracking in individual members to complete collapse of the structure. This diversity in selecting a performance level helps to consider the different target performance depending on the design life and importance of the structure. Nevertheless, targeted performance can always be associated with hazard level and drift for removing the ambiguity from engineering perspective. The improvement in analysis tools like Nonlinear Time History Analysis (NTHA), Incremental Dynamic Analysis (IDA), Modal Pushover Analysis etc., enhanced the capabilities of the PBD methodologies. Although, there is a significant recognition for these methodologies in research and academic communities, they are not widely practiced not only because of their complexity and time consumption but also, lack of adaptability to the codes of practice unlike Force Based Design (FBD).

The adaptability of the FBD procedures to codes of practice is achieved through continuous observation of the behavior of structures in past earthquakes. In general, seismic design followed by many countries is Force-Based. In the early 20th century significant earthquakes like Kanto (1920), Napier (1932), Long Beach (1933) etc., occurred around the world causing a huge destruction [1]. The intricate observation of this destruction caused by these earthquakes revealed that the buildings designed for wind performed well during earthquakes that led to the interpretation of equal distribution of force throughout the building. Hence, certain percentage of seismic weight, is distributed vertically in the seismic design without any consideration to the dynamic properties of the structure. The observation of subsequent earthquakes proved that the above understanding to be wrong and further research helped in understanding the dynamic properties of the structure. Later, by early 1960, dynamic property-based distributions like period (T) dependent seismic force distribution were developed and implemented [1]. Applications of Finite Element Analysis (FEA) and Nonlinear analysis in earthquake engineering helped in identifying a unique behavior popularly known as ductility, which helps in achieving deformation without significant loss in strength. Realization of the role played by ductility paved way for the development of another important concept called capacity design that finally led to the research on performance-based design.

Since the evolution of Force based design, every new earthquake threw a new challenge to the then existing principles and assumptions. Many of the challenges were answered but validity of some of the assumptions like initial stiffness, interdependency of strength and stiffness, ambiguity among different standard codes in time period calculation, usage of cracked sections, way in which ductility capacity and response reduction factors are calculated were never addressed, as pointed out by Priestley et.al. [2]. Many of the national codes were built on same linage and Indian national code “Criteria for Earthquake Resistant Design of Structures”, IS:1893[3] is no exception. IS 1893 is accompanied by “Ductile Design and Detailing of Reinforced concrete structures subjected to seismic forces - Code of practice”, IS 13920[4]. Since the code was commissioned in the year 1962, IS 1893 had 5 revisions. Similarly, IS 13920 was commissioned in the year 1992 with improvement in capacity design concepts. Both the codes underwent a major revision in the year 2016 according to the latest developments that are taking place throughout the world. Changes in IS 1893: 2016 were majorly influenced by IBC 2015, NEHRP 2009, ASCE/SEI 7-10, NZS 1170.5:2004. Some of the major changes which were incorporated in latest version of IS 1893:2016 and IS 13920:2016 code, are as follows:

(a) In revision 5 (IS 1893:2002), there were terminologies like MCE (Maximum Credible Earthquake) and DBE (Design Basis Earthquake). MCE is defined as “the most severe earthquakes considered by the standard” and DBE is defined as “the earthquake which can reasonably be expected to occur at least once during design life of structure”. Sudhir k Jain [5] studied the ambiguity in terminology and suggested that these terms should be dropped as the seismic zone maps are not based on Probabilistic Seismic Hazard Analysis (PSHA) and hence, use of terms such as MCE and DBE can create confusion. Hence, MCE and DBE are dropped in revision 6 (IS 1893:2016). Also, in revision 5 (IS 1893:2002), the objective of “No collapse” is very clear and hence, damage is allowable, and the design intention is not to achieve “No Damage” and the same is evident in behavior of buildings during past earthquakes. But, in revision 6 (IS 1893:2016), the performance objectives are not clearly stated leading to ambiguity.
(b) For structural analysis, revision 6 (IS 1893:2016) suggests that the moment of inertia (I), should be considered as 70 % of I_{gross} for columns and 35 % of I_{gross} for beams for the design of beams and columns
(c) Structures located in seismic zone IV and V should consider design vertical earthquake effects in estimating the seismic design forces.
(d) Equivalent static method shall be sufficient for calculating the design forces on the structure when the natural period is less than 0.4 sec.
(e) When dynamic analysis is used, Design base shear V_b estimated shall not be less than $\bar{V}_b$ calculated from equivalent static method.
(f) In the latest revision of IS 13920:2016 the column to beam strength ratio was increased from 1.1 in previous versions to 1.4 in current version of code.

In addition to the above, there are many other changes like providing two spectra for equivalent static analysis and response spectrum analysis and extending the time period of SDOF's from 4sec to 6 sec etc., which will not affect the current study and hence will be discussed wherever necessary.

Above stated changes in code clearly gives an impression of the influence that the design codes had on strength governing design. Every clause will contribute to increase in force on the structure and restrict the possible nonlinear displacements before collapse, allowing the structure to fail intermittently neither reaching complete strength nor complete drift, if not practically at least numerically to start with. Increase in forces may not impact short buildings in most cases however, they have significant influence on the behavior of medium and high-rise structures. In high-rise structures, if every joint is maintained with a column to beam strength ratio of 1.4, intended drifts and in turn plastic hinge formation cannot be achieved in upper stories. Hence, a rational method for verifying the drift required at collapse for same structure is needed to compare the behavior of structure designed with enhanced clauses of IS 1893:2016. Direct-Displacement Based Seismic Design (DDBD) proposed by Priestley et.al [2] is one such PBD methodology which overcomes the limitations of FBD methodology, thus becoming an effective tool for comparison of buildings compliant with Indian standard code.

Some of the previous studies on FBD using IS 1893 and DDBD include a study conducted by Sheth et.al., [6] on 15 storey RC building. The building was designed as per IS 1893:2002 in both FBD and DDBD procedures and verified whether target drift was achieved or not. Another study was conducted by Sil et.al [7], to obtain a target drift of 2% after multiple iterations in both the methods and final cross-sectional sizes are achieved. Later, inter-storey drifts, material strain and ductility are compared.

This paper is an attempt to understand the performance comparison of selected structures between FBD method prescribed in the latest IS 1893 & IS 13920 with DDBD methods. As DDBD is performance-based design, it helps how a building is performing at design inter-storey drift. Initially, three buildings with 3, 6 and 9 storey assumed to be located in seismic zone V are designed using DDBD and FBD methods with a performance objective of "No collapse". The reason for choosing this performance objective is attributed to behavior of buildings during past earthquakes like Bhuj (2001), Sikkim (2011) etc., where performance of the buildings failed to meet the criteria of "No Collapse" and the other performance objectives stated in earlier versions of code. Though failure of buildings is attributed to code compliance, there are many questions on the sufficiency of the clauses such as reduction factors, safety factors and load combinations used in design. With the objective to understand the performance of structures at "No collapse", critical inter-storey drift of 4% is chosen and the selected three buildings are designed in DDBD also. To compare the performance, displacement-based pushover analysis is performed on the buildings designed according to FBD and DDBD. Two major parameters; strength and lateral drift are considered for comparison between the performance of both the sets of structures. Plan and Elevation details of the three structures are shown in Fig.1.

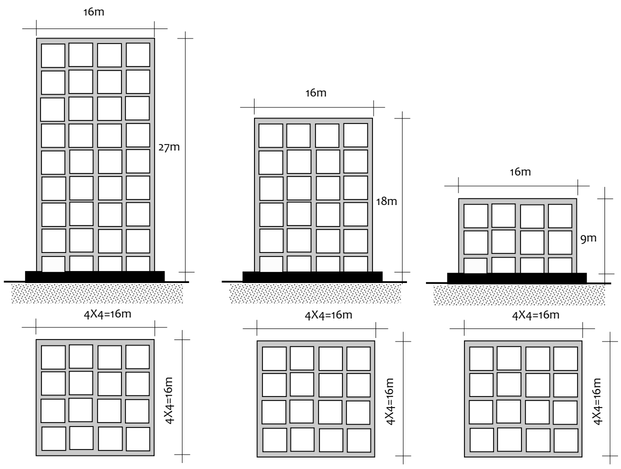


Fig. 1 – Plan and Elevation details of buildings with 3, 6 and 9 stories

2. DDBD Methodology

There are different limit states like cross section limit state and structural limit states. Cross section limit state represents at cross-sectional level and structural limit state represents the global response of the structure. Cross section limit states like cracking of concrete, yielding of rebar, spalling of cover concrete and ultimate limit state of a member where member strength significantly drops can be studied using the moment- curvature relationship of a cross section. Structural limit states like serviceability limit state, damage control limit state and No collapse limit state can be obtained by using any of nonlinear static or dynamic analysis procedures. The structural limit state depends on the performance objective chosen for the design and is the choice of designer.

For DDBD, the drift limit specified in modal code for Level 2 earthquake is 2.5%, drift limit for Level 1 earthquake is 1% without URM and 0.5% with URM. For Level 3, no limits are set. Here, for normal structures, level 1 earthquake represents serviceability limit state with 50% probability in 50 years, Level 2 earthquake represents damage control limit state with 10% probability in 50 years and Level 3 earthquake represents “No collapse” limit state with 2% probability in 50 years [2]. It is identified that the first two limit states are important for developed countries or countries having frequent earthquakes. For developing countries like India, where PSHA studies are not fully established, rather relying on probabilistic return period, it is better to achieve “No Collapse” limit state or survival limit state. This limit is exceeded when structure is no longer able to support its load and collapses. Many PBD based codes like SEAOC [8] suggests a design inter storey drift of 4% for achieving “No Collapse” limit state. Hence, a design inter storey drift for DDBD is set to 4% for the current study.

Once performance objective is selected, next important step in DDBD is selection of suitable displacement spectrum that may be specific to a region. Intuitively, the acceleration spectrum provided in IS 1893 should be used to develop the corresponding displacement spectrum in the absence of a site-specific spectrum. According to IS 1893: 2016, elastic acceleration spectrum is defined by the equations given in Eq. (1) and the resulting plot is shown in fig.2 (a). Elastic Displacement spectrum can be obtained by converting the acceleration spectrum into displacement spectrum by multiplying with $\frac{T^2}{4\pi^2} \times s_{a(T)} \times g$. The spectrum so generated is shown in fig.2 (b).

$$\frac{s_a}{g} = \left\{ \begin{array}{ll} \textit{For rocky or hard soil sites} & \begin{cases} 1+15T & T<0.1s \\ 2.5 & 0.1<T<0.4s \\ \frac{1}{T} & 0.24<T<4.0s \\ 0.42 & T>4.0s \end{cases} \\ \textit{For medium stiff soil sites} & \begin{cases} 1+15T & T<0.1s \\ 2.5 & 0.1<T<0.55s \\ \frac{1.36}{T} & 0.55<T<4.0s \\ 0.34 & T>4.0s \end{cases} \\ \textit{For soft soil sites} & \begin{cases} 1+15T & T<0.1s \\ 2.5 & 0.1<T<0.67s \\ \frac{1.67}{T} & 0.67<T<4.0s \\ 0.42 & T>4.0s \end{cases} \end{array} \right\} \quad \text{Eq. (1)}$$

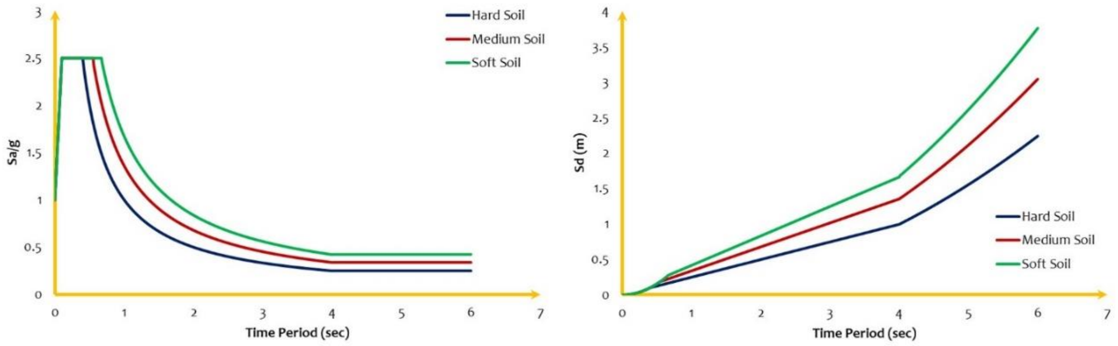


Fig. 2 – (a) Acceleration spectrum specified according to IS 1893(Part 1): 2016, (b) Displacement spectrum generated from the elastic acceleration spectrum of IS 1893: 2016[4]

Although the mere conversion yields a reliable displacement spectrum, there are two concerning issues which need to be taken care of. The corner period obtained in the displacement spectrum is too high that is not practically possible. This is due to the constant value of acceleration coefficient after a period of 4sec. This results in very high values of displacements above this period. For example, a comparison with Eurocode [9] suggests that the acceleration coefficient is inversely proportional to the T^2 value after the mentioned corner period. Secondly, the influence of low PGA values specified by the code. From the past earthquake studies, for example, 2001 Bhuj earthquake, it is identified that the PGA values observed at different locations during the earthquake are much higher when compared to those values specified in the code. This leads to a very lower level of displacement spectrum if converted from the code specified acceleration spectrum. This neglects the site-specific scenario that is actually to be considered for DDBD.

Another important aspect of DDBD in considering the displacement spectrum is that the achieved strength of the structure with respect to its design base shear. In general, a structure is usually designed for 25-30% of its seismic weight in the case of DDBD whereas it is designed for 10-15% of its seismic weight in the case of FBD. By carrying out a small case study of performing pushover analysis for 3 storey structure, it is identified that the maximum inelastic strength achieved by the FB designed structure is lower when compared to that of the DDB designed structure. This result indicates that the structure designed according to strength criteria (FBD) does not provide the minimum strength that is required. On the other hand, structure designed using DDBD provides a higher strength in addition to the designed drift required for the structure. From this, it can be concluded that the structure designed using FBD procedures lack the minimum load carrying capacity which is definitely not true. This issue may not be true with the codes of other countries as they specify comparatively higher PGA values and also rational reduction factors to achieve the inelastic drifts.

Therefore, the elastic acceleration spectrum used by Caponi C [10] for his comparison is used which is defined by the eq. (2). From the acceleration spectrum obtained, the displacement spectrum is derived using the same conversion as discussed previously.

$$S_a(T) = \begin{cases} PGA \times \left[1 + (C_A - 1) \times \frac{T}{T_A}\right] & 0 < T < T_A \\ PGA \times C_A & T_A < T < T_B \\ PGA \times C_A \times \frac{T_B}{T} & T_B < T < T_C \\ PGA \times C_A \times \frac{T_B \times T_C}{T^2} & T \geq T_C \end{cases} \qquad \text{Eq. (2)}$$

Where $S_a(T)$ is the acceleration expressed in the units of g. PGA is peak ground acceleration and in Indian standard scenario PGA varies from 0.1 to 0.36g for seismic zones II to V, respectively. C_A is amplification factor which is taken as 2.5. T_A,T_B and T_C are equal to 0.1sec, 1sec and 5sec, respectively. Fig.3 shows the displacement spectrum developed using the acceleration spectrum selected.

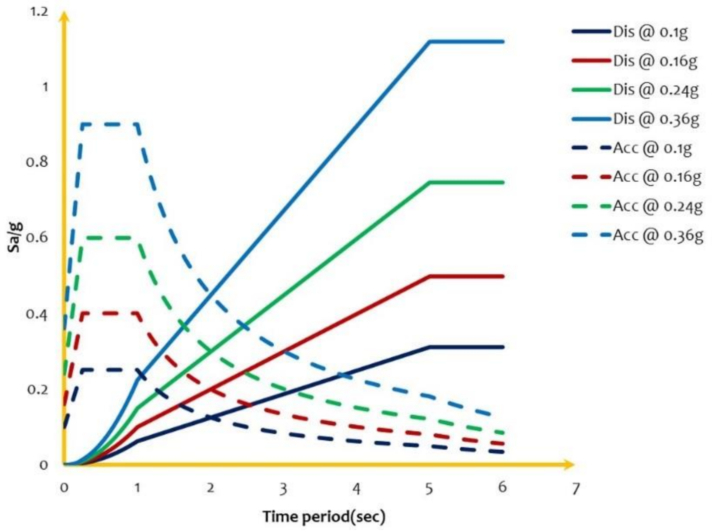


Fig. 3 – Acceleration and Displacement spectrum generated using the equations suggested by Caponi C [10]

By considering the displacement spectrum used by Caponi C [10] for comparative study, the selected three structures are designed according to DDBD methodology. The flow chart of the methodology is shown in fig.4.

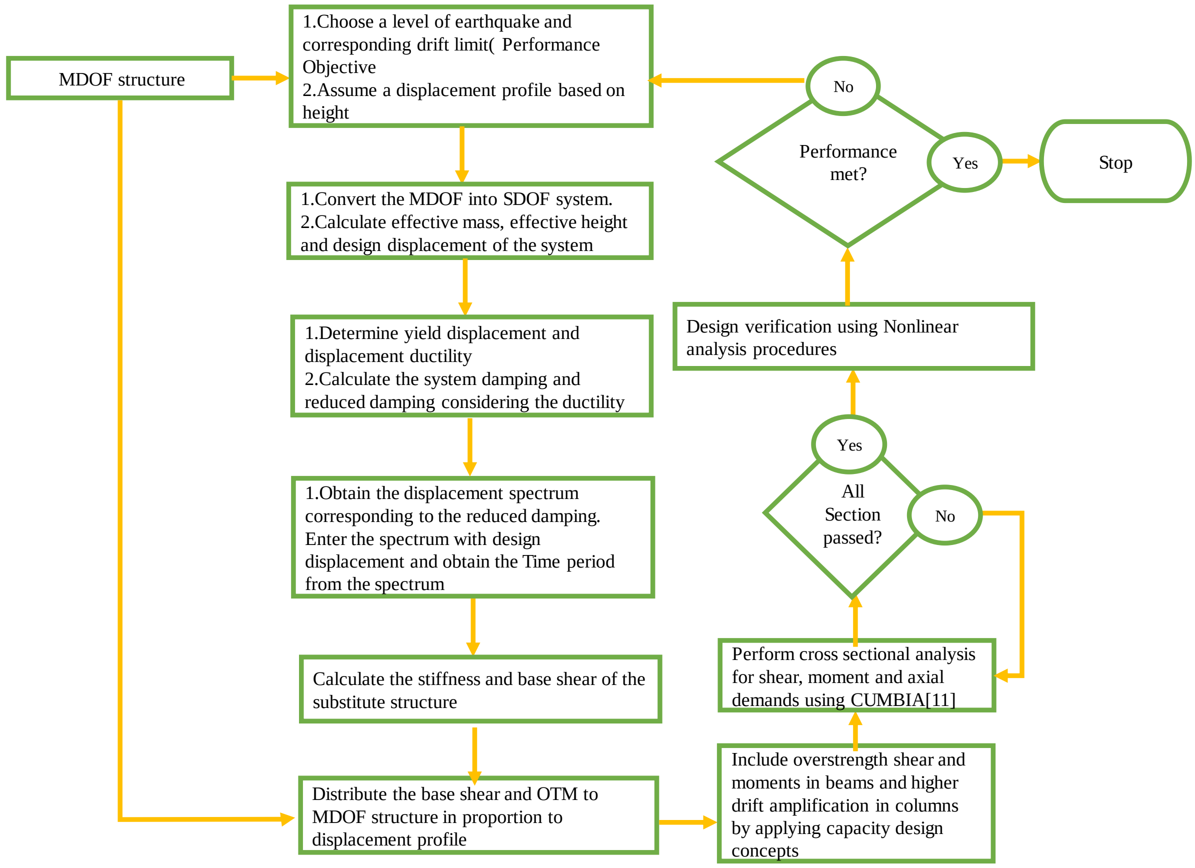


Fig. 4 – Flow chart of the DDBD procedure used for designing structures with 3, 6 and 9 stories

3. FBD methodology

As discussed earlier, the selected buildings are also designed according to FBD methodology that is suggested in Indian standard code of practice. The lateral forces are calculated distributing the design base shear calculated according to the seismic zone, importance factor and empirical time period of the building. For a proper comparison of the design methodologies of FBD and DDBD, the acceleration spectrum specified by the code is used for calculating the lateral forces. Further, the design lateral forces are obtained from the load combinations specified in the code including the influence of vertical ground motion on the structure. Performing a linear static analysis resulted in obtaining the design moments and shears that are considered as demand on the structure. The cross-section sizes of the members are obtained by revising the preliminary sections such that the load carrying capacity of the section is considerably greater than the design force demands. The obtained sections are checked for serviceability criteria as suggested by limit state design method. The design procedure adopted is mentioned in the flow chart shown in fig.5.

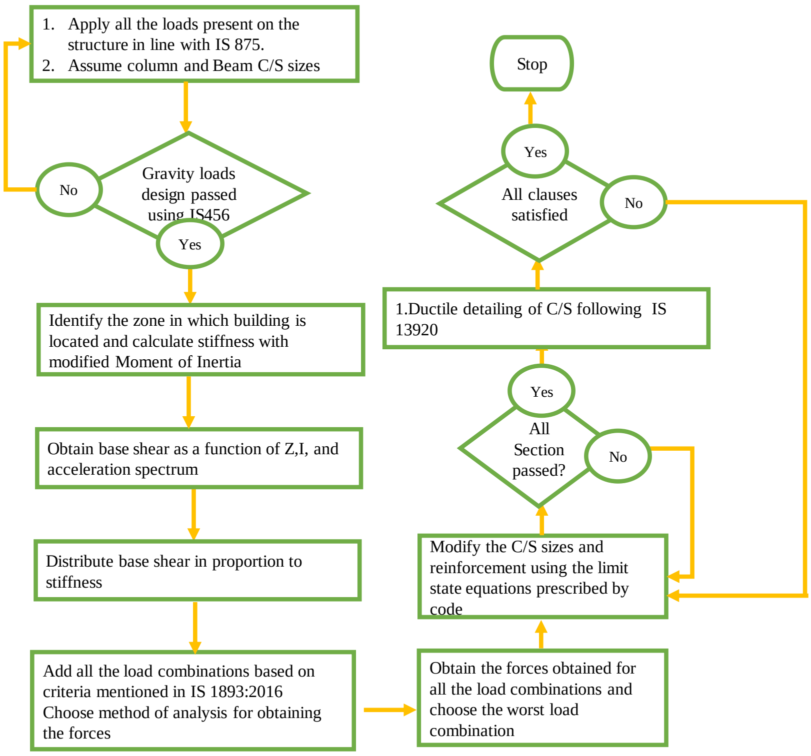


Fig. 5 – Flow chart of FBD procedure used to design the selected buildings with 3, 6 and 9 stories

4. Results and Discussions

Both the methods require calculation of base shear V_b as outlined in flow chart discussed previously and distribute the base shear vertically according to pattern chosen. There are differences in the way load pattern is chosen for FBD according to IS 1893:2016 and DDBD. FBD considers parabolic load pattern and DDBD consider triangular load distribution. Similarly, FBD utilizes load combinations and DDBD utilizes drift amplification factors for design. With inherent differences in both the methods, it is important to establish the differences at various levels i.e., design level and design verification at intended performance level.

4.1 Design Level

Key design parameters in any seismic design is obtaining the dynamic property of structures namely time period, mass and stiffness. If mass is assumed to be fixed, stiffness and time period vary according to the method chosen. Stiffness and time period are end values for DDBD methodology before proceeding for finalizing cross section sizes where FBD starts with assumptions in stiffness and time period. DDBD starts with displacement profile adopted with assumed inter-storey drift for achieving a specific performance level. The following differences can be prominent.

A. Though DDBD considers drift amplifications factors for columns and overstrength factors for both beams and columns, it will not consider material safety factors at design level. While performing capacity design calculations, value of material strength is incorporated, and cross section sizes are finalized. For capacity design calculations, DDBD suggests concrete compression strength of $f_c' = 1.7f_c$ and in general a value of $f_c' = 1.3f_c$ [2]. For rebar, yield stress as $f_y' = 1.3f_c$ for capacity design calculations. In general, $f_y' = 1.1f_c$ [2]. In order to make comparison fair, a value of $f_c' = 1.5f_c$ and $f_y' = 1.15f_c$ are considered.

B. During design stage DDBD considers triangular load distribution $\frac{w_i h_i}{\sum w_i h_i}$ and FBD according to IS code considers parabolic load distribution $\frac{w_i {h_i}^2}{\sum w_i {h_i}^2}$.

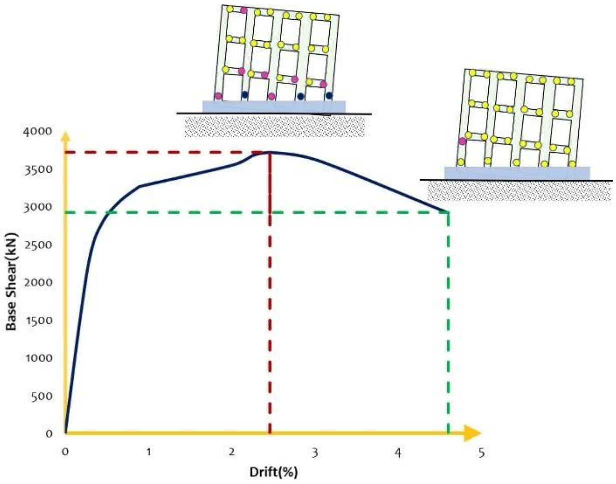
4.2 Performance Assessment of Designed Buildings

For the designed buildings, table 1 shows the dynamic properties according to FBD and DDBD procedures.

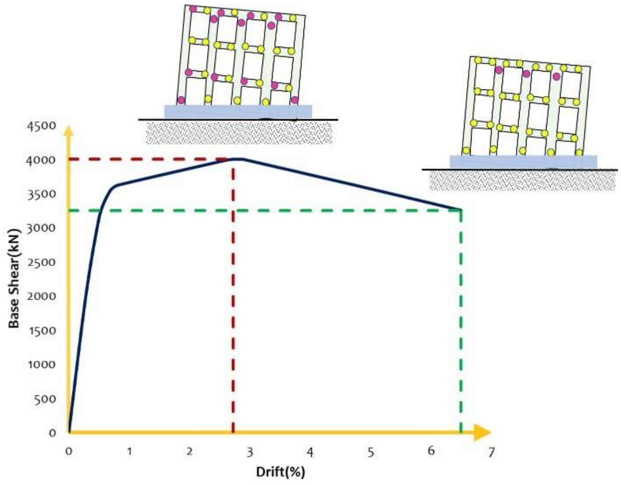
Table 1 – Dynamic properties for the selected structures obtained according to FBD and DDBD methods

Storey	Drift	$F_D = \left(\frac{V_b}{w}\right)_{DDBD}$	$F_{IS} = \left(\frac{V_b}{w}\right)_{FBD,IS}$	$F_{other} = \left(\frac{V_b}{w}\right)_{FBD,eq}$	$T_{FBD.code}$	T_{DDBD}
	2.5	0.46				1.92
3-storey	3	0.35	0.09	0.025	0.20	1.36
	4	0.23				1.08
	2.5	0.34				1.60
6-storey	3	0.25	0.09	0.0324	0.40	2.06
	4	0.16				2.94
	2.5	0.22				2.34
9-Storey	3	0.16	0.09	0.0324	0.60	3.01
	4	0.10				4.31

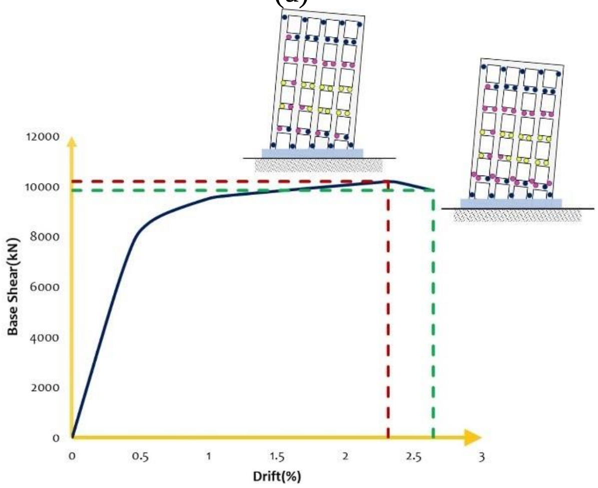
From Table 1, it is evident that design base shears according to DDBD methodology are 23%,16% and 10% for 3, 6 and 9 storey structures, respectively. Though base shears calculated according to FBD are 9-10% for highest seismic zones, it is required to understand whether the intended maximum inelastic strength is achieved in comparison with the building designed according to DDBD. To study the performance, the selected buildings are compared by performing non-linear static pushover analysis. Pushover analysis can be performed as force based or displacement based. For current study displacement-based pushover analysis for all the 3-dimensional structures is done using SAP2000 v.16[12]. Fig.6 shows the pushover curves obtained for structures with 3, 6 and 9 stories designed according to FBD and DDBD methodologies. The obtained pushover curves are normalized to understand the deviations from design base shear forces as shown in fig.7, fig.8, and fig.9 for structures with 3, 6, and 9 stories, respectively.



(a)



(b)



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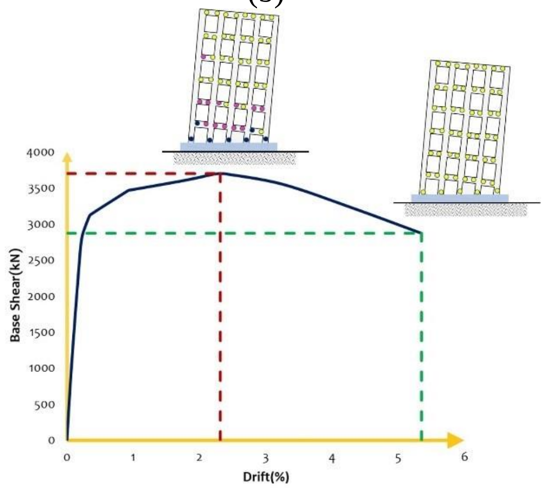
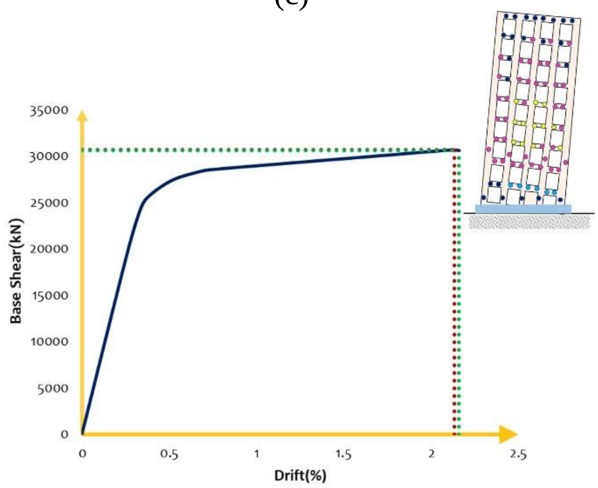
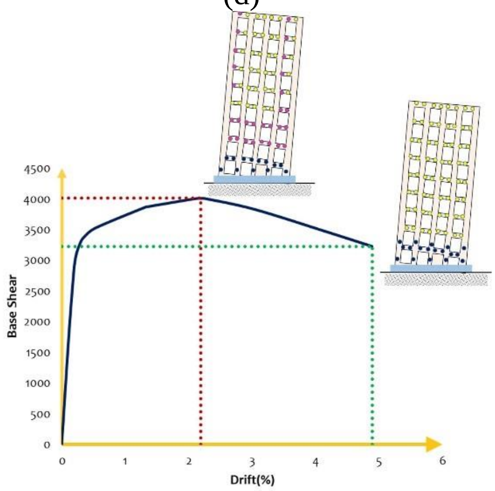


(d)



(e)



(f)

Fig. 6 – Pushover curves obtained for structures with (a) 3 storey designed for Force(FBD) (b) 3 storey designed for drifts(DDBD) (c) 6 storey designed for force(FBD) (d) 6 storey designed for drifts(DDBD) (e) 9 storey designed for force(FBD) (f) 9 storey designed for drifts(DDBD)

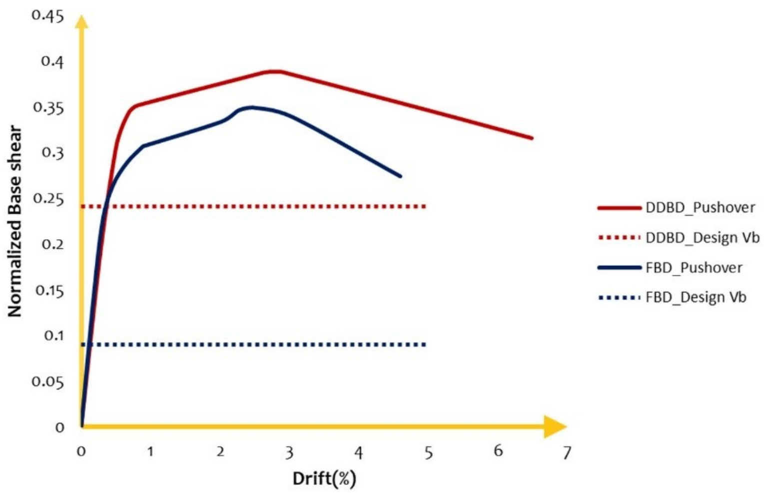


Fig. 7 – Normalized pushover curves for 3 storey structure

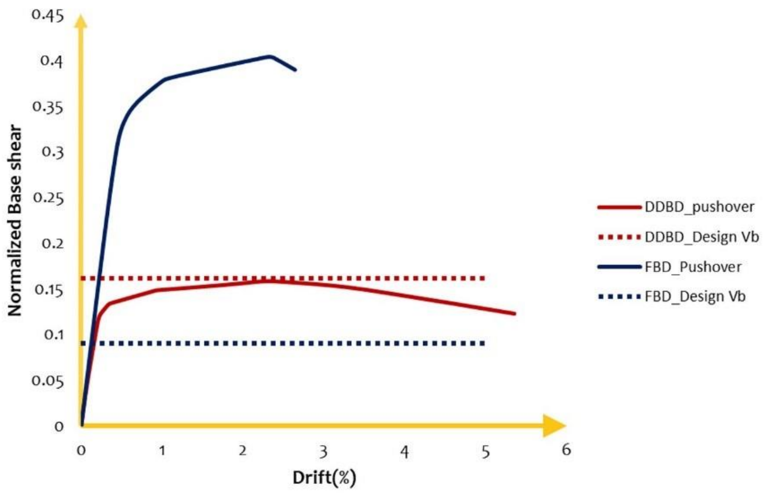


Fig. 8 – Normalized pushover curves for 6 storey structure

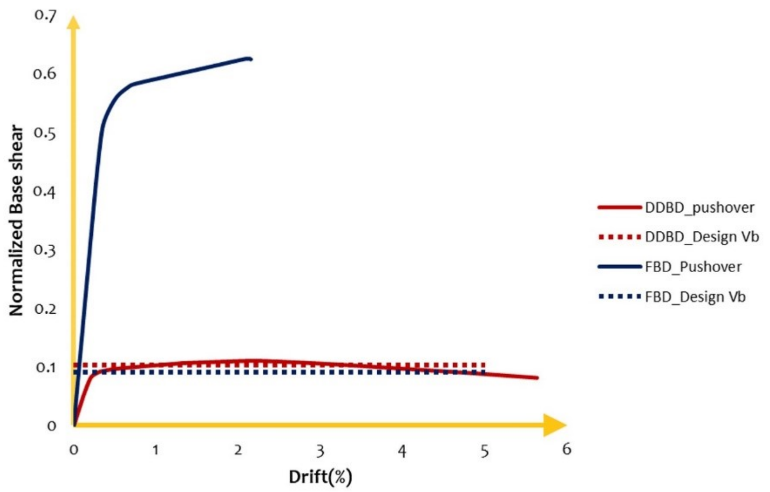


Fig. 9 – Normalized pushover curves for 9 storey structure

4.3 Hinge Mechanism

As shown in fig.9 for the 3 storey structure designed in DDBD, at maximum strength, except few in ground storey and top storey, all the beams reached collapse level. In columns, collapse hinges are formed at interior locations whereas exterior columns are at Immediate occupancy level. Also, in interior columns at top storey, hinges formed. No column sway mechanism was found. At 20% strength drop, all beams reached collapse level and top storey columns are at immediate occupancy level. At this level also, column sway mechanism at critical storey is noticed indicating a good design even at lower bound design with lesser PGA values. On the other hand, for a 3 - storey structure designed in FBD, at maximum strength collapse hinges are formed in all beams and no hinges were formed in columns. At 20% strength reduction, collapse hinges were formed in all beams and columns. Although, column sway mechanism has started to form at the final stages of pushover analysis but it not so serious as maximum capacity of all the beams were reached.

For the 6-storey structure designed in DDBD, at maximum strength, collapse hinges are formed in all beams at upper 4 stories and full capacity of first storey and second storey beams are not reached. At 20% strength reduction, collapse hinges are formed in all beams and columns representing an ideal plastic hinge mechanism. On the other hand, for the building designed in FBD, at maximum strength, collapse hinges formed only in second and third storey beams and after reaching the maximum strength, hinges are formed at the first storey column, there by triggering a storey mechanism.

For the 9-storey structure designed in DDBD, collapse hinges are formed at upper 6 stories and full capacity is not reached at lower storey columns. At 20% strength drop, all the top 8 storey beams reached maximum capacity and only at lower stories, collapse hinges in beams are not formed. At this level, column sway mechanism is noticed. In the FBD structure, the behavior of 9 storey structure is almost elastic with maximum strength equal to nearly 60% of its seismic weight. As the height of the building increases, elastic behavior is expected as design according to wind forces govern rather than earthquake. At maximum strength, column hinges are formed and mechanism is formed just when maximum strength is attained.

5. Conclusion

DDBD has high merit of achieving the intended performance. Out of 3 structures, for 2 structures normalized design base shear became tangent to load-displacement curve and overall drift of the structure is greater than 4% suggesting that the intended performance of the structure is achieved. Shorter period buildings in DDBD achieved slightly higher base shear and it can be achieved by reducing the moment overstrength factors, although not necessary as the performance is as expected. It is observed that at 4% inter-storey drift and PGA of 0.36g, strength achieved by DDBD is more than FBD for 3 - storey structure, whereas the strength for 6 - storey according to FBD is significantly high and for 9 - storey structure, the difference is very significant because of the cumulative effect of safety factors in design according to FBD. The merit of high strength is not sufficient for good performance, particularly for the 9 storey structure as it becomes very close to an elastic structure. The inherent reason is applying large number of load combinations (for e.g., 73) and maintaining beam-column strength ratio of 1.4 that enforces the column size to be bulky and in turn make it behave elastically.

In the current study, though the overall performance of DDBD is good, the inelastic strength achieved by DDBD is severely low because of underestimation of PGA. Many countries that have significant PSHA studies suggests that the PGA values should be in the range of 0.6-1.2, whereas maximum PGA considered in India is 0.36g divided by 2, considering DBE. If proper spectrum, PGA and inelastic drift are used, DDBD can easily chase down the inelastic strength achieved by FBD methodology. The drifts achieved by both the methods are comparable only for 3 - storey structure. For 6 - storey and 9 - storey structures, the inelastic drifts are significantly higher for DDBD method. With the ability to achieve the design inelastic strength and inelastic drift, it is quite easy for DDBD to achieve the inelastic strength achieved by FBD with good plastic hinge mechanism where as it would be a mounting task for an engineer to achieve the inelastic drift comparable with DDBD using FBD procedure beyond 2.5%, particularly when the height of the buildings increases. Having established the capacity of the DDBD procedure, it is important to understand the behavior of building subjected to wind especially tall buildings. As elastic design governs in wind, the occupancy comfort and safety of structures designed using DDBD should be verified. Also, above buildings should be subjected to real accelerograms and assess their behavior at collapse. Ultimately, as every method would have merits and demerits, precision and accuracy along with limitations, it should be an engineering judgement that should govern while choosing a design methodology rather than adaptability to a specific method in the information era.

6. References

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