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Comparative Parametric Study of Force Based and Displacement Based Design of a Base Isolated Structure

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Abstract*— Designing of structures for making it safe against seismic forces has been undergoing constant reappraisal in the past 25 years. The emphasis of the study has been shifted from strength to performance of a structure since it was realized that increasing strength need not necessarily increase the performance of the building nor reduce damage. The purpose of this study is to evaluate the response of the structure when subjected to different types of seismic forces using non-linear dynamic (Time history) analysis, to compare the base shear, drifts and displacements of a base isolated structure designed using the principles of force based design and displacement based design philosophies and to compare the percentage savings in reinforcement, if any, in a displacement based design.

***Index Terms*—Force Based Design, Direct Displacement Design, Base Isolated Structure, Comparative FBD and DDBD, Non Linear Time History Analysis.**

I. INTRODUCTION

The procedure of force based design which is currently practiced in India is carried out based on IS 1893-2002 where the stiffness, strength and time period are the initial inputs. The performance of the building is not evaluated in this method, but rather it emphasizes on the forces acting on the structure by using the initial stiffness, nominal material strength and an acceleration response spectrum based on 5% damping. On the other hand, the essence of direct displacement based design, which is a performance based design, is to characterize the structure with an effective stiffness (K_e) and an equivalent viscous damping (ξ_e). Direct displacement-based design uses an effective secant stiffness K_e to the design displacement Δ_D, the strength F_{max} corresponding to the design displacement, and displacement spectra for different levels of equivalent viscous damping [27].

II. ANALYSIS PROCEDURE

A building can be analyzed using Linear Static Procedure (LSP) Linear Dynamic Procedure (LDP), Non-linear Static Procedure (NSP) or Non-linear Dynamic Procedure (NDP). For regular short structures, where higher mode effects are not much significant, static procedures will be appropriate. But for tall buildings or buildings with torsional irregularities or non-orthogonal systems, dynamic procedures may be adopted.

In linear static analysis displacements, strains, stresses and reaction forces under the effect of applied loads are calculated. The results of a Linear Static Procedure may prove to be highly inaccurate for buildings with torsional irregularity or non-orthogonal force resisting system.

In a linear dynamic procedure, the building is modelled as a multi-degree of freedom system (MDOF). In linear dynamic analysis, the response of the structure to ground motion is calculated in the time domain, and all phase information is therefore maintained. Only linear properties are assumed. The analytical method can use modal decomposition as a means of reducing the degrees of freedom in the analysis.

The NSP is generally a more reliable approach to characterizing the performance of a structure than are linear procedures. However, it is not exact and cannot accurately account for changes in dynamic response as the structure degrades in stiffness; nor can it account for higher mode effects in multi-degree of freedom (MDOF) systems. The NSP shall be permitted for structures when higher mode effects are not significant. To determine if higher modes are significant, a modal response spectrum analysis shall be performed for the structure using sufficient modes to produce 90% mass participation.

The Nonlinear Dynamic Procedure consists of nonlinear response history analysis, a sophisticated approach to examining the inelastic demands produced on a structure by a specific suite of ground motion acceleration histories. As with the NSP, the results of the NDP can be directly compared with test data on the behavior of representative structural components to identify the structure's probable performance when subjected to a specific ground motion. Potentially, the NDP can be more accurate than the NSP in that it avoids some of the approximations made in the more simplified analysis. Response history analysis automatically accounts for higher mode effects and shifts in inertial load patterns as structural softening occurs. In addition, for a given earthquake record, this approach directly solves for the maximum global displacement demand produced by the earthquake on the structure, eliminating the need to estimate this demand based on general relationships.

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III. TARGET BUILDING PERFORMANCE AND ACCEPTANCE CRITERIA

1. Operational Building Performance Level: Buildings meeting this target Building Performance Level are expected to sustain minimal or no damage to their structural and non-structural components. The building is suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with power, water, and other required utilities provided from emergency sources, and possibly with some nonessential systems not functioning.
2. Immediate Occupancy Building Performance Level: Buildings meeting this target building performance level are expected to sustain minimal or no damage to their structural elements and only minor damage to their non-structural components. Although it would be safe to reoccupy a building meeting this target Building Performance Level immediately after a major earthquake, non-structural systems might not function, either because of the lack of electrical power or internal damage to equipment. Therefore, although immediate re-occupancy of the building is possible, it might be necessary to perform some clean-up and repair and await the restoration of utility service before the building can function in a normal mode.
3. Lift Safety Building Performance Level: Buildings meeting this level may experience extensive damage to structural and non-structural components. Repairs may be required before re-occupancy of the building occurs, and repair may be deemed economically impractical. The risk to Life Safety in buildings meeting this target Building Performance Level is low. This target Building Performance Level may entail more damage than anticipated for new buildings that have been properly designed and constructed for seismic resistance when subjected to their design earthquakes. Building owners may desire to meet this target Building Performance Level for severe ground shaking.
4. Collapse Prevention Building Performance Level: Structural Performance Level at Collapse Prevention, means the post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and—to a more limited extent—degradation in vertical-load carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity loads. Significant risk of injury caused by falling hazards from structural debris might exist. The structure might not be technically practical to repair and is not safe for re-occupancy because aftershock activity could induce collapse.

IV. LITERATURE REVIEW

In 1985, C.J. Derham, J.M. Kelly and A.G. Thomas [5] investigated the vertical and horizontal accelerations in 1/24th model of a containment structure by carrying out SIMQUAKE The bearings were tested for horizontal, vertical and fatigue resistance. No damage was evident for a displacement up to 38 cm which represents a shear strain of 125%. Young J. Park, Andrei M. Reinhorn and Sashi K. Kunnath [49] derived a formula for measuring the seismic damage of each component as well as storey level damage using a Damage Index, based on the linear combination of maximum deformation ratio and energy dissipation during cycle loading. Rafeal Riddell, Pedro Hidalgo, E. Cruz (1989) [37] studied the elastic and inelastic response spectra for various sets of earthquake records to study the response reduction factor to account for the energy dissipation capacity of the structures. Later in 1991, Uang [7] derived the basic formulas for establishing the response modification factor R and the displacement amplification factor Cd used in the National Earthquake Hazards Reduction Program (NEHRP) recommended provisions. During 1992, J.P. Moehle [15] had studied the idea of using structural displacement information directly in earthquake resistant design. Inaudi and Kelly in 1993[20] had defined a procedure for the optimum damping in linear isolation systems. The study has shown that the optimum isolation damping decreases with an increase in the number of degrees of freedom. J. S. Hwang and L.H. Sheng in 1994 [17] critically examines the newly available AASHTO isolation specifications. The unrealistic representation of the effective stiffness and equivalent damping ratio by the AASHTO isolation specifications is discussed. In this study, empirical formulae for the calculations of the equivalent period shift and equivalent damping ratio of the equivalent elastic system have been proposed and validated. Later, Miranda and Bertero in 1994 [11] has confirmed the above study by the results from various site investigations over the last 30 years. Jack P. Moehle in 1996[16] has elaborated the advantages and disadvantages of displacement based seismic design. According to him, damage in a yielding structure can be related more directly to displacement than to force or stress. Masaru and Ian during 1997[29] proposed an analytical hysteresis model for elastomeric seismic isolation for the purpose of accurately predicting the response of seismic isolated structure. Calvi and Kingsley in 1995[12] presented a displacement-based seismic design procedure for MDOF structures, with example designs of regular and irregular bridges. The displacement-based design process offered several advantages over a force-based process, among them the ability to consider explicitly the displacement demand (i.e. damage) in each member rather than assigning a single, force-based global behaviour factor to the structure. Simon Kim and Enzo D' Amore in 1999[41] has critically examined the fundamental assumptions on which the push over analysis procedures are based and the accuracy of the procedure has been assessed through a case study. It has been concluded that push over analysis is more realistic than existing code procedure. Typical code procedures are force based and ignores displacement based design philosophy and relies on linear elastic pseudo dynamic procedures in conjunction with the use of linear force reduction factors to estimate the seismic performance of structures. Whittaker, Gary Hart and Christopher Rojahn in 1999[47] has arrived at a draft

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formulation that represents the response modification factor as the product of factors related to reserve strength, ductility, and redundancy. The formulation splits R into factors related to reserve strength (Rs), ductility (Rm), and redundancy (RR).

In 2000, Medhekar and Kennedy[27] reviews the conceptual basis of the spectral acceleration-based design method currently used in seismic codes and its limitations. An alternative method that uses displacements as the basis for the design procedure is then presented. Peter Fajfar in 2000[35] explains a relatively simple nonlinear method for the seismic analysis of structures (the N2 method). It combines the pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system. A.M. Mwafy and A.S. Elnahsai in 2001[2] checks the validity and the applicability of inelastic static pushover analysis in comparison with 'dynamic pushover' idealised envelopes obtained from incremental dynamic collapse analysis. Satish Nagarajaiah and Xiaohong Sun during 2001[40] observes the performance of the base-isolated Fire Command and Control (FCC) building in Los Angeles which experienced strong motion during the 1994 Northridge earthquake. It was evident from the evaluation that the base isolated FCC building performed well, except for impact, which increased structure shear, and drift demands. R.D. Bertero and V.V. Bertero in 2002[38] discussed the main requirements that a reliable Performance Based Seismic Design (PBSD) should satisfy and present a conceptual comprehensive numerical procedure for the PBSD of buildings. Oscar M. Ramirez, Michael, C. Constatinou, and Chriz Z. Chrysostomou in 2002[34] has undertaken studies to support the development of the 2000 NEHRP (National Earthquake Hazards Reduction Program) Provisions for the design of buildings with energy dissipation systems. Tysh Shang Jan in 2004[44] proposed in his study, a new simplified pushover analysis procedure, which considers higher mode effects. The basic features of the proposed procedure are: the response spectrum-based higher mode displacement contribution ratios, a new formula for determining the lateral load pattern and the upper-bound (absolute sum) modal combination rule for determining the target roof displacement. Y.Y. Lin, M.H. Tsai, J.S. Hwang and K.C. Chang in 2003[48] presented a seismic displacement-based design method for new and regular buildings equipped with passive energy dissipation systems (EDS). Using the substitute structure approach for the building structure and simulating the mechanical properties of the passive energy dissipation devices (EDD) by the effective stiffness and effective viscous damping ratio, a rational linear iteration method was proposed. T.J. Sullivan, G.M. Calvi and M.J.N. Priestly in 2004[43] has explored the basis and performance of initial stiffness and secant stiffness based DBD methods. The paper identifies various challenges associated with the application of both initial stiffness and secant stiffness based DBD methods and considers whether one form is more effective than the other. Sajal Kanti Deb in 2004[39] discusses 3D nonlinear analysis procedure of base isolated buildings. The influence of soil–structure interaction (SSI) and possible effects of building and foundation rocking are examined by investigating the modal properties of the isolated system.

Later in 2005, Bommer and Mendis[22] reasoned that direct displacement-based seismic design and assessment require input in the form of displacement response spectra over long period ranges (up to the product of the yield period and the square root of the ductility demand factor) and for a number of damping levels (up to about 30% of critical). M.J.N. Priestley, G.M. Calvi, and M.J. Kowalsky in 2005[25] has shown that the influence of hysteretic characteristics has been underestimated in recent force-based studies. These assertions are supported by results of recent analytical studies, which have included refinement of ductility/equivalent-viscous damping relationships, and an examination of the important (and largely ignored) role of "elastic" damping in inelastic time-history analyses. Mehmet Inel and Hayri Baytan Ozmen in 2006[30] carried out a study to investigate the possible differences between push over analysis of default hinge and user defined hinge models. The observations showed that the user defined hinge model is better than the default hinge model in reflecting non-linear behaviour compatible with element properties.

C.P. Providakis in 2007[6] elaborates in his paper that the effects of near-fault (NF) ground motions with large velocity pulses can bring the seismic isolation devices to critical working conditions. This implies that, beyond some levels of supplemental damping, the isolated buildings still remain vulnerable to damage if drifts are not controlled carefully. Asgarian and Shokrgozar in 2009[4] evaluated the overstrength, ductility and response modification factor of buckling restrained braced frames with various number of stories and type of bracing. Chopra and Reyes in 2010[21] has proposed The PMPA (Practical Modal Pushover Analysis) procedure that is especially suitable for practical application to compute seismic demands directly from the earthquake response (or design) spectrum.

In 2010, D. Cardone, M. Dolce and G. Palmero[9] designed a procedure for Direct Displacement Based Design (DDBD) for buildings equipped with seismic isolation system. The key aspect for the proposed procedure is the definition of the target displacement profile for the structure. It is assigned by the designer in order to accomplish given performance levels, expressed in terms of maximum Isolator displacement and maximum inter-storey drift. A.B.M. Saiful Islam, Mohammed Jameel and Mohammed Zamin Jumat in 2012[1] examines the cost benefit of seismic isolation. Seismic base isolators increase the building costs with its price as well as installation cost. But the cost reduction for reinforcement in upper floors for horizontal and vertical members makes up for that cost. Jared Weisman and Gordan P. Warn in 2012[19] arrived at a procedure to assess the stability using a ratio of areas, referred to as the overlapping area method, to determine the critical load capacity of elastomeric and lead rubber bearings at a given lateral displacement that must be greater than a

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combination of axial forces imposed on the bearing. M.Mouzzoun, O. Moustachi, A. Taleb, S. Jalal in 2013[24] evaluate push over analysis as an effective tool to assess the seismic performance of buildings under different level of shaking. Young-Sun Choun, Junhee Park and In-Kil Choi in 2014[50] studied the effects of variability of the mechanical properties of lead rubber bearings on the response of a seismic isolation system. According to the study, variation provisions in the ASCE-4 are reasonable, but more strict variation limits should be given to isolation systems subjected to ground motions having low A/V ratios. Syed Ahmed Kabeer and Sanjeev Kumar in 2014[42] has analysed the adequacy of the base isolation against earthquake damage when compared to the conventional earthquake resistant design.

In 2015, Mohammed Ismail [31] proposed an inherent self-stopping mechanism of a Roll-In-Cage (RNC) isolator to limit the horizontal displacement of an isolated structure under excitation stronger than the design earthquakes or under near fault earthquakes considering limited seismic gaps without exhibiting direct pounding of the RNC isolated structure with the surrounding adjacent structures. Ima Multaji, Fransiscus Asisi and Kevin Willyanto in 2015[13] has studied the performance of Force Based Design and Direct Displacement Based Design on a concrete special moment resisting frame. It was concluded that DDBD performed better than FBD in predicting seismic demand of the structure ie storey drift because it deliberately designs the structure to achieve a given performance limit state.

Somwanshi and Pantawane in 2015[32] demonstrates how an isolation system can be efficient, evaluating its effectiveness for the building in terms of maximum shear force, maximum bending moment, base shear, storey drift and storey displacement reductions. From analytical study of an isolated building with its fixed base counterpart, , it was observed that for both models of symmetric as well as asymmetric, fixed base building have zero displacement at base of building whereas, base isolated building models shows appreciable amount of lateral displacements at base. Dilip J. Chaudhari and Tushar A. Dalawi in 2015[10] designed RC moment resisting frames to be analysed by the Performance based Plastic design method and conventional elastic design method. It is then evaluated by Non-linear static and Non-linear dynamic analysis under different ground motions. The increased hysteretic energy dissipation of the frame indicates that the structure utilizes its capacity lying in the inelastic zone. Cancellara and Angelis in 2016[8] has proposed a high damping hybrid seismic isolator (HDHSI) obtained by the assembly in series of a lead rubber bearing (LRB) and a friction slider (FS) characterized by a high friction coefficient. With respect to a traditional LRB system which is not equipped with a sliding mechanism, the HDHSI system has shown to be particularly efficient in counteracting seismic events characterized by high intensity and/or anomalous frequency content.

V. INFERENCE OF LITERATURE SURVEY

1. Comparative studies has proven that a displacement based design can bring more efficiency in terms of design and cost, compared to a force based design.
2. A base isolated structure reduces the lateral forces acting on a structural system by increased damping.
3. Introducing a direct displacement based design for a base isolated structure can prove to be more economical for such structures
4. It has been shown in recent studies that a 10% of cost of structure is incurred in properly isolating a building from the harmful effects of ground motion. But this cost is justifiable for a building like a hospital building or other important structures and services like water tanks, which should remain intact after a major event.

VI. DYNAMIC PARAMETERS OF STRUCTURE

A. The dynamic parameters considered are for San Francisco region

$S_s = 1.5g$, $S_1 = 0.638g$, $F_a = 1$, $F_v = 1.5$

$S_{xs} = F_a \times S_s = 1.0 \times 1.5g = 1.5g$

$S_{x1} = F_v \times S_1 = 1.5 \times 0.638g = 0.957g$

$S_{DS} = 2/3\ S_{xs} = 1g$

$S_{D1} = 2/3\ S_{x1} = 0.638g$

Site Classification D (Stiff Soil 15 < N value < 50)

$S_{DS} < 0.167$ g

Therefore $\rho = 1.0$

B. Set target period and target displacement

For most base isolated system, the period of building should be set to 2 to 3 seconds. The periods TD and TM corresponding to DBE and MCE can be established at 2.5 and 3.0 seconds respectively.

C. Obtain Effective Stiffness and displacement

Effective period at design displacement

$T_{D,} = 2\pi \sqrt{(W/(KDmin \times g)}$

$(2.5)^2 = (2\pi)^2 \times \frac{W}{KDmin \times g}$

$K_D min = 92974$ KNm

Similarly effective period at maximum displacement,

$T_M = 2\pi \sqrt{(W/(KMmin \times g)}$

$K_M min = 64565$ KNm

Fundamental Period

$T_a = C_t h_n^x$ (ASCE 7 – 2002 cl: 9.5.5.3.2-1)

For moment resisting RC frame, $C_t = .044$ and $x = 0.9$

$T_a = .044 \times 33.3^{0.9} = 1.03$ Seconds

Since $T_a = 1$ second, it shows that the building is stiff.

$T_0 = 0.2 \frac{SD1}{SDS}$

$= 0.2 \times \frac{0.638}{1} = 0.127$

$T_S = \frac{SD1}{SDS} = \frac{0.638}{1} = 0.638$

Since Ta > Ts,

$Sa = \frac{SD1}{Ta} = \frac{0.638}{1} = 0.638$

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D. Examine Minimum displacement permitted by dynamic analysis

$D_D = (\frac{g}{4\pi 2})(\frac{Sx1\ x\ TD}{BD1})$

$D_M = (\frac{g}{4\pi 2})(\frac{Sx1\ x\ TM}{BM1})$

Assuming a 15% damping for the isolators, from Table 9.13.3.3.1 of ASCE 7-10 2002, we get BD1 and BM1 as 1.383

Substituting the values, we get

D_D = 0.429 m

D_M = 0.515 m

E. Establish Minimum base shear below and above Isolation interface

To establish the minimum base shear, the minimum effective stiffness required is increased by 10% to accommodate for the aging effects of the bearing.

$V_B = 1.1\ x\ KDMIN\ x\ DD$ = 1.1 x 92974 x 0.429 = 43874.43 KN (Below Isolation Interface)

$V_S = \frac{1.1\ x\ KDMIN\ x\ DD}{R1}$ = 21973.22 KN

Where R1 is the response reduction factor taken as $^3/_8$ times the Response modification factor with a minimum value of 1 and a maximum value of 2. In our case, R1 = 2.

F. Drift Limits

With reference to ASCE 7 -10 2002 Cl. 9.13.4.7.4, maximum inter-storey drift corresponding to design lateral force including displacement due to vertical deformation of isolation system shall not exceed the following limits.

I) The maximum inter-storey drift of the structure above the isolation system calculated by response spectrum analysis shall not exceed 0.015hsx

II) The maximum inter-storey drift above isolation system calculated by time history analysis based on force deflection characteristics of nonlinear elements of the lateral force resisting system shall not exceed 0.02hsx

Where hsx is the total height of the building.

In this case, the total height of the building is 33.3 m. Hence the maximum drift limits are

i) For response spectrum analysis : 0.5m

ii) For time history analysis : 0.66m

VII. DESIGN OF LEAD RUBBER ISOLATOR

The design of the lead bearing isolator was done as per the following steps [32].

Maximum Vertical Reaction = 21592 KN

$Target\ period, Teff\ =\ 2\ Seconds$

Structural Damping β = 5%

$Spectral\ Accelaration, Sa\ =\ 0.638$

$Design\ Displacement,$

$$Dbd\ =\ \frac{Teff}{2\pi} * Sa\ =\ 0.2m$$

$Effective\ Stiffness,$

$$Keff\ =\ \frac{2\pi}{teff} * \frac{Wi}{g}\ =\ 21723\ KN/m$$

Energy Dissipated,

$Ed\ =\ 2Keff * Dbd2 * \beta\ =\ 89.59$ KNm

$Force\ at\ zero\ displacement,$

$F0\ =\ \frac{Ed}{(4*Dbd)}\ =\ 110.29$ KN

$Stiffness\ of\ lead\ core,$

$Kpb\ =\ \frac{F0}{Dbd}\ =\ 543.07$ KN/m

$Stiffness\ of\ rubber,$

$Kr\ =\ Keff - Kpb\ =\ 21180$ KN/m

$Total\ thickness\ of\ LRB,$

$tr\ =\ \frac{Dbd}{Y}\ =\ 0.14$m

$Diameter\ of\ lead\ rubber\ bearing,$

$Dbearing\ =\ (Kr * tr/(400\ \pi))1/2\ =\ 1.51$m

Diameter of lead core of LRB,

$$Dpb\ =\ \frac{4f0}{(\pi\sigma pb)^{1/2}}\ =\ 0.11m$$

$Area\ of\ lead\ core\ in\ LRB,$

$Apb\ =\ \pi/4 * (Dpb)2\ =\ 0.01m^2$

$Dia\ of\ force\ free\ area,$

$Dff\ =\ Dbearing\ -\ 2tr\ \ =\ 1.24$m

$Force\ free\ area,$

$$Aff\ =\frac{\pi}{4} * (Dff)^2\ =\ 1.21m^2$$

$Total\ loaded\ area,$

$AL\ =\ Aff - Apb\ =\ 1.2m^2$

$Circumference\ of\ force\ free\ section,$

$Cf\ =\ \pi * t * Dff\ =\ 0.0389m$

$Shape\ factor,$

$Si\ =\ AL/Cf\ =\ 30.74$

As per the displacement calculated, stiffness required the bearing was selected. The properties of the lead bearing isolator selected are

No	Property	Value
1.	Isolator diameter	1.36m > 1.35m
2.	Axial load capacity	27600KN > 21591 KN
3.	Maximum permitted displacement	860 mm > 250 mm
4.	Height of isolator	405 mm
5.	No: of rubber layers	20
6.	Lead plug diameter	0.38 m
7.	Compressive Stiffness	5300 KN/mm

Table I

Accordingly, the isolator properties in other directions were calculated based on ASCE 41-13

1. *Area of lead plug of diameter 0.38 m*

   $Ap = \frac{\pi}{4} d^2$ = 113400 mm^2

2. *Characteristic strength of lead plug*

   $Q = Ap * \ \sigma yl$= 903.4 KN

3. *Area of rubber layer*

   $Ar = Total\ area - Lead\ Plug\ Area$ = 1.34 * 10^{6} mm^2

4. *Post yield stiffness*

   $Kp = \frac{Ar*G*Fl}{H}$ = 1465 KN/m

Where

G = Shear modulus for rubber = 0.385 N/mm^2

F_L = Factor taken as 1.15 (ASCE 41 -13)

5. *Elastic stiffness*

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It is taken as 10 times the yield stiffness = 10 * 1465 = 14650 KN/m

6. *Yield Strength*

$$Fy = Q + Kp * Dy$$

$$Dy = Q/(5.5 * Kp)$$

$$Fy = 903.4 + \frac{903.4}{5.5*1465} = 904 \text{ KN}$$

7. *Post Yield Stiffness Ratio*

$$\frac{Kp}{K1} = \frac{1465}{14650} = 0.1$$

VIII. THE EXPERIMENTAL PROGRAM

The building is a G + 6 storied building with a total height of 33.3 m excluding the isolator height. It is an RCC framed single moment resisting frame. The model is as shown below. Two models were created, as base isolated. First one was analyzed as a force based structure. Plastic hinges were provided on the beams and shear wall of the second model so that the non-linear properties of the material is utilized. Hence the beams and walls were analyzed as deformation controlled units in the second model. The columns in the second model was kept as force controlled units thus following the strong column week beam model. The model is as shown in Fig: 1

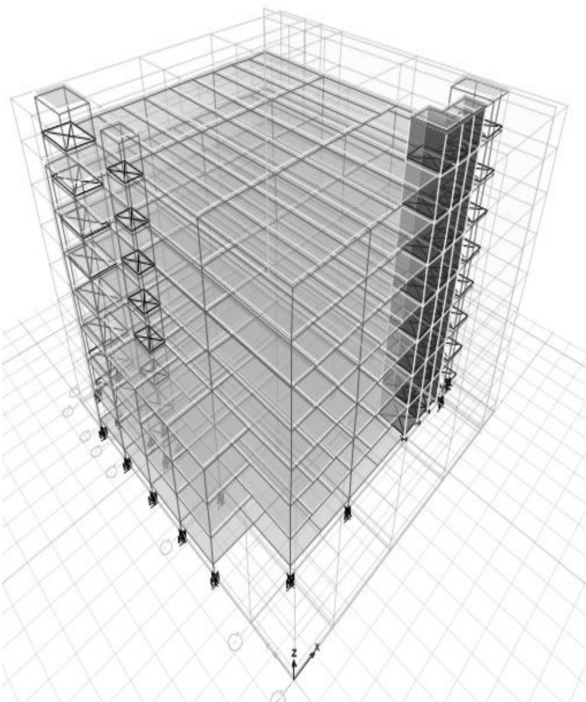

Fig: 1: Etabs Model of Structure

Step 1: Defining Material Properties

While the material properties are defined, the non-linear behavior of the material has to be defined as per the acceptance criteria mentioned in ASCE 7-10. The criteria mentioned for Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) are as shown in Fig: 2

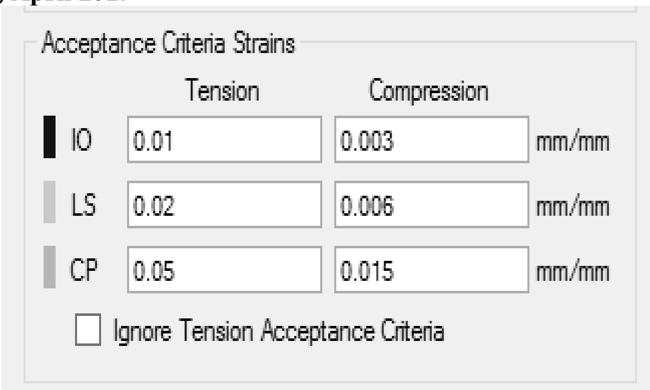

Fig: 2 Acceptance Criteria Status

Step 2: Defining the section Properties

The Sections that are used in the analysis are defined in the step. The sections being used are

Section Type	Length	Width
Column C1	1000	1000
Column C2	300	1000
Beam B1	300	700
Beam B2	600	800
Beam B3	500	700

Table II

Step 3: Response Spectrum of the building

The Response Spectrum of the building is plotted based on the Ss and S1 values provided. The plotted response spectrum of the building is given in Fig: 3

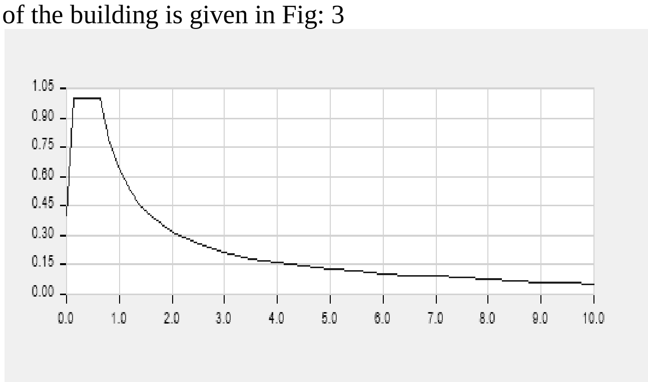

Fig: 3 Response Spectrum of Building

Step 4: Providing Ground Motion Data

The Ground Motion Data for 3 sets of ground motion, the Imperial Valley earthquake, the Bhuj quake and time history of Imperial Valley matched to the response spectrum of the building, are provided.

Step 5: Providing Load Combinations and Load Cases

The various load combinations are provided. The load cases that are then set to run are shown in Fig: 4

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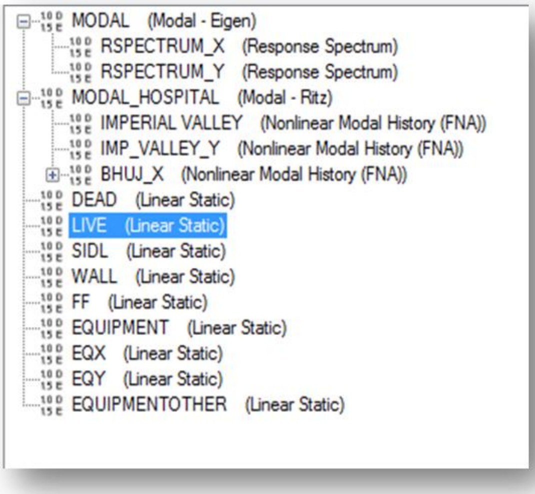

Fig: 4 Load Cases

Step 6: Providing Link Details

For a Seismically Isolated Structure the isolator is provided as links and the link details has to be provided. The required properties were calculated and a Lead bearing which matched the calculated properties were selected from the list provided by the manufacturer (Dynamic Isolation Systems). These properties are then assigned to the links drawn between the base and the ground floor. The properties assigned are

Directional Properties of Isolator required in E-Tabs

Direction U1

Effective Stiffness - 5300 KN/mm
Effective Damping - 10%

Direction U2 and U3

Linear Properties

Effective Stiffness - 13902
Effective Damping - 10%

Non Linear Properties

Stiffness - 14650 KN/m
Yield Strength - 904 KN
Post yield stiffness ratio - 0.1

Step 7: Assigning Hinges for a Displacement Based Design

Where ever a displacement based design has to be followed, hinges are to be provided where rotation can be allowed. Here it is assumed a force based behavior for the columns and a displacement based behavior for the beams, where DDBD is adopted. Hence auto hinges are provided at all the beam column joints, on beams and fiber hinges are assigned to the shear walls. The critical load combination selected here which was found out from the static analysis carried out at the beginning of the procedure was 1.2 DL+ 1.0 EQ + 1.0 LL.

Step 8: Analyzing the Model

Two models were analyzed for this study.

1. Base Isolated – Force Based Design
2. Base Isolated – Displacement Based Design

Step 9: Design of the Structure

To compare the relative merits and demerits of the two models, the design check was carried out.

IX. RESULTS AND DISCUSSION

On comparison of the Moments for the beams where plastic hinges were provided, it was found that the moments in the beams modelled with plastic hinges drops down compared to that of the beams designed as force controlled units. Hence the requirement of reinforcement is also reduced. The Design moment and reinforcement of a single beam component is provided here as an example.

For a based isolated structure with the beam components designed to act as force controlled units, the area of reinforcement required was provided in e-tabs as in Fig: 5

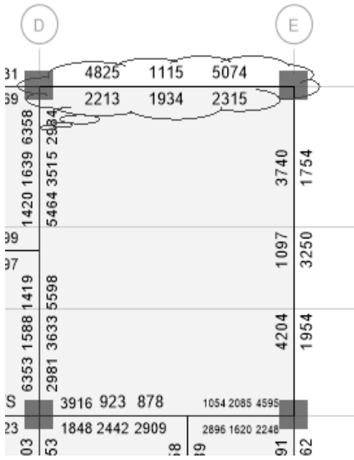

Fig: 5 Area of Reinforcement along Grid DE – 7 FBD

For the selected beam under study which is marked in fig: 5, the moments that the beams are designed to resist for are provided as in Fig: 6

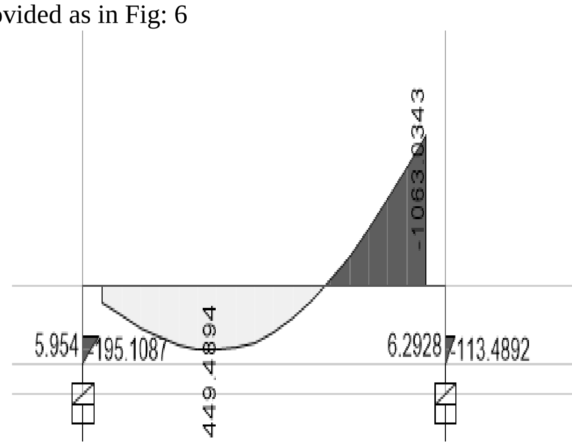

Fig: 6 Moments along Grid DE-7 FBD

Similarly, for a base isolated structure with beam components designed to act as deformation controlled units, the area of reinforcement required for the same beam as marked in Fig: 7

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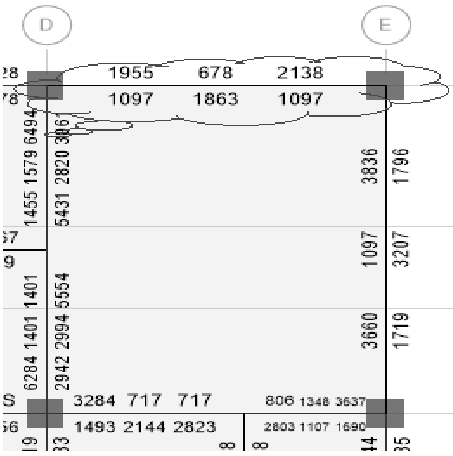

Fig: 7 Area of Reinforcement along Grid DE – 7 DDBD

And the moments were (Fig: 8)

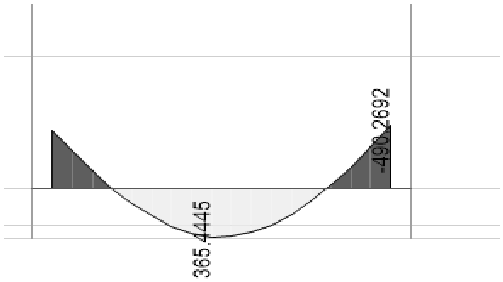

Fig: 8 Moments along Grid DE-7 DDBD

The comparison results for base shear for FBD and DDBD based design is graphically plotted as shown in Fig: 9

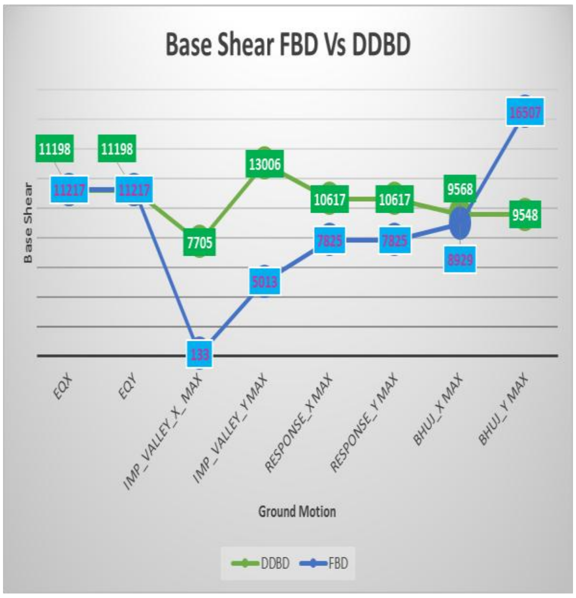

Fig 9: Comparison of Base Shear

The graphical representation of the drift values of the Force based design and the displacement based design are shown in Fig: 10

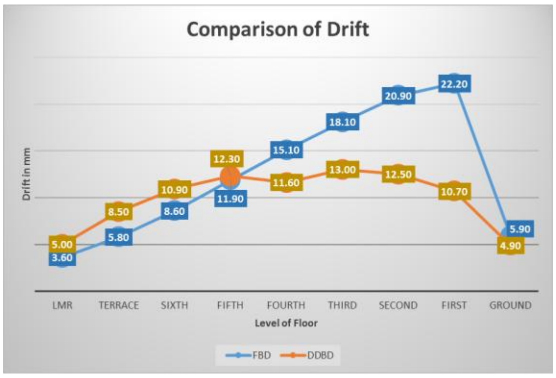

Fig: 10 Comparison of Drift Values

The results of displacement for both the methods are analysed and graphically compiled as shown in Fig 11

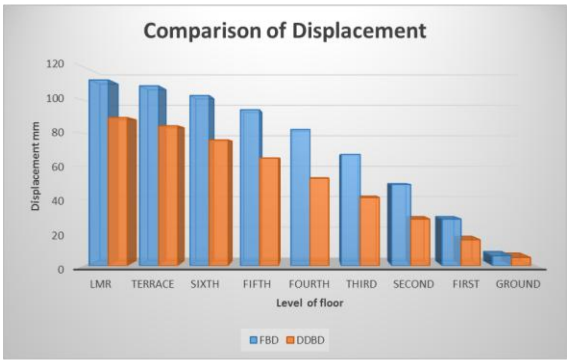

Fig 11: Comparison of Displacements

Finally, on comparing the fundamental period of the structures, the following comparison was drawn as in Fig 12

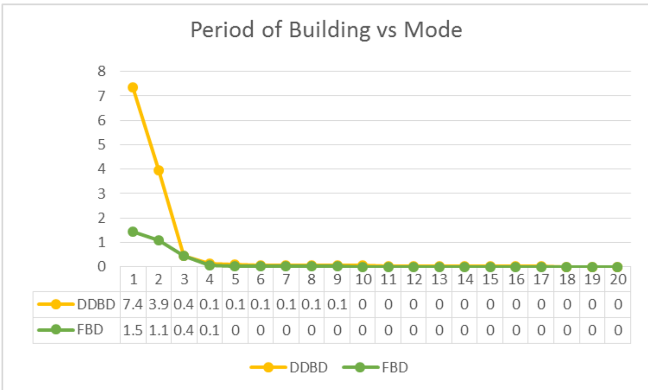

Fig 12: Comparison of Fundamental Period

X. CONCLUSION

The following conclusions were drawn based on the experimental results.

1. There is significant reduction in moments of a displacement based design due to provision of hinges. Hence there is reduction in the area of steel used for a base isolated building designed using the philosophy of DDBD compared to its FBD counterpart. By applying hinges to the beams, the

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structural member becomes plastically indeterminate. Hence plastic analysis is applied. The ultimate load is found from the strength of steel in plastic range resulting in reduction of the moment

2. The structure utilizes its capacity lying in the in-elastic zone.

3. For a structure, the base reaction is its horizontal acceleration co-efficient multiplied by the weight of the building. The acceleration co-efficient is dependent upon the importance factor of the building, the spectral accelerations, the site class and the Response Reduction Factors. Since these are constant for all the above models, there is no variation in the base reaction in any case.

4. There is no significant difference in displacement of a fixed base structure in FBD or DDBD design. A marginal decrease in displacement can be seen for a base isolated building with DDBD compared to its FBD counterpart. However, the load combination in which the maximum displacements and drifts occur were found to be different.

5. DDBD is a design philosophy. The basic parameters of the building like drift or displacement is not affected by following the principles of DDBD. An optimisation can be effected by reducing the area of steel by using its non-linear material properties.

6. A fraction of the extra cost incurred in isolating the building can be recovered by optimising the structure using the philosophy of DDBD since the method provides striking economy as regards to the weight of steel.

XI. FUTURE SCOPE

After the Bhuj earthquake, the need for seismically isolation of important structures in India has been felt. The Indian code is silent about the base isolation technique used for protecting a building from seismic hazard. It is high time that the Indian code is modified to include base isolation techniques. The code of practise followed in India is for force based design. To compare the results of the analysis, we require some acceptance criteria for displacements and drifts. These criteria needs to be set in Indian code. The DDBD design can be further optimised by iteratively reducing the frame section sizes based on the demand capacity ratio for a required performance level. The variation in base shear of the structure in FBD and DDBD for various ground motion needs to be studied further. It has been noted that the fundamental time period of the structure increases in DDBD. The relationship between the increase of time period and the use of secant stiffness in DDBD is another area of future scope.

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