Non-Linear Time History Analysis of Steel Moment Resisting Frame with Energy Disspating System

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Abstract—a new seismic protection strategy called the hybrid passive control device (HPCD) has been developed which combines typical passive energy dissipation devices. It consists of a high damping rubber (HDR) sandwich damper in series with a buckling restrained brace (BRB). The HPCD provides energy dissipation at small deformations without significantly decreasing the structural period. The significant energy dissipation capacity of a BRB is provided for significant seismic events in the second phase. The transition between these two phases consists of an increasing stiffness as the device transitions from rubber damper to BRB. The HPCD reduces deformations, forces and accelerations from seismic events. The behaviour of a 9 storied building was studied in this work with and without HPCD using Time History analysis on SAP2000.

Keywords-hybrid passive control device, buckling restrained brace, time history analysis, high damping rubber, inter-storey drift.

I. INTRODUCTION

The backbone of structural engineering practice is for a structure to perform well. The objective of designing a structure is to provide safety to life and property, keeping the design economical. Guidelines provided in design codes fulfil this objective. But, the need for seismic design is necessary. The current design codes specify guidelines which provide simple seismic design procedures, through empirical factors and simple equations. However the behaviour of the structure is complex and depends on various factors and hence the sufficiency of these designs based on simple equations cannot be predicted.

In addition to gravity loads, a structure has to be designed for anticipated lateral loads. The lateral loads subjected to a structure can be due to wind or earthquake. A structural engineer needs to develop an effective lateral load resisting system, which would prevent collapse and damage to life and property.

In the event of an earthquake, the structure is subjected to lateral forces which are generated by the structure's inertia resisting motion. These forces can be very high in magnitude. To design a structure to remain elastic during earthquake would be impractical and uneconomical.

A number of techniques and devices have been proposed, tested and successfully installed in structures for seismic protection. Base isolation is one of the techniques used to reduce the seismic demand on structures. This technique controls inter-storey drift and high floor acceleration. Energy dissipating devices in the form of dampers are used in the lateral force resisting systems. Viscous and visco-elastic dampers, friction dampers, metal dampers are known to be effective in reducing inter-storey drift.

This paper studies the behaviour of a hybrid passive control device system (HPCD) on a 9 storied composite structure with steel moment resisting-frames. Moment-resisting frames consists of beams and columns, with the beams rigidly connected to the columns. The lateral forcesare resisted primarily by rigid frame action-that is, by the development of bending moment and shear force in the frame members and joints. By virtue of the rigid beam-column connections, a moment frame cannot displace laterally without bending the beams or columns depending on the geometry of the connection. The bending rigidity and strength of the frame members is the primary source of lateral stiffness and strength for the entire frame [25].

The HPCD is a combination of two passive control devices which work complementarily. The HPCD comprises of a high damping rubber system (HDS) and a buckling restrained brace (BRB). Justin D. Marshal (2008)[22]stated that the HPCD works in 2 stages. In the first stage the material properties of the high damping rubber namely the viscoelastic property comes into play. The benefit of the first stage is energy dissipation for all deformation levels. This stage is reached during the action of wind and small earthquakes and no structural repair is required. The second stage of the HPCD is a hysteretic yield device. This stage is beneficial because it has the ability to dissipate significant energy and also increase the stiffness of the device. The displacements are reduced with increase in stiffness thereby preventing instability of the building which could lead to collapse. The second phase of the HPCD only engages during major earthquakes and focuses the permanent damage into elements that can be replaced without affecting the main structure.

Reference [23] states that "Buckling restrained braces (BRBs) or Unbonded Braces (UBs) commonly found are made from encasing a core steel cross-shape or flat bar member into a steel tube and confined by infill concrete. The steel core member is designed to resists the axial forces with a full tension or compression yield capacity without the local or global flexural buckling failure. When the brace is subjected to compressions, an unbonding material placed between the core member and the infill concrete is required to reduce the friction. Thus, a BRB or an UB basically consists of three components, including steel core member, buckling restraining part and the unbonding material. The buckling restraining part can be constructed from mortar filled in the tube, reinforced concrete, reinforced concrete covered with FRP or all-metallic steel tubes. The energy dissipation or damage prevention capacity of a steel framed structure can be greatly enhanced by employing buckling-restrained braces (BRBs). They usually consist of a steel core capable of undergoing significant inelastic deformation and a casing for restraining global and local buckling of the core element. A BRB exhibits stable hysteretic behaviour with high-energy dissipation capacity".

In this study inverted steel pipes have been used as BRB.

II. Design Data And Analysis

The study in this project is based on the non-linear analysis of the building subjected to loadings as discussed in this chapter.

A. *Frame Configuration*

The building under consideration is a symmetrical building. For analysis purpose a single frame shall be considered. The bay width is 9m and column height is 4m. There are 9 stories (G+8) and 5 bays.

B. *Load Data*

The building is intended for commercial use and hence the live loads shall be in accordance to the same as specified by IS 875 Part II.

Dead load shall be calculated as per material density specified in IS 875 Part I.

Floor finishes are assumed as 1.5 KN/m^2 on typical floor and 3 KN/m^2 for the roof.

Live load considered is 5 KN/m^2 as specified for commercial buildings in IS 875 Part II.

Although live load is to be neglected for the roof a load of 1 KN/m^2 is assumed.

C. *Material Property*

Pre-defined material property from SAP2000 was used in analysis and design. The properties are as follows:

Weight per unit volume = 76.9729 KN/m^3
Modulus of Elasticity, E = 1.999×10^8 KN/m^2
Poisson's Ratio, υ = 0.3
Coefficient of Thermal Expansion = 1.17×10^{-5}/°C
Minimum Yield Stress, f_y = 344737.9 KN/m^2
Minimum Tensile Stress, f_u = 448159.3 KN/m^2
Minimum Yield Stress, 1.1×f_y = 379211.7 KN/m^2
Minimum Tensile Stress, 1.1×f_u = 492975.2 KN/m^2

D. *Frame Design*

The building frame considered in this study is assumed to be located in Indian seismic zone III with medium soil conditions. The design peak ground acceleration (PGA) of this zone is specified as 0.16g. Each bay is divided into three panels 3m each. The design of slab is as follows:

1) *Typical Floor Slab*

a) *Design Data*

$$L_x = 3\text{m};\ L_y = 3\text{m};$$

$$\frac{L_y}{L_x} = 1$$

Therefore, slab is a 2-way slab.
Cover to the main reinforcement = 20mm
Grade of concrete (f_{ck}) = 25 N/mm^2
Grade of steel (f_y) = 415 N/mm^2

$$\frac{L_x}{D_{eff}} = 23$$

(Interpolating between 20 and 26 as per IS 456:2000)
Effective depth required = 93.17mm
Overall depth of slab = 125mm
Bar diameter in x direction = 8mm
Bar diameter in y direction = 8mm

b) *Load Calculation*

$$\text{Dead load} = 0.125 \times 25 = 3.125\ \text{KN/m}^2$$
$$\text{Live load} = 5\ \text{KN/m}^2$$
$$\text{Floor finishes} = 1.5\ \text{KN/m}^2$$
$$\text{Total load} = 9.625\ \text{KN/m}^2$$
$$\text{Factored load} = 9.625 \times 1.5 = 14.4375\ \text{KN/m}^2$$

Slab support condition: 2 adjacent sides discontinuous.

c) *Moment Calculation*

$M_{x(-ve)} = \alpha x^* W_u^* (L_x)^2$ = 6.11 KN-m
$M_{x(+ve)} = \alpha x^* W_u^* (L_x)^2$ = 4.55 KN-m
$M_{y(-ve)} = \alpha x^* W_u^* (L_x)^2$ = 6.11 KN-m
$M_{y(+ve)} = \alpha x^* W_u^* (L_x)^2$ = 4.55 KN-m

d) *Steel Calculation*

$$A_{stx} = 0.5\frac{f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 * M_x}{f_{ck} * b * d^2}}\right] d = 165.50\ \text{mm}^2$$

Spacing required for 8 mm bar = $\frac{165.50}{50.24}$ = 303 mm

Provide 8mm bar at 250 mm c/c.

Summary of steel details is as tabulated below:

2) *Roof Slab*

The design of roof slab will be same as the typical floor slab; however the load calculation will vary.

LOAD CALCULATION
Dead load = 0.125*25 = 3.125 KN/m^2
Live Load = 1 KN/m^2
Floor finishes = 3 KN/m^2
Total load = 7.125 KN/m^2

LOAD CALCULATION ON BEAM
FOR TYPICAL FLOOR BEAM

$$\text{Thickness of wall} = 23\ \text{cm}$$
$$\text{Storey height} = 4\ \text{m}$$
$$\text{Depth of beam} = 381\ \text{cm}$$
$$\text{Density of masonry} = 20\ \text{N/mm}^2$$
$$\text{Load due to wall} = 0.23 \times (4 - 0.381) \times 20 = 16.65\ \text{KN/m}$$
$$\text{Load due to slab} = \frac{1}{2} \times 3 \times 1.5 \times \frac{9.625}{3} = 7.22\ \text{KN/m}$$
$$\text{Load on central beam} = 16.65 + 2 \times 7.22 = 31.09\ \text{KN/m}$$

For Roof Beam

$$\text{Load due to slab} = \frac{1}{2} \times 3 \times 1.5 \times \frac{7.125}{3} = 5.34\ \text{KN/m}$$
$$\text{Load on central beam} = 2 \times 5.34 = 10.68\ \text{KN/m}$$

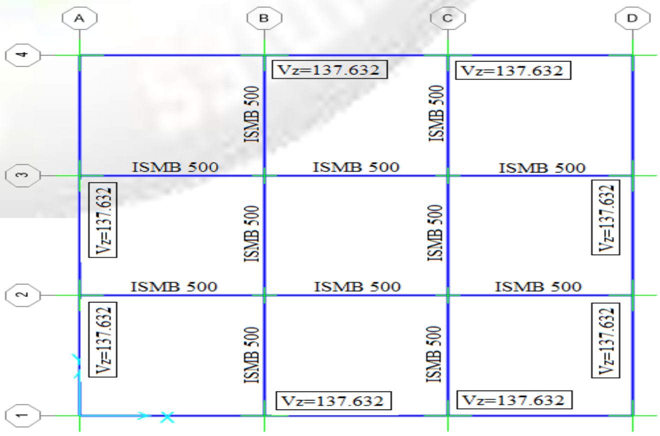


Fig.1: Slab Model in SAP 2000

A slab 9m×9m was modelled in SAP2000 as shown in figure 1. The area was divided in panels 3m×3m each. Total slab load was applied on internal beams and the grid was designed. The beam sizes are as shown in the figure above. The vertical reaction Vz=137.632KN is obtained from the analysis results. This reaction is to be applied on the main frame beam. The vertical reaction Vz for the roof beam is 103.86KN.

In addition to the point loads the main beam will have a uniformly distributed load due to slab and wall. For the typical floor the distributed load is 31.09 KN/m and for the roof the same is 10.68 KN/m.

E. Selection of Time History

Earthquake codes recommend that the selection of the accelograms should be such that the mean spectral acceleration (the SRSS of both components) covers the design spectrum. The main criterion for analysis using time history method is that the selection and scaling of the available accelograms should be compatible with the design spectrum of the location under consideration.

The accelogram corresponding to Imperial Valley, selected from the PEER Strong Ground Motion Database has been used in this study. The earthquake moment magnitude Mw for the selected record is 7. The purpose of selecting this particular time history is that the soil condition in this place is similar to the soil condition in the considered location. The spectral acceleration coefficient (Sa/g) depends upon the soil condition (clause 6.4.5, IS 1893:2002). The design horizontal seismic coefficient (A_h) is proportional to the spectral acceleration coefficient. This coefficient A_h is used to determine the design seismic base shear (V_B). Hence, it is evident that the total design lateral force is dependent on the soil condition of the location considered. The details of the Imperial Valley Time-History are tabulated below:

Sr. No.	Location,Year, Mw	Station	Type of Soil	PGA
1	Imperial Valley,1940,7	El Centro array #9	Deep Broad Soil	0.313g

Table.1: Details of Imperial Valley Earthquake

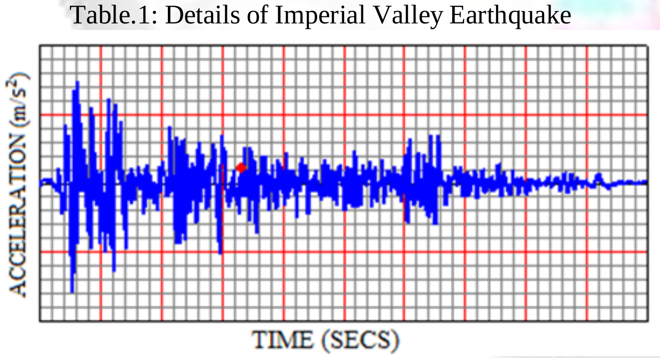


Fig.2: Time History Plot for ImperialValley

The load combinations considered as per IS 800:2007 were:

1.5D+1.5L+1.5EQ

0.9D+1.5EQ

Where D=Dead Load; L=Live Load; EQ=Earthquake Load.

F. Analysis

Non-Linear analysis was carried out and the members were designed as per IS 800:2007. The displacement at the roof level was checked. The maximum drift obtained was 152.9mm. The maximum permissible drift as per IS 1893:2002 is 0.4% of the height. That is 0.004×H, where H is the height of the structure from the ground. The height of the structure is 36m. Therefore, the maximum permissible deflection is 144mm. The design of frame members is as shown in Figures 3 and 4. British sections were used. The schedules of sections used are described below:

Member	Section
C1	As shown below
C2	As shown below
C3	UKC 356×406×634
B1	UKC 356×406×551
B2	UKC 356×406×467
B3	UKC 356×406×393
B4	UKC 356×406×340
B5	UKC 356×406×235

Table.2: Frame Sections for Bare Frame

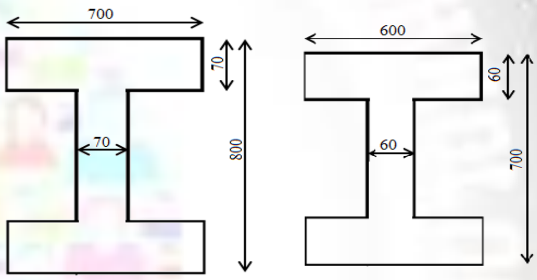


Fig.3: C1 and C2

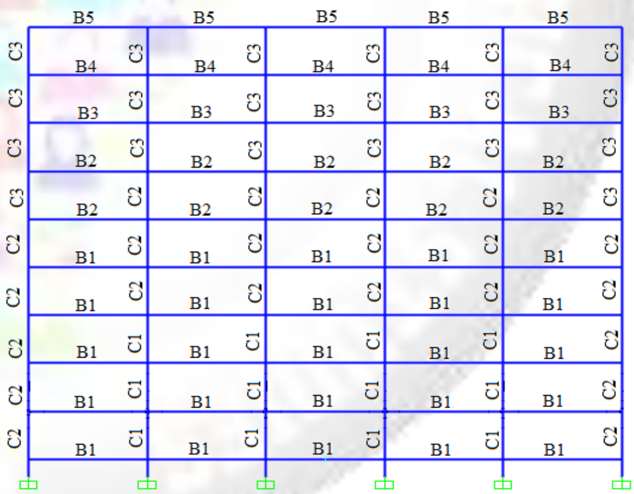


Fig.4: Frame Sections for Bare Frame

In order to reduce the roof drift, dampers were modelled as shown in figure 5. The dampers consisted of a BRB and a rubber device. The Mathematical model of the damper is as shown below

$$\frac{1}{k_{eq}} = \frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_3} + \frac{1}{k_4}$$

$$\frac{1}{k_{eq}} = \frac{k_1 + k_2}{k_1 k_2} + \frac{1}{k_3} + \frac{1}{k_4}$$

$$\frac{1}{k_{eq}} = \frac{k_1k_3 + k_2k_3 + k_1k_2}{k_1k_2k_3} + \frac{1}{k_4}$$

$$\frac{1}{k_{eq}} = \frac{k_1k_3k_4 + k_2k_3k_4 + k_1k_2k_4 + k_1k_2k_3}{k_1k_2k_3k_4}$$

$$k_{eq\,(HDS)} = \frac{k_1k_2k_3k_4}{k_1k_2k_3 + k_2k_3k_4 + k_1k_3k_4 + k_1k_2k_4}$$

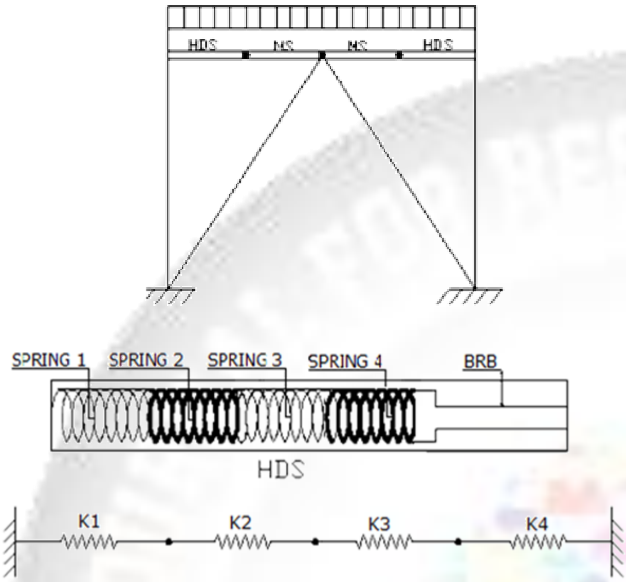
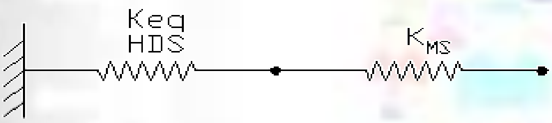


Fig.5: Mathematical model of Hybrid Damper



$$\frac{1}{[k_{eq}]_{HDS+MS}} = \frac{1}{[k_{eq}]_{HDS}} + \frac{1}{[k]_{MS}}$$

$$\frac{1}{[k_{eq}]_{HDS+MS}} = \frac{[k]_{MS} + [keq]_{HDS}}{[keq]_{HDS}[k]_{MS}}$$

$$[k_{eq}]_{HDS+MS} = \frac{[keq]_{HDS}[k]_{MS}}{[k]_{MS} + [keq]_{HDS}}$$

The schedules of sections used are described below:

Member	Section
C1	As shown in fig. 3.4
C2	As shown in fig. 3.4
C3	UKC 356×406×634
C4	UKC 356×406×551
C5	UKC 356×406×467
C6	UKC 356×406×393
B1	UKC 356×406×393
B2	UKC 356×406×340
B3	UKC 356×406×235

Table.3: Frame Sections for HPCD Frame

Member	Section
P1	Pipe 15 12
P2	Pipe 18 18

Table.4: Schedule for BRB Sections

The stiffness of the spring was varied along the height of the building to optimise the roof drift. The target displacement

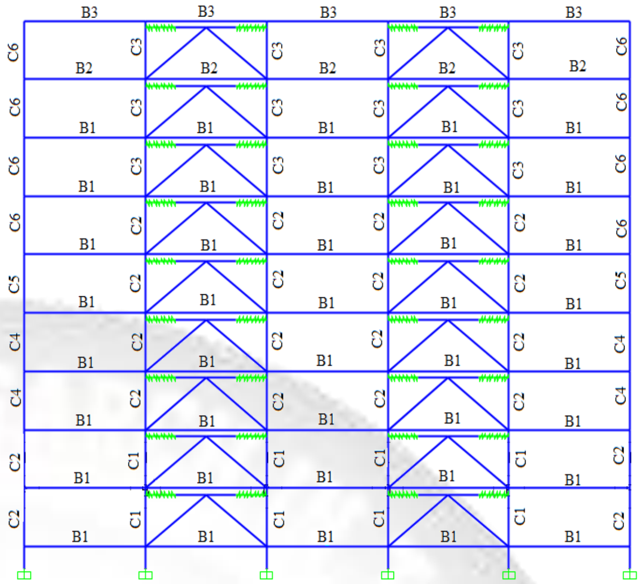


Fig.6: Frame with Dampers

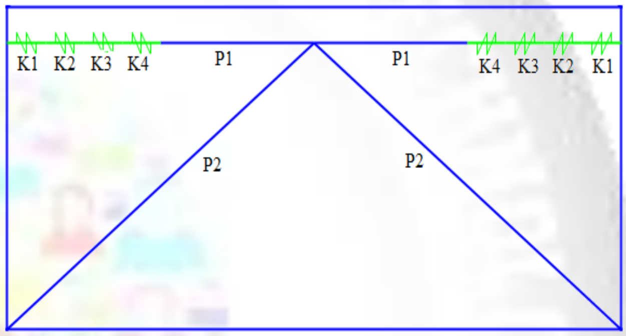


Fig.7: Enlarged View of Damper.

Was set as 30mm. The stiffness of springs was follows:

a. For Storey 1-3
   i. K1 = 25000 KN/m
   ii. K2 = 50000 KN/m
   iii. K3 = 75000 KN/m
   iv. K4 = 100000 KN/m
b. For Storey 4-6
   i. K1 = 24000 KN/m
   ii. K2 = 48000 KN/m
   iii. K3 = 72000 KN/m
   iv. K4 = 96000 KN/m
c. For Storey 7-9
   i. K1 = 23000 KN/m
   ii. K2 = 46000 KN/m
   iii. K3 = 69000 KN/m
   iv. K4 = 92000 KN/m

The roof drift obtained by this arrangement was 29mm.

III. RESULTS

The results have been presented in graphical form and have shown some strengths and some weaknesses of the hybridpassive damping system (HPCD).

The performance of a building during earthquake is assessed by various parameters such as acceleration, joint displacement, inter-storey drift, roof drift and damage to non-structural elements. The results obtained from the two frames are presented in the form of graphs

A. Base Shear

Base shearis lateral force that will occur due to seismic ground motion at the base of a structure. From the results tabulated below it is clear that the base shear in the frame with HPCD is lower than the bare frame. This is because in the HPCD frame the lateral force resisting capacity has been increased by passive energy dissipating devices, thereby reducing the demand at the foundation level.

FRAME	BARE	HPCD
SHEAR Fx (KN)	4907.41	4168.971

Table 5 IDA Results for Base Shear

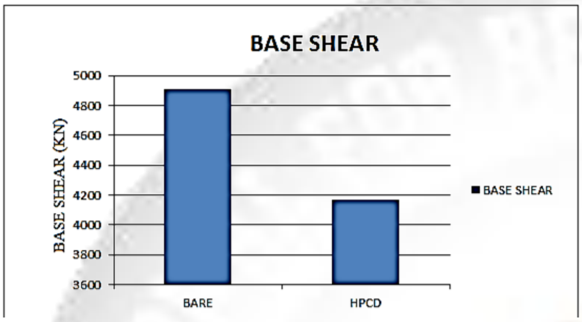


Fig.8: Graph of Base Shear for Bare Frame and Frame with HPCD

B. Roof Acceleration

In order to control the inertia force, it is desired that the acceleration be controlled. However, in this analysis it was observed that there was a slight increase in acceleration. The weakness of the HPCD system is illustrated with the roof acceleration response. The increased stiffness results in higher accelerations. The roof acceleration in the HPCD frame was increased by 2.36 percent. The acceleration values for the 2 frames are tabulated below:

FRAME	BARE	HPCD
ACCELERATION (m/s^2)	28.29567	28.97733

Table.6: IDA Results for Roof Acceleration

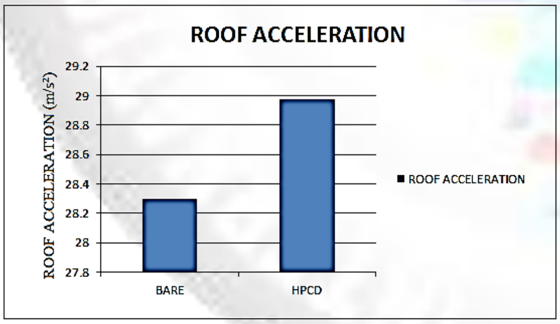


Fig.9: Graph of Roof Acceleration for Bare Frame and Frame with HPCD

C. Joint Displacement

The joint displacement tabulated below is the maximum displacement of a joint on a storey. This displacement is measured with reference to the foundation. In the HPCD frame the lateral stiffness is increased by the addition of high damping rubber system and buckling restrained brace, thus keeping the joint displacement in control. The graphs are curved in nature. The non-linear behaviour of the modelled frame is evident from the form the graph assumes.

STOREY	BARE (mm)	HPCD (mm)
0 (Foundation)	0.0000	0.0000
1	1.7011	0.6714
2	16.2330	3.9930
3	37.1431	6.6520
4	59.3460	9.9868
5	81.0288	13.3474
6	100.6693	16.3485
7	118.4301	19.0732
8	133.9945	23.7381
9	145.4413	27.3684
10 (ROOF)	152.9057	28.9948

Table.7: IDA Results for Joint Displacement

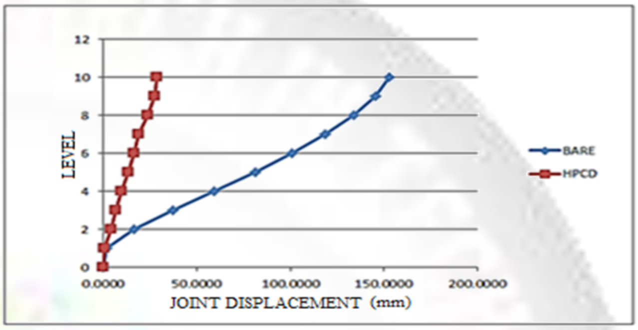


Fig.10: Graph of Joint Displacement for Bare Frame and Frame with HPCD

D. Storey Drift

STOREY	BARE (mm)	HPCD (mm)
1	1.7011	0.6714
2	14.5318	3.3215
3	20.9101	2.6591
4	22.2029	3.3347
5	21.6828	3.3607
6	19.6405	3.0011
7	17.7607	2.7247
8	15.5644	4.6649
9	11.4468	3.6302
10	7.4644	1.6265

Table.8: IDA Results for Storey Drift

Storey drift is the measure of relative displacements between two storeys. The values below are obtained by calculating the difference between joint displacements of two storeys. It is observed that the storey drift has reduced considerably in the HPCD frame. This is mainly due to two reasons. First, is due to the increase in the lateral stiffness and secondly, due to rubber properties. This can be explained by the rubber dampers acting as a restoring force and having the capacity for large elastic deformation.

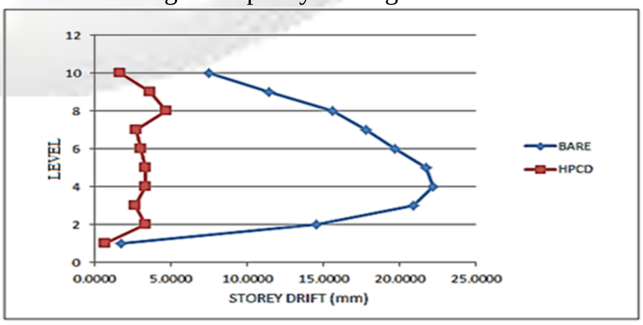


Fig.11: Graph of Storey Drift for Bare Frame and Frame with HPCD

E. Roof Drift

FRAME	BARE	HPCD
ROOF DRIFT (mm)	152.9057	28.9948

Table.9: IDA Results for Roof Drift

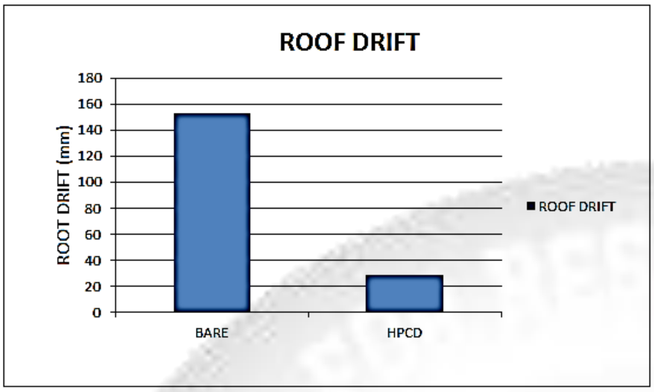


Fig.12: Graph of Roof Drift for Bare Frame and Frame with HPCD

Roof drift is the maximum displacement of a joint on the roof with respect to the ground. The maximum permissible drift should be 0.004 times the storey height.

$$Maximum\ permissible\ drift = 0.004 \times H$$
$$Where, H = Height\ of\ Structure = 36m$$
$$\therefore Maximum\ permissible\ drift = 0.004 \times 36$$
$$= 144m$$

The roof drift for the two frames is tabulated below. It is seen that the roof drift in the bare frame exceeds the maximum permissible drift. A target displacement of 30mm was to be obtained using HPCD. The drift obtained is 29mm. This could be achieved by increasing the overall stiffness of the frame.

IV. CONCLUSION

Two frames were analysed and designed. The first frame is a Bare frame without passive energy dissipating devices. Exceptionally poor performance occurred, with the structure essentially failing for some records. The addition of passive energy dissipation devices showed marked improvement in the structural response. The parameters of concern were base shear, roof acceleration, joint displacement, inter-storey drift and roof drift.

Even with the increased stiffness, reduced base shears were reported in frame with HPCD compared to the bare frame. A 15 percent reduction in base shear was observed. Lower base shear would mean smaller sizes of foundation. Thus, economy can be achieved in foundation design. The weakness in the HPCD frame was that the acceleration was greater than that in bare frame.

The joint displacements in the HPCD frame were observed to have decreased by 79 percent on average. The inter-storey drift also was reduced by 70 percent. A roof drift of 29 mm was achieved (target displacement was 30mm) by varying the stiffness of the springs along the height of the frame. Thus, by reducing the displacements and drifts the damage to non-structures and property can be minimised.

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