Performance Based Seismic Assessment of Masonry Infilled RCC Building with Diaphragm Discontinuity

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Abstract: *In multistoreyed RCC framed buildings, critical damages are due to seismic ground excitations, which cause catastrophic failuresat the weaker locations. Buildings with two types of structural irregularities namely diaphragm discontinuity and open ground story are considered. Assessment of seismic vulnerability of these buildings is done by using Nonlinear Static Pushover Analysis (NSPA). Performance Based Seismic Design of masonry infilled RCC buildings with two different shape of openings in the diaphragm is considered here with Design Basis Earthquake(DBE) and Maximum Considered Earthquake(MCE) where by selecting appropriate performance criteria in terms of Inter-story drift ratio(IDR) and Inelastic displacement demand ratio(IDDR) are critically observed. The Equivalent Linearization Procedure of Pushover analysis presented in FEMA 440, which is a modification of Capacity Spectrum Method based on ATC-40 guidelines, is performed in ETABS-2016 to study the performance of R.C.C. buildings with diaphragm discontinuity, designed as per IS-1893-2016.*

Keywords: Diaphragm Discontinuity, Infill Walls, Performance Objectives, Pushover Analysis, RCC Buildings

I. INTRODUCTION

Diaphragm is a structural system which is used to transfer the major lateral loads to shear walls or frames primarily through in-plane shear mechanism. Openings in floor diaphragms reduces the in-plane rigidity of the diaphragm thus it affects the distribution of lateral loads to the load resisting elements, which eventually causes stress concentration near discontinued joints, which significantly changes the overall dynamic behaviour of the structure. As per IS-1893:2016, a diaphragm is said to have discontinuity in their in-plane stiffness, when the floor slabs have a cut-outs or openings of area more than 50 percent of the full area of the floor slab. Pushover analysis is an approximate analysis technique in which the building structure is subjected to monotonically increasing lateral loads with an invariant height-wise distribution until a target displacement is reached or the structure becomes unstable. It also consists of a series of sequential elastic analysis, superimposed to approximate a force-displacement curve of the overall structure, which iscalled the capacity curve. The demand spectrum curve is normally estimated by reducing the standard elastic by 5% damped design spectrum by spectral reduction method. The intersection of the capacity curve and the demand spectra is referred to as the "Performance Point" of the structure as it forms the basis for assessing the performance of the structure by using certain explicit and quantifiable performance criterion. Pushover analysis procedure can be performed either under force-controlled or deformation-controlledmechanism [4]. In force-controlled pushover procedure, the full load combination is applied as specified in code only when the load is known (such as Gravity loading). In force-controlled pushover procedure some numerical error may affect the accuracy of overall results as target displacement may be associated with minor value of lateral stiffness which is developed due to mechanisms and P-delta effects. This method allows us to trace the sequence of yielding and failures of structural members along with their performance levels as well as the progress of the overall capacity curve of the structure.

The advantage of Performance Based Seismic Engineering (PBSE) is the possibility of achieving predictable seismic performance with uniform risk[8]. Performance based design begins with the selection of acceptability criteria stated in the form of one or more performance objectives. Each performance objective is essentially a statement of acceptable risk of incurring specific levels of damages i.e., structural and nonstructural due to which the consequent socio-economic losses that occur as a result of these damages, under a given seismic hazard level. The Global losses can be evaluated in the form of life-loss, direct economic costs or down-time (time for restoration) which is developed due to seismic damages. The procedure for estimating these losses is the central core crux of Performance Based Design (PBD) [8]. A series of structural simulations are performed and response of the building structure to loading is systematically worked out to estimate its probable behaviour under various possible design scenario events. If the performance of simulated model meets or exceeds the predefined performance objectives, the design is complete, or else the designed is revised in an iterative manner until the performance objectives are achieved.

Dr. G. S. Doiphode*, Applied Mechanics & Structural Engineering Department, Faculty of Technology & Engineering. The Maharaja Sayajirao University of Baroda, Vadodara, Gujarat-State, India. Email :gsdoiphode-appmech@msubaroda.ac.in

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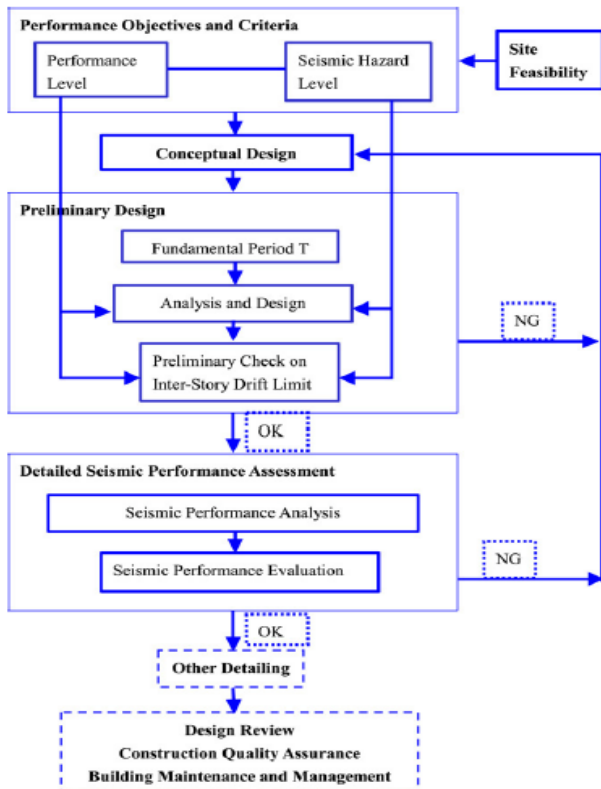Fig.1 shows here the necessary diagrammatic presentation which highlights the basic procedure of PBD

Fig. 1. Performed Based Designflow-chart for new buildings

Table – 1: Performance levels for a building

Performance Level	Characteristics
Operational	Very light overall damage, no permanent drift, structure substantially retains original strength and stiffness. Minor cracks are visible. All important systems remain functional.
Immediate Occupancy	Light overall damage, no permanent drift, structure substantially retains original strength and stiffness. Minor cracks visible. Elevators can be restarted. Fire protection operable.
Life Safety	Moderate overall damage, some permanent drift, some residual strength and stiffness left in all stories. Gravity load bearing elements function. Walls and parapets remain in-plane. Damage to partitions. Building may be beyond economical repair.
Collapse Prevention	Severe overall damage, large permanent drifts, little residual strength and stiffness. Load bearing columns and wall function. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.

The performance level can be assessed by using the displacement demand at the performance point. The adequacy of the structure can be checked by comparing the response parameters with the acceptable limits for various performance levels. Performance levels are associated with earthquake hazard and design levels.Each performance level is quantified by parameters associated with strength, stiffness and ductility respectively. Regarding the strength, OP (Operational level) corresponds to elastic behaviour. Over-strength must be ensured for other each performance levels and no strength degradation can occur beyond the ductility limit. No weak story exists and the structure has enough vertical capacity. Regarding ductility, the concept of Inelastic Displacement Demand Ratio(IDDR) [5] is employed. IDDR represents the ratio of inelastic displacement demand over the ultimate inelastic displacement capacity. Acceptable values of IDDR associated with structural system performance levels OP,IO,DC,LS and CP are 0,0.2,0.4,0.6 and 0.8, respectively.

Table-II: Allowable Inelastic Displacement Demand Ratio (IDDR)

Performance Level	OP	IO	DC	LS	CP
IDDR Value	0	0.2	0.4	0.6	0.8

Regarding stiffness, the maximum Inter-story Drift Ratio(IDR) is considered to limit building lateral displacement. In this research, based on references like FEMA 356[5], ATC 40[4], SEAOC blue book [7] and other literatures, the IDR limits in Table III are preliminarily suggested. Structural systems are mainly classified into four types namely, load-bearing walls, the frame systems, the moment resisting frames and the dual systems.

Table -III: Allowable Inter-Story Drift Ratio (IDR)

Structural system	Performance Level				
	OP	IO	DC	LS	CP
System with masonry shear walls	0.005	0.007	0.007	0.007	0.009
Other Systems	0.005	0.01	0.015	0.02	0.025

II. STRUCTURAL MODELLING

The (5 x 5) bays moment resisting frame (G+6 floors) is modelled by using commercial code ETABS. Gravity loads comprising of Dead load (DL) and Live load (LL) for sections were applied to slab and beam elements. Initially, static linear analysis is performed by using only gravity loads by using appropriate reduction factor for live loads with the combining lateral loads applied to the frame as per IS:1893-2016 [1]. The influence of non-structural walls i.e., masonry infills damage mechanism and deformation capacity are considered for all the buildings.

A. Masonry Modelling

The infill walls are modelled using Multi-linear plastic link elements, which can be used for both the linear as well as non-linear static analysis.

For linear analysis, the effective stiffness for the spring was calculated using the stiffness for equivalent compression diagonal strut, the dimensions of which can be calculated by using the equations given in IS:1893-2016. The width of strut is calculated from,

$$Wds = 0.175 * \alpha h^{-0.4} * L_{ds} \quad (1)$$

Where,

$$\alpha h = hcol(\sqrt[4]{(Em * t * sin2\theta)/(4 * Ef * Ic * hinf)})$$

In which

, h_{col} = height of column,
E_m = Modulus of elasticity of masonry,
L_{ds} = Length of diagonal strut
, E_f = Modulus of elasticity of frame,
I_c = M.O.I. of columns beside infill,
h_{inf} = height of infill,
$t = t_w$ = thickness of strut = thickness of infill wall.
L_{inf} = Length of infilled wall
So, linear stiffness = AE_m/L_{ds}, with A = W_{ds}*t

For non-linear behaviour of infill wall, the force deformation curve is supplied for the multi-linear plastic link element. The force-deformation relationship is obtained from the equations given by Dolsek and Fajfar [09], which are mentioned below,

i. Initial stiffness:

$$Ke = \frac{Gw * tw * Linf}{hinf}$$

ii. Maximum force:

$$Fmax = 0.818 * ftp * Lw * tw * B \quad (2)$$

Where B = $(1 + (C_1^2 + 1)^2) / C_1$ with
$C_1 = 1.925 * Linf/hinf$

iii. Cracking force and Residual force:
$Fcr = 0.6*Fmax$ and $Fr = 0.2*Fmax$

iv. Deformations:
$Dcr = Fcr/Ke$, $Dm = 0.2\%$ strain, $Dult = 5*Dm$
Table IV shows the values of properties of the masonry wall are tabulated below

Table– IV: Masonry properties

Infill wall length, L_{inf}	4150 mm
Infill wall height, h_w	2650 mm
Thickness, t_w	230 mm
Mortar Strength, f_{mo}	5.7 Mpa
Masonry Strength, f_m	3.65 Mpa
Elastic modulus, E_m	2007.5 Mpa
Crushing Strength, f_{tp}	2.5 Mpa
Shear Modulus, G_w	836.45 Mpa

Fig.2 depicts the force-deformation curve for masonry infill wall modelling which is used for further calculation.
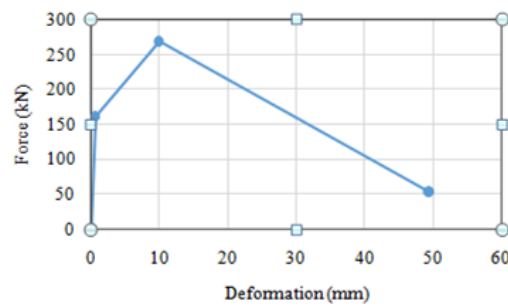

Fig. 2. Force-Deformation curve for Infilled wall

For performing pushover analysis on the space frame, three pushover load cases are to be defined. The first load case is 'Push Down' which applies Gravity Loads (force controlled case) and it is followed by two lateral load cases 'PUSH X' and 'PUSH Y'(displacement controlled cases) for each of the two orthogonal directions of the building respectively. The method of iteration used in the pushover analysis is the Newton-Raphson method. The Gravity load combination comprises of'DL+0.25LL' and for the two lateral load, load patterns from equivalent static analyses 'EQ X' and 'EQ Y' have been used to push the structure.

B. Non-Linear Plastic Hinge

Under seismic loads, plastic hinges are usually formed at the junctions of the beams and columns in the framed structures. For the beam elements, the plastic hinges are mostly caused due to uniaxial bending moments, whereas for the column elements, the plastic hinges are mostly developed due to the interaction between axial loads and biaxial bending moments. Hence, to account for material nonlinearity in the pushover analysis, different types of plastic hinges should be applied for beams and columns each.In ETABS, hinge properties can be assigned to the members using either user-defined or default hinge properties which are based on FEMA-356 and Asce-41 criteria. The default M3 hingesareapplied at both the ends of beams to simulate the plastic hinges caused by uniaxial moment, and the default P-M2-M3 hinges are applied at both the ends of columns to simulate the plastic hinges caused by axial loads and biaxial bending moments.

C. Problem Statement

Total 6 types of buildings are considered here to validate numerically the technical theme. It is necessary to know the influence of diaphragm discontinuity and masonry infills as non-structural elements on the non-linear response parameters, and based on those, performance based seismic assessment is carried out until the predefined performance criteria in terms of IDR and IDDR are satisfied. Model I representsregular building as bare frame, Model II represents building with central opening in "+" Shape in the diaphragm and Model III represents "E" Shaped building in Plan.Models IV, V and VI represent the same three buildings respectively incorporating the infill walls on external periphery. The prototype buildings are G+6 reinforced concrete buildings consisting of five bays in both orthogonal directions.

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The spacing along X and Y directions is 4.5m and the story height is 3m. Hence the overall plan dimension of the building is 22.5m x 22.5m.The primary beam dimension is taken as 230mm x 350mm, column size as 375mm x 375mm (For Model I), 350mm x 350mm (For models II, III, IV, V and VI) with fixed support at base respectively. The slab thickness is considered to be 130mm constant. To achieve the required performance objectives, sizes of beams and columns are increased until the performance meets or exceeds the predefined objectives. The building design data with plan and elevation details are shown below. Building frame with and without infilled also shown in Fig. 4 while Fig. 5 depicts rendered view of same building

Fig. 4. Discontinuous Diaphragm + shape (Model 2)

Table– V: Building Design Data

Live Load	3.0 kN/m^2 on typical floor
	1.5 kN/m^2 on terrace
Floor Finishing	1.0 kN/m^2
Terrace Finishing	2.0 kN/m^2
Water Proofing	1.0 kN/m^2
Wall Load	230 mm thick masonry walls only at the exterior periphery
Masonry wall density	20 kN/m^3
Design Seismic Load	As per IS:1893(Part 1)
Slab thickness	130 mm
Zone factor, Z	0.24
Importance factor, I	1
Response reduction factor, R	5
Soil Type	Type II, Medium as per IS:1893
Concrete Grade	M30
Steel Grade	Fe 415 (HYSD Bars)

Fig.5. Discontinuous Diaphragm E shape (Model 3)

Fig.6. Bare & Infilled Frame

Fig. 3. Continuous Diaphragm (Model 1)

Fig.7. Rendered View Building Frame

The total six frames are analyzed using ETABS 2016 commercial code for various aspects. Various graphs are drawn and necessary results are arranged in tabular format are presented here.

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III. RESULTS AND DISCUSSIONS

The performance point of the complete building frame responds to the considered hazard level which is evaluated through the Capacity Spectrum Method by using Equivalent Linearization technique [4],[5]. The structure is laterally pushed again to the target displacement associated with the performance point to assess the behaviour of both the structural system and its resisting structural and non structural elements, if such a hazard level occurs.
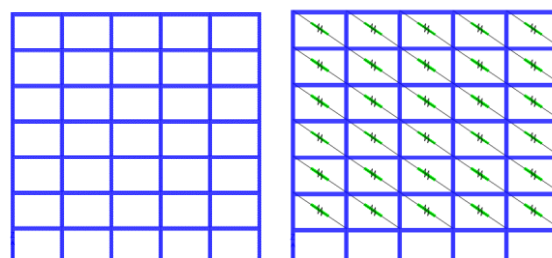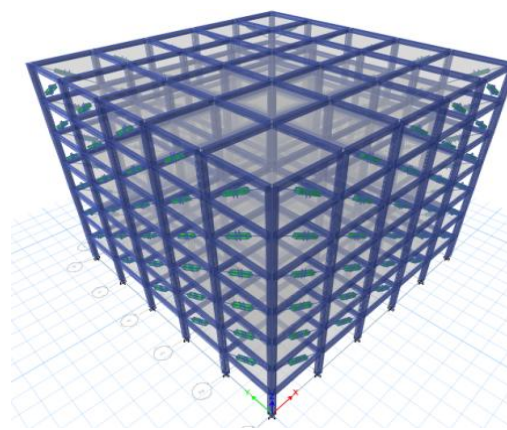
Table – VI: System Performance Evaluation regarding IDR & IDDR for Model 1

Type	Regular Building-Bare Frame			
Performance Obj.	Basic Safety Obj.		Enhanced Safety Obj.	
Seismic Hazard Level	DBE	MCE	DBE	MCE
Optimum Section dimensions (mm*mm)	Col: (375X375), Beam: (230X350)		Col: (400X400), Beam: (230X350)	
IDRa	1.5	2.5	1	2
IDR (%)	1.03	1.876	0.994	1.753
Check if IDR≤IDRa	Yes	Yes	Yes	Yes
IDDRa	0.4	0.8	0.2	0.6
IDDR	0.199	0.761	0.149	0.55
Check if IDDR≤IDDRa	Yes	Yes	Yes	Yes

Table – VII: System Performance Evaluation regarding IDR & IDDR for Model 2

Type	'+' Shaped Open-Bare Frame			
Performance Obj.	Basic Safety Obj.		Enhanced Safety Obj.	
Seismic Hazard Level	DBE	MCE	DBE	MCE
Optimum Section dimensions (mm*mm)	Col: (400X400), Beam: (230X350)		Col: (450X450), Beam: (230X350)	
IDRa	1.5	2.5	1	2
IDR (%)	0.95	1.67	0.872	1.566
Check if IDR≤IDRa	Yes	Yes	Yes	Yes
IDDRa	0.4	0.8	0.2	0.6
IDDR	0.216	0.76	0.129	0.47
Check if IDDR≤IDDRa	Yes	Yes	Yes	Yes

Table – VIII : System Performance Evaluation regarding IDR & IDDR for Model 3

Type	E Shaped Building-Bare Frame			
Performance Obj.	Basic Safety Obj.		Enhanced Safety Obj.	
Seismic Hazard Level	DBE	MCE	DBE	MCE
Optimum Section dimensions (mm*mm)	Col: (400X400), Beam: (230X350)		Col: (450X450), Beam: (230X350)	
IDRa	1.5	2.5	1	2
IDR-X (%)	0.896	1.61	0.834	1.487
IDR-Y (%)	0.997	1.714	0.921	1.569
Check if IDR≤IDRa	Yes	Yes	Yes	Yes
IDDRa	0.4	0.8	0.2	0.6
IDDR-X	0.159	0.54	0.114	0.39
IDDR-Y	0.199	0.72	0.128	0.45
Check if IDDR≤IDDRa	Yes	Yes	Yes	Yes

Table – IX : System Performance Evaluation regarding IDR & IDDR for Model 4

Type	Regular Building-Infilled Frame			
Performance Obj.	Basic Safety Obj.		Enhanced Safety Obj.	
Seismic Hazard Level	DBE	MCE	DBE	MCE
Optimum Section dimensions (mm*mm)	Col: (350X350), Beam: (230X350)		Col: (350X350), Beam: (230X350)	
IDRa	1.5	2.5	1	2
IDR (%)	0.988	1.877	0.988	1.877
Check if IDR≤IDRa	Yes	Yes	Yes	Yes
IDDRa	0.4	0.8	0.2	0.6
IDDR	0.107	0.52	0.107	0.52
Check if IDDR≤IDDRa	Yes	Yes	Yes	Yes

Table – X: System Performance Evaluation regarding IDR & IDDR for Model 5

Type	'+' Shaped Open-Infilled Frame			
Performance Obj.	Basic Safety Obj.		Enhanced Safety Obj.	
Seismic Hazard Level	DBE	MCE	DBE	MCE
Optimum Section dimensions (mm*mm)	Col: (400X400), Beam: (230X350)		Col: (500X500), Beam: (230X350)	
IDRa	1.5	2.5	1	2
IDR (%)	0.737	1.38	0.624	1.236
Check if IDR≤IDRa	Yes	Yes	Yes	Yes
IDDRa	0.4	0.8	0.2	0.6
IDDR	0.22	0.72	0.182	0.58
Check if IDDR≤IDDRa	Yes	Yes	Yes	Yes

Table – XI: System Performance Evaluation regarding IDR & IDDR for Model 6

Type	E Shaped Building-Infilled Frame			
Performance Obj.	Basic Safety Obj.		Enhanced Safety Obj.	
Seismic Hazard Level	DBE	MCE	DBE	MCE
Optimum Section dimensions (mm*mm)	Col: (400X400), Beam: (230X350)		Col: (450X450), Beam: (230X350)	
IDRa	1.5	2.5	1	2
IDR-X (%)	0.707	1.318	0.662	1.208
IDR-Y (%)	0.82	1.463	0.748	1.34
Check if IDR≤IDRa	Yes	Yes	Yes	Yes
IDDRa	0.4	0.8	0.2	0.6
IDDR-X	0.249	0.789	0.177	0.582
IDDR-Y	0.267	0.799	0.176	0.557
Check if IDDR≤IDDRa	Yes	Yes	Yes	Yes

The results highlight that when there is discontinuity in the diaphragm the base shear capacity of the building is considerably lowered. Hence, to achieve the same performance objectives, models with opening require higher sections for 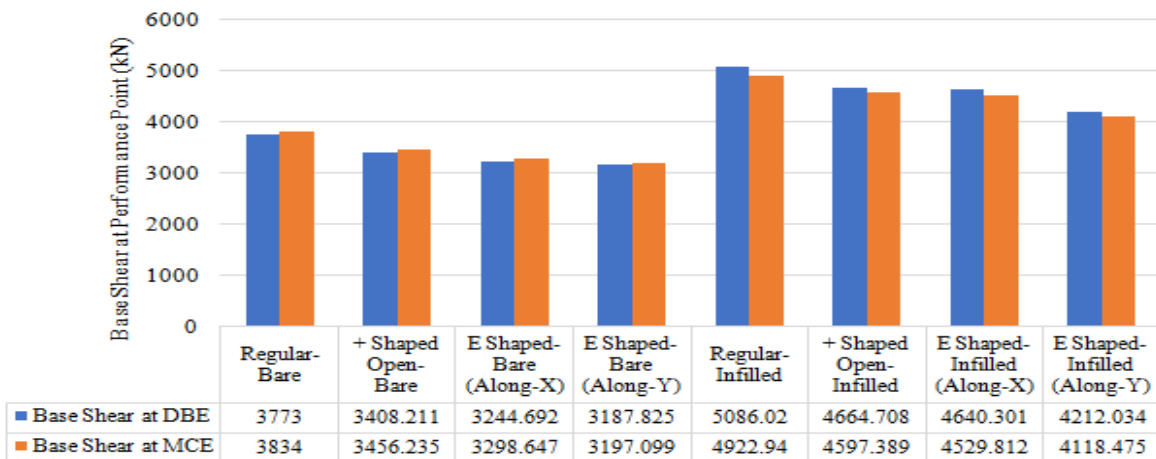Columns compared to those of models without openings in diaphragm. The infill walls on the other hand increase the initial stiffness of the structure and reduce its ultimate deformation capacity under lateral loads. The following charts show the comparison of base shear and roof displacements at DBE and MCE hazard levels among all the considered models.

	Regular-Bare	+ Shaped Open-Bare	E Shaped-Bare (Along-X)	E Shaped-Bare (Along-Y)	Regular-Infilled	+ Shaped Open-Infilled	E Shaped-Infilled (Along-X)	E Shaped-Infilled (Along-Y)
Base Shear at DBE	3773	3408.211	3244.692	3187.825	5086.02	4664.708	4640.301	4212.034
Base Shear at MCE	3834	3456.235	3298.647	3197.099	4922.94	4597.389	4529.812	4118.475

Fig. 8. Base Shear at Performance point for Basic Safety

	Regular-Bare	+ Shaped Open-Bare	E Shaped-Bare (Along-X)	E Shaped-Bare (Along-Y)	Regular-Infilled	+ Shaped Open-Infilled	E Shaped-Infilled (Along-X)	E Shaped-Infilled (Along-Y)
Displacement at DBE	155.8	143.225	133.365	147.204	120.578	93.838	88.225	106.857
Displacement at MCE	281.9	264.311	256.439	274.284	186.429	148.264	140.605	163.677

Fig. 9. Displacement at performance point for Basic Safety

	Regular-Bare	+ Shaped Open-Bare	E Shaped-Bare (Along-X)	E Shaped-Bare (Along-Y)	Regular-Infilled	+ Shaped Open-Infilled	E Shaped-Infilled (Along-X)	E Shaped-Infilled (Along-Y)
Base Shear at DBE	3834	3513.19	3401.312	3319.921	5086.02	5076.219	4898.201	4440.537
Base Shear at MCE	3879.1	3491.605	3398.59	3283.628	4922.94	5082.491	4824.879	4368.584

Fig. 10. Base Shear at performance point for Enhanced Safety

	Regular-Bare	+ Shaped Open-Bare	E Shaped-Bare (Along-X)	E Shaped-Bare (Along-Y)	Regular-Infilled	+ Shaped Open-Infilled	E Shaped-Infilled (Along-X)	E Shaped-Infilled (Along-Y)
Displacement at DBE	150.143	134.533	127.244	139.969	120.578	85.945	85.381	102.293
Displacement at MCE	273.712	266.638	256.185	268.834	186.429	139.852	138.167	162.282

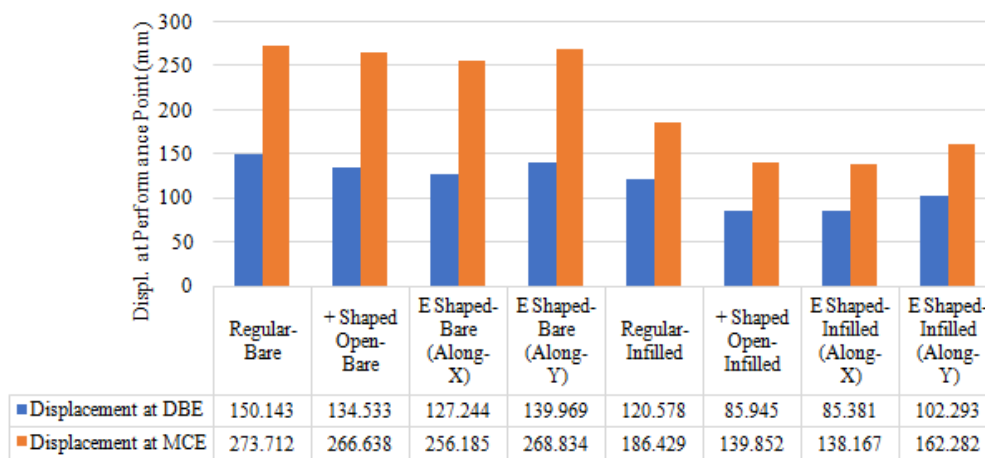**Fig. 11. Displacement at performance point to satisfy the Enhanced Safety**

IV. CONCLUSIONS

Performance based seismic assessment for (G+6) storeyed infilled buildings with and without diaphragm discontinuity is carried out by pushover analysis. The influence of varying the member dimensions on the performance of the structures was investigated. The following are the salient conclusions that were drawn.

1) Performance of the building improves on increasing the lateral stiffness of column. Thus, byincreasing the beam dimension, the ultimate drift of the pushover curve decreases.

2) Results shows that to obtain the same performance level in terms of parameters IDR and IDDR, higher column size for structural members is required for opening in plan.

3) The base shear capacity and the ultimate displacement reduces in the building with diaphragm discontinuity as compared to the regular building.

4) The pushover curve for models with masonry infills is characterised by high initial stiffness and low deformability.The base shear for all the three models with infills increases by about 30 to 35 percent and the ultimate displacement decreases by 35 to 40 percent as compared to corresponding values for those as bare frame respectively. The infills have an advantageous influence on the Model I while they have an adverse influence on the Model II and Model III due to formation of weak story at ground floor.

5) The hinge formations in bare frame buildings is distributed throughout the structure, whereas in case of buildings with infills the hinges are formed only in several lower stories before the building becomes unstable.

6) The variation in the base shear of the building from performance point at DBE to MCE is very small, whereas the increase in the displacement for the same is very large, which indicates that the buildings behave in non-linear range beyond DBE level of hazard.

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