Seismic Performance Evaluation of Torsionally Asymmetric Buildings

Dr. B.G. Naresh Kumar* and Avinash Gornale**

Abstract-In present scenario, most of the buildings are often constructed with irregularities such as soft storey, torsional irregularity, unsymmetrical layout of in-fill walls, vertical and plan irregularity, etc. Past earthquake studies shows that the most of the RC buildings having such irregularities were severely damaged under the seismic ground motion. This paper presents an overview of performance of the torsionally balanced and unbalanced buildings also called as symmetric and asymmetric buildings subjecting to pushover analysis. The buildings have un-symmetric distribution of stiffness in storeys. In this the study the effect of eccentricity between centre of mass (CM) and centre of story stiffness (CR) and the effect of stiffness of in-fill walls on the performance of the building is presented. The performance of the buildings is assessed as per the procedure prescribed in ATC-40 and FEMA 273.

Index Terms- Asymmetric structure, Pushover Analysis, Seismic Performance.

1. INTRODUCTION

Many buildings in the present day scenario have irregular configurations in the both plan and elevation. Buildings with asymmetrical distribution of stiffness, mass and strength suffer severe damage during earthquakes. This has been observed in the previous earthquakes. Such buildings undergo torsional motions. An ideal multistory building designed to resist lateral loads due to earthquake would consist of only symmetric distribution of mass and stiffness in plan at every storey and a uniform distribution along height of the building. Such a building would respond only laterally and is considered as torsionally balanced (TB) building. But it is very difficult to achieve such a condition because of restrictions such as architectural requirements and functional needs.

2. REVIEW OF LITERATURE

Structural irregularities in the building can be classified in to horizontal and vertical irregularity. One major cause of vertical irregularity is the abrupt change of mass and (or) stiffness distribution along the height. One major cause of horizontal irregularity is torsion caused by uneven distribution of mass and stiffness in plan.

* Dr. B.G. Naresh Kumar, Department of Civil Engineering, Maharaja Institute of Technology, Mysore (KS), Visvesvaraya Technological University, Belgaum, –India. Email- nareshgowda58@gmail.com
** Avinash Gornale, Department of Civil Engineering, Guru Nanak Dev Engineering College, Bibar (KS), Visvesvaraya Technological University, Belgaum, –India. Email- avinash048@yahoo.co.in

The non-uniformity in the definition of centre of rigidity among codes was taken by Tso [1990] as the subject of research and classified static eccentricity as floor eccentricity and storey eccentricity. These two approaches of calculating eccentricity has been detailed and equivalence of these two approaches of calculating eccentricity has been established.

He showed that though these two approaches result in two different value of eccentricity, they both lead to the same value of torsional moment distribution; however, these two eccentricities are dependent on the structure and lateral load distribution.

Goel and chopra [1993] proposed an analysis approach which eliminates the need for explicit computation of the centre of rigidities and yet leads to results identical to those obtained by the approach which involves calculating the location of centre of rigidity. The results from the static analysis for three sets of forces applied at the centre of mass are combined appropriately to determine the design forces. With the mathematical proof of the proposed approach, they also established its equivalence with the standard approach with the help of an example building.

Jain and Annigeri [1995] took four different types of buildings having different types of irregularities. They conducted both equivalent static approach and dynamic analysis using the torsional provisions given in IS: 1893-1984 and NBCC 1990 codes. They showed that the estimation of base shear and frame shear by the NBCC code varies depending on the irregularities of the building. They concluded that static method do not account for higher modes. The static method may be used for preliminary design of irregular buildings. For the final design, dynamic analysis must be carried out.

Aziminejad and Moghadam [2009] observed that the experience from the performance of buildings during past earthquakes has shown that asymmetric buildings often sustain more extensive damage as compared to symmetric buildings. Performance of an asymmetric building can be quantified by responses such as rotation of the floors, the maximum drift of flexible and stiff edges of the buildings or the ductility demand of the elements on the edges.

3. NATURE OF PROBLEM AND BUILDING CODE PROVISION

The fundamental natural period of vibration of a building is given by empirical formulas which depend on the height of the building and base dimensions of the structure. It also states that a free vibration analysis may be performed as per established methods to obtain the natural periods of the structure. The analysis is made to obtain seismic force and their distribution to different levels along height of the building and to various lateral load resisting elements, depending n the height of the building, severity of the seismic zone in which the building is located and on the classification of the building as regular or irregular.

SEISMIC ANALYSIS PROCEDURES

It is recognized from design philosophy that the complete protection against earthquake of all magnitude is not economically feasible and design based alone on strength criteria is not justified. The basic design criteria of earthquake resistant design should be based on lateral strength as well as deformability and ductility capacity of structure with limited damage but not collapse. The procedures to determine lateral forces in the code, IS 1893 (Part 1): 2002 are based on the approximation effects, yielding can be accounted for linear analysis of the building using the design spectrum. This analysis is carried out either by modal analysis procedure or dynamic analysis procedure. A simplified method may also be adopted that will be referred as lateral force procedure or equivalent static procedure. The main difference between the equivalent static procedure and dynamic analysis procedure lies in the magnitude and distribution of lateral forces over the height of the buildings. In the dynamic analysis procedure the lateral forces are based on properties of the natural vibration modes of the building, which are determined by distribution of mass and stiffness over height. In the equivalent lateral force procedure the magnitude of forces is based on an estimation of the fundamental period and on the distribution of forces as given by a simple formula that is appropriate only for regular buildings. The following sections will discuss in detail the above-mentioned equivalent static and the dynamic procedure to determine the design lateral forces in detail.

LINEAR STATIC ANALYSIS

The total design lateral force or design base shear along any principle direction is given in terms of design horizontal seismic coefficient seismic weight of the structure. Design horizontal seismic coefficient depends on the zone factor of the site, importance of the structure, response reduction factor of the lateral load resisting elements and the fundamental period of the structure.

$$V_B = A_h \times W$$
$$\text{Where, } A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$$

Where Z is the zone factor, I is the importance factor, R is the response reduction factor and S_a/g is the average response acceleration coefficient which depends on the nature of foundation soil (rock or soil site), natural period and the damping of the structure. The design base shear V_b computed is then distributed along the height of the structure using a parabolic distribution expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_i h_i^2}$$

Where Q_i is design lateral force, W_i is seismic weight, $h_i$is height of the i^{th} floor measured from base and n is the number of stories in the building.

LINEAR DYNAMIC ANALYSIS

According to the code, dynamic analysis may be performed either using response spectrum method or time history method. In either method, the design base shear (V_B) is compared with a base shear ($\overline{V_B}$) calculated the fundamental natural periodT_a. It suggests that when V_B is less than$\overline{V_B}$, all response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) must be suitably scaled by multiplying with$\overline{V_B}/V_B$.

NON-LINEAR STATIC ANALYSIS

Current advances in earthquake engineering favors performance based approach for the seismic design of new structures and for the assessment and rehabilitation of existing structures located in active seismic zones. Typically a performance objective is defined when a set of structural and non-structural performance levels, representing losses and repair costs, are coupled with different intensities of seismic input. The performance of a structure typically assessed on the basis of maximum deformation and for cumulative inelastic energy absorbed during earthquake. Report from past earthquake reconnaissance observations indicate that most structures designed according to current codes will sustain residual deformations in the event of a design-level earthquake, if they perform exactly as expected.

The major challenge to performance-based seismic design and engineering of building is to develop, yet effective methods for designing, analysis and checking the design of structures so that they reliably meet the selected performance objectives and there is a need for analysis procedures, which are capable of predicting the demands, force and deformation imposed on structures more realistically. In response to this need, simplified Non-Lineal Static Pushover analysis procedure to determine the displacement demand imposed on a building have been incorporated in Applied Technology Council (ATC-40) and Federal Emergency Management Agency (FEMA-273).

Pushover analysis is one of the methods available to understand the behavior of structures subjected to earthquake forces. As the name implies, it is the process of pushing horizontally with a prescribed loading pattern incrementally until the structure reaches a limit state [ATC-40 1996]. The static approximation consists of applying a vertical distribution of lateral loads to a model which captures the material non - linearity of an existing or previously designed structure, and monotonically increasing those loads until the peak response of the structure is obtained on a base shear vs. roof displacement plot.

From the Response Spectrum and Base Shear vs. Roof Displacement plot, the Target Displacement, δt, may be determined. The Target Displacement represents the maximum displacement the structure will undergo during the design event. One can then find the maximum expected deformations within each element of the structure at the Target Displacement and redesign them accordingly. The Target Displacement and Response Spectrum is shown in Figure.1 and 2.

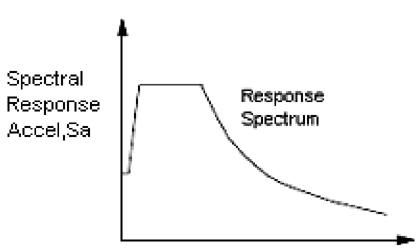

Fig 1: Responce Spectrum for 5% damping

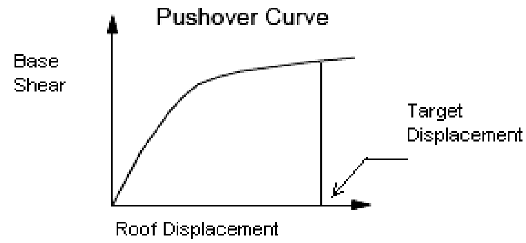

Fig 2: Target Displacement.

4. MODELING AND ANALYSIS

In the present study the gravity load analysis and lateral load analysis as per the seismic code IS 1893 (Part 1): 2002 are carried out for asymmetric buildings and buildings with open ground storey and an effort is made to study the effect of seismic loads on them and their capacity and demand is evaluated using nonlinear static pushover analysis guidelines given in ATC-40.

The plan layout of the reinforced concrete ordinary moment resisting frame building of five storied building without and with consideration of stiffness of walls is as shown in Fig.3, with open ground storey and unreinforced masonry infill walls in the upper storey's are chosen. The bottom storey height is kept 4.5m and a height of 3.2m is kept for all the other storey's, bay dimensions in both directions are kept as 4m. The building is deliberately kept symmetric in both the orthogonal directions in plan to avoid torsional response under pure lateral forces for symmetric buildings and for asymmetric buildings the plan of the building is kept symmetric but one side edge columns are made stiffer than all other columns. This makes the structure torsionaly unbalanced i.e asymmetric. The elevations of the different building models considered are shown in Fig.4 and 5. The masonry infill is modeled as equivalent diagonal strut in the upper storey. Stafford Smith equation for calculation of equivalent diagonal strut width is considered [Agarwal, P. and Shrikhande, M.].

$$\text{effective strut width, } w = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_l^2}$$

$$\alpha_h = \frac{\pi}{2}\sqrt[4]{\frac{4E_f I_c h}{E_m t \sin 2\theta}} \qquad \alpha_l = \pi\sqrt[4]{\frac{4E_f I_b l}{E_m t \sin 2\theta}}$$

Where,

Em and Ef = Elastic modulus of masonry wall and frame material, respectively t, h, l = Thickness, height, and length of infill wall, respectively

Ic, Ib = Moment of inertia of column and beam of the frame, respectively

$$\theta = \tan^{-1}(h/l)$$

Concrete frame elements are classified as beam and column frames. Columns and beams are modeled using three dimensional frame elements. Slabs are modeled as rigid diaphragms. The analytical model of the floor diaphragm represents the strength, stiffness and deformation capacity for in-plane loading. The beam column joints are assumed to be rigid. Default hinge properties available in ETABS as per the ATC-40 are assigned to the frame elements. There are four distinct building models namely.

Model I: The building is symmetric in plan and also in distribution of storey stiffness. Building has no walls in the first storey and one full brick masonry wall in the upper storeys. The building is modeled as bare frame; however the masses of the walls are included and the building is modeled as stiff frame in which stiffness of wall is considered.

Model II: The building is symmetric in plan and asymmetric in distribution of storey stiffness i.e the centre of mass and stiffness are not at the same point. The buildings in model II have 19.12% eccentricity. Building has no walls in the first storey and one full brick masonry wall in the upper storeys. In this model also the building is analyzed with and without considering the Stiffness of walls.

Model III: The building is symmetric in plan and asymmetric in distribution of storey stiffness i.e the centre of mass and stiffness are not at the same point. The buildings in model III have 29.16% eccentricity. Building has no walls in the first storey and one full brick masonry wall in the upper storeys. In this model also the building is analyzed with and without considering the Stiffness of walls.

Model IV: The building is symmetric in plan and asymmetric in distribution of storey stiffness i.e the centre of mass and stiffness are not at the same point. The buildings in model IV have 37.26% eccentricity. Building has no walls in the first storey and one full brick masonry wall in the upper storeys. In this model also the building is analyzed with and without considering the Stiffness of walls.

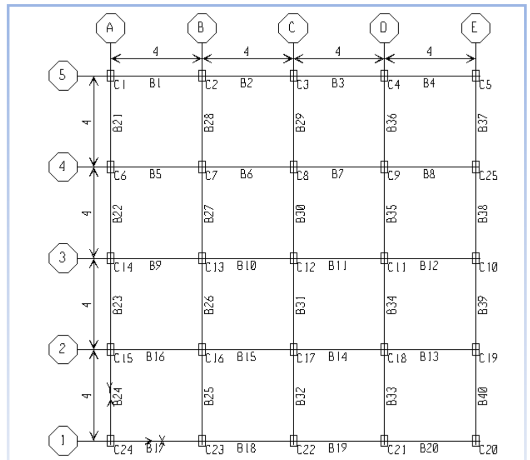

Fig 3 (a): Plan of Symmetric Building (Model I)

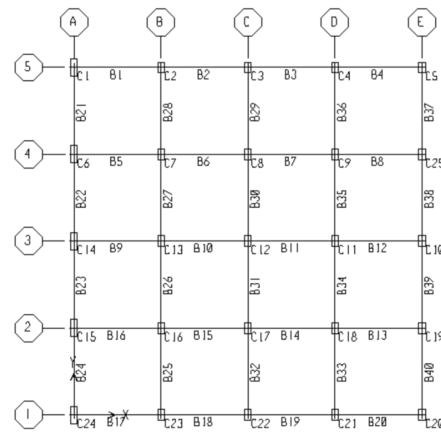

Fig 3 (b): Plan of Asymmetric Building (Model II)

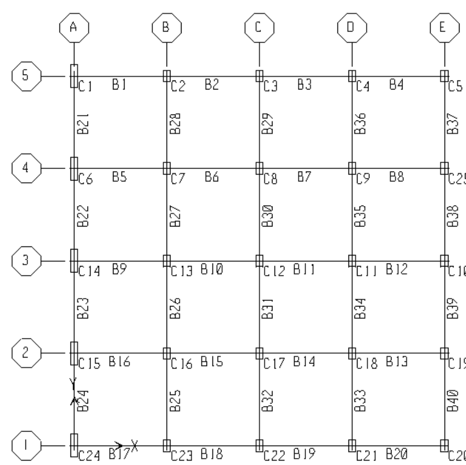

Fig 3 (c): Plan of Asymmetric Building (Model III)

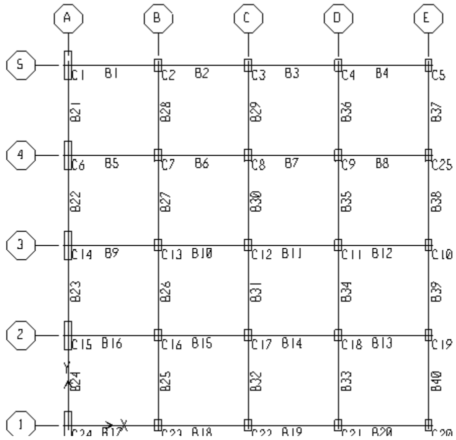

Fig 3 (d): Plan of Asymmetric Building (Model IV)

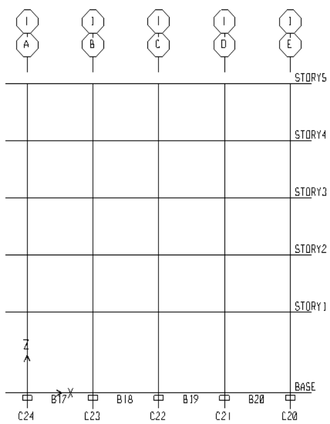

Fig 4 (a): Elevation of models neglecting the stiffness of walls

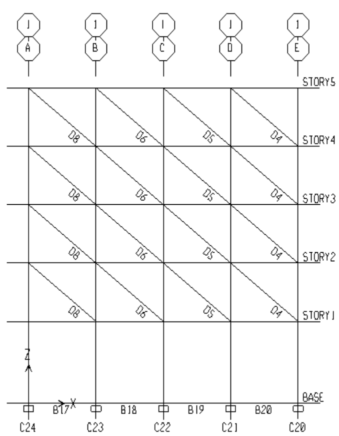

Fig 4 (b): Elevation of models considering the stiffness of walls

5. Results and Discussions

NATURAL PERIODS

The natural periods obtained from seismic code IS:1893 (Part 1)-2000 (referred to as "Codal" in the discussion) and free vibration analysis using ETABS (referred to as "Analysis" in the discussion) are shown in Tables 1. Codal and analytical values are not identical. The natural period computed analytically is higher than that given by codal provisions, for all models.

The analytical natural period depends on the mass and stiffness of each model and is therefore different for models with different amounts of eccentricity and where stiffness of infill walls is considered or ignored. It can be observed that models where stiffness of infill walls is considered (by representing them as equivalent diagonal struts) have significantly lower fundamental natural period as compared to models where stiffness of infill walls ignored. This is to be expected, and is mainly due to the stiffness contribution of the diagonal struts in models where stiffness of infill walls is considered.

From Table 1 it can be seen that the fundamental natural period obtained from analytical approach is 2.37 to 2.48 times higher than those obtained from codal approach, for models where stiffness of infill walls is neglected and 3.12 to 3.34 times higher for models where stiffness of infill walls is considered.

HINGE STATUS AT PERFORMANCE POINT

Performance point determined from pushover analysis is the point at which the capacity of the structure is exactly equal to the demand made on the structure by the seismic load. The performance of the structure is assessed by the state of the structure at performance point. This can be done by studying the status of the plastic hinges formed at different locations in the structure when the structure reaches its performance point. It is therefore important to study the state of hinges in the structure at performance point. The status of hinges at performance points are shown in Tables 2 and 3.

From the data presented in Table 2, the models are subjected to pushover analysis (ESA) by neglecting the stiffness of infill walls; the effect of asymmetry on the status of hinges at performance point can be seen. In these models as the asymmetry increases the numbers of hinges in elastic range are decreasing and numbers of plastic hinges are increasing. But as the performance objective for the building is not fixed, we can say that more the number of hinges at performance point in elastic range and fewer the number of plastic hinges is a better performance.

Type of Structure	Fundamental Natural Periods T (Sec)			
	Neglecting the Stiffness of Walls		Considering the Stiffness of Walls	
	Code	Analysis	Code	Analysis
Symmetric	0.636	1.580	0.389	1.299
Asymmetric 1	0.636	1.544	0.389	1.259
Asymmetric 2	0.636	1.526	0.389	1.238
Asymmetric 3	0.636	1.509	0.389	1.216

Table 1: Codal and Analytical Fundamental Natural Period for Different Models

Sl No	Type of Structure	Disp (m)	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
1	Symmetric	0.0994	1865.8	513	57	80	0	0	0	0	0	650
2	Asymmetric 1	0.0972	1886.1	501	69	80	0	0	0	0	0	650
3	Asymmetric 2	0.0950	1897.9	501	69	80	0	0	0	0	0	650
4	Asymmetric 3	0.1153	1969.5	485	60	105	0	0	0	0	0	650

Table 2: Hinge Status at Performance Point along X-direction for the Structures Neglecting Stiffness (ESA, EQ in X) of Walls

Sl No	Type of Structure	Disp (m)	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
1	Symmetric	0.1048	1940.1	520	50	80	0	0	0	0	0	650
2	Asymmetric 1	0.0976	1952.8	527	43	80	0	0	0	0	0	650
3	Asymmetric 2	0.0966	1969.3	523	47	80	0	0	0	0	0	650
4	Asymmetric 3	0.1142	2238.5	500	45	65	40	0	0	0	0	650

Table 3: Hinge Status at Performance Point along X-direction for the Structures Neglecting Stiffness (RSA, EQ in X) of Walls

From the data presented in Table 3 pertaining to models neglecting stiffness of infill walls and designed by RSA, the number of hinge in elastic range decreases as the asymmetry increases and number of plastic hinges increases. But when these models were designed by two different methods ESA and RSA by neglecting stiffness of infill walls, the numbers of hinges in elastic range for RSA are more in number as compared to corresponding models designed by ESA and the numbers of plastic hinges by RSA are less in number as compared with ESA.

The structure designed by ESA and RSA at performance point are safe under pushover analysis in both X and Y directions for all models analysed by neglecting the stiffness of walls, thus the performance of these models is satisfactory and does not require retrofitting. In the models where stiffness of infill walls is considering, as shown in Tables 4 and 5, the behavior of models at performance point is similar to the models in which stiffness of infill walls is neglected. As expected, in models considering the stiffness of walls the number of hinges at performance point in elastic range decreases as the asymmetry of the models increases in comparison with corresponding symmetric model. The number of plastic hinges increase as the asymmetry of the model increases. Even though the total number of hinges in

Sl No	Type of Structure	Disp (m)	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
1	Symmetric	0.1032	3144.8	883	46	16	35	0	0	0	0	970
2	Asymmetric 1	0.0575	2880.7	886	12	17	25	0	0	0	0	970
3	Asymmetric 2	0.0585	3039.3	890	15	40	25	0	0	0	0	970
4	Asymmetric 3	0.0660	3407.2	882	23	40	25	0	0	0	0	970

Table 4: Hinge Status at Performance Point along X-direction for the Structures Considering Stiffness (ESA, EQ in X) of Walls

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Sl No	Type of Structure	Disp (m)	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
1	Symmetric	0.0905	3022.9	885	20	30	35	0	0	0	0	970
2	Asymmetric 1	0.0573	3017.2	896	48	3	23	0	0	0	0	970
3	Asymmetric 2	0.0515	3025.7	844	96	25	5	0	0	0	0	970
4	Asymmetric 3	0.0558	3419.8	900	25	22	23	0	0	0	0	970

Table 5: Hinge Status at Performance Point along X-direction for the Structures Considering Stiffness (RSA, EQ in X) of Walls

the plastic state is similar to the case where stiffness of infill walls is neglected, a number of hinges lie in the LS-CP range for all models and methods of design when stiffness of infill walls is considered. This is mainly due to the fact that the design force of the members in these models is considerably less than the corresponding members in models where stiffness of infill walls is neglected, as can be seen from Tables 2 and 3.

The structure designed by ESA and RSA are safe under pushover analysis in both X and Y directions for all models analysed considering the stiffness of walls, thus the performance of these models is satisfactory and some of the elements require retrofitting. In some of the elements the hinge status is between LS-CP which indicates the need for retrofitting.

BASE SHEAR AND ROOF DISPLACEMENT AT PERFORMANCE POINT

The design base shear for symmetric and asymmetric models estimated from hand calculation (ESA) matches with that obtained using ETABS, which validates that the models in ETABS are correct and can be used for further analysis. For RSA all design quantities like base shear, moments, deflections, etc are scaled to match its design base shear to that obtained by ESA, as required by the code. Base shear and roof displacement at performance point for symmetric and asymmetric models are as shown in Tables 6 and 7.

From Tables 6 and 7 it can be observed that the base shear at performance point are higher for all models than design base shear. From Tables 6 and 7 it can be observed that for the models neglecting stiffness of walls the design base shear at performance point for asymmetric models increases as the asymmetry increases. The design base shear at performance

Sl No	Type of Structure	Design Base Shear Vb (kN)	Performance Point			
			ESA		RSA	
			V (kN)	Δ (m)	V (kN)	Δ (m)
1	Symmetric	963.21	1874.49	0.10	1911.37	0.095
2	Asymmetric 1	969.46	1887.99	0.10	1942.62	0.095
3	Asymmetric 2	973.62	1915.17	0.10	1960.50	0.095
4	Asymmetric 3	978.83	1933.51	0.10	2138.55	0.099

Table 6: Performance Point along X-direction (EQ in X) for Different Structures Neglecting the Stiffness of Walls

Sl No	Type of Structure	Design Base Shear Vb (kN)	Performance Point			
			ESA		RSA	
			V (kN)	Δ (m)	V (kN)	Δ (m)
1	Symmetric	1531.50	2926.51	0.080	2793.69	0.075
2	Asymmetric 1	1541.44	3099.90	0.076	3174.38	0.073
3	Asymmetric 2	1548.06	3258.24	0.075	3389.16	0.073
4	Asymmetric 3	1556.34	3509.15	0.074	3689.62	0.072

Table 7: Performance Point along X-direction (EQ in X) for Different Structures Considering the Stiffness of Walls

point for asymmetric models neglecting stiffness of infill walls in comparison with symmetric models increases by 0.73%, 1.22% and 1.84%, and for models considering the stiffness of infill walls increases by 0.58%, 1.08% and 1.62% in comparison with corresponding symmetric models.

From Tables 6 and 7 it can be observed that, for both models neglecting and considering stiffness of infill walls, the base shear at performance point is lower for ESA than RSA. As this is expected, due to storey shear distribution is better for RSA than ESA. The base shear at performance point for the structures considered for the study, increases as the asymmetry of the structure increases gradually and the roof displacement decreases.

In asymmetric models, as the storey stiffness increases the base shear at the performance point increases. In Table 6, the models are made asymmetric by increasing the static eccentricity, the corresponding base shear at the performance point (neglecting the stiffness of walls, X-direction) is increases in comparison with symmetric by an amount of 1.25%, 2.17% and 3.15% when analyzed by ESA and 1.63%, 2.57% and 11.88% when analyzed by RSA.

Similarly, in Table 5.16 the performance point (considering the stiffness of walls, X-direction) is increases by 5.92%, 11.33% and 19.90% when analyzed by ESA and 13.62%, 21.31% and 32.06% when analyzed by RSA as compared to symmetric structure.

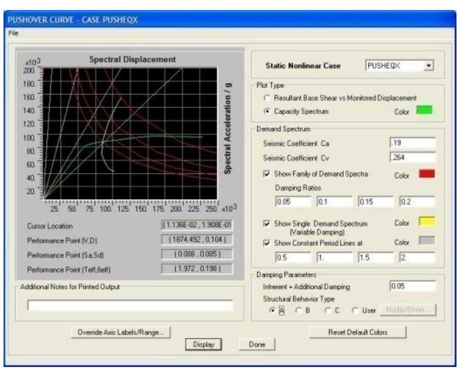

Fig 5: Performance point for symmetric model neglecting Stiffness of walls along Y-Direction (ESA, EQ in X-direction)

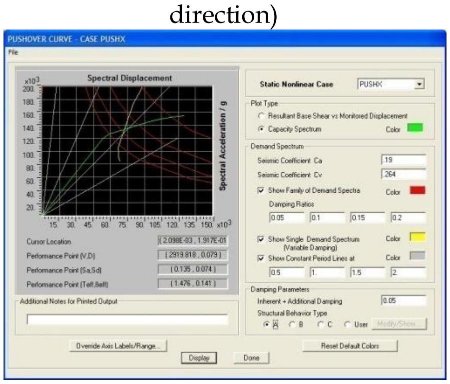

Fig 6: Performance point for symmetric model considering Stiffness of walls along Y-Direction (ESA, EQ in X-direction)

LATERAL DISPLACEMENTS

The lateral displacement of models considered for study is the displacement of centre of mass. The maximum displacement at each floor level with respect to ground for all models along X direction for different analyses studied. For the models considering the stiffness of infill walls, ground storey is a soft storey, there fore models in which stiffness of infill walls considering, as per code provision the ground storey columns and beams made 2.5 times stronger than upper storey columns and beams. As it is not done in our models a abrupt change in displacement can be seen at storey_1 as compared to models in which stiffness of infill walls neglected.

From Figs 7 and 8, it is observed that displacement profile for models neglecting stiffness of infill walls is maximum at roof and gradually reducing in lower storeys and a zero displacement at basement. This type of displacement profile is due do to neglecting the stiffness of infill walls. From the graphs 9 and 10, it is observed that displacement profile for models considering the stiffness of walls changing abruptly at storey-1; it indicates the stiffness irregularity which is due to open ground storey and presence of masonry infill walls (considering stiffness) in the upper storey.

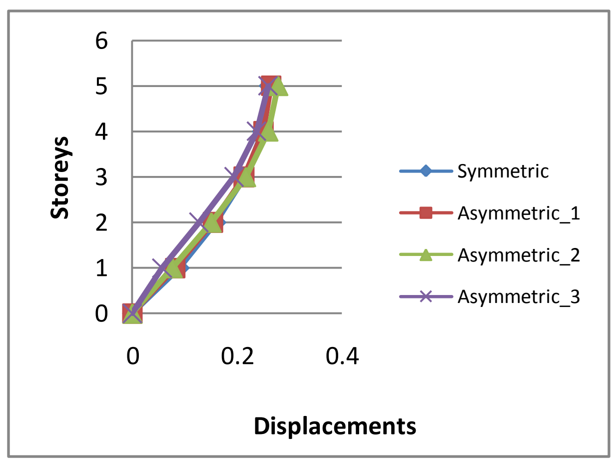

Fig 7: Lateral Displacements (m) at collapse along X for ESA(Pushover Analysis in X) for Different Models Neglecting the Stiffness of walls

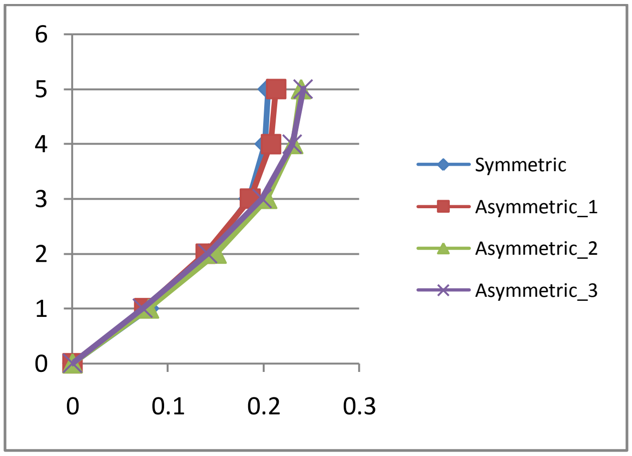

Fig 8: Lateral Displacements (m) at collapse along X for RSM(Pushover Analysis in X) for Different Models Neglecting the Stiffness of walls

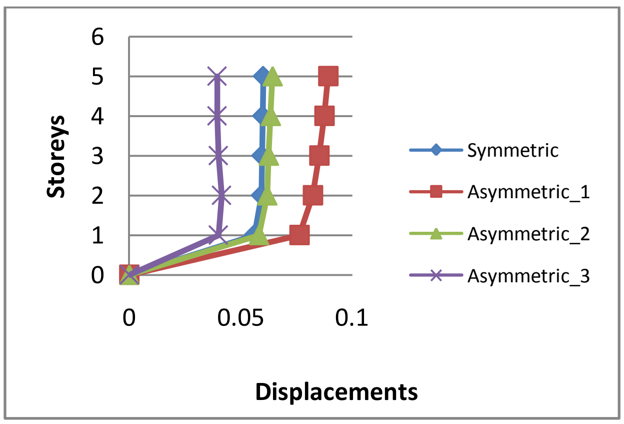

Fig 9: Lateral Displacements (m) at collapse along X for ESA(Pushover Analysis in X) for Different Models Considering the Stiffness of walls

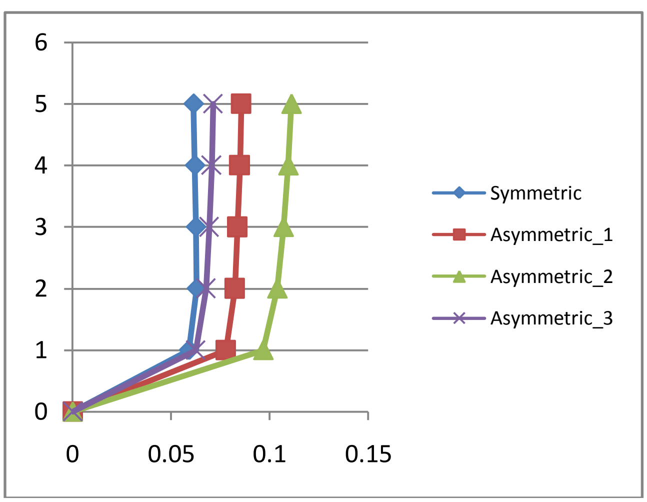

Fig 10: Lateral Displacements (m) at collapse along X for RSM(Pushover Analysis in X) for Different Models Considering the Stiffness of walls

DUCTILITY RATIOS

Reinforced concrete structures for earthquake resistance must be designed, detailed and constructed in such a way that the ductility factor will be limited to 3. The ductility ratio of the models analysed are given in Tables 8 to 11.

From Table 8 it can be seen that the ductility ratio is in the range of 3.61 to 4.10. For models neglecting stiffness of infill walls, as the asymmetry increases the ductility ratio increases as compared to corresponding symmetric building for both ESA and RSA. All the models with earthquake acting along X-direction and neglecting the stiffness of walls behave as 'Structures with restricted ductility' with $1.5 < \mu < 4$ and only asymmetric_3 structure (RSA) is fully ductile structure with $4 < \mu < 8$.

From Table 9 it can be seen that the ductility ratio is in the range of 3.85 to 7.67. For models neglecting stiffness of infill walls, as the asymmetry increases the ductility ratio increases as compared to corresponding symmetric building for both ESA and RSA. All the models with earthquake acting along Y-direction and neglecting stiffness of walls are fully ductile with $4 < \mu < 8$ and the symmetric models analysed by RSA behave as Structures with restricted ductility with $1.5 < \mu < 4$.

As the ductility ratio of the models considered for the analysis is limited to 3, from Table 8 and 9 it can be seen that, all models neglecting stiffness of infill walls have higher ductility ratio in both X and Y-direction which indicates the structure has higher strength than required leading to uneconomic structures. The models neglecting stiffness of infill walls are more ductile as compared to models where stiffness of infill walls is considered.

From the Table 10 it can be seen that the ductility ratio is in the range of 2.016 to 2.264. All the models with earthquake acting along X-direction and considering stiffness of walls are Structures with restricted ductility with $1.5 < \mu < 4$

From the Table 11 it can be seen that the ductility ratio is in the range of 2.167 to 5.375. All the models with earthquake acting Y-direction and considering stiffness of walls are Structures with restricted ductility with $1.5 < \mu < 4$ and asymmetric_2 and Asymmetric_3 (RSA) are fully ductile structures with $4 < \mu < 8$.

From data presented in Table 10 and 11 it can be seen that, all models considering stiffness of infill walls have lower ductility ratio in both X and Y-direction as compared to corresponding models where stiffness of infill walls is neglected. The models considering stiffness of infill walls behave less ductile.

Type of Structure	ESA			RSA		
	Δmax	Δy	µ	Δmax	Δy	µ
Symmetric	0.285	0.076	3.750	0.242	0.067	3.612
Asymmetric 1	0.288	0.074	3.892	0.249	0.061	4.082
Asymmetric 2	0.286	0.073	3.918	0.258	0.063	4.095
Asymmetric 3	0.283	0.072	3.931	0.246	0.060	4.100

Table 8: Ductility ratio in X-direction for Structure Neglecting the Stiffness of Walls

Type of Structure	ESA			RSA		
	Δmax	Δy	µ	Δmax	Δy	µ
Symmetric	0.280	0.047	5.957	0.258	0.067	3.851
Asymmetric 1	0.257	0.036	7.139	0.273	0.064	4.266
Asymmetric 2	0.242	0.032	7.563	0.270	0.063	4.286
Asymmetric 3	0.238	0.031	7.677	0.260	0.060	4.333

Table 9: Ductility ratio in Y-direction for Structure Neglecting the Stiffness of Walls

Type of Structure	ESA			RSA		
	Δmax	Δy	µ	Δmax	Δy	µ
Symmetric	0.127	0.063	2.016	0.116	0.055	2.109
Asymmetric 1	0.120	0.057	2.105	0.117	0.054	2.167
Asymmetric 2	0.121	0.056	2.161	0.114	0.051	2.235
Asymmetric 3	0.129	0.059	2.186	0.120	0.053	2.264

Table 10: Ductility ratio in X-direction for Structure Considering the Stiffness of Walls

Type of Structure	ESA			RSA		
	Δmax	Δy	µ	Δmax	Δy	µ
Symmetric	0.128	0.057	2.246	0.117	0.054	2.167
Asymmetric 1	0.096	0.042	2.286	0.091	0.031	2.935
Asymmetric 2	0.079	0.034	2.324	0.120	0.026	4.615
Asymmetric 3	0.071	0.030	2.367	0.129	0.024	5.375

Table 11: Ductility ratio in Y-direction for Structure Considering the Stiffness of Walls

6. CONCLUSIONS

On the basis of present study following conclusions are drawn

- In Symmetric, Asymmetric_1, Asymmetric_2 and Asymmetric_3 models, when stiffness of infill walls is neglected the base shear at performance point is 1.94, 1.95, 1.96 and 1.97 times higher in X-direction and 2.13, 2.21, 2.23 and 2.33 times higher in Y-direction than design base shear, whereas for models considering the stiffness of infill walls, base shear at performance point is 1.91, 2.01, 2.10 and 2.25 times higher in X-direction and 2.90, 3.00, 3.11 and 3.21 times higher in Y-direction than design base shear. Models are capable of resisting more base shear than it is designed for.

- In Symmetric, Asymmetric_1, Asymmetric_2 and Asymmetric_3 models, where stiffness of infill walls is neglected and when designed by RSA the base shear at performance point is higher by 1.023 times as compared to ESA and for models considering the stiffness of walls the base shear at performance point is higher by 1.023, 1.024, 1.04 and 1.05 times.

- Ductility ratios for the models neglecting stiffness of walls is varying between 3.612 to 7.677 i.e. models neglecting the stiffness of walls behaving more ductile but models in which stiffness of infill walls is considered the ductility ratio is varying between 2.01 to 2.93 i.e models considering the stiffness of walls experience brittle failure.

- The required area of reinforcement in selected columns is greater when designed for gravity and earthquake loads as compared to corresponding models designed only for gravity loads. In the case of models where stiffness of infill walls is neglected, this increase is between 30% to 60%, whereas for models where stiffness of infill walls is considered, it varies between 50% to 70%. It is therefore important to consider the seismic loads in the design of medium rise buildings.

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