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Seismic Performance Evaluation of R c-Framed Buildings - An Approach To Torsionally Asymmetric Buildings

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ABSTRACT: - *At present scenario many buildings are asymmetric in plan and/or in elevation based on the distribution of mass and stiffness along each storey throughout the height of the building. Most recent earthquakes have shown that the irregular distribution of mass, stiffness and strengths may cause serious damage in structural systems. This research quantifies the performance of the torsionally balanced and torsionally unbalanced buildings also called as symmetric and asymmetric buildings by subjecting to pushover analysis. The buildings have un-symmetrical distribution of stiffness in storeys. In this paper the effort is made to study the effect of eccentricity between centre of mass (CM) and centre of stiffness (CR) and the effect of stiffness of infill walls on the performance of the buildings. The performance of the buildings is assessed as per the procedure prescribed in ATC-40 and FEMA-356.*

Four building models are considered for study, which are constructed on hard soil in seismic zone Ш of India (as per IS: 1893-2002[9]), one symmetric and 3 asymmetric in stiffness distribution. Infills were modeled using equivalent strut approach. Static analysis (for gravity and lateral loads) and non-linear pushover analysis (assigning the hinge properties to beams and column sections) were performed. It is concluded that the performance of the models in which the stiffness of walls considered is found better when compared with the models in which the stiffness of walls ignored. And with increase in eccentricity, the performance point of the structure will be more but due to increase in stiffness, structure may fail in brittleness.

***Keywords:* –** *Asymmetric Structure, Stiffness of Infills, Pushover Analysis, Seismic Performance*

I. Introduction

Earthquakes are one of the most devastating natural hazards that cause great loss of life and livelihood. Most recent earthquakes have shown that the irregular distribution of mass, stiffness and strengths may cause serious damage in structural systems, 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 requirement and functional needs. The structures whose performances were evaluated in this study, are designed with the provisions from IS: 1893-2002. Equivalent static force method of determining earthquake force is limited to the structures having height of less than 40 meters. Hence this study deals with medium rise buildings (ten storied). The purpose of the paper is to summarize the basic concepts on which the pushover analysis is based, perform non-linear static pushover analysis of medium height RC buildings and investigate the changes in structural behavior due to consideration of infill configurations.

II. Literature Review

Dhiman Basu and Sudhir K. Jain[3] In this paper, the definition of centre of rigidity for rigid floor diaphragm buildings has been extended to unsymmetrical buildings with flexible floors. A superposition-based analysis procedure is proposed to implement code-specified torsional provisions for buildings with flexible floor diaphragms. The procedure suggested considers amplification of static eccentricity as well as accidental eccentricity. The proposed approach is applicable to orthogonal as well as nonorthogonal unsymmetrical buildings and accounts for all possible definitions of center of rigidity. Analysis results of a sample building clearly show the significance of considering the torsion provisions of design codes for asymmetric flexible diaphragm buildings. It is seen that treating the diaphragms of such buildings as rigid for torsional analysis may cause considerable error. The example also illustrates that the contribution of accidental torsion as well as the torsional amplification terms can be quite significant.

Humar et al[7] [2003] showed that eccentricities between the centers of rigidity and centers of mass in a building cause torsional motion during an earthquake. Seismic torsion leads to increased displacement at the extremes of the building and may cause distress in the lateral load-resisting elements located at the edges, particularly in buildings that are torsionally flexible. For an equivalent static load method of design against torsion, the 1995 National Building Code of Canada specifies values of the eccentricity of points through which the inertia forces of an earthquake should be applied. In general, the code requirements are quite conservative. They do not place any restriction on the torsional flexibility, however. New proposals for 2005 edition of the code which simplify the design eccentricity expressions and remove some of the unnecessary conservatism are described. The new proposals will require that a dynamic analysis method of design be used when the torsional flexibility of the building is large. Results of analytical studies, which show that the new proposals would lead to satisfactory designs.

R. Shahrin & T.R. Hossain [15] has overviewed the performance of bare, full infilled and soft ground storey buildings which is situated in Dhaka city. The building models have been designed according to BNBC (2006) and their performance based seismic investigation is assessed by pushover analysis. The performance of the buildings is assessed as per the procedure prescribed in ATC 40 and FEMA 273. For different loading conditions resembling the practical solutions of Dhaka city, the performances of these structures are analyzed with the help of capacity curve, capacity spectrum, deflection, drift and seismic performance level. For the bare frame structure they kept regular throughout its height and bay length to concentrate on the effects caused by the distribution of infill. The structure is six storeys high with a storey height of 3 meters. In order to investigate the effect of infill distribution they have considered 3 geometrical cases: The first case comprises a fully infilled structure resembling the regular structures representing a regular distribution of stiffness throughout the height. Second case examined the effects of omitting infills from ground floor only, such as with infamous soft ground storey configuration. On the other hand third case specifically dealt with the consequences of omitting the infills of the third floor of the building and observed the influences on structural performances. It has been concluded that the performance of an infilled frame is found to be much better than a bare frame structure and also the consideration of effect of infill leads to significant change in the capacity.

A. Kadid and A. Boumrkik[12] an experimental pushover analysis was carried out with an objective to evaluate the performance of framed buildings under future expected earthquakes. To achieve this objective, three framed buildings with 5, 8 and 12 stories respectively were analyzed. The results obtained in this paper shows that properly designed frames will perform well under seismic codes. Some of the conclusions made by the authors are:

The pushover analysis is a relatively simple way to explore the non linear behavior of Buildings. The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behavior of structures. The behavior of properly detailed reinforced concrete frame building is adequate as indicated by the intersection of the demand and capacity curves and the distribution of hinges in the beams and the columns. Most of the hinges developed in the beams and few in the columns but with limited damage.

III. Nature of Problem and Building Code Provision

The fundamental natural period of vibration of a building is given by empirical formulae 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 on 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.

1.1 Equivalent Static Analysis

Along any principal direction, the total design lateral force or design base shear is given in terms of design horizontal seismic coefficient and 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.

Following procedure is generally used for the Equivalent Static Analysis:

Determination of base shear (V_B) [5] of the building

$$V_B = A_h \times W \tag{1}$$

where ,
$$A_h = \frac{Z \cdot I \cdot S_a}{2 \cdot R \cdot g} \tag{2}$$

The design base shear V_b computed is then distributed along the height of the structure using a parabolic distribution expression:

$$Q_i = V_B \frac{W_i \cdot h_i^2}{\sum_{i=1}^{n} W_i \cdot h_i^2} \tag{3}$$

1.2 Static Torsional Provisions

The code defines the eccentricity as the distance between the centre of mass and centre of rigidity. The centre of rigidity is defined as the point through which the resultant of the restoring forces of a system acts. The design eccentricity, e_{di} to be used at floor is prescribed as:

$e_{di} = 1.5e_{si} + 0.05b_i$ (4)

$e_{di} = e_{si} – 0.05b_i$ (5)

whichever gives the more sever effect in the shear of any frame. In above expression e_{si} is the static eccentricity and ,b_i is the floor plan dimension of the floor i, perpendicular to the direction of earthquake force. The factor 1.5 represents the dynamic amplification factor and the factor 0.05 represents the extent of accident accidental eccentricity.

IV. Non-Linear Static Push-over Analysis

The pushover analysis of a structure is a static nonlinear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. On a building frame, plastic rotation is monitored, and lateral inelastic forces versus displacement response for the complete structure are analytically computed. This type of analysis enables weakness in the structure to be identified. The decision to retrofit can be taken in such studies.

Two key elements of a performance based design procedure are demand and capacity. Demand is a representation of the earthquake ground motion. Capacity is a representation of the structure’s ability to resist the seismic demand. The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design. Once the capacity curve and demand displacement are defined, a performance check can be done. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limit of the performance objective for the forces and displacements implied by the displacement demand.

In this study, non linear static pushover analysis was used to evaluate the seismic performance of the structures. The numerical analysis was done using ETABS 9 and guidelines of ATC-40 and FEMA 356 were followed. The overall performance evaluation was done using capacity curves, storey displacements and ductility ratios. Plastic hinge hypothesis was used to capture the non linear behavior according to which plastic deformations are lumped on plastic hinges and rest of the system shows linear elastic behavior.

The pushover or capacity curve represents the lateral displacements as the function of force applied to the structure. Location of hinges in various stages can be obtained from pushover curve as shown in Fig. 1. The range AB is elastic range, B to IO is the range of immediate occupancy, IO to LS is the range of life safety, and LS to CP is the range of collapse prevention [ATC-40]. If all the hinges are within the CP limit then the structure is said to be safe. However, depending upon the importance of structure the hinges after IO range may also need to be retrofitted.

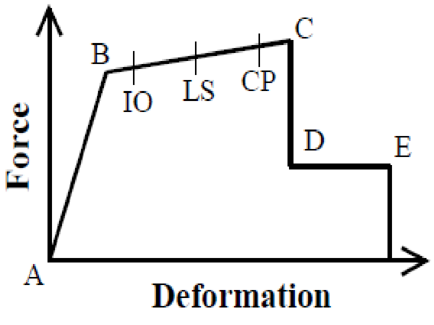

Fig 1: Different Stages of Plastic Hinge Formation [2].

V. Modeling and Analysis

In the present study lateral load analysis as per the seismic code IS: 1893-2002 is carried out for symmetric and asymmetric buildings and an effort is made to study the effect of seismic loads on them and their capacity and demand is evaluated using non linear static pushover analysis guidelines given in ATC-40 and FEMA 356.

The plan layout of the reinforced concrete ordinary moment resisting frame building of ten storied building without and with consideration of stiffness of walls is as shown in Fig. 2, 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 storeys, bay dimensions in both x and y directions are kept as 6m and 4m respectively. 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 torsionally unbalanced i.e. asymmetric. The elevations of the different building models considered are shown in Fig.4. The masonry infill is modeled as equivalent diagonal strut in the upper storey. The equation for calculation of equivalent diagonal strut width is considered from Kasim Armagon et al[11] paper.

The width is given by

$$W_{ef} = 0.175\,(\lambda_h\,\mathrm{H})^{-0.4}\ \sqrt{H^2 + L^2} \qquad (6)$$

where

$$\lambda_h = \sqrt[4]{\frac{E_i t \sin 2\theta}{4 E_c I_c H_i}} \qquad (7)$$

H and L are the height and length of the frame, Ec, and Ei are the elastic moduli of the column and of the infill panel, t is the thickness of the infill panel, q is the angle defining diagonal strut, Ic is the modulus of inertia of the column and Hi is the height of the infill panel.

Concrete frame elements are classified as beam and column elements. Columns and beams are modeled using three dimensional frame elements. Slabs are modeled as rigid diaphragms. The beam column joints are assumed to be rigid. Default hinge properties available in ETABS as per ATC-40 are assigned to the frame elements.

The following four distinct building models are used in the study.

Model I: The building is symmetric in plan and also in distribution of storey stiffness, both in plan and along height. Building has no walls in the first storey and brick masonry walls in the upper storeys. Two forms of this model are studied, one in which the stiffness of walls is ignored and the other in which stiffness of infill walls is considered. Irrespective of wheather the stiffness of infill walls is ignored or considered, the mass of the infill walls is always considered.

Model II: The building is similar to the building in Model I in both plan and elevation, but stiffness eccentricity is introduced by making the columns on the left edge larger (450 x 900 mm instead of 450 x 600 mm). This introduces a static eccentricity of 11.81%. This model is also studied by first ignoring the stiffness of the infill walls and then considering the stiffness of in fill walls.

Model III: This model is similar to Model I and stiffness eccentricity is introduced in the same way as in the case of Model II, except that the static eccentricity is larger because the column sizes on the left edge in this case in 450 x 1200 mm. This makes the static eccentricity of 17.5%. This model is also studied by first ignoring the stiffness of the infill walls and then considering the stiffness of in fill walls.

Model IV: This model is similar to Model I and stiffness eccentricity is introduced in the same way as in the case of Model II, except that the static eccentricity is larger because the column sizes on the left edge in this case in 450 x 1500 mm. This makes the static eccentricity of 22.3%. This model is also studied by first ignoring the stiffness of the infill walls and then considering the stiffness of in fill walls.

VI. Methodology

To study the effects of different configurations of centre of mass and stiffness at different levels of hazard, multi story buildings are used. Building is symmetric in the X direction and asymmetric in Y-direction. The asymmetry in Y-direction is produced by changing the size of the columns (by increasing the stiffness of columns in Y-direction). A symmetric model is also used as a reference, which is a Torsionally Balanced (TB) system. The design gravity loads and earthquake loads of TB and Torsionally Unbalanced (TU) system will be determined based on the Indian Standard IS: 875-1987, Parts 1 and 2 and IS: 1893 (Part 1)-2002 respectively. The lateral strengths of the TU systems are the same and equal to the lateral strength of the TB system.

The methodology will consist of the selected building model being designed as per IS: 1893 (Part 1)-2002. The TB and TU building models will then be subjected to Static Pushover Analysis to assess the performance of the building. The performance of the model will be compared with that of the corresponding TB model. The study will be repeated for models with different static eccentricities.

A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness and strength. Using a pushover analysis, a characteristic non linear force displacement relationship can be determined [Kadid, 2008][12].

ETABS software will be used to perform the static pushover analysis. The user establishes grid lines, places structural objects relative to the grid lines using points, lines and areas, and assigns loads and structural properties to those structural objects (for example, a line object can be as-signed section properties; a point object can be assigned spring properties; an area object can be assigned slab or deck properties). The program simplifies seismic analysis by providing modeling features such as rigid diaphragms to model slabs and infill walls. Analysis and design are then performed based on the structural objects and. It can take the results of an analysis as input to define the hinge properties of members so that pushover analysis can be carried out. Results are generated in graphical or tabular form that can be printed to a printer or to a file for use in other programs.

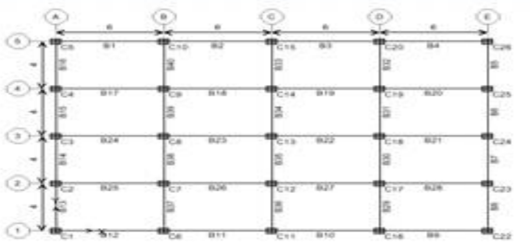
Fig 2 (a): Plan of Symmetric Building

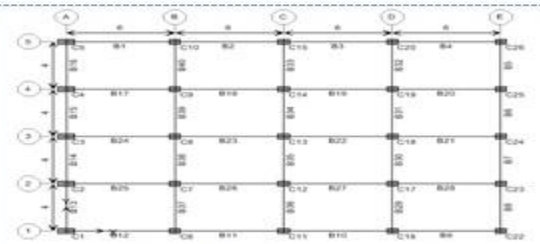
Fig 2 (b): Plan of Asymmetric-1 Building

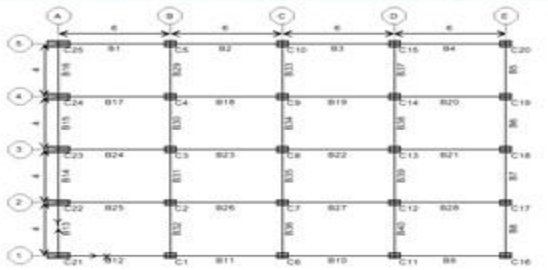
Fig 2 (c): Plan of Asymmetric-2 Building

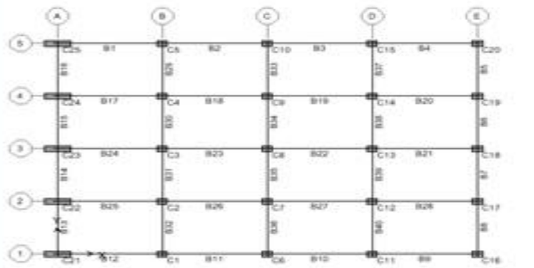
Fig 2 (d): Plan of Asymmetric-3 Building

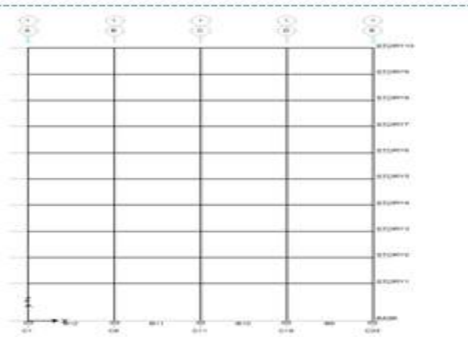
Fig 3 (a): Elevation of models neglecting stiffness of wall

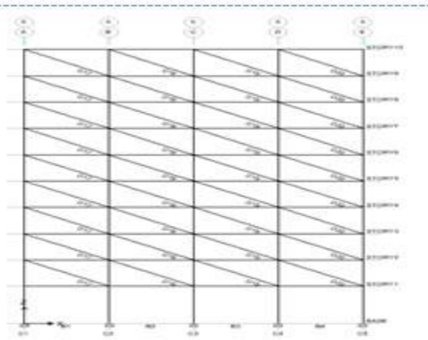
Fig 3 (b): Elevation of models considering stiffness of walls

VII. Results and Discussions

1.3 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 Table 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 codal natural period for models where stiffness of infill walls is ignored depends only on height and is the same for all models, irrespective of the amount of eccentricity and base dimension of the building models. The codal natural period for models where stiffness of infill walls is considered, depends on the height as well as the lateral dimension of the building in the direction of the earthquake. Since all models have different lengths in x and y direction, the value is different in x and y direction where stiffness of walls is considered.

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.

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

Model	Fundamental Natural Periods T (sec)						
	Neglecting the Stiffness of Walls			Considering the Stiffness of Walls			
	Codal	Analysis	Vb (kN)	Codal		Analysis	Vb (kN)
				x	Y		
Symmetric	1.04	2.37	1620.62	0.611	0.749	1.21	2581.37
Asymmetric 1	1.04	2.31	1632.66	0.611	0.749	1.18	2603.91
Asymmetric 2	1.04	2.23	1644.70	0.611	0.749	1.16	2654.94
Asymmetric 3	1.04	2.22	1663.31	0.611	0.749	1.14	2648.97

From Table 1 it can be seen that the fundamental natural period obtained from analytical approach is 2.13 to 2.28 times higher than those obtained from codal approach, for models where stiffness of infill walls is neglected and 1.86 to 1.98 times higher in x-direction, 1.52 to 1.62 times higher in y-direction for models where stiffness of infill walls is considered.

1.4 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 point for different models considered for the analysis i.e. both symmetric and asymmetric models with neglecting and considering the stiffness of infill walls are shown in Tables 2 to 5.

Table 2: Hinge Status at Performance Point along X-direction for ESA for the Models Neglecting Stiffness of Walls.

Model	Hinges at Performance Point										
	Disp (m)	Base force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total Applied
Symmetric	2990.57	0.188	69	81	236	0	**1**	**37**	**0**	0	1300
Asymmetric 1	3007.12	0.185	66	106	234	0	**3**	**41**	**0**	0	1300
Asymmetric 2	3125.36	0.178	58	133	232	0	**16**	**17**	**4**	0	1300
Asymmetric 3	3136.26	0.203	17	108	280	0	**7**	**24**	**6**	0	1300

Table 3: Hinge Status at Performance Point along Y-direction for ESA for the Models Neglecting Stiffness of Walls.

Model	Hinges at Performance Point										
	Disp (m)	**Base force**	**A-B**	**B-IO**	**IO-LS**	**LS-CP**	**CP-C**	**C-D**	**D-E**	**>E**	**Total Applied**
Symmetric	3151.06	0.158	137	128	169	0	**2**	**8**	**2**	0	1300
Asymmetric 1	3154.81	0.113	155	160	127	0	**2**	**6**	**8**	0	1300
Asymmetric 2	3232.54	0.157	138	154	178	0	**3**	**3**	**6**	0	1300
Asymmetric 3	3070.57	0.117	150	184	59	0	**3**	**7**	**0**	0	1300

From the data presented in Table 2, the models are subjected to pushover analysis (ESA) in x-direction 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.

From the data presented in Table 3 pertaining to models neglecting stiffness of infill walls and designed by ESA in y-direction, the number of hinge in elastic range decreases as the asymmetry increases and number of plastic hinges increases.

From the data presented in tables 2 and 3, it can be observed that as the asymmetry is increasing the number of hinges in plastic range is also increasing, representing as the asymmetry increases the building will be more vulnerable to seismic forces.

From the data presented in Tables 2 and 3, it can be observed that, for all models neglecting stiffness of infill walls and when designed by ESA the number of hinges in elastic range are more in number for EQ in Y-direction as compared to EQ in X-direction.

The structure designed by ESA at performance point are not safe under pushover analysis in both X and Y directions for all models analyzed by neglecting the stiffness of walls. The performance of the structures suggests an increased vulnerability of the structure with formation of column hinges at base level and beam hinges at each story level at performance point.

Most of the elements are in the range of IO-LS and some of the elements lie in the range of C-D which indicates failure of those elements, so these structural elements requires retrofitting.

Table 4: Hinge Status at Performance Point along X-direction for ESA for the Models Considering Stiffness of Walls.

Model	Hinges at Performance Point										
	Disp (m)	**Base Force**	**A-B**	**B-IO**	**IO-LS**	**LS-CP**	**CP-C**	**C-D**	**D-E**	**>E**	**Total Applied**
Symmetric	6511.98	0.064	22	0	60	0	**2**	**36**	**2**	0	2020
Asymmetric 1	6765.76	0.062	19	25	68	0	**2**	**16**	**2**	0	2020
Asymmetric 2	7000.85	0.061	20	27	66	0	**4**	**16**	**4**	0	2020
Asymmetric 3	7319.08	0.060	19	42	68	0	**8**	**2**	**0**	0	2020

Table 5: Hinge Status at Performance Point along Y-direction for ESA for the Models Considering Stiffness of Walls.

Model	Hinges at Performance Point										
	Disp (m)	**Base Force**	**A-B**	**B-IO**	**IO-LS**	**LS-CP**	**CP-C**	**C-D**	**D-E**	**>E**	**Total**
Symmetric	4762.08	0.078	37	10	68	0	**2**	**12**	**8**	0	2020
Asymmetric 1	5004.82	0.076	36	8	70	0	**2**	**10**	**12**	0	2020
Asymmetric 2	5539.71	0.074	34	36	64	0	**0**	**8**	**6**	0	2020
Asymmetric 3	6090.27	0.071	35	55	44	0	**2**	**0**	**0**	0	2020

In the models where stiffness of walls is considered as shown in tables 4 and 5, the behavior of models at performance point is similar to the models in which stiffness of 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 total number of hinges in plastic state is similar to the case where stiffness of infill walls is neglected, a number of hinges lie in the range of IO-LS and some of the elements mainly ground story columns lie in the range of C-D, this is because of the soft story mechanism in which the stiffness of walls is not considered in the ground floor which increases the seismic vulnerability of the structure.

The structure designed by ESA are not safe under pushover analysis in both X and Y directions for all models analyzed considering stiffness of walls, thus the performance is not satisfactory and the elements in which the hinge status is between IO-LS and C-D indicates the need for retrofitting.

1.5 Base Shear and Roof Displacement at Performance Point

The design base shear for symmetric and asymmetric models obtained from hand calculation match with those obtained by using ETABS, which validates the models in ETABS, can be used for further analysis. Base shear and roof displacement at performance point for symmetric and asymmetric models are as shown in Tables 6 to 9.

The Seismic Performance Evaluation is comprises of comparison between some of the 'demand' that earthquake places on Structure to measure of the 'capacity' of the building to resist. Base Shear (total horizontal force at the lower level of the building) is the normal parameter that is used for this purpose. The Base Shear demand that would be generated by a given earthquake or intensity of ground motion and compare this to the base shear capacity of the building.

Table 6: Performance Point along X-direction for ESA for the Models Neglecting Stiffness of Infill Walls.

Model	Design Base Shear Vb(kN)	Performance Point ESA X	
		V(kN)	d(m)
Symmetric	1620.62	2990.57	0.188
Asymmetric 1	1632.66	3007.12	0.185
Asymmetric 2	1644.70	3125.36	0.178
Asymmetric 3	1656.74	3136.26	0.203

Table 7: Performance Point along Y-direction for ESA for the Models Neglecting Stiffness of Infill Walls.

Model	Design Base Shear Vb(kN)	Performance Point ESA Y	
		V(kN)	d(m)
Symmetric	1620.62	3151.06	0.158
Asymmetric 1	1632.66	3154.81	0.113
Asymmetric 2	1644.70	3232.54	0.157
Asymmetric 3	1656.74	3270.57	0.117

Table 8: Performance Point along X-direction for ESA for the Models Considering Stiffness of Infill Walls.

Model	Design Base Shear Vb(kN)	Performance Point ESA X	
		V(kN)	d(m)
Symmetric	2250.26	6511.98	0.064
Asymmetric 1	2266.98	6765.76	0.062
Asymmetric 2	2283.69	7000.85	0.061
Asymmetric 3	2300.41	7319.08	0.060

Table 9: Performance Point along Y-direction for ESA for the Models Considering Stiffness of Infill Walls.

Model	Design Base Shear Vb(kN)	Performance Point ESA Y	
		V(kN)	d(m)
Symmetric	2250.26	4762.08	0.078
Asymmetric 1	2266.98	5004.82	0.076
Asymmetric 2	2283.69	5539.71	0.074
Asymmetric 3	2300.41	6090.27	0.071

From Tables 6 to 9 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 point for asymmetric models neglecting stiffness of infill walls in comparison with symmetric models increases by 0.55%, 4.5% and 4.8%, and for models considering the stiffness of infill walls increases by 3.89%, 7.5% and 12.4% in comparison with corresponding symmetric models.

From the above tables it can be observed that the performance points in X-direction are higher as compared to performance points in Y-direction, this is because of higher stiffness in X-direction and the roof displacement in X-direction is lesser than in Y-direction.

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 0.55%, 4.5% and 4.8%. Similarly, in Table 7 the performance point (neglecting the stiffness of walls, Y direction) increases by 0.1%, 2.58% and 3.79%.

Similarly, in Table 8 the performance point (considering the stiffness of walls, X direction) is increases by 3.9%, 7.5% and 12.4%. Similarly, in Table 9 the performance point (considering the stiffness of walls, Y direction) is increases by 5.1%, 16.33% and 27.89%.

The base shear at performance point for the structures considered for the study, increases as the asymmetry of the structure increases gradually and roof displacement decreases.

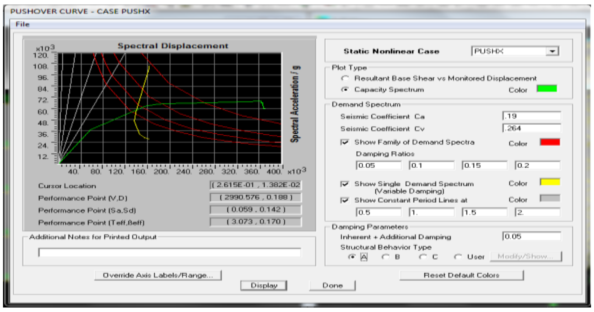

Fig 4: Performance Point for Symmetric Model Neglecting Stiffness of Walls along X-Direction (ESA, EQ in X-direction)

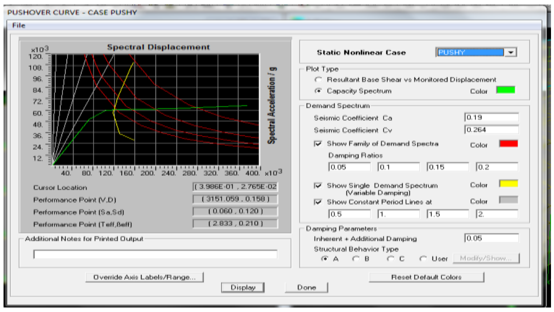

Fig 5: Performance Point for Symmetric Model Neglecting Stiffness of Walls along Y-direction (ESA, EQ in Y-direction)

1.6 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 and Y directions obtained from Equivalent Static analysis are shown in Fig. 6 to 9.

For the models considering the stiffness of infill walls, ground storey is a soft storey, therefore 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 Fig. 6 and 7, 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 Fig. 8 and 9, 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.

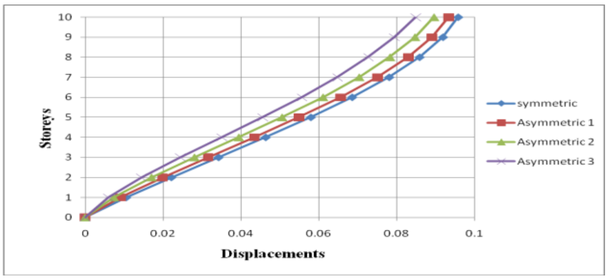

Fig 6: Lateral Displacements along X-direction for ESA (Pushover Analysis in X) for Different Models Neglecting the Stiffness of Walls.

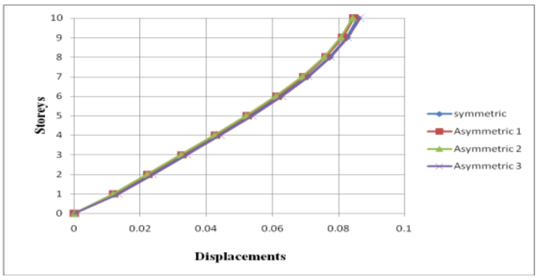

Fig 7: Lateral Displacements along Y for ESA (Pushover Analysis in Y) for Different Models Neglecting the Stiffness of Walls.

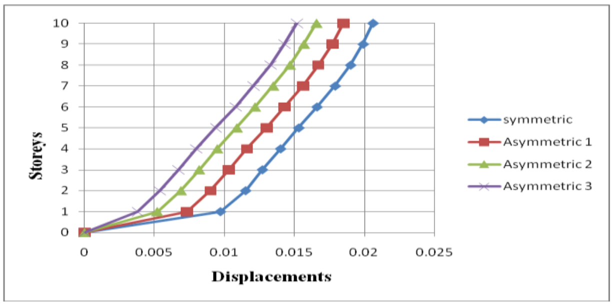

Fig 8: Lateral Displacements along X for ESA (Pushover Analysis in X) for Different Models Considering the Stiffness of Walls.

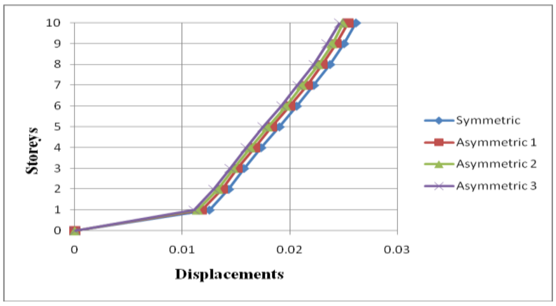

Fig 9: Lateral Displacements along Y for ESA (Pushover Analysis in Y) for Different Models Considering the Stiffness of Walls.

1.7 Ductility Ratios

Ductility of a structure, or its member, is the capacity to undergo large inelastic deformation without significant loss of strength or stiffness[1]. This is important for an earthquake resisting system because if the structure is incapable of behaving in ductile fashion then the structure collapse without yielding. Reinforced Concrete structures for earthquake resistance must be designed, detailed and constructed in such a way that the ductility factor will be at least 3 up to the point of beginning of visible damage and even greater, to point of beginning of structural damage and limitations[1]. Table 10 to 13 gives the ductility ratio for the symmetric and asymmetric building models in transverse and longitudinal direction.

Table 10 : Ductility Ratio in X-direction for the Models Neglecting Stiffness of Walls.

Type of Structure	Δ_{max}	Δ_y	μ
Symmetric	0.470	0.077	6.104
Asymmetric 1	0.486	0.073	6.657
Asymmetric 2	0.491	0.073	6.726
Asymmetric 3	0.514	0.084	6.119

Table 11 : Ductility Ratio in Y-direction for the Models Neglecting Stiffness of Walls.

Type of Structure	Δ_{max}	Δ_y	μ
Symmetric	0.498	0.094	5.298
Asymmetric 1	0.284	0.056	5.071
Asymmetric 2	0.585	0.092	6.358
Asymmetric 3	0.435	0.098	4.438

Table 12 : Ductility Ratio in X-direction for the Models Considering Stiffness of Walls.

Type of Structure	Δ_{max}	Δ_y	μ
Symmetric	0.092	0.027	3.407
Asymmetric 1	0.095	0.027	3.518
Asymmetric 2	0.100	0.028	3.571
Asymmetric 3	0.105	0.029	3.620

Table 13 : Ductility Ratio in Y-direction for the Models Considering Stiffness of Walls.

Type of Structure	Δ_{max}	Δ_y	μ
Symmetric	0.096	0.032	3.000
Asymmetric 1	0.093	0.035	2.657
Asymmetric 2	0.078	0.037	2.108
Asymmetric 3	0.078	0.040	1.950

From Table 10 it can be seen that the ductility ratio is above 4. For models neglecting stiffness of infill walls, as the asymmetry increases the ductility ratio increases as compared to corresponding symmetric building. All the models with earthquake acting along X-direction and neglecting the stiffness of walls behave as fully ductile structures with $4 < \mu < 8$.

From Table 11 it can be seen that the ductility ratio is also above 4. For models neglecting stiffness of infill walls, as the asymmetry increases the ductility ratio increases as compared to corresponding symmetric building. All the models with earthquake acting along Y-direction and neglecting stiffness of walls are fully ductile with $4 < \mu < 8$.

As the ductility ratio of the models considered for the analysis is limited to 3, from Tables 10 and 11 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 12 it can be seen that the ductility ratio is in the range of 3.407 to 3.620. 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 13 it can be seen that the ductility ratio is in the range of 1.95 to 3. All the models with earthquake acting along Y-direction and considering stiffness of walls are Structures with restricted ductility with $1.5 < \mu < 4$.

From data presented in Table 10 to 13 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.

VIII. Conclusions

Based on the present study following conclusions are drawn.

- The natural period decreases as the stiffness(i.e. eccentricity) of the building increases and thereby leading to increase in base shear. From analysis, it is found that for the models when stiffness of infill walls neglected and considered, the natural period decreases by 0.94 to 0.97 times when compared to the symmetric model.
- For the building models considered in the study, when stiffness of infill walls is neglected the base shear at performance point is 1.84, 1.85, 1.90 and 1.91 times higher in X-direction and 1.94, 1.93, 1.96 and 1.97 times higher in Y-direction than design base shear, whereas for models considering the stiffness of infill walls, base shear at performance point is 2.89, 2.98, 3.06 and 3.18 times higher in X-direction and 2.12, 2.21, 2.42 and 2.65 times higher in Y-direction than design base shear. Hence, the building models considered are capable of resisting more base shear than it is designed for.

- For the buildings studied in, it is found that the plastic hinges are more in case of models where stiffness of walls neglected than the models where stiffness of infill walls is considered, this is because the lateral stiffness is more in case of models where stiffness of walls considered due to the introduction of infill walls in the structure which helps in resisting lateral forces. Hence the structural elements which lies in the range of collapse point increases the seismic vulnerability of the structure and such elements requires retrofitting.
- The lateral displacements in which the stiffness of walls considered has shown the abrupt change in the displacement profile at storey-1 which indicates the stiffness irregularity due to soft storey mechanism and increases vulnerability towards seismic forces where as the models in which the stiffness of walls is neglected has shown the smooth displacement profile.
- Ductility ratios for the models neglecting stiffness of walls is varying between 5.071 to 6.726 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 1.950 to 3.620 i.e. models considering the stiffness of walls and experiences brittle failure.

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