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Torsional Behaviour of Asymmetrical Buildings in Plan under Seismic Forces

Mr. Sandesh N. Suryawanshi[1], Prof. S. B. Kadam[2], Dr. S. N. Tande[3]

[1] M. Tech. Student, Detriment of Applied Mechanics, Walchand college of Engineering, Sangli,

[2] Assistant Prof., Department of Applied Mechanics, Walchand college of Engineering, Sangli,

[3] Professor, Department of Applied Mechanics, Walchand college of Engineering, Sangli,

Maharashtra, India

Abstract: *Most recent earthquakes have shown that the irregular distribution of mass, stiffness and strengths may cause serious damage in structural systems Due to several reasons structures acquire asymmetry. Asymmetry in structures makes analysis of the seismic behaviour very complicated. Seismic demand in peripheral elements is enhanced. Uniformity in load distribution gets disturbed. Torsional behaviour of asymmetric building is one of the most frequent causes of structural damage and failure during strong ground motions. The paper first concentrates in understanding the complex behaviour of structure under asymmetric form; a study on the influence of the torsional moment effects on the behaviour of structure is done by using Response spectrum method. Then a simplified nonlinear pushover analysis has been used to find structural descriptors required in seismic vulnerability assessment. Deformation demand for different story for low-medium rise framed building has been found by using software SAP2000.*

Keywords: *Symmetric and Asymmetric plan, Earthquake, Torsion, Response spectrum, Pushover Analysis, Seismic Performance.*

1. Introduction

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. However an accurate evaluation of the seismic behavior of irregular buildings is quite difficult and a complicated problem Due to the variety of parameters and the choice of possible models for torsionally unbalanced systems, there is as yet no common agreement or any accurate procedure advised by researchers on common practice in order to evaluate the torsional effects. Seismic damage surveys and analyses conducted on modes of failure of building structures during past severe earthquakes concluded that most vulnerable building structures are those, which are asymmetric in nature. Asymmetric building structures are almost unavoidable in modern construction due to various types of functional and architectural requirements. Torsion in buildings during earthquake shaking may be caused from a variety of reasons, the most common of which are non-symmetric distributions of mass and stiffness Modern codes deal with torsion by placing restrictions on the design of buildings with irregular layouts and also through the introduction of an accidental eccentricity that must be considered in design. The lateral-torsional coupling due to eccentricity between centre of mass (CM) and centre of rigidity (CR) in asymmetric building structure generates torsional vibration even under purely translational ground shaking during seismic shaking of the structural systems, inertia force acts through the centre of mass while the resistive force acts through the centre of rigidity as shown in Fig.1

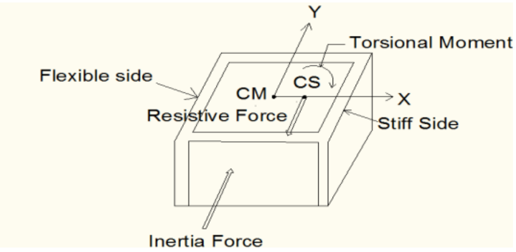

Figure 1. *Generation of torsional moment in asymmetric structures***.**

In order to design buildings in earthquake prone regions, seismic codes present different torsional provisions according to the seismicity of the region. Seismic provisions introduce design eccentricity to estimate the value of the torsion in buildings as accurately as possible. The dynamic eccentricity results from the irregular mass, resistance or stiffness distribution of the system, while the accidental eccentricity is expected to account for factors not explicitly considered, such as uncertain estimation of the mass, stiffness and rotational component, which is believed to play a highly important role. This accidental eccentricity, ea, which is a fraction of the plan dimension, b, is considered in design to be on either side of the center of stiffness. The coefficient, proposed by different seismic building codes is equal to 0.05 or 0.1. The design eccentricity ed1 is generally presented as;

$$ed1 = \alpha\ esi + \beta bi,\ ed2 = \delta esi - \beta bi$$

2. Methods of Seismic Analysis

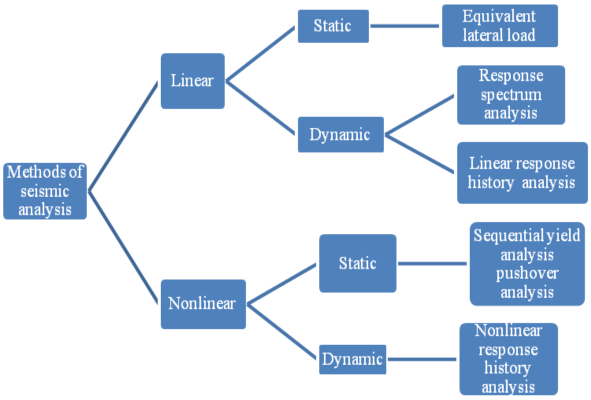

Figure 2. *Methods of seismic analysis*

The methods of seismic analysis used here are Response Spectrum Method and Non linear static push over analysis method.

2.1 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 (VB) of the building-

$VB = A_h \times W$ (1)

Where,

A_h = the seismic response coefficient

$$A_h = \frac{Z}{2} X \frac{I}{R} X \frac{Sa}{g} \quad (2)$$

Vertical Distribution of Seismic Forces

$$Q_i = V_b \frac{w_i h_i^2}{\sum_{j=1}^{n} w_j h_j^2} \quad (3)$$

The lateral force, F (kip or KN), induced at any level shall be determined from the following equations

2.2 Response Spectrum Method

The Response Spectrum is a method of estimation of maximum responses (acceleration, velocity and displacement) of a family of SDOF systems subjected to a prescribed ground motion. The RSM utilizes the response spectra to give the structural designer a set of possible forces and deformations a real structure would experience under earthquake loads.

Response Spectrum method, being time consuming and tedious process, most of time, it resort to computer applications. Now while, modeling the structure, in most of available software's, usually, we model the space frame, neglecting the in-fill wall stiffness. These results in flexible frames, and due to which, in most of Cases, the program gives a higher Time Period and results into lower base shear. Today with the availability of Powerful Computers and Software, the seismic coefficient method should not be applied to anything other than mass concrete!! In such a case a reduction coefficient would not be applicable. The infill walls and slabs should be modeled. If software has plate modeling capability, these can be modeled as plates. Otherwise an "equivalent" pair of diagonal members connecting the four corners of the slab or wall (in each bay) would simulate the shear behavior The diagonal members shall be 'truss' members - i.e. capable of only carrying axial load. The elastic properties can be derived from first principle, by matching forces and deformations in a plate and the equivalent diagonals.

2.3 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 structures 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 SAP2000 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.

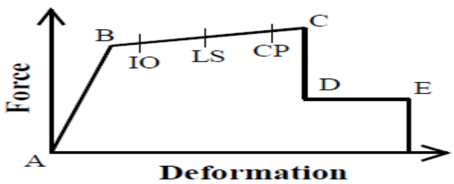

Figure 3. *Different Stages of Plastic Hinge Formation*

3. BUILDING DETAILS

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 three buildings one is symmetric and other two are asymmetric in plan for building height G+3 and G+6 and for comparison criteria is that numbers of columns are kept same for all three buildings and an effort is made to study the effect of seismic loads on them also determine torsional moments, base shear, displacement and time period by using response spectrum method and their capacity and demand is evaluated using nonlinear static pushover analysis guidelines given in FEMA-356 and ATC-40 by using software SAP2000.

Problem statement –A G+3 and G+6 storied bare RC Special Moment Resisting Frame has plan as shown in fig. is situated in seismic zone IV

Beam size -	0.30m x 0.45m
Column size -	0.30m x 0.45m
Thickness of slab-	150mm
Height of storied –	3m
Plinth height above GL –	2m
Unit weight of concrete –	25kN/m3
Live load on floor –	3kN/m3
Live load on roof –	2kN/m3
Grade of concrete –	M20
Grade of steel –	Fe415
Soil type –	Medium soil

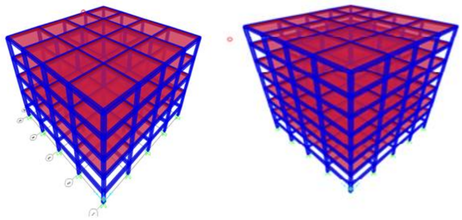

Figure 3(a). *Symmetric G+3 and G+6*

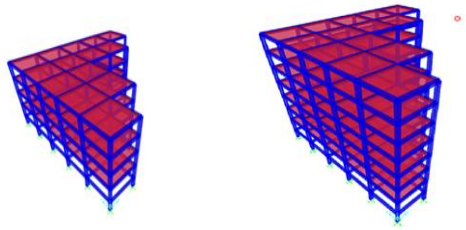

Figure 3(b). *L shape G+3*and *G+6*

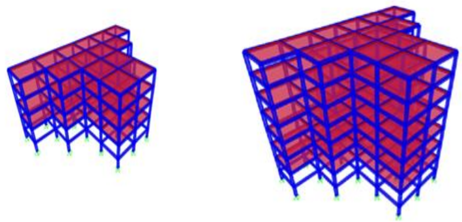

Figure 3(c). *T shape G+3 and G+6*

4. RESULTS AND DISCUSSIONS

Table 1(a). *Base shear and Time period for G+3*

G+3 Storied building	By using software SAP2000 Response spectrum method			
	Seismic weight (*kN*)	Time period (*sec*)	Base shear (*kN*)	Disp. (*mm*)
Symmetric	10812.5	0.85	537.77	7.64
L shape	9790.00	0.83	501.22	8.93
T shape	9790.00	0.83	504.45	8.39

Table 1(b). *Base shear and Time period for G+6*

G+6 Storied building	By using software SAP2000 Response spectrum method			
	Seismic weight (*kN*)	Time period (*sec*)	Base shear (*kN*)	Disp. (*mm*)
Symmetric	18996.87	1.41	532.8	12.38
L shape	17198.425	1.36	484.91	14.55
T shape	17198.425	1.38	493.62	13.55

Table 2(a). *Comparison of Torsional moments of column and beam for G+3*

G+3 Storied building	Torsional moment (*kN-mm*)					
	Symmetric building		L shape building		T shape building	
	Column	Beam	Column	Beam	Column	Beam
Story no 3	0	0	25.44	64.35	22.41	61.25
Story no 2	0	0	21.54	57.35	19.34	54.35
Story no 1	0	0	15.68	49.54	13.64	41.58

Table 2(b). *Comparison of Torsional moments of column and beam for G+6*

G+6 Storied building	Torsional moment (*kN-mm*)					
	Symmetric building		L shape building		T shape building	
	Column	Beam	Column	Beam	Column	Beam
Story no 6	0	0	37.96	76.28	34.98	68.32
Story no 5	0	0	32.94	70.65	30.25	62.22
Story no 4	0	0	27.54	64.22	26.98	58.85
Story no 3	0	0	25.98	51.12	23.56	43.15
Story no 2	0	0	21.25	37.25	19.88	31.14
Story no 1	0	0	18.12	25.58	17.12	21.98

Table 3(a). *Performance and collapse point for different structures along x direction for G+3*

Type of Struct-ure G+3	Design base shear (kN)	Performance point		Collapse point	
		Base shear (kN)	Disp. (mm)	Base shear (kN)	Disp. (mm)
Sym-metri-c	537.77	2465.82	40.12	3863.28	167.45
L shape	501.22	2223.93	42.00	3850.22	176.22
T shape	504.45	2277.11	41.12	3834.44	171.81

Table 3(b). *Performance and collapse point for different structures along x direction for G+3*

Type of Struct-ure G+6	Design base shear (kN)	Performance point		Collapse point	
		Base shear (kN)	Disp. (mm)	Base shear (kN)	Disp. (mm)
Sym-metri-c	532.8	2319.47	60	3556.47	186.16
L shape	484.91	2177.38	68	3557.27	225.73
T shape	493.62	2224.58	68	3543.50	212.50

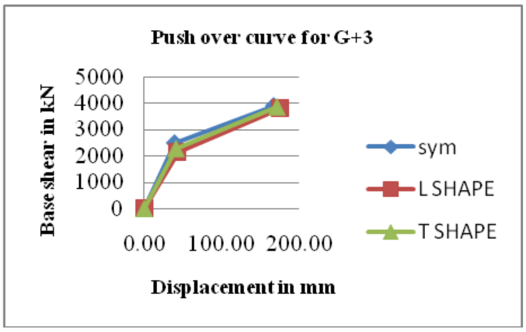

Figure 4(a). *Different Stages of Plastic Hinge Formation*

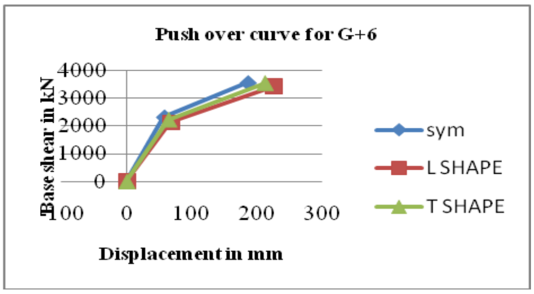

Figure 4(b). *Different Stages of Plastic Hinge Formation*

5. CONCLUSION

1. Time period and base shear calculation by using equivalent static method is approximately equal with response spectrum method in SAP.
2. While comparing the torsional moment (TM) in beam the result shows that for asymmetrical building the TM is more than symmetrical, therefore it is necessary to design the beam and column for torsional moment.
3. By using equivalent static method and response spectrum method in SAP it shows that, base shear and roof displacement for asymmetrical building is more than symmetrical building.
4. By using push over analysis performance of symmetrical building is better than asymmetrical building.
5. Formation of hinges in asymmetrical building is more and early than symmetrical building.

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