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Effect Of Combined Plan, Vertical And Mass Irregularity On Torsional Performance Of High Raised Buildings

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Abstract - **Recently, reports showing the damage caused by earthquakes indicate that, torsional effects often cause considerable damage to structures leading to their collapse. The response of asymmetric buildings towards torsion is one of the most crucial factors for their damage. Torsion in such buildings is due to irregularity in plan, mass and stiffness which may cause severe damage in structural systems. Due to various reasons structures acquire asymmetry. Asymmetric structures have irregular distribution of plan, stiffness and mass, its centre of mass and centre of rigidity do not coincide and hence cause the torsional effect on the structures which is one of the most important factor influencing the seismic damage of the structure. For the purpose of study, asymmetrical buildings of 12 storeys, 15 storeys and 18 storeys with same columns sizes subjected to gravity loads and seismic loads are analysed using non-linear dynamic analysis. The structure is evaluated in accordance with IS 456-2000 and seismic code IS: 1893-2002 using non-linear time history method with the help of ETABS. Percentage of torsion and joint rotation are significantly decreased by minimizing the stiffness eccentricity. Torsion reduces to 72%, 76% and 72% for 12 storey U shaped building, 15 storeys U shaped building and 18 storeys U shaped building. Similarly torsion reduces to 79%, 85% and 90% for 12 storey building with openings, 15 storey building with openings and 18 storey building with openings. Joint rotation reduced to 84%, 58%, and 65% for 12 storey U shaped building, 15 storeys U shaped building and 18 storeys U shaped building. Joint rotation reduced to 78%, 75 %, and 86% for 12 storey building with openings, 15 storey building with openings and 18 storey building with openings. Thereby a maximum decrease in torsion is 90% for 18 storey building with openings. And maximum decrease in joint rotation is 86% for 18 storey building with openings.**

keywords - **Plan irregularity, Vertical irregularity, Mass irregularity, Torsion**

I. INTRODUCTION

1.1 General: The earthquakes are the most unpredictable and devastating among all natural disaster. Earthquake is a phenomenon that occurs due to the geotechnical activities in the strata of the earth and causes heavy loss to both life and property if it occurs in populated regions. Thus, it is the responsibility of a structural engineer to draw out the parameters from previous experiences and consider all the possible hazards that the structure may be subjected to, in the future, for the purpose of safe design of the structure. For the simplicity of the practicing engineers, different societies in different countries have specified different earthquake codes for the design of the structures subjected to different type of hazards, which are modified as and when newer knowledge is acquired, keeping in view the hazards of the natural calamities.

1.2 Types of Structural Irregularities: There are various types of irregularities in the buildings depending upon their location and scope, but mainly, they are divided into two groups- plan and vertical irregularities. In the present paper, the irregularities are considered and described as follows.
Plan Irregularities: According to clause 7.1 from Sixth revision of IS 1893-2002 (Part 1). Plan irregularities are classified as torsion irregularity, re-entrant corners, floor slabs having excessive cut-outs or openings, out-of-plane offsets in vertical elements and non-parallel lateral force system.
Torsion Irregularity: A building is said to be torsionally irregular, when maximum horizontal displacement of any floor in the direction of the lateral force at one of the floor is more than 1.5 times its minimum horizontal displacement at the far end in that direction
Vertical Irregularities: According to clause 7.1 from Sixth revision of IS 1893-2002 (Part 1). Vertical irregularities are classified as mass irregularity, vertical geometrical irregularity, stiffness irregularity, In-plane Discontinuity in Vertical Elements Resisting Lateral Force and
Mass Irregularity: Mass irregularity shall be considered to exist, when the seismic weight of any floor is more than 150 percent of that of its adjacent floors. This provision of 150 percent may be relaxed in case of roofs.
Vertical Geometric Irregularity: Vertical geometric irregularity shall be considered to exist, when the horizontal dimension of the lateral force resisting system in any storey is more than 125 percent of that in its adjacent storey

1.3 Importance of Seismic Analysis

The prediction of the response of a structure to a particular type of loading is of utmost importance for the design of structure. Basically the codes and previous experiences provide us with a lot of information regarding the type of loads and their intensities for different types of structure and the site condition. The analysis procedure to be adopted purely depends upon the engineers choice as per the accuracy of the work required. The non-linear time history method analysis can be regarded as the most accurate method of seismic demand prediction and performance evaluation of structures. A non-linear dynamic analysis or inelastic time history analysis describes the actual behavior of the structure during an earthquake. The method is based on the direct numerical integration of the motion differential equations by considering the elasto-plastic deformation of the structure element. This method captures the effect of amplification due to resonance, the variation of displacements at diverse levels of a frame, an increase of motion duration and a tendency of regularization of movements result as far as the level increases from bottom to top. Seismic analysis is a subset of structural analysis and is the calculation of the response of a building to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment in regions where earthquakes are prevalent.

Equivalent Static Analysis: Linear static analysis or equivalent static analysis can only be used for regular structure with limited height. All design against seismic loads must consider the dynamic nature of the load. However, for simple regular structures, analysis by equivalent linear static methods is often sufficient. This is permitted in most codes of practice for regular, low-to medium-rise buildings. It begins with an estimation of base shear load and its distribution on each story calculated by using formulas given in the code. The base shear is the total horizontal force on the structure which is calculated on the basis of structure mass and fundamental period of vibration and corresponding mode shape.

Response Spectrum Analysis: It is a representation of the maximum response of idealized single degree freedom system having certain period and damping, during earthquake ground motions. The maximum response plotted against undamped natural period, for various damping values, can be expressed in terms of maximum absolute acceleration, maximum relative velocity or maximum relative displacement. For this purpose, response spectrum case of analysis has been performed according to IS 1893. In the calculation of structural response (whether modal analysis or otherwise), the structure should be so represented by means of an analytical or computational model that reasonable and rational results can be obtained by its behavior.

Non-linear Static Analysis: This method is also known as push-over analysis. This method is an improvement over the linear static and dynamic analysis in the sense that it allows the inelastic behavior of the structure. The method still assumes a set of static incremental lateral load over the height of the structure. It provides information on the strength, deformation and ductility of the structure and the distribution of demands. These permits to identify critical members likely to reach limit states during the earthquake, for which attention should be given during design and detailing process. The pushover analysis of a structure is a static nonlinear analysis is a static nonlinear analysis under permanent vertical loads and gradually increasing lateral loads.

Non-linear Dynamic Analysis: A non-linear dynamic analysis or inelastic time history analysis describes the actual behavior of the structure during an earthquake. The method is based on the direct numerical integration of the motion differential equations by considering the elasto-plastic deformation of the structure element. This method captures the effect of amplification due to resonance, the variation of displacements at diverse levels of frame, an increase of motion duration and a tendency of regularization result as far the level increases from bottom to top.

1.4 Need of the Study: Structural eccentricity is a useful parameter which is responsible to co-relate the seismic elastic response of asymmetric structures. When the structural system is excited into the inelastic range, yielding of the resisting elements complicates the behaviour. Therefore there is a need to study this parameter that captures the inelastic response of the structure.

1.5 Objectives of the Study: Based on the study, the following objectives are drawn:
• To study the variation in torsion by minimizing the stiffness eccentricity.
• To study the effect of stiffness variation on joint rotation of the structure.
• Comparison of torsion and joint rotation before and after varying the stiffness eccentricity.

1.6 Scope of the Study: The present work aims at an objective to identify the non-linear response of asymmetrical structures with respect to stiffness assignments. The buildings studied in this section are asymmetrical buildings of 12 storeys, 15 storeys and 18 storeys with same columns sizes subjected to gravity loads and seismic loads using non-linear dynamic analysis. The structure is evaluated in accordance with IS: 456-2000 and seismic code IS: 1893-2002 using non-linear time history method with the help of ETABS.

1.7 Organization of Dissertation: Chapter 1 is the discussion of the importance of earthquake and torsional behaviour of buildings, types of structural irregularities and introduction to importance of seismic analysis. The scope and objective of the study has also been discussed.
Chapter 2 deals with the various literatures that have been published on effect of irregularities on buildings.
Chapter 3 covers the complete study on analysis procedures and procedure adopted in the present work.
Chapter 4 completely take care of the case study of building under consideration and the various building data surveys done to gather the information for modelling of the structure.
Chapter 5 copes with the numerical study and presentation of results of non linear dynamic analysis of for the current buildings under study

Chapter 6 details of discussions drawn based on the present work and the scope of the study.

II. LITERATURE REVIEW

The general philosophy for earthquake-resistant design of structures has undergone some major change in the past 15 years, following some of the most devastating earthquakes all over the world. The prediction of the earthquake response of a structure became more significant for the engineers to design the structures and this became more significant for the engineers to design the structures and this became much easier with the availability of seismic data and software enhancement. Newer analysis methodologies are being proposed with focus on a realistic characterization of seismic structural damage and its direct incorporation in the design methodology. In addition, a major emphasis is given to the characterization of all the uncertainties in the process of design. In general, these approaches are categorized under performance-based seismic design (PBSD). The various ways of modelling structural damage for PBSD lead to various design approaches. The most commonly adopted approach for PBSD so far is the displacement based design approach, where the design criterion is set usually by a limit on the peak roof (inelastic) displacement, the peak (inelastic) inter-story drift, or the peak ductility demanded. Generally when the earthquake hits a structure, before actual failure of the building, it passes both the linear and nonlinear stages. The linear analysis has been the source of interest of the researchers as mentioned before, but the non-linearity of the structure has now taken the stage. The strength of structure in the non-linear yield is significant for the structures and could be utilized with application of several limit states for a monitored performance and damage. Most buildings have some degree of irregularity in the geometric configuration or the distribution of mass, stiffness or strength.

2.2 Non-Linear Dynamic Procedures: The response of buildings to earthquakes is a complex, three dimensional, nonlinear, dynamic problems. Limitations in technology and the depth of our understanding of this problem have lead to the profession developing a number of simplified methods for representing it, most of which disregard one or more of its fundamental aspects: the Linear Dynamic Procedure (LDP) ignores nonlinearity; the Nonlinear Static Procedure (NSP) ignores dynamic effects; the Linear Static Procedure (LSP) ignores both. In contrast, the Nonlinear Dynamic Procedure (NDP) attempts to fully represent the seismic response of buildings without any of these major simplifying assumptions.

2.3 Overview: Moehle (1984) studied the seismic response of four irregular reinforced concrete test structures. These test structures were simplified models of 9-story 3 bay building frames comprised of moment frames and frame-wall combinations. Irregularities in the vertical plane of these structures were introduced by discontinuing the structural wall at various levels. Based upon measured displacements and distributions of storey shears between frames and walls, it was apparent that the extent of the irregularity could not be gauged solely by comparing the strengths and stiffness's of adjacent stories in a structure. Structures having the same stiffness interruption, but occurring in different stories didn't perform equally. It was observed that the curvature ductility demand in beams varied from 3.9 to 7.2 and for columns from 1.8 to 2.9, for an abrupt termination of shear walls at different levels along the height.

Han Seon Lee and Dong Woo Co (2002) they studied the seismic response of high rise RC wall buildings with different irregularities. In this study they have consider three models with three different cases. First model has a symmetrical moment resisting frame, second has in-filled shear wall at the central frame and third one is with in-filled shear walls in most exterior frames at the bottom two storey.

Dubey and Sangamnerkar (2011) have studied torsional response due to plan and vertical irregularity to analyze T-shaped building, while earthquake forces acts and to calculate additional shear due to torsion in the columns, and concluded that additional shear due to torsional moments needs to be considered because, this increase in shear forces causes columns to collapse, so in design procedure this additional shear must be taken into account.

Gupta and Pajagade (2012) have presented a review about the investigation done on torsional behaviour of multi-storey buildings with plan as well as vertical irregularities. It also focused on codal provision made for torsion.

2.4 Critical Appraisal: Despite a lot of researches being done in this field of non linear dynamic analysis, a few shortcomings in the available literature have been observed such as Dubey and Sangamnerkar, having carried out the work on T-shaped building with vertical geometrical irregularity, have not considered the torsion irregularity and mass irregularity. Similarly, Sagar and Gururaj have studied the parameters like deflection, story displacement response due to plan and vertical irregularity for cross shape, + and T shape buildings. But they have not considered the torsion parameter. Gowthami et al. carried out work on torsional effect on buildings with re entrant corners but they have not performed the work on vertical irregularities. Hence, the proposed study concentrates on torsional performance of buildings with combined plan, vertical and mass irregularity that are subjected to earthquake.

III. METHODOLOGY

3.1 The purpose of the non-linear dynamic analysis is evaluating non-linear response of the structure with respect to torsion. Nonlinear dynamic analysis utilizes the combination ofground motion records with a detailed structural model, therefore is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square-root-sum-of-squares.

3.2 Analysis Procedure

Already the methods for the assessment of the vulnerability of building based on score assignment are rather elaborate and therefore time consuming. More sophisticated methods, implying a more detailed analysis method and more refined models, take even more times and serve therefore for evaluation of individual buildings only, possibly as a further step after the rapid screening of potential hazardous buildings in multi phase procedure. They are not suitable for the earthquake scenario projects where a large number of buildings have to be evaluated. Nevertheless, the concept behind those methods can be valuable for the development of a new simple method and hence, the main analysis procedure shall be briefly outlined. The analysis procedure can be divided into linear procedure (linear static and linear dynamic) and non linear procedure (non-linear static and nonlinear dynamic).

3.2.1 Linear Static Analysis: In a linear static analysis procedure, the building is modelled as an equivalent single degree of freedom (SDOF) system with the linear elastic stiffness and an equivalent viscous damping. The seismic is modelled by an equivalent lateral force with the objective to produce the same stress and strain as the earthquake it represents. Based on an estimation of the first fundamental frequency of the building using empirical relationships or Rayleigh's method, the spectral acceleration is determined from the appropriate response spectrum which, multiplied by the mass of the buildings, results in the equivalent lateral force. The coefficient take into account not only issue like second order effect, stiffness degradation, but also force reduction due to anticipated inelastic behaviour. The lateral force is then distributed over the height of the building and the corresponding internal force and displacements are determined using linear static analysis. These linear static procedures are used primarily for design purpose and are incorporated in most codes. Their expenditure is rather small. However, their applicability is restricted to regular buildings for which the first mode of vibration is predominant.

3.2.2 Linear Dynamic Analysis: In a linear dynamic procedure the building is modelled as a multi degree freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix.

$$V = Sa * m * \sum Ci \qquad (3.1)$$

The seismic input is modelled using spectral analysis or time history analysis. Modal spectral analysis assumes that the dynamic response of a building can be found.

3.2.4 Non-Linear Dynamic Analysis: In a non-linear dynamic analysis procedure, the building model is similar to the one used in nonlinear static procedure incorporating directly the inelastic, lateral response using general finite elements. The main difference is that the seismic input is modelled using a time-history analysis which involves time-step-by-time-step evaluation of building response. This is the most sophisticated analysis procedure for predicting force and displacement under seismic input. However, the calculated response can be very sensitive to the characteristic of the individual ground motion used as seismic input therefore several time-history analysis are required using different ground motion records.

3.3 Non-Linear System: The force-deformation relationship in the structure after a deformation point becomes non-linear. This point is called as yield deformation, because at this point the yielding begins. On initial loading this yielding system is linearly elastic as long as the force is less than fy (i.e. the stiffness is zero). fy generally is called as the yield strength. The structure now enters into the plastic zone and it exhibits elasto-plastic behaviour. Now the deformation increases at constant force. The pattern of cyclic loading unloading and reloading continues till the material deformation comes to rest. This implies that the force Fs corresponding to deformation u is not single valued and depends on the history of the deformations and on whether the deformation is increasing (positive velocity) or decreasing (negative velocity) as shown in Fig. 3.1

3.4 Governing Equation of Motion for SDOF and MDOF System

$$M\ddot{u} + c\dot{u} + f_s(u,\dot{u}) = -m\ddot{u}_g \qquad (3.2)$$

Where $f_s(u,\dot{u})$ is the resisting force for an elasto-plastic system

M is the mass of the structure

C is the damping coefficient

The function $f_s(u,\dot{u})$ describes the force-deformation relation in partially dimensionally form. Numerical methods are discussed in the subsequent topics to solve for the single degree of freedom system equation of motion. Similarly the equation of motion for MDOF system subjected to external force or earthquake excitation for inelastic systems:

$$[M]\ddot{u} + [C]\dot{u} + [K]u = -1m\ddot{u}_g(t) = p_{eff}(t) = -s\ddot{u}_g(t) \qquad (3.3)$$

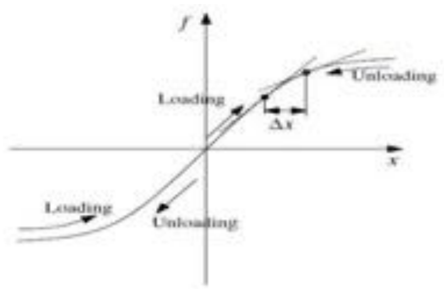


Fig. 3.1 General Non-Linear Force-Deformation Relationships

Where

[M] = Mass matrix of the MDOF system.

[C] = Damping matrix of the MDOF system.

[K] = Stiffness matrix of the MDOF system.

3.5 Centre of Mass and Centre of Rigidity: Centre of mass is the point where entire mass of the system is concentrated. During an earthquake, acceleration induced inertia forces will be developed at each floor level, where the mass of the entire storey may be assumed to be calculated. Hence the location if particular level will be determined by the centre of accelerated mass at that level. The coordinates of centre of mass is given by

XCM = ∑wxi xi/ ∑ wi

YCM = ∑wxi yi/ ∑ wi

Centre of rigidity is the point that position of storey shear which will cause only relative floor translation. It also referred as centre of stiffness of the system. The coordinates of centre of stiffness is given by

XCR = ∑ kyi xi/ ∑ki

YCR = ∑kxi yi/ ∑ ki

3.6 Development of Torsion in Buildings: When a building is subjected to seismic excitation, horizontal inertia forces are generated in the building. The resultant of these forces is assumed to act through the centre of mass of the structure. The vertical members in the structure resist these forces and the total resultant of these systems of forces act through a point called as centre of stiffness (C.S). When the centre of mass and centre of stiffness does not coincide, eccentricities are developed in the buildings which further generate torsion. The following Fig. 3.2 showsthe development of torsion in buildings.

e

.CM .CR

Fig. 3.2 Development of Torsion in Buildings

3.7 Eccentricity: Stiffness Eccentricity is defined as a system with non-coincident centre of mass and centre of stiffness. When such system is subjected to dynamic excitations (like earthquake, wind), the inertia force can be modelled as acting through centre of mass while the resultant of resisting forces respond through the centre of stiffness. This creates a moment between the two opposing forces, resulting into a torsional effect. Strength eccentricity is defined as the distance between centre of mass and centre of strength for a considering floor.

V. CASE STUDY DESCRIPTION

4.1 Models of the Buildings: Model 1(a) is an U shaped 12 storied building which is asymmetric in X direction and symmetric in Y direction in plan with vertical irregularity and mass irregularity, all column sizes are 750X300mm, except C2&C3 for first storey and beam sizes are 500X300mm. Model 1(b) is an 15 storied building which is asymmetric in X direction and symmetric in Y direction in plan with vertical irregularity and mass irregularity, all column sizes are 750X300mm, except C2&C3 for first storey and beam sizes are 500X300mm. Model 1(c) is an 18 storied building which is asymmetric in X direction and symmetric in Y direction in plan with vertical irregularity and mass irregularity, all column sizes are 750X300mm, except C2&C3 for first storey and beam sizes are 500X300mm.

Model 2(a) is floors having excessive cut-outs or openings having 12 storied building with openings and symmetric in X direction and asymmetric in Y-direction in plan with vertical irregularity and mass irregularity, all columns sizes are 750X300mm and beam sizes are 500X300mm. Model 2(b) is floors having excessive cut-outs or openings having 15 storied building with openings and symmetric in X direction and asymmetric in Y-direction in plan with vertical irregularity and mass irregularity, all columns sizes are 750X300mm and beam sizes are 500X300mm. Model 2(c) is floors having excessive cut-outs or openings having 18 storied building with openings and symmetric in X direction and asymmetric in Y-direction in plan with vertical irregularity and mass irregularity, all columns sizes are 750X300mm and beam sizes are 500X300mm.

4.2 Torsion Irregularity Condition for U Shaped Models:

Model 1(a) is 12 storeys U shaped building:

Δmin= 21.783 mm

Δmax= 34.795 mm

Δmax> 1.5x (Δmin)

Δmax> 1.5x (34.795)

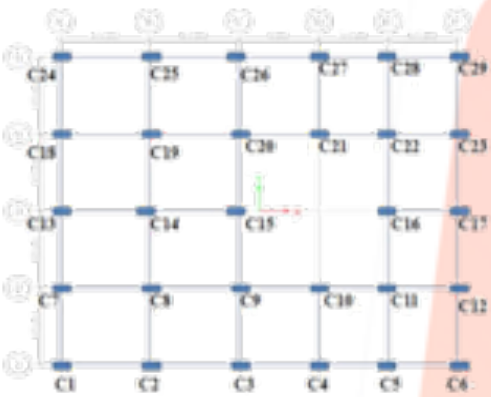
Δmax> 32.674 mm

Model 1(b) is 15 storeys U shaped building:

Δmin= 23.004 mm
Δmax= 38.387 mm
Δmax> 1.5x (Δmin)
Δmax> 1.5x (23.004)
Δmax> 34.506 mm

Model 1(c) is 18 storeys U shaped building:

Δmin = 29.085 mm
Δmax= 50.045 mm
Δmax> 1.5x (Δmin)
Δmax> 1.5x (29.085)
Δmax> 43.627 mm



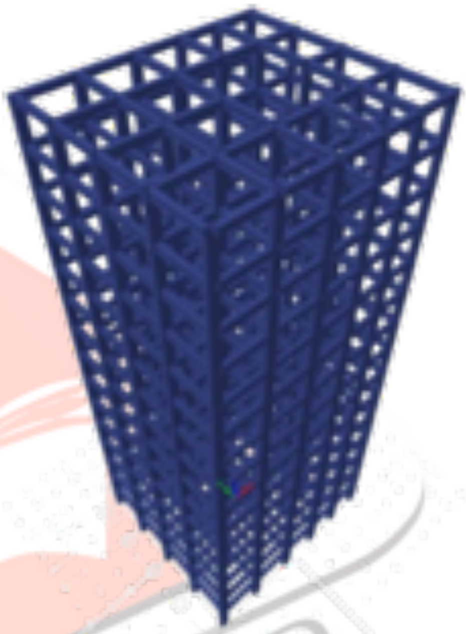


Fig. 4.3 Plan View of Model 2 Columns sizes = 750X300mm Beam sizes = 500X300mm   Fig. 4.4 Isometric View of Model 2

VI. RESULTS AND DISCUSSIONS

The following results are obtained by carrying out the non linear dynamic analysis (FNA) in ETABS to study the combined effect of plan, vertical and mass irregularity on torsion. Subsequent discussions are made about the torsion in buildings by non linear dynamic analysis and reducing torsion with respect to stiffness eccentricity.

5.1 Comparison of Torsion

Model	Number of storeys	Eccentricity	Torsion (kN-m)
Model 1(a)	12	1.776	5825.631
Modified model 1 (a)	12	0.639	1588.826

Table 5.1 Results of Eccentricity and Torsion for Model 1(a) and Modified model 1(a)

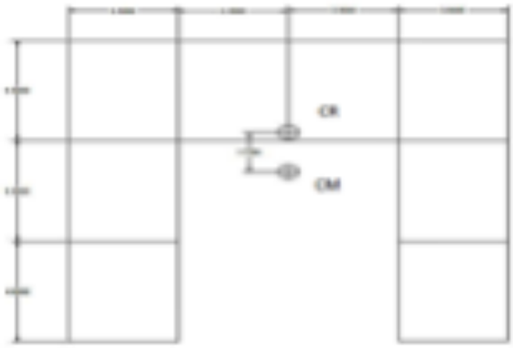


Fig. 5.1 Position of Centre of Mass and Centre of Rigidity for Model 1(a)

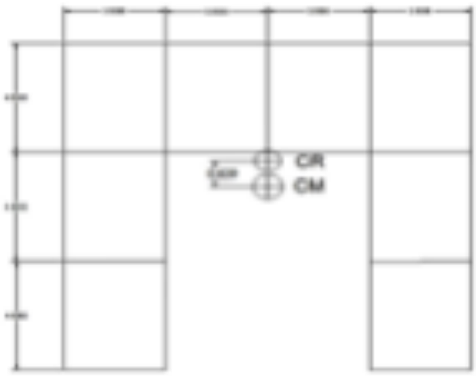


Fig. 5.2 Position of Centre of Mass and Centre of Rigidity for Modified Model 1(a)

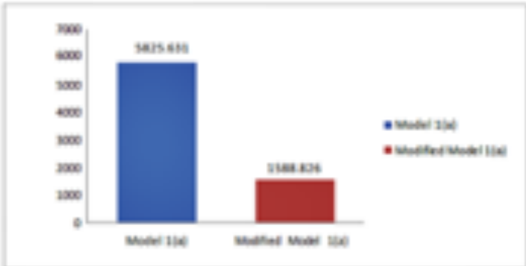


Fig. 5.3 Results of Torsion for Model 1(a) and Modified model 1(a)

It is observed that the variation of torsion for 12 storey U shaped building is decreased by 72.72% after varying the stiffness parameters as shown in Fig. 5.3

Model	Number of storeys	Eccentricity	Torsion (kN-m)
Model 1(b)	15	2.058	7466.670
Modified model 1(b)	15	1.046	1749.878

Table 5.2 Results of Eccentricity and Torsion for Model 1(b) and Modified Model 1(b)

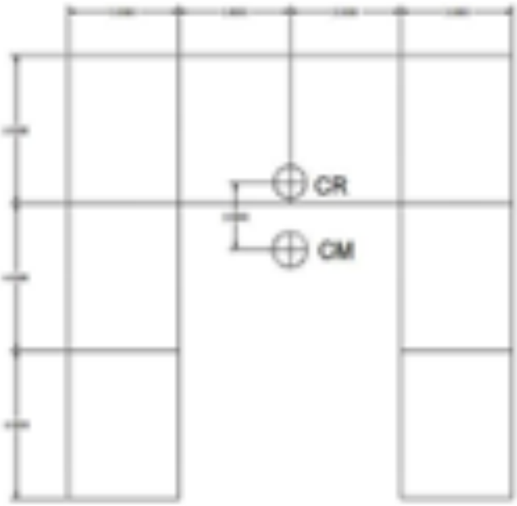


Fig. 5.4 Position of Centre of Mass and Centre of Rigidity for Model 1(b)

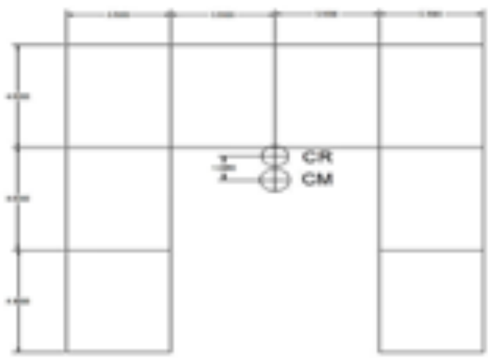


Fig. 5.5 Position of Centre of Mass and Centre of Rigidity for Modified Model 1(b)

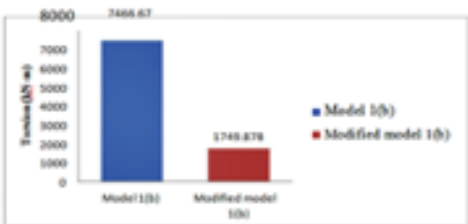


Fig. 5.6 Results of Torsion for Model 1(b) and Modified Model 1(b)

It is observed that the variation of torsion for 15 storey U shaped building is decreased by 76.56% after varying the stiffness parameters as shown in Fig. 5.6

Model	Number of storeys	Eccentricity	Torsion (kN-m)
Model 1(c)	18	2.32	9344.246
Modified model 1(c)	18	1.39	2566.915

Table 5.3 Results of Eccentricity and Torsion for Model 1(c) and Modified Model 1(c)

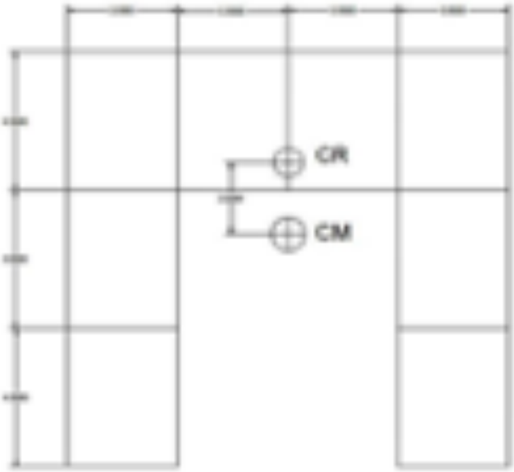


Fig. 5.7 Position of Centre of Mass and Centre of Rigidity for Model 1(c)

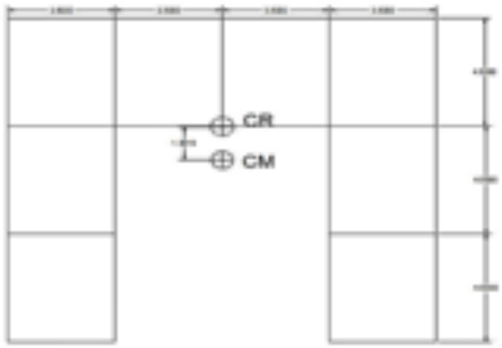


Fig. 5.8 Position of Centre of Mass and Centre of Rigidity for Modified Model 1(c)

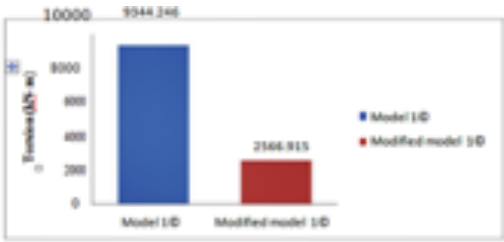


Fig. 5.9 Results of Torsion for Model 1(c) and Modified Model 1(c)

It is observed that the variation of torsion for 18 storey U shaped building is decreased by 72.53% after varying the stiffness parameters as shown in Fig. 5.9

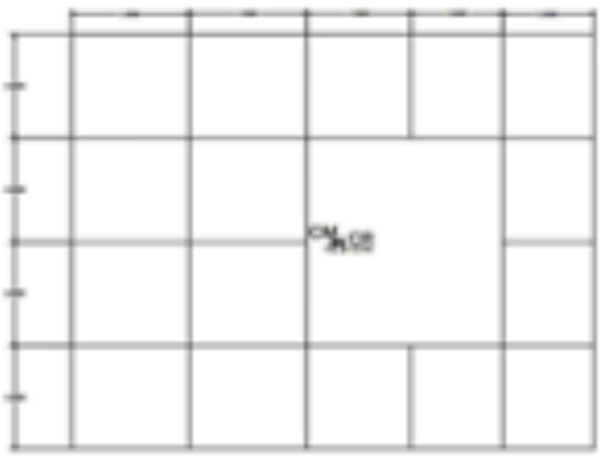


Fig. 5.10 Position of Centre of Mass and Centre of Rigidity for Model 2(a)

Fig. 5.11 Position of Centre of Mass and Centre of Rigidity for Modified Model 2(a)

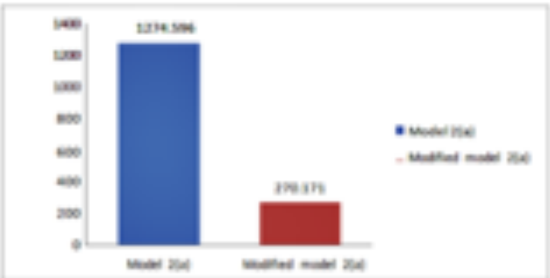


Fig. 5.12 Results of Torsion for Model 2 and Modified Model 2(a)

It is observed that the variation of torsion for 12 storey building with openings is decreased by 78.80% after varying the stiffness parameters as shown in Fig. 5.12

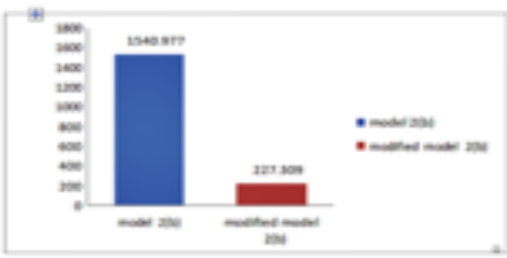


Fig. 5.15 Results of Torsion for Model 2(b) and Modified Model 2(b)

It is observed that the variation of torsion for 15 storey building with openings is decreased by 85.23% after varying the stiffness parameters as shown in Fig. 5.15

Model	Number of storeys	Eccentricity	Torsion (kN-m)
Model 2(c)	18	0.258	1678.938
Modified model 2(c)	18	0.017	162.205

Table 5.6 Eccentricity and Torsion Values of Model 2(c) and Modified Model 2(c)

Fig. 5.17 Position of Centre of Mass and Centre of Rigidity for Modified Model 2(c)

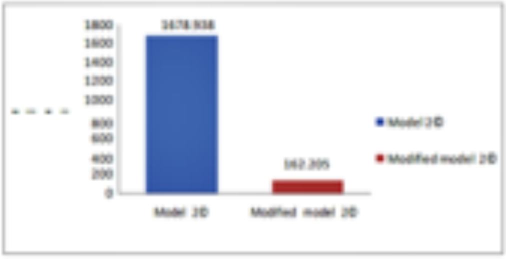


Fig. 5.18 Results of Torsion for Model 2(c) and Modified Model 2(c)

It is observed that the variation of torsion for 18 storey building with openings is decreased by 90.33% after varying the stiffness parameters as shown in Fig. 5.18

5.2 Comparison of Joint Rotation

Model	Number of storeys	Rotation at a joint (Rz)	
		NLTH_X Max(x10^{-3})	NLTH_Y Max
Model 1(a)	12	4.512	0
Modified model 1(a)	12	0.727	0

Table 5.7 Results of Joint Rotation of Model 1(a) and Modified Model 1(a)

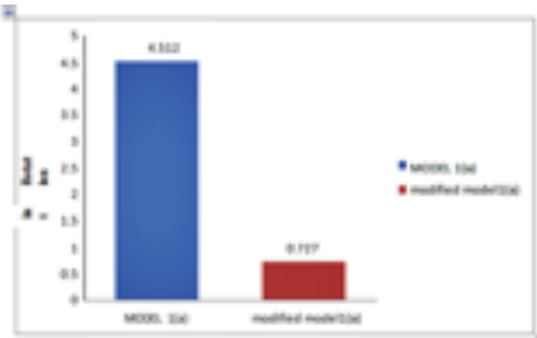


Fig. 5.19 Joint rotation Variation for Model 1(a) and Modified model 1(a)

It is observed that the variation of joint rotation for 12 storey U shaped building is decreased by 83.88% after varying the stiffness parameters as shown in Fig. 5.19

Model	Number of storeys	Rotation at a joint (Rz)	
		NLTH_X Max(x10^{-3})	NLTH_Y Max
Model 1(b)	15	7.476	0
Modified Model 1(b)	15	3.119	0

Table 5.8 Results of Joint Rotation for Model 1(b) and Modified Model 1(b)

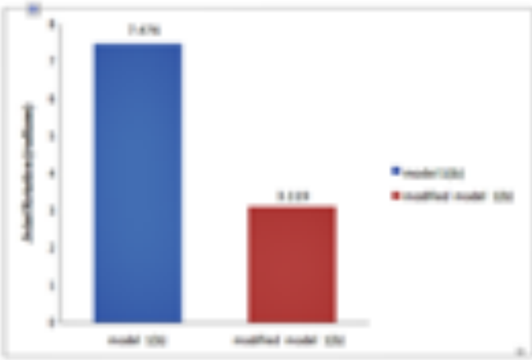


Fig. 5.20 Variation of Joint Rotation for Model 1(b) and Modified model 1(b)

It is observed that the variation of joint rotation for 15 storey U shaped building is decreased by 58.27% after varying the stiffness parameters as shown in Fig. 5.20

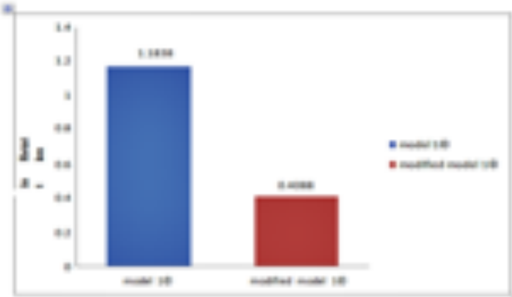


Fig. 5.21 Variation of Joint Rotation for Model 1(c) and Modified Model 1(c)

It is observed that the variation of joint rotation for 18 storey U shaped building is decreased by 65.46% after varying the stiffness parameters as shown in Fig. 5.21

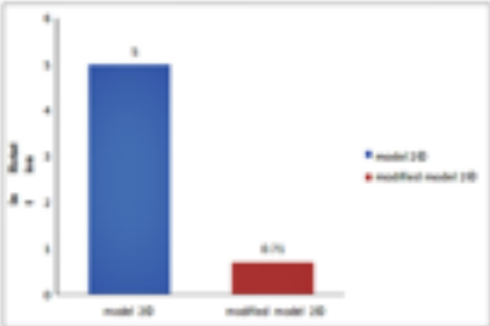


Fig. 5.24 Variation of Joint Rotation for Model 2(c) and Modified Model 2(c)

It is observed that the variation of joint rotation for 18 storey building with openings is decreased by 85.81% after varying the stiffness parameters as shown in Fig. 5.24

VII. CONCLUSION

The following conclusions drawn from the present work:

- Torsion percentage in asymmetrical buildings is significantly decreased by minimizing the stiffness eccentricity. Torsion reduces to 72%, 76% and 72% for 12 storey U shaped building, 15 storeys U shaped building and 18 storeys U shaped building.
- Percentage of torsion in asymmetrical buildings is decreased by minimizing the stiffness eccentricity. Torsion reduces to 79%, 85% and 90% for 12 storey building with openings, 15 storey building with openings and 18 storey building with openings.
- Percentage of joint rotation is significantly reduced by varying the stiffness parameters. Joint rotation reduced to 84%, 58%, and 65% for 12 storey U shaped building, 15 storeys U shaped building and 18 storeys U shaped building.
- Percentage of joint rotation is significantly reduced by varying the stiffness parameters. Joint rotation reduced to 78%, 75 %, and 86% for 12 storey building with openings, 15 storey building with openings and 18 storey building with openings.
- Thereby a maximum decrease in torsion is 90% for 18 storey building with openings. And maximum decrease in joint rotation is 86% for 18 storey building with openings.

6.3 Scope for Further Study

As the various researches are getting attracted towards the computer software, the scope of the studies under the particular topic can be done using different seismic zones and also using different earthquake data and effect of bracing systems on torsion for buildings with combined plan, stiffness and mass irregularity.

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