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Full Length Research Paper

Dynamic behavior of torsionally sensitive reinforced concrete framed structures

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Torsional irregularity leads to increased unequal displacements at the extremes of the building and may cause distress in the lateral load resisting elements at the edges. Torsional irregularity is caused by plan asymmetry, asymmetrical distribution of mass and stiffness, asymmetrical configuration of the lateral load resisting systems. In the present study, a model G+2 storey was considered to analyze the torsional behavior of the structure subjected to base excitation using shake table test setup. A physical model was constructed and tested to study its torsional behavior under seismic loading. A similar model was developed and analyzed using SAP-2000 software. From the experimental test results, it was observed that, three translational modes and two torsional modes of vibrations were found to exist in the structure.

Key words: Dynamic loading, Torsion, stiffness, symmetrical structures, shake table.

INTRODUCTION

Torsional effects are the catastrophic occurrences in many multi-storied buildings during strong earthquakes and have a major influence for the collapse of structure. This led to the inclusion of torsional effects in existing codes and modification of the existing codal provisions to include the torsional effects. It has been a real challenge to reduce the inelastic translational displacements of critical elements of a storey during twisting moment. There had been good research studies to analyze the torsional effects of multistoried structures and discussed further in detail.

The methodology for modeling the inelastic torsional response of buildings in nonlinear static (pushover) analysis enables reproduction to the highest possible degree the results of inelastic dynamic time history analysis. The load vectors were defined using dynamic elastic spectral analysis while the dynamic characteristics of an equivalent single mass system, which incorporates both translational and torsional modes, were derived using an extension of earlier methods based on the single-degree-of-freedom (SDOF) approach. A superposition-based analysis procedure was proposed by Basu and Jain (2004) 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 and extended the definition of center of rigidity for rigid floor diaphragm buildings to un-symmetrical buildings with flexible floors. The proposed approach was applicable to orthogonal as well as non orthogonal unsymmetrical buildings and accounts for all possible definitions of center of rigidity.

Gluck et al. (1975) modeled a three-dimensional reinforced concrete framed building using finite element method. Two types of elements, the beam-column element and flat shell element were used for modeling the frame and floor slabs, respectively. A computer program had been developed for the analysis of 3D framed building by integrating the finite element and stiffness method. The lumped inelasticity model with three-dimensional point hinges at the ends of the beam-column element was implemented. A single storey one bay reinforced concrete space frame was analyzed for twist loading to study the inelastic response of the reinforced concrete frame. The results indicated that the consideration of torsion in defining the yielding surface plays a significant role in the inelastic behavior and estimation of failure load for reinforced concrete frames

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under torsional loading.

Silvia and Ilia (2003), presented an analytical estimation of the dynamic effects, caused by shifting of the centre of mass with accidental eccentricity in symmetric structures. The approximate analytical solution proves, that even under small accidental eccentricities the symmetric structures exhibit “irregular behavior” and the accidental torsional effects cannot be described properly by static application of torsional moments. The prescribed application rule by Eurocode 8 for multimodal analysis underestimates the accidental torsional effects up to 21% for 5% eccentricity for the structures considered in the paper. An expression for the correction of member responses was derived. It is proved by numerical simulations of the dynamic response of three-dimensional models of symmetric structures, that the proposed correction coefficient gives accurate results in cases of single-storey and multi-storey structures. It gives a convenient way for the design practice to estimate accurately the accidental torsional effects on planar and 3-D models of symmetric structures.

Han-Seon and Dong-Woo (2007), modeled three 1:12 scale 17-story RC wall building models having different types of irregularity at the bottom two stories and was subjected to the same series of simulated earthquake excitations to observe their seismic response characteristics. The first model has a symmetrical moment-resisting frame (Model 1), the second has an infilled shear wall in the central frame (Model 2), and the third has an infilled shear wall in only one of the exterior frames (Model 3) at the bottom two stories. The estimated fundamental periods for other structures than moment frames and bearing wall structures in UBC 97 and AIK 2000 appeared to be reasonable. The total amount of energy absorption by damage was similar regardless of the existence and location of the infilled shear wall. The largest energy absorption was due to overturning, followed by that due to shear deformation. The rigid upper system renders rocking behavior in the lower frame, and thereby, the self weight contributes up to about 23% of the resistance against the total overturning moment.

Reinhorn et al. (1977), proposed an approximate method for the dynamic analysis of torsionally coupled tall building structures by utilizing the properties of their uncoupled counterparts. An exact solution was first given for the particular case in which the lateral and torsional stiffness matrices were uncoupled by same transformation. The method was then applied to a wider class of structures where that condition was only approximately satisfied by reducing the dynamic coupling problem to an approximate two-degrees-of-freedom system. Simple formulae and graphical representations of dynamic magnification of static eccentricity were given. Anil (1994) presented a procedure for evaluating building-code provisions for accidental torsion from analysis of earthquake-induced motions of nominally symmetric-plan buildings. This procedure was used to analyze the motions of three buildings recorded during recent California earthquakes. Two alternative approaches to evaluate the code accidental torsion provisions were developed. The first one considers the response histories of base shear and base torque in the building, and the second, the "actual" forces in the structural elements during the earthquake. The results showed that base rotational motions cause between 25 and 45% of the total accidental torsion in the buildings. They also demonstrate that the accidental torsional moments specified by the Uniform Building Code are sufficient in representing the torsion in the recorded motions of the three buildings. Further, it was shown that accidental torsion need not be considered in the design of two of the three buildings studied. The observation, however, may not necessarily apply to other structures, such as buildings with torsional vibration periods much longer than their lateral vibration periods.

SHAKE TABLE TEST PROCEDURE

A model RC frame was placed in the shake table at the required orientation. After that, raft foundation of the frame was tightened with the shake table to make it as a fixed base. Now a reference steel frame (I section) was fixed near the model but just away from the shake table. LVDT and NCDTs are fixed in the steel frame at each floor levels. These are used to measure the linear displacement of the frame for every shaking of the table. Strain gauges which are already fixed in the steel reinforcement are connected to the recorders. Accelerometers are fixed at each floor which is used to record the acceleration values. Seismic recorders are placed at each floor which is used to record the acceleration of the floor for each excitation. After all the instrumentation setup is over, all features are checked by giving a small shaking to the RC frame. The acceleration data in the form of charge is transferred into voltage by compatible signal conditioners. This voltage was sent to the 2-band FFT – analyzer (Fast – Fourier – Transformation). FFT- analyzer computes the required displacement from the applied voltage signal. The calculated displacement was given to the actuators which are located to the lower part of shake table. These actuators are moved from its original position to achieve its input and this is called the table excitation. All the displacements, acceleration values are measured along the direction of table excitation only. Dynamic strain values for all the columns are received from the strain gauge and stored to the computer. Acceleration values from all the accelerometers are measured and stored in the computer. Displacement values from all the LVDT and NCDTs were recorded to the computer.

Free vibration test

The force is given as an impact and the body vibrates under the action of the applied load. In the free vibration test, the impact is given using an impact hammer and the response is measured in terms of accelerations using pickups which are directly attached to the concrete surface. The impact load is given at various floor levels and the response is measured at different floor levels to get the translational and torsional frequencies of the structure under free vibration.

Translational mode

In order to get the frequencies of translational mode of the structure, the impact was given at the centre of the beam of first floor and the response was measured at the centre of the beam of all the three floor levels. Similarly the impact was given at second and third floor levels and the response was measured at all the floor levels.

Torsional mode

In order to measure the frequencies of the torsional mode of the structure, the impact was given at one end of the beam of first floor and the response was measured at the other end of the beam at all the floor levels. Similarly, the impact was given at the ends of the beam at second and third floor levels and the response was measured at the other end of the beam at all the floor levels.

MODELING IN SAP 2000

The model was created using SAP 2000 as a beam slab model and rigid diaphragm model. Modal analysis was performed for the model to obtain the frequency and mode shapes under earthquake excitation. In the beam slab model, the rigidity of the joints was taken care of by the slab provided. In the rigid diaphragm model, the rigidity was given by providing a rigid diaphragm as shown in Figures 1 and 2.

EXPERIMENTAL TEST RESULTS AND DISCUSSION

Frequencies

Three translational modes and two torsional modes were found for the model by experimental analysis. The frequencies of each mode obtained from analysis and experiments are provided in Table 1.

Acceleration

Base acceleration

Two different base accelerations of 0.2 and 0.132 m/s^2 were given to the base of the model in shake table testing and the frames were tested within the elastic limit. The PGA (in terms of 'g') values of the two elastic-force inducing earthquakes were 0.02 and 0.013 g respectively. A typical acceleration versus time history data for a typical earthquake is shown in Figure 4.

Z-direction acceleration

The acceleration obtained in Z direction for the floor levels are shown in Table 2. For both earthquakes all the columns in the stiffer side move identically. Similarly, flexible side columns also move identically. However, there is a difference of 26.3% between the flexible and stiff side columns for earthquake 1 and 12.12% for earthquake 2. Magnification factors were 3.6 for earthquake 1 and 2.8 for earthquake 2, at the third floor level. (The magnification factor is the ratio of maximum floor acceleration to maximum base acceleration).

X-direction acceleration

Earthquake is applied only in the Z-direction. However due to the torsional coupling, a good cross-axis (X-direction) response is seen. The acceleration obtained in X direction for the floor levels are shown in Table 3. The acceleration in flexible and stiffer sides columns make them move identically and their magnitudes were equal for both earthquake 1 and earthquake 2. Magnification factors obtained were 1.38 for earthquake 1 and 0.56 for earthquake 2. The cross-axis (X direction) acceleration obtained is 25 and 20% of the direct axis (Z direction) acceleration for earthquake 1 and earthquake 2 respectively.

DISPLACEMENTS AND STOREY DRIFTS

The displacements obtained along the Z-direction for the two given earthquakes are shown in Table 4. The displacements obtained show that the stiff side columns have lower value compared with the flexible side columns due to higher stiffness in the columns, for both the earthquakes. The displacements obtained along the X-direction for the two given earthquakes are shown in Table 5. The displacements obtained for stiff side columns and flexible side columns are equal in X direction since there is no excitation along the X direction. The storey drift ratio is the ratio between the inter-storey displacement to the storey height. The inter-storey drift ratios obtained for the floor levels are given in Table 6. The storey drift ratio obtained in level II is higher than the values obtained in levels III and I and the drift ratio profile is shown in the Figure 3. The different mode shapes and torsional mode shape obtained are shown in Figures 5 to 14.

Conclusions

A physical model was constructed and tested to study its torsional behavior under seismic loading. A similar model was developed and analyzed using SAP-2000 software and the results were compared experimentally. From the test results and discussions the following conclusions were drawn.

i) From the analysis and experiments done, three translational modes and two torsional modes of vibrations were found to exist in the structure.
ii) The first bending mode was in the cantilever mode with a frequency of 5.5 Hz. The first torsional mode was found with a frequency of 8.375 Hz.
iii) The model was subjected to two small earthquakes of PGA values (0.025 and 0.01 g). The excitation was given

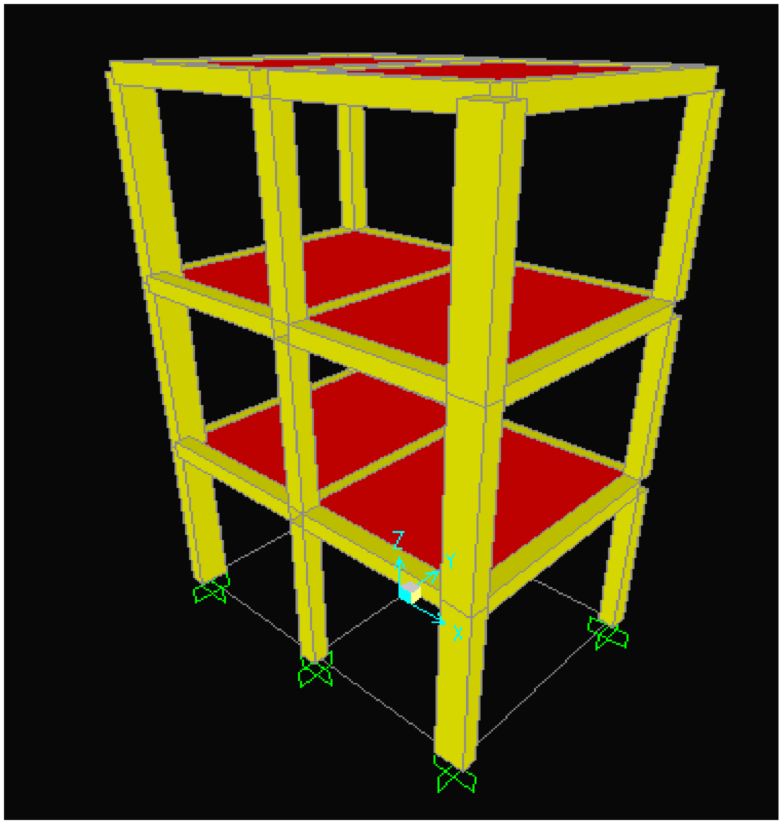

Figure 1. Beam-slab model.

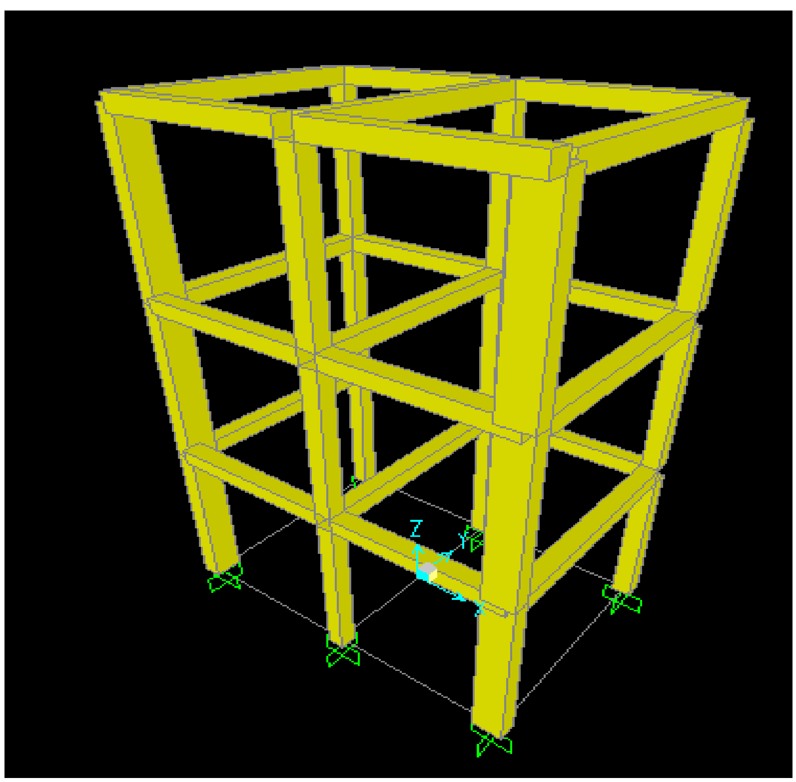

Figure 2. Rigid-diaphragm model.

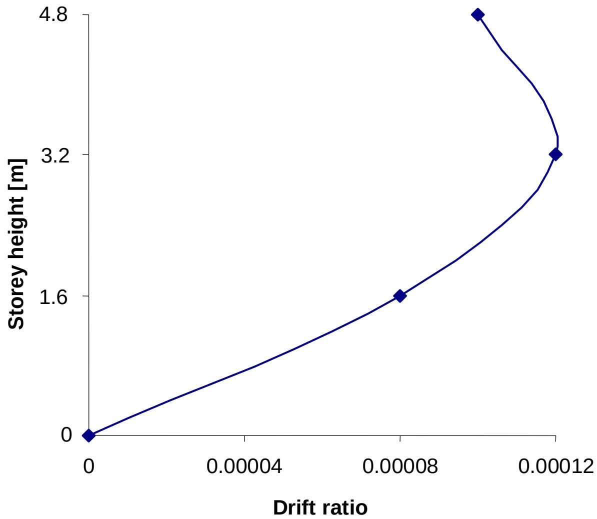

Figure 3. Drift ratio profile.

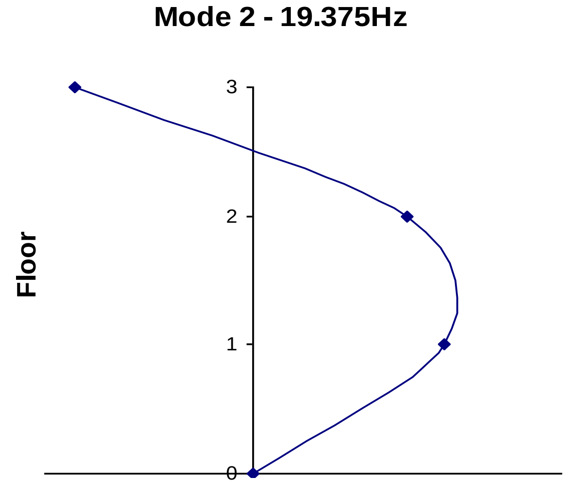

Figure 6. Mode-2.

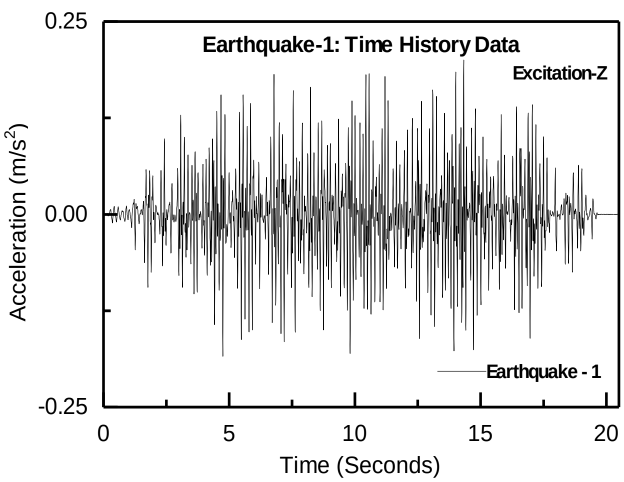

Figure 4. Acceleration Vs Time history data.

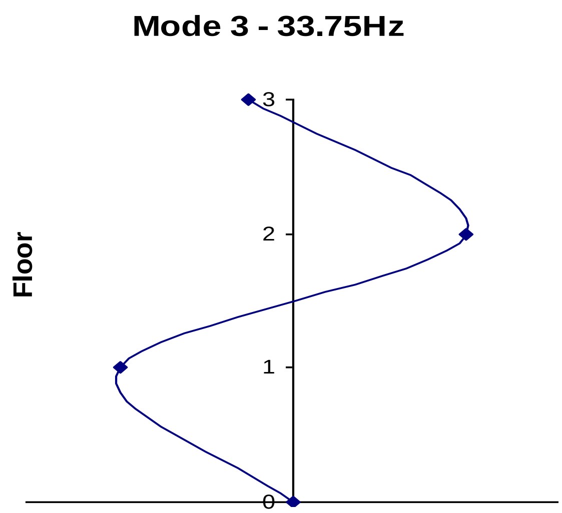

Figure 7. Mode-3.

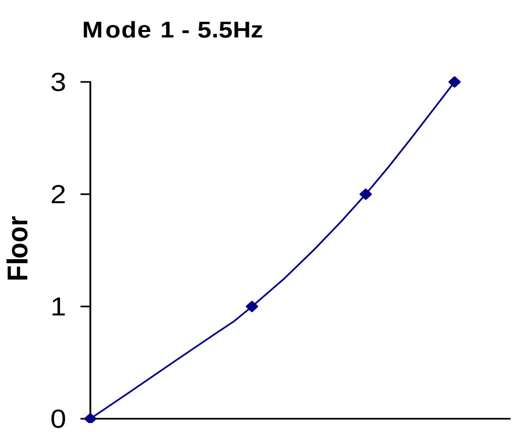

Figure 5. Mode-1.

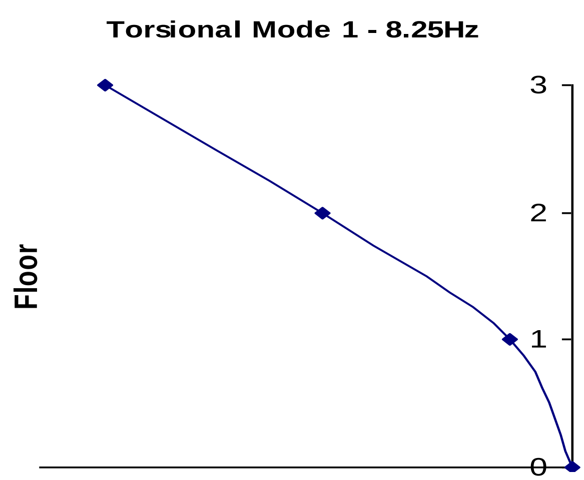

Figure 8. Torsional Mode-1.

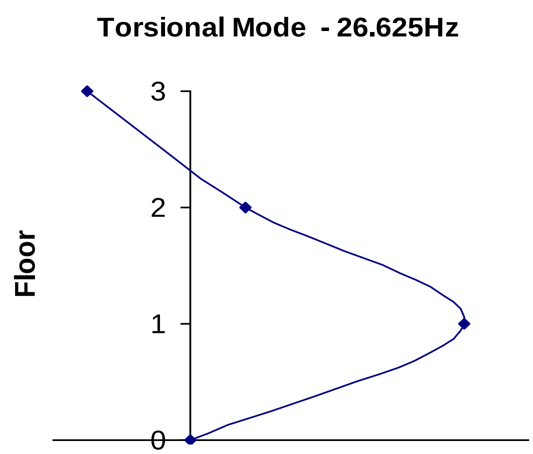

Figure 9. Torsional Mode-2.

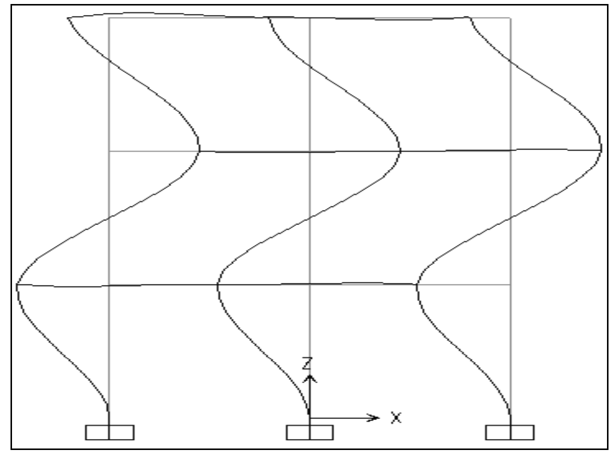

Figure 12. Mode-3.

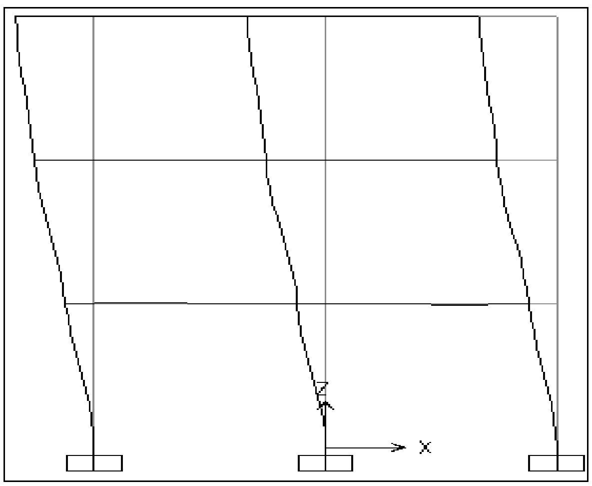

Figure 10. Mode-1.

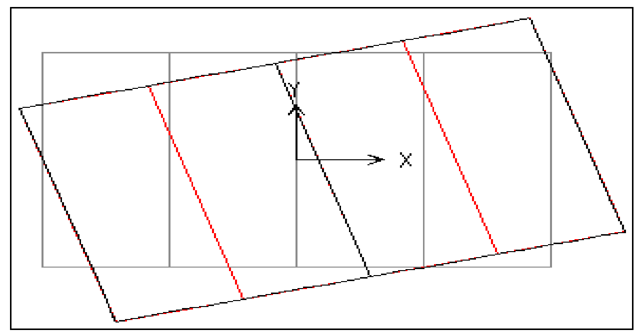

Figure 13. Torsional Mode-1.

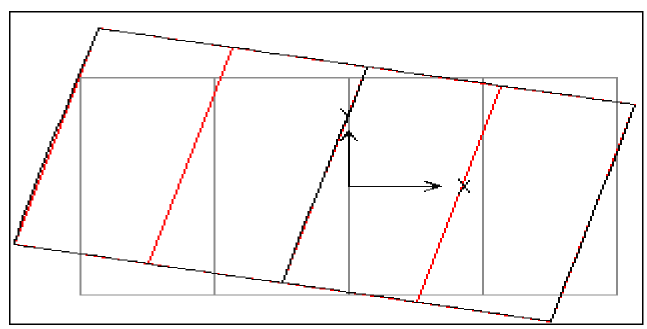

Figure 14. Torsional Mode-2.

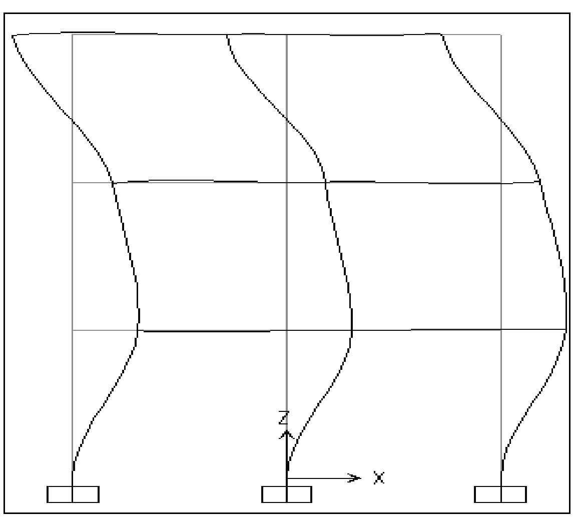

Figure 11. Mode-2.

in the longer direction (Z direction) of the frame. The response was measured on both longer and shorter directions (X and Z directions) of the frame using a LVDT.
iv) The magnification factor (magnification factor = peak acceleration on roof level/ peak base acceleration) for earthquake 1 is 3.6 and earthquake 2 is 2.7.
v) Under earthquake 1, the row of columns in flexible side moves identical in Z direction. The row of columns in stiff

Table 1. Frequency obtained in various modes.

Method	Translational Mode 1	Translational Mode 2	Translational Mode 3	Torsional Mode 1	Torsional Mode 2
Free vibration test	5.5	19.375	33.75	8.25	26.625
Shake table test	5.45	19.53	33.61	8.22	26.755
Beam-slab model	5.43	16.57	27.86	6.00	19.29
Rigid diaphragm model	6.96	22.27	39.90	7.81	25.51

Table 2. Acceleration obtained in different column and floor levels in Z direction.

Earthquake	Columns	Floor III (m/s^2)	Floor II (m/s^2)	Floor I (m/s^2)
EQ1	Stiffer side columns	0.57	0.41	0.24
	Flexible side columns	0.72	0.495	0.30
EQ2	Stiffer side columns	0.33	0.23	0.16
	Flexible side columns	0.37	0.26	0.2

Table 3. Acceleration obtained in different column and floor levels in Z direction.

Earthquake	Columns	Floor III (m/s^2)	Floor II (m/s^2)	Floor I (m/s^2)
EQ1	Stiffer side columns	0.182	0.136	0.076
	Flexible side columns	0.182	0.136	0.077
EQ2	Stiffer side columns	0.074	0.051	0.029
	Flexible side columns	0.074	0.051	0.029

Table 4. Direction displacements.

Earthquake	Columns	Floor III (m)	Floor II	Floor I
EQ1	Stiffer side columns	0.00039	0.00028	0.00012
	Flexible side columns	0.00047	0.00034	0.00015
EQ2	Stiffer side columns	0.00021	0.00015	0.00007
	Flexible side columns	0.00024	0.00017	0.00008

Table 5. X direction displacements.

Earthquake	Columns	Floor III (m)	Floor II (m)	Floor I (m)
EQ1	Stiffer side columns	0.0001	0.00008	0.00004
	Flexible side columns	0.0001	0.00008	0.00004
EQ2	Stiffer side columns	0.00003	0.00002	0.00001
	Flexible side columns	0.00003	0.00002	0.00001

side also moves identical in Z direction. However, there is an acceleration difference of 30% between the flexible and stiff side columns.

vi) Under earthquake 2, the row of columns in flexible side moves identical in Z direction. The row of columns in stiff side also moves identical in Z direction. However, there is an acceleration difference of 30% between the flexible and stiff side columns.

Table 6. Storey drift ratios.

Earthquake	Floor level	Stiffer side column	Flexible side column
Earthquake 1	Level III	0.00004	0.00004
	Level II	0.00006	0.00006
	Level I	0.00005	0.00005
Earthquake 2	Level III	0.00007	0.00008
	Level II	0.00011	0.00012
	Level I	0.00009	0.00010

vii) Since the frame is vertically irregular, it undergoes torsional mode of vibration in direction perpendicular to the direction of excitation.
viii) Under earthquake 1, the row of columns in flexible side moves identical in X direction. The row of columns in stiff side also moves identical in X direction. However, there is an acceleration difference of 30% between the flexible and stiff side columns.
ix) Under earthquake 1, the row of columns in flexible side moves identical in X direction. The row of columns in stiff side also moves identical in X direction. However, there is an acceleration difference of 30% between the flexible and stiff side columns.
x) The maximum displacement under earthquake 1 was 0.4 mm in Z direction and 0.1 mm in X direction. The maximum displacement under earthquake 2 was 0.4 mm in Z direction and 0.1 mm in X direction.
xi) Drift ratio was high for level II and then followed by levels I and III for both earthquakes 1 and 2.
xii) The base shear obtained for earthquake 1 was 3205.15 N, which is 0.45% by weight of the structure. The base shear obtained for earthquake 1 was 3205.15 N, which is 0.45% by weight of the structure.

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