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Shake table tests on a non-seismically detailed RC frame structure

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Abstract. A reinforced concrete (RC) framed structure detailed according to non-seismic detailing provisions as per Indian Standard was tested on shake table under dynamic loads. The structure had 3 main storeys and an additional storey to simulate the footing to plinth level. In plan the structure was symmetric with 2 bays in each direction. In order to optimize the information obtained from the tests, tests were planned in three different stages. In the first stage, tests were done with masonry infill panels in one direction to obtain information on the stiffness increase due to addition of infill panels. In second stage, the infills were removed and tests were conducted on the structure without and with tuned liquid dampers (TLD) on the roof of the structure to investigate the effect of TLD on seismic response of the structure. In the third stage, tests were conducted on bare frame structure under biaxial time histories with gradually increasing peak ground acceleration (PGA) till failure. The simulated earthquakes represented low, moderate and severe seismic ground motions. The effects of masonry infill panels on dynamic characteristics of the structure, effectiveness of TLD in reducing the seismic response of structure and the failure patterns of non-seismically detailed structures, are clearly brought out. Details of design and similitude are also discussed.

Keywords: shake table test; structural engineering; RC structure; dynamic loads; masonry infill panels; tuned liquid dampers; seismic response; nonlinear behaviour; similitude requirements

1. Introduction

Seismic response of reinforced concrete (RC) framed structures has been one of the major topics of interest for past few years among the structural engineering researchers around the world. The main reason behind this interest is the complex behaviour of the RC structures and their vulnerability to earthquakes. Past earthquakes have exposed the vulnerability of RC structures under seismic loads. Two most important and most frequently found deficiencies in old structures are no shear reinforcement in the joint and poor end anchorages leading to joint shear and bond failures respectively. Often such failures lead to partial or complete collapse of structures. Fig. 1 shows near complete collapse of structures due to joint failures during 2004 Sumatra Earthquake (Saatcioglu *et al*. 2004).

Over the years, researchers have investigated the behavior of RC structures under earthquakes and updated the design guidelines so that the structural performance during earthquakes can be improved.

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Fig. 1 Joint failures causing partial to total collapse of structures

However, there are numerous buildings around the world built without considering such provisions. In order to make them seismically acceptable, good models are required to evaluate the existing state of the structure against seismic forces and also to develop various retrofit strategies. Realistic shake table tests are the best possible way to generate information on real-life behavior of structures that can be used to validate the existing or proposed models as well as to evaluate the efficacy of a retrofit solution. In this work, such an attempt to conduct shake table tests on a reinforced concrete framed structure that was designed following the non-seismic provisions (IS 456:2000) and generate useful information is made. The tests were conducted in three different stages as explained below.

One of the most important parameter that needs to be estimated correctly for better prediction of forces that will be attracted by the structure is its fundamental frequency. It is well known that the natural frequency of RC structures is largely dependent on whether it has masonry infill panels or not. Infill panels are known to contribute significantly towards the stiffness of the structure. In order to obtain the effect of masonry infill panels on the stiffness of the structure, sine sweep tests were conducted on the structure with and without infill panels. However, it must be noted that this was not the main objective of the tests and was included as the first stage of the program to get additional information.

Often, it is found that old structures need retrofitting to qualify for the current seismic requirements. There are numerous such retrofitting techniques now available and many more are continuously being developed. However, most of the conventional retrofit methods are quite invasive and they require lot of alterations to the original structure. One retrofit solution which is extremely low invasive and virtually does not disturb any of the inhabitants is the Tuned Liquid Dampers (TLD). Preliminary tests on a small steel structure (Sharma *et al*. 2008) provided promising results that encouraged the authors to try the system on this structure under realistic ground motions. Although, TLD were initially used to control the wind response of the structures (Soong 1988, Modi *et al*. 1990, Fujino *et al*. 1992), more recently, the use of TLD in controlling the seismic response of structures is studied (Reed *et al*. 1998, Banerji *et al*. 2000, Banerji 2004, Li *et al*. 2004, Jin *et al.* 2007, Bairrao 2008). It has been found that if properly proportioned, TLD can be quite effective in controlling the seismic response of structures, even to broadband excitations (Banerji 2004). In the second stage of this experimental program, tests were carried on the 3D RC framed structure without and with TLD and the seismic response were studied and compared.

Inelastic behaviour of structures plays a vital role in the current seismic design and re-qualification practice for RC structures. In this practice, the estimated forces on the structure obtained by the linear analysis are reduced to account for nonlinearity in the structure, or sometimes some kind of nonlinear analysis such as pushover analysis or nonlinear time history analysis is performed to have a better estimate of demand on the structure due to a seismic event. Due to minimum flexural and shear design requirements, non-seismically designed structures may possess an inherent lateral strength to resist minor to moderate earthquakes, but the performance of such structures under severe earthquakes may be extremely poor (Bracci *et al.* 1995). A comparison of seismic design provisions to the non-seismic design provisions points out to various deficiencies in the non-seismically detailed structures. Weak column strong beam configurations, minimal transverse reinforced in columns for shear and confinement, especially in potential hinge zones, providing lap splices in potential hinge zones, no ties in the joint core, and inadequate anchorage to beam bars in the joints are the most commonly found deficiencies. In this test program, the deficiencies that were included in the structure comprised of no special confining reinforcement and no ties in the joint. In the third stage, experiments were conducted on the structure with earthquake ground motions in two axes with a gradually increasing level of PGA that represented earthquakes from low to moderate to severe intensities.

2. Objectives and research significance

Non-conforming RC structures make the majority of RC structures existing worldwide, especially in developing countries, where the seismic detailing provisions were enforced only in 90's. In such structures, several nonlinearities exist apart from beam hinging and column hinging such as joint shear failure, bond failure etc that make the modeling difficult. Also interactions between infill panels and frame make things complicated. Shake table tests offer a way to have the valuable insight to the structure in terms of the failure modes, study the interactions that can be utilized to develop and validate the analytical models. Moreover such tests on realistic structures and ground motions are the best way to prove the efficacy of the retrofit system. In view of the above the test program was designed with following three primary objectives:

1. To investigate the usefulness of TLD to reduce the seismic response of realistic RC structures under realistic ground motions
2. To investigate the seismic behavior and failure modes of non-conforming RC structure till failure.
3. To generate data for calibrating the available analytical models for better prediction of response of such structures during earthquakes.
4. To study the influence of masonry infill panels on the stiffness and fundamental frequency of the structure.

3. Experimental program

In this work, a 3D RC frame structure was tested under seismic ground motions. The structure was symmetric in plan with 2 bays in both directions but in one direction, the structure was softer than in other direction, due to the use of rectangular columns. The structure had a plinth storey, whose height was 1/3rd of the height of other three stories. The test was conducted at the

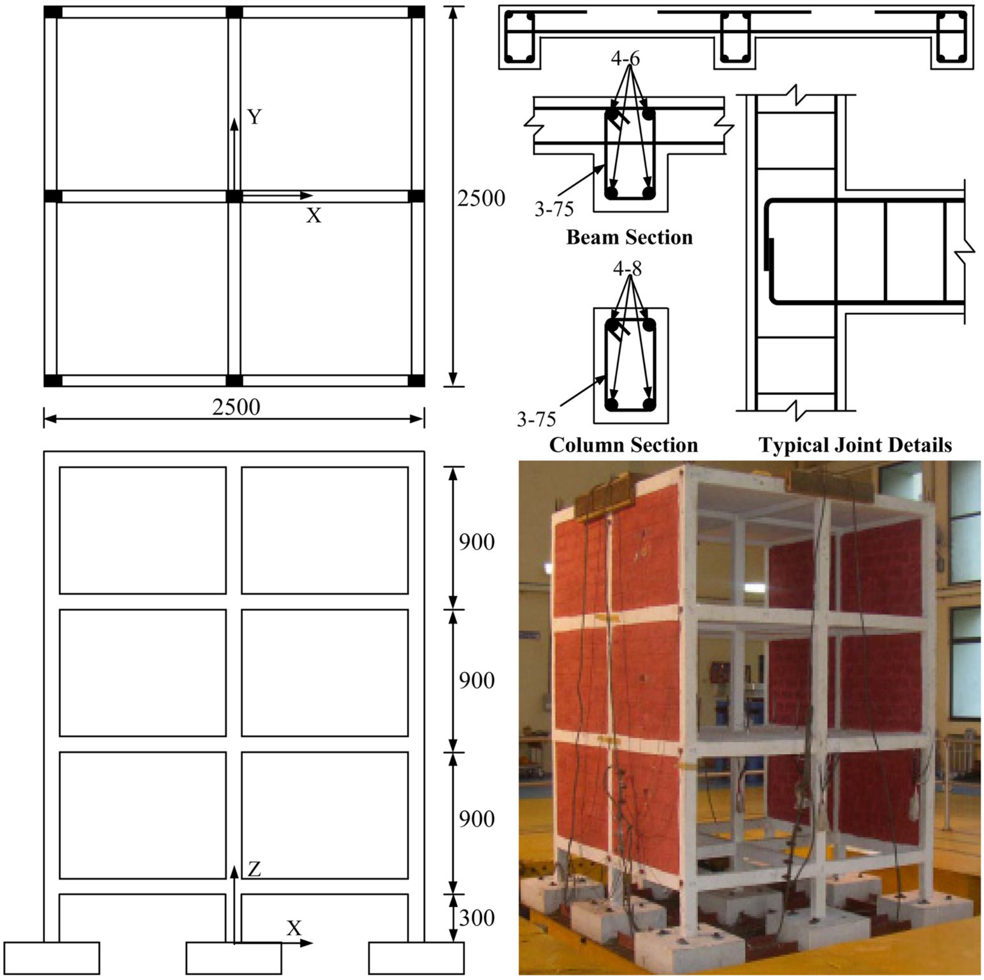

Fig. 2 Details of the test structure

Earthquake Engineering and Vibration Research Centre (EVRC) of Central Power Research Institute (CPRI) Bangalore. The shake table size is 3 m × 3 m and the payload capacity is 10 tons. Due to the shake table size and weight restrictions, the structure had to be scaled down to approximately 1/3rd scale. The section sizes of beams and columns were 75 mm × 100 mm with a 50 mm thick slab. Fig. 2 shows the details and photographic view of the structure before the test. All the reinforcement of 6 mm or less was Mild steel reinforcement and 8 mm diameter bars were tor steel reinforcement. The slab reinforcement consisted of 6 mm diameter bars at the rate of 100 mm c/c. The tested average cube compressive strength of concrete was obtained as 33.0 N/mm^2.

The structure originally had masonry infill completely filling the frame panels in *X*-direction, whereas the frame panels in *Y*-direction (weaker direction) were not having any masonry infill. This was mainly due to the weight restrictions of the shake table. The first set of the test was therefore conducted on the structure with infill panels. Sine sweep tests were conducted on the structure with a PGA of 0.075 g, 0.1 g and 0.125 g in *X*, *Y* and *Z* (vertical) directions from 1 Hz to 50 Hz at the rate of 1 Octave per minute, i.e. the frequency of sine wave was doubled in a minute. Fig. 3 shows a typical input time history for the sine sweep tests corresponding to PGA of 0.1 g.

The masonry walls were then removed and almost equivalent masses were applied on the floors and the sine sweep tests were again carried out. The criterion for placing the masses was such that the frequency of the structure could be brought to the desirable range of tuning with TLD. In this way the fundamental frequency of the structure without and with walls was compared and the tuning between the sloshing frequency of TLD and fundamental frequency of the structure could be

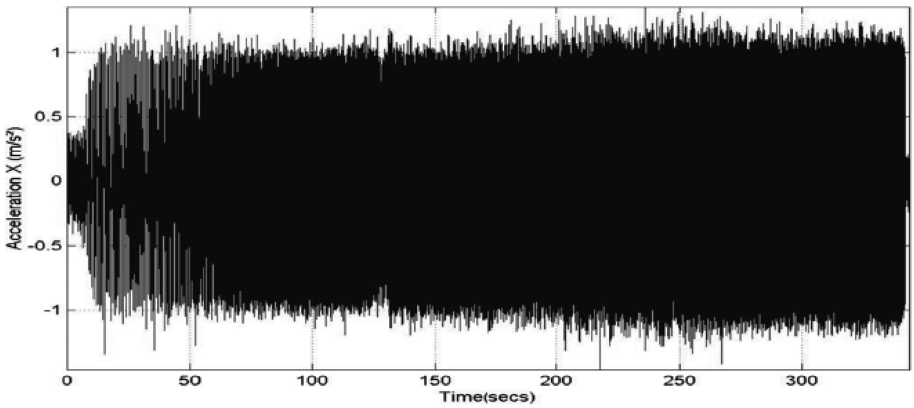

Fig. 3 Typical input time history for sine sweep tests (PGA = 0.1 g)

Fig. 4 Structure with masses and TLD but without walls

obtained. The second stage of tests consisted of tests on the structure without and with TLD kept on the roof of the structure. The TLD were proportioned beforehand to have dimensions that would make the sloshing frequency approximately equal to the estimated fundamental frequency of the structure. Fig. 4 shows the photograph of the structure with removed walls, added masses and TLD in the form of liquid containers at the roof. The added masses were 500 kg on the roof and 1st floor slab and 1000 kg on the 2nd floor slab. First, a series of tests were performed with the TLD and later the TLD were removed and the same series of tests were performed without TLD to compare the seismic response of the structure with and without dampers.

In the 3rd stage of the experimental program, biaxial ground motions were provided to the shake table in X and Y directions simultaneously with equal excitations, while no input motion were given in the Z direction. The time history was artificially generated such that the response spectrum generated corresponding to the time history would closely envelope the target response spectrum (Fig. 5).

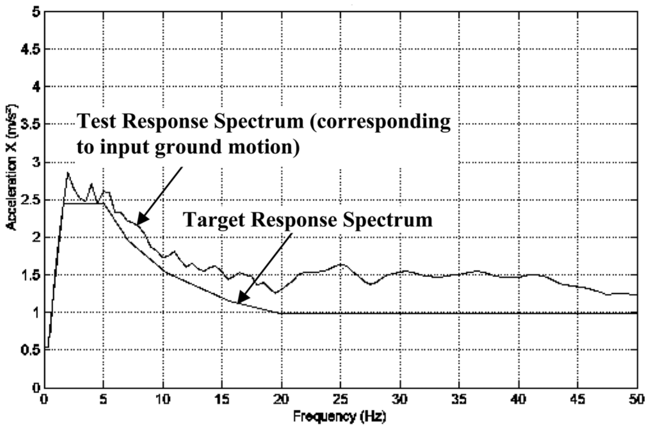

Fig. 5 Comparison of target and test response spectrum normalized to 0.1 g

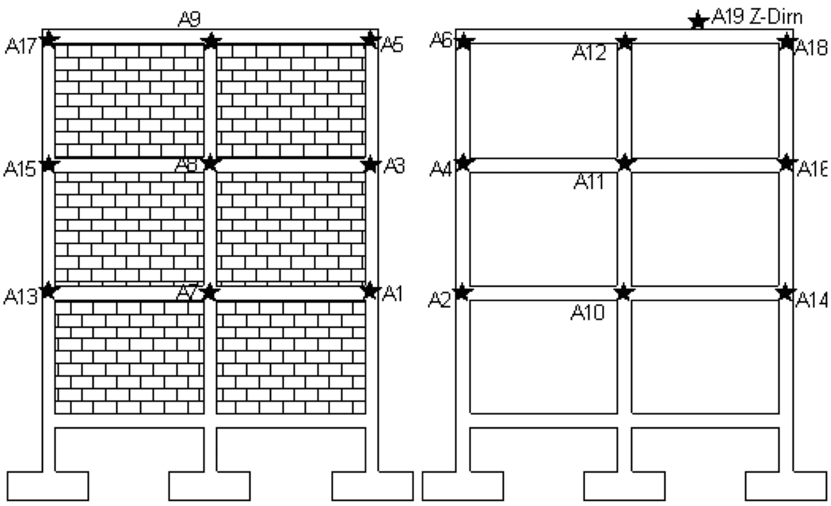

Fig. 6 Accelerometer locations on the structure

The tests started with the biaxial time histories provided to the table with a PGA of 0.1 g and gradually it was increased in steps of 0.1 g till 0.7 g. The limit of 0.7 g was due to the shake table stroke limitations. The maximum available displacement stroke for the actuators of shake table was +/- 150 mm. The maximum displacement in either direction for a PGA of 0.1 g was 20 mm and

Table 1 Test Matrix of experimental program

S. No.	Ground Motion	Axis of loading	PGA (g)	Description
1	Sine sweep	*X*-Axis	0.075	With infill panels
2	Sine sweep	*Y*-Axis	0.075	With infill panels
3	Sine sweep	*X*-Axis	0.1	With infill panels
4	Sine sweep	*Y*-Axis	0.1	With infill panels
5	Sine sweep	*Z*-Axis	0.1	With infill panels
6	Sine sweep	*X*-Axis	0.125	With infill panels
7	Sine sweep	*Y*-Axis	0.125	With infill panels
8	Sine sweep	*X*-Axis	0.1	Without infill panels, with added mass
9	Sine sweep	*Y*-Axis	0.1	Without infill panels, with added mass
10	Simulated Seismic	*X*-Axis	0.075	With TLD
11	Simulated Seismic	*X*-Axis	0.1	With TLD
12	Simulated Seismic	*X*-Axis	0.15	With TLD
13	Simulated Seismic	*Y*-Axis	0.075	With TLD
14	Simulated Seismic	*Y*-Axis	0.1	With TLD
15	Simulated Seismic	*Y*-Axis	0.15	With TLD
16	Simulated Seismic	*X*-Axis	0.075	Without TLD
17	Simulated Seismic	*X*-Axis	0.1	Without TLD
18	Simulated Seismic	*X*-Axis	0.15	Without TLD
19	Simulated Seismic	*Y*-Axis	0.075	Without TLD
20	Simulated Seismic	*Y*-Axis	0.1	Without TLD
21	Simulated Seismic	*Y*-Axis	0.15	Without TLD
22	Simulated Seismic	*X*-*Y*	0.1	Biaxial ground motion
23	Simulated Seismic	*X*-*Y*	0.2	Biaxial ground motion
24	Simulated Seismic	*X*-*Y*	0.3	Biaxial ground motion
25	Simulated Seismic	*X*-*Y*	0.4	Biaxial ground motion
26	Simulated Seismic	*X*-*Y*	0.5	Biaxial ground motion
27	Simulated Seismic	*X*-*Y*	0.6	Biaxial ground motion
28	Simulated Seismic	*X*-*Y*	0.7	Biaxial ground motion
29	Sine sweep	*Y*-Axis	0.1	To verify the change in frequency after the induced damage in the structure

correspondingly, the maximum displacement in either direction for a PGA of 0.7 g was around 140 mm, which was very close to the limit of the table and the tests could not be continued further. In the end, a sine sweep test was conducted to study the frequency change of the structure due to induced damage. Table 1 shows the test matrix of the experimental program.

In order to obtain the seismic response, accelerometers were mounted on the structure at various locations as shown in Fig. 6 and were connected to the high speed data acquisition system. The output readings from the accelerometers were recorded digitally at a frequency of 256 samples per second.

4. Proportioning of TLD

The TLD are characterized by three major ratios (Chaiseri *et al.* 1989, Sun *et al.* 1991, Koh *et al.* 1994, Reed *et al.* 1998, Yu *et al.* 1999, Fujino *et al.* 1992, Banerji *et al.* 2000) namely tuning ratio, mass ratio and depth ratios. A TLD that reduces the structural response significantly for the given set of values of these ratios may be considered as properly designed.

4.1 Tuning ratio

The fundamental linear sloshing frequency of the TLD is given by (Jin *et al.* 2007)

$$f = [\{\tanh(3.68\Delta)*3.68g/D\}^{0.5}]/2\pi \tag{1}$$

Where, Δ is the water depth ratio defined as ratio of undisturbed water depth h to the tank diameter, D.

As the name suggests, tuning ratio of a TLD, is the ratio of the fundamental, linear sloshing frequency, f, given by Eq. (1), to the natural vibration frequency of the structure. Generally a tuning ratio of unity or very close to unity is considered to be optimum (Chaiseri *et al.* 1989, Sun *et al.* 1991, Reed *et al* 1998, Yu *et al.* 1999, Fujino *et al.* 1992, Banerji *et al.* 2000). Banerji *et al.* concluded that it is reasonable to consider this tuning ratio to have a value of unity for strong earthquake motions also. Based on these arguments, the target tuning ratio for the TLD was set to unity.

4.2 Depth ratio

As seen in Eq. (1), the only two parameters that can be varied to achieve the desired tuning ratio are undisturbed water height, h and the tank diameter D. Thus, the water depth ratio Δ, which is the ratio of water depth h to tank length D, is the significant parameter to mark the effectiveness of TLD. Most of the earlier studies (Chaiseri *et al.* 1989, Koh *et al.* 1994, Yu *et al.* 1999) have restricted this value to less than 0.1. However, Banerji *et al.* (2000) pointed out that the water depths required for the short period structure with frequencies more than 2 Hz, and a tuning ratio of unity are too small for practical implementation if such a restriction is maintained. They tested a depth ratio of 0.3 and concluded that the optimum value for such structures under high excitation levels is around 0.3. They supported it with the argument that at a high excitation level, the energy transmitted is large and even a TLD with relatively larger water depths dissipates energy through sloshing and wave breaking. Therefore for strong earthquake motions, it is more effective to use larger water depths, within the constraints of shallow water theory (Banerji *et al.* 2000). Based on these arguments, the water depth ratio in this case was considered as 0.3.

4.3 Mass ratio

The mass ratio, is the ratio of the mass of water in the containers to the mass of the structure itself. In order to make sure that TLD do not influence the dynamic characteristics of the structure and also due to the practical limitations of space on the roof of the structure to place dampers, a mass ratio of 1% was considered in this case.

4.4 Design of TLD

From pretest analysis, the fundamental frequency of the structure was estimated to be around 3.05 Hz corresponding to design material properties. Since it was decided to consider tuning ratio as unity, From Eq. (1), for a water depth ratio of 0.3 and $f = 3.05$, the diameter of the tank comes to about 80 mm. Therefore in this test, plastic containers with internal diameter of 80 mm were used with a water height of 24 mm. The total mass ratio in this case came out to a little less than 1% but due to space constraints, it was considered suitable.

5. Test structure and prototype structure

In past, efforts have been put by researchers to perform such tests. Tests reported by El-Attar *et al.* (1991), Bracci *et al.* (1995), Lee and Woo (2002), Cardone *et al.* (2004), Dolce *et al.* (2005), Dolce *et al.* (2006a, b, c), Hwang *et al.* (2006), Hashemi and Mosalam (2006a, b, 2007), Kakaletsis *et al.* (2011) are a few examples of similar tests and many more tests may be found in literature. However, one of the biggest technical limitations of the shake table tests is to design the structure suitable enough to represent the real life structures without much distortion. Mostly this limitation comes due to the size and weight capacity of the shake table. Though theoretically it would be possible to design a specimen that can almost accurately simulate the real life structure at a smaller scale by following dimensional analysis (Buckingham 1914), the construction of a 'true replica' model that satisfies all the similitude requirements needed by dimensional analysis is almost an impossible task due to material limitations (Morcarz *et al.* 1981, Quintana-Gallo *et al.* 2010). Therefore, the challenge is to design a least distorted model within the constraints of shake table capacity and material availability.

The structure tested in this experimental program represented a scaled down model of a part of a three storey reinforced concrete frame structure designed as per non-seismic design guidelines as per Indian Standards (IS 456:2000). Although the structure was detailed non-seismically, it did not represent a typical 70's structure where plain round bars with hooks were used; rather it represented structures built in later decades but not following seismic design and detailing provisions. Such structures make a large number of existing structures in India.

The design concrete cube strength was considered as 25 N/mm^2 (corresponding to M25 specified by IS 456:2000) and deformed bars having characteristic yield strength of 415 N/mm^2 were selected for the design. The choice of the set of materials is one of the most popular material parameters considered for design of such structures in India. The prototype structure was considered to have a

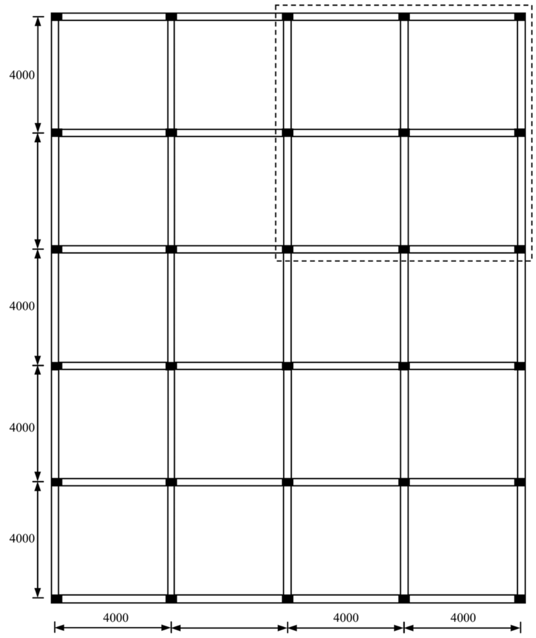

Fig. 7 Structural layout of prototype structure considered for study

storey height of 3.0 meters and beam width of 4.0 meters, which is typical for a residential building in India giving a popular room size. The design was performed on the basis of the guidelines provided by the code (IS 456:2000) with no seismic considerations. Fig. 7 shows the beam and column layout of the hypothetical prototype structure considered for this study. The portion of the structure enclosed within the dashed lines was considered to be scaled and tested on the shake table. Although, ideally a scaled model of the prototype designed as per scaling laws and dimensional analysis would be best suited, but as mentioned earlier due to the constraints set by shake table capacities and material availability, such a scaling is not possible. In this case, in general, a scale factor of 3.33 was considered to design the model. The scale factor was basically governed by the size limitation of the shake table. The table used for this experiment is 3 m × 3 m in size and therefore the plan dimension of the structure to the outer of the columns was set as 2.5 meters, leaving just sufficient space for overhang of footings beyond columns. Thus, the center to center distance between two columns was set as 1.2 meters leading to a linear scale factor, S_L of 4/1.2 = 3.33.

6. Similitude requirements

6.1 Linear dimension scaling

As mentioned earlier, due to the shake table size limitation, a linear scale factor of $S_L = 3.33$ was enforced. Thus, the bay width was scaled from 4.0 meters to 1.2 meters and the storey height was scaled from 3.0 meters to 0.9 meters. The plinth storey height was reduced from 1.0 meters to 0.3 meters. The beam and column dimensions for the prototype structure were maintained at 250 mm × 325 mm so that they could be easily scaled to 75 mm × 100 mm.

6.2 Material scaling

As mentioned earlier, the prototype structure was considered to be built by using concrete of grade M25 (characteristic cube strength = 25 MPa) and deformed bars with characteristic yield strength of 415 MPa. To avoid too much scaling from material point of view, it was decided to use concrete mix of same grade. However, to maintain similitude, the maximum size of aggregate (msa) of 6 mm was used (corresponding to msa of 20 mm in prototype). The design of the concrete mix yielded in a ratio of 1:1.35:2.02 with a water-cement ratio of 0.45. The mean concrete cube strength for the mix was found to be 33.0 N/mm^2, which was very close to the target mean strength of 33.75 N/mm^2 as recommended by IS Code (IS 456:2000). Thus, for concrete it can be said that the material scale factor given by the ratio of concrete strength in the prototype to concrete strength in model, S_{FC} was unity.

In case of reinforcement, it was not possible to use deformed bars with characteristic yield strength of 415 MPa except as longitudinal bars in columns due to size availability issues. Due to area scaling, the sizes for rebars other than longitudinal bars in columns were 6mm diameter or less and only mild steel bars with characteristic yield strength of 250 MPa was available for these sizes. Defining material scale factor for steel, S_{FS} as the ratio of specified yield strength for rebars in prototype to that in model, two different values of S_{FS} were used. Thus, S_{FS} for column main rebars was unity and for other rebars, S_{FS} = 415/250 = 1.66.

6.3 Reinforcement area scaling

According to similitude requirements (Sabnis *et al.* 1983), the required model reinforcement area to provide the scaled bar yielding force is calculated as

$$A_m = \frac{A_p}{S_{FS} \times S_L^{\,2}} \tag{2}$$

The model reinforcement area were scaled according to the law given by Eq. (2), though, a perfect scaling could not be done due to size and type of reinforcements available.

6.4 Mass scaling

Since the density and modulus of elasticity of the concrete in prototype and model were essentially equal, a mass simulation approach suggested by Quintana-Gallo *et al.* (2010) that corresponds to an extension of what is suggested by Morcarz *et al.* (1981) was followed. The additional inertial mass on the i^{th} floor level in the model structure, ΔM_i corresponding to a unit mass in prototype is given by Eq. (3).

$$\Delta M_i = \frac{1}{S_L^{\,2}}\left(1 - \frac{1}{S_L}\right) \tag{3}$$

Therefore for every ton of mass located at a storey level of the prototype structure, 0.063 tons of mass was to be applied at the corresponding storey level for prototype structure. However, this restriction was not cent percent satisfied due to shake table weight restrictions. Moreover, as mentioned in part 1 of the paper, another criterion for added mass was to bring the frequency of the structure in the desirable range to test tuned liquid dampers.

6.5 Comment on similitude

As shown above, similitude between the prototype and model was attempted for, following the scaling rules. However, it was found that every aspect of the structure could not be scaled due to shake table capacity and material availability constraints. But it should not be considered as a matter of big concern since the objective of the test was not to qualify the prototype structure by testing but to study the behaviour of reinforced concrete structures, in general, under earthquakes.

7. Results and discussion

7.1 Results of 1st stage of experiments

7.1.1 Sine sweep tests on structure in plane of walls (X- direction)

The tests were carried out for a frequency range of 1 to 50 Hz, with acceleration levels of 0.075 g, 0.1 g and 0.125 g. During these tests, cracks in the mortar at the corner of walls were observed first on ground storey walls (at an acceleration of 0.1 g) and then on middle storey walls at an acceleration of 0.125 g). This is as expected since the maximum diagonal tension will come at the bottom storey. Consequently, there was a gradual reduction in frequency for three different acceleration levels as 9.75 Hz, 9.25 Hz and 8.75 Hz for 0.075 g, 0.1 g and 0.125 g respectively. Moreover, due to degradation in the load carrying capacity and stiffness of the walls, the amplification from the base to roof acceleration increased as 4.04, 4.5 and 5.2 for 0.075 g, 0.1 g and 0.125 g respectively. However, no resonant frequency corresponding to second mode was obtained in any case. The results of sine sweep tests in X-direction (in plane of the walls) are summarized in Table 2.

7.1.2 Sine sweep tests on structure in vertical direction

A single test with an acceleration level of 0.1 g was performed in Z-direction and no modes were observed in the range of 1-50 Hz.

7.1.3 Sine sweep tests on structure out-of-plane of walls (Y-direction)

The tests were carried out for a frequency range of 1 to 50 Hz, with acceleration levels of 0.075 g, 0.1 g and 0.125 g. During these tests, corner mortar cracks formed earlier due to in-plane tests for corresponding accelerations propagated throughout the periphery of the walls first on middle storey walls (at an acceleration of 0.1 g) and then on bottom storey walls at an acceleration of 0.125 g). However, there was no significant cracking in the top storey walls. This clearly highlights the fact that the out-of-plane strength of the walls is significantly influenced by simultaneous action of in-

Table 2 Summary of results obtained during sine sweep tests in *X*-direction with walls

PGA	Visual Observation	Resonant Frequency		Amplification
		1st Mode	2nd Mode	
0.075 g	No cracking, No damage	9.75 Hz	Not Found	4.04
0.1	Cracks in mortar at corner of ground storey walls.	9.25 Hz	Not Found	4.5
0.125 g	Cracks in mortar at corner of middle storey walls.	8.75 Hz	Not Found	5.2

Table 3 Summary of results obtained during sine sweep tests in *Y*-direction with walls

PGA	Visual Observation	Resonant Frequency		Amplification
		1st Mode	2nd Mode	
0.075 g	No cracking, No damage	4.25 Hz	17.0 Hz	13.69
0.1	Peripheral cracks in middle storey	4.25 Hz	16.25 Hz	13.34
0.125 g	Peripheral cracks in bottom storey	4.00 Hz	15.375 Hz	10.85

plane vibrations (as in the case of real earthquake scenarios). No significant change in the frequency for first mode of the structure was noticed with the propagation of cracks. However, a reduction in the second mode frequencies was observed with increase in acceleration levels. The frequencies of the structure were obtained as 4.25 Hz, 4.25 Hz and 4.0 Hz for first mode and 17.0 Hz, 16.25 Hz and 15.375 Hz for the second mode against the acceleration levels of 0.075 g, 0.1 g and 0.125 g respectively. The amplifications were 13.69, 13.34 and 10.85 against the acceleration levels of 0.075 g, 0.1 g and 0.125 g respectively. The results of sine sweep tests out-of-plane of the walls are summarized in Table 3. It is interesting to note that amplifications were much higher in case of *Y*-direction, where there are no walls. This is attributed to the fact that due to the presence of the masonry infill panels, the behaviour of the frame structure is essentially changed from a moment-resisting frame action to truss action (Murty and Jain 2000).

7.1.4 Sine-sweep tests on bare frame structure in X- and Y- directions

Sine sweep tests were conducted in two orthogonal horizontal directions in order to determine the frequency of the structure after removal of walls and with added mass. It was found that the frequency of the structure in *X*-direction has lowered to 3.5 Hz, and that in *Y*-direction is lowered to 3.0625 Hz.

7.2 Discussion on 1st stage of experiments

The 1st stage experiments clearly displayed that the masonry infill panels contribute greatly to the stiffness of the structure leading to higher natural frequencies. The increase of stiffness is generally associated with attracting higher seismic forces for the cases of most of the real life reinforced concrete framed structures. However, there is another important aspect to the contribution of masonry walls, which is associated with the change of load transfer mode from predominant moment resisting frame action for bare frames to predominant truss action for frames with infill walls. Therefore due to the inclusion of walls, the moments at the ends of beams and columns are reduced while the axial forces are increased.

Another important issue is the stability of walls itself. Masonry infill panels are traditionally constructed after the bare frame structure is ready, thereby not allowing any good bondage between the walls and the frame members and the only thing resisting the forces at the interface is the weak and thin mortar layer. As seen by the experiments, walls tend to develop brick-mortar interface cracks at the corners at a relatively low PGA level under in-plane loads. These cracks propagate through the periphery of the walls under out of plane loads making the walls unstable. Therefore, under the combined action of in-plane and out-of-plane accelerations as would be the case in a real earthquake, the stability of the masonry walls are often endangered, which is one of the major causes for the loss of life and property during earthquakes. In such cases, even though the structural framework may be stable but the walls itself may become unstable and lead to partial, non-structural but life endangering failures. One solution to this problem is using dowels to anchor walls to the frame or to prevent out of plane movement of the walls by using fibre mats etc. However, this is out of scope of this paper.

8. Results of 2nd stage of experiments

In the 2nd stage of the experiments, simulated seismic ground motion was applied to the shake table. The accelerations were applied only in one direction at one time to the table, i.e., the ground motion was essentially uniaxial. The input ground motion was the artificially generated time history, response spectrum of which enveloped the desired response spectrum (Fig. 5). Fig. 8 shows the corresponding acceleration time history actually provided to the shake table.

8.1 Simulated seismic tests on structure in X-direction with and without dampers

The simulated seismic tests were conducted on the structure in *X*-direction with and without dampers for PGA levels of 0.075 g, 0.1 g and 0.15 g. The PGA level was limited to 0.15 g in order to not induce any damage to the structure. Fig. 9 shows the typical acceleration response recorded at the roof of the structure for the ground motion corresponding to 0.1 g PGA in *X*-direction without dampers and Fig. 10 shows the same for the case with dampers. The maximum responses of the structure were observed as 5.5 m/s^2, 6.5 m/s^2 and 8.7 m/s^2 for the case of without dampers and as 4.2 m/s^2, 5.1 m/s^2 and 6.8 m/s^2 for the case with dampers against the acceleration levels of 0.075 g, 0.1 g and 0.15 g respectively displaying that the TLD are capable to control the seismic response of the structure.

8.2 Simulated seismic tests on structure in Y-direction with and without dampers

The simulated seismic tests were also conducted on the structure in *Y*-direction with and without

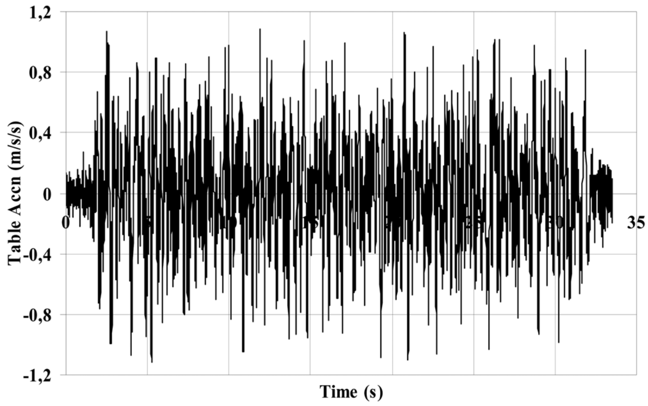

Fig. 8 Acceleration time history provided to the shake table

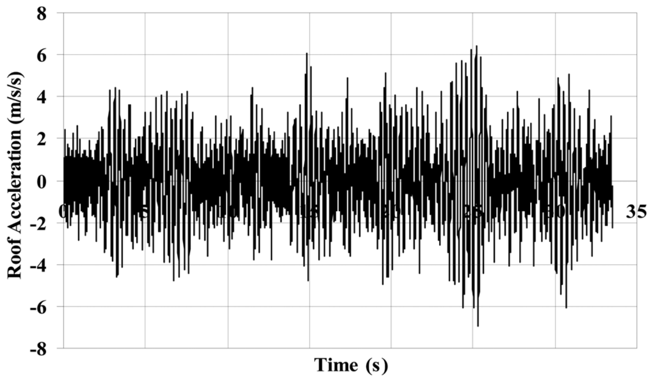

Fig. 9 Acceleration response of the structure without dampers recorded at roof in *X*-direction due to 0.1 g PGA ground motion

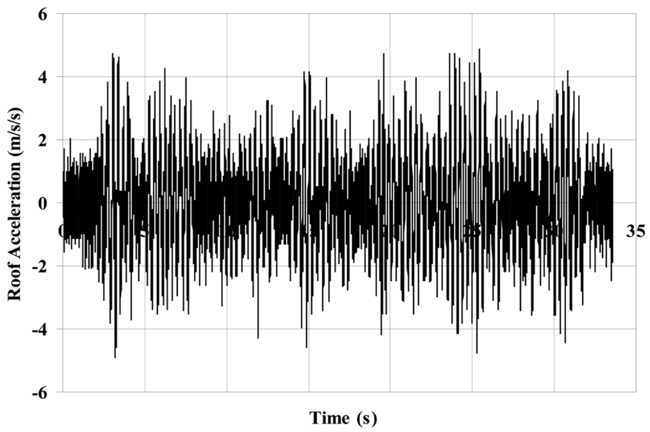

Fig. 10 Acceleration response of the structure with dampers recorded at roof in *X*-direction due to 0.1 g PGA ground motion

dampers for PGA levels of 0.075 g, 0.1 g and 0.15 g. The maximum responses of the structure were observed as 5.2 m/s^2, 6.7 m/s^2 and 8.5 m/s^2 for the case without dampers and as 3.0 m/s^2, 4.2 m/s^2 and 5.8 m/s^2 for the case with dampers against the acceleration levels of 0.075 g, 0.1 g and 0.15 g respectively again displaying the suitability of TLD in controlling the response of the structure. The summary of results of simulated seismic tests for *X*-direction is given in Fig. 11 and that for *Y*-direction is shown in Fig. 12.

The 2nd stage experiments displayed the efficiency of using tuned liquid dampers in controlling the seismic response of reinforced concrete frame structures. As mentioned earlier, the response is controlled by tuning and closer the tuning ratio is to unity, in general, the response reduction should be higher. The same effect is displayed in this case also. As discussed in the section of design of TLD, the TLD were designed to have a sloshing frequency of around 3.05 Hz. From the sine sweep tests performed in the two horizontal directions after removal of walls and added mass, it was found that the frequency of the structure in *X*-direction was 3.5 Hz, and that in *Y*-direction was 3.0625 Hz. Therefore, of course the TLD were better tuned to the *Y*-direction frequency rather than the *X*-

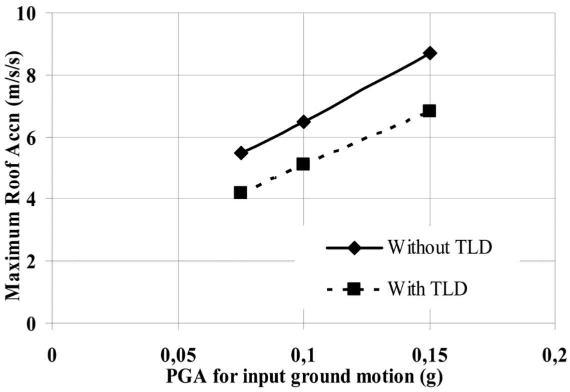

Fig. 11 Maximum response of the structure for *X*-direction without and with TLD

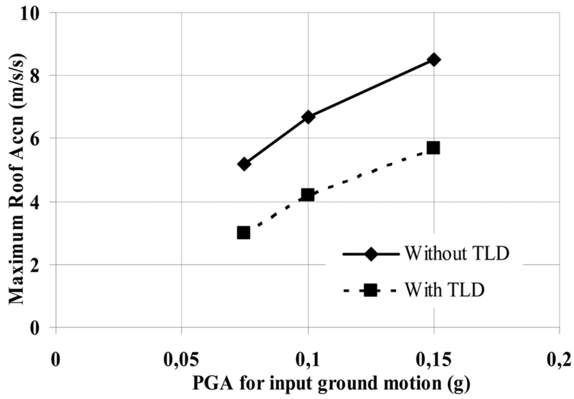

Fig. 12 Maximum response of the structure for *Y*-direction without and with TLD

direction frequency. Therefore, it can be expected that the response control is better in *Y*-direction than in *X*-direction. Results summarized in Figs. 10 and 11 confirm this prediction. The average reduction in the response was found to be around 22% in *X*-direction and around 37% in *Y*-direction. However, it must be remembered that in this case the excitation was given only in one direction. If the structure would be excited in bidirectional motion, the response reduction even for *Y*-direction would be lesser. Moreover in such cases when circular TLD are used, there is a risk for the phenomena called 'whirling' to occur, which would result in reducing the effectiveness of TLD.

9. Results of 3rd stage of experiments

The third stage of tests started with simulated bidirectional earthquake with a PGA of 0.1 g. The acceleration time history for 0.1 g PGA was shown in Fig. 8 and the corresponding displacement time history for the ground motion is shown in Fig. 13. Same time history was provided in both *X*-

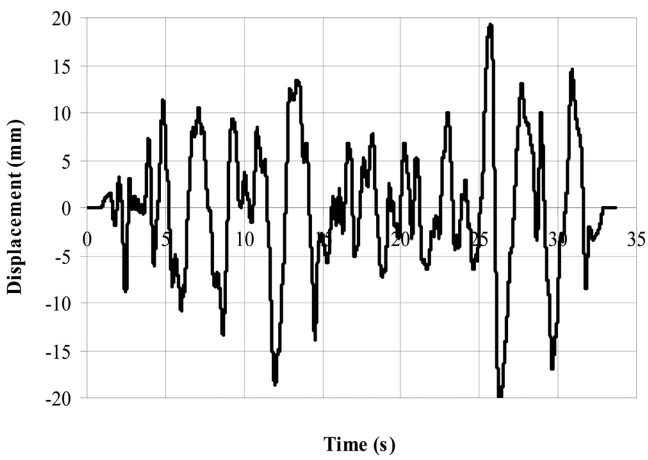

Fig. 13 Displacement time history provided to the shake table

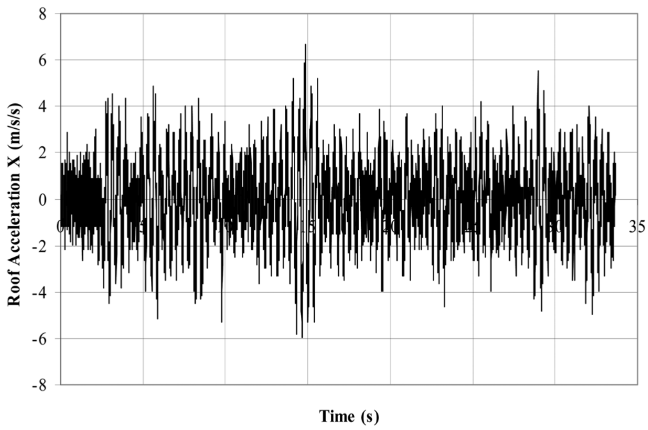

Fig. 14 Floor acceleration time history for *X*-direction recorded at the roof level for a PGA of 0.1g PGA bi-directional base excitation

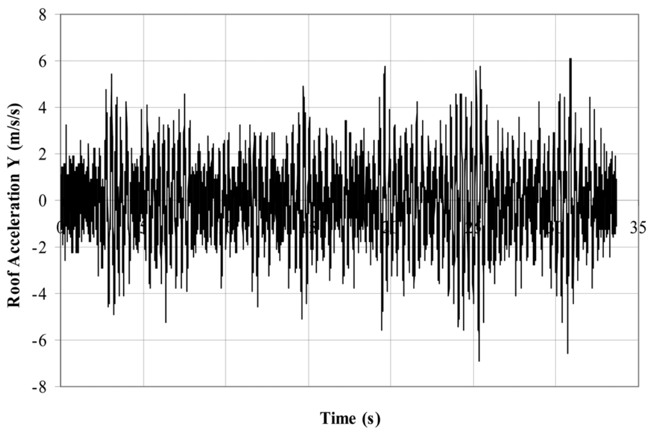

Fig. 15 Floor acceleration time history for *Y*-direction recorded at the roof level for a PGA of 0.1g PGA bi-directional base excitation

and *Y*- directions. The response of the structure recorded at roof in *X*-direction is shown in Fig. 14 and that in *Y*-direction is shown in Fig. 15. Similar time histories were recorded at each floor level for both directions corresponding to each PGA level. A plot of the maximum acceleration recorded v/s the PGA of base excitation for *X*- direction is shown in Fig. 16 and the same corresponding to *Y*- direction is shown in Fig. 17.

The plots of Figs. 16 and 17 clearly show that at lower PGA base excitations, the maximum response acceleration increase steeply with the base excitation suggesting a linear behaviour of the structure, whereas as the base excitation is increased, the response becomes flat suggesting inelastic behaviour.

It can be observed from Figs. 16 and 17 that both the curves display an almost linear increase of maximum recorded roof acceleration in both the directions till a PGA level of 0.4 g. However, as can be noticed, the acceleration values are not directly proportional to the PGA values, e.g., the

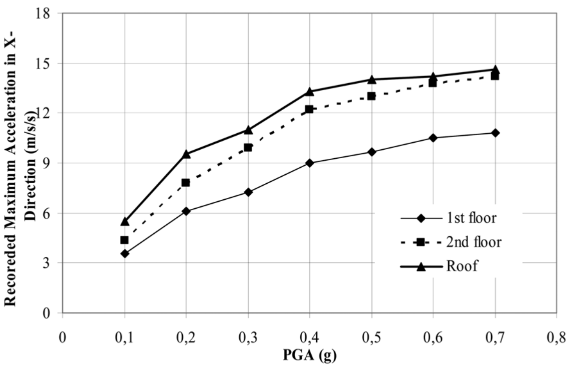

Fig. 16 Recorded peak roof acceleration v/s PGA for *X*-direction

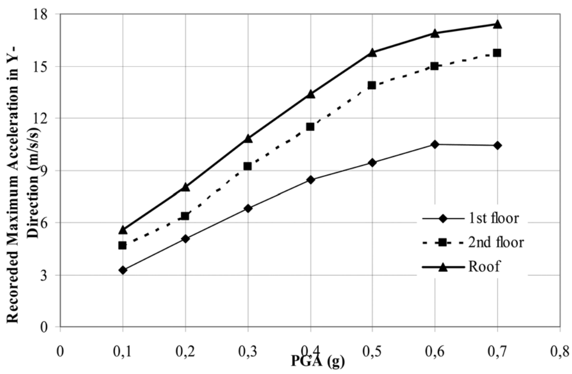

Fig. 17 Recorded peak roof acceleration v/s PGA for *Y*-direction

(a) PGA = 0.2g **(b) PGA = 0.3g** **(c) PGA = 0.4g**

Fig. 18 Crack patterns observed during initial stages of the test

recorded maximum acceleration does not get doubled when the PGA is doubled. This essentially suggests, though the behaviour is linear, it is not elastic, i.e., certain damage must have occurred in the system, which would have changed the dynamic characteristics of the structure thereby reducing the response, but the damage was not so severe to cause any nonlinearity in the global response of the structure. This can be further confirmed with the pictures shown in Fig. 18.

Fig. 18 (a) shows the very first cracks that appeared at the base of the columns, close to the footing. This definitely is as expected since due to the lateral loads generated by the ground motion, maximum moments are experienced at the base of the columns and therefore the first cracks appear there. However, as seen in Fig. 18 (a), the cracks are only initiated and they only indicate that the concrete tensile stresses are exceeding their limits but definitely there is no yielding of the reinforcement. Therefore, due to the cracks, it is expected that the stiffness is reduced but the structure shall remain essentially linear. This explains a linear but not proportional rise in the maximum response of the structure corresponding to increasing PGA.

As the PGA level of the base excitation was increased to 0.3 g and 0.4 g, a few more cracks started to appear. For a PGA of 0.3 g, cracks appeared on the beams of first floor at the beam-column interface (Fig. 18 (b)). However, again the cracks did not suggest any yielding of bars. At 0.4 g, the beam

Fig. 19 Damage patterns observed at PGA = 0.5 g

flexural cracks opened up significantly and some spalling was also observed (Fig. 18 (c)) suggesting the initiation of yielding of rebars inside. Since the cracks suggest a yielding of bars, linear response of the structure can no longer be expected. Therefore, now the dynamic response of the structure must be modified not only because of the change in dynamic characteristics of the structure but also due to the additional hysteretic damping coming into picture, that should further reduce the response of the structure. This is exactly what can be observed from Fig. 16 and 17 that after the PGA of 0.4 g, the response curves for the structure tend to become flatter.

Fig. 19 shows the damage patterns observed for the base excitation with a PGA level of 0.5 g. It was observed that many new cracks were formed in the beams and columns and the old cracks opened widely and led to spalling of concrete. The major damages were observed for the columns of 1st and 2nd storey levels. This definitely suggests that the reinforcement in these critical locations (such as end of beams and columns) have yielded. However, there was a very interesting thing observed that the cracks formed at the very base of the columns at the PGA of 0.2 g (Fig. 18 (a)) neither grew any further nor opened significantly. This seems to be something unexpected in the first look, since these areas seem to be the most critical and generally the first hinges would be expected to form at the base of the columns. It was observed that instead of the cracks developed at the column bases opening up, the newer cracks that were formed at the plinth beam level (Fig. 19) became significant. This clears some of the doubts, since the presence of the plinth beam shifts the critical zones from the base of the columns to the plinth level. This is because the presence of plinth beam reduces the shear span for the columns and therefore instead of having a flexural mechanism, the column tends to develop a strut type of mechanism, thereby avoiding any further damage at the column base. This suggests that plinth beam definitely has beneficial effects as it prevents the structure to form a soft story type of mechanism. Such a mechanism might be possible in this case had the plinth beam not been there because in such a case, after the hinge formation at column bases, general tendency of next hinge formation is at the first story column level that may finally lead to a storey mechanism. However, these failures at the end of columns and beams, which are mainly flexural type, definitely indicate significant energy dissipation through hysteresis leading to reduced response.

After the PGA of 0.5 g, the spalling was significant and the already formed cracks were seen to open up widely. Fig. 20 shows the damage patterns observed at the PGA of 0.6 g and Fig. 21 shows that for the PGA of 0.7 g, which was the maximum base acceleration level applied. In the first photograph of Fig. 20, the bottom beam bars are visible and in the third photograph it can be

Fig. 20 Damage patterns observed at PGA = 0.6 g

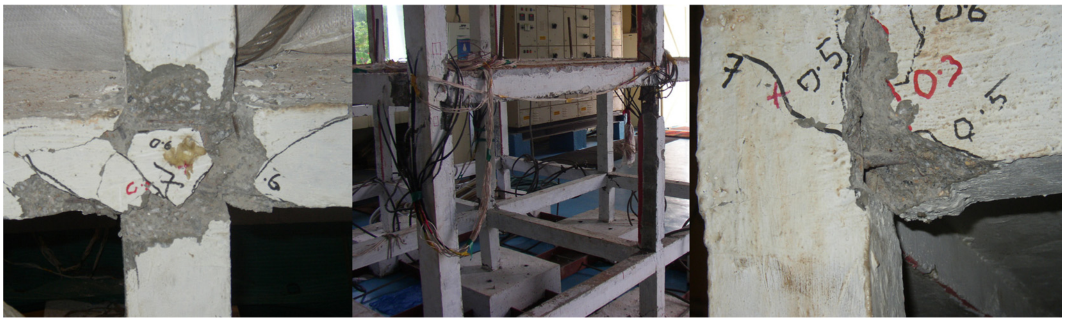

Fig. 21 Damage patterns observed at PGA = 0.7 g

seen that the interior column just below the first storey level has lost almost all of the concrete forming a total hinge. Not only the cover concrete has spalled off but even the core concrete within the reinforcement is lost. This shows the effect of lack of proper confinement with ties in the critical zones. However, the failure is essentially a flexural failure that may be attributed to large span to depth ratio for the columns, but the plastic hinge length is quite large in this case.

Fig. 21 mainly concentrates on showing the damages in the joints. The first photograph shows a typical shear failure of the interior joint. However, it must be noted that in this case it is not a pure joint shear (JS) failure but more of a so-called BJ failure mode where the members framing into the joint yield first and the shear failure of the joint occurs later. Similarly joint shear cracks can be seen propagating in the exterior joint shown in the last photograph, which is the same joint that is shown in the first picture of Fig. 13. These pictures indicate that the joints could not remain essentially elastic, which is mainly due to the lack of shear reinforcement in the joint core, but in this case, the joint failure was not so critical since the members of the structures failed first and then the cracks in the joints were induced.

As mentioned earlier, the tests could not be carried out for further higher PGA values due to the limitation of the stroke of the shake table. However, as seen above, the structure already suffered significant damage and therefore, not being able to carry out the tests further was not a big loss. As shown in Table 1, after the completion of simulated seismic tests, a sine sweep test was conducted in *Y*-direction to obtain the frequency of the damaged structure. This test, which was carried out with a PGA of 0.1 g showed that the structural frequency in *Y*-direction has come down to 1.025 Hz. As mentioned earlier, the original fundamental frequency of the structure in *Y*-direction was 3.0625 Hz, which means the structural frequency is reduced to almost $1/3^{rd}$ due to the damage. Since the mass of the structure is not significantly varied (except for a small spalling), this reduction in frequency points to a corresponding reduction in the stiffness of the structure by 9 times. This reduction is stiffness is attributed partly to the reduced modulus of the materials and partly to the change in load resisting mechanism due to hinge formations.

10. Conclusions

In this experimental program, a 3D RC frame structure was tested on the shake table to study the effects of infill panels on the dynamic characteristics of the structure, to verify the efficiency of TLD system to control the seismic response of the structure and to study the nonlinear behaviour of RC structure under increasing dynamic loads till failure. The studies on the effect of masonry infill further verified that the masonry infill panels contribute greatly to the stiffness of the structure leading to higher natural frequencies and change in load transfer mechanism from moment frame action to truss action. However, the vulnerability of the walls themselves from stability point of view was another important issue pointed out in the program. It was seen during the experiments that walls tend to develop interface cracks at the corners at a relatively low PGA level under in-plane loads, which further propagate through the periphery of the walls under out of plane loads making the walls quite unstable. The solution to this problem may be proper doweling of walls with the frame. The studies on TLD suggested that a properly designed TLD can offer a very good solution for controlling the seismic response of the RC frame structures under seismic excitation. The TLD is probably one of the least invasive solutions for retrofitting and can be installed by virtually not disturbing the occupants of the building at all. However, as with all other retrofit solutions, it has its own limitations too. One major limitation is to use TLD for relatively stiffer structures. As seen in this case, the damper size for a structure with fundamental frequency of 3Hz is only 80mm, which may seem to be too small for real life buildings. For softer structures, the size increases as the square of time period and the sizes become more practical. However, for structures having fundamental frequencies equal to or more than 2 Hz, a higher water depth ratio such as 0.3 may lead to better sizes. Also, under strong seismic excitations, such water depth ratios are found to have better response control, even though they strictly do not fall under the shallow wave theory applicability criteria.

In the third stage of tests under gradually increasing seismic loads, the responses were recorded at various levels due to the ground motions and the same were compared with the PGA of the base excitation. The curves suggested a gradual induction of damage in the structure, which was also confirmed by physically observing the damage patterns. It was observed that the very first cracks appear at the base of the columns but these cracks do not grow further, which may be attributed to the presence of plinth beams. Instead most of the latter cracks are formed at the first storey level which can be attributed to the high inertia at the storey level.

In the beginning the response of the structure raised steeply, though not proportionally, to the PGA level, which is attributed to gradual decrease in the elastic modulus of the material due to cracking. Later, as the PGA level was increased, the response of the structure was found to increase nonlinearly with respect to the PGA, which happened due to the higher damping achieved by the structure due to hysteretic energy dissipation. Mostly the damage could be classified as flexural failures of beams and columns, with some contribution of shear and torsion modes. Large spalling of concrete in the critical regions at the ends of beams and columns was observed, even from the core, due to the lack of confinement by lateral ties. In the end, the beam-column joints of the structure developed shear cracks displaying the so-called 'BJ' failure mode.

Although from the test results it seems that the model structure could resist an earthquake with a PGA level of 0.7 g, this may be misleading. This is because, the base acceleration was gradually increased in this case and at for acceleration level, certain damage occurred that altered the stiffness of the structure and also enhanced the damping through hysteretic energy dissipation. If the structure would have been exposed directly, in the as-built condition, to the higher PGA levels, the response could have been quite different and the structure might actually withstand a lower acceleration level only. However, as mentioned earlier, the objective of this test was not to qualify a real life structure by testing but to observe the various failure modes and sequence of events that may take place in such structures under earthquakes. Still, it may be concluded that the model structure could resist the low to moderate earthquakes reasonably well, but the performance of the structure under severe earthquakes was poor.

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