SEISMIC SHAKE TABLE TESTS ON A GRAVITY LOAD DESIGNED FRAME STRUCTURE – LINEAR TO NONLINEAR TESTING

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ABSTRACT

In order to understand the behavior of RC structures under seismic loads and to validate the models developed, experimental results are needed. 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, and generate useful information is made. The structure tested in this program had 3 storeys with an additional storey to simulate the footing to plinth level. In plan the structure was symmetric with 2 bays in each direction. The structure was a 1/3.33 scaled model of a part of a hypothetical prototype low rise structure. The structure was built with masonry infill panels in one direction and open frame in other direction. The first set of tests was done with masonry infill panels in one direction to obtain information on the stiffness increase due to addition of infill panels. The studies on the effect of masonry infill 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. In order to understand the inelastic dynamic behaviour of the reinforced concrete frame structures, after the tests with infill walls were over (within linear range), the walls were removed and the structure without walls was tested under biaxial ground motion with gradually increasing PGA levels till failure. 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. 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.

INTRODUCTION

Seismic response of reinforced concrete (RC) framed structures has been one of the major topics of interest for past few years now among the researchers around the world. The main reason behind this interest is the complex behaviour of the RC structures and their vulnerability to earthquakes, if not detailed properly. Past earthquakes have exposed the vulnerability of reinforced concrete structures and have shown that 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. 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. 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 [1] and generate useful information is made.

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, however whether the contribution of a panel would be more towards the mass or stiffness depends on the location of the same in the frame [2]. In the first stage of tests, 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.

Inelastic behavior 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. Past earthquakes have exposed the vulnerability of such 'non-seismically detailed' reinforced concrete frame structures and have shown that two most important and most frequently found deficiencies in such structures are no shear reinforcement in the joint and poor end anchorages leading to joint shear and bond failures respectively that may finally lead to partial or complete collapse of structures. Although, due to minimum flexural and shear design requirements, such structures may possess an inherent lateral strength to resist minor to moderate earthquakes, the performance of such structures under severe earthquakes may be extremely poor [3]. In this program, deficiencies that were included in the structure comprised of no special confining reinforcement and no ties in the joint.

Shake table tests are generally considered as the best way to gather useful information and understand the behavior of such structures under earthquakes. Therefore, in past, significant efforts have been put by researchers to perform such tests. Tests reported by [3,4,5,6] are a few examples of similar tests and some 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 [7], 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 [8]. Therefore, the challenge is to design a least distorted model within the constraints of shake table capacity and material availability.

In this work, shake table tests were conducted on the scaled model of a portion of a hypothetical low rise 3D frame structure that had a 3 (+ 1 Plinth) storey and 2 bay by 2 bay configuration and was designed following the provisions of Indian standard code for design of concrete structures without considering seismic action [1]. The experiments were conducted on the shake table with earthquake ground motions in two axes with a gradually increasing level of PGA that represented earthquakes from low to moderate to severe intensities.

DESCRIPTION OF STRUCTURE

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 Earthquake Engineering and Vibration Research Centre (EVRC) of Central Power Research Institute (CPRI) Bangalore. The shake table size is 3m x 3m 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 75mm x 100mm with a 50 mm thick slab. Fig 1 shows the details and photographic view of the structure before the test. All the reinforcement of 6mm or less was Mild steel reinforcement and 8 mm diameter bars were tor steel reinforcement. The slab reinforcement consisted of 6mm diameter bars at the rate of 100mm 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.

EXPERIMENTAL PROGRAM

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 0.075g, 0.1g and 0.125g 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 2 shows a typical input time history for the sine sweep tests corresponding to PGA of 0.1g. The masonry walls were then removed and almost equivalent masses were applied on the floors and the sine sweep tests were again carried out.

In the next 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 3). All the tests were conducted at the Earthquake Engineering and Vibration Research Centre (EVRC) of Central Power Research Institute (CPRI) Bangalore.

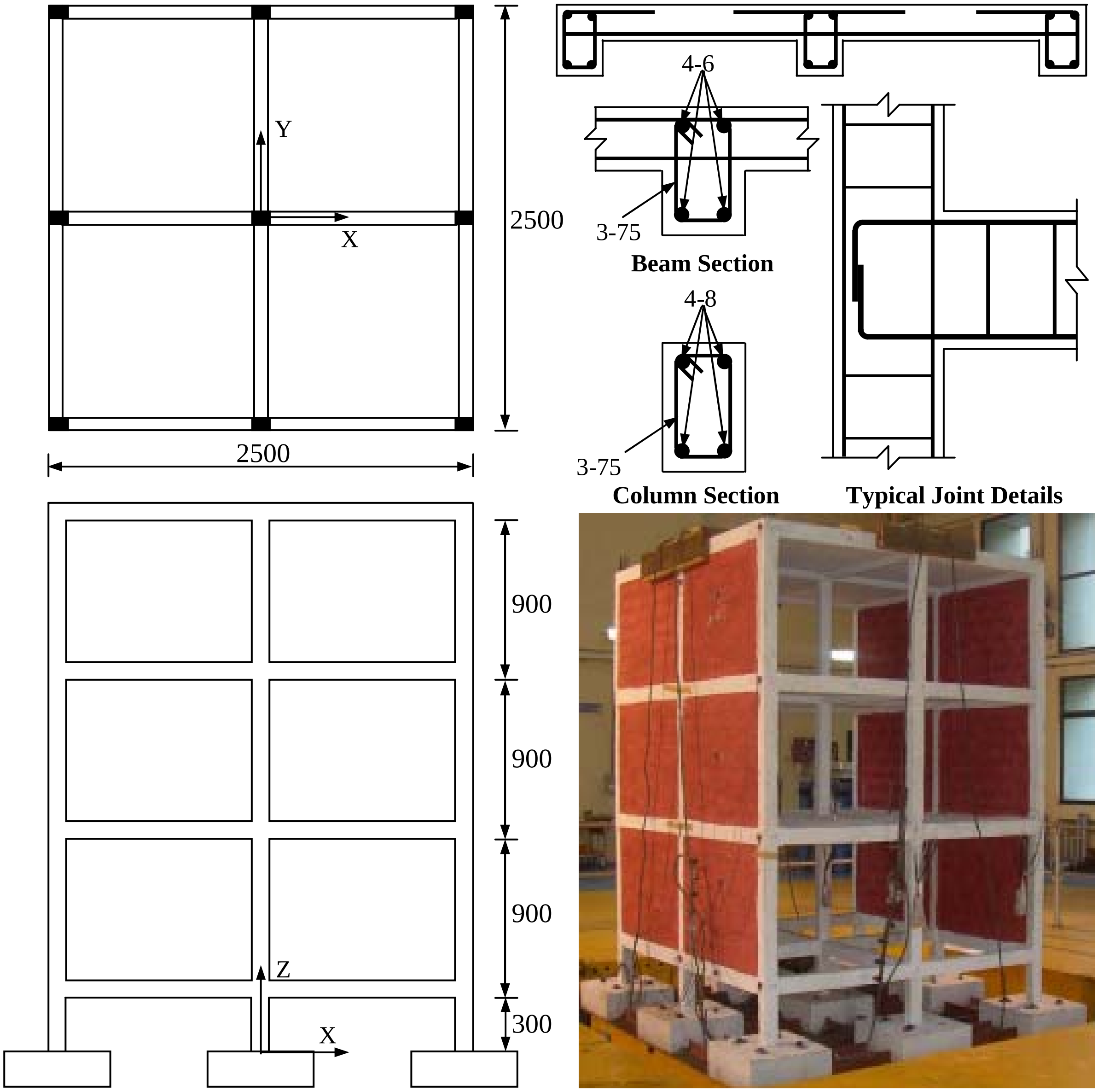


Fig 1 Description of Structure

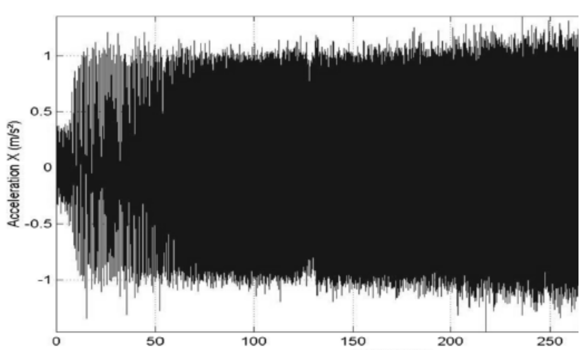


Fig 2 Time history for sine sweep tests

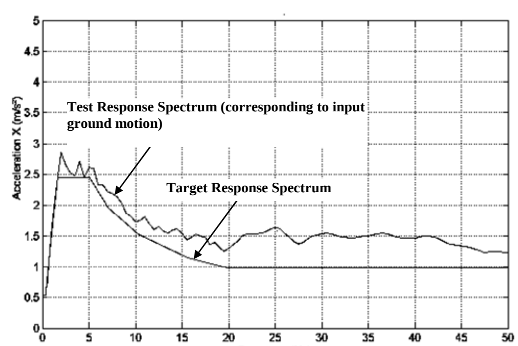


Fig 3 Target and test response spectrum for seismic tests

The tests were started with the biaxial time histories provided to the table with a PGA of 0.1g and gradually it was increased in steps of 0.1g till 0.7g. The limit of 0.7g was due to the shake table stroke limitations. The maximum available displacement stroke for the actuators of shake table was +/- 150mm. The maximum displacement in either direction for a PGA of 0.1g was 20mm and correspondingly, the maximum displacement in either direction for a PGA of 0.7g was around 140mm, 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.

RESULTS AND DISCUSSION

Results of 1st Stage of Experiments

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 0.075g, 0.1g and 0.125g. During these tests, cracks in the mortar at the corner of walls were observed first on ground storey walls (at an acceleration of 0.1g) and then on middle storey walls at an acceleration of 0.125g). 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 0.075g, 0.1g and 0.125g 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 0.075g, 0.1g and 0.125g respectively. However, no resonant frequency corresponding to second mode was obtained in any case.

Sine Sweep Tests on Structure in Vertical Direction

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

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 0.075g, 0.1g and 0.125g. 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 0.1g) and then on bottom storey walls at an acceleration of 0.125g). 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-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 0.075g, 0.1g and 0.125g respectively. The amplifications were 13.69, 13.34 and 10.85 against the acceleration levels of 0.075g, 0.1g and 0.125g respectively. 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 [9].

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.

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. A resultant of these two aspects may be beneficial or detrimental for the structure. For example if the bare frame structural frequency is such that further increase in frequency may be associated with constant or reducing forces, then addition of walls will reduce the base shear as well as moments on the frame members. However, if the frequency rise is associated with the increase in base shear to such an extent that the truss action is not beneficial anymore may be because the axial forces are too high or the walls themselves fail, then addition of walls may be detrimental to the seismic safety of the structure.

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 quite 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 cause 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.

Results of 2nd Stage of Experiments

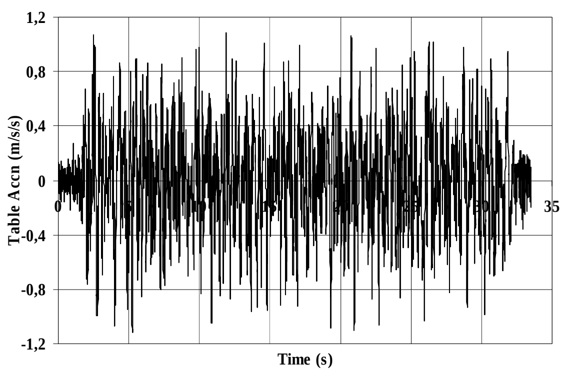


Fig 4 Acceleration time history of the shake table

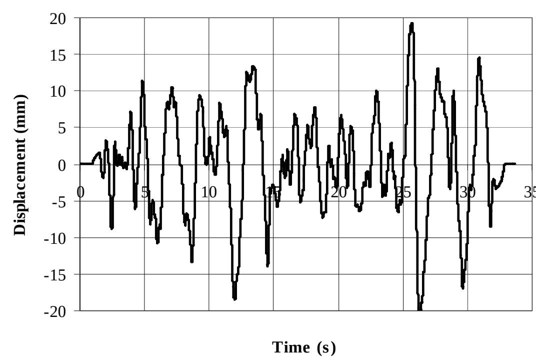
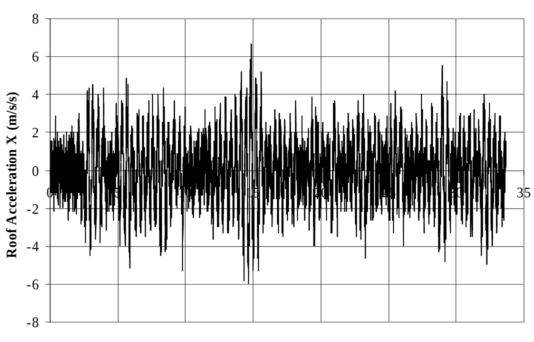
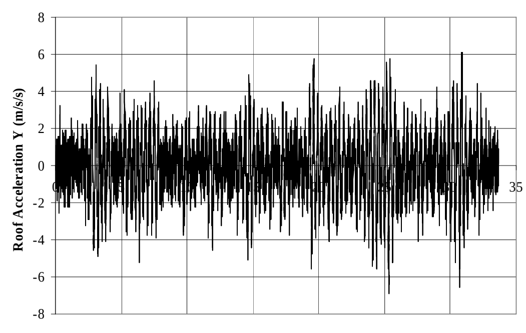


Fig 5 Displacement time history of the shake table

The series of tests started with simulated bidirectional earthquake with a PGA of 0.1g. The acceleration and the corresponding displacement time history for the ground motion are shown in Fig 4 and Fig 5 respectively. Same time history was provided in both X- and Y- directions. The response of the structure recorded at roof in X-direction is shown in Fig 6 (a) and that in Y-direction is shown in Fig 6 (b). Similar time histories were recorded at each floor level for both directions corresponding to each PGA level. A plot of the maximum acceleration v/s the PGA of base excitation for X- direction is shown in Fig 7 and the same corresponding to Y- direction is shown in Fig 8.



(a) X-Direction



(b) Y- Direction

Fig 6 Floor acceleration time history recorded at the roof level for a PGA of 0.1g PGA base excitation

The plots of Fig 7 and 8 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 7 and 8 that both the curves display an almost linear increase of maximum recorded roof acceleration in both the directions till a PGA level of 0.4g. However, as can be noticed, the acceleration values are not directly proportional to the PGA values, e.g. the 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.

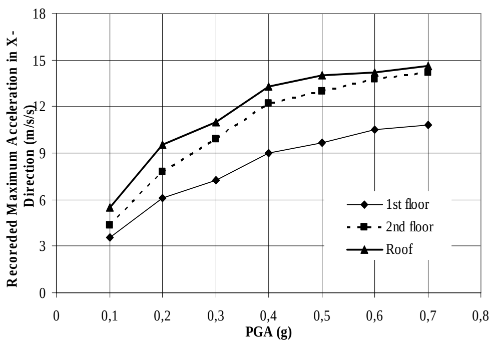


Fig 7 Peak roof acceleration v/s PGA for X-direction

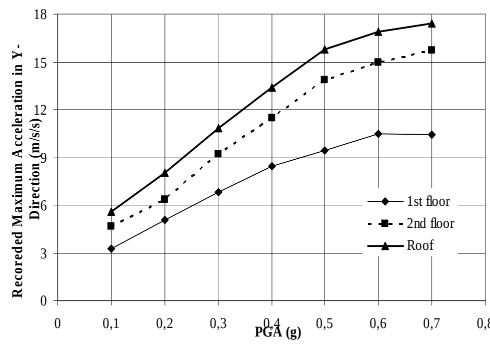


Fig 8 Peak roof acceleration v/s PGA for Y-direction

Fig 9 (a) shows the very first cracks that appeared at the base of the columns, close to the footing. This 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 9 (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 0.3g and 0.4g, a few more cracks started to appear. For a PGA of 0.3g, cracks appeared on the beams of first floor at the beam-column interface (Fig 9 (b)). However, again the cracks did not suggest any yielding of bars. At 0.4g, the beam flexural cracks opened up significantly and some spalling was also observed (Fig 9 (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.

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

Fig 9 Crack patterns observed during initial stages of the test

Fig 10 shows the damage patterns observed for the base excitation with a PGA level of 0.5g. 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. However, the cracks formed at the very base of the columns at the PGA of 0.2g (Fig 11 (a)) neither grew any further nor opened significantly. It was observed that the newer cracks that were formed at the plinth beam level (Fig 10) became significant. This suggests that plinth beam definitely has beneficial effects as it prevents the structure to form a soft story type of mechanism.

Fig 10 Damage patterns observed at PGA = 0.5g

Fig 11 Damage patterns observed at PGA = 0.6g and 0.7g

After the PGA of 0.5g, the spalling was significant and the already formed cracks were seen to open up widely. Fig 11 shows the damage patterns observed at the PGA of 0.6g and 0.7g, which was the maximum base acceleration level applied. The bottom beam bars became visible and 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 12 shows the damages in the joints. However, 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. 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.

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 0.1g showed that the structural frequency in Y-direction has come down to 1.025 Hz. 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.

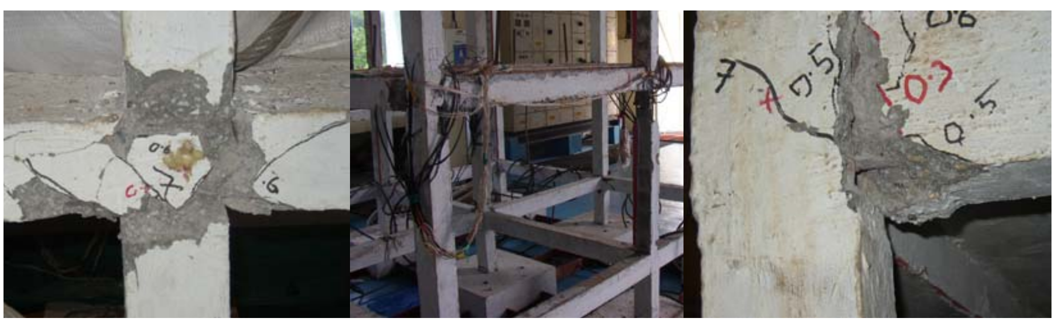


Fig 12 Damage patterns in Joints

CONCLUSIONS

The studies on the effect of masonry infill 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, which may or may be beneficial or even detrimental for the structure depending on a case to case basis. 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 curves of response of structure v/s PGA of input motion 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. 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. 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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