Structural Analysis and Design of Multi-Storey Complex for Earthquake Resistance

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Abstract: The changes in the modal parameters due to the changes in the characteristics of the structure are studied which can be used for structural damage identification and also new directions in the area of damage detection are explored. Two models have been tested with the varying condition of induced damage in the two different laboratories. The first model tested was a one bay by one bay by three story steel non shear frame model with two shear panel in both the planes in the form of bracing parallel to the direction of motion. The structure was given an input motion of artificially generated pink noise at the base in the shake table with different structural configuration such as with removal of single bracing on both sides of plane and removal of all the bracing of the structure. Since the damage in the structure was given by removal of bracings which were connected to the roof of the structure hence the stiffness reduction identified in the third story damage with varying amount of damage as per the respective cases was significant. The idea of keeping the input motion same in all the cases of the experiments led us to conclude clearly that the various dynamic parameters such a frequency, mode shape, modal strain energy, flexibility matrix and Frequency Response Function (FRF) changed according to the varying degree of damage for a particular system. Thus the purpose of the test to be able to determine any change or to locate the change was achieved. To develop a substantially accurate one dimensional analytical model of the structure, sensitivity analysis was also carried out for the beam column fixity. The matching of the frequency was set as the benchmark for the comparative study of the analytical and the experimental results. The transfer functions of the different floors for various cases correlated wellfor the experimental and analytical model.

Keywords: Structure, Analysis, Model, Building, Earthquake

I. Introduction:

To study the correlation of the degree of damage in a structure with the lateral displacement and to study the energy dissipation in a structure another test was carried out in the quasi-static laboratory on a one bay by one bay by one story concrete frame model without any infill. The damage was created by introducing crack in the model by applying quasi-static force and thus reducing the stiffness of elements which was measured in the form of decreasing frequency. Damage thus introduced through increased displacement led to the damage in the particular location which was determined through the clearly evident variations of the other derived modal parameters. The analysis results depict that the proposed empirical equation based on strain energy gives a good indication of damage location. Damage indices based on various parameters such as displacement, stiffness degradation given by various authors are compared and the result of which are in form the increasing or decreasing trends with the increase of damage. The result also shows that the state of the structure evolved after the energy dissipation of the input energy if determined from the free vibration test will be on a higher side than state of the structure obtained through the quasi static test. However the variation of stiffness defining the state of the structure decreased in the free vibration test and the quasi static test decreases as the damage is increased. The free vibration test carried on the structure was also used to locate the damaged element in the structure. The study of Neural Network based approach for damage detection was further carried out through two different approaches to determine the degree of damage in the structure. The first approach was to train the network with the frequency changes and mode shape changes while the second approach was to training the network with change in transfer function carried out for each floor independently. Through both the approach it was possible to obtain satisfactorily accurate degree of damage in floor of the case studies undertaken. The purpose of both approaches was to quantify the damage and to apply in different operational level buildings. After the earthquake event, the determination of frequency and mode shape of building would require ambient vibration test of the building again whereas in case of transfer function approach it has been assumed that the structure is instrumented during the earthquake. To validate both the approaches four storey and eight storey reinforced concrete building model was considered. The input data was artificially generated for the neural network from the program codes written. The training of the network was carried out for different combination of damage cases and the result showed that the accuracy of degree of damage detected in structure increased with the increase in the number of combination of damage considered for neural network training. Frequency and mode shapes being a linear system property was assumed to be determined through ambient vibration whereas the transfer function also is assumed to be taken for that part of the vibration in which the system vibrates linearly after the structure has been damaged. From the two methodologies developed it has been found that the accuracy to determine severity of damage decreases with increase in the number of storey being damaged. Further the instrumentation of first floor of the building would give best result in case the damage is detected based on transfer function change approach.

II. Experimental Study of Three Storey Steel Frame

It is a well-known fact that damages in the different types of structure are different. In the case of steel structure the damages are in the form of loosening of bolts, crack in the member, failure of rivets or bolts, failure of the welding, tearing of the member and corrosion of members. Each type of failure would accordingly induce the change in the structural behaviour. The different types of failure under laboratory conditions, being controlled failure, may not truly represent the actual mode of failure in the field. The most suitable and practical form of damage that can be induced in the structure to study in the laboratory is the member failure. The damages induced in the structure unless are not significant will not be able to produce significant change in the dynamic parameters.

2.1 Description of shake table laboratory:

This bi-directional shake table facility capable of generating longitudinal, vertical and pitch motion was developed at the Department of Earthquake Engineering IIT Roorkee in 1986 for testing of equipment and structures under earthquake excitation. Since then it has been used for seismic qualification of equipment and scaled model testing of civil engineering structures such water tank, masonry structures, base isolated masonry structures etc. The input motion to the shake table can be given either that of an earlier recorded real earthquake or it can be an artificial earthquake motion compatible to a given response spectra. The platform of shake table facility has a size of 3500 mm x 3500 mm in plan and it can take a maximum payload of 20tons.

2.2 Sensor and data acquisition: This shake table was recently upgraded with digital controls and 128 channels online data acquisition system (Shrikhande et al. 2002). The sensors used were single channel forced balanced accelerometers having natural frequency of about 100 Hz and damping of 70%. Accelerometers provide voltage output, which is proportional to acceleration of points where these are mounted. This analog acceleration time history is then fed to data acquisition system. The data acquisition system first conditions the analog signal to its requirement through amplifier and anti aliasing filter and then this conditioned analog signal is fed to AD Converter where digitization at prescribed sampling rate takes place. This digital data is then stored in hard disks of a computer. The basic functions of data acquisition system are achieved by using vendor supplied interactive software and hardware system. These systems can be standalone or coupled to a computer and have the facility of acquiring simultaneously multiple channels of data from various sensors. The data acquisition system acquired for the particular case study for generating output of the response of the structure had the desired specification such that the output generated had sufficient information to extract the modal parameter after processing accurately.

2.3 Description of model:

Model used in this work was made for testing for upgradation of shake table facility. The model is a steel frame of one bay by one bay with three storeys. The columns of this model were made of ISMB 150 and the beams of ISMB125. Angle ISA 100 x 100 x 8 was used as braces. Cleat angle with nuts and bolts arrangement has been used for connecting beams to the column. The slab of each floor except the roof was a pre-cast composite of 8 mm mild steel plate of size 2.0 m x 1.5 m in the bottom and concrete of 97 mm depth above plate. The concrete was held intact over the plate of 2.0 m by 1.5 m by 8 mm thick strips. The floor slab was supported over a grid of welded as well as bolted angles and channels, which transferred the load of slab to the beams of the steel frame. The grid was supported on all the beams at 0.25m and 1.75 m from centre of the column.

III. Proposed Method

Following are the parameters that were decided in order to perform the experiment under the same setup so that the records obtained are not affected by the various other parameters with the exception of changes being made to the structure.

Sensor location: To obtain the distributed response of the structure, one sensor was placed in the horizontal direction of motion of the shake table. On the model, sensors were placed on each beam-ends alternatively in the direction of motion. Thus, in total six sensor were installed with two in each floor. Figure below shows the details of the position of the sensor. The direction of the installed acceleration pick up was accounted for during the analysis of the data.

Figure 3.1: Typical sensor locations at beam - column joint of experimented model

Input Motion: For all the case of the experiment the model was subjected to pink noise (an artificially generated time history which contains full range of frequencies with the same power spectrum). The basis of the selection of the time history for experimentation is that all the modes of the structure should get excited.

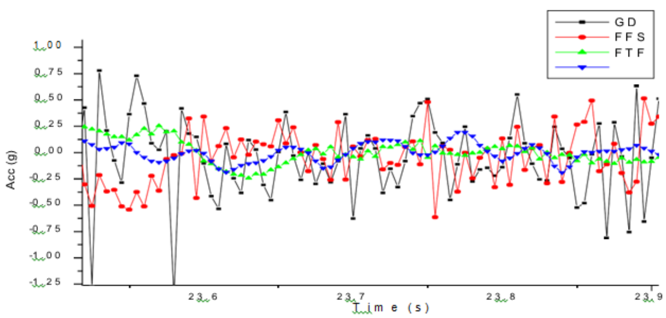


Figure 3.2: Enlarged view of the typical responses at different floors of the steel frame

Sampling frequency: Throughout the experiment the sampling rate was fixed at 200 samples per second. This sampling rate is sufficient to provide information for modal frequencies upto 100 Hz (Nyquist frequency) and three modes can be covered in this range.

Induced damage: The various damages that can be induced in the structure was change of mass by increasing the weight over the slab, removal of beam or braces depicting the reduction in the stiffness of the structure, loosening of the bolts that connects the beams to the columns which will also depict the reduction in the stiffness of structure. But carrying out these induced damages in laboratory are not practically feasible due to the limitation of the facility available for experimentation. The change in weight of the structure is not justified since during the earthquake or due to ageing effect the mass of the structure does not change. The beam column joint connection bolt gets loosened due to the dynamic loading on the structure. Such effect does lead to the decrease in the stiffness of the structure. Among all the options sequential removal of braces was an effective form of damage that can be introduced in the structure practically to match the expected target. The Damage on this frame was introduced by removing cross bracings in different combinations.

Frequency changes

Although steel is a homogenous material but various component of the structure i.e. structural member, connection plates, bolts, connecting beam and column all made of steel undergo different mode of failure when subjected to forces beyond their capacity. In case of structural members, cracks develop at a particular location when either tensile, bending or shear failure occurs. In case of connection plate tearing occurs from the edges of the drilled holes for bolts or rivet whereas in the case bolts either they get loosened of fails in shear. All these different types of failure lead to different stiffness reductions in the structure. While for the analysis purpose it is immaterial whether the joint or member has failed so long as the vibrational parameters are able to depict that the particular structural member is not being utilized to its full capacity for carrying load. Hence it is so necessary that the vibration based techniques should be able to pin point the damage the structure due to a particular component failure of the element as the indication of the failure of the element would be sufficient to approach at the address of the damage in the structure. This led to the next level of damage detection parameter that is mode shape changes. The difference in the mode shape is detected through comparison of the graphical output of the undamaged and the damage state of the structure. To what extent the mode shape is affected depends on the quantity and the location of damage. In the present study since the structure was held secured from the top in both plane of vibration by the bracings hence the deflection in the case of the intact structure at the roof level was less than the deflections at the first and the second floor. Once the damage was induced in the form of removal of bracing the displacement of the roof level increased and the behaviour changed.

Modal Strain Energy Change Ratio:

The elemental modal strain energy (MSE) is defined as the product of the elemental stiffness matrix and the second power of the mode shape component. The MSE for the jth element and ith mode before and after the occurrence of damage.

Transfer function changes:

A transfer function is frequency domain mathematical representation of the amplification or de-amplification between the input and output signal for a system. The transfer function thus tells us the frequency at which the resonance and the anti resonance condition of the systems will be achieved and the system's amplification or de-amplification at those particular frequencies will determine the behaviour of the system for a specific input. In case of buildings the transfer function is the ratio of Fourier amplitude of the response at the floors (acceleration / velocity / displacement) to Fourier amplitude of the ground.

Comparison of transfer function:

The comparison shows varying degree of amplification of the acceleration response at different floor for various cases with the frequency range from 0 to 16 Hz. For case1 the second frequency of the experimental result was more amplified for the first floor and second floor whereas for the third floor the results did not matched perfectly. For case 2 the second frequency of the experimental result was more amplified for the first floor whereas the second frequency of the analytical model was more amplified for the second floor and third floor. For case 3 the second frequency of the experimental result was more amplified for the first floor, second floor and third floor. The analytical models updated considering the experimental modal parameters as the baseline can be used to check the compliance of the structure with the updated codal provision by performing various analyses such as time history analysis or pushover analysis or response spectrum analysis on the structures, thus real time responses of the floors or stiffness degradation of the storeys if any can be generated and further retrofitting measures of the experimented structure can be suggested.

IV. Result

Table 4.1 Comparative value of natural frequencies (Hz) for experimented cases

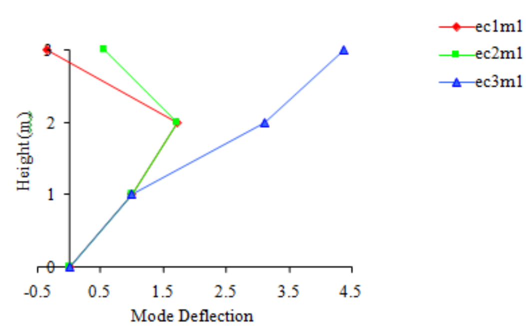
Mode	ec1	ec2	ec3	(ec1 - ec2) %	(ec1 - ec3) %	(ec2 - ec3) %
1	6.34	5.43	2.38	14.30	62.44	56.17
2	14.10	13.44	8.58	4.68	39.15	36.16
3	81.20	81.10	14.35	0.12	82.33	82.31

For this particular structure following observations can be made:

- Frequency decrease for case 2 is not substantial in comparison to case 3. This implies that the introduction of the extra cross bracing did not added much stiffness in the structure.
- For case 2 frequency decrease is in the decreasing order as we move towards higher mode. The reduction in the frequency of various modes is not same which is supportive to the fact that the damage does not cause same modal frequency changes.
- For case 3 frequency decrease is not with respect to some particular trend as we move towards higher mode.

Table 4.2 Comparison of Modal Strain Energy Change Ratio for all floors

Floor	ec1 vs ec2	ec1 vs ec3	ec2 vs ec3
1	0.3276	0.6795	0.8989
2	0.1409	0.3749	0.4683
3	1.0000	1.0000	1.0000



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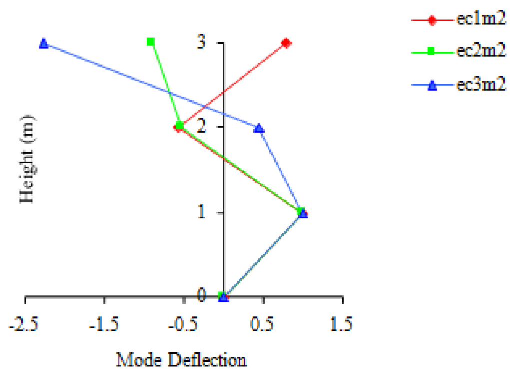

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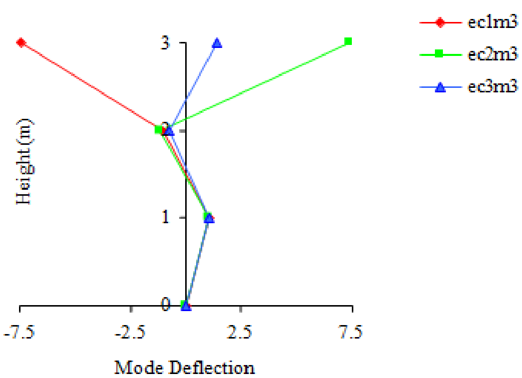

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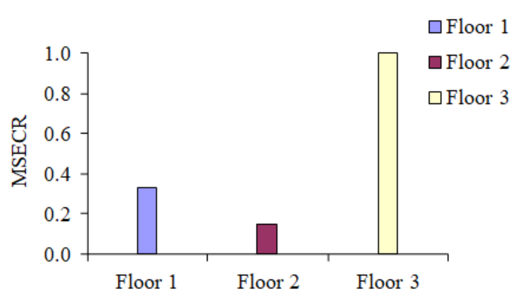
Figure 4.1 Comparison of mode shapes of experimented cases for (a) mode 1 (b) mode 2



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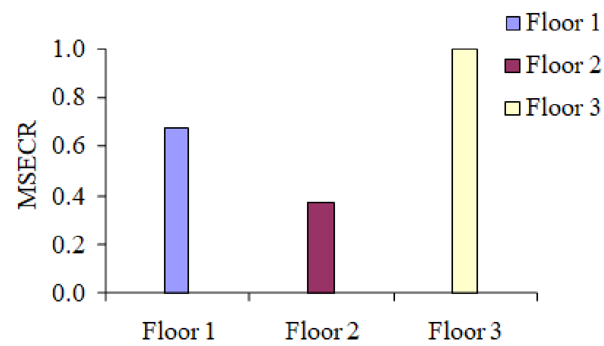

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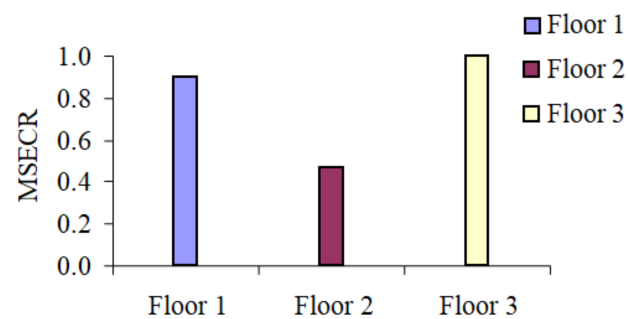

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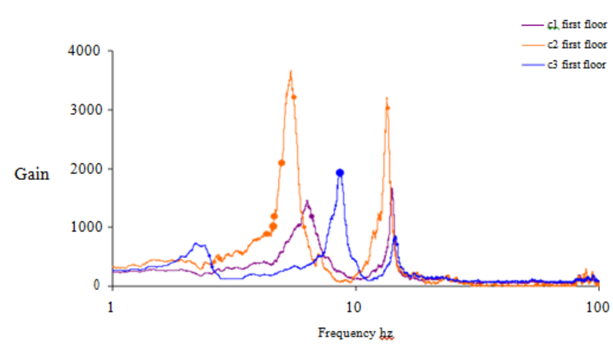
Figure 4.2: Comparison of modal strain energy change ratio of all floors for

c1 vs c2 (b) c1 vs c3 (c) c2 vs c3



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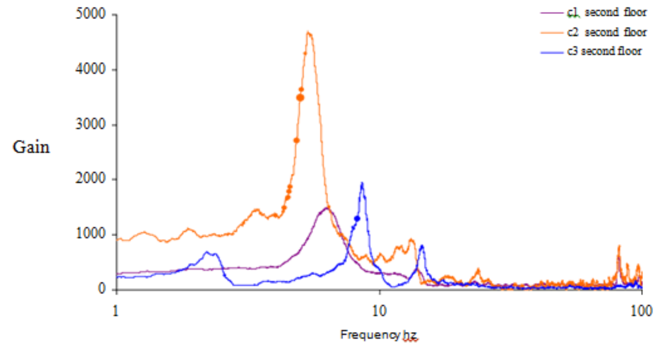

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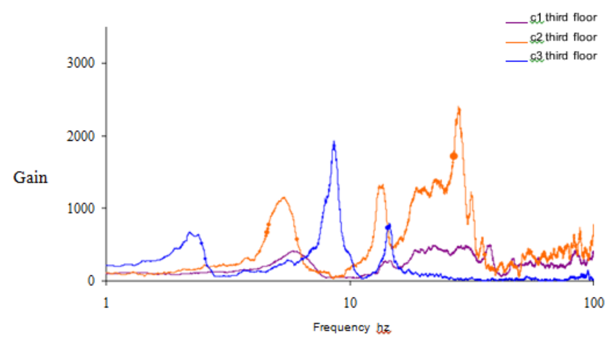
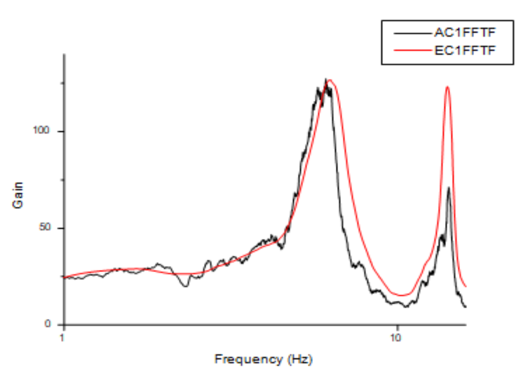


(c)

Figure 4.3: Comparison of transfer function of experimented cases for (a) first floor (b) second floor (c) third floor

Table 4.3 Modal Assurance Criteria between experimented cases

ec1 vs ec1	ec1m1	ec1m2	ec1m3	ec2 vs ec1	ec1m1	ec1m2	ecm3
ec1m1	1.0000	0.0084	0.0158	ec2m1	0.8098	0.0245	0.0979
ec1m2	0.0084	1.0000	0.1602	ec2m2	0.0199	0.0889	0.5663
ec1m3	0.0158	0.1602	1.0000	ec2m3	0.0565	0.5009	0.8526
ec3 vs ec1	ec1m1	ec1m2	ec1m3	ec3 vs ec2	ec2m1	ec2m2	ec2m3
ec3m1	0.1872	0.1199	0.7046	ec3m1	0.6065	0.3394	0.5207
ec3m2	0.2563	0.0856	0.8346	ec3m2	0.0091	0.5934	0.7334
ec3m3	0.0402	0.9353	0.3664	ec3m3	0.0173	0.0030	0.7401



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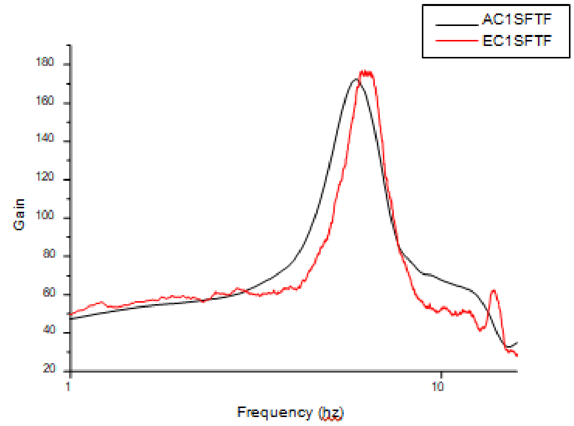

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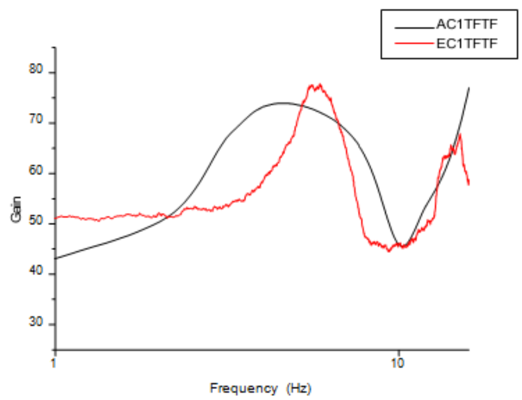

(c)

Figure 4.4: Comparison analytical and experimentally obtained transfer function of case 1 for (a) first floor (b) second floor (c) third floor

Conclusions

- The experimentation carried out on the steel frame model showed that the various modal parameters that is frequency, mode shape and modal strain energy and transfer function either changed or got shifted with respect to the change in structural configuration.
- The experimentally obtained decreased frequency for different cases is an indication of the reduction of the stiffness offered by the structure and this is only possible incase either number of structural members are reduced or the capacity of member or members are reduced.
- Since the induced damage was a global damage hence the change in the mode shape indicated a global behavioral change in the structure with relatively more deviation in the floor from where the structural member was detached.
- The normalized modal strain energy change ratio obtained quantified the relative reduction of stiffness in thedifferent storeys of the structure.
- The multiple frequency shifts from the transfer function plots showed the effect of induced structural change on different mode of the structure with a comparatively very clear result in the lower modes than in the higher mode.
- The decreased modal assurance criteria values as the damage was increased in the structures showed decreasing correlation between the mode shapes of the structure indicating increased damage.
- The first two frequencies obtained from the analytical model matched well with the experimental results but

the third frequency was notcomparable.

- The mathematically obtained mode shapes were found to be consistent with the experimental mode shape although the simulated model showed non orthogonality between the modes.

- The transfer function plots matched well for the experimental test case in which all the bracings were removed but the results varied for the cases in which all the bracings were attached.

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