Structural Engineering and Mechanics, Vol. 15, No. 4 (2003) 381-394 381
DOI: http://dx.doi.org/10.12989/sem.2003.15.4.381

Stochastic finite element based seismic analysis of framed structures with open-storey

M. Manjuprasad†, S. Gopalakrishnan‡ and K. Balaji Rao†

Structural Engineering Research Centre, CSIR Campus, Taramani, Chennai - 600 113, India

(Received April 3, 2002, Accepted March 8, 2003)

Abstract. While constructing multistorey buildings with reinforced concrete framed structures it is a common practice to provide parking space for vehicles at the ground floor level. This floor will generally consist of open frames without any infilled walls and is called an open-storey. From a post disaster damage survey carried out, it was noticed that during the January 26, 2001 Bhuj (Gujarat, India) earthquake, a large number of reinforced concrete framed buildings with open-storey at ground floor level, suffered extensive damage and in some cases catastrophic collapse. This has brought into sharp focus the need to carry out systematic studies on the seismic vulnerability of such buildings. Determination of vulnerability requires realistic structural response estimations taking into account the stochasticity in the loading and the system parameters. The stochastic finite element method can be effectively used to model the random fields while carrying out such studies. This paper presents the details of stochastic finite element analysis of a five-storey three-bay reinforced concrete framed structure with open-storey subjected to standard seismic excitation. In the present study, only the stochasticity in the system parameters is considered. The stochastic finite element method used for carrying out the analysis is based on perturbation technique. Each random field representing the stochastic geometry/material property is discretised into correlated random variables using spatial averaging technique. The uncertainties in geometry and material properties are modelled using the first two moments of the corresponding parameters. In evaluating the stochastic response, the cross-sectional area and Young's modulus are considered as independent random fields. To study the influence of correlation length of random fields, different correlation lengths are considered for random field discretisation. The spatial expectations and covariances for displacement response at any time instant are obtained as the output. The effect of open-storey is modelled by suitably considering the stiffness of infilled walls in the upper storey using cross bracing. In order to account for changes in soil conditions during strong motion earthquakes, both fixed and hinged supports are considered. The results of the stochastic finite element based seismic analysis of reinforced concrete framed structures reported in this paper demonstrate the importance of considering the effect of open-storey with appropriate support conditions to estimate the realistic response of buildings subjected to earthquakes.

Key words: stochastic finite element method; seismic analysis; framed structure.

1. Introduction

With the increased demand for urban habitat infrastructure, there is an increased need for the construction of multi-storey buildings with reinforced concrete frames. While constructing such

† Scientist
‡ Director Grade Scientist

buildings it is a common practice to provide parking space for vehicles at the ground floor level. This floor will generally consist of open frames without any infilled walls and is called an open-storey. The common practice is to design the reinforced concrete frames of buildings by neglecting the presence of infilled walls in the upper floors. It is well known that even engineered structures, such as, reinforced concrete frames, located in the seismically active zones suffer damage to varying degrees depending on the intensity resulting in socio-economic losses. During the January 26, 2001 Bhuj (Gujarat, India) earthquake of magnitude 6.8, a large number of reinforced concrete framed buildings with open-storey at ground floor level, suffered extensive damage and in some cases catastrophic collapse. In several places, the presence of sandy soil due to deep riverbeds below the buildings and liquefaction of the soil in the region during the earthquake resulted in weak support conditions leading to failure of the buildings. This has brought into sharp focus the need to carry out systematic studies on the seismic vulnerability of buildings with open-storey and the necessity for evaluating the design of such buildings for resistance to earthquakes.

In general, the system parameters and loads are uncertain in nature and vary stochastically over the space (and time). The uncertainties in system parameters such as material properties and geometric dimensions are due to variations in the quality of construction. The estimation of realistic response and vulnerability of the reinforced concrete framed structures with open-storey subjected to seismic excitations should therefore take into account the stochasticity in the loading and the system parameters by modelling them as random fields.

It is known that the finite element method is an effective tool available to analyse large and complex structures encountered in engineering practice. In the conventional finite element analysis, it is assumed that the system parameters are deterministic. The stochastic analysis, in general, refers to the explicit treatment of uncertainty in any quantity considered. The stochastic finite element analysis technique combines the two methodologies, namely, the finite element analysis and the stochastic analysis. The stochastic finite element technique can be effectively used to model the random fields due to loading and system parameters while carrying out the stochastic dynamic response analysis of structures.

Experience gained from post-disaster damage survey of buildings conducted after the Bhuj earthquake motivated the authors to initiate studies on the seismic vulnerability of reinforced concrete framed buildings with open-storey. This paper presents the results of stochastic finite element analysis of a five storey reinforced concrete framed structure with open-storey subjected to standard seismic excitation, carried out at Structural Engineering Research Centre, Chennai. From the review of literature, it is found that published literature on such studies is scanty. The effect of open-storey is modelled by suitably considering the stiffness of the infilled walls in the upper storey using cross bracing. In order to account for changes in soil conditions observed during earthquakes, both fixed and hinged supports are considered for analysis. In the present study, only the stochasticity in the system parameters is considered. The stochastic finite element method used for carrying out the analysis is based on perturbation technique. In evaluating the stochastic response, the cross-sectional area and Young’s modulus are considered as independent random fields. To study the influence of correlation length of random fields, different correlation lengths are considered for random field discretisation. The spatial expectations and covariances for displacement response at any time instant are obtained as the output. The results of the stochastic finite element based seismic analysis of reinforced concrete framed structures reported in this paper demonstrate the importance of considering the effect of open-storey with appropriate support conditions to evaluate the realistic response of buildings against earthquakes.

2. Stochastic finite element technique

2.1 *Brief review*

From the review of existing literature on stochastic finite element technique it is found that several approaches are being pursued in this field. Some of the well known approaches include spectral methods (Ghanem and Spanos 1991), perturbation technique (Kleiber and Hien 1992) weighted integral approach (Choi *et al*. 2000), etc.

The approach based on perturbation technique has been widely used in the field of stochastic mechanics to estimate the response statistics primarily due to its analytical tractability and savings in computational time. This technique is most effective when fluctuations of the random field variables are small and the probability density functions have decaying tail. It is reported that it performs well when the maximum variations are about 15 percent. In practice, for the construction of multistorey buildings, system parameter variations more than 15 percent are not admissible. Hence, this technique has been used in this study to evaluate the stochastic dynamic response of framed structures with system uncertainties.

2.2 *Methodology*

A computer program for stochastic finite element dynamic analysis of framed structures based on perturbation technique (Kleiber and Hien 1992) has been implemented and used for carrying out the studies reported in this paper. The program can handle the analysis of framed structures with parameters described deterministically and/or stochastically. The computer program has been designed to deal with dynamic deterministic and stochastic problems of medium and large-scale three-dimensional frames. In the estimation of stochastic response of the structure, time invariant uncertainties in geometry and material properties such as cross-sectional area, length, Young's modulus and mass density of the structural members were modelled using the first two moments of the corresponding parameters. In the present study, each random variable representing the field value of an element is approximated by its spatial average over the element, i.e., the element random variable is defined as the spatial average of the random field over the element domain and is specified at the centroid of the element. The spatial expectations and covariances for displacement response at any time instant can be obtained as the output.

In order to solve the problem efficiently, an approach based on the random variable transformation (standard normal transformation) and the so-called two-fold transformation technique (two-fold superposition technique) is used. The main concepts of the technique are:

- Transformation of the system with correlated nodal random variables to a system with uncorrelated nodal random variables through a standard eigen-problem
- Transformation of the coupled system of equations of motion, to an uncoupled system by generalised eigen-problem
- Superposition of the resulting uncorrelated and modal responses to obtain complete solution

In this technique, only a few highest modes (random variable transformation) are sufficient to approximate randomness in the system and only a few lowest modes (generalised coordinate transformation) are sufficient to describe the dynamic response of the system. A common feature of both procedures is that they are based on the change of basis of coordinates through orthogonal transformation. The stochastic finite element algorithm based on two-fold superposition technique offers an efficient tool for the stochastic analysis of structures. The transformation of correlated random variables to the set of uncorrelated variables is used to significantly reduce the complexity of the problem. Integration of uncoupled equations of motion is performed using the Newmark's method.

3. Stochastic finite element model of the structure

3.1 Geometry

Based on the experience gained during extensive post-disaster damage survey of buildings carried out by the first author after the recent Bhuj earthquake, a five-storey three-bay reinforced concrete framed structure with open-storey was selected to study the seismic response. The structure typically represents the type of buildings found extensively damaged during the earthquake. The cross-sectional dimension of reinforced concrete beams and columns is taken as 230×450 mm. The effect of open-storey is modelled in the frame by suitably considering the stiffness of infilled brick walls in the upper storey using cross bracing. The infilled wall is modelled as an equivalent diagonal strut/tie (Achyutha *et al*. 1994). The model is based on the equivalent diagonal strut concept proposed by Smith and Carter (1969). The thickness of the infilled wall is assumed to be 230 mm. The details of the models used to idealize the bare frame (without infilled walls) and open-storey frame (with infilled walls) are shown in Figs. 1(a) and 1(b).

3.2 Loading

The frame is subjected to horizontal seismic base excitation due to El Centro earthquake. This is generally considered as a standard excitation because it contains energy over a wide range of

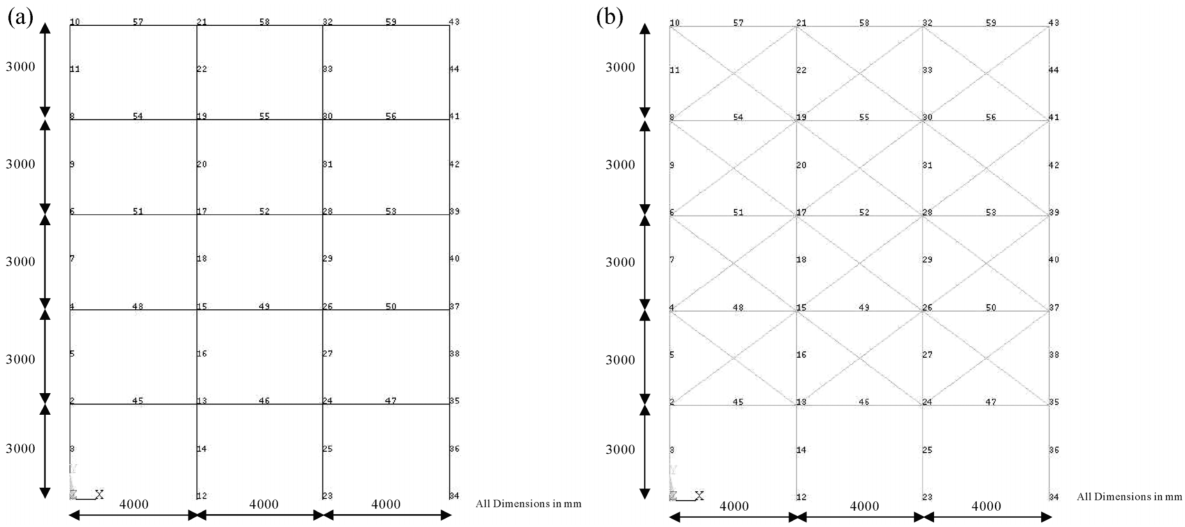

Fig. 1 (a) Geometry and finite element model of the bare frame, (b) Geometry and finite element model of the open-storey frame)

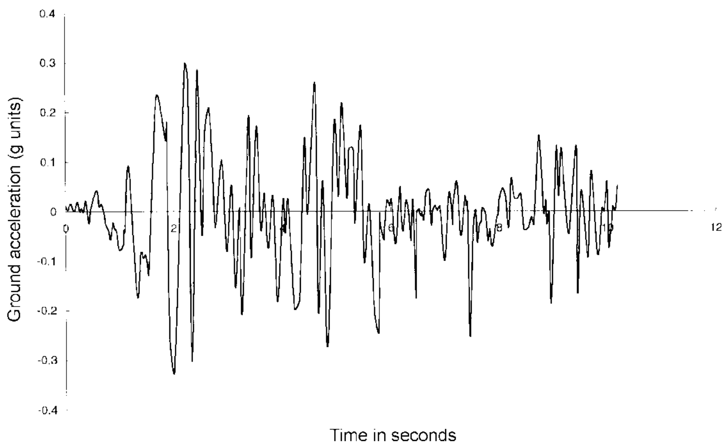

Fig. 2 Horizontal ground acceleration time history for N-S component of El Centro earthquake

frequencies (0.33 Hz - 3 Hz) (Fintel 1974). The north-south component of the horizontal acceleration time history of the earthquake is shown in Fig. 2.

3.3 Material properties

For concrete, the mean values of the Young's modulus, mass density, Poisson's ratio and damping factor, are assumed as 2.0×10^4 N/mm^2, 2500.0×10^{-12} N-sec^2/mm^4, 0.15 and 0.05, respectively. For brick masonry, the mean values of Young's modulus, mass density, Poisson's ratio and damping factor, are assumed as 0.132×10^4 N/mm^2, 1900.0×10^{-12} N-sec^2/mm^4, 0.15 and 0.05, respectively in both tension and compression.

3.4 Finite element discretisation

Each of the vertical continuous columns is discretised by 10 equal-length three dimensional beam elements (with 2 elements between any two floors). Each of the horizontal continuous beams (at each floor level) is discretised by 6 equal-length three dimensional beam elements (with 2 elements between any two columns). Each of the equivalent diagonal strut/tie representing an infilled wall panel is modelled using a three-dimensional beam element with bending stiffness neglected. The five-storey three-bay framed structure was thus discretised by using 40 elements for columns, 30 elements for beams and 24 elements for the struts/ties. The discretisation has resulted in a total of 94 elements.

3.5 Random field discretisation

The lengths of various members and mass densities are treated as deterministic. The member cross sectional areas and Young's modulus are treated as independent random fields. Each random field is modelled as a vector of correlated element random variables using spatial averaging technique. The discretisation used for modelling the random fields is the same as that used for finite element modelling resulting in a vector of 94 element random variables for each random field. By default, it is assumed that the 10 element random variables in each vertical column are correlated, the 6 element random variables in each continuous beam are correlated and each of the struts/ties is modelled by an element random variable. It is further assumed that there exists no correlation among/between the element random variables of different vertical columns, horizontal continuous beams and diagonal struts/ties. The correlation function for each column, beam and strut/tie for the various random fields is assumed to be an exponential function as given below (Kleiber and Hien 1992):

$$\mu(X_\rho, X_\sigma) = \exp\left(-\frac{|x_\rho - x_\sigma|}{\lambda}\right) \tag{1}$$

Here, λ is the correlation length factor, X_ρ, X_σ are the element random variables obtained by discretising the independent random fields (namely Young's modulus or element cross-sectional area) and x_ρ, x_σ are the normalised centroidal distances of respective elements from the first element. In the above, ρ, σ refer to the element random variable numbers. For example, for the columns with 10 element random variables the values of ρ, σ vary from 1 to 10 and the values of x_ρ, x_σ are given by $x_1 = 0.0, \ldots, x_{10} = 1.0$. Similarly, for beams with 6 element random variables $x_1 = 0.0, \ldots, x_6 = 1.0$ and for struts/ties with single element random variable $x_1 = 0.0$. A separate program was developed to generate the covariance matrix for the structure for any given random field.

3.6 Solution parameters

To solve the initial-value problem, the two-fold superposition technique, as described earlier, is used with 10 lowest eigen-pairs. The set of 94 correlated random variables in each random field is transformed to a set of uncorrelated variables, out of which the 10 highest modes are used in the calculations. The equations are integrated with respect to time using 1020 sampling intervals (time step length $\Delta t = 0.01$ seconds). The secular terms are eliminated using the frequency range factor $r = 0.3$ and 1020 Fourier terms.

4. Results of seismic response analysis

Unless otherwise specified the following parameters are assumed while carrying out the analysis: open-storey-frame configuration, fixed support condition, random field model for the cross sectional area with a coefficient of variation of 0.05. The correlation length factor required for stochastic field modelling is taken as 0.50.

4.1 Effect of open storey

From the deterministic free vibration analysis of the bare frame considered, it was noted that the frame vibrates predominantly in MDF mode (discrete Multi Degree Freedom system mode), with a fundamental frequency of 3.03 Hz, as shown in Fig. 3(a). However, the open-storey frame vibrates in SDF mode (Single Degree Freedom system mode), with a fundamental frequency of 2.84 Hz, as shown in Fig. 3(b). The mode shapes shown in Figs. 3(a) and 3(b) correspond to 1st mode of vibration.

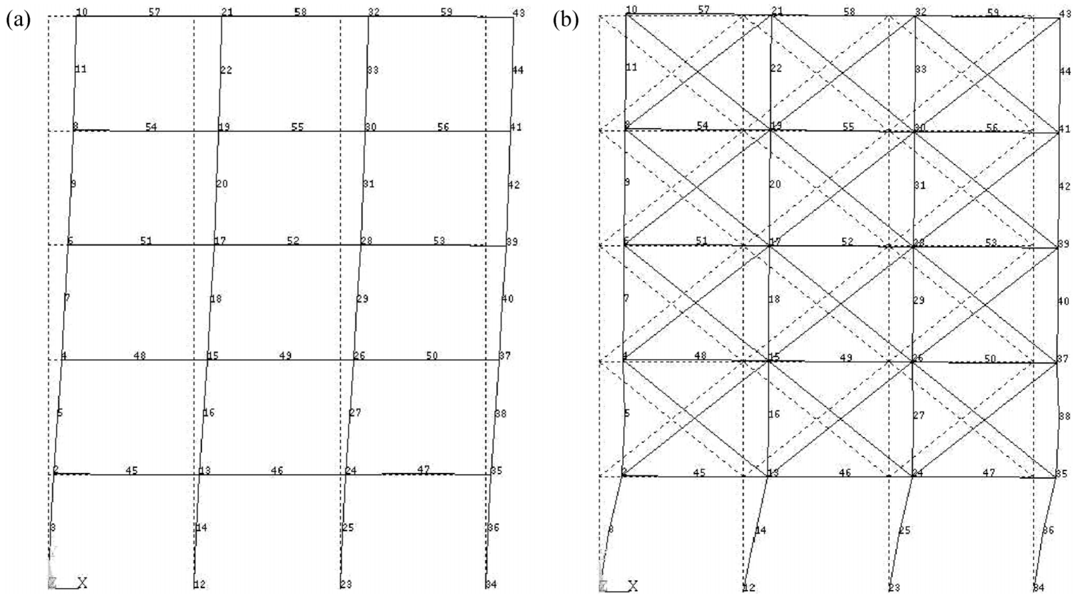

Fig. 3 (a) First vibration mode of the bare frame, (b) First vibration mode of the open-storey frame

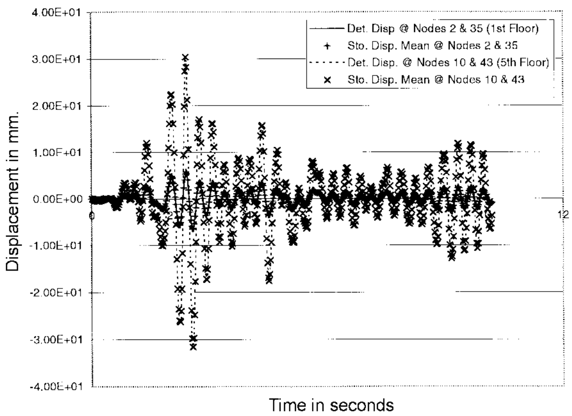

Fig. 4 Horizontal displacement time history for bare frame - fixed base (coefficient of variation of C/S area = 0.05, correlation length factor = 0.50)

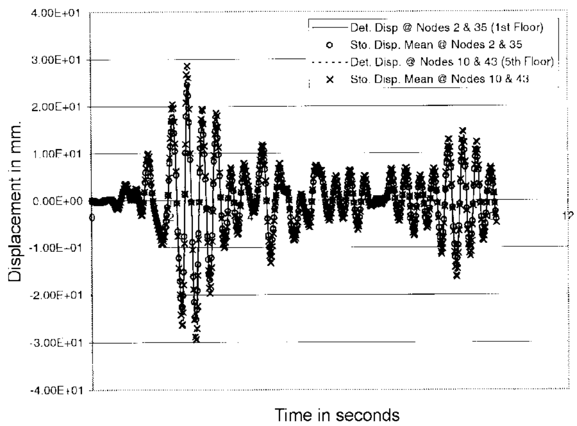

Fig. 5 Horizontal displacement time history for open-storey frame - fixed base (coefficient of variation of C/S area = 0.05, correlation length factor = 0.50)

The time history of the mean values for horizontal displacement estimated from SFEA, at 1st floor level (Nodes 2 and 35) and 5th floor level (Nodes 10 and 43) are shown in Fig. 4 for the bare frame and in Fig. 5 for the open-storey frame. The deterministic solutions are also given in the same figure for comparison. From Figs. 4 and 5 it is seen that the mean displacements coincide with the respective deterministic displacements at all times for all nodes considered. From the values of displacements obtained at various floor levels it is found that the 1st mode of vibration contributes significantly to the response of the frames. The mean displacement at 5th floor level is more than that at 1st floor level for the bare frame, which is governed by MDF mode of vibration. However, for the open-storey frame, which is governed by SDF mode of vibration, the mean displacement at 1st floor level is almost the same as that at 5th floor level and is approximately equal to the maximum mean displacement for the bare frame case. This observation suggests that the situation is more critical in the case of open-storey frame for which the maximum displacement is attained at the 1st floor level itself and the top floors move almost like a rigid body. This results in the release of most of the energy at the 1st floor level. This can lead to nonlinear behaviour at the 1st floor level under strong motion earthquakes resulting in increased chances of failure. This behaviour is in corroboration with the observations made during the post-disaster damage surveys. That is, the reinforced concrete framed structures with open-storey at ground floor level have shown severe distress at the junction of beam column joints at 1st floor level with no significant damage in the upper floors. However, the structures without open-storey suffered uniform damage, of lesser degree, throughout the height of the structure.

The time history of the covariances for horizontal displacement at the 1st floor level (Nodes 2 and 35) and 5th floor level (Nodes 10 and 43) are shown in Figs. 6(a), 7(a) for the bare frame and in Figs. 6(b), 7(b) for the open-storey frame. As can be observed from Figs. 6(a) and 6(b), the values of displacement variances are significantly less (by approximately one order) for the case of open-storey frame indicating that the fluctuations of response around the mean are not significant. This could be due to change in mode of vibration from MDF mode for the bare frame to SDF mode for the open-storey frame. In bare frame, the response is influenced by the stiffness of various members of the structure. In the open-storey frame the stiffness of ground floor column members alone have significant influence on the response. Hence, randomness in cross-sectional area of various members, which affects the stiffness of the structural members, has insignificant influence on variances of displacement response of the open-storey frame.

From Figs. 7(a) and 7(b), it is observed that, for the open-storey frame, the values of displacement covariance between nodes in the 1st floor, between the nodes in the 5th floor and between the nodes

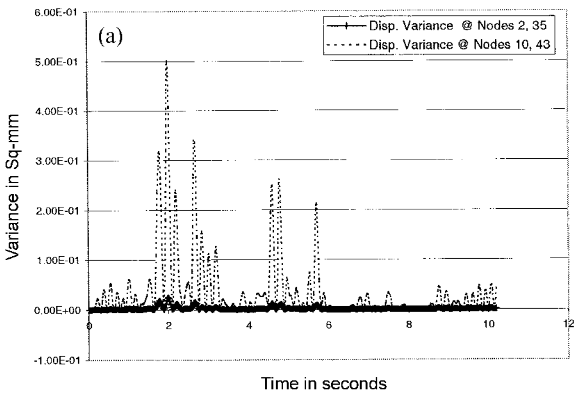

Fig. 6(a) Horizontal displacement variance time history for bare frame - fixed base (coefficient of variation of C/S area = 0.05, correlation length factor = 0.50)

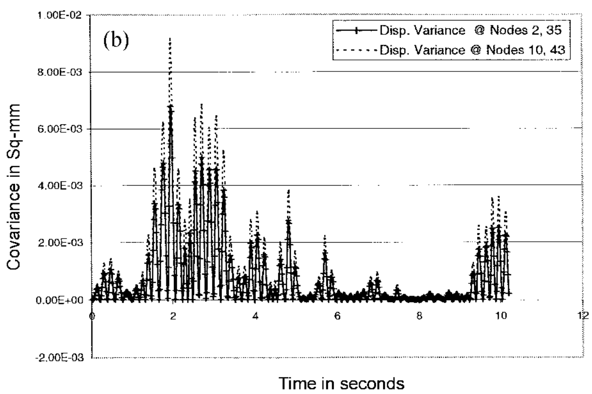

Fig. 6(b) Horizontal displacement variance time history for open-storey frame - fixed base (coefficient of variation of C/S area of frame members = 0.05, coefficient of variation of C/S area of walls = 0.05, correlation length factor = 0.50)

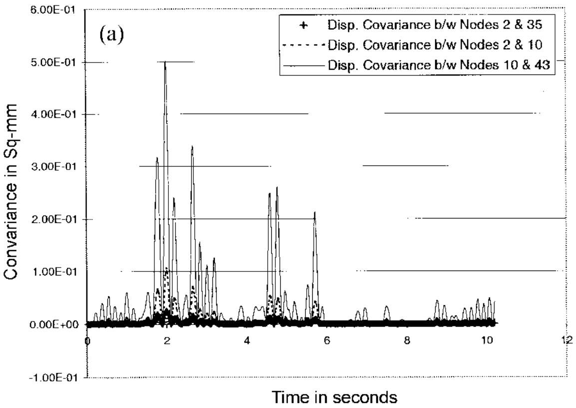

Fig. 7(a) Horizontal displacement covariance time history for bare frame - fixed base (coefficient of variation of C/S area = 0.05, correlation length factor = 0.50)

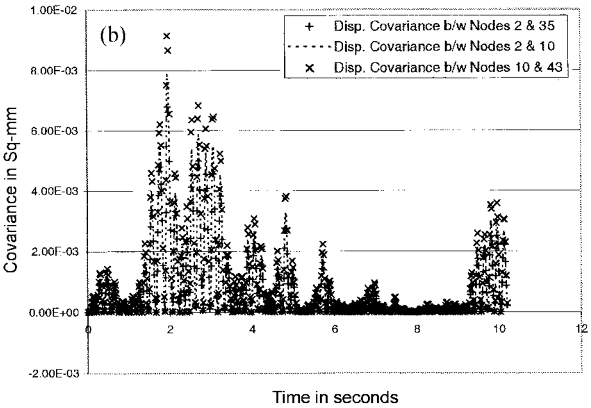

Fig. 7(b) Horizontal displacement covariance time history for open-storey frame - fixed base (coefficient of variation of C/S area of frame members = 0.05, coefficient of variation of C/S area of walls = 0.05, correlation length factor = 0.50)

at 1st and 5th floor levels are approximately the same. Hence, for the open-storey frame the displacements are perfectly correlated within the floors as well as across the floors. This could be because the mass of the structure above the 1st floor level acts like a single lumped mass. This is in contrast to the bare frame, which behaves more like a multi degrees of freedom system with distributed mass across the floors. This behaviour is to be expected since there is a change in the mode of vibration of the structure with open-storey from MDF mode to SDF mode.

4.2 Effect of support conditions

Results obtained for the open-storey frame in the previous section by assuming fixed support condition give a lower bound on the response of the structure. In order to account for possible changes in foundation soil conditions due to liquefaction during strong motion earthquakes, hinged support condition is considered for the present study. The condition is expected to represent the worst condition and hence give an upper bound on the response of the structure.

From the free vibration analysis carried out, it was observed that the open-storey frame with hinged support also vibrates in SDF mode. However, the fundamental frequency is reduced to 1.45 Hz, which is also in the range of the dominant frequency contained in the excitation.

The time history of the mean values for horizontal displacement at 1st floor level (Nodes 2 and 35) and 5th floor level (Nodes 10 and 43) are shown in Fig. 8. The deterministic solutions are also given for the purpose of comparison. From Fig. 8, it is seen that the stochastic mean displacements coincide with the respective deterministic displacements at all times for all nodes. As expected, for the open-storey frame, governed by SDF mode of vibration, the mean displacement at 1st floor level is almost the same as that at 5th floor level.

By comparing Fig. 8 (with hinged supports) with Fig. 5 (with fixed supports), it is observed that for the hinged case the amplitude of vibration is magnified by nearly two times and the period of vibration is elongated. The effect of actual soil condition can be expected to be in between these

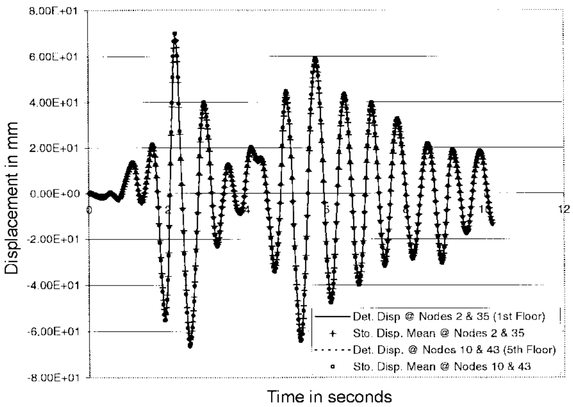

Fig. 8 Horizontal displacement time history for open-storey frame - hinged base (coefficient of variation of C/S area = 0.05, correlation length factor = 0.50)

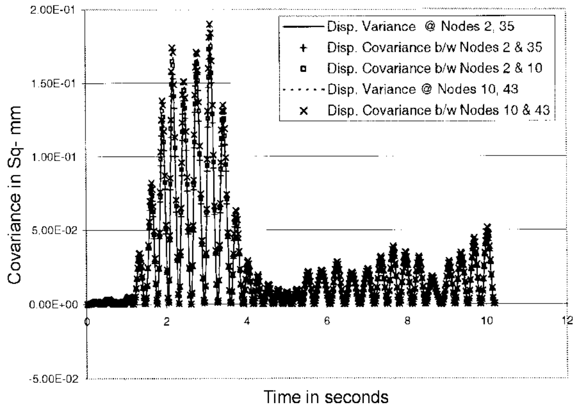

Fig. 9 Horizontal displacement covariance time history for open-storey frame - hinged base (coefficient of variation of C/S area of frame members = 0.05, coefficient of variation of C/S area of walls = 0.05, correlation length factor = 0.50)

two support conditions. This study brings out the importance of considering the effect of soil stiffness in order to evaluate the realistic response. An increase in the amplitude/deflection will result in increased chances of failure of the structure. During post-disaster damage survey, it was observed that the buildings in certain zones of the Ahmedabad city which were founded on Sabarmathi riverbed, were extensively damaged during the Bhuj earthquake. Similar buildings, not located on the riverbed were not so extensively damaged. This observation could be attributed to the liquefaction of the soil during the earthquake resulting in weak support conditions.

The time history of covariances for horizontal displacement at the 1st floor level (Nodes 2 and 35) and 5th floor level (Nodes 10 and 43) is shown in Fig. 9. As can be observed from these plots, the values of displacement covariances are significantly high (by approximately one order) for the case with hinged supports when compared to the case of fixed supports (Figs. 6(b) and 7(b)). This indicates that the fluctuations of response around the mean are quite significant for the case of hinged support. This uncertainty in the response due to changes in support conditions has to be taken into account by using appropriate safety factors while designing the structures. It can also be observed that for open-storey frame with hinged support, the values of displacement covariance between nodes in the 1st floor level, between the nodes in the 5th floor level, as well as between the nodes at these floor-levels are almost coincident. Thus, for the open-storey frame with hinged support the displacements are more perfectly correlated within the floors as well as across the floors. This reemphasizes that the complete structure above the 1st floor level behaves like a single lumped mass.

4.3 Effect of randomness in the value of Young's modulus

The Young's modulii of concrete in the frame and brick masonry in the infilled walls are stochastic in nature. To account for this, these quantities are considered as random fields. To study the effect of changes in the coefficient of variation of Young's modulus of masonry on response of

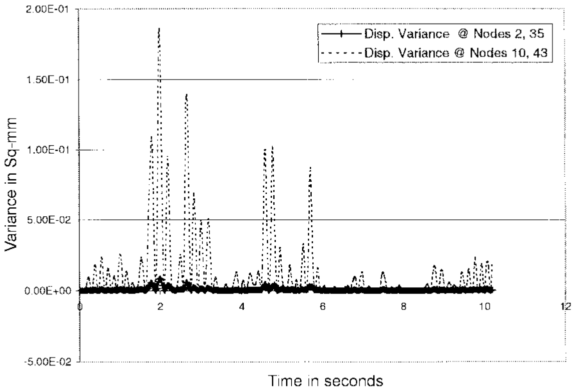

Fig. 10 Horizontal displacement variance time history for bare frame - fixed base (coefficient of variation of Young's modulus = 0.05, correlation length factor = 0.50)

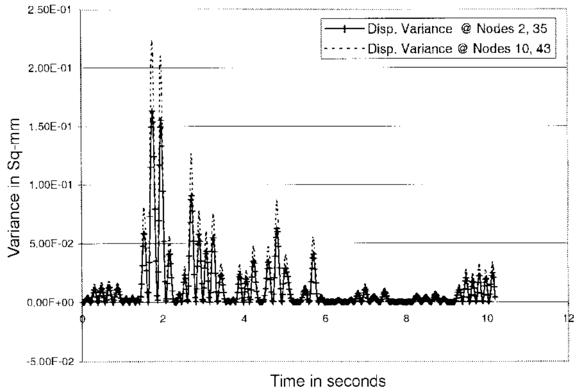

Fig. 11 Horizontal displacement variance time history for open-storey frame - fixed base (coefficient of variation of Young's modulus of concrete = 0.05, coefficient of variation of Young's modulus of masonry = 0.05, correlation length factor = 0.50)

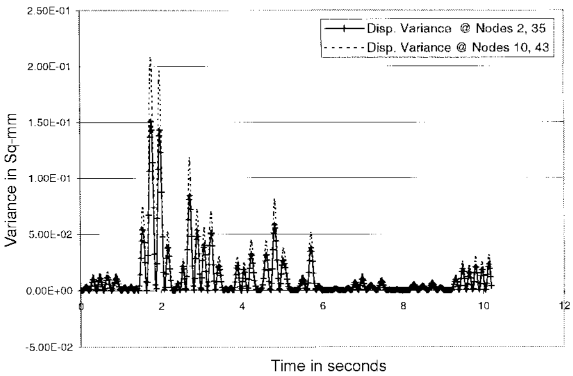

Fig. 12 Horizontal displacement variance time history for open-storey frame - fixed base (coefficient of variation of Young's modulus of concrete = 0.05, coefficient of variation of Young's modulus of masonry = 0.15, correlation length factor = 0.50)

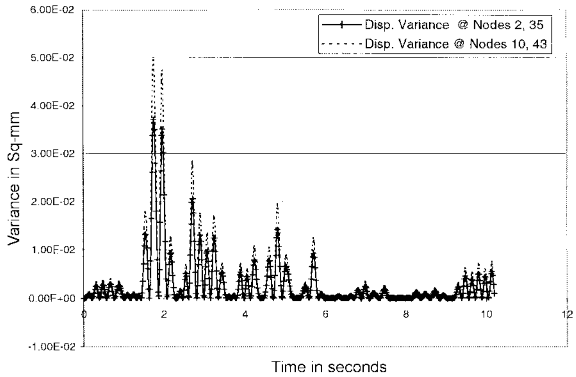

Fig. 13 Horizontal displacement variance time history for open-storey frame - fixed base (coefficient of variation of Young's modulus of concrete and masonry = 0.05, correlation length factor = 0.50, correlation within a segment *b*/*w* any two joints)

structures, two values namely, 0.05 and 0.15 are considered. The coefficient of variation of Young's modulus of concrete is assumed as 0.05. It may be noted that the actual value of coefficient of variation of Young's modulus can be higher than 0.05. However, in order to facilitate the comparison of results with those obtained by accounting for randomness in the cross-sectional area the above value is considered.

It was observed that there was no change in the time history of the mean values for horizontal displacement at 1st floor level and 5th floor level and remain the same as shown in Figs. 4 and 5. The time histories of variances for horizontal displacement at the 1st floor level (Nodes 2 and 35) and 5th floor level (Nodes 10 and 43) are shown in Fig. 10 for the bare frame and in Fig. 11 for the open-storey frame. These time histories were obtained by taking the coefficient of variation of Young's modulii of concrete and masonry as 0.05. The response of the open-storey frame with coefficient of variation of Young's modulus of masonry taken as 0.15 is shown in Fig. 12.

4.3.1 Variations in the Young's modulus of masonry

From Figs. 11 and 12 it can be observed that the values of displacement variances are fairly insensitive to variations in the coefficient of variation of the Young's modulus in the infilled walls. This can be explained as follows. As mentioned earlier, the entire portion of the structure above is behaving like a single well-correlated lumped mass, resulting in the SDF mode of vibration of the structure. Hence, since it is the mass of upper floors that influences the response, the variations in Young's modulus of walls may not significantly influence the behaviour of the structure.

From Figs. 11 and 12 it can also be observed that the values of displacement variances for the open-storey frame are smaller for larger coefficient of variation of Young's modulus of masonry. This could be because, with large variations in the Young's modulus of masonry the influence of the stiffness contributed by the infilled wall in the open-storey frame gets reduced and the behaviour of the structure tends towards the behaviour of the bare frame (Fig. 10).

4.3.2 Variations in the Young's modulus of concrete

In the case of bare frame, by comparing the values of displacement variances (Figs. 6(a) and 10) it is noted that the displacement variance values are higher for the case with randomness in element cross-sectional area than for the cases with randomness in Young's modulus. This is consistent with the observations reported by the authors in an earlier publication (Manjuprasad *et al.* 2001). However, in the case of open-storey frame (Figs. 6(b) and 11), it can be observed that the values of displacement variances for the case with randomness in Young's modulus are higher than the corresponding values for the cases with randomness in element cross-sectional area. This may be due to the change of mode of vibration of the structure from MDF mode to SDF mode. Hence, variations in Young's modulus of concrete have to be properly controlled by ensuring good quality control during construction.

4.4 Effect of correlation length

In general, the correlation length of random parameters in reinforced concrete structures represents the quality of workmanship. In stochastic finite element analysis, the correlation length is used to define the covariance structure of the element random variables. In order to study the effect of correlation length on the response of the open-storey frame this study is carried out. The Young's modulus is assumed as a random field since it's variation is found to have greater influence on the behaviour of the frame. Following two cases are considered for this study.

Case (i) - Narrow-band random field

In this case it is assumed that the uncertainties are correlated over a particular type of structural member. The random field is assumed to be narrow-band type and discretised such that:

(a) The 10 element random variables in each vertical column are correlated,
(b) The 6 element random variables in each continuous beam are correlated, and
(c) Each of the strut/tie is modelled by an element random variable.

Case (ii) - Wide-band random field

In this case it is assumed that the uncertainties are correlated over a segment of the structural member between any two joints. The random field is assumed to be wide-band type and discretised such that:

(a) The 2 element random variables in each segment of the column between any two floors are correlated,
(b) The 2 element random variables in each segment of the continuous beam between any two columns are correlated, and
(c) Each of the strut/tie is modelled by an element random variable.

The characteristics of raw materials and workmanship can vary during different sequences and phases. The two cases considered above represent typical situations that can arise in practice depending on the sequence and phase of construction.

The time behaviour of variances obtained for the horizontal displacement at the 1^{st} floor level (Nodes 2 and 35) and 5^{th} floor level (Nodes 10 and 43) are shown in Fig. 11 for case (i) and Fig. 13 for case (ii). From the analysis it is observed (figure not shown) that there is no change in the time history of the mean values for horizontal displacement at 1^{st} floor level and 5^{th} floor level and remain the same as before (Fig. 5). However, from Figs. 11 and 13 it can be seen that the values of displacement variances in the case (ii) are approximately 4.5 times lesser than that of case (i). This shows that the use of decreased value of the correlation length (i.e., wide-band random field) will result in the estimation of lesser probability of failure of the structure when the analysis is carried out to evaluate the safety of the structure. The results highlight the importance of considering correct value of correlation length in the modelling of random fields using stochastic finite elements.

5. Conclusions

During the January 26, 2001 Bhuj (Gujarat, India) earthquake, a large number of reinforced concrete framed buildings with open-storey at ground floor level, suffered extensive damage and in some cases catastrophic collapse. Experience gained from post-disaster damage survey of buildings conducted after the Bhuj earthquake motivated the authors to initiate studies on the seismic vulnerability of reinforced concrete framed buildings with open-storey. This paper presents the results of stochastic finite element analysis of a five-storey reinforced concrete framed structure with open-storey and subjected to standard (El Centro) seismic excitation, carried out at Structural Engineering Research Centre, Chennai. From the studies carried out by considering stochastic system parameters, the following conclusions are drawn:

- Presence of open-storey in a frame changes the governing mode of vibration from MDF mode to SDF mode.
- The structures with open-storey at ground floor level shows severe distress at the junction of beam column joints at the 1^{st} floor level while there will not be much damage in the upper floors. This is in contrast to the structures without open-storey, which will show a uniform damage of lesser degree throughout the height of the structure. This observation holds good for both the support conditions (fixed and hinged) considered. This could be due to the SDF of mode of vibration in the open-storey frame leading to large deformations and consequent release of energy at junctions in the 1^{st} floor level.

- Presence of a hinged type of support (representing weak support condition due to liquefaction of soil) results in magnification of amplitude of vibration by nearly two times and elongation of the period of vibration.
- For the open-storey frame the lateral displacements are perfectly correlated within any given floor level as well as across the floors. The complete structure above the 1st floor level behaves like a single lumped mass. This is in contrast to the frame without open-storey, which behaves more like a distributed mass across the floors.
- The stochastic dynamic displacement response of the frame with open-storey is more sensitive to variations in Young's modulus. This is in contrast to the frame without open-storey, which is more sensitive to variations in cross-sectional area.
- The variations in the stochastic dynamic displacement response of the frame with open-storey are fairly insensitive to variations in the coefficient of variation of the Young's modulus of masonry in the infilled walls.
- Higher the correlation length of the random field, higher would be the variations in the stochastic response of the structure.

The results of seismic analysis of reinforced concrete framed structures with open storey and appropriate boundary conditions reported in this paper demonstrate the importance of using stochastic finite element technique in characterising the stochastic response of structures with system parameters treated as random fields. Further investigations are being carried out at SERC, Chennai, on the effect of considering the stochasticity in input base excitation. This would provide useful information to carryout risk analysis and to develop guidelines for reliability based design of structures.

Acknowledgements

This paper is being published with the kind permission of the Director, Structural Engineering Research Centre, Chennai, India. The authors would like to thank the Director for the excellent support and encouragement provided to carryout the research work reported in the paper.

References

Achyutha, H., Injaganeri, S.S., Satyanarayanan, S. and Krishnamoorthy, C.S. (1994), "Inelastic behaviour of brick infilled reinforced concrete frames", *J. Struct. Eng.*, SERC, Chennai, **21**(2), 107-115.

Choi, C.K. and Noh, H.C. (2000), "Weighted integral SFEM including higher order terms", *J. Eng. Mech.*, ASCE, **126**(8), 859-866.

Manjuprasad, M., Gopalakrishnan, S. and Balaji Rao, K. (2001), "Stochastic finite element analysis of RC framed structures subjected to seismic excitations", *Proc. of National Symposium on Advances in Structural Dynamics and Design*, SERC, Chennai during January 9-11, 391-399.

Mark Fintel (1974), *Handbook of Concrete Engineering*, Van Nostrand Reinhold Company, New York.

Kleiber, M. and Hien, T.D. (1992), *The Stochastic Finite Element Method: Basic Perturbation Technique and Computer Implementation*, John Wiley & Sons, Chichester.

Roger G. Ghanem and Pol D. Spanos (1991), *Stochastic Finite Elements: A Spectral Approach*, Springer-Verlag, New York.

Stafford Smith, B. and Carter, C. (1969), "A method of analysis for infilled frames", *Proc. of the Institution of Civil Engineers*, **44**, 31-48.