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Updating of FE models of an instrumented G+9 RC building using measured data from strong motion and ambient vibration survey

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Abstract. A number of structural and modal parameters are derived from the strong motion records of an instrumented G + 9 storeyed RC building during Bhuj earthquake, 26 Jan. 2001 in India. Some of the extracted parameters are peak floor accelerations, storey drift and modal characteristics. Modal parameters of the building are also compared with the values obtained from ambient vibration survey of the instrumented building after the occurrence of earthquake. These parameters are further used for calibrating the accuracy of fixed-base Finite Element (FE) models considering structural and non-structural elements. Some conclusions are drawn based on theoretical and experimental results obtained from strong motion records and time history analysis of FE models. An important outcome of the study is that strong motion peak acceleration profile in two horizontal directions is close to FE model in which masonry infill walls are modeled.

Keywords: instrumentation; strong motion record; ambient vibration testing; fixed base FE models; dynamic time history analysis

1. Introduction

A large number of multi-storeyed RC buildings have undergone damage during the Bhuj earthquake, 2001. These buildings range from *G*+4 to *G*+10 and the causes of damage are again the same as identified in other previous earthquakes such as soft storey failure: vertical irregularity in stiffness/ strength, floating column failure, complex load path to transfer forces, mass irregularities, eccentric loading and P-Δ effect, poor and old construction, corrosion of reinforcement, pounding, hammering of adjacent buildings, design deficiency, lack of ductility and ductile detailing and construction consideration: lack of sliding and moveable joints (EERI 2002). In spite of these common causes, one question often arises – How far the provisions of Code-of-Practices are sufficient enough to predict the dynamic characteristics of the building. The code estimates these parameters empirically for different categories of buildings.

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Incidentally one of the multistoried RC buildings in Ahmedabad namely Regional Passport Office Staff Quarter (RPOSQ) building was instrumented just 2 days before the occurrence of Bhuj earthquake Jan. 26, 2001. Instrumentation of the building consisted of fourteen channels of recording at different floor levels including tri-axial sensor (Force Balance Accelerometer) at ground floor and at the roof. The sensors of the building were in position to record the response of the building during an earthquake. It gave an opportunity to extract the important building parameters and to know the structural response during an earthquake. This study is focused on the identification of structural and modal parameters such as drift index in two horizontal directions; modal characteristics using the Fourier transform. These parameters are further used in calibrating the accuracy of different FE models of the instrumented building by considering the number of structural and non-structural elements.

2. Brief description of the structure and strong motion instrumentation

RPOSQ building was constructed in the year 2000 following the IS: 456-2000 Indian standard. The best part of the study is that all blue prints of as-built drawings were available. Therefore, geometry and material property used in the building are confirmed. This has certainly enhanced the reliability of the study.

Plan dimensions of RPOSQ building were 18.09 m x 20.36 m with a total height 30 m above ground along with each storey height 3.0 m, Fig. 1. The total floor area was about 250 m^2 upto fifth floor level, about 208 m^2 between sixth and ninth floor level and it reduced to 167 m^2 at roof/tenth floor level. The structural system was a RC moment resisting frame. The size of columns at periphery was 1000×300 mm while inner side sizes are 835×300, 600×300, 780×230 and 755×230 mm. Overall, fourteen types of rectangular beams were used and the variation of width of the beam was in between 230 mm and 660 mm while the variation in depth existed at 300 mm and 700 mm. Floor slabs were 100 mm and 110 mm thick. A lift well of size 3.46×2.265 m was at the centre of the building. Clear lift well height from machine room floor level to pit floor was 34.10 m. In front of the lift well, staircase was provided for the residents along with the two lifts. The foundation system consists of a RC raft slab of size 25×23×1.58 m founded at 3.25 meters from the ground floor/ground level (Fig. 1b). Fig. 2a shows the typical plan of the building, orientation of columns and thickness of floor slabs (in circles).

The building was instrumented with 14 channels of Force Balance Accelerometers (FBA), Fig. 2b. One orthogonal tri-axial FBA was placed at the ground floor and one at the top floor. Two uniaxial sensors in two horizontal directions were located near the beam – column joint of each floor at 3rd, 5th, 7th and 9th floors whereas tri-axial sensors (ground and top floor) were installed on top of the floor slab.

Based on cross borehole tests, shear wave velocity profile is shown in Fig. 3. From the bore logs tests it come to be known that the site under consideration consists of alluvial deposit. Average shear wave velocity above bedrock (Takewaki 2005) was 344 m/s i.e., Type II soil (medium soil) by Indian Standard IS 1893: 2002 (Part 1).

3. Structural and modal parameters from strong motion records of Bhuj earthquake

Fig. 1(a) A view of the RPOSQ building

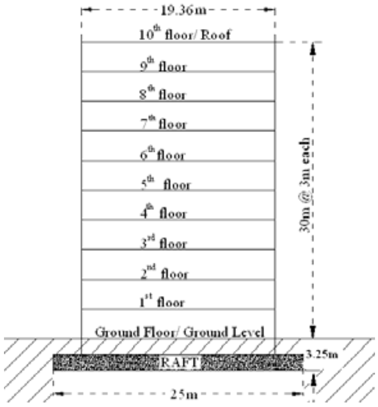


Fig. 1(b) Elevation in N-S direction of RPOSQ building

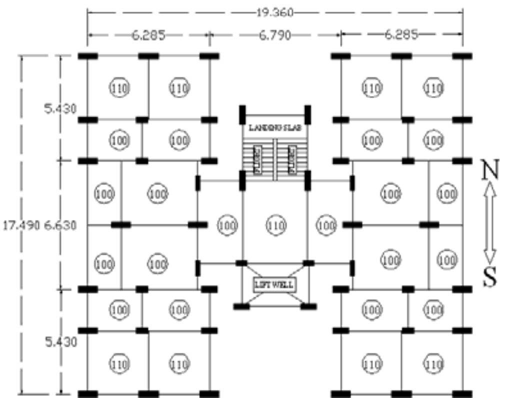


Fig. 2(a) Typical plan of upto 5^{th} floor level, orientation of columns and thickness of floor slabs

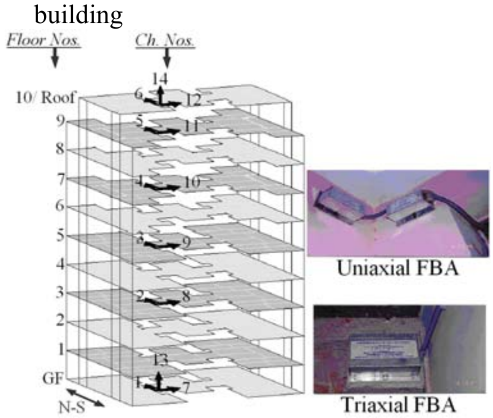


Fig. 2(b) locations of sensors at various floors and channel numbers

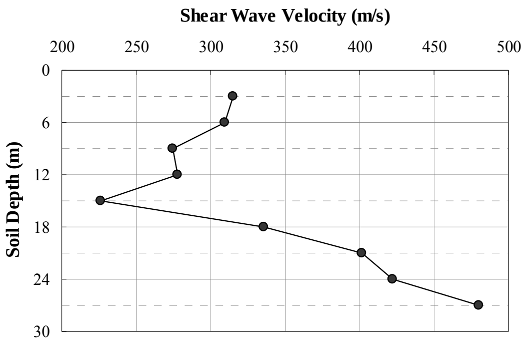


Fig. 3 Shear wave velocity at vertical points from cross borehole tests

Acceleration time history records of 133.53 sec duration at 200 samples per second (SPS) are obtained from all the 14 channels installed in the building (Fig. 4) during Bhuj earthquake, 2001.

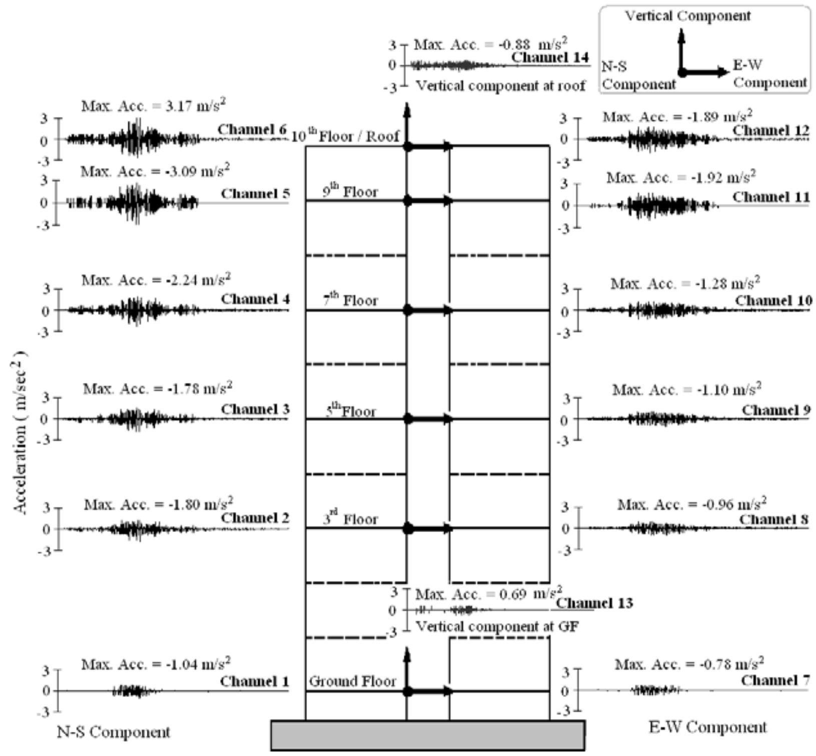


Fig. 4 Corrected acceleration time histories and the peak value of accelerations at various floors of RPOSQ building

Table 1 Peak accelerations and calculated parameters from records of Bhuj earthquake, January 26, 2001

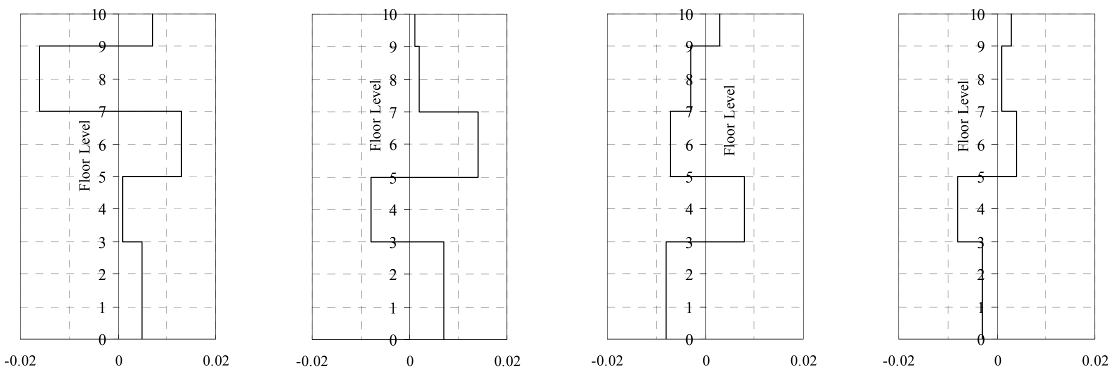
Ch. No.	Floor No.	Component	Peak Floor acceleration, A (m/s^2)	Peak relative velocity (m/s)	Peak relative displacement (m)	Amplification factor (A_{floor}/A_G)
6	10/Roof	N-S	3.17	0.3682	-0.11961	3.04
5	9		-3.09	-0.3671	0.11555	2.97
4	7		-2.24	-0.2740	0.12201	2.15
3	5		-1.78	-0.1789	-0.07395	1.71
2	3		-1.80	0.1163	-0.07572	1.73
1	GF/GL		-1.04	-	-	1.00
12	10/Roof	E-W	-1.89	0.2217	0.10216	2.42
11	9		-1.92	0.1955	0.05042	2.46
10	7		-1.28	0.1800	0.09959	1.64
9	5		-1.10	0.1461	0.08572	1.41
8	3		-0.96	0.1341	0.09829	1.23
7	GF/GL		-0.78	-	-	1.00
14	10/Roof	Vertical	-0.88	0.0637	0.05372	1.29
13	GF/GL		0.69	-	-	1.00

The peak values of recorded accelerations are obtained from processed accelerograms (Fig. 4). It is observed that in vertical direction, acceleration is amplified by a factor of 1.29. Further the peak value of vertical ground acceleration is about 2/3 times of the peak value in horizontal direction, while at the top floor this ratio is about 1/4. The peak value of relative velocity and displacement is calculated by using relative acceleration time history at given floors; velocity and displacement time histories are computed by integrating once and twice respectively. Base motion acceleration time history is subtracted from the instrumented floor acceleration time histories to get the relative time histories and these have been integrated twice to get the relative displacements at the floors (Kojic *et al.* 1984). Table 1 shows the relative peak values of velocity and displacement with their time of occurrence. The storey drift is estimated from the acceleration time history records of building at the 3rd floor, 5th floor, 7th floor, 9th floor and at the roof. Linear interpolation is applied to find out relative displacements of non-instrumented floors.

It is observed that the peak or maximum value of relative displacement of all floors has not occurred at the same instant of time. Therefore, the relative values of drift, drift index and inter-storey drift are plotted at those instant of time where one of the peaks of instrumented floor has occurred. Fig. 5 shows the drift index of floors in N-S direction at the four time instants. The maximum inter-storey drift index is found to be 0.014 between 7th and 6th floor and 6th and 5th floor. The maximum overall drift i.e., the maximum relative displacement of top floor with respect to ground is calculated as 0.11961 m and 0.10216 m and overall drift index of the building is about 0.003987 and 0.003405 in N-S and E-W directions respectively.

3.1 Modal parameters

Frequency Domain Decomposition (FDD) technique is used to perform the modal identification of the structures (Brincker *et al.* 2000). The peaks in the FDD of response measurements are required for different data sets, taken from various locations on the building. They are used to estimate natural frequencies. ARTeMIS software (Structural Vibration Solution 2004) is used for identification of modal parameters of instrumented building from strong motion records using building geometry. Geometry of the building can be very helpful in deciding the real



At 35.575s time instant At 45.485s time instant At 51.020s time instant At 71.295s time instant

Fig. 5 Drift index$(\Delta_{i+1} - \Delta_i)/h$ of floors in N-S direction at different instant of time

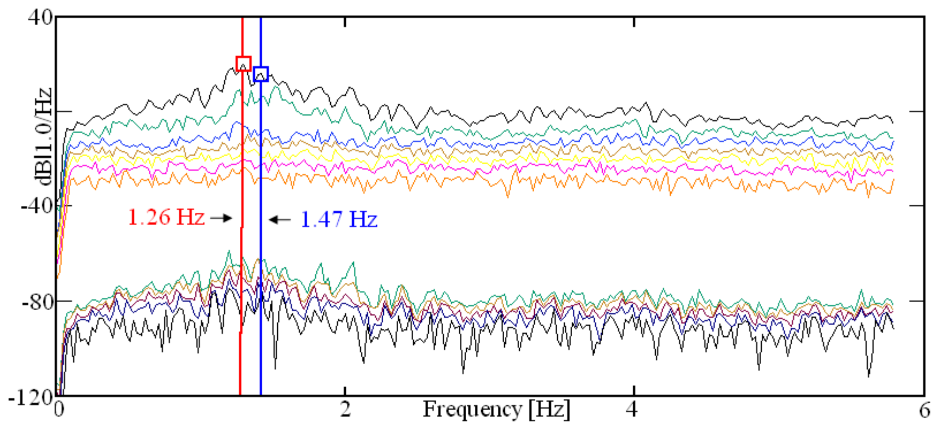


Fig. 6 Peaks in the FDD

modes of building, which leading to a better understanding of the dynamic behavior of the building. To minimize the effect of noise in records, 200 SPS data is decimated by a factor of two to have ultimate data of 100 SPS or having nyquist frequency of 50 Hz. Table 2 gives modal parameters of the instrumented building based on strong motion records of Bhuj earthquake and Fig. 6 give frequency response curves. Details of the modal extraction are described in reference (Singh 2008).

4. Modal parameters from ambient vibration testing (AVT)

Modal parameters of RPOSQ building are studied under ambient environmental forces. The sources of ambient vibrations are traffic around the building, wind, human activity in the building. The objective of AVT of the instrumented multi-storied RC building (*G* +9) is twofold. First, is to give additional information of dynamic properties of the building and secondly, because of very low level of vibration it is expected that participation of soil in the modal parameters of the building will be minimum. This additional information plays an important role for the development of finite element model and throws light on the difference of dynamic behaviour in the low level of ambient vibration and strong motion. Modal parameters obtained are given in Table 2 and details of the study are given in reference (Singh 2008).

Table 2 Modal parameters estimated from strong motion data and AVT

Mode	Strong motion		AVT	
	Frequency (Hz)	Damping ratio (%)	Frequency (Hz)	Damping ratio (%)
1	1.26^{NS1}	5.0	1.725^{NS1}	1.1
2	1.47^{EW1}	2.9	1.907^{EW1}	1.2
3	2.34^{T1}	2.7	2.198^{T1}	0.9
4	3.91^{NS2}	2.4	5.068^{NS2}	1.5
5	4.98^{T2}	1.4	6.207^{T2}	1.3

Note: EW: East –West; NS: North-South; NS1: first N-S mode; EW1: first E-W mode; T1: first torsion mode; NS2: second N-S mode; T2: second torsion mode.

Table 3 Details of FE models

FE Models	Elements considered	FE modeling
M1	Columns and beams – bare frame	Beam and columns as 3-D beam-column element, floor slabs and waist slab of staircase as 4-node plate elements and masonry infill walls as strut element (Garevski *et al.* 2004) using material properties given in Table 4
M2	Columns and beams + floor slabs	
M3	Columns and beams + stair case	
M4	Columns and beams + floor slabs + stair case	
M5	Columns and beams + floor slabs + stair case + infill walls	

5. FE models updating study

To correlate experimental and analytical results of the building, different FE models are used for analytical results. The whole analysis is carried out under fixed base condition to predict the behaviour of the building because these types of FE models are frequently used in design offices. Analyzed structure has foundation on alluvial deposits shown in Fig. 3, which is ignored in the fixed-base assumption. Therefore, one more FE model is developed to cater this effect to find out modal parameters only.

5.1 3D fixed-base FE models

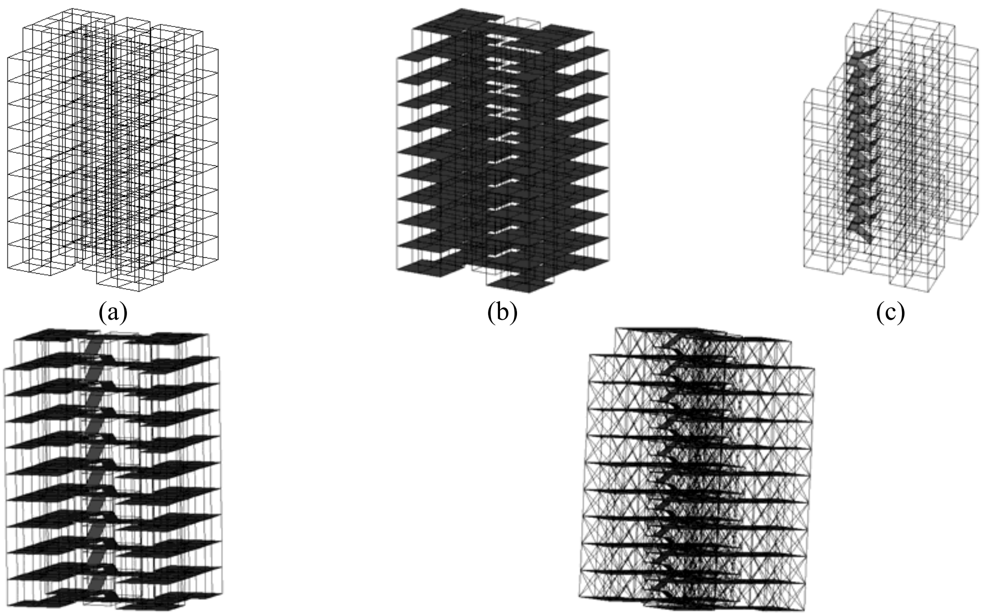
Seismic behavior of the RPOSQ building is studied using five FE models shown in Fig. 7 and details are given in Table 3. These FE models are developed using general purpose finite element program Ansys (Ansys 2006). As described earlier, geometry and material properties are known from the blueprints of the building. Therefore, FE models are updated manually by adding structural and non-structural elements to the bare frame model (M1) and keeping material properties same as given in as-built drawings. Though, manual updating has limitations (Ventura *et al.* 2000) but in broader sense, it gives the effect of each type of element on the seismic behaviour of building. Gravity loads considered in the analysis are: dead loads for beams, columns, slabs (with finishes), masonry infill walls (exterior and interior), machine room of the lift, water tank, balcony and a percentage of live load. Moment due to cantilever in balcony is also considered with the gravity load at the appropriate nodes. The gravity loads remain the same in all FE models and the change in stiffness are accounted in FE models.

5.1.1 FE Modeling of floor slabs

In the present study all slabs are considered as plate element with the defined thickness. A four noded plate element *Shell63* of Ansys of thickness 0.10 m has been used to model the floor slabs of the building. Total numbers of plate elements are 386.

5.1.2 FE Modeling of masonry infill walls

Various studies have been reported on the scaled models to find out the response of the building with infill walls (Garevski *et al.* 2004) by studying the lateral forces due to infill walls own inertia in an earthquake. This type of lateral deformation demands elongation in one diagonal length and compression in another diagonal length. If the frames are filled with infill walls which happen generally in the buildings than the infill walls try to act against these actions. Due to resistance offered by the infill walls in the diagonal lengths the brick infill within the panel can be



(a) (b) (c) (d) (e)

Fig. 7 FE models M1 to M5 shown above from (a) to (e) respectively

Table 4 Material properties of the elements of the building

Grade of Mix	Char. Compr. Str. (f_{ck}) (N/mm^2)	Used in the construction of	Mod. of Elas. (E) N/mm^2 ($E = 5000\sqrt{f_{ck}}$)
1:2:4	15	Columns, Beams, Floor Slabs, Staircase, Raft slab	19.365×10^3
1:1.5:3	20	Columns, Water Tank	22.361×10^3

modeled as strut elements in the two diagonal lengths. In the present study the effect of infill walls on building response is studied by modeling the infill walls as truss element in the fifth FE model M5 using *Link8* element of Ansys. Thicknesses of exterior and interior infill walls are taken as 9 inches (0.2286 m) and 4.5 inches (0.1143 m) respectively. The modulus of elasticity and Poisson ratio of infill considered in analysis are 1.2x 10^{10} N/m^2 and 0.15 respectively. The equivalent area of strut element is calculated by $\frac{1}{2}\sqrt{\alpha_h^2+\alpha_l^2}\times t$, where $\alpha_h = \frac{\pi}{2}\left[\frac{E_f I_c h}{2E_m t\sin 2\theta}\right]^{1/4}$, $\alpha_l = \pi\left[\frac{E_f I_b h}{E_m t\sin 2\theta}\right]^{1/4}$ and $\theta = \tan^{-1}\frac{h}{l}$; E_f and E_m elastic modulus of frame material and masonry wall; t, h and l thickness, height and length of infill wall; I_c and I_b moment of inertia of columns and beams.

5.1.3 Material specifications

Two types of cement concrete mix 1:2:4 and 1:1.5:3 used in the construction are given in Table 4. According to IS 456: 2000, modulus of elasticity (E) of the concrete is given by

$E = 5000\sqrt{f_{ck}}$ N/mm^2. Main reinforcement used in columns are high yield strength steel bars having 415 N/mm2 yield strength and confining steel properties is as per IS 13920-1993.

5.2 3D FE Model 'M6' for SSI effect on modal parameters only

Previous FE models (M1 to M5) were used to calculate modal parameters as well as time history analysis assuming base-fixed assumption. Therefore in order to include soil flexibility an additional FE model M6 was generated to calculate modal parameters especially modal frequency. The details of this comprehensive FE model M6 (Fig. 8) are as follows.

5.2.1 Superstructure

Five FE models (M1 to M5) were considered for fixed base analysis of the building. At a later stage, the modal parameters of the fifth FE model (M5) were found to be close to those obtained from ambient vibration testing. Hence, the fifth FE model M5, in which a combined effect of all structural elements was considered as superstructure for SSI analysis.

5.2.2 Substructure

In the substructure, layered soil and raft foundation were included. Columns were connected to the raft foundation to transfer the applied load to the soil strata. Layered soil media was modeled by the FE method and the boundary conditions were implemented around the soil block. The raft foundation and soil medium were modeled by eight-noded solid elements (SOLID45) of Ansys. These elements are very successful in predicting the behavior of DSSI (Ottaviani 1975).

5.2.3 Size of the soil block

In the horizontal direction, the width of the soil block was considered as three times of the size of raft foundation in that direction. The size of the raft foundation was 25×23 m. Hence, the size of the soil block was taken as 75×69 m. After considering the shear wave velocity profile (Fig. 3), engineering bedrock level was assumed at a depth of 30m. Therefore, the height of the soil block in vertical direction was taken as 30 m below the foundation level.

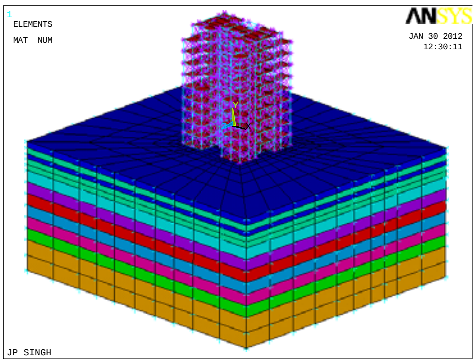


Fig. 8 FE model of Building-Raft-Soil System (M6)

5.2.4 Size of FE elements of the soil block and boundary conditions

The size of the element in vertical direction i.e., height (h) is very important in case of the shear wave transmitted vertically and according to a study by Gupta *et al*. (1982), maximum height of the FE element (h_{max}) can be taken as $h_{max} = \left(\frac{1}{5} \sim \frac{1}{8}\right) V_s \Big/ f_{max}$ where, V_s and f_{max} are velocity of shear wave and highest wave frequency respectively. Keeping in mind the building frequency range and magnitude of the problem, highest wave frequency intercepted was taken as 10 Hz. In the present study the maximum size of the soil element was taken as 3.0 m which fulfilled the above requirement. In the horizontal direction, the limitation of the size of soil element is not as strict as in the vertical direction and the maximum size of the element can be taken as three to five times the maximum size of the soil element (Lu *et al.* 2005).

In the present analysis, radiation damping was incorporated as viscous boundary condition at four vertical faces of the soil block. All nodes were fixed at the bottom of the soil block which was assumed as bedrock level.

5.3 Modal parameters

The free vibration analysis of FE model M1 to M5 is carried out and the first five frequencies are given in Table 5. Modal patterns of first five modes are also given in Table 5 as superscript to modal frequencies. From the natural frequencies of FE models it is found that the frequencies of bare frame model M1 are lowest while the frequencies of model M5 are highest which is expected due to addition of stiffness of other building elements. It is seen that the maximum increment in frequencies is found after adding stiffness of infill walls. After incorporating the soil flexibility (model M6), modal frequencies were reduced and are given in Table 5.

5.4 Seismic response

Seismic response of FE models of the building is carried out by performing mode-superposition dynamic time history analysis using the recorded strong motion at GF as input motion. According to post earthquake survey and analysis of the recorded strong motion data, this building remains in the elastic range during strong motion of Bhuj earthquake (Singh 2008). Therefore dynamic analysis of all FE models is performed considering the linear behaviour of all elements. Dynamic analysis is performed for whole duration of the strong motion record 133.525 s and the sampling interval for the analysis is taken as 0.005 s for which the strong motion data is available.

Table 5 Modal frequencies of first five modes of FE models M1 to M5 and percentage variation with respect to the bare frame model M1

Mode	M1 (Hz)	M2 (Hz)	M3 (Hz)	M4 (Hz)	M5 (Hz)	$\left(\frac{\text{M5-M1}}{\text{M1}}\right)$ (%)	M6 (Hz)
1	0.91^{NS1}	0.94^{NS1}	0.93^{NS1}	0.97^{NS1}	1.98^{NS1}	118.3	1.36^{NS1}
2	1.03^{T1}	1.10^{T1}	1.04^{T1}	1.11^{T1}	2.22^{EW1}	115.0	1.52^{EW1}
3	1.08^{EW1}	1.12^{EW1}	1.12^{EW1}	1.15^{EW1}	2.29^{T1}	110.8	1.61^{T1}
4	1.58^{mix}	2.75^{mix}	1.59^{mix}	2.85^{mix}	5.78^{NS2}	265.3	2.17
5	1.84^{mix}	3.15^{mix}	1.87^{mix}	3.21^{mix}	6.20^{T2}	236.9	2.21

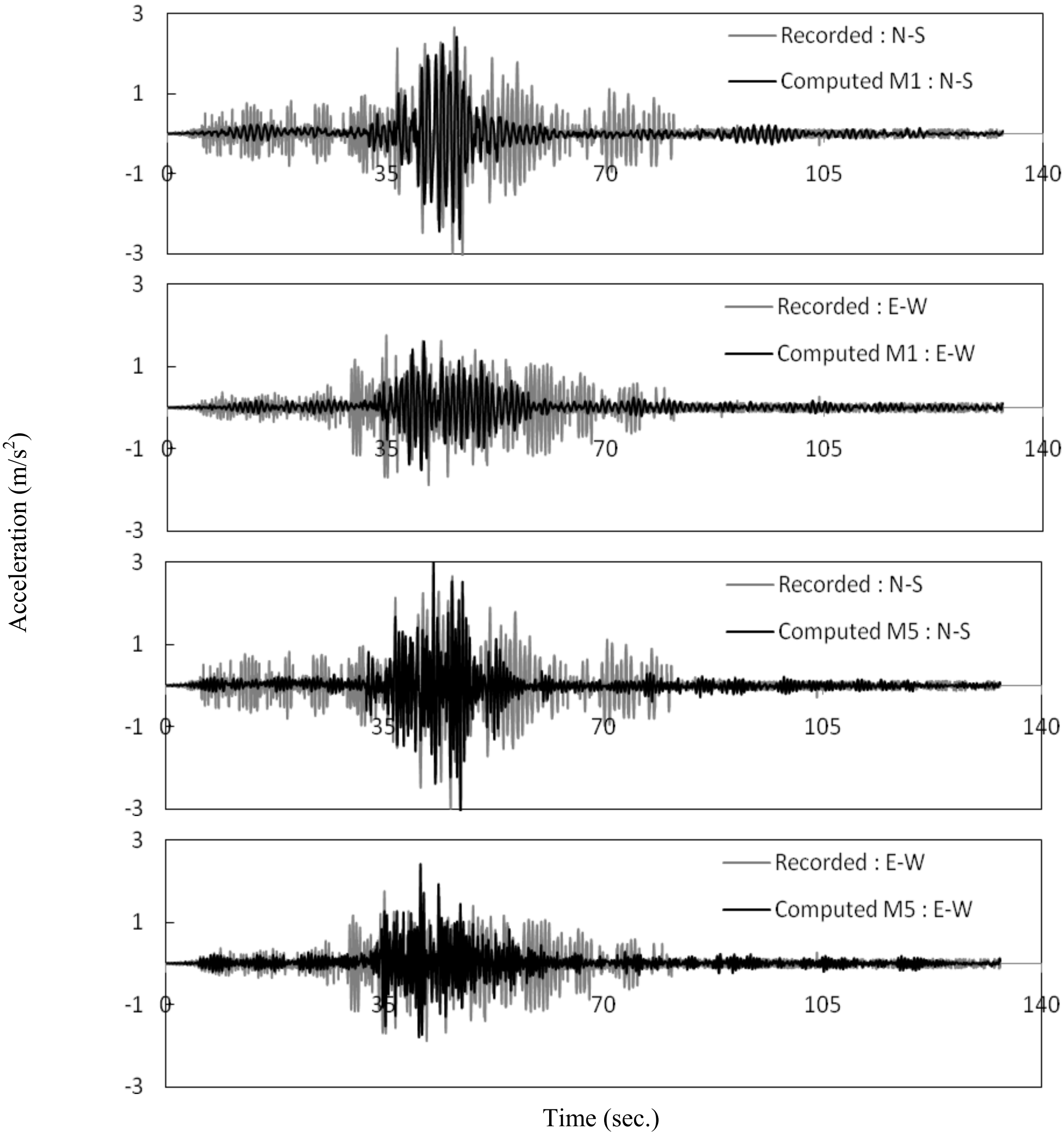


Fig. 9 Recorded and computed time history at 10th floor of FE model M1 and M5

Table 6 Characteristics of input excitation

Direction	Peak acceleration (m/s^2)	Time of occurrence of peak acceleration (s)
NS	1.038	46.940
EW	0.782	34.945
Vertical	0.686	44.060

5.4.1 Input excitation characteristic

Strong motion record at the ground floor in N-S, E-W and vertical directions of the building is used as input excitation at the base of FE models for the dynamic time history analysis. The characteristics of input excitation in three perpendicular directions are given in Table 6.

5.4.2 Material damping

The material damping is incorporated as Rayleigh damping. Rayleigh damping coefficients α and β are computed by using observed damping of the first and second modal frequencies in the RPOSQ building (Table 2) during strong motion. Material damping is assumed to be constant throughout the entire seismic event.

5.4.3 Results of dynamic analysis

The dynamic time history analysis of FE models M1 to M5 is performed using strong motion record of ground floor as input excitation at the fixed base of the FE models as described. Typical signature of recorded and computed time history for FE model M1 and M5 are given in Fig. 9 at top of building (10^{th} floor level).

6. Comparison of responses

6.1 Modal parameters

For FE model M1 to M4, only first three modes are pure modes (Table 5) namely translational and torsional mode while other modes are mixed modes (mix). While FE model M5 shows a clear modal pattern for first five modes. Model M5 shows first five modes as pure modes, three translational and two torsional. A comparison has been given in Table 5 to observe the effect of structural and non-structural components on the modal pattern of five FE models M1 to M5. In addition, modal patterns of FE model M5 are same as observed from strong motion data and AVT records. Therefore, it may reasonably be justified that the infill walls play a significant effect on the modal parameters of the building and it is desirable that these should be modeled in order to get a good correlation between the experimental and analytical results. By modeling infill walls the higher modes are identical with the observed modes and in the design of multistorey buildings higher mode effects can be considered (Humar and Rahgozar 2000).

First two modal frequencies of the building-raft-soil system (model M6) were calculated as 1.36 and 1.52 Hz and the recorded frequencies of the building were 1.26 and 1.47 Hz, which are close to calculated frequencies of model M6.

6.2 Floor accelerations

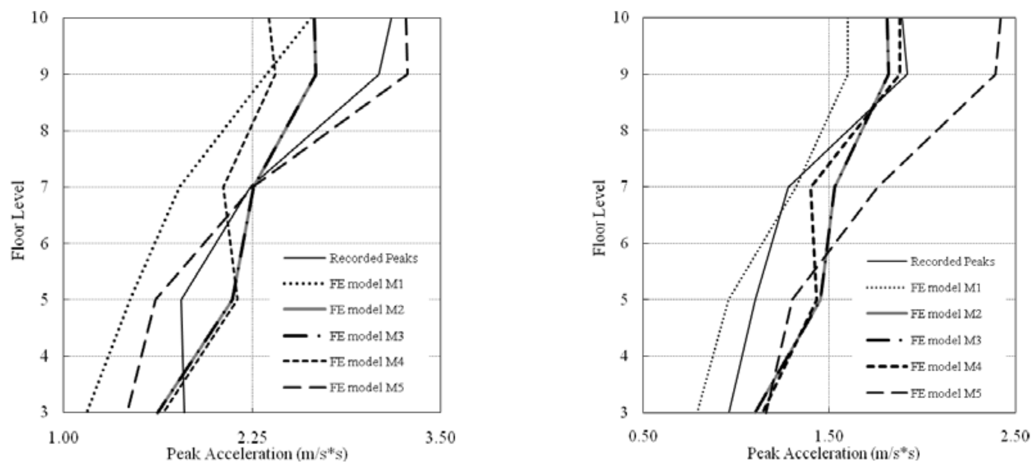
Peak floor acceleration of bare frame FE model M1 is lowest in both horizontal directions (Tables 7 and 8). FE models M2 and M3, which include floor slabs and staircase respectively, give the same peak accelerations, which reflect that the lateral stiffness due to floor slabs and stair case is the same for the present analysis. The increment of peak acceleration of floors of FE models M2 and M3 with respect to bare frame FE model M1 is higher in EW direction, than during the first mode in the NS direction. The peak acceleration increases from 1.60 m/s^2 to 1.81 m/s^2 in EW direction, and 2.64 m/s^2 to 2.66 m/s^2 in NS direction, suggests the data after adding floor slabs or staircase in the bare frame model. Also the increment of FE model M2 and M3 is higher for lower floors of the building. When floor slabs and staircase are taken into consideration with bare frame model, FE model M4 peak acceleration does not change much in both NS and EW direction. But when the stiffness due to masonry infill is added the increment of peak acceleration is noticeable. The maximum increment of peak acceleration is 51.30 percent, at the roof in EW direction. It reflects that the inclusion of infill walls in the FE model gives noticeable rise in the floor accelerations in the building.

Table 7 Peak floor accelerations in NS direction

Floor No.	Peak floor acceleration (m/s^2) computed from FE models					strong motion record (m/s^2)
	M1	M2	M3	M4	M5	
10	2.64	2.66	2.66	2.36	3.27	3.17
9	2.34	2.67	2.67	2.40	3.28	3.09
7	1.77	2.26	2.26	2.06	2.25	2.24
5	1.44	2.12	2.12	2.15	1.61	1.78
3	1.15	1.62	1.62	1.66	1.42	1.80

Table 8 Peak floor accelerations in EW direction

Floor No.	Peak floor acceleration (m/s^2) computed from FE models					strong motion record (m/s^2)
	M1	M2	M3	M4	M5	
10	1.60	1.81	1.81	1.88	2.42	1.89
9	1.60	1.82	1.82	1.88	2.39	1.92
7	1.32	1.53	1.53	1.40	1.76	1.28
5	0.96	1.45	1.45	1.43	1.30	1.10
3	0.79	1.10	1.10	1.14	1.16	0.96



(a) (b)

Fig. 10 Peak absolute accelerations, recorded and from FE models in (a) N-S direction and (b) E-W direction

Fig. 10 shows the comparison of peak accelerations of FE models and recorded response. It is observed that the peak values of recorded and analytically obtained accelerations are very much comparable at all the locations except 7th and 3rd floor' in transverse direction. The maximum difference in peak value is +37.5% at 7th floor of E-W component and -21.1% at the 3rd floor of N-S component. In vertical direction, the analytical response and measured response are very much similar.

7. Conclusions

The structural dynamic parameters of an instrumented G+9 storeyed RC building are identified from the strong motion records of Bhuj earthquake, 2001. An ambient vibration testing of the same building are carried out after the earthquake and a number of modal parameters are also identified from the records of ambient vibration. These parameters are further used to update FE models of the building by considering the number of structural and non-structural elements. The main observations from this study are;

1. The modal frequencies computed from FE model including the effect of infill walls with fixed base are reasonably close to ambient vibration testing. The difference in modal frequencies of first mode of FE model with respect to ambient vibration testing is about 16 percent while with respect to strong motion record it is about 57 percent. The 57 percent difference in first mode indicates the influence of dynamic soil structure interaction during earthquake which was verified by the building-raft-soil system FE model. Therefore, modal frequencies based on ambient vibration testing can be used for calibration of fixed base FE models and these fixed-base FE models can further be upgraded to investigate SSI effects.
2. Infill walls exercise a significant effect on the modal parameters of the building and it is desirable that these should be modeled in order to get a good correlation between the experimental and analytical results.
3. It was found that inclusion of stiffness of infill walls gives fairly close agreement with recorded peak accelerations at the instrumented floors in comparison to the computed peak accelerations without infill walls.
4. It was found that the computed peak accelerations from FE model, which includes infill stiffness, are higher than the recorded peak accelerations. The observed difference in the recorded and measured response may be due to approximation in modeling of infill with openings as well as soil structure interaction effect that has been ignored in the present study.

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