13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1122

SEISMIC RESPONSE OF AN OFFICE BUILDING: COMPARISON STUDY BY FEA MODELING AND RECORDED DATA ANALYSIS FROM DIFFERENT EARTHQUAKE EVENTS

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SUMMARY

Presented is a detailed case study on structural identification and Finite Element (FE) modeling of a 14-story office building. Specifically, system identification tools were used to identify the structural dynamic properties based on recorded seismic data. This identified model was compared with a model based on IBC 2000 using the fundamental period, design response spectral acceleration and base shear. The two models were found to be about 20% different in these parameters. Then, a series of three-dimensional FE models were created to study various approaches to improve the accuracy of FE models quantitatively. The sensitivity of natural periods to certain factors was also addressed. Simulated seismic responses using recorded free-field acceleration as input were compared with the recorded responses and it was found that a FE model can be calibrated to give a good prediction of earthquake response.

INTRODUCTION

During the past twenty years, extensive strong motion instrumentation programs have been developed in the United States to monitor seismic events and structural performances. Many earthquake events have been recorded and analyzed to improve our understanding of the seismic behavior of civil structures (Shakal [1], Uang [2] and Celebi [3]). Moreover, these recorded data are being utilized to assess and improve various modeling techniques, e.g. Finite Element modeling (Boroschek [4], Snaebjornsson [5]) so that they can be applied in new structure design or existing structure retrofitting.

Alaska is one of the most seismically active areas in the world. For example, the largest earthquake in U.S. history, the 1964 Good Friday Earthquake, occurred in South-Central Alaska ($M_W = 9.2$) and lasted over 5 minutes. This earthquake caused a large amount of damage in the Anchorage basin. As part of the strong-

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motion network, an extensive strong-motion network has also been developed in the Anchorage metropolitan area. An office building in the midtown area of Anchorage was instrumented in 1989. Since then, many seismic events have been recorded.

This paper presents a detailed case study on structural identification and FE modeling of this office building using recorded seismic data. Specifically, system identification tools were used to identify the structural dynamic properties. Then the identified dynamic properties were compared with those based on the IBC 2000 code (IBC [6]). Finally, detailed FE analyses were conducted to investigate several approaches to improve the accuracy of FE models in order to capture the fundamental dynamic properties of the structural system. These approaches include refining the mass calculation, considering these factors like eccentricity of the mass, the stiffness contribution of composite action between the floor deck and beams, the panel zone rigidity of beam/column connections and the stiffness contribution due to non-structural elements. The sensitivity of natural periods to composite action and panel zone rigidity was also investigated.

BUILDING AND INSTRUMENTATION DESCRIPTION

The office building is a steel moment-resisting frame structure with reinforced concrete spread footings constructed in the early 1980's. The building is 54.7 m tall, 32.3 m wide and 52.1 m long. The current instrumentation was completed in 1989 under the United States Geological Survey (USGS) National Strong-Motion Program (NSMP). The original analog recorder system was upgraded to a digital, on-site/real-time system with dial-up capability in August 1999. The 12-channel system (CH 1 – CH 12) has accelerometers located in the basement, on the 8th and 14th (roof) floors of the building and at its free-field station, as shown in Figure 1. The free-field site is located in a 60 cm deep downhole, 65.5 m east of the northeast corner of the building.

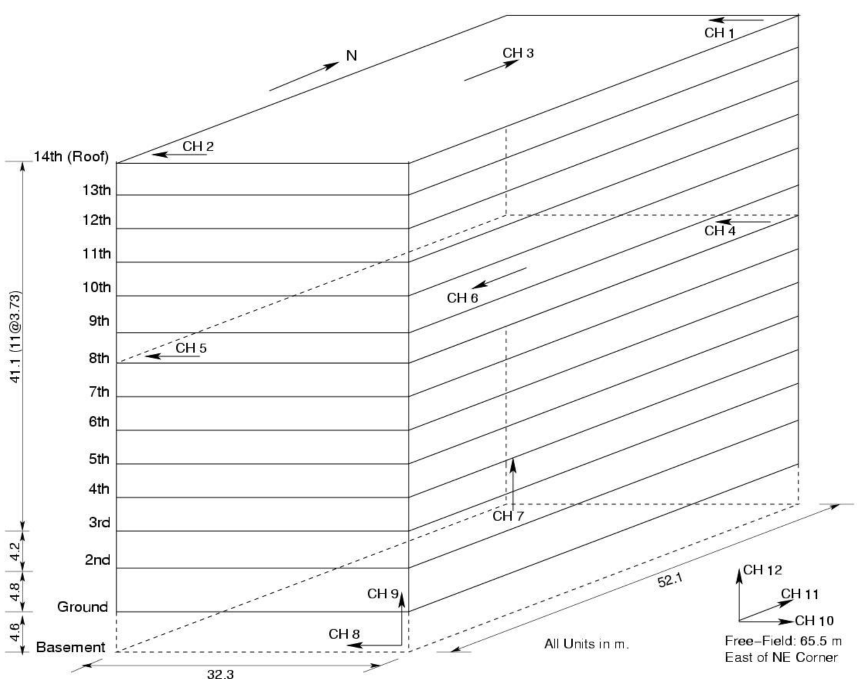

Fig. 1 Sketch of Building Instrumentation with the Free-Field Station

EARTHQUAKE RECORDS

Among the recorded events in this building, two earthquakes were considered in this paper: the October 23, 2002 Nenana and the November 3, 2002 Denali events. The November event ($M_W = 6.7$) occurred at 5 km depth at a hypocentral distance of 286 km. The October event ($M_W = 7.9$) occurred at 10 km depth at a hypocentral distance of 279 km. Figure 2 shows the recorded E-W acceleration and displacement time histories for these two events. On the left, the recorded acceleration time histories at the basement (CH 8) and the roof level (CH 2) of the building for the two seismic events are shown. On the right, the corresponding displacement time histories are shown. Figure 3 shows the Fourier Amplitude Spectra (FAS) of recorded acceleration time histories at CH 8 for the two events. Apparently, durations, peak accelerations and frequency contents are not similar in these two events. It can be seen that the November event has a rather broad frequency content, while most of the energy of the October event is concentrated around 2.0 sec, which approximates the later discovered fundamental period of the structure. Moreover, when the rocking behavior of the building in these two events was studied using the recorded data from CH 7 and CH 9, no significant rocking behavior was observed.

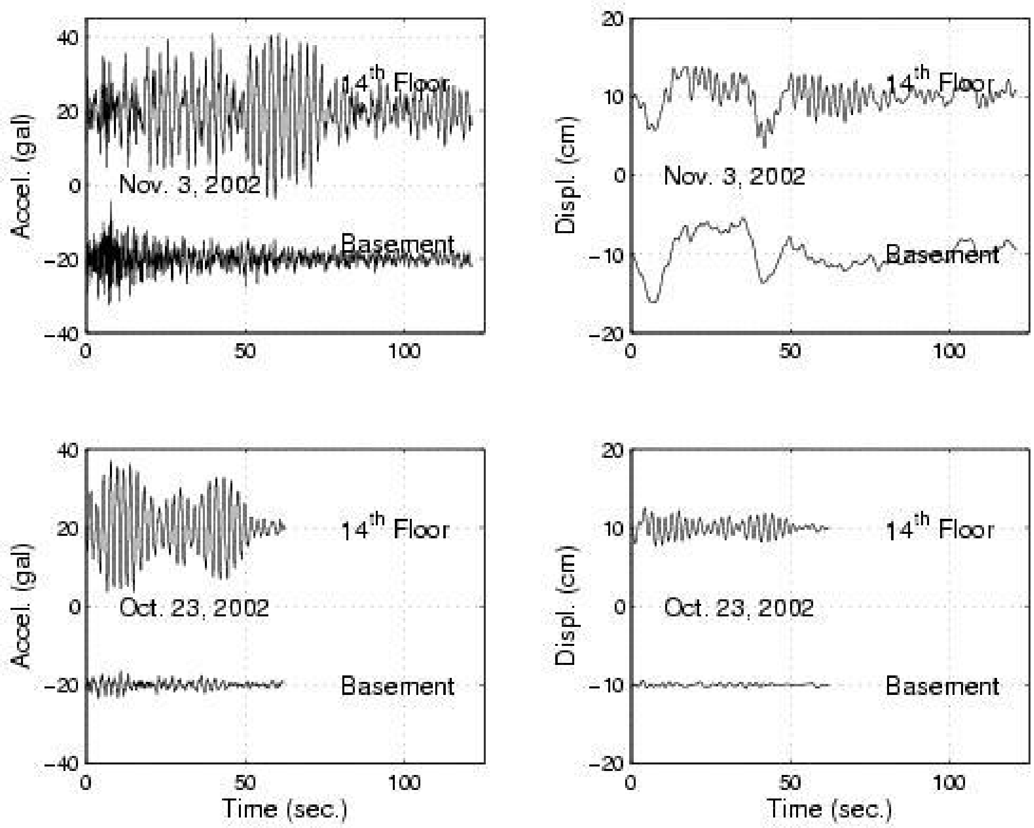

Fig. 2 Acceleration & Displacement Time Histories from Recorded Data in the E-W Direction at the Basement (CH 8) & 14th Floor (CH 2)

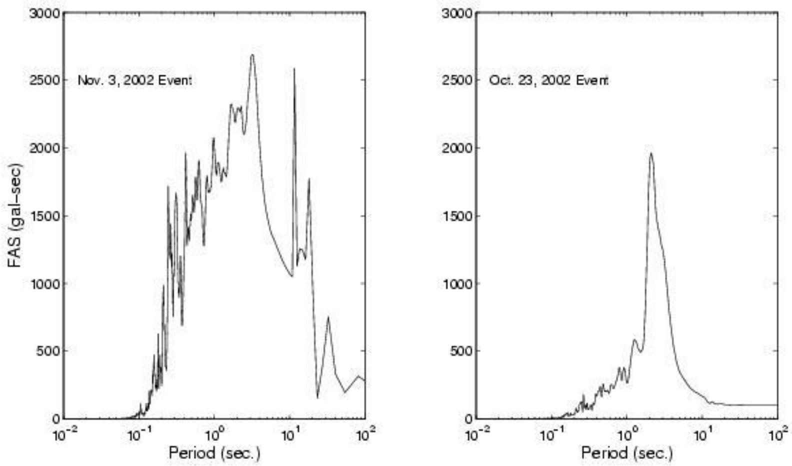

Fig. 3 FAS of Recorded Accelerations in the E-W Direction at the Basement (CH 8)

SYSTEM IDENTIFICATION AND RESULTS

Identification Method

The recorded earthquake motions have relatively low amplitudes, hence the building responses are assumed to be linear elastic. The system identification technique (Safak [7, 8]) was used to identify the in situ dynamic properties of the building. Specifically, the structural response in the discrete time domain was represented in terms of parameters of ARX (Auto-Regressive Exogenous) model. The system model can be examined in the frequency domain by applying the shifting theorem and Z-transform. To compute the frequencies and damping ratios, the poles p_k of the transfer function (i.e. the roots of the denominator polynomial) is determined. The damping ratios ξ_k and frequencies f_k (in Hz) are calculated from the poles by the following equations:

$$\xi_k = \frac{\ln\left(\frac{1}{|p_k|}\right)}{\left[v_k^{\,2} + \ln^2\left(\frac{1}{|p_k|}\right)\right]^{\frac{1}{2}}} \qquad \text{(Eq. 1)}$$

and

$$f_k = \frac{\ln\left(\frac{1}{|p_k|}\right)}{2\pi\xi_k\Delta} \qquad \text{(Eq. 2)}$$

where Δ is sampling interval; v_k = argument of p_k, i.e. $\tan(v_k) = (p_k^I/p_k^R)$, with superscript R and I denoting the real and imaginary parts, respectively.

Model Selection and Validation

The use of this particular model requires the estimation of order of the model, n_a and n_b and the time delay, d. The most widely used method (Glaser [9]) is to increase the orders or time delay until the squared estimation errors become white noise. In this paper, the procedure proposed by Safak [10] is adopted to select the model parameters. The acceleration time histories recorded at CH 8 and CH 2 were taken as the input and output, respectively.

One method of validating the model is to determine whether the estimated transfer function is close to the spectral ratio computed from recorded data. The modulus of the estimated transfer function was computed and shown in Figure 4, together with the spectral ratio of the input-output data set. The transfer function and the spectral ratio agree very well, especially around the dominant frequencies. In addition to transfer function comparison, the agreement between the recorded and estimated output can also be used to validate the model. The estimated E-W output at Node 772 (refer to Modeling Approach) and the recorded output at CH 2 are shown in Figure 5, and very good agreement can be observed.

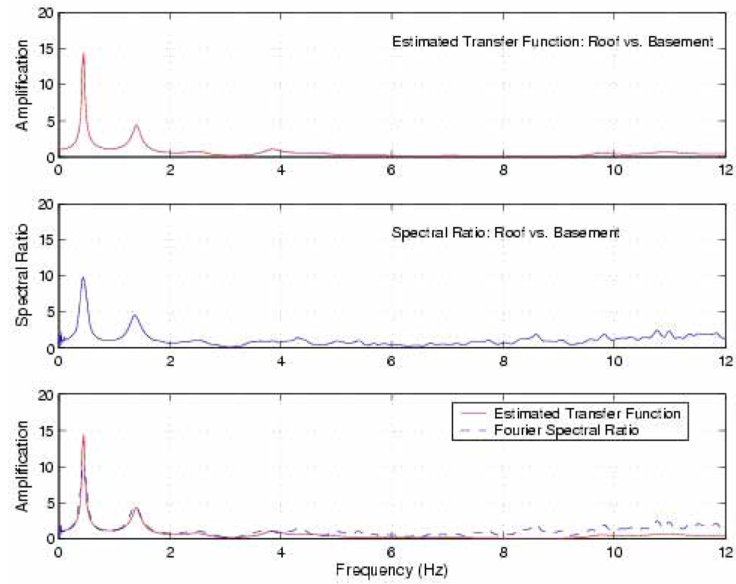

Fig. 4 Comparison of Estimated Transfer Function with Fourier Spectral Ratio for Basement-Roof Pair in the E-W Direction, November Event

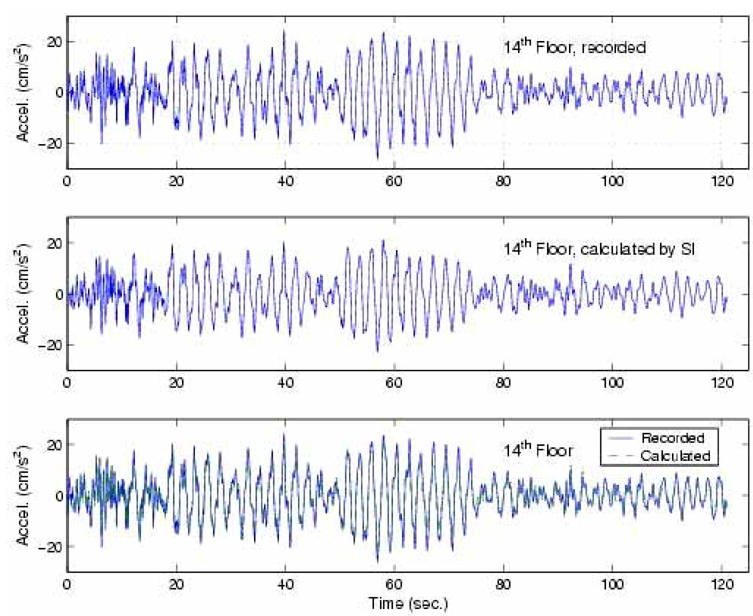

Fig. 5 Comparison of Recorded & Estimated E-W Acceleration Time Histories at 14th Floor for the November Event

Identification Results

Since the stiffness of the building in both directions are not the same, it would be interesting to identify dynamic properties of the building in the both directions. However, N-S input motion at the basement is not available from the current instrumentation system. Although it is possible to use the input in the E-W direction at the basement and output in the N-S direction at the 14th floor to identify natural modes in the N-S direction, the identification would be more effective if the input and output in the same direction was used (Safak [7, 8]). Therefore, we attempted to use the N-S free-field motion (CH 11) as input and recorded motion at CH 3 as output to identify the system. The usage of free-field motion as input can be justified by the fact that the recorded motions in UP (vertical) and E-W directions in both locations have very similar Fourier amplitude spectra, as can be seen from Figure 6.

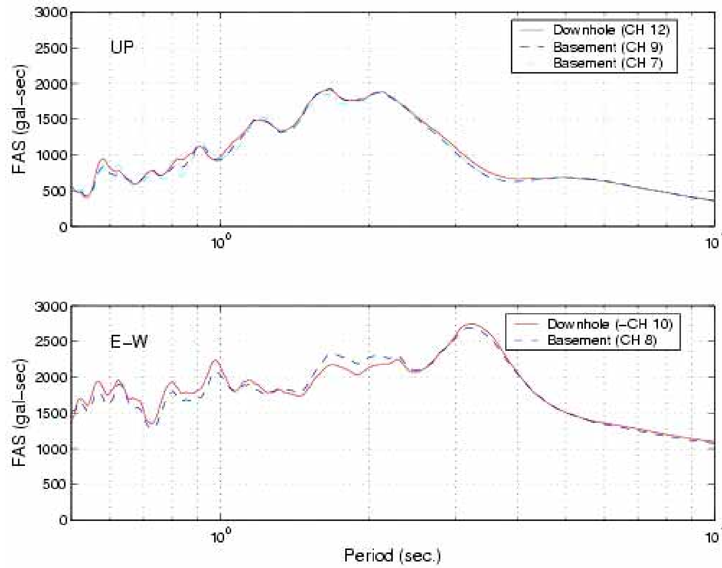

Fig. 6 Comparison of Basement & Free-Field (Downhole) Accelerations in Frequency Domain for the November Event

Identified modal frequencies and damping ratios using recorded data in the October and November events are shown in Table 1. Only the first natural periods and corresponding damping ratios in the E-W and N-S directions are shown and later used to compare with the results from the FE analyses. The table also includes the peak ground acceleration for each event to show the level of shaking. It can be seen from Table 1 that the natural periods and damping ratios in both events are slightly different from each other. This might be caused by nonlinear behavior of the building and possible site effects depending on the level of shaking.

Table 1 Summary of System Identification Results

Event	PGA (cm/sec^2)	E-W		N-S	
		Natural Period (sec)	Damping Ratio (%)	Natural Period (sec)	Damping Ratio (%)
October	3.4	2.19	3.1	2.13	3.6
November	15.6	2.23	4.0	2.17	4.1

Comparison with IBC 2000

It would be interesting to compare the identified dynamic parameters such as fundamental period with those computed based on IBC 2000, since this period would be used for seismic load computation. The periods from both events were averaged and taken as the fundamental period of the identified system (referred to as the identified model). As can be seen from Table 2, the computed fundamental period based on the approximation formula (Eq. 16-39 in IBC 2000, referred to as the IBC 2000 model) is 1.75 sec, compared with the identified period 2.21 sec. The period of the identified model is 20.8% longer than that of the IBC 2000 model. To further assess the impact of the differences in the fundamental period on the seismic design, the design response spectra suggested by IBC 2000 was computed and plotted in Figure 7 with these two fundamental periods noted. The design spectral acceleration for the identified model is 0.25g, while that for the IBC 2000 model is 0.31g.

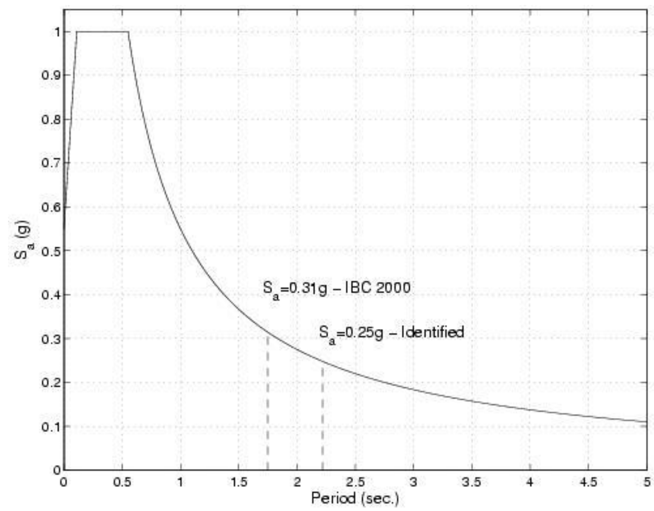

Fig. 7 Design Acceleration Response Spectrum for the Building Site According to IBC 2000

Table 2 Comparison of Design Parameters for the Identified Model and IBC 2000 Model

Model	Fundamental Period (sec)	Design Response Spectral Acceleration (g)	Base Shear (kN)
IBC 2000	1.75	0.31	9,439
Identified	2.21	0.25	7,896

FINITE ELEMENT MODELING AND ANALYSIS

An inaccurate estimation of fundamental period by the approximation formula may lead to either increased cost or inadequate design. Therefore, it would be useful to study possible approaches for improving the modeling accuracy. This section describes a series of three-dimensional linear FE models of the structure with increasing important details of design and construction. The effects of these details on the natural periods are discussed. Time history analyses were performed to assess the improvement between the basic model and the refined model.

Modeling Approach

FE models were developed using SAP2000 and an example of the models is shown in Figure 8. First, a basic model (Model 1) based on general engineering practice and design procedures was created based on the following assumptions:

(1) Each floor diaphragm and the roof was assumed to be rigid in its own plane since the 75 mm deep composite metal decking is topped with 64 mm of concrete.

(2) The composite action was ignored in the basic model since no special provisions (i.e. shear studs) to develop full composite action between the floor deck and beams were indicated on the plans.

(3) Center-to-center dimensions of the steel frame elements were used in the basic model to simplify the model geometry.

(4) The mass was simply calculated without considering the openings for the six elevators, the two stairwells, the large exterior windows at each floor, and other non-symmetric masses.

(5) The structure is almost double symmetrical, therefore, according to IBC 2000, two-direction 5% eccentricity was incorporated in the basic model to account for accidental torsional effects.

(6) The contribution of non-structural elements to the overall building stiffness was not considered.

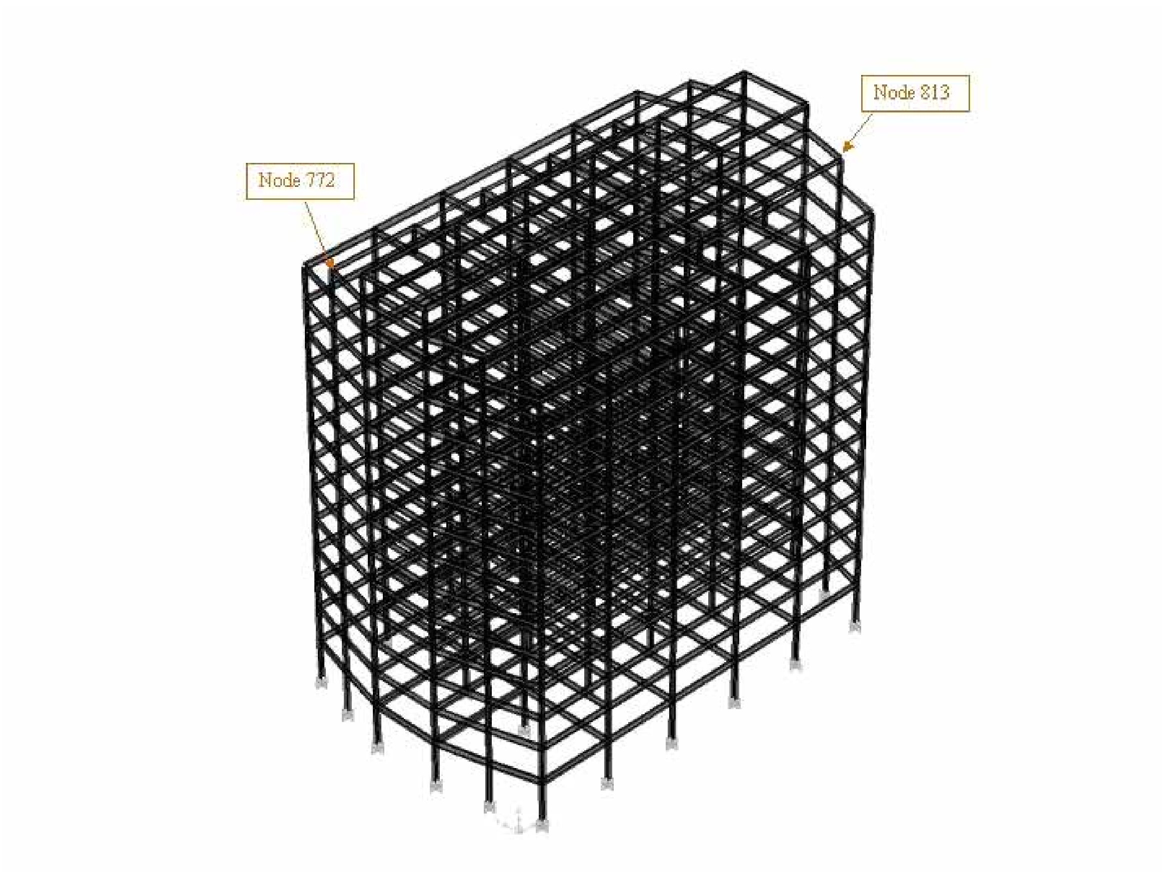

Fig. 8 3-D FE Model of the Building

The fundamental period obtained from Model 1 is 33.5% longer than the identified period, and 68.6% longer than the period calculated using the approximation formula in IBC 2000. Therefore, model improvements were undertaken. Five (5) different FE models were developed to identify the parameters most affecting the natural periods and the results are summarized in Table 3.

To evaluate the effects of the mass distribution on the natural periods, Model 2a and 2b were created. Model 2a had concentrated mass at the center 4 nodes of each floor. Model 2b incorporated the mass distributed in a relatively larger area of 6 nodes so that the eccentricities caused by the openings at the elevator core, the stairwells and non-symmetrically distributed masses were considered. The results show that the distribution of the mass had a negligible effect on the translational natural periods, but a clear effect on the torsional period. This is mainly due to the change in rotational inertia. The mass distribution at 6 nodes was used in the rest of all models.

Table 3 Summary of Finite Element Models

Model No.	Mass			Increase in Beam Stiffness	End Offset	Non-Struct. Stiffness	Natural Period (sec)		
	Orig.	Ref.	Eccent.				E-W	N-S	Tor.
1	X		5.0%, 4 nodes				2.97	2.85	1.00
2a	X		0.0%, 4 nodes				2.95	2.84	1.07
2b	X		3.0%, 6 nodes				2.95	2.84	1.13
3a		X	0.0%, 6 nodes				2.66	2.56	1.02
3b		X	3.0%, 6 nodes				2.67	2.57	1.02
4a		X	3.0%, 6 nodes	5%			2.64	2.54	1.01
4b		X	3.0%, 6 nodes	10%			2.62	2.51	1.00
4c		X	3.0%, 6 nodes	15%			2.59	2.49	0.99
5a		X	3.0%, 6 nodes	10%	50%		2.40	2.33	0.92
5b		X	3.0%, 6 nodes	10%	70%		2.32	2.26	0.88
5c		X	3.0%, 6 nodes	10%	90%		2.25	2.19	0.85
5d		X	3.0%, 6 nodes	10%	95%		2.23	2.18	0.94
5e		X	3.0%, 6 nodes	10%	100%		2.21	2.16	0.84
6		X	3.0%, 6 nodes	10%	95%	1%	2.20	2.15	0.83

Model 3a was developed to evaluate the effects of the mass calculation on the dynamic properties. The mass calculation was refined to more accurately represent the actual building mass. The openings for the elevators and stairwells at each floor, the large windows of the exterior wall areas and other non-symmetric masses were all considered. The refined overall mass of the building, i.e. 9.816×10^6 kg, is approximately 21% smaller than the original mass. The natural periods decrease by approximately 10% from Model 2b to Model 3a. Model 3a did not have any eccentricity, while Model 3b had about 3% of eccentricity which was computed from the actual mass distribution. Comparing Model 3a and 3b, it is found that 3% of change in eccentricity only slightly affects the natural periods. Hence, 3% of eccentricity and mass distribution in 6 nodes were used throughout the rest of the models.

Model 4 was created to investigate the effects of partial composite action between the floor deck and beams (Boroschek [7]). The beam stiffness was increased by a certain percentage to represent the possible partial composite action. Since typical connections between the metal deck and beams were not specified in the design documents, the beam stiffness was increased by 5% (Model 4a), 10% (Model 4b), and 15% (Model 4c) to observe the effects. The corresponding natural periods are included in Table 3 and shown in Figure 9 (a). Only 2% of changes in periods were observed between these three cases. Therefore, 10% beam stiffness increase was used in the rest of the models.

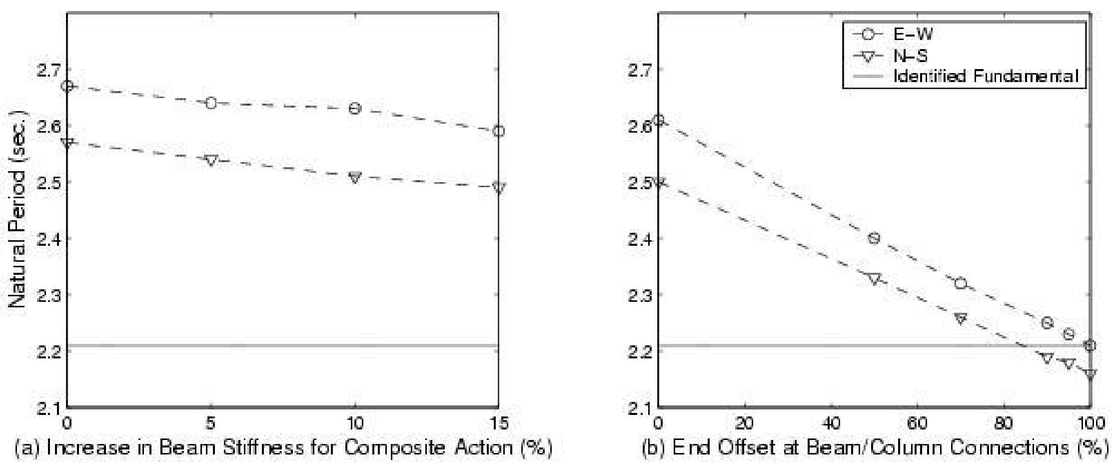

Fig. 9 Sensitivity of Natural Periods

Model 5 was created to study the effects of panel zone rigidity at the beam/column connections. A typical beam/column connection in this building is a fully bolted moment connection with A490-X 28.6 mm diameter bolts with column web double plates (13 mm to 25 mm thick) on both sides. To evaluate the effects of panel zone rigidity on the natural periods, end offsets ranging from 50% to 100% were used in Model 5a through Model 5e. The corresponding natural periods were shown in Table 3 and plotted Figure 9 (b). For comparison purpose, the results from Model 4b were treated as these for 0% end offset. The natural periods decrease by 14% to 16% as the end offset increases from 0% to 100%. Obviously, the effects of panel zone rigidity on the natural periods are very prominent. As a result, if more than 90% end offset is used, the E-W and N-S natural periods are both within 2% of the identified natural periods. Considering the beam/column connection, it is believed that 95% end offset can represent the actual panel zone rigidity of this type of construction and was used in the next model.

Finally, the stiffness contribution from non-structural elements was considered in Model 6. Braces were added around the exterior of the model to simulate the stiffness contribution of the external walls. The overall stiffness of the bare frame was 37.3-kN/mm in the E-W direction and 39.8-kN/mm in the N-S direction. The braces were adjusted to increase the lateral stiffness of the bare frame by about 1% in both E-W and N-S directions so that Model 6 produces natural periods in both E-W and N-S directions were within 1% of the identified model.

Time History Analysis

In order to assess the improvement, Model 1 and 6 were selected to conduct time history analyses to obtain model responses. The recorded accelerations at the free-field station (Ch 10, Ch 11, and CH 12) were used as the input. 5% damping ratio was used in the time history analysis of Model 1 based on code suggestions, whereas the identified average damping ratios for the two events were used in the analysis of Model 6.

The computed absolute acceleration and displacement time histories at nodes 772 (refer to Figure 10) and the recorded time histories at Ch 2 for the two events were plotted together in Figures 10, 11, 12, and 13, respectively. It is pointed out that the location of Node 772 is close but not exactly the same as the sensor location of CH 2. For better comparison, the Fourier amplitude spectra of acceleration time histories were also shown. The results for Model 1 are shown in Figures 10 and 11 while the results for Model 6 are shown in Figures 12 and 13. Comparing these results of Model 1 and Model 6 with the recorded, it is easy to see the large improvements in the models. Model 6 produces a much improved match between the computed and the recorded results in both time and frequency domains.

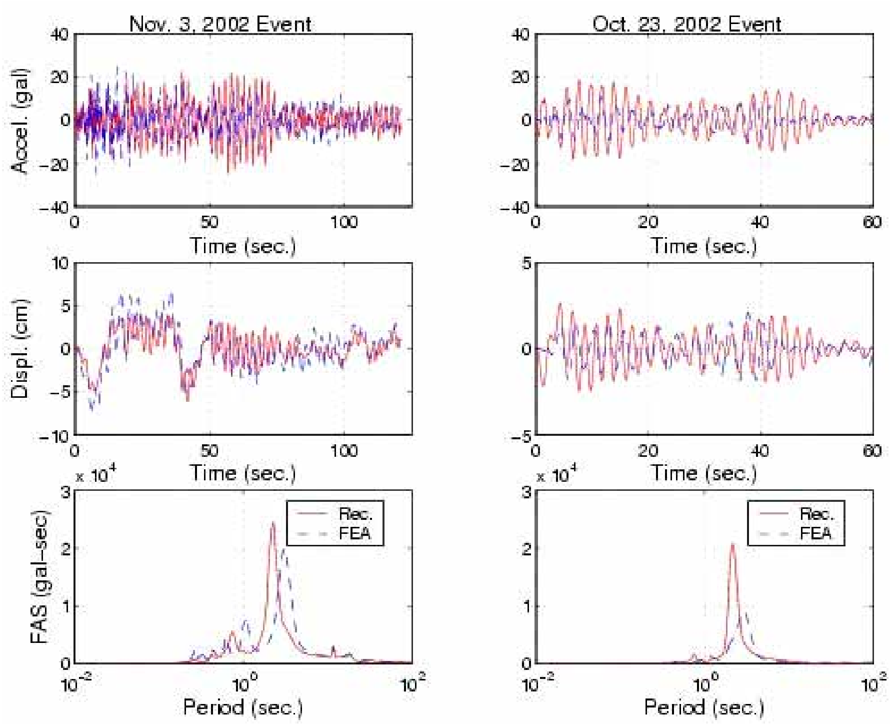

Fig. 10 Comparison of the Recorded & FEA (Model 1) Simulated Results in the E-W Direction

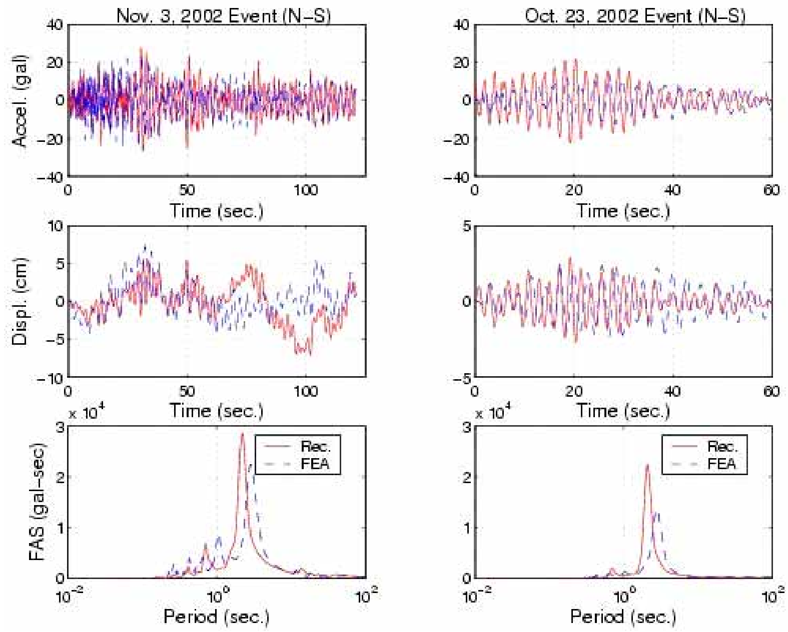

Fig. 11 Comparison of the Recorded & FEA (Model 1) Simulated Results in the N-S Direction

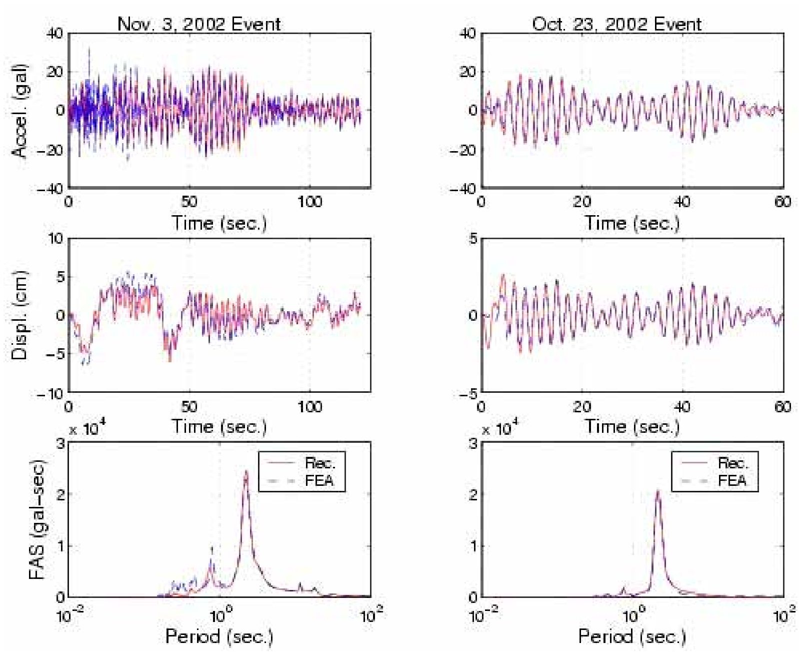

Fig. 12 Comparison of the Recorded & FEA (Model 6) Simulated Results in the E-W Direction

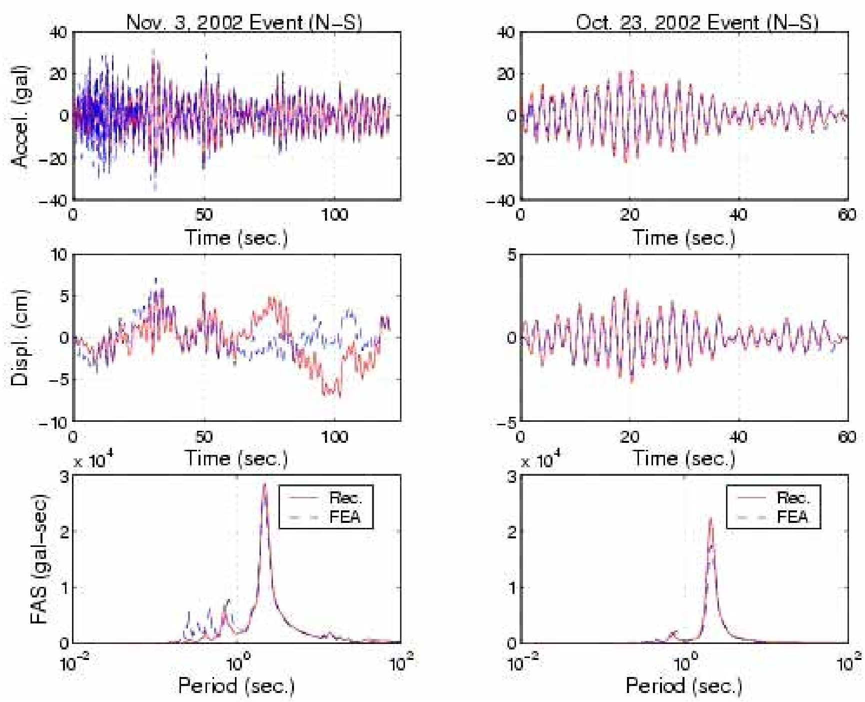

Fig. 13 Comparison of the Recorded & FEA (Model 6) Simulated Results in the N-S Direction

CONCLUSIONS

In this paper, the dynamic properties of an office building were identified by system identification technique based on recorded earthquake motions. A series of FE models were created to improve the accuracy of FE modeling. It was found that the fundamental period of the building computed using the approximation formula in IBC 2000 was 20.8% shorter than the identified. While the fundamental period calculated by FE modeling (Model 1) considered generally accepted engineering practices and assumptions suggested by IBC 2000 was 33.5% longer than the identified.

Several parameters that may be used to improve FE modeling accuracy were studied quantitatively. Among the parameters studied, the modeling study shows that the most effective ways to improve the accuracy of FE models are to refine the mass calculation (quantity and location) and consider the panel zone rigidity of the beam/column connections. Comparison of the recorded motions with dynamic analysis results of the "best fit" FE model shows a FE model can be refined to give very good fit to measured response. Therefore, linear FE models, after careful refinement according to design and construction details, can be used for prediction purposes.

The case study shows recorded seismic data can be very useful in improving the accuracy of FE modeling used in typical engineering design. Improving FE modeling will result in more cost effective structures and more reliable seismic design.

ACKNOWLEDGMENT

This work was supported primarily by the National Science Foundation's Experimental Program to Stimulate Competitive Research (EPSCoR) under a Seed Grant Award.

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