CORRELATING MEASURED AND SIMULATED DYNAMIC RESPONSES OF A TALL BUILDING TO LONG-DISTANCE EARTHQUAKES

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SUMMARY

For almost a decade, a 66-storey, 280 m tall building in Singapore has been instrumented to monitor its dynamic responses to wind and seismic excitations. The dynamic characteristics of the tall building have been investigated via both the finite element method and the experimental modal analysis. The properties of the finite element model have been shown to correlate well with those derived from data recorded during the ambient vibrations. During the study period, twenty sets of earthquake ground motions have been recorded at the building site. The ground motions may be divided into three categories based on their predominant frequency components with respect to the building's fundamental frequency. The calibrated three-dimensional finite element model is employed to simulate the seismic response of the tall building. Correlation analysis of the time histories between the recorded data and the simulated results has been carried out. The correlation analysis results show that the simulated dynamic response time histories match well with those of the recorded dynamic responses at the roof level. The results also show that the simulated maximum response at the roof level is close to the peak response recorded during the earthquakes.

INTRODUCTION

Dynamic response due to lateral loading is one of the most important design criteria for high-rise buildings. Depending on local situations, it may be controlled by either winds or earthquake ground motions. Singapore, an island republic 1.3 degrees north to the equator, is located in a neither seismic active nor strong wind region. However, in local practice, the design lateral load for building structures is based on a wind speed of 35 m/s,

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or the notional horizontal load specified in BS 8110 [1] as 1.5% of the characteristic dead weight applied laterally at each level simultaneously, whichever is greater.

From the instrumental measurements of several tall buildings in Singapore during the last decade, it has been found that the responses of tall buildings due to long-distance earthquakes, which are mainly large tremors propagating from Sumatra of Indonesia, were greater than those caused by winds [2]. A few events of earthquake tremors are believed to have generated base shear forces up to a magnitude comparable to that of the notional horizontal load, i.e. 1.5% of the building's total characteristic dead weight [3, 4]. Thus, the study on the response of high-rise buildings in Singapore to major, long-distance earthquakes has since attracted more research interests.

One of the tallest buildings in Singapore that has been instrumented to monitor its seismic response has recorded 20 sets of seismic response data during the last decade. A detailed three dimensional finite element (FE) model of the building is constructed to simulate the seismic structural response. The correlation analyses of the recorded and the simulated seismic responses of the instrumented tall building are presented in this paper.

REGIONAL SEISMICITY

Singapore is located at the southern tip of Peninsular Malaysia, which is an aseismic region. The closest earthquake belt to Singapore comprises the 1,500 km long Sumatra fault system and the subduction zone off the west coast of Sumatra, with the closest point about 350 km away from Singapore. Sumatra Island is part of the Indonesia island arc. The Indo-Australia plate subducts below the Eurasia plate at a rate of about 67 mm per year [5]. The displacement between the two plates is partly accommodated by sudden movements. Very large earthquakes have thus been generated along the interface between the two plates. The sizes, locations and timings of earthquakes are generally erratic. Figure 1 shows the epicentres of earthquakes that happened around the region from January 1, 1960 to April 30, 2002. However, it is important to note that two of the largest earthquakes in the region happened in last century, $M_w = 8.75$, 1833 and $M_w = 8.4$, 1863, respectively. Fortunately, at that time, there was no high-rise building or reclaimed land in Singapore. These major long-distance Sumatra earthquakes therefore didn't cause any problem to the local buildings. Historically, earthquake hazard has thus hardly been an issue for Singapore. However, with the rapidly increasing number of tall buildings constructed over the last two decades, more ground tremors which propagated from long-distance Sumatra sources have been reportedly felt by residents of Singapore's high-rise buildings. The latest event felt locally was from the Southern Sumatra event occurring on June 7, 2000 with Ms = 6.7 at an epicentral distance of 692 km, which is the aftershock of the Bengkulu event on June 4, 2000 with Ms = 8.0 [6].

BUILDING AND INSTRUMENTATION

The Meteorological Service Singapore (MSS) established the first national network of seismic stations in September 1996 for acquiring seismic data to characterize the ground motions in Singapore. Before that, no instrumental seismic ground motion time history was available in Singapore. The MSS network comprises seven seismograph stations equipped with seismometers and strong motion accelerometer arrays. One of the stations is located at rock site which is also part of the global seismographic network, two at soft soil sites, and the rest of four at firm soil sites. Among the four stations located at firm soil sites, one has a set of surface seismometers on the campus of Nanyang Technological University. In addition, several residential and commercial buildings have since been instrumented with bi-axial accelerometers. The experiences with one of these instrumented buildings are described in this paper.

The instrumented building, the Republic Plaza tower, is one of the tallest buildings in Singapore. It is a 66-storey, 280 m high tower that consists of a frame-tube structural system with a central core wall connected to a ring of external columns by a horizontal steel framing system at every floor. The typical floor plan is as shown in Figure 2. The reinforced concrete central core wall with a plan area of 21.5 m by 22.65 m extends almost the full height of the building, except a major opening at the southeast part at a few top storeys. The perimeter of the building comprises 16 large steel tubes filled with concrete. There are two tapering sections between storeys 20 and 27 and between storeys 44 and 46, respectively, where mechanical equipments are located and where outriggers are employed to enhance the lateral rigidity of the building. The structure sits on a deep, stiff caisson foundation system. The foundation system consists of six 60 m deep interior caissons connected by a 5.5 m thick concrete mat, and eight 40 m deep exterior caissons linked by transfer beams. The whole structure is symmetrical about the y-axis and slightly unsymmetrical about the x-axis due to the central core wall openings and the internal thin elevator core walls. More detailed descriptions of the structural system are given elsewhere [7].

Initially the instrumentation was designed to study the characteristics of winds and dynamic response of the tall building. After realizing the greater effects from long-distance earthquakes, the scope was switched to include monitoring of seismic responses. The instrumentation system consists of four servo-accelerometers, two 3-component anemometers and other hardware for purpose of converting digital data, storage and remote accessing, etc. Two servo-accelerometers are installed at the basement level (B1) along the x and y directions as shown in Figure 2, and the other two are at the roof level (Storey 65). The signal ranges of the accelerometers are set to ±10 mm/s^2 at the B1 level, and ±50 mm/s^2 at the roof level. The effects of machinery noises may be significant due to the small magnitudes of weak motions being recorded. A lowpass filter at 3 – 3.5 Hz is therefore used to reduce the machinery noises. Normally the wind can only excite the first natural mode of the building. Thus, a significant second mode response indicates the occurrence of ground tremors. Hence, isolated second-mode root mean square (RMS) spikes in both directions are used as the criteria for triggering the recording system for seismic monitoring. More detailed descriptions of the instrument system can be found elsewhere [2].

RECORDED MOTIONS

The ground tremors experienced in Singapore usually originate from large earthquakes occurring in Sumatra region that is at least 350 km away. Having travelled a long-distance to arrive in Singapore, the ground motions usually have relatively low frequency components and last for a few minutes. The magnitudes of the ground motions are much smaller than the typical damaging strong motions. The list of earthquakes that have triggered the instrumentation system in the Republic Plaza tower is summarized in Table 1. In total, 20 events have been recorded between March 1997 and January 2001. Among them, the record of the March 1997 Sunda earthquake is the first set of seismic ground motions recorded in Singapore with modern instruments. The epicentres of these earthquakes are shown in Figure 3. The epicentres of these events scatter around a very large region. Among the recorded events, the closest one was south of Medan city, 397 km away from Singapore, while the farthest one took place at Taliabu, 2,372 km away from Singapore. The peak values of the recorded acceleration time histories range from 0.05 mm/s^2 to 7.3 mm/s^2 at the basement level (B1) and from 0.17 mm/s^2 to 50 mm/s^2 at the roof level (Storey 65). They are also summarised in Table 1. The largest value of the recorded accelerations resulted from the main shock (Ms=8.0) of the Bengkulu earthquake (No. 14 of Table 1) which happened on June 4, 2000 [6].

Normally the time histories recorded at the basement level of a building may be different from the true free-field ground motions because of structure-soil interactions. However, in this case, it is believed that the difference is negligible because of the great rigidity of the caisson foundation, which was indicated by a small modal ordinate at the basement level [2]. Thus, the basement signals are used as the ground acceleration inputs for the FE model in the time history analyses discussed in a later section.

THE FINITE ELEMENT MODEL

A three dimensional mathematical model was constructed for numerical analyses. The FE model shown in Figure 4 contains 3,185 frame elements and 2,223 shell elements, and has 20,808 degrees of freedom. Besides the main structural elements, major openings, secondary openings and internal thin walls within the central core have all been considered in this model. Ambient vibration tests were carried out to assess the first few natural modes and frequencies of the building. The simulated and measured frequencies match well for the lower modes, as shown in Table 2 [8]. However, the higher mode frequencies do not match as well, and the correlation errors are generally larger for the higher modes. The long duration, low frequency ground motions experienced in Singapore would likely excite only the first few lower frequency modes of the building. Hence, the FE model is adequate for this response correlation study.

Figure 5 shows the first six natural mode shapes of the FE model: two for the translational motions in each horizontal direction and two for the torsional motions about the vertical axis. For each mode, the views of three coordinate planes are shown in the figure where solid lines indicate the deflected mode shapes and grey lines indicate the undeformed configurations. Modes 1 and 2 are the translational modes in the x and y directions, respectively. Thus, the x and y directions in the FE model are the principal directions of the building, although the x direction modes are slightly coupled with torsion. The slight coupling results from the slight unsymmetrical structural layout about the x-axis due to the major opening of central core walls and the internal thin walls in the upper part of the building. The coupling of the x translational mode with the torsional mode becomes more prominent in the higher modes (modes 3 and 6). It appears that modes 1, 2, and 3 are the fundamental x, y, and torsional modes.

Table 3 shows the modal participation factors and the cumulative modal participation mass ratios of modes 1 to 6. Having compared the relative values of these two parameters for each individual mode, it appears that modes 1, 3 and 4 are primarily the x-direction response and that modes 2 and 5 are primarily the y-direction response. The cumulative modal participation mass ratios for the first 20 modes reach 96.7% and 96.2% in the x and y directions, respectively. Therefore, the first 20 modes are used in the modal superposition analyses that follow.

DAMPING RATIOS

The value of damping ratio is an important parameter in dynamic analysis and is also one of the most uncertain factors. The modal damping ratios for the translational modes of the Republic Plaza tower were determined based on the ambient vibration test results [2] and are shown in Table 4. The values fall in a small range between 0.52% and 0.87%. They were the upper bounds and used for each respective translational mode. The averaged value of 0.73% was used for the higher frequency modes for all the events analysed.

However, the damping ratios for the torsional modes have to be assumed for the FE model because the ambient vibration tests didn't provide sufficient information for the torsional response of the building. The transfer functions of the recorded and the simulated accelerations between the basement and the roof were studied. Figure 6 shows the transfer functions for the recorded x-direction response, the simulated x-direction response with 0.73% torsional damping ratio which is the averaged translational damping ratio, and the simulated x-direction response with 5% torsional damping ratio. The transfer function for the recorded response is computed based on several sets of recorded data. As shown in Figure 6, all the transfer functions peak at the natural frequencies of about 0.19 Hz, 0.70 Hz and 1.55 Hz corresponding to the translational modes. The troughs in the transfer functions indicate the torsional response frequencies of the FE model. The transfer function of the simulated x-direction response with 0.73% torsional damping ratio shows a trough at 0.6 Hz, 1.3 Hz and 2.3 Hz, corresponding to the first three torsional mode frequencies identified in the ambient vibration tests. However, the troughs are not obvious in the recorded response. The absence of troughs in the transfer function of the recorded data in the x-direction may suggest that the torsional components in the actual building response are smaller than those simulated by the FE model. The torsional damping ratios for the FE model may therefore be increased to reduce the contribution of torsional response towards the simulated response. A 5% damping ratio was thus used for the first three torsional modes. Figure 6 shows that the troughs in the transfer function of the simulated x-direction response with 5% torsional damping ratio become less significant.

CORRELATIONS OF TIME HISTORIES

The simulated and recorded roof response time histories are compared in this section. The power spectrum was calculated through the Fast Fourier Transformation (FFT) method for every recorded acceleration time history to study its frequency properties. According to the frequency components identified, the time histories can be generally divided into three groups with respect to the building fundamental frequency. One representative event of each group is analyzed in the sections that follow.

Signals dominated by low frequency

These earthquakes are normally large but occur at a very long distance away from Singapore. Most of the high frequency components of the waves would have been filtered out by the earth during the travel, and only the low frequency surface waves and a small portion of higher frequency body waves arrive in Singapore. The Taliabu event which was of magnitude Ms = 8.3 and occurred at 2,375 km away is chosen to represent this group of events. Figure 7 shows the recorded ground motions in the x- and y-direction and their corresponding power spectra. It can be seen that the components with frequency lower than 0.1 Hz are dominant in the recorded signals. Under such excitations, the building is likely to undergo mainly a rigid-body motion, plus some first mode responses. The ground displacement time histories in the x- and y-direction integrated from the acceleration time histories are shown in Figure 8.

Figures 9a and 9b show the comparison of the acceleration, velocity and displacement time histories between the recorded and the simulated responses at the roof level in the x and y directions. The solid lines in the figures represent the recorded time histories or power spectra, and the dotted lines show the corresponding simulated results. Comparing the ground displacements in Figure 8 and their corresponding roof displacements in Figure 9a and 9b, it can be seen that the whole building experienced mainly a rigid-body motion plus the first mode response during the event. The first mode response is manifested via the small ripples whose frequency corresponds to the first mode frequency in the respective direction in the roof displacement time histories. The FE model reproduces well the roof response time histories that consist of the rigid-body motion plus the first mode response. However, the FE model underestimates the response in the x direction while overestimates that in the y direction during the high frequency response in the beginning of the event. From the power spectra (Figures 9c and 9d), it can be seen that only the first natural mode in each direction was excited. The two sets of power spectra for the recorded and the simulated results match well. However, the simulated results at the first mode frequency are larger than the recorded ones in the y direction.

Table 5 shows the maximum values of the recorded and the simulated roof responses. The maximum simulated roof displacements are 7.5 mm and 4.3 mm in the x and y directions, respectively. Compared with the maximum recorded roof displacement responses of 8.1 mm and 4.9 mm, the correlation is good. The recorded and the simulated maximum accelerations and velocities also correlate well with each other as shown in Table 5.

Signals dominated by high frequency

These seismic events mainly originate from the Sumatra fault system with a relatively small magnitude (Ms between 5.0 and 6.5). Upon arriving in Singapore, the waves consist of relatively high frequency motions between 0.3 Hz and 4 Hz. Figure 10 shows the motions recorded during one of the Southern Sumatra earthquakes in the year 2000 of magnitude Ms = 5.8. The surface waves are negligible in this case, compared with the body waves considering some wind and noise effects in the low frequency components. Such ground motions could excite some higher modes of the structure.

Because the low frequency components are negligible, the basement signals are high passed at 0.15 Hz before being used as inputs to the FE model. Choosing 0.15 Hz is to minimize the effects of very low frequency components and at the same time avoid a significant loss of the first mode response at 0.19 Hz. Figures 11a to 11d show the comparison of the roof response time histories and the power spectra between the recorded signals and simulated results for this event. From the power spectra of the recorded roof responses, it appears that the first two natural modes in the x direction, at 0.19 Hz and 0.69 Hz, were excited during the event. The first two translational natural modes in the y direction, at 0.2 Hz and 0.71 Hz, were also excited. However, the torsional mode response is not obvious. The magnitudes of the peaks on the recorded and the simulated power spectra are similar for the first and the second modes in both translational directions. The FE model has a difficulty in simulating the third translational mode response and the torsional response. Even though the responses with high frequency in this case don't match as well as those in the low frequency dominant case, the FE model does reproduce the general trends of motions, especially in the displacement time histories.

As shown in Table 5, the recorded and the simulated maximum roof responses correlate well, except the acceleration and velocity in the x direction. During the event, the maximum roof displacements simulated are 0.15 mm and 0.19 mm in the x and y directions, respectively, compared with those of 0.15 mm and 0.20 mm recorded.

Signals dominated by both low and high frequencies

This group of events mainly occurs around the Sumatra Island but with a larger magnitude. The ground acceleration records and their power spectra from the earthquake occurred in year 2000 at Bengkulu, 685 km away from Singapore, are shown in Figure 12. The earthquake is of magnitude Ms = 8.0 and is the biggest event recorded in terms of the peak ground accelerations since the operation of the instruments [6]. According to the power spectra, the magnitudes of low frequency and high frequency components are comparable for this event. From the relatively quiet signals prior to the earthquake arrival, it can be seen that the data were well recorded with little noise or wind disturbance.

The simulated results and the recorded roof motions are shown in Figures 13a to 13d. The responses consist of mainly the first two natural modes in both directions and the rigid-body motions. The smaller spikes in the power spectra of the simulated results at the first translational mode frequency in each direction, 0.19 Hz and 0.21 Hz for the x and y directions respectively, suggest that the FE model may be stiffer than the actual structure. However, for this largest event recorded, the simulated and the recorded time histories match very well in both the displacement and velocity responses, except in the later portion of the responses in the y direction, where the simulated results are smaller compared with the field records. Such phenomena were also observed in some other events. The correlation of acceleration time histories also appears to be much better for this largest event than for the other events.

The maximum recorded and simulated roof responses of the event are summarized in Table 5. As shown, the maximum simulated roof displacements are 16 mm and 31 mm in the x and y directions and those of the recorded roof displacements are 21 mm and 33 mm.

MAXIMUM BASE SHEARS

The maximum base shear forces from nine selected time history analysis cases are summarized in Table 6. The absolute maximum value of the base shear force results from the Bengkulu event (No. 15), 1,158 kN in the y-direction. This is about 0.15% of the total characteristic dead weight of the building, which is around 760,000 kN. Compared with the notional horizontal load of 1.5% of the total weight as specified in the BS8110 code, the 0.15% base shear coefficient is relatively small.

DISCUSSIONS AND CONCLUSIONS

During the last decade, the Republic Plaza tower has been instrumented to monitor the roof response to ground motions resulting from long-distance earthquakes that mainly occurred in the Sumatra region. An FE model, whose first few natural frequencies were verified with the ambient vibration testing results, was used to study the frequencies and mode shapes of the building. The dynamic characteristics of the FE model show that the x translational modes are coupled with the torsional modes because of a slightly unsymmetrical structural layout of the building along the x-axis.

Time history analyses were carried out to study the structural response during the recorded long-distance earthquakes. The following conclusions can be drawn from the analyses:

1. The weak seismic ground motions originated from the long-distance earthquakes consist of only low frequency signals and could only excite the first and the second translational modes of the building. The torsional response was not as obvious in the recorded building response.
2. The damping ratios of torsional modes appear to be higher than those of translational modes.
3. It is shown that the FE model can reproduce the general trends of building movement time histories as well as the peak responses at an acceptable accuracy level. In general, the correlations of the roof displacement responses are much better than those of the velocity and acceleration responses.
4. The FE model works better in the lower modes than the higher modes. This may be partially due to the participation of torsional response in the higher modes, as well as the lower quality of the high frequency signals which have a small magnitude that may be contaminated by external sources. The better performance of the FE model in a larger event, e.g. the Bengkulu event, is a good indication. Nevertheless, under the ground excitations originated from the long-distance seismic events, the building movements are normally dominated by the first mode. The influence of higher modes is thus usually less significant.

5. The maximum base shear force induced by the long-distance earthquakes recorded was found to be around 0.15% of the total building dead weight. This is much smaller than the notional horizontal load specified as 1.5% of the characteristic dead weight of the building in the BS8110 code.
6. Because of the complexity in the dynamic response of a real structure, it is difficult for even an accurate FE model, which has been verified by good correlation between simulated and measured natural modes and frequencies, to generate accurate time history responses. In this case, it may be due to less understanding on the torsional response and damping of the structure. Hence, it needs to be extremely careful when a finite element model is used to predict the seismic response of a large building, especially when the model has not been calibrated against experimental data.

ACKNOWLEDGEMENTS

The authors are grateful for the continual support of the City Developments Ltd, Meteorological Service Singapore, and Shimizu Corporation, Japan. Without their help, this work would not have been possible.

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Table 1. Earthquakes recorded at Republic Plaza

No.	Event Time		Epicenter			Epicentral Distance (km)	Ms	Max. Response(mm/s2)	
	Date	GMT	Long	Lat	Location			B1	L65
1	17-Mar-97	08:05:48	105.5E	6.6S	Sunda	892	6.4	0.27	4.70
2	18-May-97	22:14:18	99.74E	1.87S	Southern Sumatra	572	5.4	0.44	2.40
3	7-Jul-97	11:24:37	97.6E	1.1N	Northern Sumatra	695	5.9	0.64	3.20
4	20-Aug-97	07:15:00	96.5E	4.4N	Aceh	883	6	0.81	8.60
5	18-Dec-97	05:46:57	99.6E	1.95S	Southern Sumatra	591	5.7	0.64	3.90
6	29-Nov-98	14:10:32	124.9E	2.1S	Taliabu	2372	8.3	0.61	3.10
7	18-Feb-99	04:35:57	102.0E	2.8S	Southern Sumatra	494	5.1	1.20	6.40
8	14-Aug-99	00:16:52	104.7E	5.9S	Southern Sumatra	798	6.5	0.40	4.10
9	18-Sep-99	12:52:35	123.3E	4.0S	Southern Sumatra	590	5.2	0.53	5.40
10	11-Nov-99	18:05:43	100.3E	1.28N	Central Sumatra	397	6.2	0.87	4.80
11	2-Dec-99	12:40:39	101.5E	2.9S	Southern Sumatra	523	5	0.13	1.20
12	21-Dec-99	14:14:57	105.6E	6.8S	Southern Sumatra	918	6.6	0.57	8.00
13	4-May-00	04:21:17	125.6E	1.11S	Sulawesi	2216	7.6	0.05	0.17
14	4-Jun-00	16:28:25	102.1E	4.75S	Bengkulu	685	8	7.30	50.00
15	4-Jun-00	16:39:25	102.1E	4.65S	Bengkulu	683	6.7	5.20	40.00
16	22-Jul-00	20:56:12	102.4E	4.1S	Southern Sumatra	605	5.8	0.38	2.60
17	12-Sep-00	16:27:24	101.8E	5.4S	Bengkulu	777	6.1	0.16	1.80
18	22-Sep-00	18:22:02	102.1E	5.0S	Bengkulu	717	6.2	0.38	5.20
19	25-Oct-00	09:32:22	105.6E	6.55S	Sunda	884	6.8	0.56	8.10
20	16-Jan-01	13:25:00	101.7E	3.96S	Bengkulu	633	6.7	1.10	8.80

Table 2. Correlation of first few natural mode frequencies

Mode	X1	X2	X3	Y1	Y2	Y3	T1	T2	T3
Field(Hz)	0.19	0.70	1.55	0.20	0.75	1.73	0.57	1.34	2.31
FEM(Hz)	0.19	0.82	1.70	0.21	0.83	1.90	0.60	1.28	2.26
Error(%)	1.57	16.64	9.48	7.54	11.26	9.65	6.36	-4.25	-2.16

Table 3. Modal participation factors and participation mass ratios

Mode	Frequency (Hz)	Modal Participation Factor		Cumulative Participation Mass Ratio	
		UX	UY	UX	UY
1	0.19	-6978.1	-0.3	62.6	0.0
2	0.21	-0.6	6926.8	62.6	61.7
3	0.60	2676.6	0.7	71.8	61.7
4	0.82	-2803.2	-3.5	81.9	61.7
5	0.83	-2.8	3920.5	81.9	81.5
6	1.28	-1413.4	-0.8	84.5	81.5

Table 4. Modal damping ratios for the first three translational modes

Direction Mode	X			Y			Average
	X1	X2	X3	Y1	Y2	Y3	
Damping (%)	0.66	0.85	0.87	0.70	0.52	0.77	0.73

Table 5. Peak roof responses of the three events analysed

No.	Event	Direction	Acceleration(mm/s^2)		Velocity(mm/s)		Displacement(mm)	
			Recorded	Simulated	Recorded	Simulated	Recorded	Simulated
6	Taliabu	X	2.9	2.4	3.4	3.1	8.1	7.5
		Y	3.3	3.1	1.5	1.5	4.9	4.3
16	Southern Sumatra	X	2.6	1.5	0.59	0.31	0.15	0.15
		Y	2.3	2.6	0.49	0.52	0.2	0.19
14	Bengkulu	X	50	31	18	13	21	16
		Y	47	54	23	20	33	31

Table 6. Maximum base shear forces from simulation results

No.	Events	Shear Force in X (kN)	Base Shear Coefficient (%)	Shear Force in Y (kN)	Base Shear Coefficient (%)
1	Sunda	96	0.013	96	0.013
4	Aceh	135	0.018	68	0.009
6	Taliabu	99	0.013	73	0.010
10	Central Sumatra	64	0.008	63	0.008
13	Sulawesi	100	0.013	80	0.010
14	Bengkulu	539	0.071	1007	0.133
15	Bengkulu	873	0.115	**1158**	**0.152**
16	Southern Sumatra	23	0.003	37	0.005
17	Bengkulu	21	0.003	22	0.003

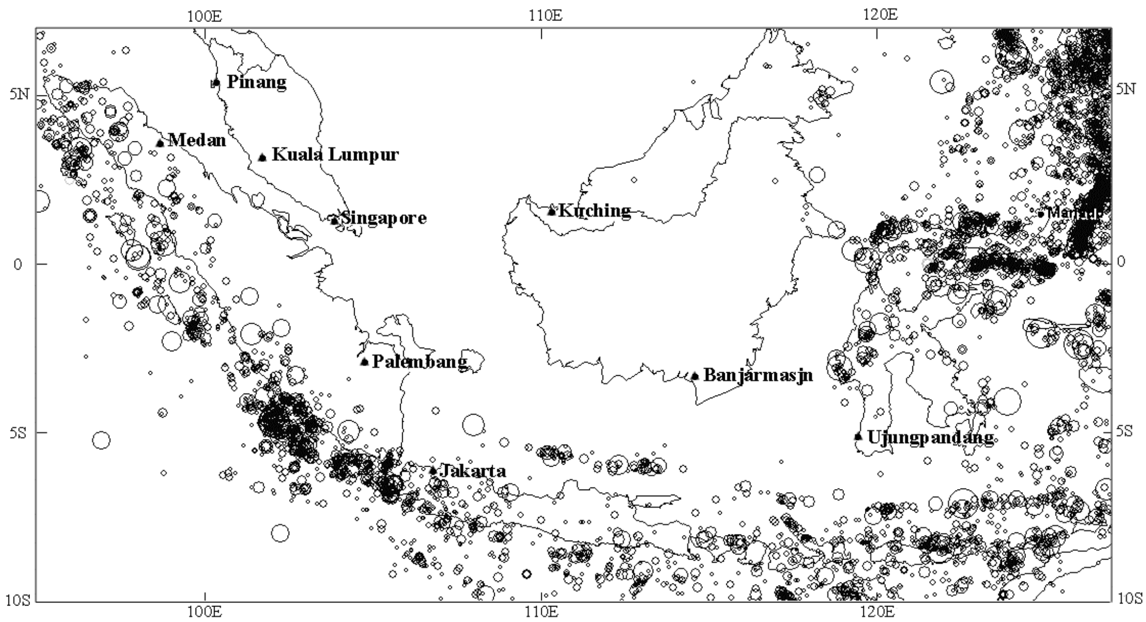

Figure 1. Epicentres of earthquakes in Sumatra region (1960.01.01 – 2001.04.30)

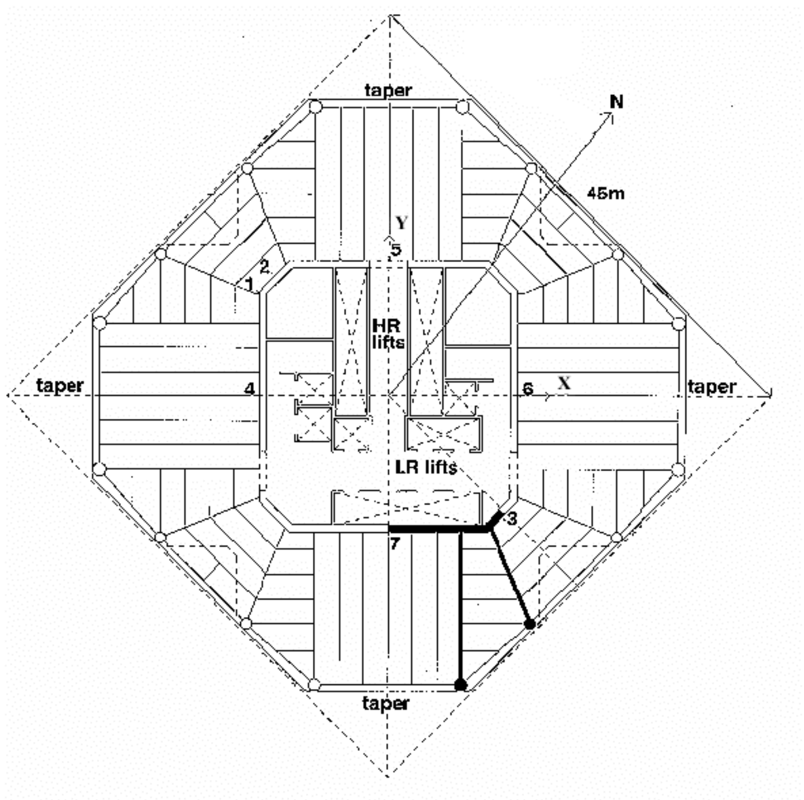

Figure 2. Floor plan of the Republic Plaza tower

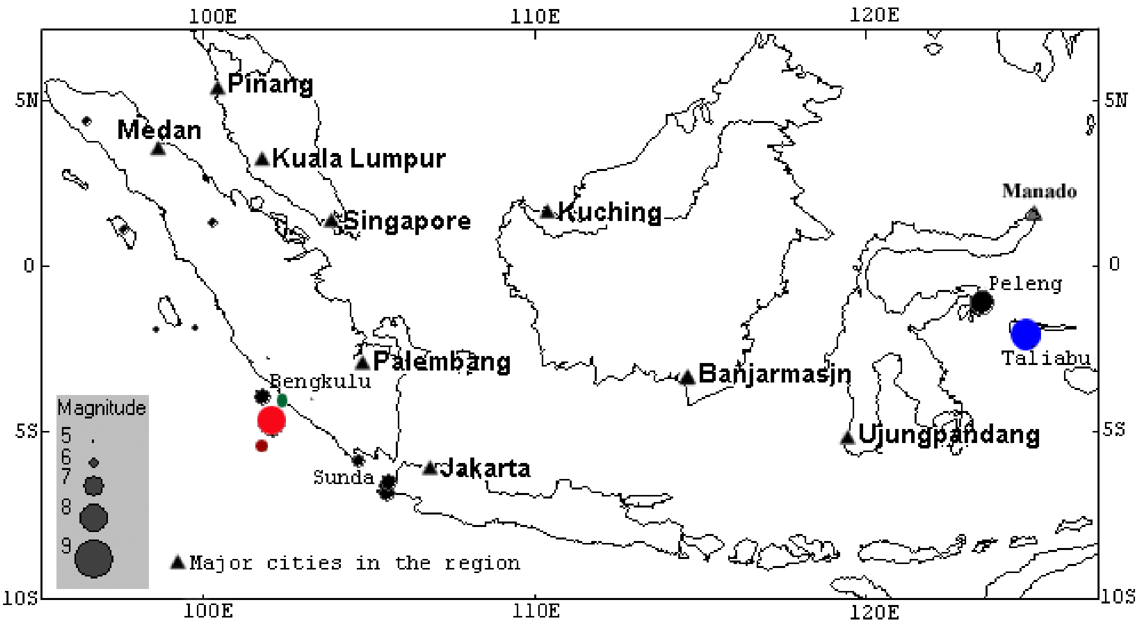

Figure 3. Epicentres of recorded earthquakes

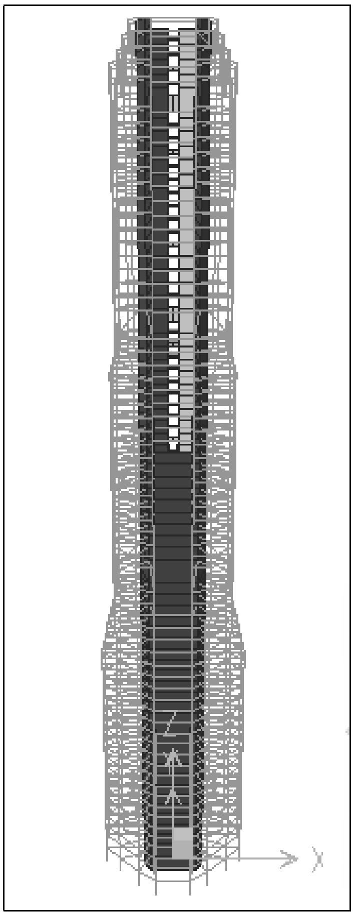

Figure 4. 3D view of the FE model

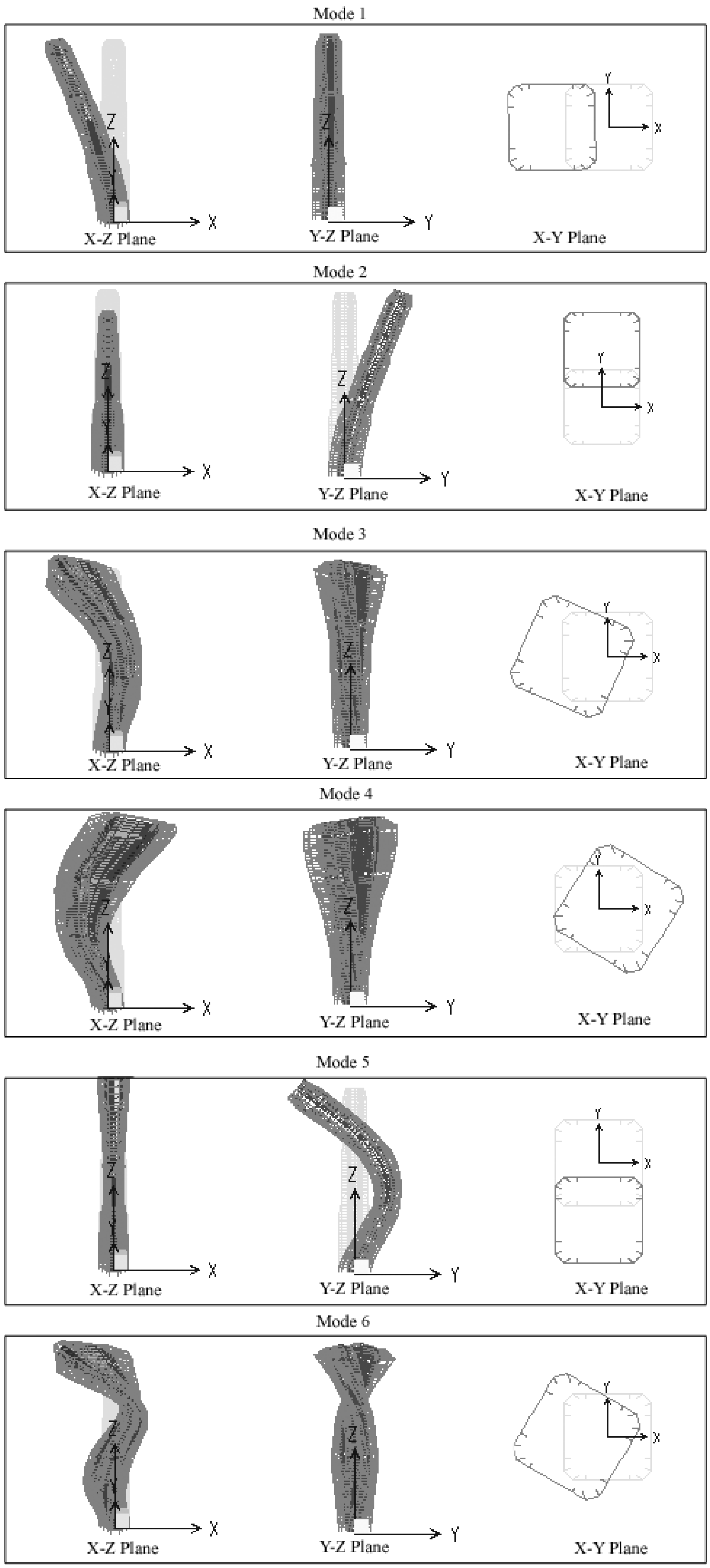

Figure 5. The first 6 natural mode shapes of the FE model

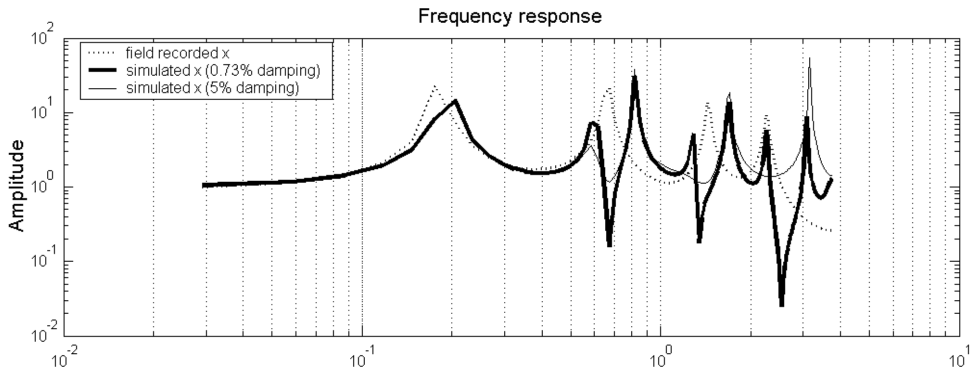

Figure 6. Transfer functions of the recorded and the simulated responses

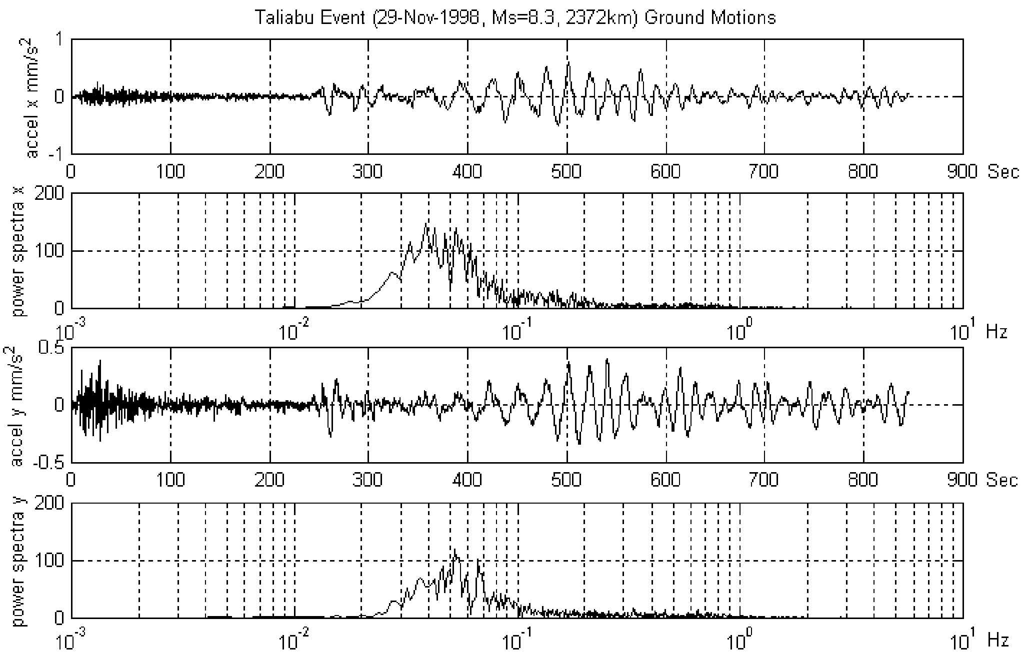

Figure 7. Ground accelerations of Taliabu event (1998.11.29)

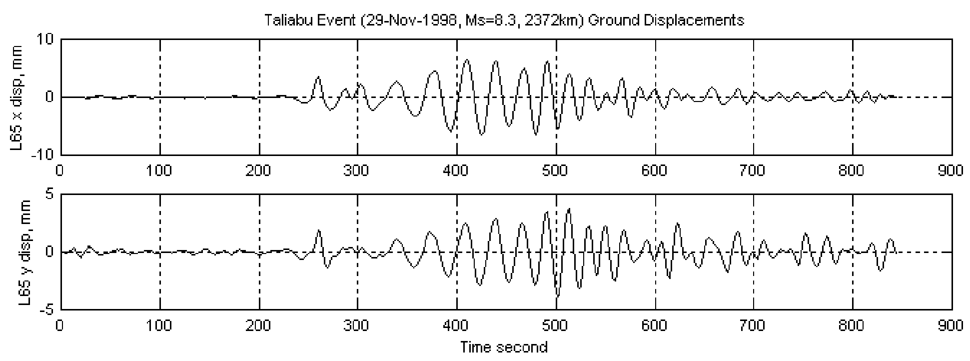

Figure 8. Ground displacements of Taliabu event (1998.11.29)

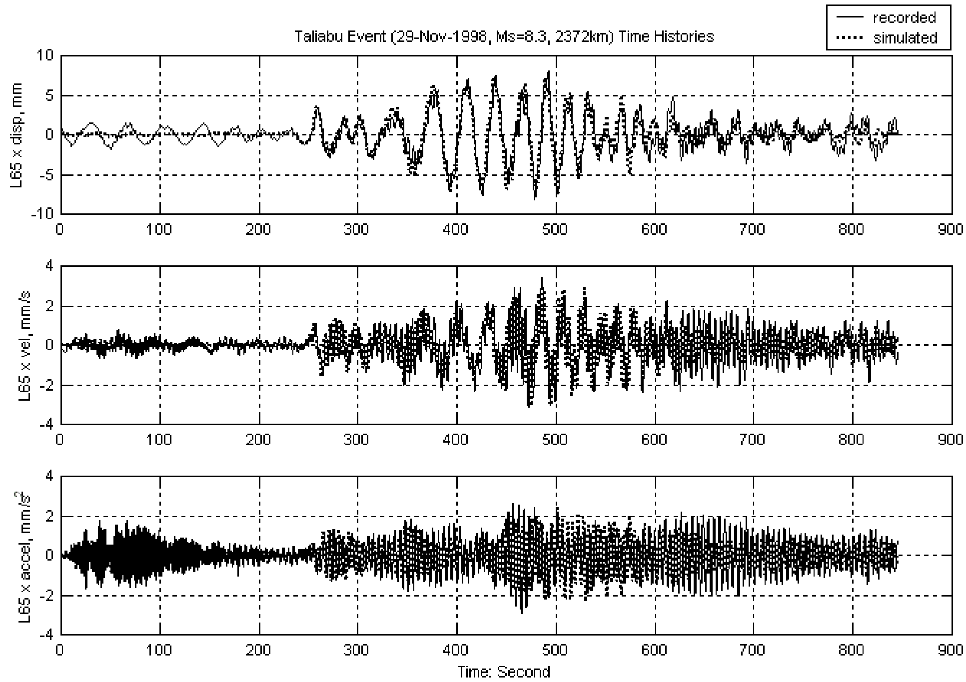

Figure 9a. Taliabu event (1998.11.29): comparison of the time histories at roof level in the x direction

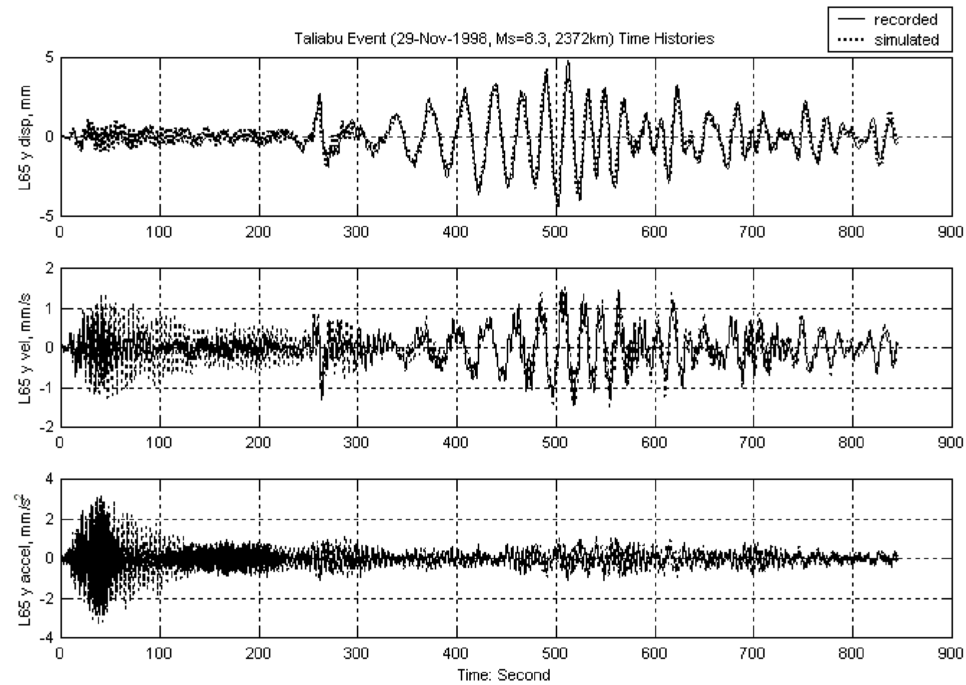

Figure 9b. Taliabu event (1998.11.29): comparison of the time histories at roof level at roof level in the y direction

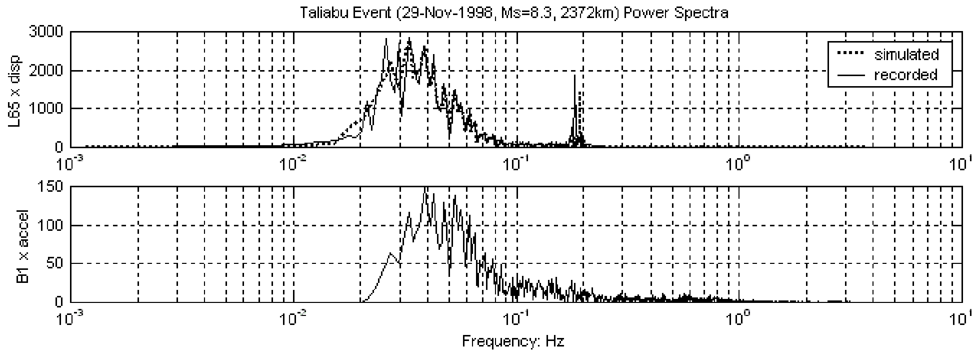

Figure 9c. Taliabu event (1998.11.29): comparison of power spectra at roof level in the x direction

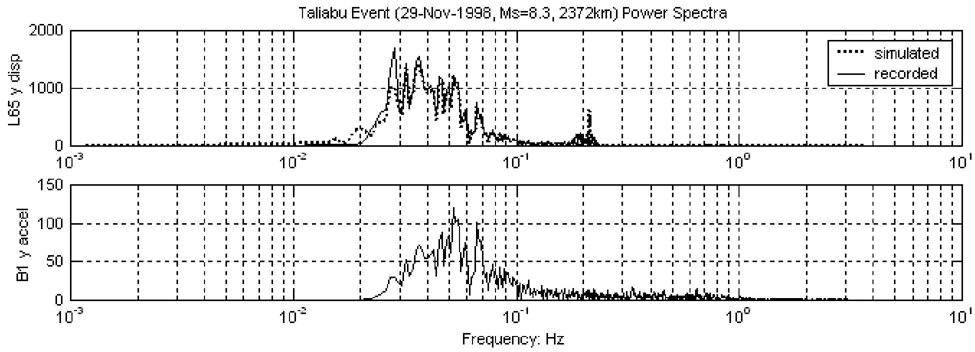

Figure 9d. Taliabu event (1998.11.29): comparison of power spectra at roof level in the y direction

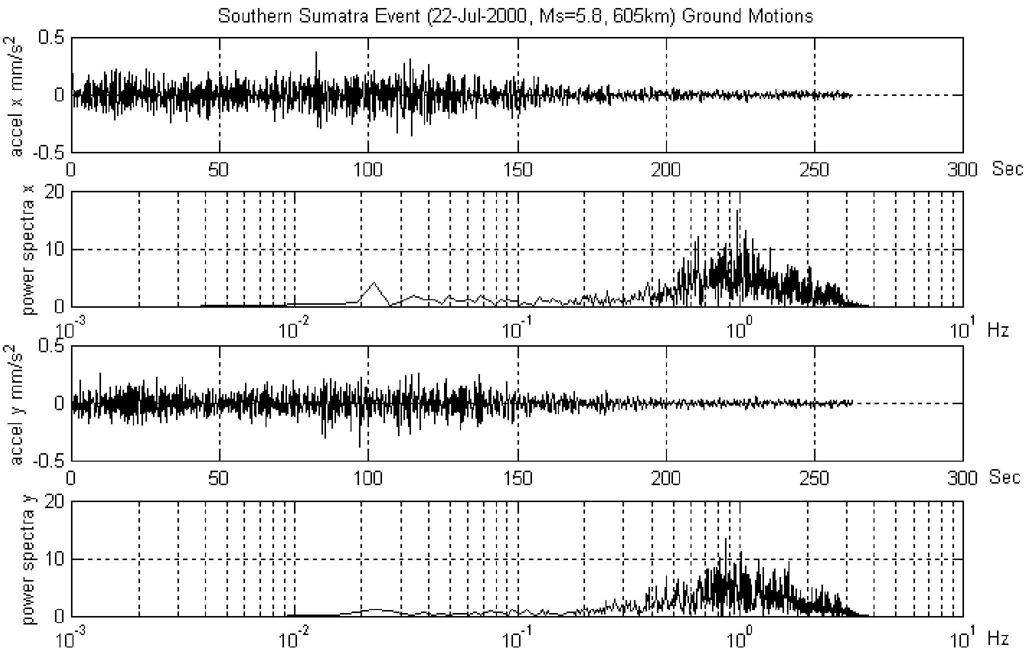

Figure 10. Ground accelerations of Southern Sumatra event (2000.07.22)

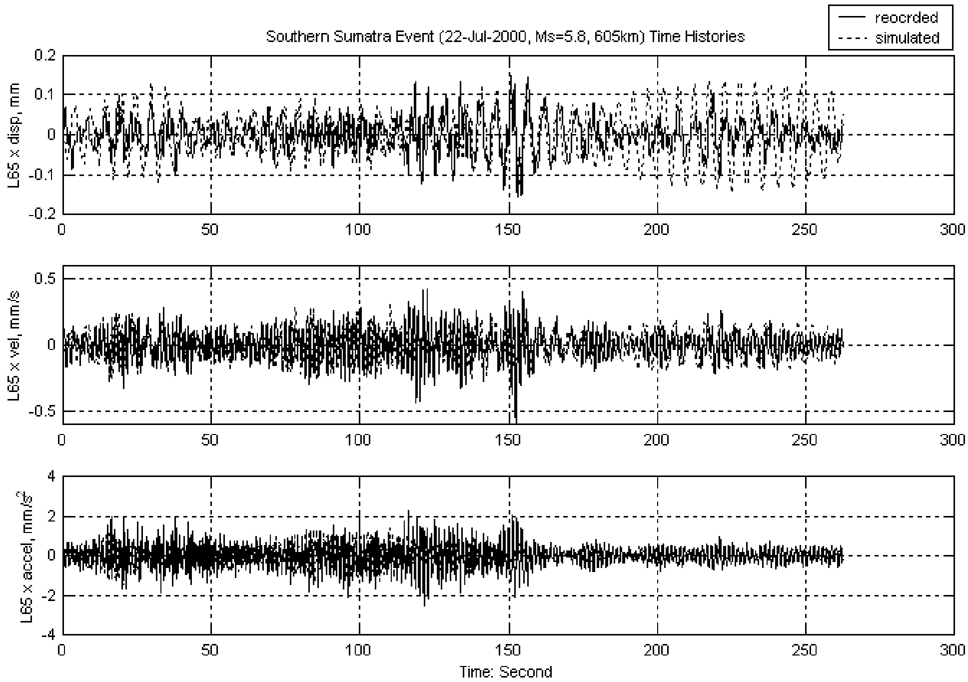

Figure 11a. Southern Sumatra event (2000.07.22): comparison of the time histories at roof level in the x direction

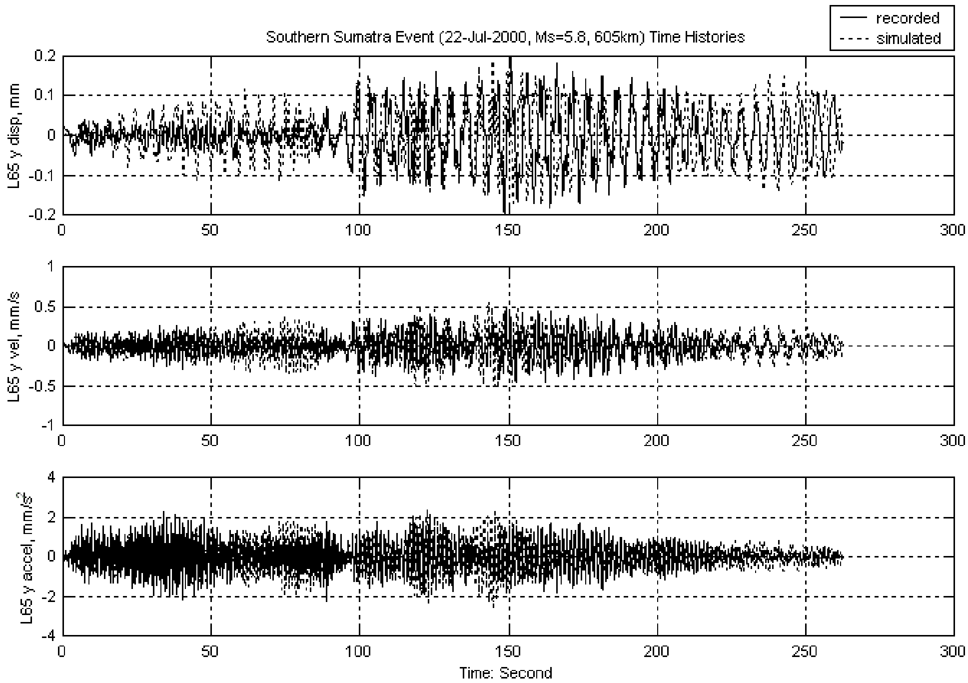

Figure 11b. Southern Sumatra event (2000.07.22): comparison of time histories at roof level in the y direction

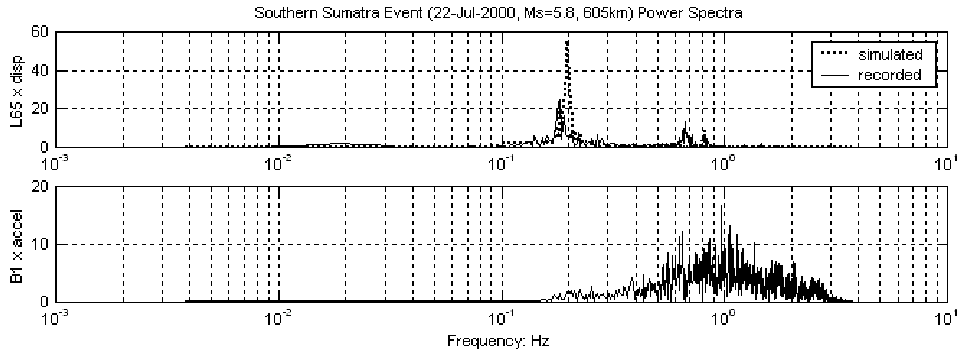

Figure 11c. Southern Sumatra event (2000.07.22): comparison of power spectra at roof level in the x direction

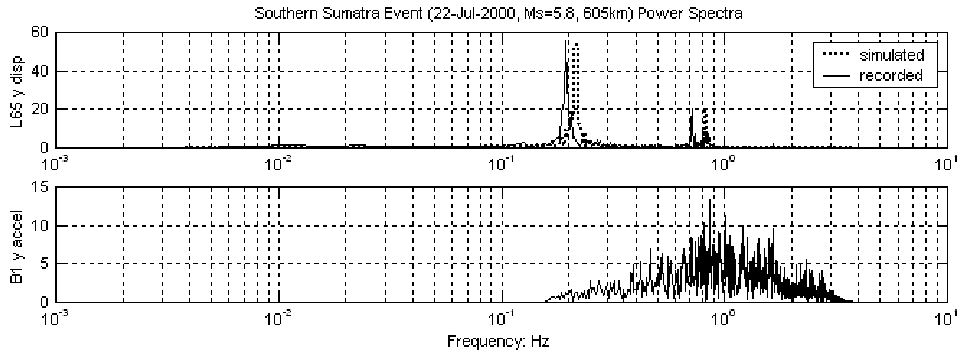

Figure 11d. Southern Sumatra event (2000.07.22): comparison of power spectra at roof level in the y direction

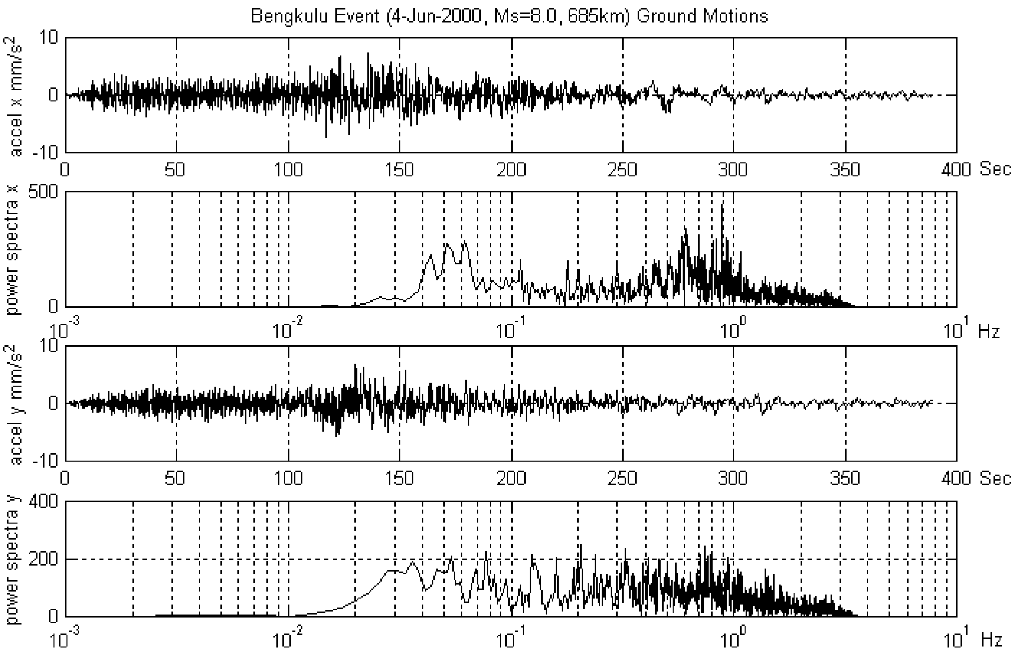

Figure 12. Ground accelerations of Bengkulu event (2000.06.04)

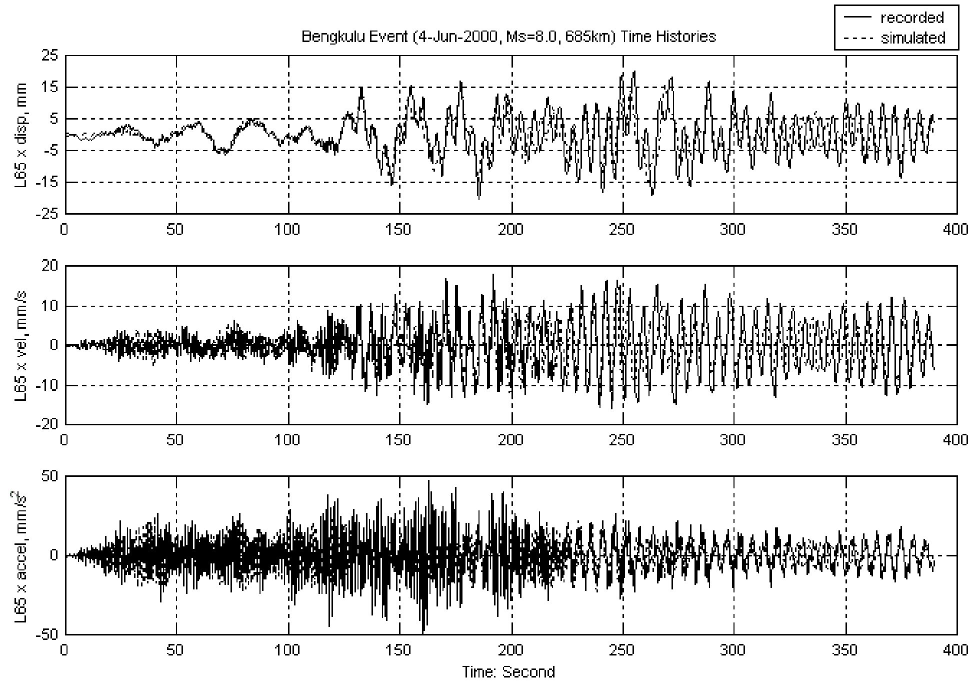

Figure 13a. Bengkulu event (2000.06.04): comparison of time histories at roof level in the x direction

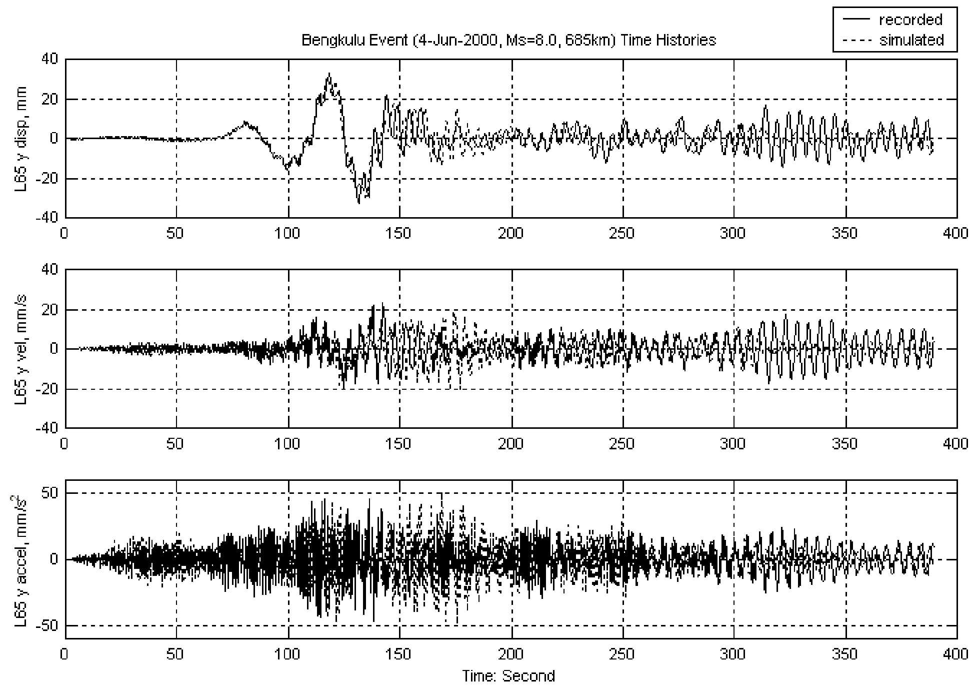

Figure 13b. Bengkulu event (2000.06.04): comparison of time histories at roof level in the y direction

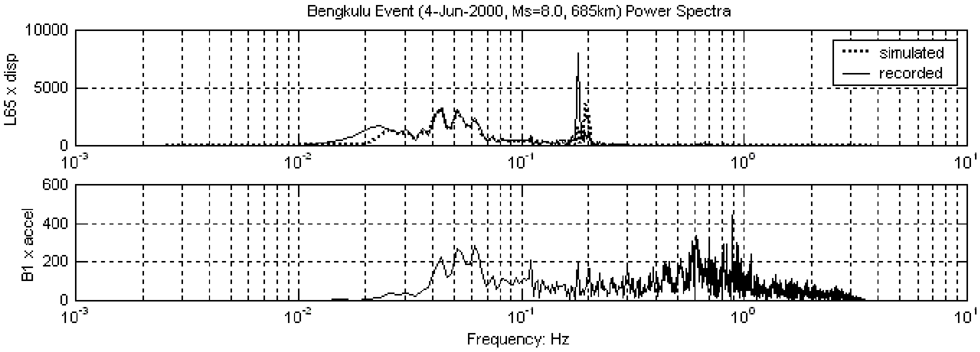

Figure 13c. Bengkulu event (2000.06.04): comparison of power spectra at roof level in the x direction

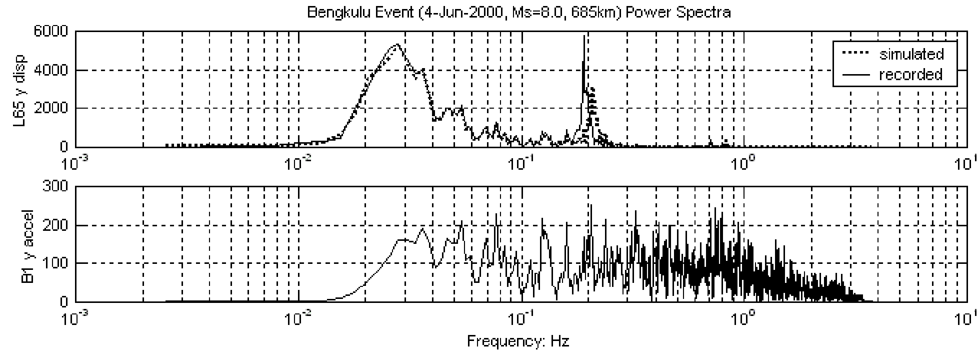

Figure 13d. Bengkulu event (2000.06.04): comparison of power spectra at roof level in the y direction