Strong Ground Motion by September 30, 2009 Pariaman Earthquake and Damage To Large Scale Buildings

M.H. Pradono
Agency for the Assessment and Application of Technology, Jakarta, Indonesia

Y. Goto
Earthquake Research Institute, the University of Tokyo, Tokyo, Japan

R.P. Rahmat
Graduate School of Kyoto University, Kyoto, Japan

A. Hayashi & K. Miyatake
Oriental Consultants Co. Ltd., Tokyo, Japan

SUMMARY:
On September 30, 2009, an earthquake of Mw 7.6 struck the west coast of Sumatra of Indonesia, affecting Padang and Pariaman, causing significant damage to about 150,000 houses and buildings as well as claiming more than 1,000 lives. One of the remarkable features of this disaster was damage to large-scale reinforced concrete buildings in Padang, the capital city of West Sumatra. Since the shaking during this earthquake was not recorded instrumentally in the downtown area of Padang, the authors estimated its intensity and predominant frequency through several approaches. One of the damaged buildings located in the downtown area of Padang was selected. It was severely damaged but remained almost upright. The third-floor columns were severely damaged. This damage obviously resulted from the reduction of column cross-section on this floor. The demand moment at the columns exceeded the capacity moment by double. Consequently, the columns were heavily damaged.

Keywords: reinforced concrete building, earthquake damage, earthquake wave synthesis

1. INTRODUCTION

According to BMKG (Agency for Meteorology, Climatology and Geophysics of Indonesia), the epicenter was located at latitude -0.84, longitude 99.65 degrees, and at a depth of 71 km. This is about 80 km west of Padang. The magnitude on the Richter scale was 7.6. The earthquake was assumed to be inner-plate type and the fault mechanism to be thrust faulting. No significant tsunami was generated.

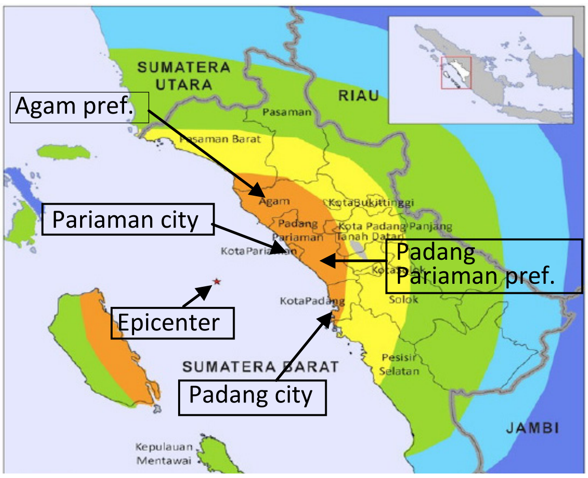

Figure 1.1. Area Map and Epicenter, Attached to OCHA Indonesia Earthquake, Situation Report No.16 (20 Oct. 2009)

The significant damage area extended over Padang city (Population about 840,000), Pariaman city (70,000), Padang Pariaman prefecture (380,000) and Agam prefecture (420,000). The damage features differed site-dependently. Pandang city, which is the capital of West Sumatra State, suffered significant damage to its many modern large-scale buildings. In the hilly area of Pariaman, the remarkable feature was severe damage to low-rise non-engineered residential houses.

In this paper, the authors discuss damage analysis of large scale reinforced concrete buildings. These buildings have important roles in supporting the backbone function of the capital city and the damage they suffered might indicate common deficiencies of the same kinds of buildings in many other cities.

2. GROUND SHAKING IN PADANG

Since the shaking during this earthquake was not recorded instrumentally in the downtown area of Padang, the authors estimated its intensity and predominant frequency through several approaches, namely, a questionnaire survey on the intensity of the shaking, a micro-tremor observation, a wave synthesis using a recorded wave on a rock site near Padang, and a monitoring video recorded at a building in Padang.

2.1. Questionnaire survey

The questionnaire survey, obtained from residents in the area, comprised 34 questions, and was developed to estimate the JMA (Japan Meteorological Agency) Intensity by Ota et al (1979). This method has been widely applied to many areas of many earthquakes since the 1970s and has been useful for estimating seismic intensities in areas where no seismometer was located. The authors converted JMA Intensity to MM Intensity.

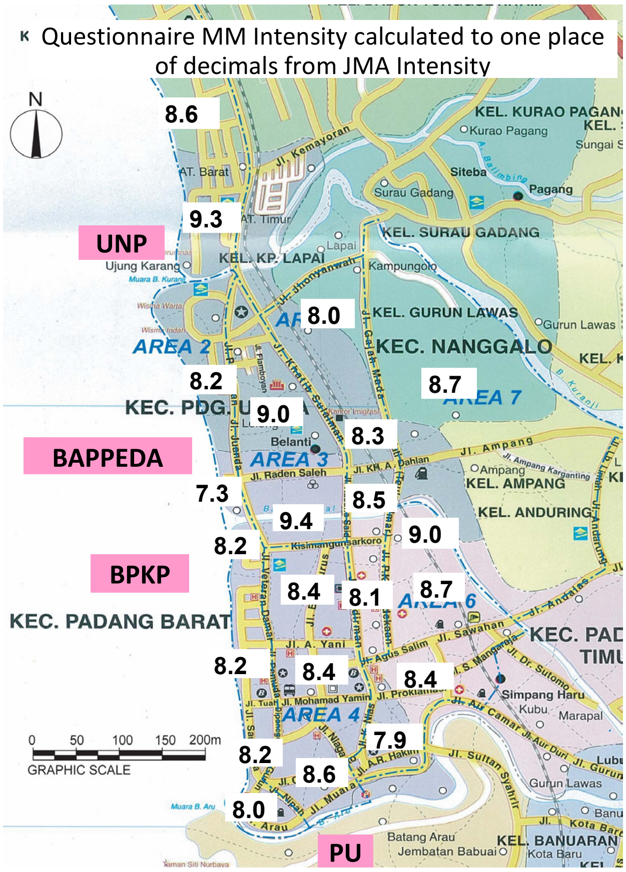

Figure 2.1. MMI from questionnaire survey

The original questionnaire sheet had, of course, been written in Japanese, but Honda et al had translated it to Bahasa Indonesia and used it for the residents of Banda Aceh in 2005. We modified it a little to make it more relevant to the life style of Padang. About 720 residents in the downtown area of Padang were interviewed by ten local students and the answers were analyzed following the method improved by Ota et al (1998) to fit high-intensity areas. The extracted MM Intensity as the average of all the answers was VIII-IX (5 upper in JMA Intensity) and the areal distribution is shown in Figure 2.1. The students explained each item of the questionnaire to residents, discussed the answers with them, and marked one of the answers listed on the sheet. Thus, the intensities were slanted according to the students who conducted the interviews. However, the areal deviation of the intensities was still helpful in interpreting the damage distribution in the downtown area of Padang.

2.2. Micro-tremor observation

We observed micro-tremors at typical sites in the downtown area of Padang as one way to estimate the seismic response feature of the ground, and analyzed the observed data by the H/V spectrum method (Nakamura method). Figure 2.2. shows the spectra. The rather long-period components of around one to two seconds are clearly predominant, and they seem to lengthen from south to north. There is a hilly area on the left bank of the river mouth of Mata Air Timur (Jiraku river), so the surface soil layer is assumed to be shallow in the southern area and to become thick in the north. This assumption is supported by the variation of the predominant period of micro-tremors from south to north.

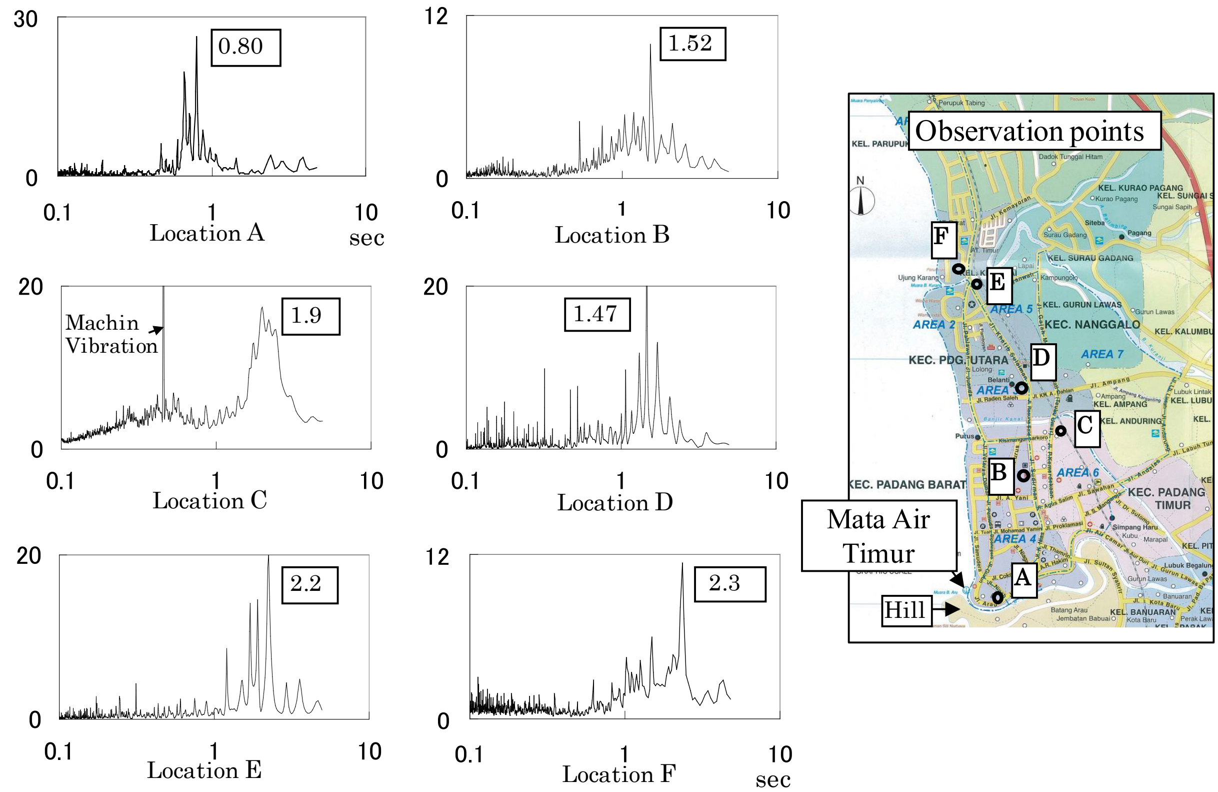

Figure 2.2. H/V spectra of micro-tremor in Padang city

2.3. Wave synthesizing

There was no instrumentally observed record of the shaking in the downtown area of Padang during the earthquake. However, BMKG recorded the shaking by a strong-motion seismograph placed on a rocky site in Andalas University, which is about 11km east of the downtown area of Padang. Additionally, an array observation using three seismographs located at the Andalas University site, a stiff soil site and a soft soil site of the downtown area was being operated by EWBJ (Engineers without Borders, Japan) and Andalas University. Although it did not record the September 30 earthquake, it did record several other rather small earthquakes. We used these earthquake records to synthesize a provisional shaking in the downtown area during the September 30, 2009 earthquake.

Figure 2.3. shows a conceptual diagram of the synthesizing process.

1) Extract a Fourier transfer function averaging 4 earthquake records recorded from April to December 2009, which were 0.25-1.25 kine at maximum (Figure 2.4.).
2) Compose an assumed soil column model fitting the transfer function. The depth and Vs of the soil layer were assumed using the data presented by Kiyono and Kubo of Kyoto Univ. The other profiles were determined by a trial and error method (Figure 2.5.).
3) Calculate a non-linear (equivalent linearized) response of the soil column model using the BMKG record as the incident wave. An improved SHAKE code with a frequency dependent damping was used.

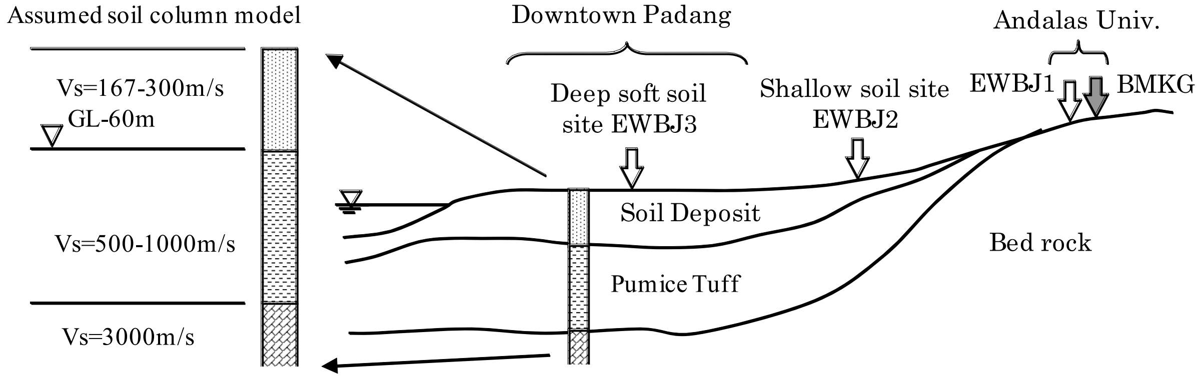

Figure 2.3. Conceptual diagram of synthesizing process

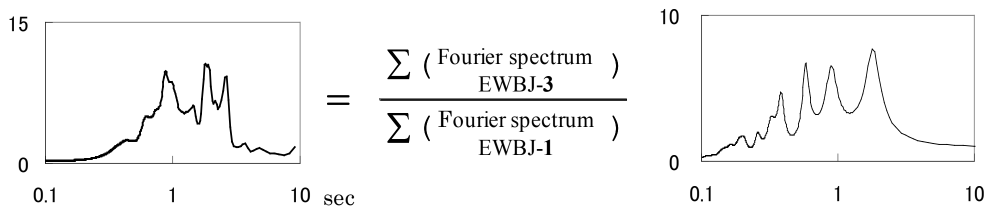

Figure 2.4. Average Fourier transfer function **Figure 2.5.** Synthesized Fourier transfer function

The synthesized wave and the response spectrum are shown in Figure 2.6.. Due to the effect of the soft soil layer, the short-period component of the incident wave is cut off and the peak acceleration is decreased. However, the long-period component is amplified and the relative response velocity at 1.8 seconds reaches 150 kine. The JMA instrumental seismic intensity is 5.3 and MMI is about VIII.

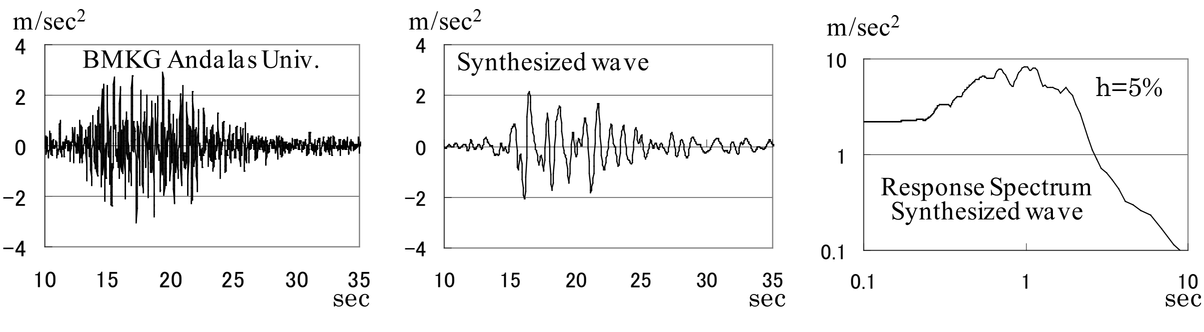

Figure 2.6. Incident wave, Synthesized wave and Response spectrum

2.3. Period of shaking recorded by a monitoring video camera

The Emergency Operation Center (EOC) of West Sumatra State is located in the downtown area of Padang. A monitoring video camera was located on the ceiling of the operation room and recorded the responses of chairs on casters and an unlocked door during the initial stage of the strong shaking.

Figure 2.7. Clips of the video records

By tracking the movement of the chairs and the door in the frames (Figure 2.7.), we could draw time histories of the movements, as shown in Figure 2.8. The chairs and the door moved at a one- to two-second period, which is consistent with the predominant period of the synthesized wave.

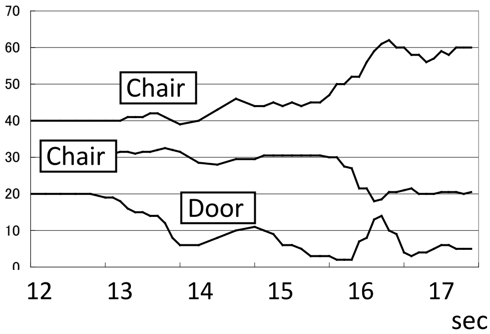

Figure 2.8. Time history of the movements

3. SURVEY ON DAMAGE TO LARGE SCALE BUILDING

3.1. Detail survey on a typically damaged building

The BPKP (Financial and Development Supervisory Board) building was located at the center of the downtown area (Figure 3.1.). It was severely damaged but remained almost upright. Construction of this building started in 2003. After the first construction stage, the first and the second floors were completed and ready for use. In 2006, all five floors were completed. In the September 2007 Sumatra earthquake (offing Bengkulu) the terracotta roof collapsed. That roof was then replaced by a lightweight thin-steel roof (Figure 3.2.).

During the on-site survey, we removed the cover materials near the top and bottom of all columns and evaluated the damage degree of each one using the seismic damage evaluation method developed by the Japan Building Disaster Prevention Association (1991). Table 3.1. shows the damage degree frequencies for each floor. Figures 3.3.-3.5. show typical damaged columns according to damage degree. The third-floor columns were severely damaged. This damage obviously resulted from the reduction of column cross-section on this floor.

Figure 3.1. BPKP building **Figure 3.2.** Lightweight roof

Figure 3.3. Damage degree III **Figure 3.4.** Damage degree IV **Figure 3.5.** Damage degree V

Table 3.1. Damage degree frequencies by floor

	1F	2F	3F	4F	5F
0	34	15	2	35	35
I	0	0	3	0	0
II	0	6	3	0	0
III	1	11	10	0	0
IV	0	2	10	0	0
V	0	1	7	0	0

We measured all the major building components, namely, the columns, the floor heights, the beams, the plate thicknesses, and the reinforcing bars. The main bars were from φ19×16 to φ17×12 and the stirrups were φ10 spaced at 120mm to 150mm. The concrete strength of representative portions was measured with a Schmidt Hammer, and the steel bar strength was measured with a Vickers Hardness Tester. The micro-tremor on the building was also measured. The Fourier spectra of the tremor are shown in Figure 3.6. The sway and the torsion vibration periods of the building are clearly observed.

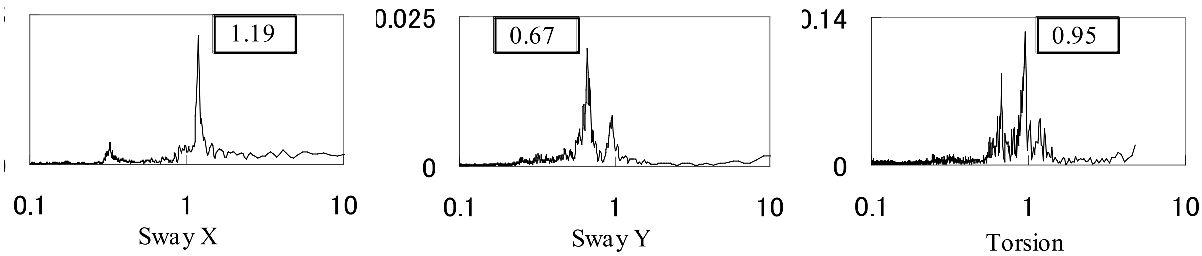

Figure 3.6. Time history of the movements

3.2. Numerical analysis on the damaged building

Using these measured dimensions, a lumped mass frame model was developed. The weights of the inner and perimeter walls of the building were included in the floor plate lumped mass. The stiffness of the columns and beams were assumed to be 100% of the original elastic-range value. The frame model was analyzed by a versatile software system for structural analyses (SAP2000), and the natural periods of free vibration and the response to a design earthquake load were evaluated. Figure 3.7. shows the analyzed modes and the natural periods. The periods in the horizontal X direction and in torsion coincided with those of the micro-tremor. However, the period in the horizontal Y direction was different. The model did not take into account the remaining stiffness of the brick walls and the decreasing stiffness of the damaged columns. These two factors must have compensated in the horizontal X direction and in torsion. The damage was considerably smaller in the horizontal Y direction than in the horizontal X direction, thus suggesting that the wall stiffness in the Y direction might not have decreased much.

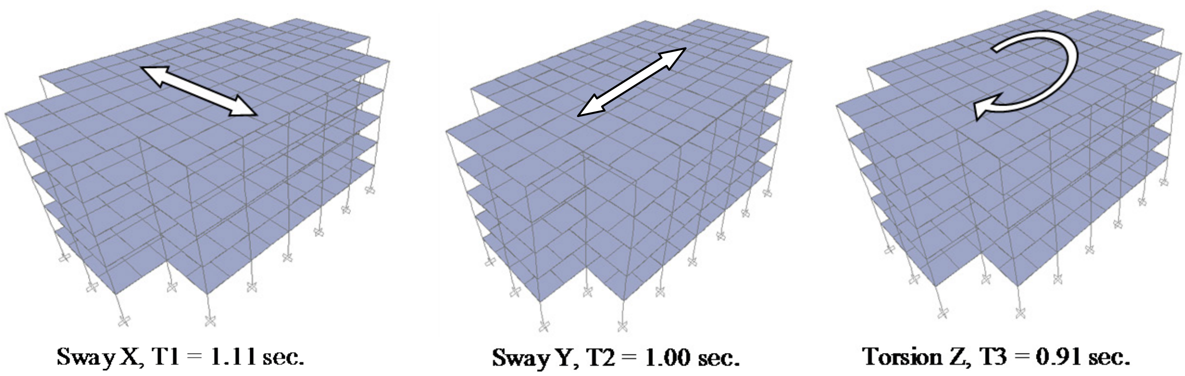

Figure 3.7. Frame model and analyzed natural vibration

Using the concrete and steel bar properties obtained from the survey, the axial force and moment capacity diagram of the columns at all floor levels were calculated, as shown in Figure 3.8.

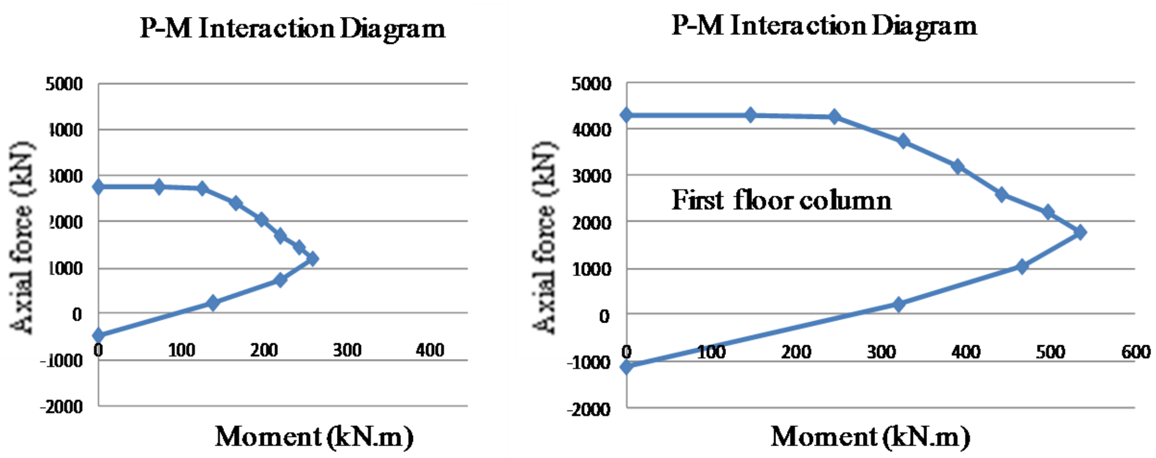

Figure 3.8. Axial force and moment capacity interaction curves at typical columns

To compare these capacities with the actual stress levels in the columns, the equivalent static seismic loading method was applied based on SNI-03-1726-2002 (recent seismic code used in Indonesia). The dead weight was assumed to include the weights of the structural members (columns, beams, and plates) and perimeter walls, and live load (20N/mm2 for office). The equivalent static seismic coefficient was also assumed based on the SNI-03-1726-2002.

Thus, the nominal static equivalent base shear force V is:

$$V = \frac{C_1 I}{R} W_t \qquad (1)$$

where C_1 is obtained from Figure 18 using first natural period T_1, and W_t is total building weight, including an appropriate live load.

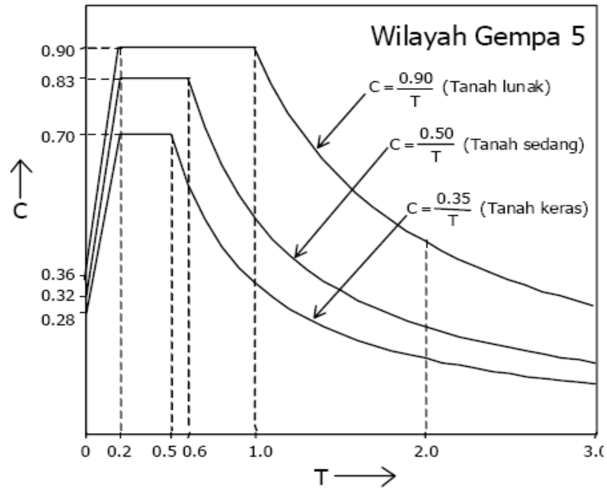

Figure 3.9. C1 curves by SNI-03-1726-2002

In the BPKP case, T_1 is 1.11 seconds. Therefore, based on the figure and soft soil (Tanah lunak) condition, C_1 is $0.9/T_1 = 0.9/1.11 = 0.811$. I is importance factor, taken as 1.0 for office buildings. R is seismic reduction factor. For a normal moment resisting frame, R is taken as 3.5. Therefore, V becomes 0.232. The weight is quantified as W_t = 39,005 kN following the assumption mentioned above. Thus, the base shear force is V = 0.232×x 39005= 9,036 kN. The base shear V must be distributed along the height of the structure, i.e.,

$$F_i = \frac{W_i \, z_i}{\sum_{i=1}^{n} W_i \, z_i} V \qquad (2)$$

where F_i is the load acting on the mass center at floor -i, W_i is weight of floor -i, Z_i is height of floor -i, and n is the total number of stories.

Table 3.2. Seismic force distribution

Floor Name	Height (m)	Weight (kN)	Force per floor (kN)
Floor-4	20	4847	3012
Floor-3	16	8540	2410
Floor-2	12	8540	1807
Floor-1	8	8540	1205
Floor-G	4	8540	602

Table 3.3. Demand moment vs. Capacity moment

Floor Name	Axial force (kN)	Demand moment (kNm)	Capacity moment (kNm)
Floor-3	865	504	230
Floor-1	1525	792	515

Based on three-dimensional frame analyses, the axial force and the demand moment at a typical column of the 1st and 3rd floor are shown together with the moment capacity in Table 3.3. The demand moment exceeds the capacity, so the column could collapse if the actual seismic force reached the level of the design seismic load denoted by SNI-03-1726-2002.

Figure 3.10. compares the C_1 curve to the synthesized wave response spectrum. The C_1 curve shifted to the capacity moment level of the third floor column is also shown. It is clear that the seismic force acting on the BPKP building during the September 30, 2009 earthquake far exceeded its capacity and reached the level of the design seismic load denoted by SNI-03-1726-2002. According to the design document of the BPKP building, it was designed based on the previous design code of the SNI-03-1726-2002. The design calculation was based on the seismic coefficient method and the design seismic coefficient was 0.07. In spite of the very small design load, the advantage of a moment frame structure might have contributed to avoiding collapse. If the reinforcing bar arrangement, especially the hoops, had been adequate, the columns might not have been so severely damaged. Unfortunately, this was not so.

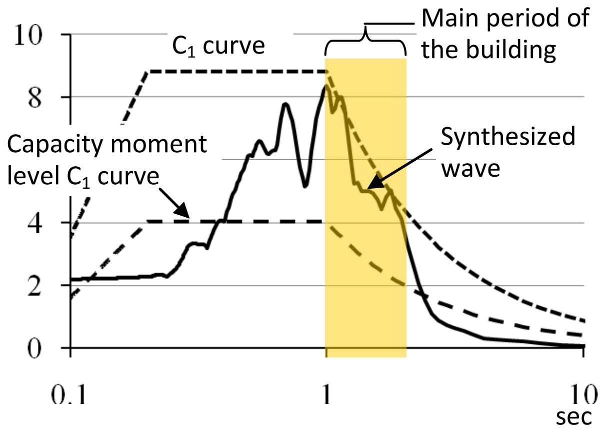

Figure 3.10. Comparison of C1 curve to the synthetic response spectrum

4. CONCLUSIONS

(1) From the questionnaire survey, the MM intensity in the downtown area of Padang was estimated to be XIII – IX (5-upper in JMA intensity) on average.
(2) The micro-tremor H/V spectra predominated at the rather long period of 1.0-2.0 seconds in the downtown area of Padang.
(3) The synthesized wave using the EWBJ array observation and the BMKG record was also MMI XIII level of the intensity and had a predominant period in 0.5-2.0 seconds. Its response spectrum nearly reached the design spectrum level denoted by SNI-03-1726-2002.
(4) One of the damaged large scale buildings was designed by the seismic coefficient method using a design seismic coefficient 0.07. Although the demand moment at the columns exceeded the capacity moment and the columns were heavily damaged, they did not collapse. If the reinforcing, especially the hoops, had been arranged adequately, the columns might not have been so severely damaged.

AKCNOWLEDGEMENT

The data used in this paper was obtained during field surveys by the authors under "The Preparatory Study on Disaster Management Program for Indonesia" implemented by JICA, 2009. The authors would like to express their sincere gratitude to JICA. They also express their sincere gratitude to Professor Yoshiaki NAKANO of the University of Tokyo, and to Professor Junji KIYONO of Kyoto University for their kind discussions and advice. Finally, the authors would like to acknowledge the contributions of EWBJ and BMKG for their efforts in operating the strong earthquake motion recording system.

REFERENCES

Ohta Y., Goto N. & Ohashi H. (1979). "Estimation of Seismic Intensity during Earthquake by Questionnaire Method." Research Report Of Engineering Department Of Hokkaido University, No. 92, 117-128

Ohta Y., Koyama M. & Nakagawa K. (1998). "Revision of algorithm for seismic intensity determination by questionnaire survey –in high intensity range-" *J. JSNDS* **16-4**, 304-323

Honda R., Takahashi Y., Pradono M. H. & Kurniawan R. (2005) "2004 Off Sumatra Earthquake - Questionnaire Seismic Intensity Survey in Banda Aceh" *Proc. 28th JSCE Earthquake Engineering Symposium*, 2005