CHAPTER 3 LARGE-SCALE RC BUILDING DAMAGE SURVEY

3.1 Outline

The purpose of this survey is to determine the extent of damage to the RC building facilities brought about by the Earthquake. The primary objective is to determine the causes of the failure of the structures so that remedial measures could be provided to avert the occurrence of similar tragedy in the future. Listed hereunder is the specific component of the survey/study.

1. Determining the conditions of the damaged building facilities.
2. Review and recommend risk assessment for damaged building facilities to be conducted by Indonesian Group.
3. Examination and recommendation for the method of retrofitting damaged building structures.
4. Recommendations of seismic forces resisting criteria to be adopted for the design of building structures.
5. Collation of the study results with those conducted by Australia-Indonesia Facility for Disaster Reduction.

It is noted in this connection that the study will focus on the buildings of major RC moment resisting frames considering that these facilities were designed by structural engineers to ensure its stability.

3.2 Features of Major Damage to RC Building Structures

3.2.1 Seismic Intensity

Immediately after the earthquake, news circulated about the damage to major building structures designed as moment resisting RC frames. The news was quite alarming in comparison with the destruction of residential houses constructed as conventional building type such as confined masonry or un-reinforced masonry. Based on rational judgment from the point of view of Japanese structural engineering, the seismic forces could have been overwhelming considering that the design analysis of the RC moment resisting frames was undertaken based on modern techniques with the use of computers. The report therefore was felt unexpectedly.

For this reason, the causes why major RC building structures collapse should be given emphasis to avert the occurrence of the same incident in the future. The first question would be the seismic intensity that caused the heavy damage. This would have been known if measurements were taken by the seismometer device that was installed in Padang City. Consequently, the survey cannot commence without the benefit of any reliable measurement. On the basis of the foregoing, the seismic intensity was estimated pursuant to the 35 items of questionnaires suggested by Dr. Ota1 and this was used in 560 locations over Padang city. The 35 items of questionnaires are listed hereunder.

Q1	Have you notice the ground motion of the earthquake?
Q2	Where were you at that time? (In the house or outside of the house)
Q3	What were you doing there?
Q4	Where were you when earthquake occurred? (Specify on the map.)
Q5	Describe the place where you were at the time of the earthquake?
Q6	Describe the ground of that place.
Q7	Describe the house.
Q8	How high/tall is the house?
Q9	On which floor were you at the time of the earthquake?
Q10	When was the house built?
Q11	Were hanging objects swinging? (e.g. lamp hanging from the ceiling, calendar on the wall, paintings.)
Q12	Did water in the vase, bowl, aquarium, etc move?
Q13	Did tableware, window, door, etc. rattle?
Q14	Did unstable decors such as vases, bottles etc. move?
Q15	Did heavy furniture, such as bookshelves, tables and chests, move?
Q16	Did the house shake?
Q17	Was the house damaged?
Q18	How long do you think was the ground motion?
Q19	What was the strongest ground motion?
Q20	Were you surprised by the earthquake?
Q21	How scary was the earthquake?
Q22	What did you do at the moment of the earthquake?
Q23	What precautions did you take against fire occurrence for kitchen appliance such as extinguishing of kerosene stove, gas stove, and electric stove?
Q24	Were you awaken because of the earthquake?
Q25	Did you have difficulty moving around during the earthquake?
Q26	Were trees or cars swaying because of the ground motion?
Q27	Did you experience difficulties in driving your car during the earthquake?
Q28	Did you feel swaying when your car was parked?
Q29	How many people in your surroundings excluding yourself felt the ground motion?
Q30	Were wooden/brick/concrete hollow block fences, chimneys etc. near you damage?
Q31	Were houses seriously damaged?, Were there fissures (crack) on the ground?, Did pavement cracked? Did erosions or landslides occur, Were the roads damaged?,etc.?
Q32	Did the water and electrical supply system in the vicinity of your area stopped?
Q33	How old are you?
Q34	What is your sex?
Q35	Could you give us your name, address and phone number? (Not compulsory.)

Based on the results of the inquiries, the distribution of estimated seismic intensity over Padang city is shown in Figure 3.2.1. It is informed in this connection that the Japanese Meteorological Agency (JMA) is applying the same method for estimating intensity and scale/extent of an earthquake in Japan. Based on the Modified Mercalli Intensity (MMI), transformed to Trifunac and Brady (1975) Method through graph, the Peak Surface Ground Acceleration (PGA) was

[1] Hiroshi Ota, Noritoshi Goto, Hitomi Ohashi: A Questionnaire Survey for Estimating Seismic Intensities, Bulletin of the Faculty of Engineering, Hokkaido University, Vol.92, pp.117-128, 31-Jan.1979

obtained. For the whole of Padang city, the intensity based on the JMA method is estimated at more than 5 while the intensity based on the MMI method is estimated at more than 8.

Figure 3.2.1 Distribution of Seismic Intensity over Padang city (Estimated through Replies of Questionnaire Surveys)

Based on the results of the survey, the extent of damage other than Padang City is not known.

Figure 3.22 shows the spots surveyed in Pariaman. As shown, the average of the intensities (I_{JMA}) based on the result of the replies to the questionnaire surveys for the north-east area of Pariaman was estimated at 6 (MMI: 9.5).

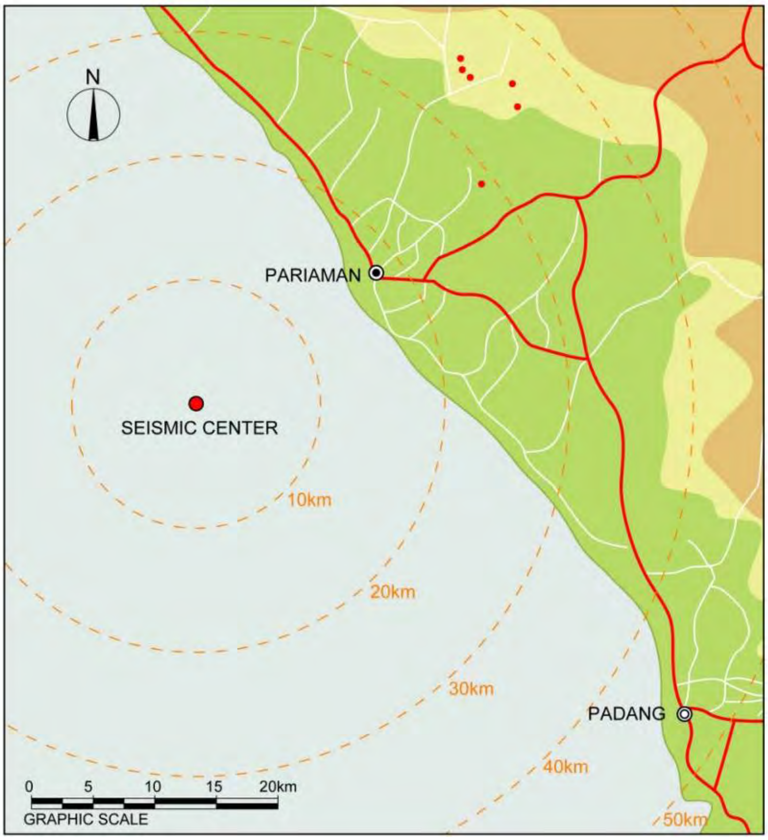


Figure 3.2.2 Survey Points in Pariaman

Questionnaire Survey in Padang (to BPKP staff)

Questionnaire Survey in Pariaman

【Check References from EERI Special Earthquake Report】

There was only one strong ground motion record provided by "EERI Special Earthquake Report dated December 2009" (BMKG/USGS 2009). The time history of ground acceleration is shown in Figure 3.2.3 and the acceleration response spectrum is shown in Figure 3.2.4. It was reported that the instrument site was located at the base of the mountains, about 12 km in from the coast and on stiff soil. Therefore the ground motions in the center of Padang, on softer deeper soil deposits, are likely to have been larger.

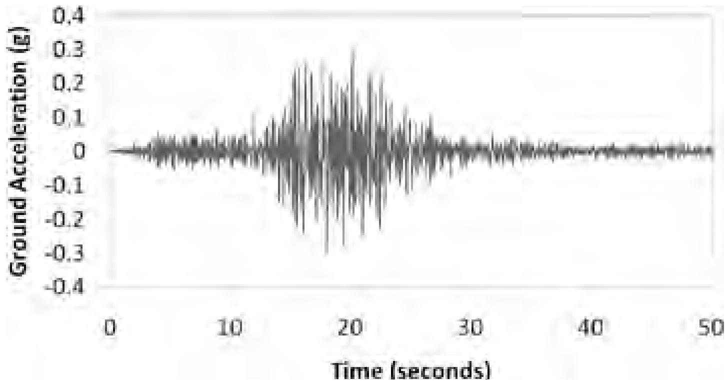


Figure 3.2.3 Strong Ground Motion Record (from EERI Special Earthquake Report)

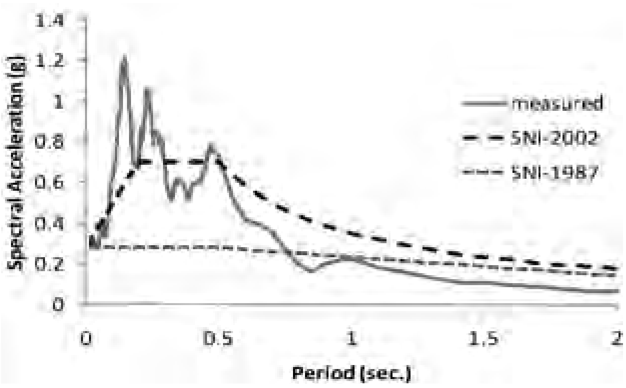


Figure 3.2.4 Acceleration Response Spectrum (from EERI Special Earthquake Report)

3.2.2 Characteristic of Building Damage

The second question is why a building structure should be damaged by a nominal intensity earthquake when it was designed as rigid RC moment resisting frames. In connection thereto, the table hereunder shows the survey carried out to assess the damage of building structures, for purposes of focusing on this point.

No	Items	① BPKP	② PU	③ UNP	④ BAPPEDA	⑤ DEP. KEUANGAN	⑥ DPRD
1	Measurement of the member section (columns, beams and walls)	○	○	○	○	○	○
2	Measurement of Re-bar arrangement (main bar, hoop bar, etc.)	○	○	—	○	○	○
3	Strength test of material (Re-bar and Concrete)	○	○	○	○	○	○
4	Measurement of transformation	○	○	—	—	—	—
5	Evaluation of damage level (columns, beams and walls)	○	—	—	—	—	—
6	Microtremor measurement	○	○	○	—	—	—

*1) BPKP: State Finance and Development Surveillance Committee

*2) PU: Public Work State Office

*3) UNP: University of Padang State

*4) BAPPEDA; Regional Body for Planning and Development

*5) DEP. KEUANGAN: Local Finance Bureau Padang District Third Office

*6) DPRD: State Parliament House

1. Measurement of structural member sections

2. Measurement of Re-bar arrangements

3. Strength test of materials	4. Measurement of transformations
	
5. Assessment of the level of damage structural members	6. Microtremor measurement
	

1) General View

All damages of major building structures were investigated as primary part of the report considering that the facilities were designed as RC moment resisting frames and is particularly more serious than the destruction of common residential houses constructed only of unreinforced masonry materials. The damage of major RC building structures is comparatively much bigger based on the premise that the structure is more reliable because as mentioned above it was constructed of RC moment resisting frames for which structural analysis was conducted. The RC moment resisting frames is illustrated in the presentation documents of the Australia-Indonesia Facility for Disaster Reduction. The damage of RC moment resisting frames and residential houses built with unreinforced masonry were both investigated. However, comparison of damages between the 2 types of construction was not made other than the explanatory notes indicated in the survey results.

2) Analysis of Surveyed Damages to Building -1 (BPKP Building)

Figure 3.2.5 Exterior View of BPKP Building

The detailed investigation of BPKP building was conducted based on the request from PU. As a backgrounder, construction of the building was completed in 2003, but was partially damaged by Bengkulu earthquake in 2007. However, the damage of the structures at that time was minimal and only the brick walls of the second floor and the fourth floor were damage and repaired for use thereafter.

The second floor as shown in Figure 3.2.6 incurred the heaviest damage.

In addition, evaluation results show that the degree of damage was distributed to the columns for each floor as shown in Table 3.2.1. The proportion of Damage for structures on the 2nd floor is summarized in Table 3.2.2.

This assessment followed the method for the "Report on Technical Cooperation for Temporary Restoration of Damaged RC School Buildings due to the 1999 Chi-Chi Earthquake." [2] The document however used a modified method[3] considering that the original one previously applied to buildings constructed mostly of brick walls. This application is based on earthquake-resistant method analysis for existing RC building structures in Japan.

[2] Architectural Institute of Japan: Report on the Technical Cooperation for Temporary Restoration of Damaged RC School Buildings due to the 1999 Chi-Chi Earthquake

[3] The Japan Building Disaster Prevention Association: Standard on Earthquake-Resistant diagnosis for existing RC Building

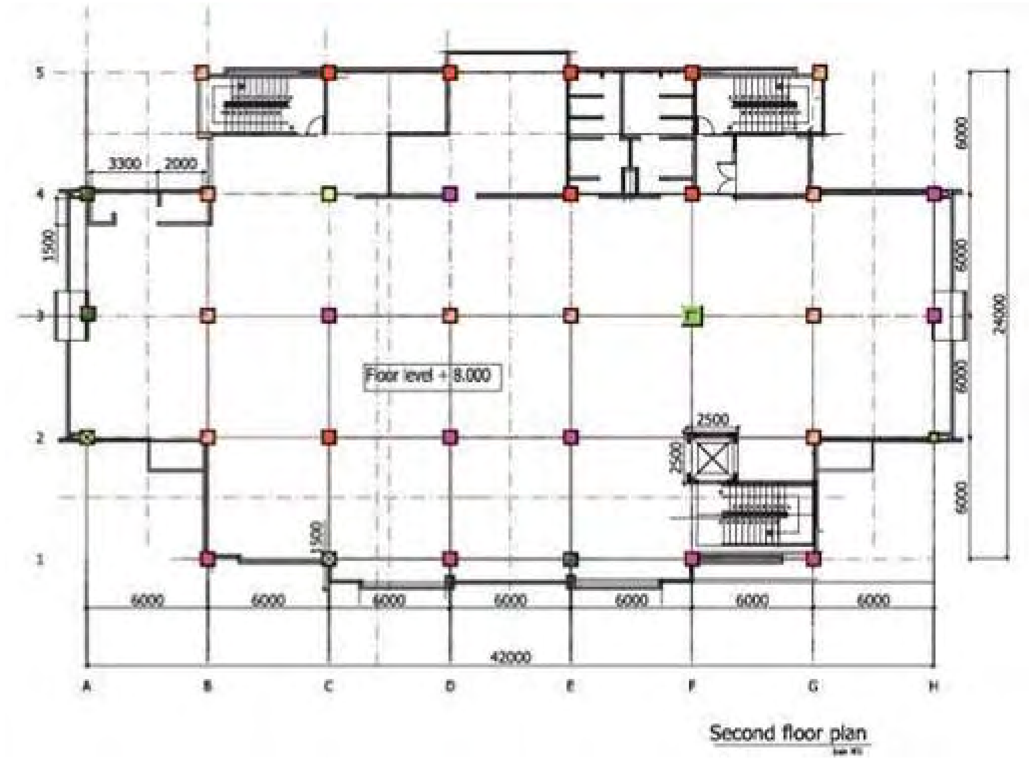


Figure 3.2.6 BPKP Second Floor Plan

Table 3.2.1 Evaluation Result of the Degree of Damage Distribution on Columns for Each Floor

Damage Level	GF	1 F	2 F	3 F	4 F
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

Table 3.2.2
Damage Ratio of the Structures on the 2nd Floor

Damage Level	No. of column (Bi)	Bi/A	Di
0	2	0.057	-
I	3	0.086	0.857
II	3	0.086	2.229
III	10	0.286	17.143
IV	10	0.286	28.571
V	7	0.200	28.571
Total	35(=A)	1.000	77.371

Table 3.2.3
Criteria to Estimate the Degree of Damage

No.	Damage Ratio	
1	No Damage	(D=0)
2	Slight Damage	(D≦ 5)
3	Small Damage	(5＜D≦10)
4	Medium Damage	(10＜D≦50)
5	**Severe Damage**	**(50＜D)**
6	Collapse	(D5=50)

Based on the result of the damage assessment as shown in Table 3.2.2, the second floor incurred the heaviest at an estimated level of 77.371. The result shows that when the value of Di damage level exceeds 50, it is considered as “Severe” as shown in Table 3.2.3.Therefore, the 2F floor of the building is considered to be Severely Damaged because it exceeded the 50 factor.

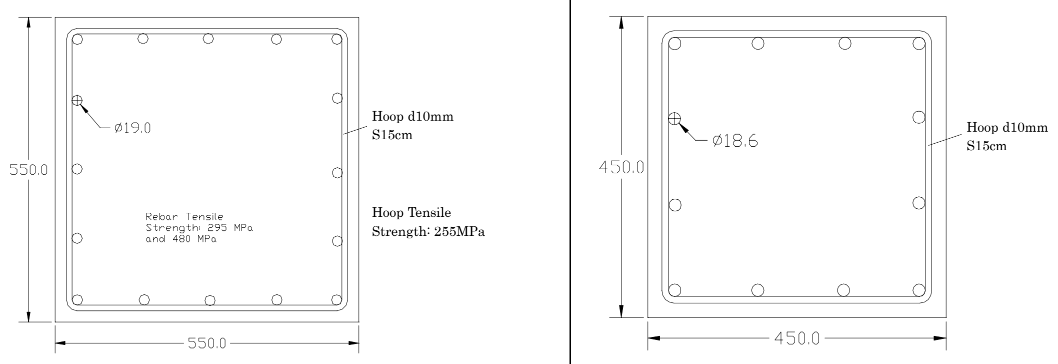
The most intense destruction of the structure is buckling of the top column of the 2F floor of the northwest side of the building as shown in Figure 3.2.7.

Figure 3.2.7 Buckling of the Top of the Column

As shown in the photograph, buckling occurred due to shear intensity or bending stress that occurred at the top of the column when the earthquake struck, causing the sagging of the floor and shortening the height of the column by 5~6cm. Because the hoop spacing is more or less 12cm of 10mm diameter rebars, reinforcing steel bars apparently is lacking to resist shear stresses.

Survey result of the whole building showed that the structure has collapsed and wall of the buildings which is made of brick materials was originally designed with low stress because the structural calculations neglected seismic forces, based on the concept that the rigid RC frames comprising of the columns and beams are designed to withstand seismic forces thereby stabilizing the walls. The brick wall collapsed due to inertial force acting in the right angle direction when the earthquake struck. Because the structure was not designed to resist seismic activity, it was not resistant to seismic stress. It is also considered that the column of the building has exceeded its allowable buckling point caused by seismic stress exceeding the design strength of the coulomb section. More discussions on this topic follow hereafter.

The reason for the destruction of the second floor is due to compression of the column section as shown in Figure 3.2.8 It is noted in this connection that the section of the column was not adequately designed to resist dead loads of the second floor combined with seismic load..



GF-E-5 Column	2F-E-5 Column

Figure 3.2.8 Column Section

In addition, the lack of materials strength is difficult to consider other than the inadequacy of steel bars as pointed out earlier, taking into account that based on the results of the Vickers hardness measurement and the Schmidt hammer measurement as shown in Figure 3.2.9 concreting appears to have passed specification requirements. Nevertheless, casting of e concrete for the connection of the column and the beam is difficult, and poor construction appears to be the most likely caused of failure because that segment of the structure requires high structural resistance/strength.

Concrete Compressive Strength	Re-bar Tensile Strength
33N/mm	340Mpa
	

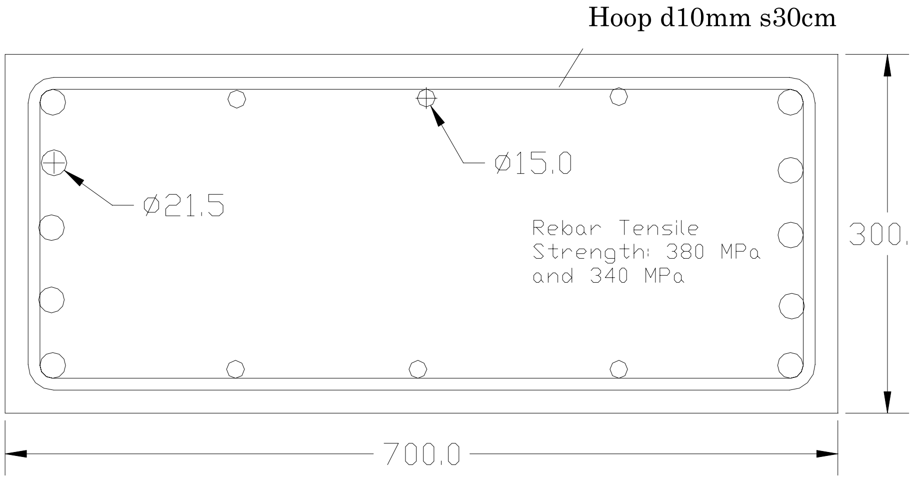
Figure 3.2.9 Field Materials Strength Testing by Nondestructive Testing Method

For irregular shaped buildings, it is said that torsion occurs when vibration takes place causing damages to structural members. However, the building is almost symmetrical, with virtually slight irregularity. The damage incurred to the column on this segment of the building was severe because of the presence of an elevator shaft at the right side, which was covered with brick wall thereby causing the instability coupled with the heavy concentrated loadings from the vicinity of the restrooms.

3) Analysis of Surveyed Damages to Building -2 (PU Building)

Figure 3.2.10 Exterior Appearance of PU Building

This building was completed in 1970. As can be seen, the entire first floor incurred severe damage and is almost in the state of collapsed and therefore is no longer usable because of its detrimental condition. All the column were distorted as shown in Figure 3.2.11 and the inadequacy of hardness in this direction appears to be overwhelmingly large considering the low resistance to secondary moment at the crossbeam area direction. Therefore, the building is about to collapse like a domino as shown in Figure 3.2.12.



1.

Figure 3.2.11 1st Floor Column Section

Figure 3.2.12 State of Transformation(Structure is inclined at 4 degrees from vertical direction)

Due to the higher height of the second floor, more walls were provided than the first floor, and as a result hardness has risen, and the resultant is a reversal of layer hardness. Due to this development, the first floor became a soft layer of the building segment and with the inertial force , the concentrated between the two layers, had caused the transformation.

4) Analysis of Surveyed Damages to Building -3 (UNP Building)

Figure 3.2.13 Outdoor View of UNP Building

Figure 3.2.14 Collapsed Brick Wall of the rest room for females at the fifth floor

Construction of this building was completed in 1998, as shown in Figure 3.2.13. The design of the exterior wall was modified to aluminum panel after the original brick wall broke down by the earthquake that occurred in 2007. The primary structural components of the building, however, were totally undamaged (Damage Level at 0). However, the tremors that occurred in the upper floors of the building, had displaced the exterior brick wall due to inertial force acting towards the right angle direction, had distorted the exterior appearance/ aesthetics of the building. .

In a sense, the horizontal direction inertial force of the earthquake caused the disintegration of the brick wall but could have prevented damage to the structural frames of the building. This phenomenon appears to be a contributory factor to quake resistance of buildings but falling of debris is hazardous to pedestrians below.

5) **Analysis of Surveyed Damages to Building -4 (BAPPEDA-MAIN Building)**

Figure 3.2.15 Outdoor View of BAPPEDA MAIN Building

The first floor of this 3 stories high building completely had collapsed. The brick wall of the first floor with the top not connected to the upper structure had collapsed inwards as shown in Figure 3.2.15 above. As a result, most of the brick walls of the first floor did not contribute to the hardness of the structure. On the one hand, the hardness of the floors on the 2nd and 3rd floors rose because of the presence of RC walls. Consequently, the first floor became a Soft segment. It is noted in this connection that the inertial force acting on irregular shaped buildings of this type, particularly at the transformation area of the center of the layers between the soft segment and the hard segment has tendencies to amplify thus facilitating the collapsed.

Figure 3.2.16 Photo Showing the Collapsed Brick Wall of the first floor

6) Analysis of Surveyed Damages to Building -5 (BAPPEDA-SUB Building)

Figure 3.2.17 Outdoor View of BAPPEDA-SUB Building

The building was separated into three segments along the expansion joints. The column broke down due to the destruction of the top of the sixth column from the left side. The brick walls which did not contribute in restraining the transformation between layers, separated along the joints.

7) Analysis of Surveyed Damages to Building -6 (DEPARTMEN KEUANGAN R.I Building)

Figure 3.2.18 Outdoor View of DEPARTMEN KEUANGAN R.I. Building

Destruction of the left side ridge is particularly intense in addition to the destruction incurred to the right side ridge as shown in the photos hereunder. The building is irregularly shaped, but considering the segments to have been separated by expansion joints, it is difficult to imagine the cause to be due to asymmetrical form. Rather, the inadequate sectional dimensions, and insufficiency of Re-bars coupled with deficient Re-bar arrangement particularly at the column / beam joint could most likely be the causes of the destruction as shown in Figure 3.2.19, and Figure 3.2.20 hereunder.

Figure 3.2.19 Inadequate Column Size and Inadequacy of Re-bars

Figure 3.2.20 Deficient Re-bar Arrangement at the Column / Beam Joint

Figure 3.2.19 shows the bent column at the bottom end of the structure resulting from inadequacy of column size coupled with the insufficient quantity of re-bars. The lack of hoop steel rods of the column is quite remarkable for this building and it is difficult to distinguish if the destruction was caused by shearing or from ordinary causes.

Figure 3.2.20 shows the deficiencies of the column / beam joint. As shown, axial re-bars were excessively placed at the small column section. The re-bars to resist axial forces have also been extended upto the bottom of the floor slab of the 1st floor even surpassing the beam, but the shear re-bars which should be placed in position with ties is not absent. Moreover, the hoop bars placed for the column are short.

By comparison, the nearby mosque surrounded by office buildings did not sustain any damage at all. Based on this occurrence, the seismic intensity in the area could have not been the direct major cause of the collapsed of the building, but the dominating factor is rather to deficiencies caused by poor design and construction.

Figure 3.2.21 Exterior View of the Mosque

8) Analysis of Surveyed Damages to Building -7 (DPRD Building)

Figure 3.2.22 Outdoor View of the DPRD Building

This building was completed in 1995 but was partly damaged by Bengkulu Earthquake of in 2007. The building was in used while the repairs were being conducted. Consequently, the building was devastated by the occurrence of the last earthquake. The destruction of structural members of the building was partially limited. The damage large visual damage is the southeast side brick wall in front of the assembly hall which was blown out by an inertial force. The remarkable destructions to the structural components is the level IV damage of the columns supporting the roofs at the right side of the entrance hall as shown in Figure 3.2.23. Moreover, destruction to other columns is slight. This building appears to have been elaborately constructed with good quality materials and based on the results of field testing concrete strength is 33MPa, while tensile strength of re-bars is 400 MPa.

Figure 3.2.23 Level IV Damage to the Columns

The caused level IV damage is due to the provision of "Short columns" without supports at the sides. Moreover, the other columns with crossbeams are provided only with brick walls.

As shown, only the columns of this side of the structure are short supported with crossbeams underneath. For the large span of the 2 story hall of the entrance, the horizontal strength was distributed the column was quite enormous.

In addition to the six items of observations carried out for the above, 3 other RC Hotel buildings sustained major damages. The state of damages based on survey results are summarized hereunder.

ANBACANG Hotel	
	demolition has already begun at the time of the survey. Based on the condition of the damaged components, it is apparent that proper re-bar arrangement was not made. As can be seen, the columns also appeared to have constructed of bricks, with small RC columns.

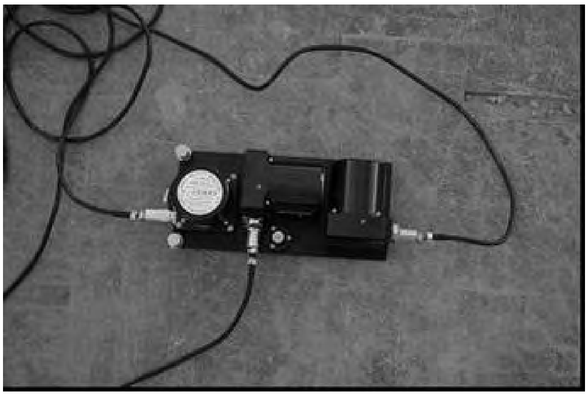
Hotel BUMIMINANG	
	As shown, the columns and the walls made of bricks sustained heavy damages but those constructed with RC frames appears to have almost not incurred damages.
INNA MUARA Hotel	
	As shown, damage to the adjoining buildings varied greatly. As can be seen, the size of the column and the beam are quite small. Re-bar arrangement appears deficient for the damage building.

3.2.3 Shake Characteristic of Buildings and Ground

(1) Buildings

The micro tremor measurement was carried out using a portable servo type acceleration meter as shown in Figure 3.2.24 The micro tremor measurement for building structures, shaken by unspecified excitation, were studied based on frequency analysis.

Adequacy of building model for dynamic analysis can be examined by comparing the fundamental natural period of the model with the predominant period obtained by frequency analysis.



a) Sensor

b) Amp

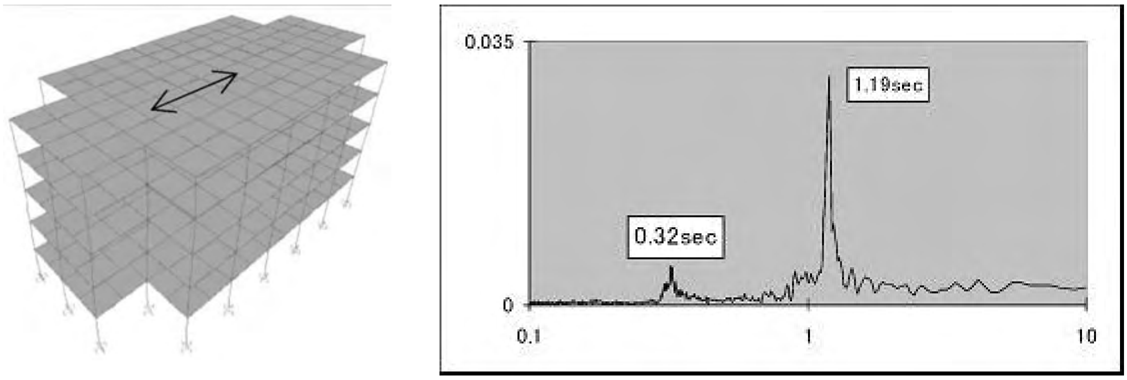
Figure 3.2.24 Instrument and Sensor

【Stiffness of Masonry Wall】

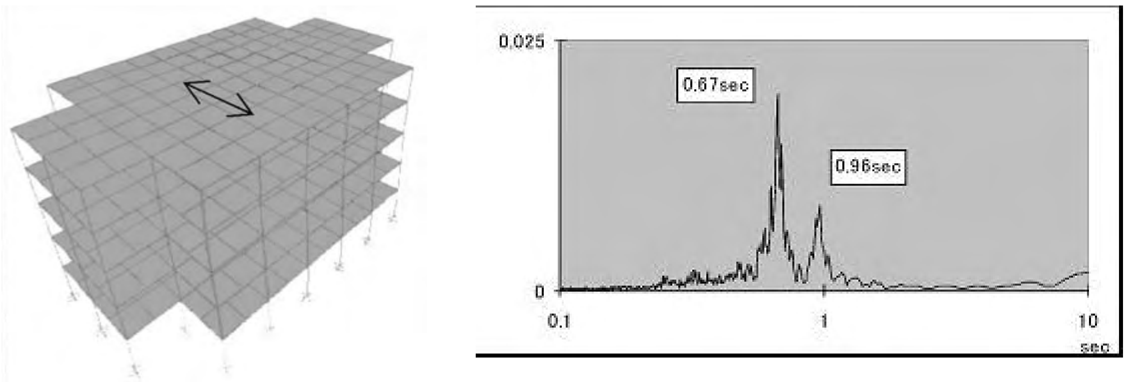
The masonry wall stiffness in RC buildings in Indonesia is usually neglected in the dynamic analysis for the design although the mass of the walls are taken into account. If the micro tremor of partially damaged building is analyzed the result of the findings can be effective in examining the validity of the above mentioned design assumptions.

The vibration characteristic of the BPKP building is shown in Figure 3.2.25 for purposes of this examination.

The predominant period of the tremor is about 1 second. This is considerably longer than the generally recognized value. Based on Japanese structural engineering, the fundamental natural period for 5-storey high building of *RC Moment Resisting Frame with RC Shear wall* could be assumed at about 0.5 second.



a) Longitudinal Direction



b) Latitudinal Direction

Figure 3.2.25 Vibration Characteristic in BPKP Building (five stories)

The measurement of vibration characteristic of undamaged 3-story high building in Aceh for reference purposes is shown in Figure 3.2.26. As shown, the predominant period of the graph is about 0.3 second. Therefore, the fundamental natural period for 3-story high building of *RC Moment Resisting Frame with Masonry wall* may be assumed at about 0.3 second. Presumed value based on structural engineering and the predominant period of Figure 3.2.26 appear to match well. Since the wall in this building did not cracked, the stiffness contributed to the rigidity of the structure.

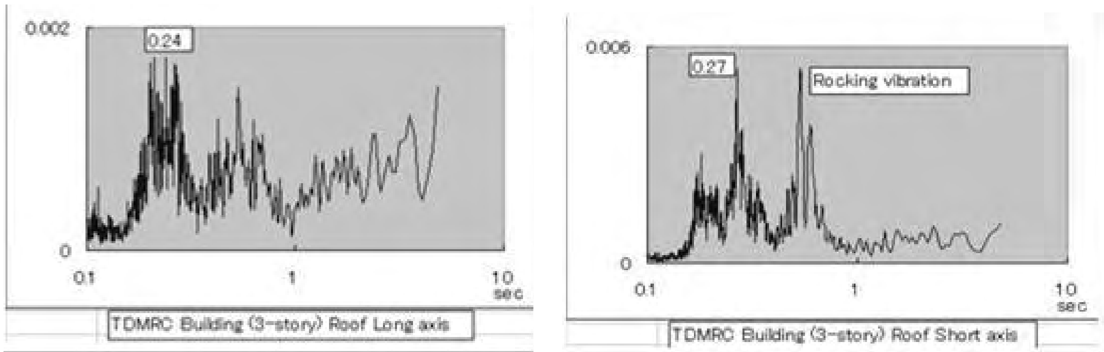


Figure 3.2.26 Vibration Characteristic in no Damage Building (3 stories)

The concept that masonry wall stiffness will not function as means of resisting lateral forces can concluded as follows.

- The wall will maintain rigidity until cracking occurs even if it is made of brick.
- When the wall is partially damaged, the rigidity will decrease remarkably, and the shearing resistance will diminish.

【Regarding the rigidity of RC flat column】

The geometrical moment of inertia of the columns in the PU building has a big difference depending on the direction because it is very flat as shown in Figure 3.2.27

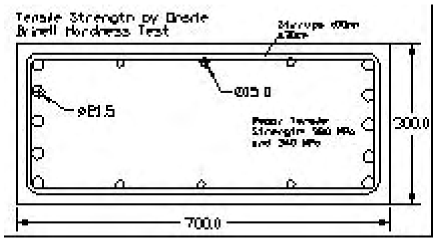
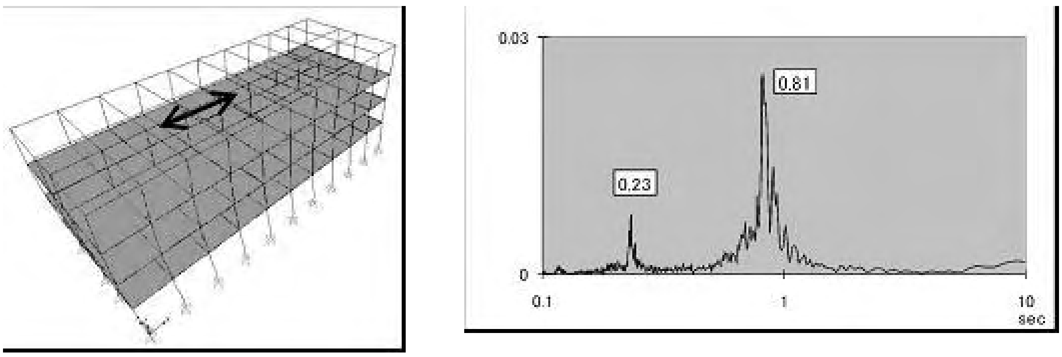


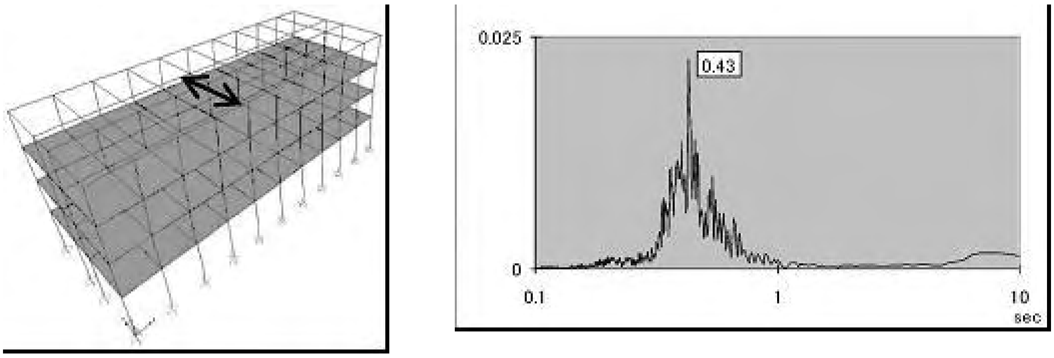
Figure 3.2.27 Column Section of PU Building

The predominant period of longitudinal direction is remarkably longer than that of latitudinal direction due to stiffness difference brought about by the building direction.

The result of the micro tremor measurement also brings a good suggestion for this respect.



a) Longitudinal Direction



b) Latitudinal Direction

Figure 3.2.28 Vibration Characteristic of PU Building

(2) Vibration Characteristic of Ground Surface

Micro tremor measurements of ground surface were carried out in few locations in Padang city and Pariaman. Micro tremor ground measurement, shaken by unspecified excitation, were observed and analyzed based on frequency analysis and the distribution of predominant period in three locations in Padang city are shown in Figure 3.2.29.

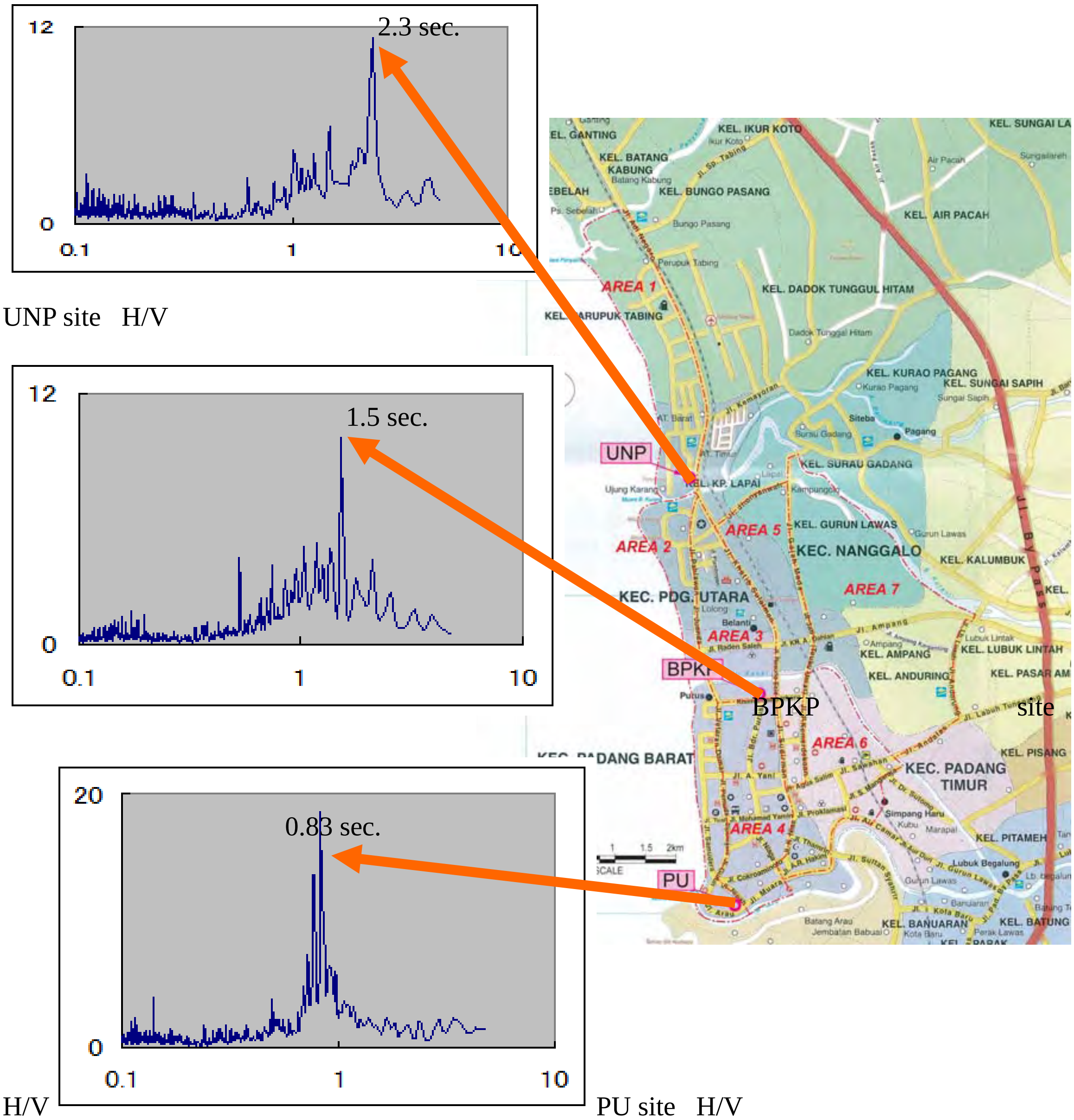


Figure 3.2.29 Vibration Characteristic of Ground in Padang City

Based on the analysis, it can be said that the deeper the surface layer in a comparatively shallow part of the ground, the predominant period is longer, while the softer the surface layer, the predominant period is longer.

The measurement sites were located in the south north, parallel to the coastline as shown in the map above. Therefore, the ground surface layer in Padang city can be assumed as either deep or softer for the northern part.

Prof. Kiyono (Kyoto Univ.) et al. had surveyed the predominant period of microtremor and the shear wave velocity distribution (velocity structure) of surface soil layer at many points of Padang. Above results are consistent with their results.

The accumulation of this kind of ground information is indispensable to make the earthquake hazard map for the Padan city. It is needed to execute the additional micro tremor measurement and PS logging, etc. in the future.

3.3 Countermeasure Responses after the Earthquake

3.3.1 Post-Earthquake Temporary Risk Evaluation

Earthquake aftershocks can cause significant damage to buildings. Occasionally, they can result to building collapse. This is the highest risk among previously damaged buildings. Hence occupants should not be allowed entry to damaged buildings which are at risk to occurrence of aftershocks. For the implementation of “POST-EARTHQUAKE TEMPORARY RISK EVALUATION,” the decision to permit entry must consider both the level of initial damage and the probability of aftershocks.

The team of Prof. I Wayan Sengara has conducted reconnaissance inspection of the damages incurred to building structures on 4 Oct. 2009, immediately after the earthquake occurred and evaluated the risk of danger to damaged building structures at the rate of 10-20 buildings per day for 4 days net, primarily for educational facilities. The inspections was carried out based on the ATC-20 method by posting one of the three colors of signed placard, green, yellow or red, on a noticeable wall of the damaged building indicating the results of the post-earthquake temporary risk assessment conducted for the risk of collapse of the structure due to possible occurrence of aftershocks. Figure 3.3.1, Figure 3.3.2 and Figure 3.3.2 show samples of the placard. Similar method was also applied for the 2007 earthquake as shown in Figure 3.3.3 hereunder.

Figure 3.3.1
Placard Sample used for the 2009 Offshore West Sumatra Earthquake

Figure 3.3.2
Placard Sample used for the 2009 Offshore West Sumatra Earthquake

Figure 3.3.3
Placard Sample used for the 2007 Earthquake

The result of the investigations for the post-earthquake temporary risk evaluation is shown in Table 3.3.1.

Table 3.3.1 Samples of Investigation Results for Post-Earthquake Temporary Risk in 4 Oct. 2009

no	Name of Building	Address	Altitude/Latitude Longitude	Result of Rapid Evaluation				Description (Damage Level)
				Green	Yellow		Red	
					Need Structural repair	Need Non-Structural repair		
4 okt 09								
1	Kopertis X (Aula)	Jl. Khatib Sulaiman Padang	8 m, S: 00 55' 21,6" E: 100 21' 48,8"		v	v		50%
2	Kopertis X (Pepustakaan)	sda	sda			v		25%
3	Kopertis X (Hall Badminton)	sda	sda				v	100%
4	Kopertis X (Laboratorium)	sda	sda			v		30%
5	Univ. Eka Sakti (Ged. F1)	Jl. Bandar Purus No. 11 Padang	6m, S: 00 56' 31,1" E: 100 21' 22.3"				v	100%
6	Univ. Eka Sakti (Ged F2 & G)	sda	sda			v		30%, Dilatasi antar bangunan diperlebar
7	Univ. Eka Sakti (Ged.C Rektorat)	Jl. Veteran Dalam No. 26 B Padang	6 m, S: 00 56' 36.3" E: 100 21' 23.4"				v	100%
8	Univ. Eka Sakti (Ged B Perkuliahan)	sda	sda			v		30%
9	Univ. Eka Sakti (Ged. D)	sda	sda				v	100%
10	Univ. Eka Sakti (Ged. A)	sda	sda				v	100%
11	Univ. Eka Sakti (Lab. Komputer)	sda	sda	v				0%
12	Ged. DepKeu RI, Ditjen Perbendaharaan Kanwil III Padang	Jl. Khatib Sulaiman No. 3 Padang	10 m, S: 00 55' 30,4" E: 100 21' 40,3"				v	100%
13	Bappeda Sumbar	Jl. Khatib Sulaiman No. 1 Padang	sda				v	100%
14	Dinas Pengelolaan Keuangan Daerah	Jl. Khatib Sulaiman No. 43	8 m, S: 00 55' 08,2" E: 100 21' 38,8"				v	100%
15	LB-LIA	Jl. Jhonny Anwar	7 m, S: 00 59' 27,6" E: 100 21' 10,8"				v	100%
16	UNP (Rektorat)				v	v		35%, perbaikan struktural ringan pada sayap kiri dan kanan
17	GOR UNP					v		25%

Trial use of the ATC-20 Method of USA was conducted for the 1989 Loma Prieta Earthquake. The method was further enhanced and used for the 1994 Northridge earthquake. Similar method was also developed in Japan at almost the same time and was fully applied for the 1995 Kobe Earthquake that occurred one year after the 1994 Northridge earthquake. High professional skill is needed in carrying out these services because of the immediate need for specific decision on site at the time of the survey. This kind of survey will therefore require an abundant experience/expertise in structural engineering. Development of Indonesian engineers for this field of expertise is therefore highly desirable as part of the emergency response measures.

Entry into damaged buildings as soon as possible is often necessary for a variety of emergency reasons, including the following: search and rescue, building stabilization, and salvage and retrieval of possessions. While building evaluation on permit for entry is needed for this type of services as part of the emergency response, decision for the delivery of needed subsidy based on the level of destruction is considered equally important. The latter decision is essential from the standpoint of impartiality. However, while the two aspects of evaluation appear to look well, it also creates confusion to a number of displaced people. There was an incident on "POST-EARTHQUAKE TEMPORARY RISK EVALUATION" which occurred at this time about the decision to provide subsidy which appeared to have been misunderstood by many inhabitants near the mountainous region of Pariaman during an interview.

3.3.2 Rehabilitation of Damaged Building

If the building is slightly damage, such as the occurrence of some cracks in masonry walls, immediate rehabilitation can be undertaken through the owner's financial means. If the building totally collapsed, then there is no other recourse but for the owner to demolish the structure for the reconstruction of a new one. The case about partially collapsed buildings may be crucial because of the difficulty in judging whether repair work is possible or if the repair work would restore sufficient strength. Owners of public building facilities should have the determination to provide the response because they are the concerned individuals to issue building permits.

The recommended measures in assessing restoration works of damaged buildings based on the survey results are described as follows:

【Verification of the structural stability in the case of BPKP building】

Some quantitative study for this building was carried out and for the I_s values, in estimating the existing earthquake resistant capacity, the evaluation procedure was conducted based on the simplified procedure. Firstly, stability calculations were conducted assuming the condition of the buildings before the earthquake occurred and the state of the structures after the earthquake occurred. This method is generally applied in Japan for the strengthening of structures but was modified to suit RC building code of practice in other countries with masonry wall. A detailed calculation method is described in the entitled, "Report on the Technical Cooperation for Temporary Restoration of Damaged RC School Buildings due to the 1999 Chi-Chi Earthquake". Obtained I_s values are shown in Table 3.3.2 hereunder.

Table 3.3.2 I_s Values for Various State of the BPKP Building

No	condition	***Is***	
		ridge direction	beam direction
1	before Earthquake	0.32	0.298
2	after Earthquake	0.094	0.085
3	restoration(non-RC wall、5 stories)	0.296	0.254
4	restoration(non-RC wall、remove top floor (=4 stories))	0.379	0.326
5	restoration(RC wall、5 stories)	0.576	0.656

Item No.1 of the table shows the earthquake evaluated resistant capacity based on the situation of the building before the earthquake occurred. The standard, by which the earthquake resistant capacity of the building is assessed is according to the I_s value, as explained hereunder.

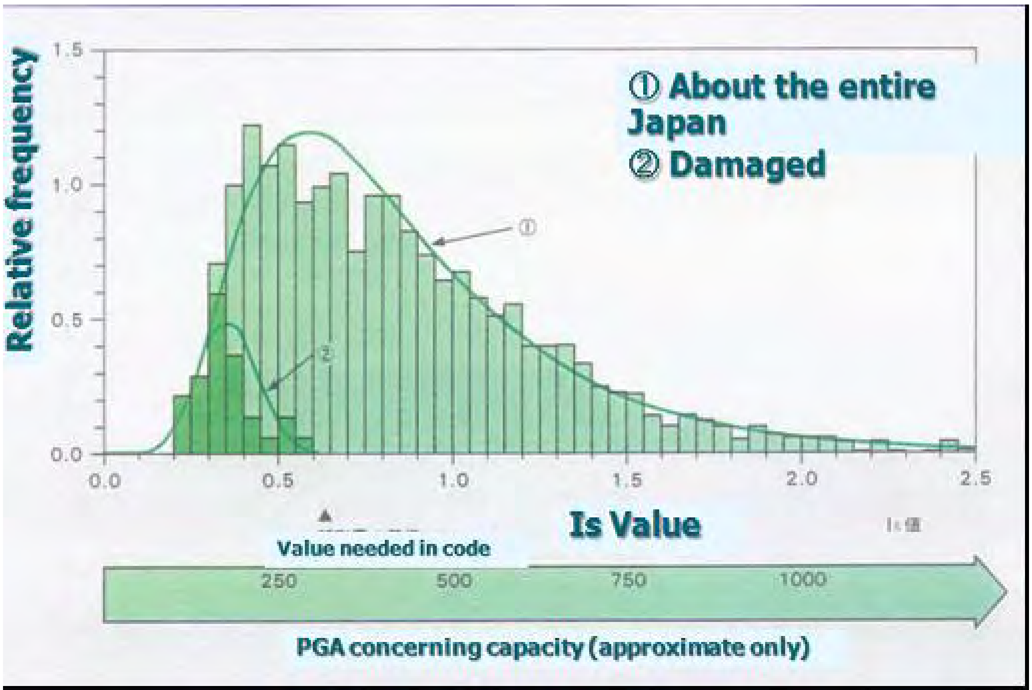


Figure 3.3.4 Distribution of the I_s Values

Figure 3.3.4 shows the distribution of “Is” value for past earthquakes. The light green columns with ①subscript show the distribution of the “I_s” value, obtained for 700 existing public buildings in Shizuoka Prefecture in Japan. The median value of this distribution is about 0.6. The dark green columns with ②subscript show the distribution of surveyed buildings, which were heavily damaged or collapsed in either of two past earthquakes in Japan in 1968 or in 1978 in Miyagi Prefecture. The median value of this distribution is about 0.3. Thus it was discovered that with buildings of “I_s” value greater than 0.6, may be able to withstand a seismic intensity equal to that of the Miyagi-oki earthquake with approximately MMI 9. The I_s value of 0.6 is a empirical standard value in Japan.

The evaluated I_s value based on the situation of the building structure before the earthquake occurred is about 0.3. As such, from the result, the BPKP building may not be able to resist an earthquake with intensity equal to Miyagi-oki earthquake.

Item No.2 of Table 3.3.2 shows the evaluated earthquake resistant capacity based on the situation of the structure after the earthquake occurred.

Item No.3 of Table 3.3.2 shows the evaluated earthquake resistant capacity based on the condition that all columns and walls are repaired to similar size of its original but the I_s value for this assumption will not regain the level before the occurrence of the earthquake.

Item No.4 of Table 3.3.2 shows the earthquake evaluated resistant capacity based on the condition that the top floor is removed with less improvement.

Item No.5 of Table 3.3.2 shows the evaluated earthquake resistant capacity based on the condition that all columns are repaired to similar size of its original with some of the masonry walls replaced with RC shear wall. For the location of masonry walls to be replaced RC shear walls, see Figure 3.3.5. In this case, this is just a recommended scheme to satisfy the standard I_s value of 0.6.

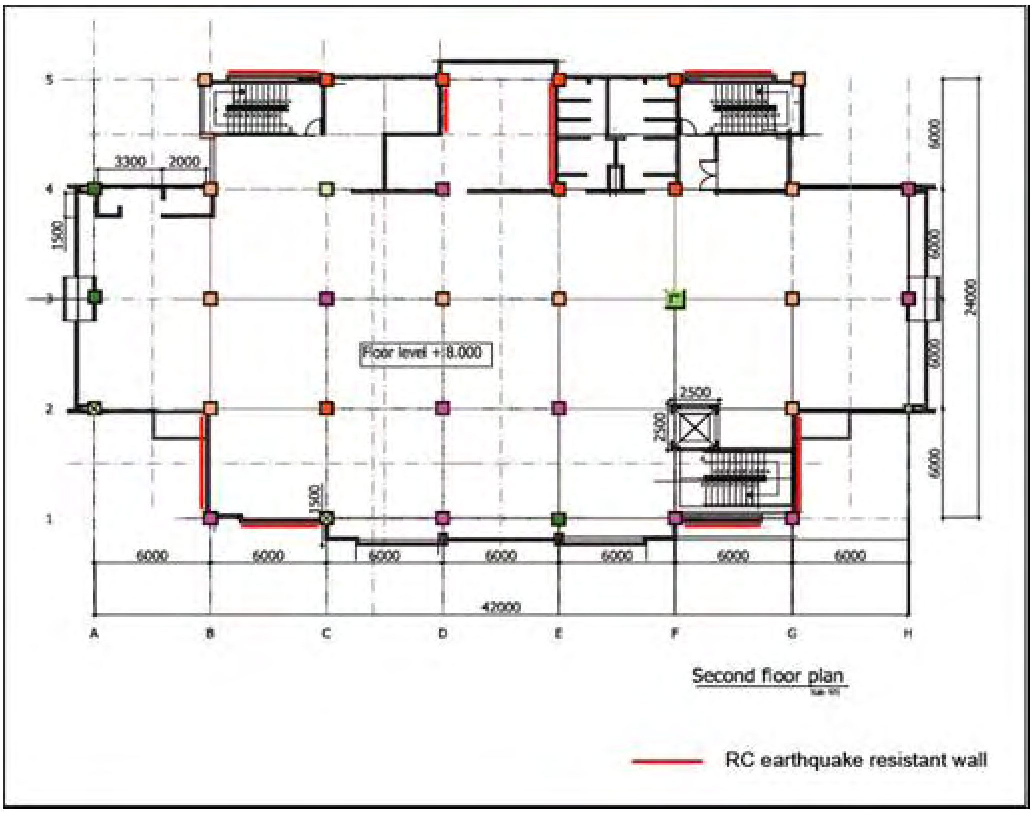


Figure 3.3.5 Location of Replaced RC Shear Wall

This section will describe how the BPKP building can be repaired and strengthened to the level, in which the building can withstand a seismic intensity equal to that of the 2009 Offshore West Sumatra Earthquake. This is a recommended scheme to recover building resistance within the realistic range. Of course, further detail examination is needed before practice.

3.4 How to Improve the Earthquake Resisting Capacity of a Building Structure

3.4.1 Prospect of an Earthquake- Resisting Capacity

In many case of the design analysis of the buildings in Padang safety of structure is confirmed by comparing required stress based on SNI 03-1727-1989 with capacity calculated by utilizing elastic assumption of stress-strain relation. In this context the value of *Base Shear Coefficient CIK* shall be 0.07 then this corresponds to 0.07G of inertia.

The intensity of ground motion of “the 2009 Offshore West Sumatra Earthquake” is presumed about I_{JMA} 5 (as Japanese intensity expression and this correspond to the value between 7 and 8 as MMI) in the result of questionnaire survey done by this team. The peak ground acceleration of ground surface can be presumed as following when the value of I_{JMA} 5 is presumed based on.

Relation between I_{JMA} and the *peak ground acceleration of ground surface a* is given by Eq. 3.4.1 then I_{JMA} 5 correspond to 223gal. (Equal to 0.227G)

$$I_{JMA}=0.55+1.90\log(a) \qquad \textbf{3.4.1)}$$

The peak value of response spectrum shall be given as 2.5 times of this value. (see Figure 3.4.1)

$$0.227G\times2.5=0.568G$$

Assuming that the ground is medium soil, the inverse proportion curve starts at 0.6sec. As a result the acceleration response spectrum value for the objective building with 1.1 sec fundamental natural period is 0.31 G.

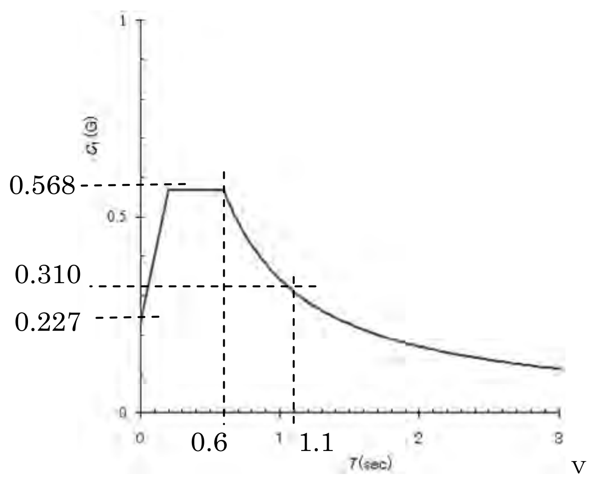


Figure 3.4.1 Presumed Response Spectrum

The *C* value of some surveyed buildings are 0.07. If the safety factor assumed for allowable stress is 3.0, critical structural member of the building may yield to horizontal inertia force of 0.21 times gravity.

$$0.07\times3.0=0.21 \ \ (G)$$

At generated inertia of 0.31 G compared with the 0.21G inertia when critical structural members of the building starts to give way, the damage incurred to many building in Padang is possible to consent.

From the foregoing, the following countermeasures concerning post-earthquake is suggested.

1. Repair of building damages for reuse is possible when the level of damage is light.
2. Partially damaged building can be repaired and strengthened to withstand similar level of earthquake as the 2009 Offshore West Sumatra Earthquake.
3. Strengthening the damaged buildings to comply with the latest Indonesian code for seismic loading however is not possible. Should it be required to comply with the latest Indonesian code, then there is no other recourse but to demolish and rebuild.

3.4.2 Repair and Retrofitting as Countermeasures (BPKP as a Case Study)

1) Construction Method

The BPKP building was included in the scope of detailed investigation of the study. As such the method of repair and retrofitting is explained hereafter. Because the level and condition of damage of building structures and sufferings of inhabitants are the same in almost all areas of Padang City, the repair and retrofitting of the BPKP building was taken as a case study for the needed repairs and retrofitting to be used as reference materials not only for this building but also for the other structures. Therefore, the proposed restoration works for the BPKP building is described hereafter.

(1) Restoration Work

A. The repair/restoration work for columns which sustained remarkable buckling is shown hereunder.

The sequence to repair the severely damaged column should be conducted as follows (to be read in conjunction with the illustrations):

a. Provide vertical load supporters as shown.

b. Remove damaged portions of the concrete.

c. Cut the column rebars (jack-up the beams if necessary).

d. Place reinforcing bars.

e. Cast concrete

f. Remove the supporters after the concrete gained sufficient strength.

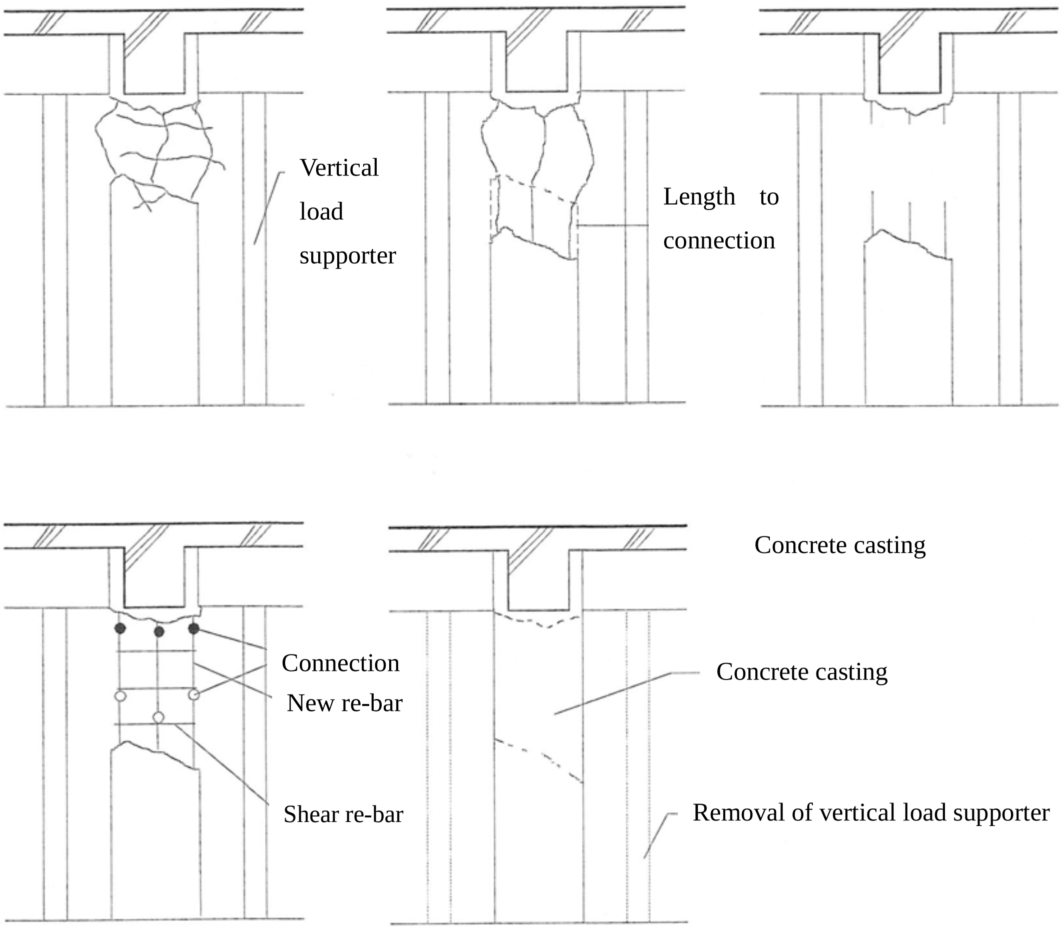


Figure 3.4.2 Restoration Columns

B. Restoration work of columns with crack(s)

Sequence of restoration works to columns with cracks shall be as follows:

a. Remove damaged concrete by chipping. Clean chipped off surface concrete with water.

b. Place Mortar Grout to prepared cracked sections.

c. Repair damaged existing hoops (Damaged reinforcing bars should be welded if necessary).

d. Place scaffolding/mould.

e. Place/cast mortar or concrete.

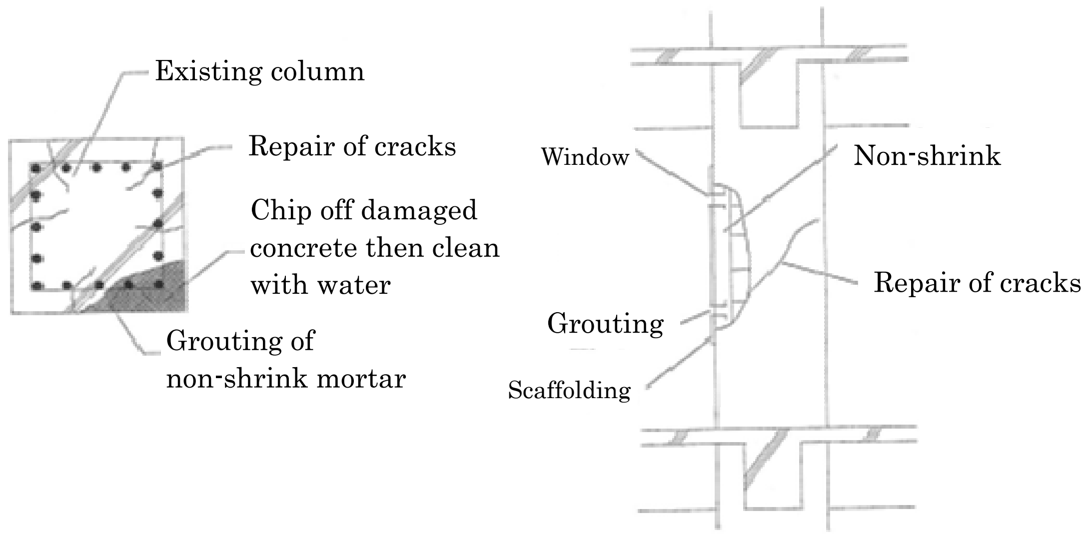


Figure 3.4.3 Restoration of Column Rebars (filling of cracks)

(2) Seismic Retrofitting

Two methods of retrofitting as described hereafter are suggested to maintain the aesthetic view of the building considering locally available expertise.

A. Restraining the relative story displacement (place RC shear walls)

To control the relative story displacement, the existing RC frames should be provided with 200mm thick RC walls to enhance its stiffness against horizontal stress.

Sequence of Construction

a. Setting post-installed anchor

b. Setting re-bars

c. Install the mould

d. Cast concrete and grout mortar

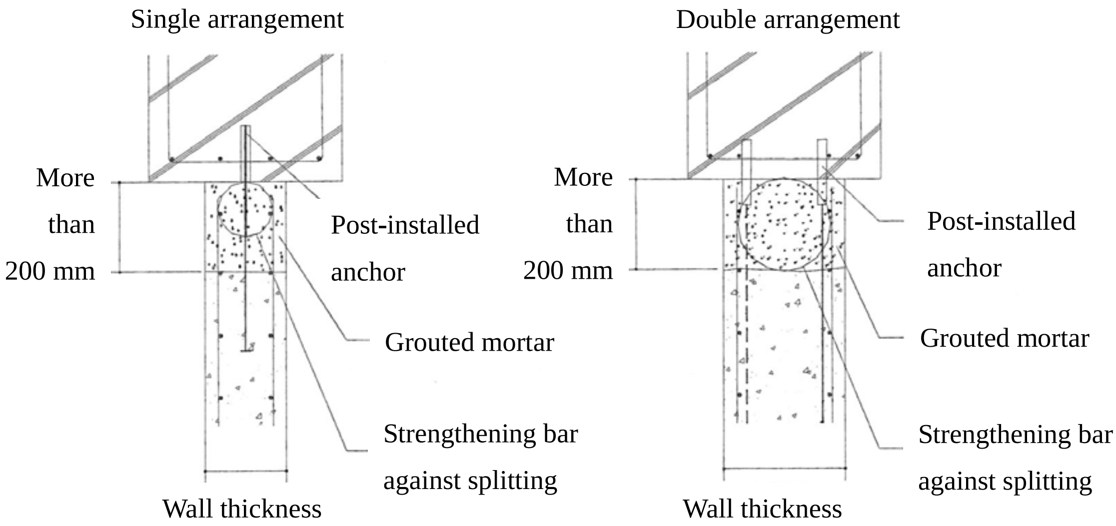


Figure 3.4.4 RC Shear Wall Section

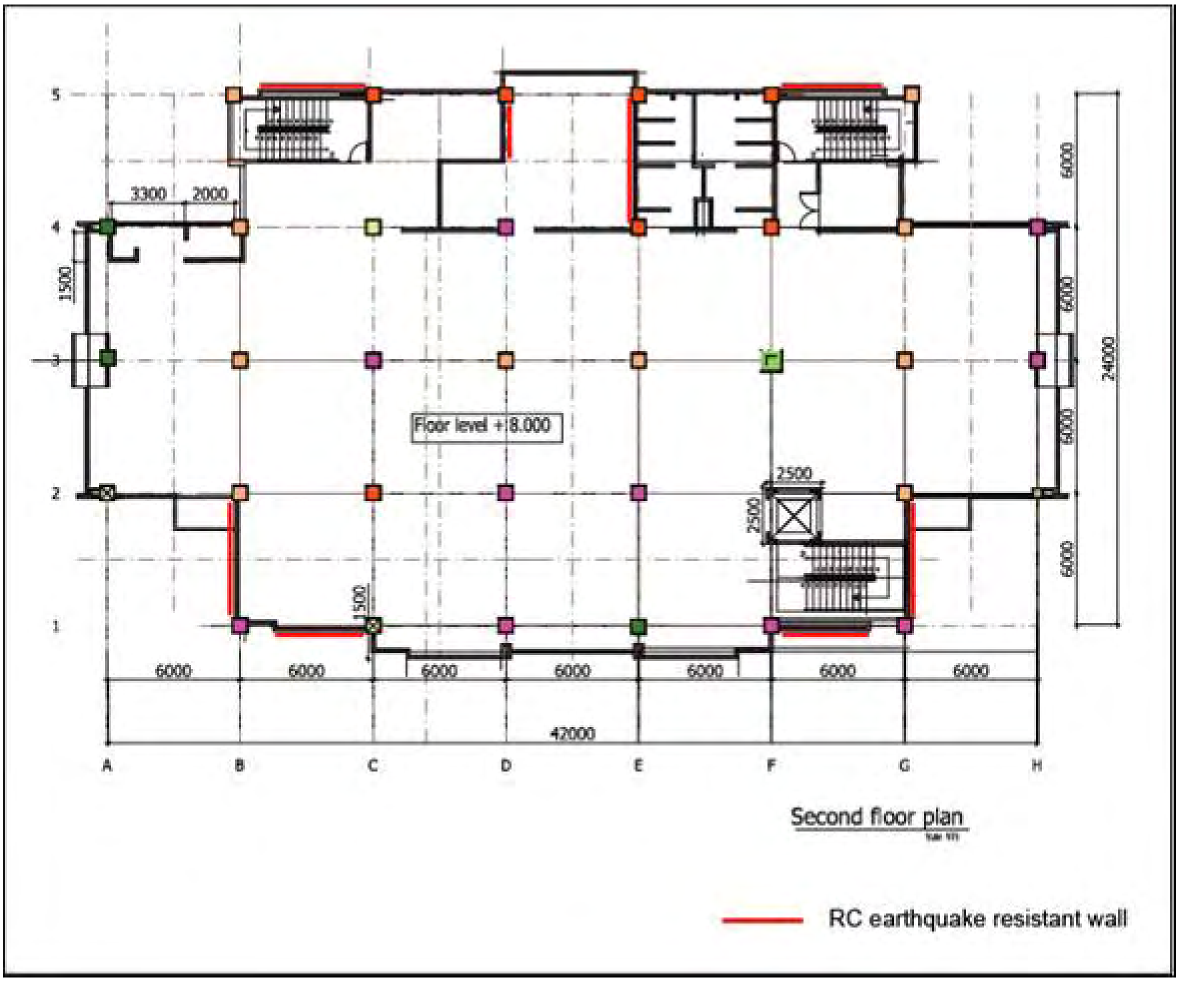


Figure 3.4.5 Location Plan of RC Shear Walls

The provision of bracings is also considered as one option to control the relative story displacement, but the construction would be more difficult than the setting of RC shear walls.

B. Restraining inertial force (Removal of top floor)

An effective method to control inertial force, is to lighten the dead-load of the building by reducing the dead-load of one floor.

The other option is to reduce the dead-load of the building by replacing the brick wall with panel wall. This method would be effective in preventing the occurrence of a second disaster because of the absence of brick walls that would collapse in the event of an earthquake.

The proposed retrofitting methods would enhance the resistance of the building against seismic force by improving the Is level as shown in the table below. The installation of RC shear walls like those being applied in Japan could also be adopted not only for BPKP and Padang City but also for the whole of Indonesia to increase the stiffness factors of building against seismic forces thereby minimizing the occurrence of possible disaster.

Table 3.4.1 Seismic Reinforcement Plan

No	condition	Is	
		ridge direction	beam direction
1	before Earthquake	0.320	0.298
2	after Earthquake	0.094	0.085
3	restoration(non-RC wall、5 stories)	0.296	0.254
4	restoration(non-RC wall、remove top floor (=4 stories))	0.379	0.326
5	restoration(RC wall、5 stories)	0.576	0.656
6	restoration(RC wall、remove top floor (=4stories))	0.739	0.843
	Table showing the estimated dead-load of each floor of BPKP		

floor	Weight(kN)
4	4,847
3	8,540
2	8,540
1	8,540
G	8,540

2) Construction Cost

The estimated cost for restoration including reinforcement of the BPKP Building is shown in the table below.

Table 3.4.2 Estimated Cost of Restoration including Reinforcement for BPKP Building

Work	Item	Cost(million rupiah)	Remark
Structure	restoration of columns	1,435	Inclusive of the cost for jack rental
	Construction of shear wall (thickness 20cm)	334	
Finishing	Floor, Wall, Ceiling, Doors & Windows	2,721	
Mech. & Elec.	Mechanical including Plumbing and Electrical	906	
Landscape	Guard house, Fence, etc.	85	
Total		**5,481**	

The total cost for restoration and reinforcement construction is estimated at Rp 5,500 million as shown in Table 3.4.2 while the cost for reconstruction of the building is estimated at Rp 18,320 million. Based on this premise, restoration cost and seismic reinforcement construction cost accounts for 30-40% of the rebuilding cost of the facility. ;In light of the above findings, it would be more economical to repair buildings with the same degree of damage as that of BKPK building as compared with rebuilding.

In terms of percentage, the estimated cost for the provision of RC shear walls is about 2% of the total cost for construction. It may therefore be possible to improve the earthquake-resistant capacity of existing buildings through the provision of shear walls estimated at 2% of the total construction cost.

3.5 Points to be Considered in Reconstruction

3.5.1 Historical Consideration

1) Transition of Design Method

It is necessary to look back at the historical background of the Indonesian earthquake-resistant design method before attempting to describe the demand for earthquake resistant buildings in Indonesia. While building technology has been in existence since old time, it was just recently that design analysis for earthquake resisting structures has been considered quantitatively. . The code for earthquake-resistant design has been revised whenever an devastating earthquake occurs.

The principle of the findings on earthquake experience is summarized in Figure 3.5.1. In a sense, the design guidelines is reviewed every time a big earthquake is experienced, and the result of the findings were incorporated into code in Japan.

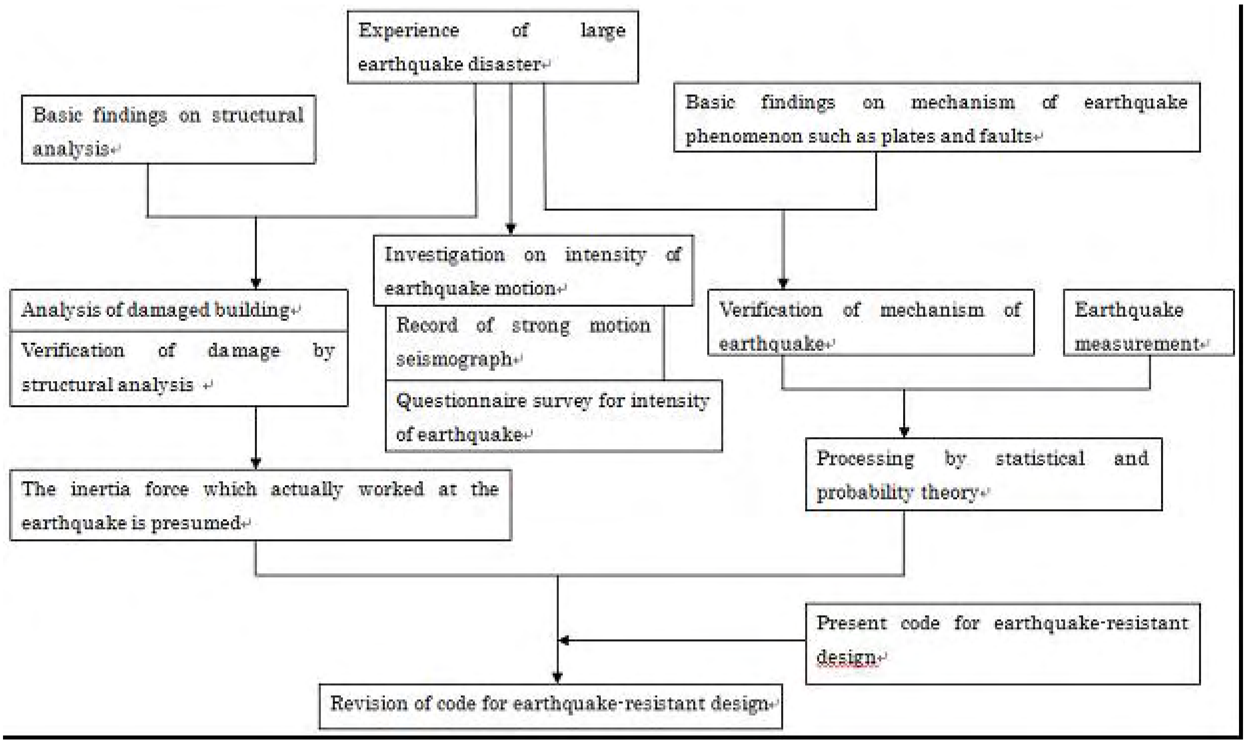


Figure 3.5.1 Findings on Earthquake Experience and Applications

The first code for seismic load was provided in Japan following the 1923 Kanto earthquake (M7.9). The value of 0.2 was adopted as a base shear coefficient because some buildings designed by T. Naito resisted seismic forces with relatively slight damage. This means that the horizontal inertia force corresponding to 0.2 times gravity acceleration shall be considered as load to analyze the model then calculate the stress and deformation generated by the analysis of the model. At this era, the structural calculation was conducted based on elasticity consideration . Termed as *allowable stress design method* because the generated stress is compared with the calculated capacity using the yield strength value multiplied by an allowable factor. The building standard law was then established in 1950.

The next epoch emerged after the occurrence of the 1968 offshore Tokachi earthquake (M7.9). The building standard code was revised with the incorporation of certain findings of the damage brought about by this earthquake.

The next era emerge with the occurrence of the 1978 offshore Miyagi earthquake (M7.4).

The next epoch existed after having occurred the 1978 offshore Miyagi earthquake (M7.4). Necessity of the larger design load of earthquake than that of previous code have been recognized by the experience gained from this earthquake. This improvement to the standard considered the elastic-plastic features of construction materials while the previous structural calculation has considered the elasticity only.

This method was termed as the elastic-plastic assumption design. The building standard law for seismic load was actually revised in 1981 based on e response spectrum in which the peak value reaches to about 1.0G.

In light of the foregoing, the chronology of earthquake-resistant design code in Indonesia can be summarized as shown in Table 3.5.1.

Table 3.5.1 Chronology Events for the Indonesian Earthquake-Resistant Design Method

	Year	Building Code		Seismic Load	Description
		Appellation	Description		
1	Before 1970	PBI 50			There is no regulation concerning earthquake.
2	1970~1980	PBI 71		Based on NZ code	Details are uncertain.
3	1980~ 1990	PBI 71		Response spectra are shown in Figure 3.5.2 and Figure 3.5.3	Allowable stress design method was obviously adopted based on the response spectrum.
4	1990~2002	PBI 90	SCI 1983 is reflected	Same as 3	The ultimate ductility check method was not reflected during this point although it was included in the 1983ACI
5	2002~	SNI 03-1726-2002	ACI 1997 is reflected	Response spectra are shown in Figure 3.5.5	(See Figure 3.5.4 and Figure 3.5.5) for the design seismic coefficient. The ultimate ductility check method was reflected during this time.

【before 2002】

The effective earthquake load covering the period from 1970 to 2002 for which PBI71 was enacted is shown in Eq.3.5.1, Eq.3.5.2, Figure 3.5.2 and Figure 3.5.3. Those correspond to the previous code for seismic load under “SNI 03-1727-1989”.

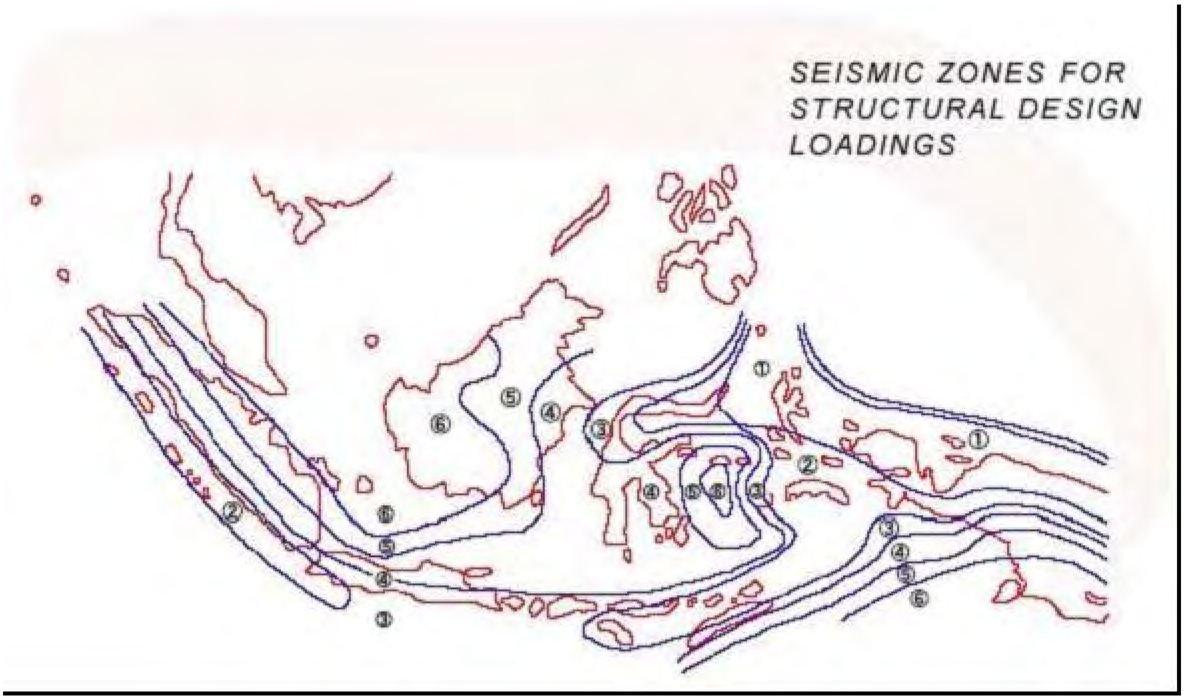


Figure 3.5.2 Zone Map (SNI 03-1727-1989)

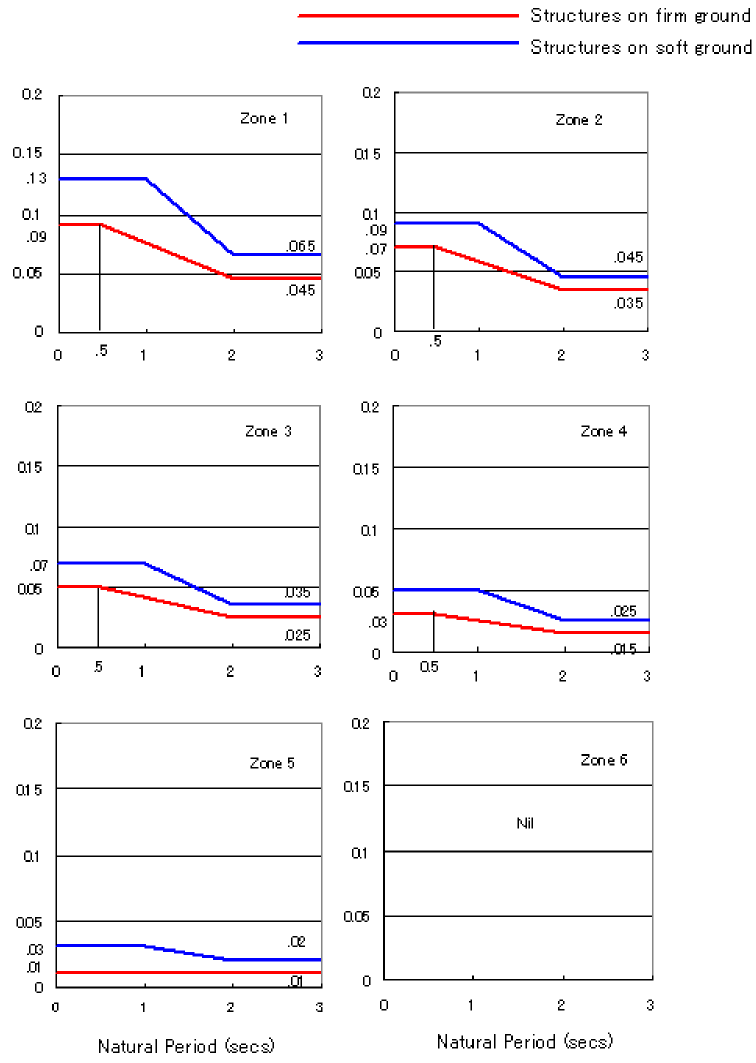


Figure 3.5.3 Response Acceleration Spectrum (*C* value in SNI 03-1727-1989)

For the previous earthquake load regulations, the *Nominal Static Equivalent Base Shear Force V* under the 1989 SNI 03-1727 Code is shown in Eq. 3.5.1. This value represents the total of the horizontal inertia force to act on the structure.

$$V = CD \cdot W_t \qquad \textbf{3.5.1)}$$

Where; W_t is the *Total weight of dead load and reduced live load,*

$$CD = C \cdot I \cdot K \qquad \textbf{3.5.2)}$$

Where; *C* is the *Base shear coefficient* (obtained from Figure 3.5.3), *I* is the *Importance factor*, *K* is the *Structure type factor* (*K* = 1.0 for RC Moment Resisting Frame structure)

Obviously the seismic load for structural design given by this standard is for elasticity design.

【Current】

The current Indonesian earthquake resistant design conditions for major RC buildings is described as follows:

The legislation "UNDANG-UNDANG 28/2002", published by the Indonesian central government, is supposed to provide the seismic load for the whole country[4] However another by-law state/prefectural provision can be published which is in contrast with "THE JAPANESE BUILDING STANDARD LAW." This legislation only provides unique fundamentals of building design procedure.

SNI 03-1726-2002 [5] describes the design seismic load generated by earthquake motion quantitatively with the seismic zone Map shown in Figure 3.5.4 and the Response Acceleration Spectrum shown in Figure 3.5.5.

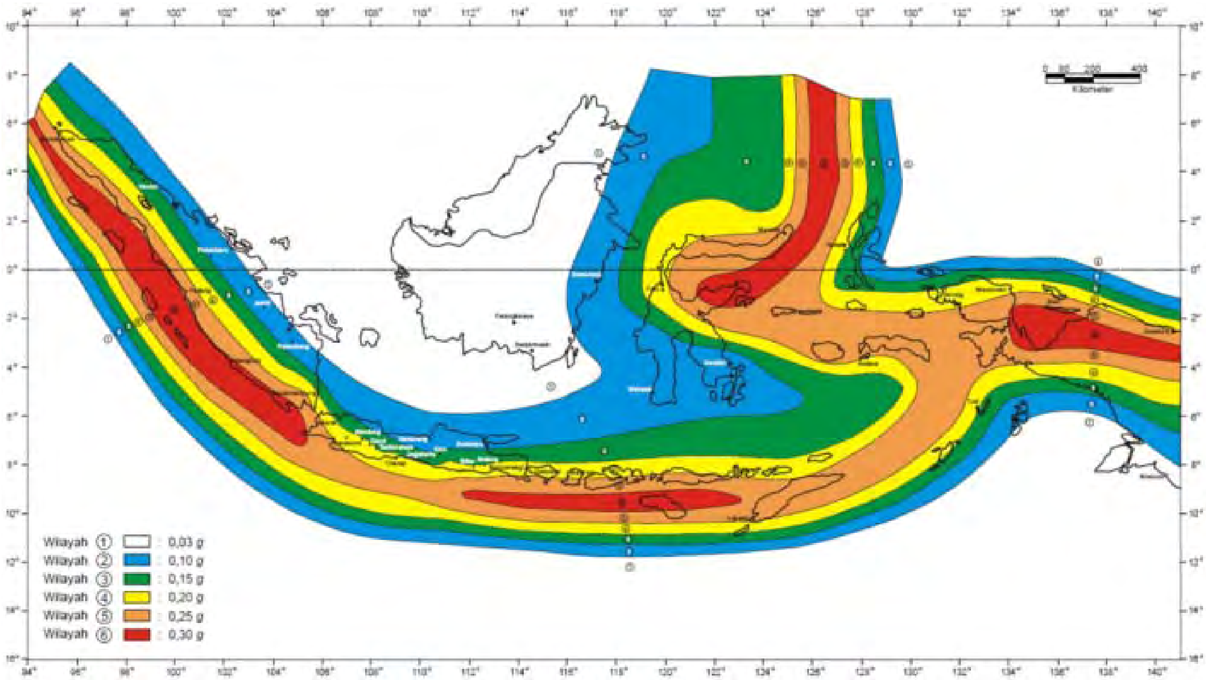


Figure 3.5.4 Indonesian Seismic Zone with Peak Base Rock Acceleration at 500 year return period

4 However this legislation stipulates to follow the Government restriction "PERAURAN PEMERINTAH 36/2005". The PP33(5) in "PERAURAN PEMERINTAH 36/2005" is corresponding article but this article provides as follows; "Further stipulation regarding loadings, resistance toward earthquake and/ or wind, and structural calculations conform to prevailing guidelines and technical standards."

5 STANDAR PERENCANAAN KETAHANAN GEMPA UNTUK STRUKTUR BANGUNAN GEDUNG SNI – 1726 – 2002 APRIL 2002

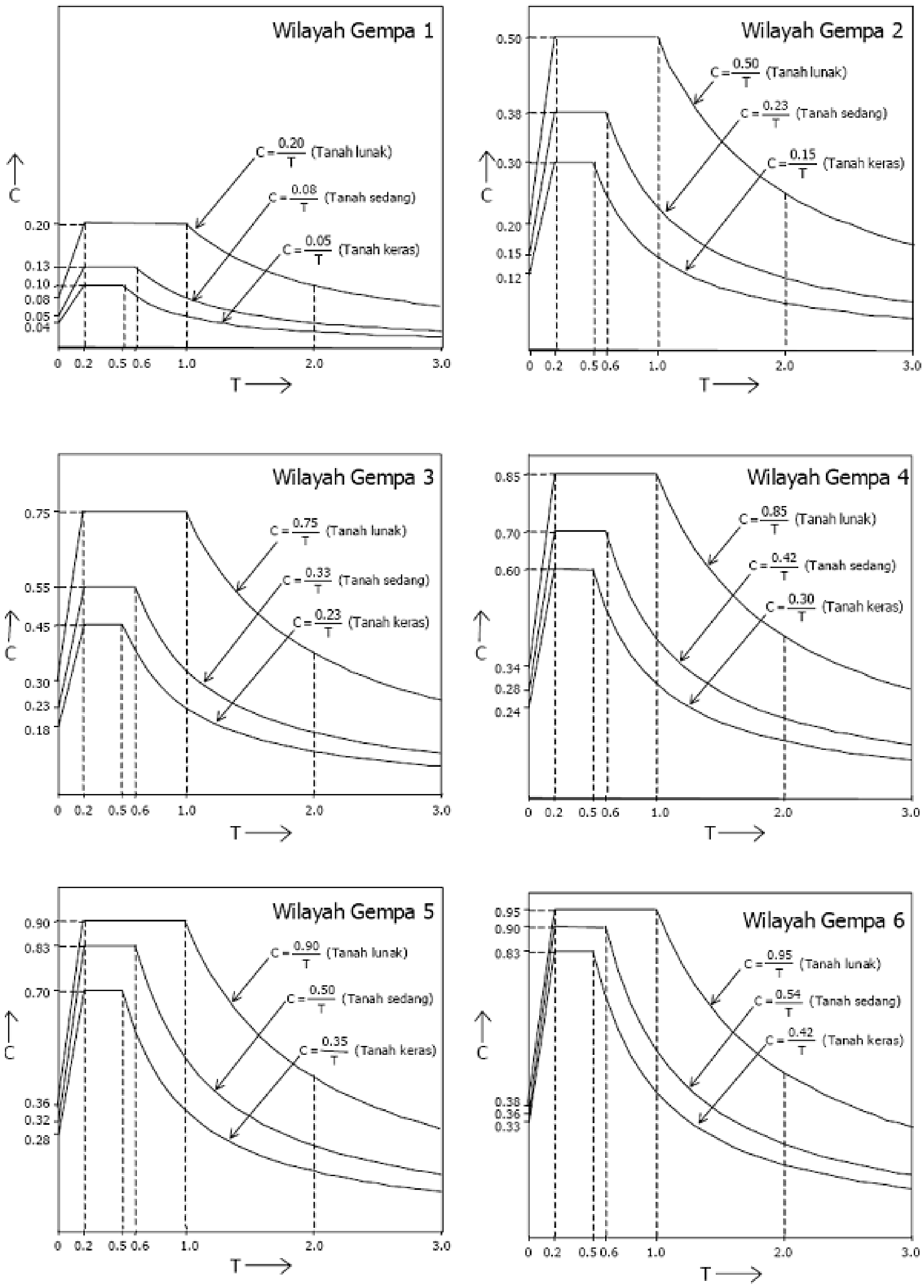


Figure 3.5.5 Response Acceleration Spectrum (SNI 03-1726-2002)

Taking Padang City into example, to the area is classified under Zone 5. Based on Figure 3.5.4, the PGA (Peak Ground Acceleration) is 0.25G. This value refers to base rock, located at great depths below ground level, so that when vibration is amplified through propagation passing through the surface ground layer ,a peak ground acceleration of 0.28G is obtained for stiff surface layer, 0.32G for medium surface layer and 0.36G for soft surface layer.

The Y axis value for the graph on the left end side as shown is 0.28G, 0.32G and 0.36G which as previously mentioned corresponds to the response of a pendulum, an analytical model for "single degree of freedom" with value at 0 seconds natural period and 5% damping ratio.

The Y axis at 2.5 times value on the left end side is given for the peak value of the response spectrum. The acceleration response value for comparatively a long period is given as a value, which is inversely proportional to the natural period.

The *Nominal Static Equivalent Base Shear Force V* is then given by Eq. 3.5.3). This value represents the total of the horizontal inertia force that will act on the structure.

$$V = \frac{C_1 I}{R} W_t \qquad 3.5.3)$$

Where; C_1 is obtained from Figure 3.5.5 for *fundamental natural period* T_1, R is the *Maximum Seismic Reduction Factor*, I is the *importance factor* and W_t is the *total weight of the building*, included as appropriate *Live load*.

The earthquake intensity is taken as a design condition, based on the latest earthquake design code "SNI 03-1726-2002". This code provides the distribution of peak ground acceleration value at 500 years return period[6]

Obviously the seismic load given by this code is the plastic-elastic design procedure.

The method of checking the structural stability is supposed to be provided by SNI 03-2847-2003 but the American design code " UBC", "IBC" or "ACI" are usually utilized in many cases because the method of analysis is generally carried out by computers with the use of USA software programs, to meet American design code of practice such as SAP2000, ETABS or SANSPRO)

Brief description of the method of design analysis follows hereafter. The required stress to act on each structural member is calculated based on the application of load force to the analytical model.

The capacity of each structure member on the other hand is calculated based on ultimate ductility check method for which the safety of structure is checked by comparing the required stress with the capacity of each structural member.

Based on the above-mentioned design procedure, the concept of the latest Indonesian code for seismic load (SNI 03-1726-2002) almost correspond to the Japanese code for seismic load although the value of seismic force for the Indonesian code is still lower than that of the Japanese code.

[6] It corresponds to 10% probability of exceedence in the service time (50years)

2) Issues of Concern

In regard to the earthquake load in the present code, value of *Seismic value Reduction Factor R* in Eq. 3.5.3 could be a matter of major concern. The Indonesian code for seismic load provides an *R* value of 8.5 maximum for *Moment Resisting Frame System* because the *Ductility Factor* μ[7] of that type of structure is expected to get to 5.2 maximum.

The above described *Nominal Static Equivalent Base Shear Force V* will be applied to the analytical model to estimate the stress of each member of the structure for the section design based on elastic-plastic/stress-strain relation. Certain setbacks however, were discovered in this part of the design procedure as described hereunder.

(1) Application of Excessively high *Seismic Reduction Factor R*

A large R value is not expected when ductility is not sufficient. Most RC structures in the region have insufficient ductility because the rebar arrangement for the column to beam connections is inappropriate coupled with the insufficiency in quantity and arrangement of hoop rebars to prevent shear failure.

The 2009 Offshore West Sumatra Earthquake is not as huge as initially conceived when compared to the assumed design earthquake based on the design code, so that certain parts of the structure may have to give way though the number of totally collapsed building was limited. However, based on the survey results, most of the failures occurred at the portion of the column to beam connections for RC Moment Resisting Frame structures. It is therefore apparent that the 8.5 value of *R* is excessive if the structures were designed based on SNI 03-1726-2002. Those failures however occurred because the capacity of each structural member was estimated based on ultimate limit state design although excessively small value was adopted for the *Nominal Static Equivalent Base Shear Force V.*

The estimated stress that was obtained based on SNI 03-1726-2002 was compared with the estimated stress capacity of the sectional dimensions of the structural members measured on site and the result of the field testing of material strength by nondestructive method as shown in“Appendix3-1 Structural Analysis of Surveyed Building” of this report. It was concluded that most buildings could not withstand the seismic load of the present “SNI 03-1726-2002” Code.

[7] *Ductility Factor* μ is defined as a ratio of generated deformation δ and deformation of yield point δ_y $\mu = \frac{\delta}{\delta_y}$

(2) Inappropriate Use of the Previous Earthquake Load Standard

In the previous earthquake load standard SNI 03-1727-1989, the capacity of each member structure is calculated based on the elastic stress-strain method. The safety of the structure is determined by comparison of the estimated stress with the capacity of each member structure. Thus, the stress intensity for each member structure would be maintained within the elastic domain. This is the major focal point of difference between the SNI 03-1727-1989 Code and the SNI 03-1726-2002 Code.

In certain cases, there are instances by which the *Nominal Static Equivalent Base Shear Force V* of the SNI 03-1727-1989 could be applied in the design of structures but the use of the SNI 03-1726-2002 for the region is mandatory.

As mentioned earlier, the concept of the latest Indonesian code for seismic load (SNI 03-1726-2002) almost correspond to the Japanese code for seismic load except that the value for seismic force under the Indonesian code is smaller than that of the Japanese code. Nonetheless, some crucial points regarding the code of practice still exist.

A. Complete compliance with the Earthquake-Resistant Design Code is not being Practice

In the design of structures in Indonesia, strict compliance with the earthquake-resistant design code is not completely abided except for large cities such as Jakarta. One of the primary causes is the deficiency in building confirmation system.

B. The Absence of a Mechanism to Induce Builders to Adopt the Code

In the case of Japan, the design code itself and introductory book for beginning designer is readily available in big bookstores. These materials however are not readily available in Indonesia.

The computer is usually used as a practical tool to facilitate design analysis. In the case of Japan, the software program is developed in own country to induce engineers to follow the code with the use of a software program that has been certified by the Ministry of Construction.

C. The Indonesian Code is not provided with Structural Details sufficiently

The Indonesian code for earthquake-resistant design provides only the seismic load to be adopted for the design. An earthquake-resistant building cannot be fully formulated unless both of the procedure regarding structural analysis and details of every individual part of the structure are provided carefully.

This concept differs from the tendency to conduct “performance based design” but the above mentioned code for structural details is indispensable to improve the technical understanding level of the general Indonesian engineer when the frequency and intensity of earthquake occurrence in Indonesia are considered.

Description of certain points on structural details follows hereafter.

3.5.2 Some Points Required for RC Structure Detail

Decision to demolish and reconstruct a building would be simple if the building was damaged irreversibly. Immediate decision can also be made if the owner has the financial capacity to pursue the rebuilding.

It shall be noticed that newly reconstructed building may not be earthquake resistant if similar kind of construction utilizing the same off-base construction method is used.

While recommendation was made to follow the latest Indonesian code for seismic loading (SNI 03-1726-2002) this would be insufficient because the Code only prescribed the seismic load that must be applied to undertake the frame stress analysis but the specific way to design each individual member of the structure was not provided.

In particular, SNI 03-1726-2002 prescribed the formula to be adopted for the seismic load based on the expectation that some part of the structural members would yield to seismic load and the deformation of the structure may be developed further. (see Eq.3.3.1))

Therefore careful consideration is needed to prevent the occurrence of sudden brittle failure due to improper rebar arrangement and insufficiency of reinforcement of structural members after they are designed and constructed. Sufficient ductility is required to enhance the resisting capacity of the structure. In order to obtain sufficient ductility of the structure, the following aspects should be considered.

1) Bending Should Occur First to Avoid Shear Failure

Shear failure will cause sudden brittle failure when the inertia force exceeds a certain limit.

For the effective prevention of shear failure, anti-shear failure rebars, including the hoops in beams and tie hoops in columns should be carefully provided and properly arranged.

The head of each tie hoop should be roundly bended to prevent detachment even when the column section is swollen by compression. (see Figure 3.5.6)

Figure 3.5.6 Detachment of tie hoop head

2) The brittle shear failure at column and beam connection part must be prevented.

The rebar that transmit axial tension between column and beam may break when configuration and arrangement of rebar is not appropriate.

3) Excessively rich layout of axial rebar must be prevented.

Sufficient concrete placing cannot be done when overlapped axial rebar occupy the dimension of column. Therefore bonding effect between concrete and rebar may be ruined in that case.

4) Yielding of column must be prevented from happening earlier than that of beam.

Column yielding before the effect of inertia reduction takes place would result to total building collapse.

This issue was already advocated in many documents but regrettably, the real condition of RC building structures in Padang is not yet based on this knowledge.

3.6 Collation and Confirmation with the Results of Other Investigation

The survey on damages generated by the earthquake was conducted jointly by Bandung Institute of Technology and Geoscience Australia with the support of Australia-Indonesia Facility for Disaster Reduction. The aim of this survey was the systematic collection and analysis of data on earthquake damage, with a particular focus on house, medical facilities and schools.

More than 70 collaborations were established in Padang from 27th Oct. – 11th Nov. Detailed surveys were conducted for medical facilities and schools while broad survey was undertaken for household buildings. Listed hereunder are the institutions and international agencies that participated in the surveys.

- Institute Technology Bandung
- Andalas University
- Geoscience Australia
- University of Adelaide
- University of Auckland
- Cardno Consulting
- Geophisical and Nuclear Sciences, NZ
- Nanyang Technical University, Singapore

It is reported that 50+ building types were identified in schema with 5 broad structural systems through 4,000 buildings surveyed over three week period. Local felt seismic intensity was assessed using the Modified Mercalli Scale.

Some 4000buildings were surveyed over a 3-week period of which more than 50 buildings were identified to have been subjected to severe damages. Because the seismometers in Padang were not operating at the time the earthquake occurred, the intensity was estimated using the Modified Mercalli Scale, based on how people felt at the time the earthquake occurred.

Attention should be paid on Figure 3.6.1 to Figure 3.6.3 regarding the relation between the seismic intensity and the building damage ratio. Those three figures can offer effective suggestion for the preparation of hazard map in the future.

The purpose of the JICA survey at this time is to know whether damage of large-scale RC building was severer than that of small scale building such as confined masonry etc. Therefore some effective findings are expected to be obtained from those three figures. However it was not possible because there is a slight difference of definition in those three figures.

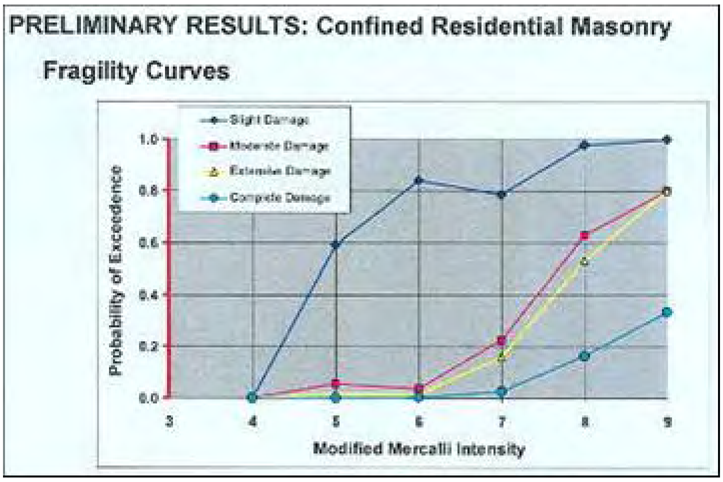


Figure 3.6.1 Relation between Seismic Intensity and Building Damage Ratio (Part1)

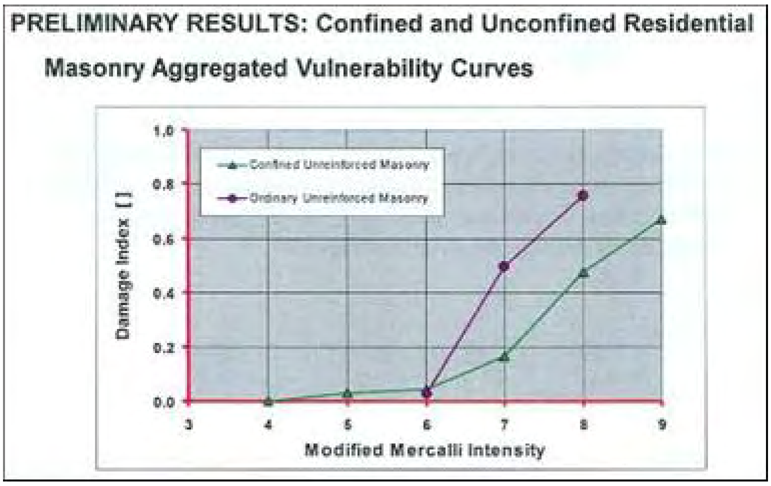


Figure 3.6.2 Relation between Seismic Intensity and Building Damage Ratio (Part 2)

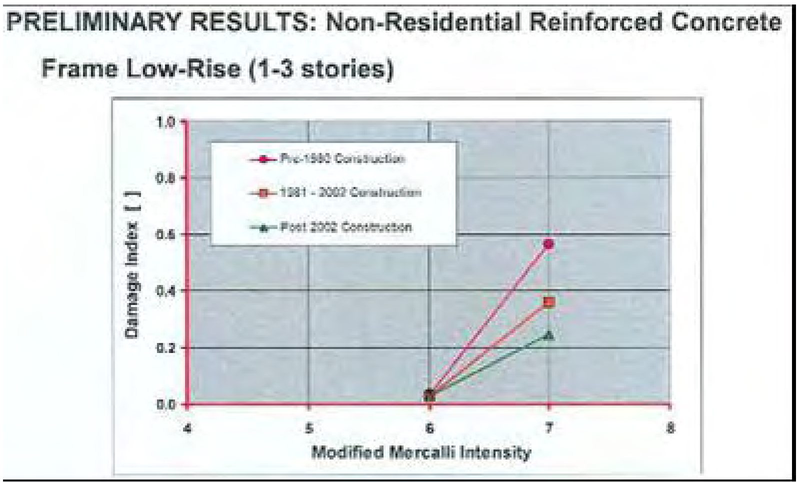


Figure 3.6.3 Relation between Seismic Intensity and Building Damage Ratio (Part 3)

The JICA Study Team also estimated the seismic intensity based on replies to the 35 items of questionnaires suggested by Dr. Ota which was distributed in 560 locations over Padang city. Comparison of the procedure used by AIFDR with the method of Dr. Ota could offer interesting progress for this type of technique.