Analytical Estimation of Seismic Response of a Typical R/C Building Damaged by the 2009 West Sumatra Earthquake in Indonesia

H. Choi & Y. Nakano
Institute of Industrial Science, The University of Tokyo, Japan

Y. Sanada & H. Kuramoto
Osaka University, Japan

T. Tomonaga, Maidiawati, Y. Hayati & J. Tanjung
Toyohashi University of Technology, Japan & Andalas Univertisy, Indonesia

SUMMARY:
The objective of this research is to analytically estimate the seismic responses represented by experienced maximum deformations of buildings based on ambient vibration measurements after earthquakes. In this paper, damage ratings of each story in selected building are evaluated based on residual seismic capacity ratio index *R*, and predominant periods and mode shapes are simulated using ambient vibration data. Furthermore, the maximum deformation of the building is assumed by proposed procedure using pushover analysis and modal analysis, and the estimated and the surveyed damage classes of R/C columns are compared to judge the compatibility of the procedure proposed herein. As a result, the surveyed damage classes of columns are approximately reproduced by proposed procedure in the stage of assumed maximum deformation. In future research, varying axial forces, unreinforced concrete walls and lateral force distributions will be taken into consideration to evaluate the seismic responses of selected building more precisely.

Keywords: Damage Rating, Damage Class, Ambient Vibration, Pushover Analysis, Modal Analysis

1. INTRODUCTION

On September 30, 2009, at 5:16 pm local time, a magnitude 7.6 earthquake (USGS) struck off the southern coast of West Sumatra, Indonesia (0.72S, 99.86E, Depth: 81km) as shown in Fig. 1.1, and Padang city which is the capital of West Sumatra province suffered moderate/heavy damage by the earthquake. In particular, a large number of reinforced concrete (R/C) structure buildings are damaged seriously.

Authors conducted two earthquake damage investigations in Padang city as shown in Fig. 1.2. These investigations were performed during December 13 to 20, 2009 and August 25 to 30, 2011, respectively. During the investigations, a typical five story R/C public office building located in damage survey areas as shown in Fig. 1.2 are selected, and the evaluation of the damage classes to each member based on the Japanese guideline (JBDPA, 1991) and the ambient vibration measurement using micro tremor are carried out in detail.

If ambient vibration measurements are performed before and after earthquakes in a building, experienced maximum deformation of the building can be commonly estimated from their measurement data. However, it is quite difficult to perform ambient vibration measurements to all the building before an earthquake. The objective of this research is, therefore, to analytically estimate the seismic responses represented by experienced maximum deformation of a building with only the ambient vibration measurements after an earthquake. In this paper, damage ratings of each story in selected building are evaluated based on residual seismic capacity ratio index *R* (Appendix and JBDPA, 1991), and predominant periods and mode shapes of the building are simulated using ambient vibration measurement data. Furthermore, the maximum deformation is assumed by proposed procedure using pushover analysis and modal analysis, and the estimated and the actually surveyed damage classes of R/C columns are compared to judge the compatibility of the proposed procedure.

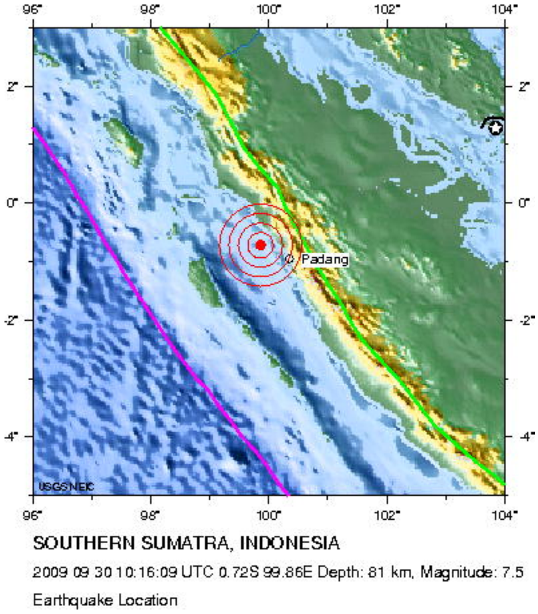


Figure 1.1. Epicenter of the 2009 West Sumatra eq.

Figure 1.2. Damage survey areas

2. DAMAGE OVERVIEW OF AN INVESTIGATED TYPICAL R/C BUILDING

In this study, Finance and Development Audit Agency building (BPKP: Badan Pengawasan Keuangan Dan Pembangunan), which is a five story R/C public office building built in 2003 located in the central part of Padang city as shown in Fig. 1.2, is selected to analytically assume the seismic responses after the earthquake.

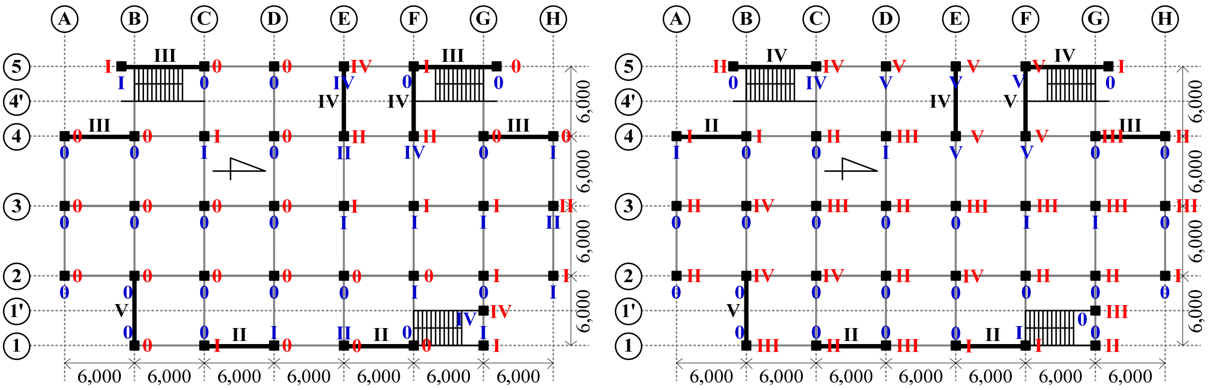
Photo 2.1 shows the whole view before and after earthquake damage of BPKP building, and Fig. 2.1 shows the second and the third floor plans together with the damage classes of R/C columns and unreinforced masonry (URM) infills evaluated using the Japanese guideline (JBDPA, 1991). The basic concept of this guideline is briefly described in Appendix. Since some structural plan was not able to be obtained, non-destructive tests using Schmidt hammer and re-bar locator were carried out during the investigations. Column cross-sections are shown in Table 2.1.

(a) Before earthquake damage (b) After earthquake damage

Photo 2.1. Whole view before and after earthquake damage of BPKP building

In the third floor suffered the most serious damage, the damages to west side of the building are relatively severe as shown in Fig. 2.1(b), and the buckling and the rupture of reinforcing bars are found in some columns of rows 4 and 5 as shown in Fig. 2.1(b) and Photo 2.2(a). In rows 2 and 3, exposed reinforcing bars are also observed in many columns as shown in Fig. 2.1(b) and Photo 2.2(b). The residual seismic capacity ratio index R values calculated by Japanese guideline are shown in Table 2.2. The R value for the transverse (EW) direction of the third floor is evaluated to be 54.1%, and the damage rating is judged as "Heavy".

0 - V: longitudinal (NS) direction of column, 0 - V: transverse (EW) direction of column, 0 - V: URM wall



(a) Second floor (b) Third floor

Figure 2.1. Floor plans and damage classes of members

Table 2.1. Column cross-section of each story

	1F & 2F	3F & 4F & 5F
Size (*mm*)	550×550	450×450
Longitudinal reinforcement	16-D19	12-D19
Shear reinforcement	Unknown	

(a) Damage class V (b) Damage class IV

Photo 2.2. Damage classes of R/C columns

Table 2.2. Damage ratings based on residual seismic capacity ratio index R

Floor	Longitudinal (NS) direction		Transverse (EW) direction	
	R value	Damage rating	R value	Damage rating
1	100.0	Slight	94.5	Light
2	85.9	Light	89.4	Light
3	85.0	Light	54.1	Heavy
4	100.0	Slight	96.7	Slight
5	98.6	Slight	99.3	Slight

3. MEASUREMENT OF AMBIENT VIBRATION USING MICRO TREMOR

In this chapter, predominant periods and mode shapes of BPKP building after the earthquake are computed from the ambient vibration measurement data using micro tremor. Furthermore, predominant periods of ground level at BPKP building site and around Padang city are also simulated. The measurement cases and computed results are described in the following sections.

3.1. Measurement Cases

Micro tremor data-logger (GEODAS-10-24DS by Buttan-Service Corporation in Japan) and pick-up equipment (velocimeter CR4.5-2 by Buttan-Service Corporation) are used for measuring ambient vibrations. Sampling frequency is set for 100*Hz* and record duration is 300seconds. The measurement positions of two cases used for calculating periods and mode shapes among all the 7 cases are shown in Fig. 3.1, and the situation at the measurement is shown in the Photos 3.1 and 3.2. In Case 1 and Case 2, the velocimeters are installed in the vertical directions of row D-5 and row D-2, respectively, as shown in Fig. 3.1.

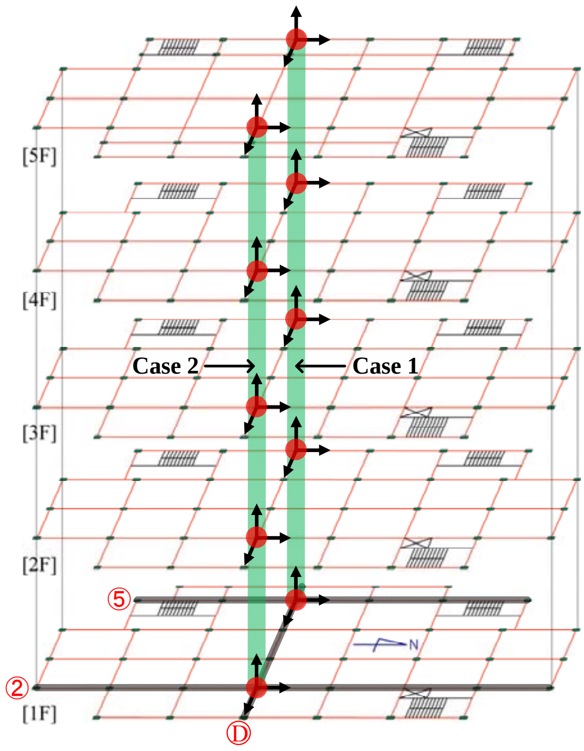


Figure 3.1. Two measurement cases

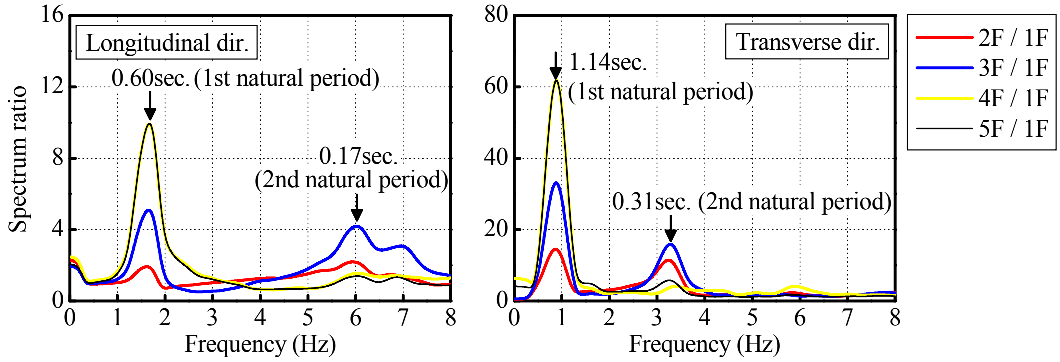
Photo 3.1. Build up the measurement instruments

Photo 3.2. Set up the pick-up equipment

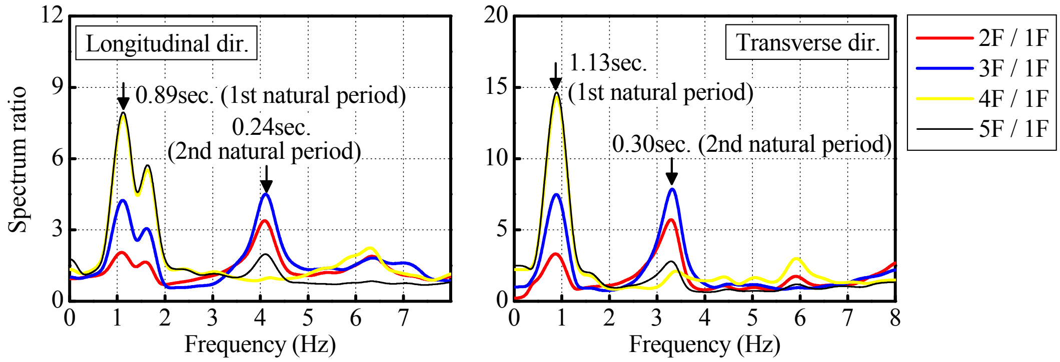
3.2. Measurement Results

3.2.1. Predominant periods of BPKP building after the earthquake

Fourier spectrum ratios (H/H Spectra) of each floor to first floor in Case 1 and Case 2 are shown in Fig. 3.2(a) and (b), respectively. As can be found in these figures, first natural periods of the longitudinal (NS) direction of the building after the earthquake are about 0.60second in Case 1 and 0.89second in Case 2, respectively, while those of the transverse (EW) direction have approximately the same value of 1.14second in both cases. The natural periods of the transverse direction is larger than those of the longitudinal direction, and this result corresponds well with the actual damage condition of the building.



(a) Case 1 (row D-5)

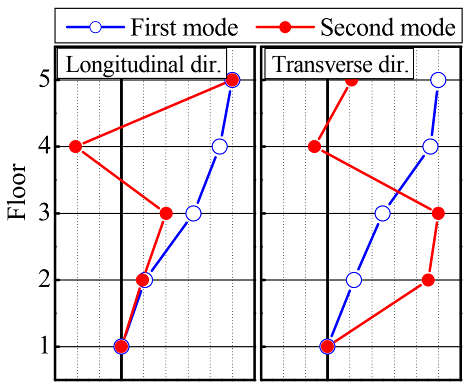


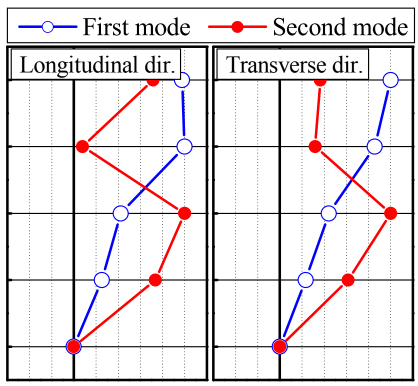
(b) Case 2 (row D-2)

Figure 3.2. Fourier spectrum ratios (H/H Spectrum) of each floor to first floor

3.2.2. Mode shapes in vertical plane after the earthquake

The first and the second mode shapes in vertical plane of the building after the earthquake are shown in Fig. 3.3. The phase differences $\Delta\theta$ between the maximum value and the other values at each natural period point shown in Fig. 3.2 are calculated from the Fourier phase spectrum. In this paper, it is assumed that each spectrum ratio value multiplied by $\cos\Delta\theta$ represents the mode shape of each floor. As shown in Fig. 3.3, this building has almost the same mode shapes for each predominant period regardless of measurement positions and directions. In particular, second mode shapes are consistent well with the actual damage condition that the damage to column capital of third story is the largest.





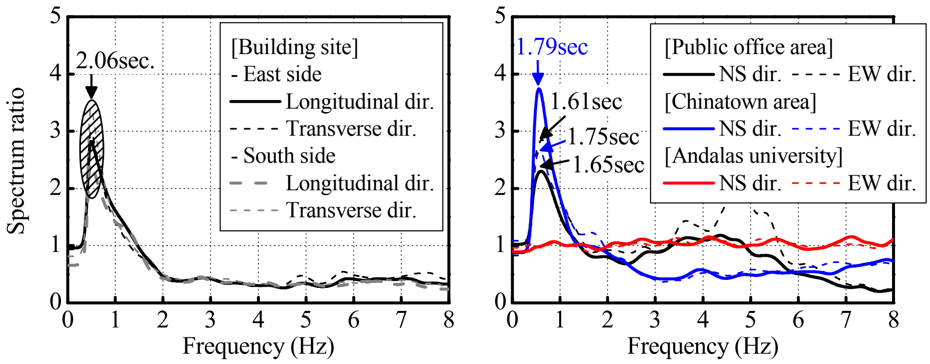
(a) Case 1 (row D-5)

(b) Case 2 (row D-2)

Figure 3.3. First and second mode shapes in vertical plane

3.2.3. Predominant periods of ground level at BPKP building and around Padang city

In these damage surveys, the ambient vibration measurements of ground level were performed at east and south side in the building site, public office area, Chinatown area, and Andalas university (refer to Fig. 1.2), respectively. Fig. 3.4 (a) and (b) show the Fourier spectrum ratio (H/V spectrum) at each ground level. As can be found the figure, it is revealed that the grounds of BPKP building site, public office area and Chinatown area have quite long periods as 2.06second, 1.61 through 1.65second and 1.75 through 1.79second, respectively. On the other hand, since H/V spectrum ratios of Andalas university located in the foot of a mountain are distributed in the range of about 1.0, its ground is judged to be near a bedrock.



(a) BPKP building (b) Public office area, Chinatown area & Andalas univ.

Figure 3.4. Fourier spectrum ratios (H/V Spectrum) of each ground level

4. ESTIMATION OF SEISMIC RESPONSE IN BPKP BUILDING

In this chapter, the maximum deformation of the transverse (EW) direction in BPKP building which earthquake damage is relatively large is estimated by the proposed procedure using pushover analysis and modal analysis, and the estimated damage classes of each column are compared with the actually surveyed values to evaluate the compatibility of the proposed procedure.

4.1. Proposed Procedure for Estimation of Maximum Deformation

In this section, in order to estimate the maximum response experienced during the earthquake, the following procedure is applied.

(1) Pushover analysis for overall frames of BPKP building is carried out for each step j.
(2) Unloading is performed at each step j, and each story stiffness K_u^i (i: the number of story) at unloaded state is calculated.
(3) Overall frames are replaced by multi-degrees-of-freedom model having K_u^i obtained in (2), and the first natural period T_j is then calculated using modal analysis.
(4) Iterative calculation of (1) through (4) is carried out until T_j is equal to T_T which is the target natural period (In this paper, T_T=1.14 roughly for the transverse direction of the building, refer to Fig. 3.2).
(5) The experienced maximum deformation of this building is determined at T_j=T_T.

4.2. Assumptions and Results for Pushover Analysis

In this study, pushover analysis is carried out using SNAP Ver.5, which is a wide use structural analysis program developed in Japan, for the transverse direction of BPKP building. The assumptions on the analysis are as follows.

(1) The lateral force distribution is assumed as the elastic first mode shape.
(2) The foundation, the slab, and the beam-column joint are considered as fix, rigid diaphragm, and rigid zone, respectively.
(3) R/C columns are modelled as having the nonlinear flexural springs to their both ends, and having the elastic shear spring and the elastic axial spring to their center, respectively. The Takeda model (Takeda. T et al., 1970) is employed for the nonlinear flexural spring as shown in Fig. 4.1. Each value shown in Fig. 4.1 is calculated by Eqns. 4.1 through 4.4 (AIJ, 1999 and 1988).
(4) Since the damages to beams are slight, they are assumed to be elastic.
(5) The URM walls located in frames are disregarded.

$$M_C = 0.56\sqrt{\sigma_B} Z_e + \frac{ND}{6} \tag{4.1}$$

$$M_U = 0.8 a_t \sigma_y D + 0.5ND\left(1 - \frac{N}{bDF_C}\right) \tag{4.2}$$

$$\alpha_y = \left(0.043 + 1.64 n p_t + 0.043\frac{a}{D} + 0.33\eta_0\right)\left(\frac{d}{D}\right)^2 \tag{4.3}$$

$${}_iK_U = \begin{cases} K_e & {}_i\theta < \theta_C \\ \dfrac{{}_iM + M_C}{{}_i\theta + \theta_C} & \theta_C \le {}_i\theta < \theta_U \\ \dfrac{M_U + M_C}{\theta_U + \theta_C}\sqrt{\dfrac{{}_i\theta}{\theta_U}} & \theta_U \le {}_i\theta \end{cases} \tag{4.4}$$

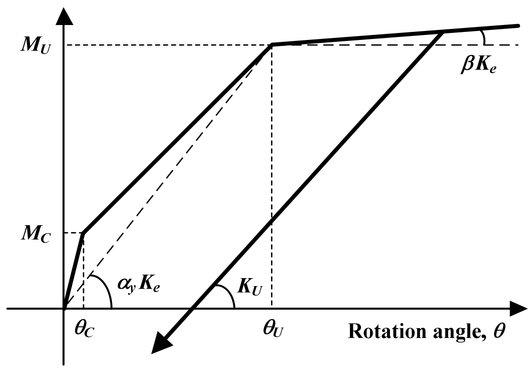


Figure 4.1. Hysteretic characteristic of column

where,

M_C, θ_c : flexural strength and rotation angle at cracking point, respectively
M_U, θ_U : flexural strength and rotation angle at yielding point, respectively
${}_iM$, ${}_i\theta$: flexural strength and rotation angle at *i* step, respectively
σ_B, F_C : compressive strength of concrete
Z_e, N : section modulus and axial force, respectively
b, D : width and depth of column, respectively
a_t, p_t : area of tensile reinforcement and tensile reinforcement ratio, respectively
σ_y : yield strength of longitudinal reinforcement
α_y : stiffness degradation factor at yielding point
n, a, d : ratio of Young's modulus, shear span and effective depth of column, respectively
η_0 : axial force ratio ($=N/bdF_C$)
K_e, ${}_iK_U$: initial stiffness and unloading stiffness at *i* step, respectively
β : stiffness degradation factor after yielding (=0.001)

The relationship between story shear force and inter-story drift angle obtained from the pushover analysis based on the assumptions above is shown in Fig. 4.2. As can be found the figure, the building deformations concentrate on the third floor. This result is consistent well with the damage states shown in Fig. 2.1 and Table 2.2.

4.3. Estimation of Maximum Deformation and Damage Class

In this section, the experienced maximum deformation of the building after the earthquake is estimated by the procedure described section 4.1, and the damage classes of R/C columns are analytically evaluated and compared to judge the compatibility of the procedure proposed in this research.

The relationship between story shear coefficient and inter-story drift angle of the third story is shown in Fig. 4.3 together with the development of the first natural period with respect to the its inter-story drift. As shown in the figure, the drift angle of the third story when becoming the same as the first natural period obtained from ambient vibration measurements (1.14sec., refer to Fig. 3.2) is

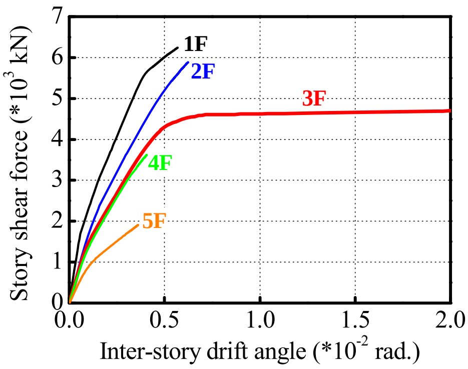


Figure 4.2. Story shear force vs. inter-story drift angle

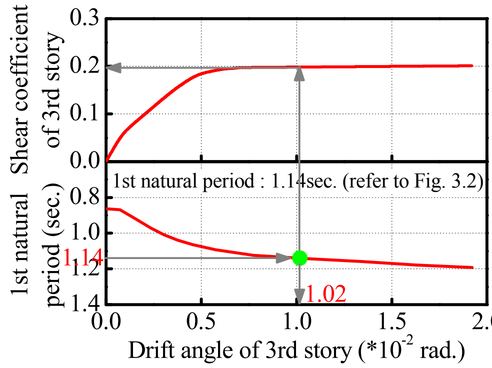


Figure 4.3. 1st period vs. drift angle of 3rd story

approximately 1.02×10^{-2}rad., and the shear coefficient value of the third story at its drift angle is about 0.20.

In this paper, the relationship between the load-deformation curve and the damage class for ductile member is defined as shown in Fig. 4.4 (Uchida, T. et al., 2005). Fig. 4.5 shows the damage classes of each column estimated by the proposed procedure together with the surveyed damage classes shown in parentheses. As shown in the figure, the surveyed damage classes of columns are approximately reproduced by proposed procedure in the stage of the assumed maximum deformation, although the estimated results to west side of the building are evaluated relatively smaller and those to east side are evaluated larger. It is seemed that the estimated damage classes of R/C columns are closely reproducible to the surveyed damage classes if the varying axial force of the building is taken into account. In future research, URM walls and/or lateral force distributions including the varying axial forces will be taken into consideration in order to evaluate the seismic responses of BPKP building more precisely.

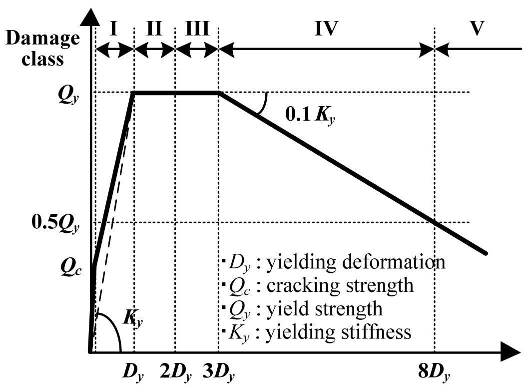


Figure 4.4. Schematic illustration of damage class vs. load-deformation curve

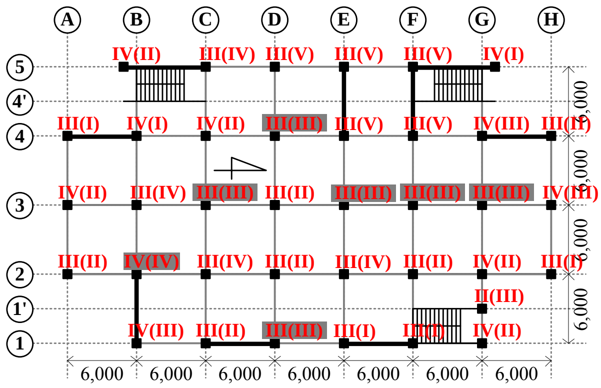


Figure 4.5. Estimation of damage class of each column in third floor

5. CONCLUDING REMARKS

In this paper, the seismic responses represented by experienced maximum deformation of BPKP building during the earthquake are analytically estimated with only ambient vibration measurement after the earthquake. The results can be summarized as follows.

(1) In BPKP building, the residual seismic capacity ratio index R value for the transverse (EW) direction of the third floor which suffered the most serious damage is evaluated to be 54.1%, and the damage rating is judged as “Heavy”.

(2) The first natural periods of the longitudinal (NS) direction are about 0.60 and 0.89seconds in Case 1 and Case 2, respectively, while those of the transverse direction are approximately 1.14second in both cases. The natural period of the transverse direction which the damage is relatively severe is larger than that of the longitudinal direction, and this result corresponds well with the actual damage condition.
(3) The ground predominant periods at the building site, public office area and Chinatown area are calculated about 2.06, 1.61-1.65 and 1.75-1.79second, respectively. On the other hand, since the H/V spectrum ratios of Andalas university are distributed in the range of around 1.0, its ground is judged to be near a bedrock.
(4) The drift angle of the third story when becoming the same as the first natural period obtained from ambient vibration data is approximately 1.02×10^{-2}rad. based on the proposed procedure.
(5) The surveyed damage classes of each column are approximately reproduced in the stage of the maximum deformation calculated by the procedure proposed herein. In future research, unreinforced masonry walls and/or lateral force distributions including the varying axial forces will be taken into consideration in order to evaluate the seismic responses of BPKP building more precisely.

APPENDIX SIMPLIFIED RESIDUAL SEISMIC CAPACITY RATIO INDEX *R* (Nakano. Y. et al., 2004)

A damage classification of R/C columns and walls is performed based on the damage definition shown in Table A.1. As defined in the Table, columns and walls are classified in one of five categories I through V.

Table A.1. Damage class definition of RC columns and walls

Damage class	Description of damage
I	- Visible narrow cracks on concrete surface (Crack width is less than 0.2*mm*)
II	- Visible clear cracks on concrete surface (Crack width is about 0.2 -1.0*mm*)
III	- Local crush of concrete cover - Remarkable wide cracks (Crack width is about 1.0 - 2.0*mm*)
IV	- Remarkable crush of concrete with exposed reinforcing bars - Spalling off of concrete cover (Crack width is more than 2.0*mm*)
V	- Buckling of reinforcing bars - Cracks in core concrete - Visible vertical and/or lateral deformation in columns and/or walls - Visible settlement and/or leaning of the building

Considering the normalized strength index *C* shown in Table A.2 and the seismic capacity reduction factor η listed in Table A.3 for a damaged member, the simplified residual seismic capacity index *R* can be expressed as shown in Eqn. A.1.

$$R = \frac{\sum_{j=0}^{5} A_j}{A_{org}} \times 100 \quad (\%) \tag{A.1}$$

where,

$A_0 = S_0 + M_0 + W_0 + 2CW_0 + 6CWC_0$

$A_1 = 0.95S_1 + 0.95M_1 + 0.95W_1 + 1.9CW_1 + 5.7CWC_1$

$A_2 = 0.6S_2 + 0.75M_2 + 0.6W_2 + 1.2CW_2 + 3.6CWC_2$

$A_3 = 0.3S_3 + 0.5M_3 + 0.3W_3 + 0.6CW_3 + 1.8CWC_3$

$A_4 = 0.1M_4$

$A_5 = 0$

$A_{org} = S_{sum} + M_{sum} + W_{sum} + 2CW_{sum} + 6CWC_{sum}$

A_0, A_1, A_2, A_3, A_4, A_5, A_{org}: Sum of normalized residual seismic capacity of members having damage class 0 through V and normalized seismic capacity of a building in pre-damaged condition, respectively

S_0, S_1, S_2, S_3, S_4, S_5, S_{sum}: Number of brittle columns having damage class 0 through V and their total number, respectively

M_0, M_1, M_2, M_3, M_4, M_5, M_{sum}: Number of ductile columns having damage class 0 through V and their total number, respectively

W_0, W_1, W_2, W_3, W_4, W_5, W_{sum}: Number of walls without boundary columns having damage class 0 through V and their total numbers, respectively

CW_0, CW_1, CW_2, CW_3, CW_4, CW_5, CW_{sum}: Number of columns with wing wall(s) having damage class 0 through V and their total number, respectively

CWC_0, CWC_1, CWC_2, CWC_3, CWC_4, CWC_5, CWC_{sum}: Number of walls with boundary columns having damage class 0 through V and their total number, respectively

Table A.2. Normalized strength index *C* for simplified procedure

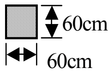
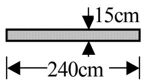
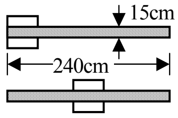
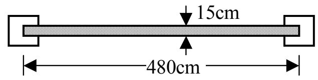
	Ductile / Brittle column	Wall without boundary columns	Column with wing wall(s)	Wall with boundary columns
Section				
τ_u (N/mm^2)	1	1	2	3
C	1	1	2	6

Table A.3. Seismic capacity reduction factor η

Damage class	Brittle column	Ductile column	Wall without boundary columns	Column with wing wall(s)	Wall with boundary columns
I	0.95	0.95	0.95	0.95	0.95
II	0.60	0.75	0.60	0.60	0.60
III	0.30	0.50	0.30	0.30	0.30
IV	0	0.10	0	0	0
V	0	0	0	0	0

The residual seismic capacity ratio index *R* defined above can be considered to represent damage sustained by a building. For example, it may represent no damage when R = 100 % (100 % capacity is preserved), more serious damage with decrease in *R*, and total collapse when R = 0 % (no residual capacity). The guideline defines the damage rating criteria shown below.

[Slight]	95(%) $\leq R$
[Light]	80(%) $\leq R$ <95(%)
[Moderate]	60(%) $\leq R$ <80(%)
[Heavy]	R <60(%)
[Collapse]	Building which is deemed to have $R\approx 0$ due to overall/partial collapse

REFERENCES

The Japan Building Disaster Prevention Association (JBDPA). (1991, revised in 2001). Guideline for Post-earthquake Damage Evaluation and Rehabilitation.

Takeda, T., Sozen, A. and Nielsen, N.M. (1970). Reinforced Concrete Response to Simulated Earthquakes. *Journal of Structural Division, ASCE*. **Vol.96:No.ST12**, 2557-2573.

Architectural Institute of Japan (AIJ). (1999). Design Guidulines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept.

Architectural Institute of Japan (AIJ). (1988). Standard for Structural Calculation of Reinforced Concrete Structures.

Uchida, T., Ohara M. and Maeda, M. (2005). Post-earthquake Capacity Evaluation of R/C Buildings Composed of Columns with Different Ductility. *Summries of Technical Paper of Annual Meeting, Architectural Institute of Japan*. **Vol.C-2**, 703-704.

Nakano, Y., Maeda, M., Kuramoto, H. and Murakami, M. (2004). Guideline for Post-Earthquake Damage Evaluation and Rehabilitation of RC Buildings in Japan. *13th World Conference on Earthquake Engineering*, **Paper No.124**.