RELATIONSHIP OF SEISMIC RESPONSES AND STRENGTH INDEXES OF GROUND MOTIONS FOR NPP STRUCTURES

Seckin Ozgur CITAK[1] Hiroshi KAWASE[2] and Shinya IKUTAMA[3]

[1] *Research Engineer, Ohsaki Research Institute, Inc., Tokyo, JAPAN.*
[2] *Professor, Safety control of urban space, Disaster Management for Safe and Secure Society, Disaster Prevention Research Institute, Kyoto University, Kyoto, JAPAN.*
[3] *Senior Research Engineer, The Japan Atomic Power Company, Tokyo, JAPAN.*
Email: citak@ohsaki.co.jp, kawase@zeisei.dpri.kyoto-u.ac.jp, shinya_ikutama@japc.co.jp

ABSTRACT:
In this study we investigate the relationship of seismic responses of ABWR type of Nuclear Power Plant structures, represented by shear force at each floor, and strength indices of simulated ground motions such as PGA, PGV, A_0 (the measurement parameter for Japanese Meteorological agency (JMA) seismic intensity), and PGA*PGV. Through the study we identified basic shear indices that can indicate or predict damage level that we can expect in the future without complex calculations. For this purposes Nonlinear Direct Integration Time History Analysis are performed for the ABWR Model using simulated strong motion records, and shear force responses of linear members are correlated with strength indices of ground motions in order to obtain shear indices. We found that the regression lines, especially with PGA for all floor levels can be used for the easy assessment of seismic response and could be the basic indices for the prediction of damage level of ABWR type of NPP structures. Also we were able to examine the ABWR model using acceleration data recorded during the 2007 Niigata-ken Chuetsu-Oki earthquake, M=6.8, at the basement level of seven Kashiwazaki-Kariwa Nuclear Power Plant units.

KEYWORDS:
Nuclear Power Plant, ABWR, Damage estimation, Shear indices, Strong ground motion parameters, seismic response.

1. INTRODUCTION

When earthquake occurs, the strong motion generated by the fault rupture causes damages and destructions to buildings thus harshly affecting human lives. The most crucial issue in earthquake disaster prevention of an urban area is how to reduce building damages, which inevitably leads to importance of knowing how to estimate quantitatively the damage levels that buildings would endure due to strong ground motion. Traditional quantitative expression for damage level of a structure is shear force, and recently story drift angle is also used. Also, there is a need to have an appropriate index to express strength of earthquake vibration. The most common representative indices are peak ground acceleration (PGA) and velocity (PGV), multiplication of them (PGA*PGV) and A_0, which is a filtered peak acceleration value used for JMA seismic intensity.

There are two levels in evaluation of possible damage levels in buildings. First and the most common way is through a damage evaluation analysis of an individual building. Second way is to perform damage evolution for a whole building stock of an urban area by making use of statistical earthquake survey data and classifying buildings according to their age, height and structural type.

Evaluation of seismic performance of buildings has become one of the topics of interest among researchers especially after the 1995 Kobe earthquake. However, most of them focused their research on evaluation of seismic performance of individual buildings, while only few studies were conducted on seismic performance of a building stock of urban area as a whole. For example Masuda and Kawase (2002) have studied relationship between the strength indices and damage ratio of middle-rise structures and established vulnerability functions for building models (Nagato and Kawase 2004) with observed ground motions as input waves. According to the study for middle-rise structures vulnerability functions were better predictable for PGA*PGV or JMA seismic intensity than PGA or PGV.

Establishing the relationship of ground motion strength indices and structural responses can be used for finding preliminary damage indices or shear indices. Our previous studies (Citak et al. 2000, 2006) were devoted to understanding of seismic behavior of Nuclear Power Plant (hereinafter NPP) structures, to predicting their damage level in such a way that could give us quick information on possible condition of concerned structures immediately after strong ground shaking.

Here in this paper we calculated shear indices using simulated strong ground motions predicted by the Central Disaster Prevention Council of Japan for Tokai-Tonankai-Nankai Earthquake (2003) as an extension to our previous studies, where recorded strong motions were used as an input force. Also, using recorded strong motion data of 2007 Niigata-ken Chuetsu-Oki earthquake additional simulation analyses were carried out for an ABWR type of NPP structures.

2. OBJECTIVES AND SCOPE OF STUDY

This study focuses on seismic response of NPP structures and relationship of seismic responses of NPP structures and strength indices of ground motions or ground motion parameters. Seismic parameters are represented by strength indices of ground motions such as PGA, PGV, A_0, PGA*PGV, and structural responses are represented by shear forces. First, numerical modeling of an ABWR type of NPP structure was carried out and its dynamic characteristics such as modes, transfer functions were analyzed. Second, simulated strong ground motions predicted by the Central Disaster Prevention Council of Japan for Tokai-Tonankai- Nankai Earthquake were used as an input force for the Nonlinear Direct Integration Time History Analysis. For the analysis the computer program SAP2000 Nonlinear 9, one of the most widely used static and dynamic Finite Element Analysis software was used. Finally, the shear force responses of the linear members were correlated with the strength indices of simulated strong ground motions, and shear indices were obtained.

In addition simulation analyses were carried out for Kashiwazaki-Kariwa NPP buildings. During the 2007 Niigata-ken Chuetsu-Oki earthquake, M=6.8, strong ground motion accelerations were recorded at the basement level of seven Kashiwazaki-Kariwa NPP units. First unit of this power plant was built in 1985, and seventh (and the last) one built in 1997 to resist different design input acceleration levels (167gal~273gal) (Table 4). In Japan NPP structures are built on Tertiary or earlier bedrock, thus design accelerations are set to a relatively low level when compared to a surface ground acceleration level. Surprisingly, at the Niigata-ken Chuetsu-Oki earthquake event, peak ground accelerations were of average 2.3 times (3.6 times at unit 2) larger than those used for the design. Using the recorded acceleration data (Tokyo Electric Power Company 2007) the ABWR model simulation analyses were conducted. Soil-structure interaction was not considered, because direct acceleration records observed at the basement floor levels were available.

3. ANALYSIS MODEL OF THE NPP (ABWR) STRUCTURE

Physical properties, used in the analysis model, of NPP (ABWR) structure, were obtained from “The report of nuclear power plant facilities limit characteristics evaluation method” (Nuclear Power Engineering Corporations (hereinafter NUPEC) 1998). The structural NPP (ABWR) model was represented as a lumped mass-spring-damper system (Figure 1) with lumped masses at each floor level, weightless frame members, springs and dampers that represent interaction of structure with ground. In more detail, the lumped masses at floor levels (m1, m2… m16) were connected via massless bars (B1, B2…B15). The lumped masses M9 and M10 represent the foundation of the structural model. The system was joined to the ground by horizontal, vertical, and rotational springs and dampers, which represent soil structure interaction. The NPP (ABWR) Model consisted of an outer wall with joints from 1 to 8, and an inner core with joints from 11 to 16. These two parts of the structure were rigidly connected at the floor levels. Two joints at each floor level were constrained together for all degrees of freedom. The foundation was represented by B9 at the foundation level.

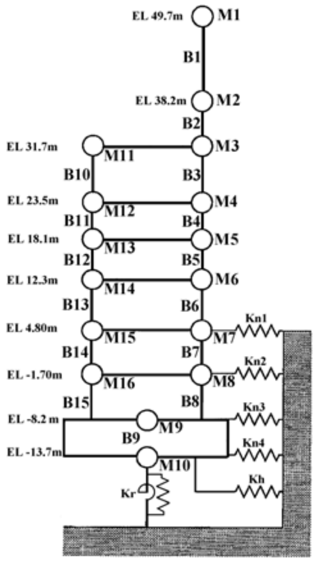

Figure 1 Analysis model of NPP (ABWR) structure

4. STRONG GROUND MOTIONS USED FOR DYNAMIC ANALYSIS

In dynamic analysis of the NPP (ABWR) model, we used strong motions simulated by the Central Disaster Prevention Council (the work of Tokai earthquake investigation committee, Tokai earthquake measure investigation committee, and Tonankai and Nankai Earthquake investigation committee) for Tokai-Tonankai-Nankai Earthquake. The simulated strong motion data consisted of NS, EW and UD components for each 1 km mesh at the engineering bedrock of the concerned area. These waveforms were first calculated for 5 km mesh, then adjusted and interpolated by the spline function for seismic intensity values. Therefore, the real calculated waveform has been synthesized for 5 km mesh only. In the analyses we used about 5986 waves including EW and NS components of 2993 records, which consisted of randomly chosen 300 records of 100-400 gal level and of all records, which have acceleration levels of more than 400 Gal. The records are summarized in Table 1 and PGA distribution for the concerned area is shown in Figure 2.

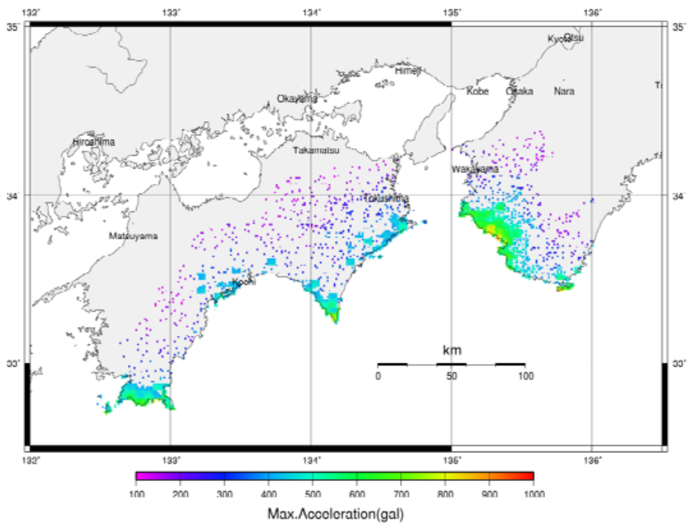

Figure 2 Peak Ground Acceleration distribution for selected data (Gal (cm/sec^2))

Table 1 Analyzed data

	Prefecture	Points	Maximum PGA(Gal)	Maximum PGV(cm/s)
1	Wakayama	1263	966.7	120.3
2	Tokushima	585	533.9	86.4
3	Kochi	1155	815.2	130.0
Total		2993		

5. DETERMINATION OF GROUND MOTION PARAMETERS

The correlation between shear responses of the NPP (ABWR) structure and the seismic indexes were examined. Seismic indexes were determined by the following methods of calculation. PGA (maximum ground motion acceleration) is an assumed value of the maximum absolute acceleration value of a strong ground motion record. Before calculating this seismic index, the records are baseline corrected and long period vibrations filtered by using BUTTERWOTH high pass filter. PGV is the maximum absolute value from velocity waveform, which is obtained by integrating acceleration record. PGA*PGV is the value obtained by multiplying PGA and PGV (not in time domain). A_0 is a measurement parameter of the JMA seismic intensity (Earthquake Research Committee 1998). The value of A_0 is an index that used to calculate seismic intensity by Equation (1). It is also called strength of the earthquake movement (Shabestari and Yamazaki 1998).

$$\mathrm{I} = 2 \cdot \log(A_0) + 0.7 \tag{1}$$

Here,
I – The JMA seismic intensity;
A_0 - Maximum value of *a*, which satisfies $\int w(t \cdot a)dt \geq 0.3$ (the range of integration is assumed to be the time duration that strong shaking continues);
t - Time (sec);
a – parameter related to the strong ground motion (cm/sec^2);
$w(t \cdot a)$ - Parameter, which depends on the following relations:
$v(t) < a$ ------ $w(t \cdot a) = 0$
$v(t) \geq a$ ------ $w(t \cdot a) = 1$
Where,
$v(t)$ is a vector-combined acceleration of three components at each time interval t filtered by a frequency-domain filter defined in Table 2.

Table 2 Types of Filter and its formula

Filter Type	Calculation formula
Filter for effect at cycle	$(1/f)^{1/2}$
High-cut Filter	$(1+0.694X^2+0.241X^4+0.0557X^6+0.009664X^8+0.00134X^{10}+0.000155X^{12})^{-1/2}$
Low-cut Filter	$(1-exp(-(f/0.5)^3))^{1/2}$

f - frequency of the earthquake motion (Hz);
$X = f / f_c$ (f_c =10 H_z), f_0=0.5 H_z

6. ANALYSES RESULTS

The purpose of the study was to examine the relationship between the responses of the NPP structure and the strength indices of ground motions. Direct integration non-linear time history analysis (Chopra 1995) (Wilson 2002) was adopted with 5% of model damping. The total of 5986 simulated acceleration data (see Table 1) were used as an input force for the dynamic analysis. As for the analysis results shear forces for each individual member were obtained and plotted against ground motion strength indices. Also linear regression line passing through the coordinate system 0 point were drawn (Figure 3 for PGA and A_0, Figure 4 for PGV and PGA*PGV). Table 3 summarizes the results represented in the graphs. From the prospect of predicting building damage for a certain level earthquake, better correlation would indicate a better adequacy of a particular strength index for its use in damage estimation.

Table 3 NPP (ABWR) Model Shear indices and Regression line coefficients R^2

Floor No	PGA		A_0		PGV		PGA*PGV	
(Member No)	Shear Index	R^2	Shear Index	R^2	Shear Index	R^2	Shear Index	R^2
Floor8 (B1)	11.625	0.789	17.260	0.531	96.816	0.078	$71.306x^{0.428}$	0.700
Floor7 (B2)	27.833	0.791	41.312	0.527	231.738	0.093	$164.920x^{0.431}$	0.701
Floor6 (B3)	44.242	0.798	65.661	0.536	368.377	0.110	$253.986x^{0.434}$	0.706
Floor5 (B4)	65.207	0.804	96.825	0.550	543.433	0.129	$366.996x^{0.436}$	0.715
Floor4 (B5)	77.932	0.810	115.750	0.558	650.122	0.146	$430.529x^{0.438}$	0.723
Floor3 (B6)	103.763	0.818	154.150	0.569	867.036	0.175	$551.529x^{0.442}$	0.737
Floor2 (B7)	86.181	0.830	128.129	0.589	721.874	0.211	$437.574x^{0.447}$	0.756
Floor1 (B8)	55.397	0.837	82.552	0.621	465.530	0.242	$274.512x^{0.450}$	0.780
Floor6 (B10)	15.692	0.806	23.303	0.551	130.796	0.128	$83.305x^{0.437}$	0.718
Floor5 (B11)	30.803	0.816	45.753	0.565	257.064	0.156	$167.496x^{0.440}$	0.730
Floor4 (B12)	38.829	0.822	57.675	0.571	324.290	0.173	$205.366x^{0.443}$	0.737
Floor3 (B13)	44.801	0.833	66.547	0.582	374.616	0.197	$226.297x^{0.447}$	0.751
Floor2 (B14)	37.085	0.837	55.158	0.598	310.936	0.222	$184.482x^{0.449}$	0.766
Floor1 (B15)	37.173	0.837	55.435	0.629	312.618	0.246	$184.078x^{0.450}$	0.785
Average		**0.816**		**0.570**		**0.165**		**0.736**

It can be seen from Table 3, where each strong ground indices are presented for each floor level with its correlation coefficient for all analyzed simulated ground motions, shear responses have good correlation (R^2=0.789-0.837) with PGA for all floors. On the other hand, it is clear that the strength index PGV does not show good correlation with shear force for any floor (R^2=0.078-0.246). A_0, which is max of filtered acceleration that used to calculate JMA intensity, show considerably good correlation (R^2=0.527-0.629) with shear responses. It shall be pointed out that index PGA*PGV did have bad linear correlation, but situation improved greatly (R^2=0.700-0.785) when the correlation was assumed to be nonlinear (i.e., parabolic) as it can be clearly seen in Figure 4.

7. SIMULATION ANALYSES USING OBSERVED STRONG GROUND MOTION DATA

Simulation analyses were carried out for the ABWR building of Kashiwazaki Kariwa NPP, which experienced M=6.8 Niigata-ken Chuetsu-Oki Earthquake in 2007. During this earthquake acceleration exceeding up to 3.6 times of design acceleration level was recorded. Table 4 gives brief information on the NPP buildings and the observed strong motions. Buildings No.6 and No.7 are ABWR type structures. Strong motions were recorded at the basement level and an upper level of the buildings. The upper level corresponds to M4-M12 floor of our model. Figure 5 shows spectral ratios calculated using the recorded and simulated acceleration data. The spectral ratio (M4-M12/M9) of our model is close to those calculated from the recorded data, especially for the building No.7 (EW-direction).

Since the strong motions were recorded at the basement floor, springs and dampers representing soil-structure interaction can be omitted. Table 5 presents maximum shear forces and story drift angle results obtained from the analyses of the simplified ABWR model (with soil-structure interaction omitted). The maximum story drift angle is 1/2000 at the Floor 4 of the building No.6 (EW-direction), which shows that the relative displacements are still in a reasonable range, even the recorded PGA levels exceeded the design PGA value.

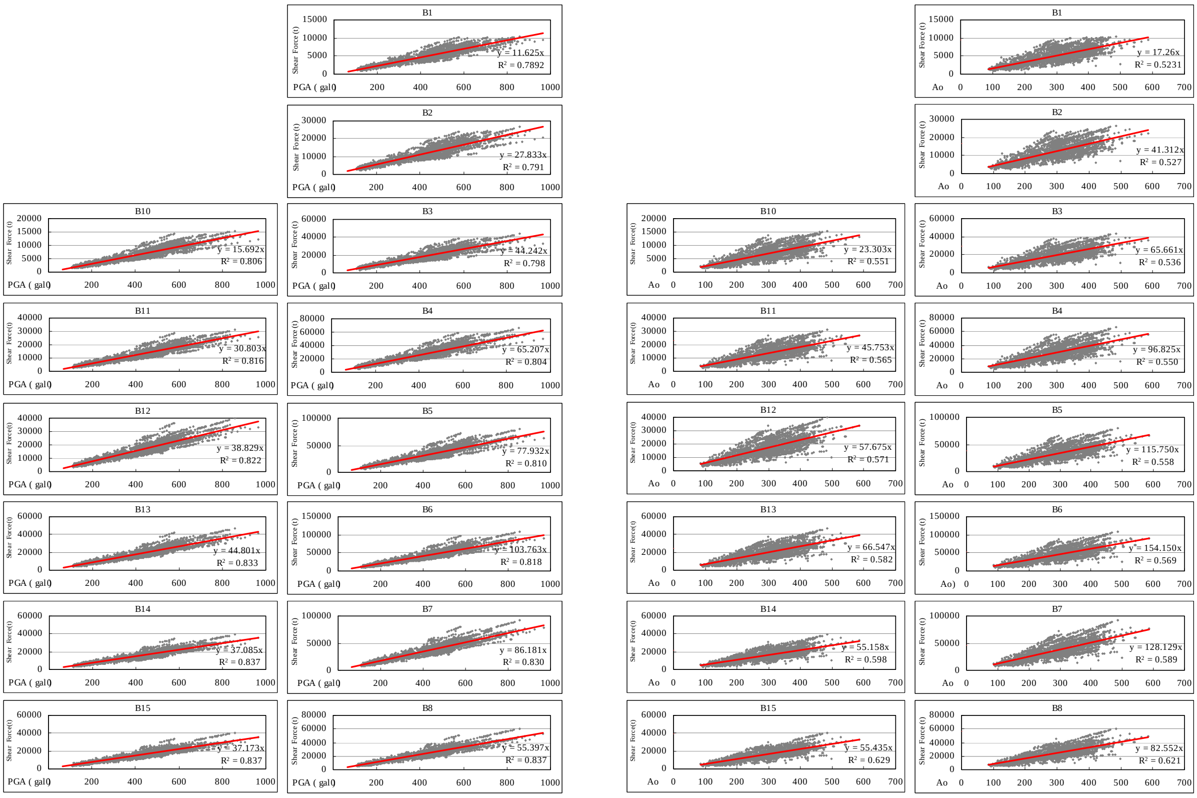

Figure 3 NPP (ABWR) Model Shear Force and PGA (left), A_0 (right) Correlation Graphs

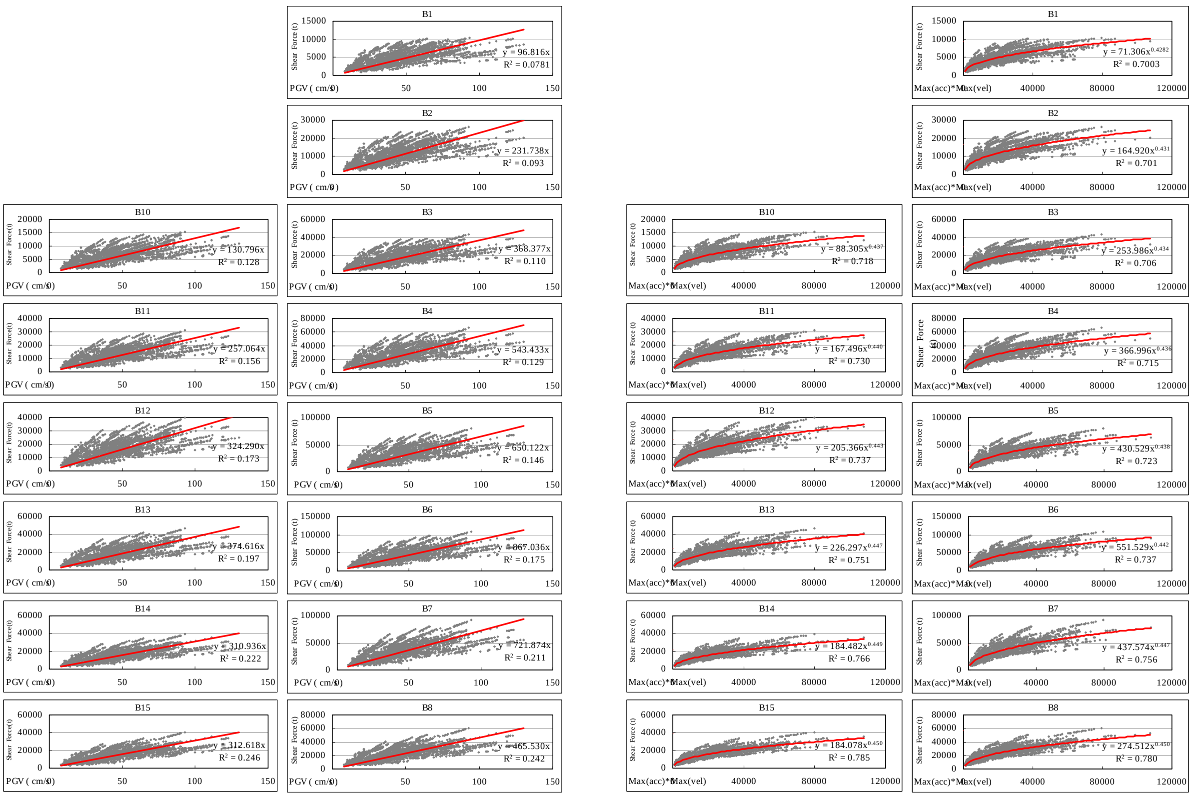

Figure 4 NPP (ABWR) Model Shear Force and PGV (left), PGA*PGV (right) Correlation Graphs

Table 4 Design and observed acceleration values at Kashiwazaki Kariwa NPP (TEPCO 2007)

Unit No	Structure Type	Construction year	Design Acc.*(Gal)			Observed Acc. (Gal)		
			NS	EW	UD	NS	EW	UD
1	BWR	1985	274	273	235	311	680	408
2	BWR	1990	167	167	235	304	606	282
3	BWR	1993	192	193	235	308	384	311
4	BWR	1994	193	194	235	310	492	337
5	BWR	1990	249	254	235	277	442	205
6	**ABWR**	**1996**	**263**	**235**	**322**	**271**	**322**	**488**
7	**ABWR**	**1997**	**263**	**263**	**235**	**267**	**356**	**355**

* Peak acceleration at the foundation level calculated considering soil amplification effect for NS and EW directions, acceleration values for UD direction, however, estimated from static design.

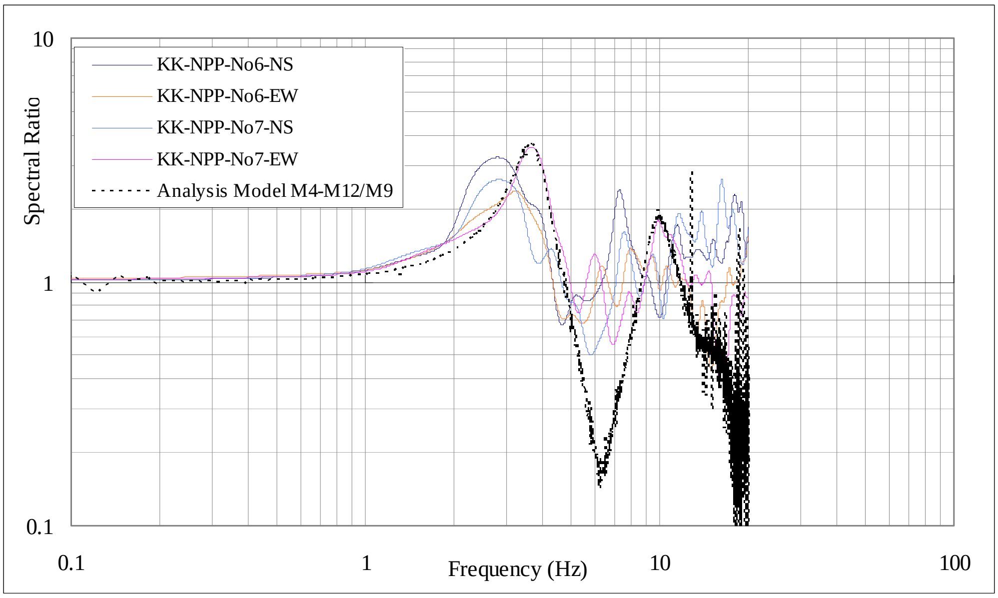

Figure 5 Spectral Ratios of No.6, No.7 and NPP (ABWR) Model

Table 5 Maximum shear force responses and story drift angle values for each floor as analysis results of recorded acceleration data at No.6 and No.7 of Kashiwazaki Kariwa NPP

Floor No	Floor h. (cm)	No6-NS			No6-EW			No7-NS			No7-EW		
		Shear Force (t)		Drift Angle	Shear Force (t)		Drift Angle	Shear Force (t)		Drift Angle	Shear Force (t)		Drift Angle
		B10-15	B1-8		B10-15	B1-8		B10-15	B1-8		B10-15	B1-8	
Floor8	1150		5566	0.00026		7501	0.00037		5343	0.00026		3978	0.00017
Floor7	650		13062	0.00027		18511	0.00039		12347	0.00026		9074	0.00019
Floor6	820	7190	19723	0.00027	10199	30084	0.00041	6491	18648	0.00025	4910	13847	0.00019
Floor5	540	12834	28220	0.00032	20017	44216	0.00049	12049	26616	0.00030	9395	20383	0.00023
Floor4	580	15476	33064	0.00032	**24936**	**52370**	**0.00050**	14877	31326	0.00030	11564	24068	0.00021
Floor3	750	17080	42025	0.00029	27657	67748	0.00047	16926	40834	0.00028	13599	31200	0.00021
Floor2	650	20003	45512	0.00026	31919	72324	0.00041	19836	44284	0.00025	16059	35036	0.00020
Floor1	650	28747	41640	0.00020	45164	65380	0.00032	28480	40977	0.00020	23417	33497	0.00016

8. CONCLUSION

In general, it is difficult to predict or explain expected building damage using only one particular strength index. PGA is relatively the best index to predict the damage level of the structure. A_0, though has complicated calculation method, can be the second best index for damage prediction calculations. As for PGA*PGV, correlation coefficient values are between those of PGA and A_0, when correlation assumed to be nonlinear. PGV did not show good correlation with shear force, therefore can not be considered as a reliable index in the damage calculations.

Using shear indices, PGA, A_0 as well as PGA*PGV we can predict damage levels of an ABWR type of Nuclear Power Plant (NPP) buildings immediately after an earthquake, without any complex calculations.

Also, a simple ABWR model was constructed for simulation analyses using records of 2007 Niigata Chuetsu-Oki Earthquake. Soil-structure interaction was not considered, because acceleration records observed at the basement floor levels were available for. The maximum story drift angle was found to be 1/2000 at the Floor 4 of the building No.6 (EW-direction), which shows that the relative displacements are still in a reasonable range, even the recorded PGA levels exceeded the design PGA values.

ACKNOWLEDGEMENT

We appreciate all institutions shown in the reference for providing us strong motion data as well as building data.

REFERENCES

Masuda, A., Kawase, H. (2002). Study on construction of vulnerability function by earthquake response analysis for reinforced concrete buildings. *J. of Structural and Construction Eng., Trans. of AIJ*. **No.558**, 101-107.

Nagato, K., Kawase, H. (2004). Damage Evaluation Models of Reinforced Concrete Buildings Based on the Damage Statistics and Simulated Strong Motions During the 1995 Hyogo-ken Nanbu Earthquake. *Earthquake Engineering and Structural Dynamics*. **Vol.33, No.6**, 755-774.

Citak, S., Kawase H., Fushimi M., Ikutama S. (2004). Relationship of seismic responses and strength indexes of ground motions for a large-scale rigid structure. *Architectural Institute of Japan, Hokkaido annual conference.* 955-956.

Seckin Ozgur CITAK, Hiroshi KAWASE, Shinya IKUTAMA. (2006). Relationship of seismic responses and strength indexes of ground motions for a Large-Scale Rigid Structures. *1st ECEES First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland, 3-8 September 2006.* Paper Number: 718.

Central Disaster Prevention Council (2003). Ground motion acceleration data of Simulated Tokai Tonankai Nankai Earthquake.

Tokyo Electric Power Company (2007). Acceleration data recorded at Kashiwazaki-Kariwa Nuclear Power Station during July 16, 2007 Niigata-Chuetsu-Oki Earthquake (Data CD No.023)

Nuclear Power Engineering Corporation (NUPEC) (1998). The report of nuclear power plant facilities limit characteristics evaluation method.

Earthquake Research Committee (1998). Seismic activity in Japan-Regional perspectives on the characteristics of destructive earthquakes-(excerpt). http://www.hp1039.jishin.go.jp/eqchreng/eqchrfrm.htm

Shabestari, K., T., Yamazaki, F. (1998). Attenuation Relationship of JMA Seismic Intensity using Recent JMA Records. *Proceedings of the 10th. Earthquake Engineering Symposium*, **Vol. 1**, pp. 529-534.

Chopra, A. (1995). Dynamics of Structures. Theory and Applications to Earthquake Engineering. Prentice-Hall.

Wilson, E.L. (2002) Three Dimensional Static And Dynamic Analysis Of Structures. Computers and Structures Inc. Berkeley, California.

Tokyo Electric Power Company (TEPCO) (2007).The report on the analysis of strong ground motion data recorded at Kashiwazaki-Kariwa Nuclear Power Plant during 2007 Niigata-Chuetsu-Oki Earthquake. http://www.tepco.co.jp/cc/press/betu07_j/images/070730d.pdf