NON-LINEAR SEISMIC RESPNSE OF A PWR-TYPE REACTOR BUILDING SIMULATED BY A 3-D FEM MODEL

Yasuhiro KASUGA[1], Atsushi KAMBAYASHI[2], Sadatomo ONIMARU[3], Hideo NAMBA[4], Takehiko KITANO[5], Minoru FUSHIMI[6], Seiya KATAYAMA[7] And Noboru MAEDA[8]

SUMMARY

In Japanese seismic design of nuclear power plants, non-linear dynamic responses are estimated using multi-stick lumped mass models. However, actual buildings have 3-dimesional (3-D) and complex shapes, and there have been few studies on non-linear dynamic behavior of 3-D building models. Therefore, we investigated the dynamic behavior of a reactor building during strong earthquakes by carrying out 3-D non-linear analysis. We also verified the seismic design method of reactor buildings and estimated the seismic safety of a reactor building. A typical Pressurized Water Reactor (PWR) building was selected for this simulation. The building was modeled as exactly as possible using the 3-D finite element method (FEM). We confirmed that the analytical results coincided well with the results of the RC wall test model up to its ultimate state. First, in order to confirm the accuracy of the 3-D FEM model simulation analyses of earthquake records observed at the reactor building were carried out. The 3-D model response was compared with the observed one. Waveforms of acceleration time histories and response spectra computed by the 3-D FEM model corresponded well with the observed data. The 3-D FEM model could accurately represent the dynamic characteristics of the actual building. Next, we calculated non-linear responses against the design earthquake using a 3-D model. A part of the RC wall reached its initial yield point. Finally, we calculated the non-linear dynamic responses against the amplified design earthquake in order to estimate the seismic safety of the reactor building. It was verified that the reactor building has a sufficient earthquake-resistant capacity.

INTRODUCTION

PWR building was selected for this simulation. These buildings consist of O/S, C/V, FH/B, E/B and I/C and these are all modeled by 3-D FEM. RC wall is made of layered shell element, steel is made of beam element and foundation mat is made of solid element. The target buildings in this study are O/S and E/B, and they are modeled by non-linear properties. Other buildings are modeled by linear properties. Figure 1 shows 3-D FEM model. Table 1 shows the numerical data of material properties and building scale used in this study. The ground is modeled by scalar spring elements of 2-horizontal and vertical way and these elements are located under the foundation discretely. Spring values are calculated by assuming rigid foundation on elastic half space, whose Vs is 2340m/sec. The vertical spring stiffness is assumed zero during the building's foundation is uplifted from the ground.

[1] *Takenaka Corporation, Chiba, Japan, E-mail:kasuga.yasuhiro@takenaka.co.jp*
[2] *Takenaka Corporation, Chiba, Japan, E-mail:kanbayashi.atsushi@takenaka.co.jp*
[3] *Takenaka Corporation, Chiba, Japan, E-mail:onimaru.sadatomo@takenaka.co.jp*
[4] *Takenaka Corporation, Tokyo, Japan, E-mail:nanba.hideo@takenaka.co.jp*
[5] *The Kansai Electric Power Co.,Inc, Osaka, Japan, FAX:+81-6-6441-3879*
[6] *The Kansai Electric Power Co.,Inc, Osaka, Japan, FAX:+81-6-6441-3879*
[7] *The Kansai Electric Power Co.,Inc, Osaka, Japan, FAX:+81-6-6441-3879*
[8] *NEWJEC, Osaka, Japan, FAX:+81-6-6243-2774*

OUTLINE OF 3-D FEM MODEL

In Japanese seismic design of nuclear power plants, non-linear dynamic responses are estimated using multi-stick lumped mass models and we design this building to have enough resistance force. However, actual buildings have 3-dimesional (3-D) and complex shapes, and we need to make a detail 3-D model to evaluate non-linear responses accurately. This study will be useful of rational design methods in the future. It is difficult to calculate a large-scale model by non-linear dynamic analysis before because of poor machines, and now, such a large-scale model is being calculated by improved machines. Therefore, we investigated the dynamic behavior of a reactor building during strong earthquakes by carrying out 3-D non-linear analysis. We have two purposes. One purpose is to improve the precision of analysis of non-linear responses using a 3-D model and another is to estimate the seismic safety of a reactor building. First, in order to confirm the accuracy of the 3-D FEM model, simulation analysis of earthquake records observed at the reactor building is carried out and we simulated to refer to a past experiment of static hysteresis load, which model is RC wall [Nagashima, 1979][Ueda and Seya, 1997]. Secondly we calculate non-linear responses against the design earthquake using the 3-D model. Finally, we calculate the non-linear dynamic responses against the amplified design earthquakes in order to estimate the seismic safety of the reactor building.

Table 1 Material properties and building scale

Concrete		Reinforcing bar	
Compressive strength (kgf/cm2)	392	Young’s modulus (kgf/cm2)	2100000
Young’s modulus (kgf/cm2)	294000	Yield strength (kgf/cm2)	3500
Poisson ratio	0.167	Building scale	
Cracking strength (kgf/cm2)	39.2	The number of nodes	3481
Reduction factor of shear modulus	36.0	The number of elements	5102
Reduction factor of compressive strength	0.63	The number of freedom	16515

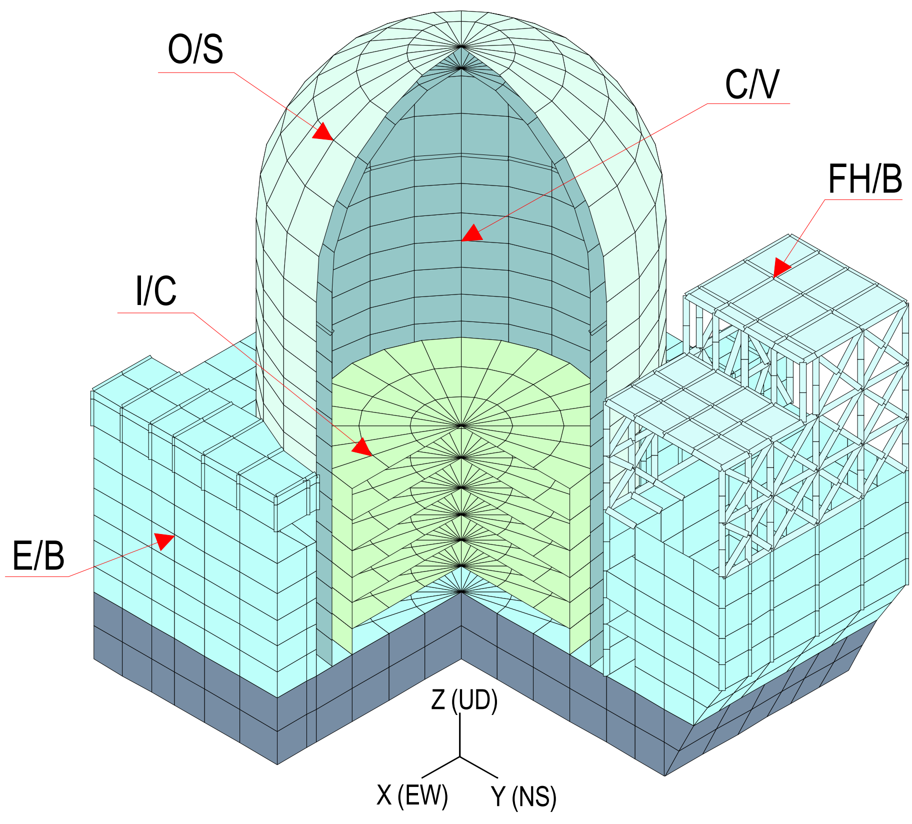


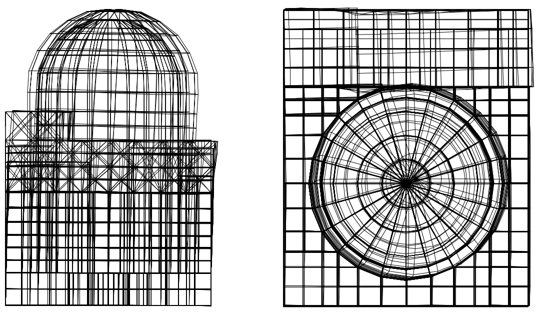
Figure 1 3-D FEM Model

VERIFICATION OF 3-D FEM MODEL

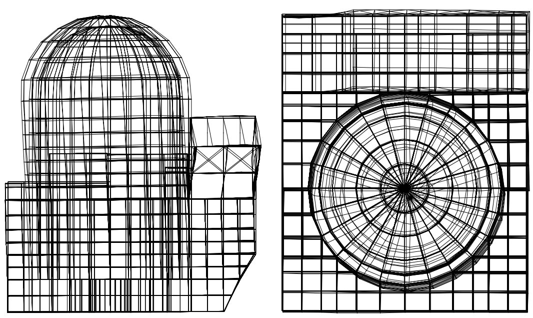
We verified the propriety of 3-D FEM models through O/S to compare 3-D model responses with observed data. Table 2 shows the first frequencies of O/S by forced vibration test and this analysis. The frequencies by this model corresponded generally with the observed data. Figure 2 shows the first eigen modes of O/S in the direction of NS and EW. It is found that each direction of mode is not independent but affects each other. Generally, 3-D shapes like this study cause this behavior. Next, in order to confirm the accuracy of the 3-D FEM model□simulation analyses of earthquake records observed at the reactor building were carried out. The observed earthquake happened near the site, is a small-scale one whose acceleration is about 10gal on the foundation and about 90 gal at the top of O/S. In order to verify the property of this building, we analyzed the building responses with observed wave on the foundation. Here, two-way of horizontal waves and vertical wave were given at the same time. The buildings do not use non-linear property in this chapter because of a small-scale earthquake not damaging these buildings. Figure 3 shows the acceleration time histories and the response spectrum at top of O/S for observed data and analysis one. Waveforms of acceleration time histories and response spectra computed by the 3-D FEM model corresponded well with the observed data. The 3-D FEM model could accurately represent the dynamic characteristics of the actual building.

Table 2 First frequencies of O/S (Hz)

Forced vibration test		Analysis	
NS	EW	NS	EW
5.72	5.74	5.16	5.20

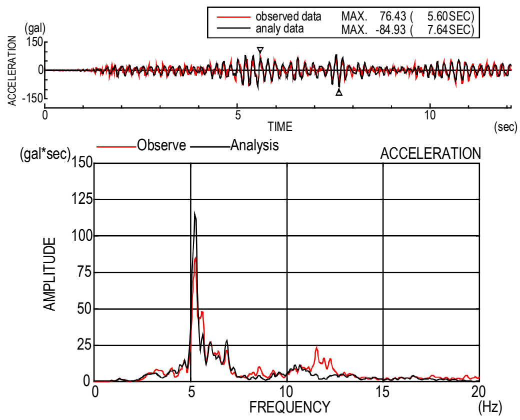


(a) NS (5.16Hz)

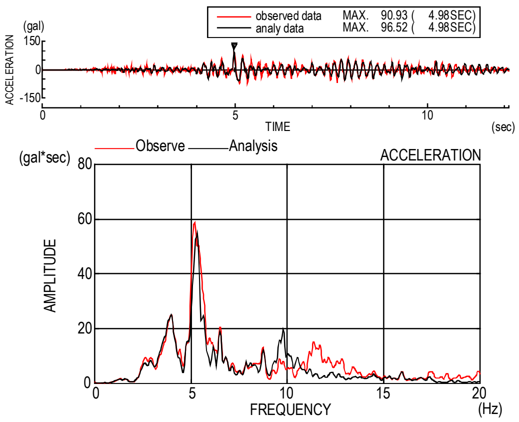


(b) EW (5.20Hz)

Figure 2 First mode



(a) NS



(b) EW

Figure 3 Acceleration time histories and the response spectrum

NON-LINEAR PROPERTIES AND VERIFICATION OF THIS ANALYTICAL METHODS

Figure 4 shows the element, which is composed of concrete plates and reinforcement planes. Concrete elements are divided into several layers of the same thickness, and the non-linear behavior varies in each layer. The non-linear properties of material are considered only in plane direction. Under compressive stress field, the Drucker-Prager's law is assumed as elasto-plastic material properties (Figure 5). Figure 6 shows the relationship between axial stress and strain in the cracking direction. The maximum strength parallel to cracking direction decrease by a half compared to the strength before cracking [Fafitis and Shah, 1985]. Rules for hysteresis loops of cracked concrete are shown in Figure 7[Stevens and Collins, 1987]. The hysteresis loops is composed of a series of connected lines. hysteresis rule in compressive stress is expressed by the plastic strain proposed by Karsan[Karsan and Jirsa, 1969]. Degradation of compressive strength under cyclic loading is based on the Yamada's rule[Yamada, 1977]. Shear stiffness decreases as a function of strain perpendicular to the cracking direction. The degradation relations refer to the Yamada and Aoyagi rule as shown in Figure 8[Yamada and Aoyagi, 1983]. To verify this analytical methods, we simulated to refer to a past experiment of static hysteresis load, which model is RC wall required this building as principal elements[Nagashima, 1979][Ueda and Seya, 1997]. We could validate this non-linear analytical method because of good agree to restoring force characteristics.

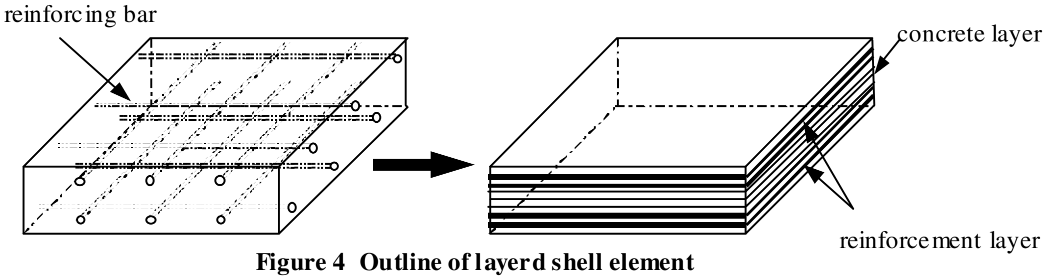


Figure 4 Outline of layerd shell element

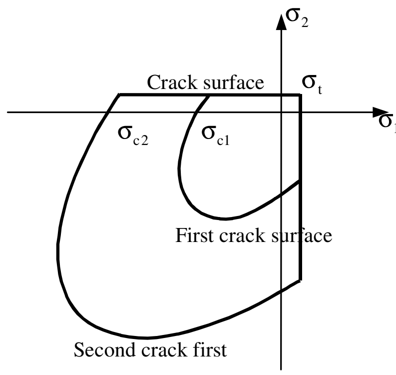


Figure 5 crack and yield surface

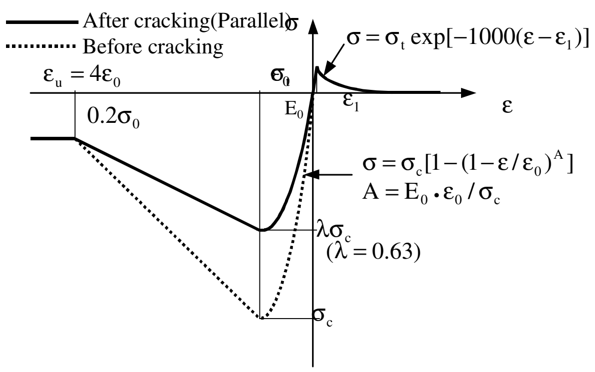


Figure 6 Envelop curve of stress-strain of concrete

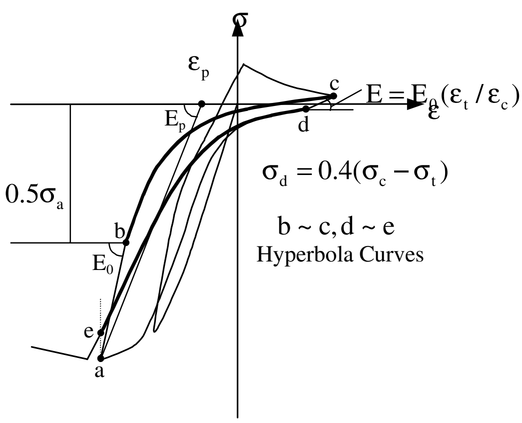


Figure 7 Stress-strain of concrete under cyclic loading

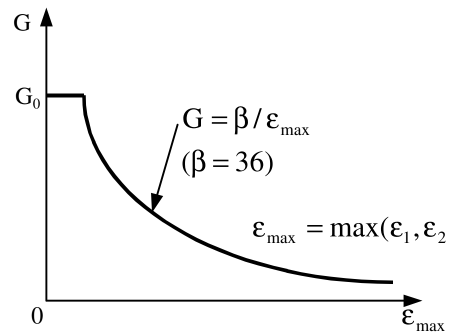


Figure 8 Shear rigidity after cracking

RESULT OF ANALYSIS AGAINST THE DESIGN EARTHQUAKE

We calculated non-linear responses against the design earthquake using a 3-D model. Figure 9 shows the design earthquake. The RC walls were modeled using layered shell elements of concrete and reinforcing bars taking their non-linearity into account. Figure 10 shows the response waves at the top and the middle of O/S. Figure 11 shows the crack state of O/S. Many cracks are happened the lower part of O/S and all of reinforcing bars do not yield. The responses at NS direction almost never have a difference of that at EW direction about O/S. Figure 12 shows the skeleton curve of static analysis and plotted the maximum story drift ratio in the middle height of O/S under the design earthquake. This value shows 0.7/1000 (rad) and generally, ultimate shear strain value for design is 4/1000. Therefore it is found that this buildings have enough safety margin. It is concluded that many cracks are happened the lower part of O/S and all of reinforcing bars do not yield, but tension strain of the concrete still is small. We confirmed to have enough safety margins for the element's strain and the story drift ratio against the design earthquake.

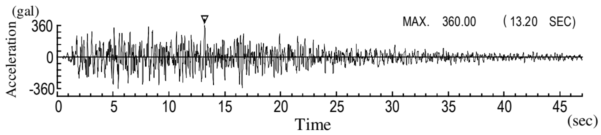


Figure 9(a) Design earthquake

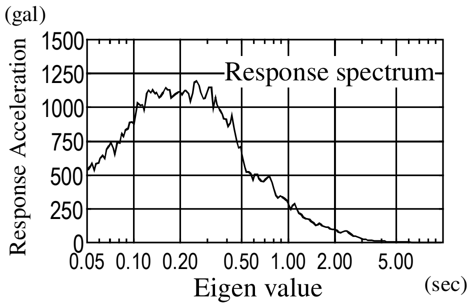
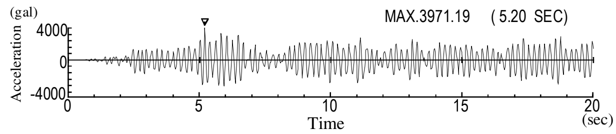
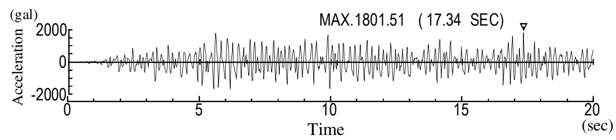


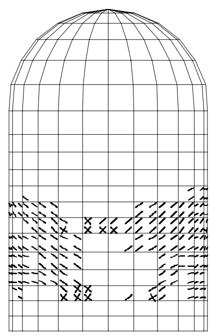
Figure 9(b) Response spectrum



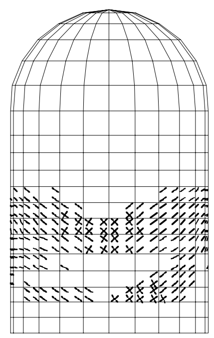
(a) Top



(b) middle (EL36.5m)
Figure 10 Acceleration time histories



(a) NS analysis (East side output)



(b) EW analysis (North side output)
Figure 11 Crack state

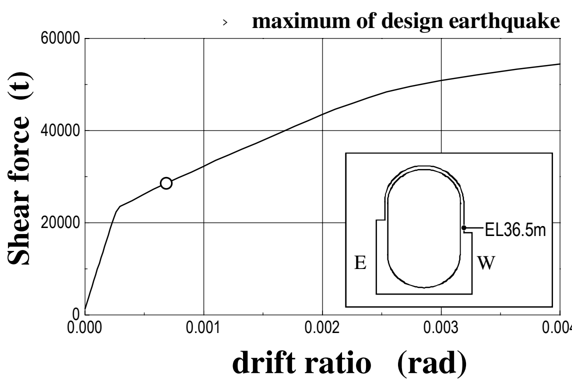
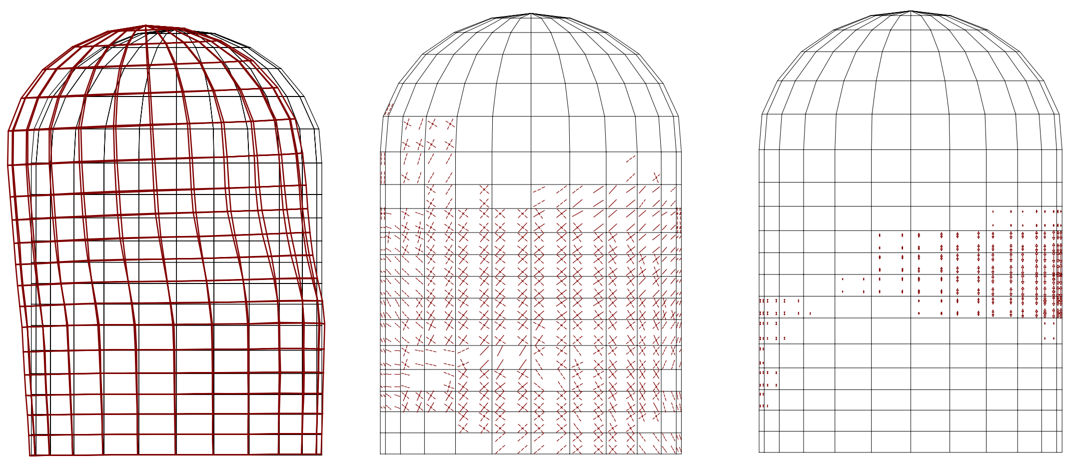


Figure 12 Skelton curve of static analysis

RESULT OF ANALYSIS AGAINST THE AMPLIFIED DESIGN EARTHQUAKES

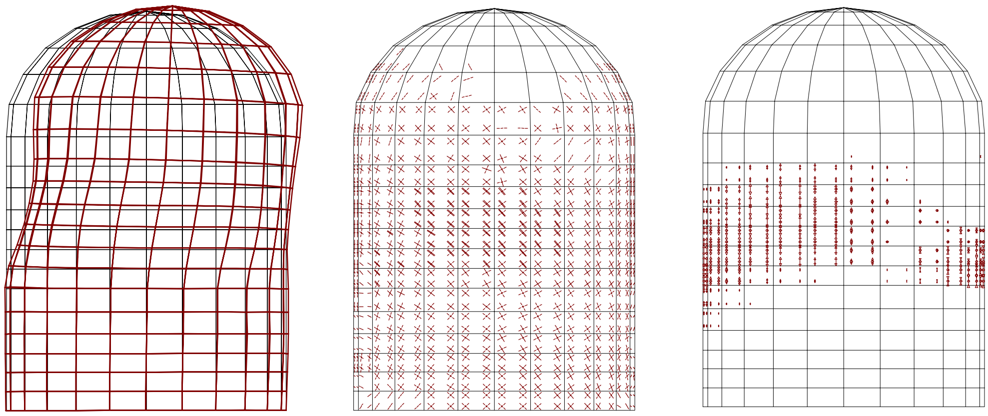
We calculated the non-linear dynamic responses against the amplified design earthquake in order to estimate the seismic safety of the reactor building. Result of the analysis by three times of design earthquake is shown in Figure 13 and by five times of design earthquake is shown in Figure 14. Figure 13(a), 14(a) shows the displacement of O/S. Figure 13(b), 14(b) shows the cracks of O/S and Figure 13(c), 14(c) shows the vertical reinforcing bar. Here the displacement and the vertical reinforcing bar state do not show the maximum values but show that (13.5sec) near the maximum and the crack state shows the final response time. Where, all of their situations are printed at Eastside of O/S. The cracks are happening almost elements expect of upper of O/S (Fig.13(b), Fig.14(b)). It was found that the story drift ratio is larger in the middle of O/S between EL43.6m and EL47.3m. The vertical reinforcing bars were yielded right side partly at O/S in Fig.13(c) and yield range is extended by five times design earthquake in Figure 14(c). Many cracks are happened at E/B wall in Figure 15 but the vertical reinforcing bars are not yielded. A compressive failure of concrete caused ultimate one was not happened at O/S and E/B. Figure 16 shows the story drift ratio and acceleration by one, three, five times of design earthquake. East Side of O/S is printed. The maximum story drift ratio is 0.7/1000(EL32.8m) by design earthquake, 3.0/1000(EL43.6m) by three times of that and 5.0/1000(EL47.3m) by five times of that. It was found that the yield range extended by the cracks is increasing as design earthquake is amplified. Figure 17 shows the uplifted displacement distribution on the ground at the time that the most northern point shows the maximum value by three and five times of design load. This reactor building is not uplifted on the ground by design earthquake, but uplifted displacement on the ground is 0.9cm by three times of design earthquake, and that is 2.5cm by five times of that. The proportion of attachment on the ground is 45% by three times of that, and 20% by five times of that. We adopt discrete spring elements to estimate the ground but it will be needed further study to model the ground accurately against a large uplifted displacement. All in all, it is reasonable to suppose that increases of uplifted displacement by each of design earthquake reflect a change of response of reactor building.

Figure 18 shows the relation of maximum story drift ratio to the magnification of design load. Here, the location to take maximum values differs at each design earthquake (cf.Fig16). One find to emerge is that the story drift ratio is linear related with the magnification of design load. The maximum value exceeds 4/1000 to be often used as ultimate shear strain under the design criterion, but we can not see the compressive failure of concrete caused ultimate one. This result agrees to the result of experiment by cylindrical shell, because this experiment does not reach an ultimate failure when the shear strain reaches 5/1000[Matsumura, 1987][Tanaka, 1987]. Here, it should be noted that the story drift ratio includes rotary motion of building and bending displacement of frames and that the shear strain is smaller than the story drift ratio.



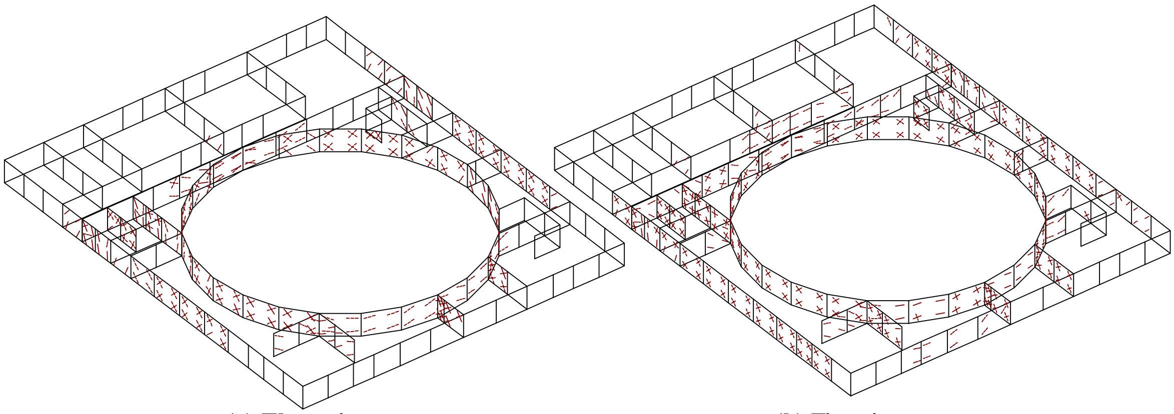
(a) Displacement **(b) Crack state** **(c) Vertical Reinforcing bar**

Figure 13 Response analysis by three times of design earthquake



(a) Displacement (b) Crack state (c) Vertical Reinforcing bar

Figure 14 Response analysis by five times of design earthquake



(a) Three times (b) Five times

Figure 15 Crack state of E/B

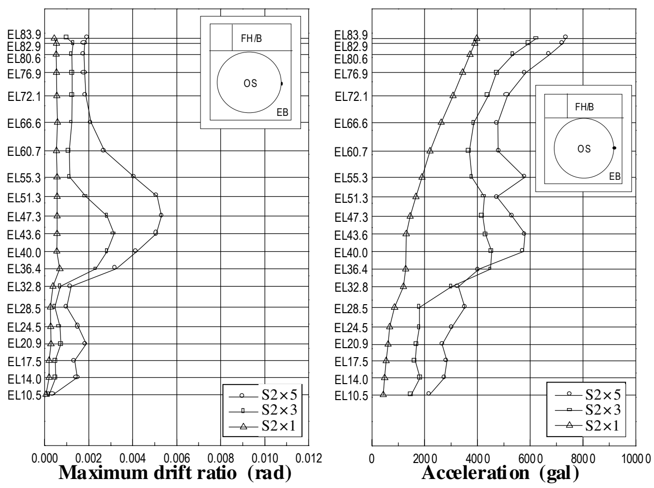


Figure 16 Maximum drift ratio and acceleration

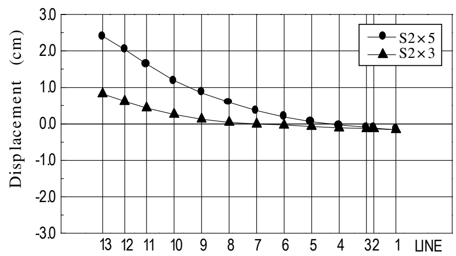


Figure 17 Uplift displacement

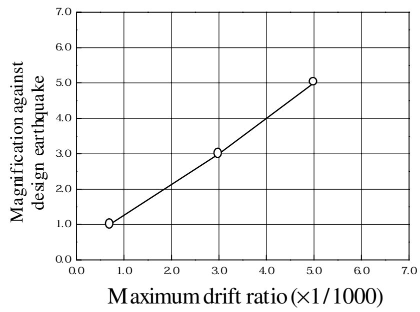


Figure 18 A relation between maximum drift ratio and the magnification of design load

CONCLUSIONS

In order to confirm the accuracy of the 3-D FEM model simulation analyses of earthquake records observed at the reactor building were carried out. Waveforms of acceleration time histories and response spectra computed by the 3-D FEM model corresponded well with the observed data. The 3-D FEM model could accurately represent the dynamic characteristics of the actual building.

We calculated non-linear responses against the design earthquake using a 3-D model. A part of the RC wall reached its initial yield point, and all of reinforcing bars do not yield. We confirmed to have enough safety margins for the element's strain and the story drift ratio the design earthquake.

We calculated the non-linear dynamic responses against the amplified design earthquake in order to estimate the seismic safety of the reactor building. The compressive failures do not happen against five times of design earthquake. Therefore it was verified that RC walls of the reactor building have some earthquake-resistant capacity against five times of design earthquake at least.

Current design criterion is considered that ultimate shear strain is 4/1000, and the half of that is allowable design value. A part of story drift ratio exceed 4/1000 in this study, but we can not found the ultimate damages. Therefore we can say that current seismic design has enough safety margin.

REFERENCES

Fafitis A. and Shah S. P. (1985), "Lateral Reinforcement for High-Strength Concrete Columns", *ACI Special Publication*, No, SP-87, pp213-232.

Karsan, I.D and Jirsa, J.O. (1969), "Behavior of Concrete under Compressive Loading", *Journal of Structual Division, ASCE*, 12, pp. 2543-2563.

Mastumura T., et al. (1987), "Allowable limit of shear walls of reactor buildings", *AIJ*.

Nagashima H., Ueda K., et al. (1979), "Cyclic loading tests of cylindrical RC shell", *Summaries of Technical Papers in Kinki Area, AIJ*, pp.97-pp.104.

Stevens, I.D., Uzumeri, S.M. and Collins, M.P. (1987), "Analytical Modeling of Concrete Subject to Monotonic and Reversed Loading", *Department of Civil Engineering, Publication* No.87-1, University Toronto.

Tanaka H., et al. (1987), "Evaluation method for restoring force characteristics of RC shear walls of reactor buildings", *AIJ*.

Ueda M., Seya H., Omiya Y., Taniguchi H., Kambayashi A. (1997), "Nonlinear analysis on RC shear wall shaking table test", *Transaction of the 14th International Conference on Structural Mechanics in Reactor Technology*.

Yamada, M., Kawamura, H and Morishita, H. (1977), "Study on plastic fatigue of concrete", *Summaries of Technical Papers in Kinki Area, AIJ*, pp.21-24.

Yamada and Aoyagi. (1983), "Shear transfer model for cracking faces", *Conference on Shear of Concrete in Analytical Study* .