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THE BEHAVIOR OF THE REACTOR BUILDING AT FUKUSHIMA DAI-ICHI NUCLEAR POWER PLANT DURING THE GREAT EAST JAPAN EARTHQUAKE

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ABSTRACT: The GREAT EAST JAPAN EARTHQUAKE which occurred on 11 March 2011 struck Fukushima Dai-ichi Nuclear Power Plant and observation records were obtained at the base mat of reactor buildings. The simulation analysis of the reactor building using observed records shows that the whole building response is supposed to be basically elastic during the earthquake. In order to analyze this result, comparison of original design seismic load and that of simulation analysis based on the observation records was carried out.

Key Words: the Great East Japan Earthquake, Nuclear Power Plant, Reactor Building, Soil-Structure Interaction, Seismic Design

INTRODUCTION

The GREAT EAST JAPAN EARTHQUAKE which occurred on 11 March 2011 struck Fukushima Dai-ichi Nuclear Power Plant (NPP) and thus observation records were obtained at the base mat of reactor buildings. The purpose of this report is to show the behavior of the reactor buildings under the earthquake by carrying out the seismic response analysis using the observation records.

Although Fukushima Dai-ichi NPP suffered devastating damage by hydrogen detonation which results from the loss of cooling capacity because of station black out caused by the tsunamis, this report is focused on the effect of the earthquake and the effect of the tsunamis is not intended.

Seismometers are installed in each reactor buildings and seismic observation had been conducted. And the acceleration time histories on the base mat of reactor buildings were obtained. Maximum acceleration values from the observation records are shown in Table 1. And maximum response acceleration values to Design Basis Ground Motion (DBGM) are also shown in Table 1. DBGM is the ground motion for seismic safety analysis based on “Regulatory Guide: Reviewing Seismic Design of Nuclear Power Reactor Facilities (2006)”. Maximum acceleration values observed at Unit #2, #3 and #5 during the earthquake were bigger than those of DBGM. Particularly the biggest maximum acceleration value was observed in the east-west direction of Unit #2.

Table 1 Observation records at the base mat of Reactor Buildings

	Maximum acceleration value from observation records (Gal)			Maximum response acceleration value to Design Basis Ground Motion (Gal)		
	NS	EW	UD	NS	EW	UD
Unit #1	460	447	258	487	489	412
Unit #2	348	550	302	441	438	420
Unit #3	322	507	231	449	441	429
Unit #4	281	319	200	447	445	422
Unit #5	311	548	256	452	452	427
Unit #6	298	444	244	445	448	415

exceeded DBGM

Estimation of Behavior of Reactor Buildings Using the Observation Records

Method of the Earthquake Response Analysis

The method of simulation analysis in this report is basically elastic response analysis using observation records on the base mat of reactor building.

Seismic response of reactor building is calculated by transfer functions from the base mat to each floor inputting observation records into the base mat of seismic response analysis model. Figure 1 shows the outline of elastic response analysis in horizontal direction.

Observation records on the base mat are input for the analyses for the reactor buildings. The response at each floor is calculated as shown in the flowchart.

First, the dynamic soil responses at the embedded part of the building are calculated from one dimensional wave propagation theory.

Second, the dynamic soil responses are applied to the building as the input motions for the simulation analysis in horizontal direction. As a result, the responses of the building are evaluated taking into account the soil structure interaction. Based on the above, the transfer functions from the base mat to each floor are calculated.

Third, the earthquake responses of individual parts of the building are obtained by multiplying the transfer function calculated as above by the Fourier spectrum obtained from the Fourier transform of the observation records of the base mat.

If the response of the building exceeded elastic range in above analysis, elasto-plastic response analysis is adopted. As shown in Figure 2, elasto-plastic response analysis is conducted based on the input motions of the soil that almost reproduce observation records of the base mat.

In the case of Unit #2, the response partially exceeded elastic range and thus the elasto-plastic response analysis was adopted.

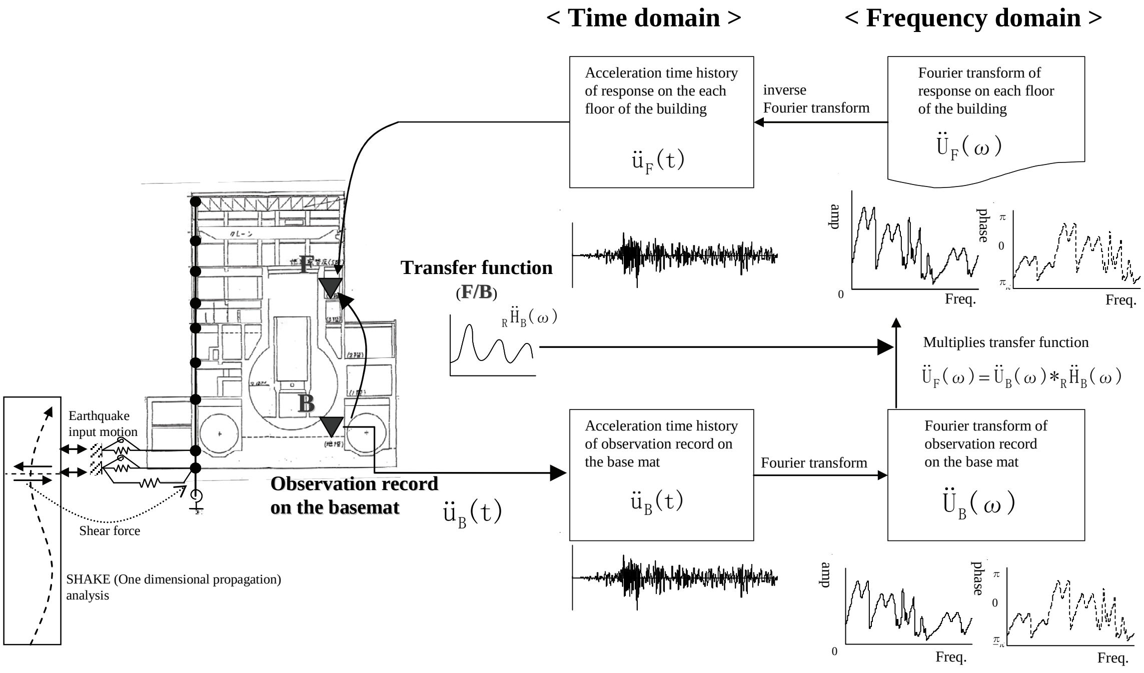


Fig. 1 Outline of elastic response analysis

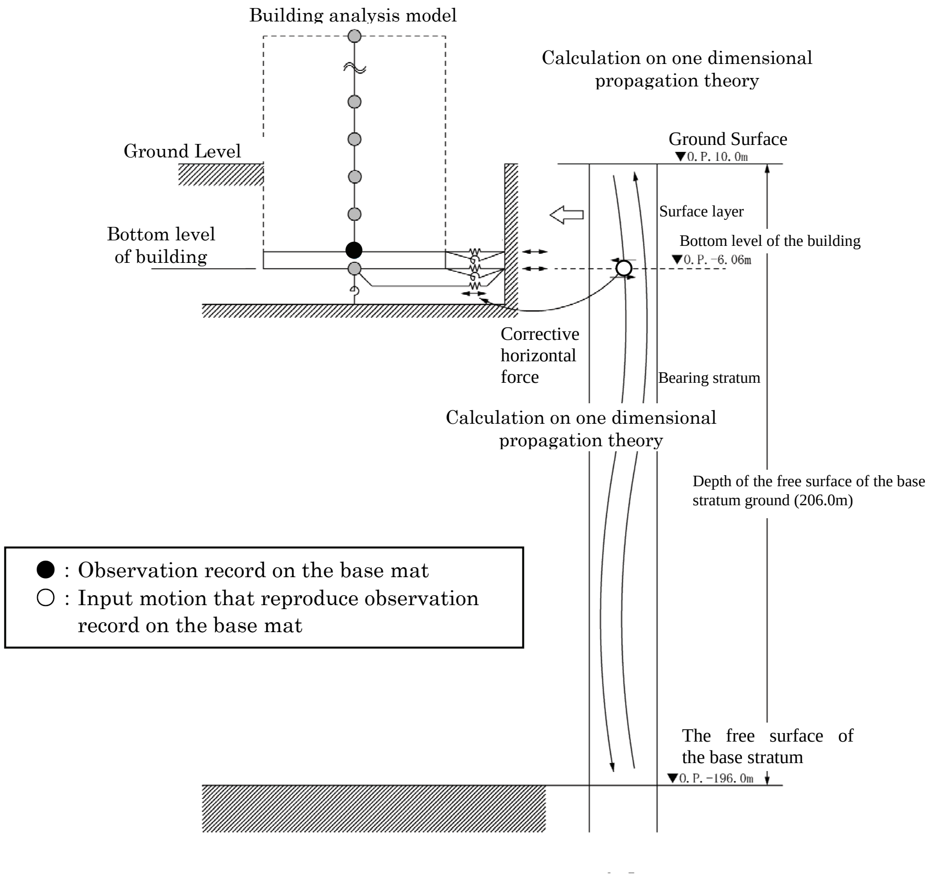


Fig. 2 Outline of elasto-plastic response analysis

Detail of the Earthquake Response Analysis Model

Reactor building is reinforced concrete structure, placed on bedrock and made as rigid as possible. Figure 3 shows the seismic response analysis model. So-called embedded Sway-Rocking model, which is composed of the combination of a building model and soil springs, is applied to the analysis. The building model is a stick model with a concentrated mass located at each floor, taking into account bending and shear stiffness. Young's modulus of concrete is evaluated from the measured compressive strength of test pieces sampled from actual building walls.

Damping factor of the building was determined to be 5%. The nonlinear characteristics of shear walls are determined by the method specified in the guideline for seismic design of nuclear power plants “JEAC 4601-2008”.

The soil springs for the base mat are employed to consider soil structure interaction. The rotational and horizontal soil springs underneath the base mat were decided based on the admittance vibration theory. And those of beside the base mat were based on the Novak’s method. Soil springs were calculated as complex stiffness depending on frequency as shown in figure 4. Spring constant was approximated by maximum value of real part. And damping coefficient was approximated by adopting the slope of the line through the origin and the point of the imaginary part according to the first natural frequency of soil-structure interaction model.

The soil properties were taken into account the strain dependency of the stiffness and damping. Degrading stiffness and damping value according to the strain level were considered and adopted in the evaluation of soil spring.

Analysis condition is shown in Table 2.

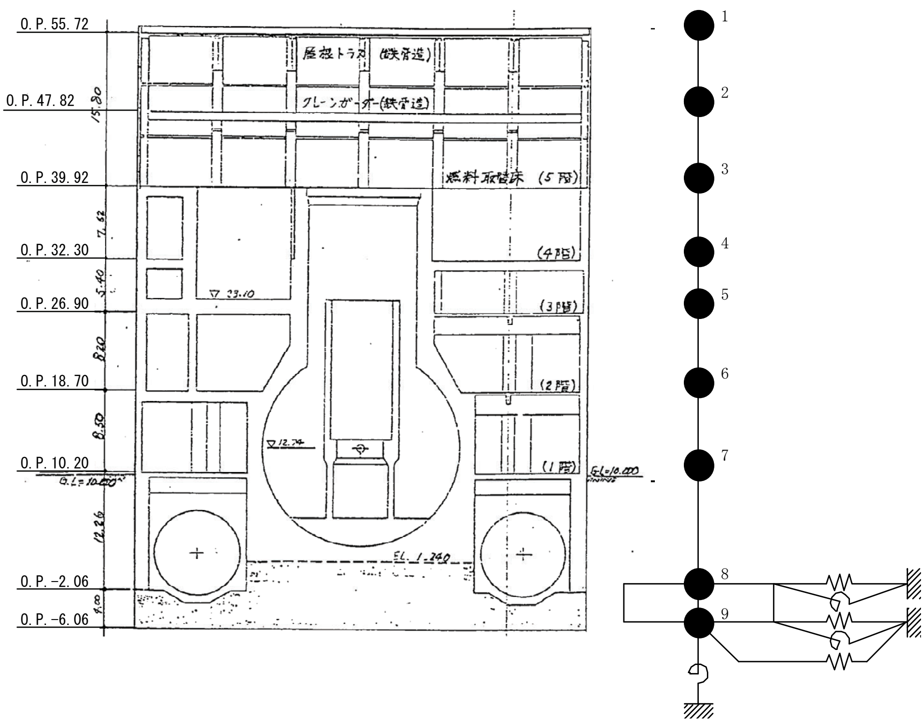
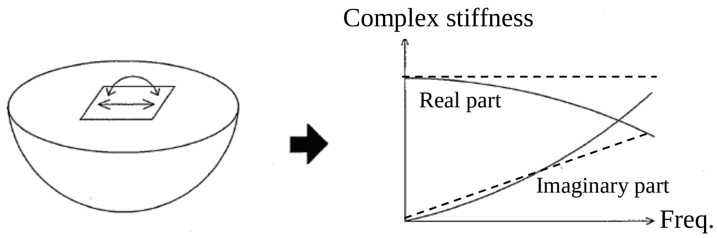
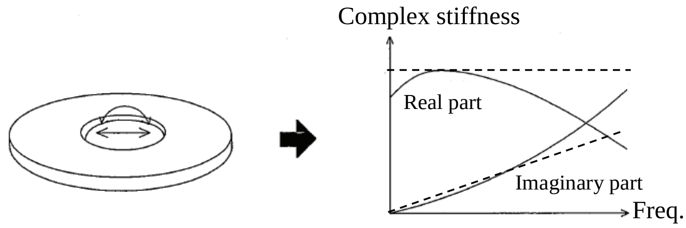


Fig. 3 Seismic response analysis model



Vibration admittance theory



Novak’s method

Fig. 4 Evaluation method of soil springs

Table 2 Analysis condition and physical value

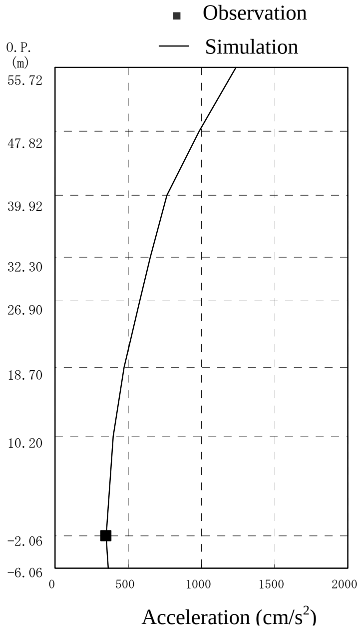
Analysis condition for structure		
Stiffness	Young's modulus of concrete	Based on material testing $Ec=2.57\times10^4 N/mm^2$ ($35.0N/mm^2$) ( ) : compressive strength
Damping		5% (Based on Strain energy proportional method)

Analysis conditions for soil-structure interaction		
Soil spring	Underneath the base mat	Based on Vibration admittance theory (horizontal and rotational)
	Beside the base mat	Based on Novak's method (horizontal and rotational)
Soil properties		Stiffness and damping factors based on maximum strain levels
Uplift of the base mat		Nonlinear

Seismic Response Analysis Result

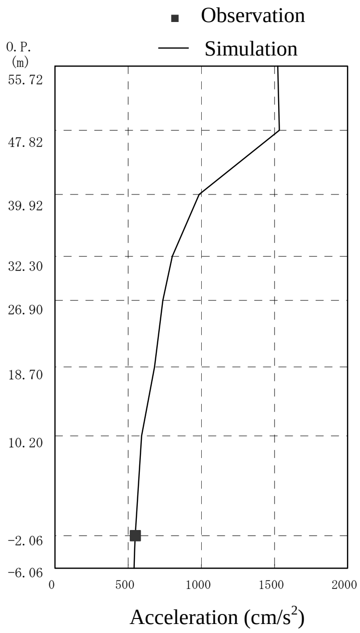
The analytical results based on above-mentioned procedures are shown in Figure 5 and Figure 6. Figure 5 shows the distribution of maximum response acceleration in height for Unit #2 obtained from the analysis. Figure 6 shows the maximum shear stress and strain plotted on nonlinear characteristics of shear wall on each floor.

The maximum shear strain obtained from the analysis is 0.43×10^3 of the east-west shear wall on 5^{th} floor. All shear walls only except the above are within elastic range. Therefore, the analysis shows that the whole building response is supposed to be basically elastic under the earthquake.



(cm/s²)

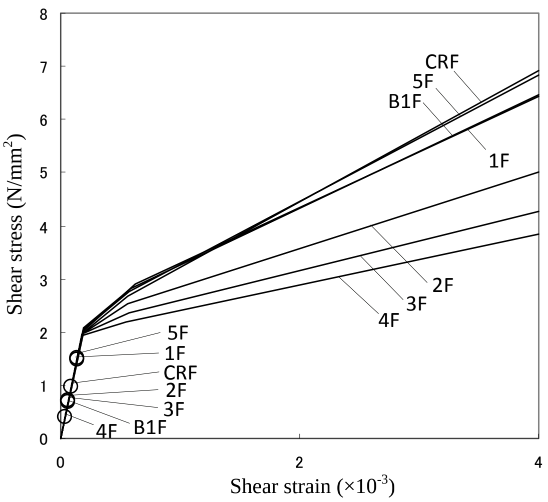
O.P. (m)	Obs.	Sim.
55.72		1236
47.82		984
39.92		764
32.30		651
26.90		579
18.70		472
10.20		397
-2.06	348	349
-6.06		364



(cm/s²)

O.P. (m)	Obs.	Sim.
55.72		1523
47.82		1534
39.92		985
32.30		802
26.90		737
18.70		680
10.20		593
-2.06	550	548
-6.06		541

Fig. 5 Maximum acceleration



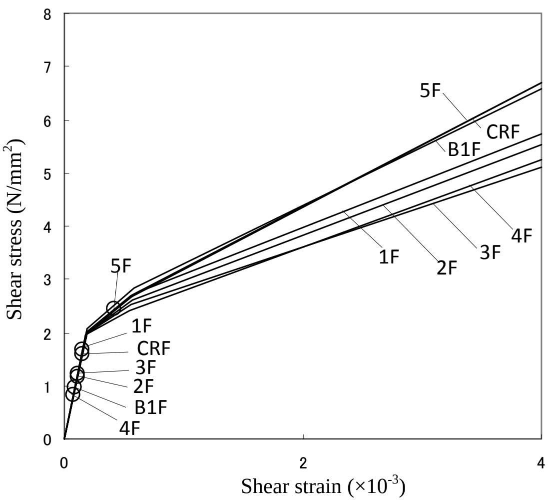


Fig. 6 Maximum shear strain

Comparison of seismic loads condition

Comparison of Seismic Loads

Maximum acceleration value of the observation record on the base mat of Unit #2 under the Great East Japan Earthquake exceeded that of DBGM. However, the simulation analysis shows the building response is basically within elastic range.

In order to analyze the reason, original design seismic load was compared to results of simulation analysis.

Philosophy of Original Seismic Design

Philosophy of original seismic design is discussed below.

In order to decide the design seismic loads, static and dynamic seismic loads are considered and the design seismic loads are decided so as to be bigger than above-mentioned two seismic loads. And the reactor building is designed so as to be within allowable stress.

Amount of rebar arrangement is calculated assuming that only reinforcing bars bear all shear stress caused by design seismic load. On the other hand, shear wall thickness is decided in such a way that shear stress to some extent can be covered by only concrete. In addition, shielding performance is also taken into account in deciding wall thickness. Figure 7 shows the seismic design flow mentioned above.

In order to decide dynamic seismic load for Unit #2, dynamic responses calculated from the past observation records such as ELCENTRO (1940.5.18, NS direction) and TAFT (1952.7.21, EW direction) are considered. The maximum acceleration values of the records were normalized at 180 gal. According to the original design documents, Taft and Elcentro were selected by reason of typical strong ground motion record. Furthermore, the seismic response analysis model at that time was different from the present one in soil dissipation damping and so on.

Static seismic load is decided so as to be three times as big as that of the Building Standard Act at that time. Thus shear coefficient in the first basement was 0.48.

As far as Unit #2 reactor building is concerned, the original design seismic load was mainly determined by dynamic seismic loads.

As explained above, the original design seismic load depended greatly on engineering judgment.

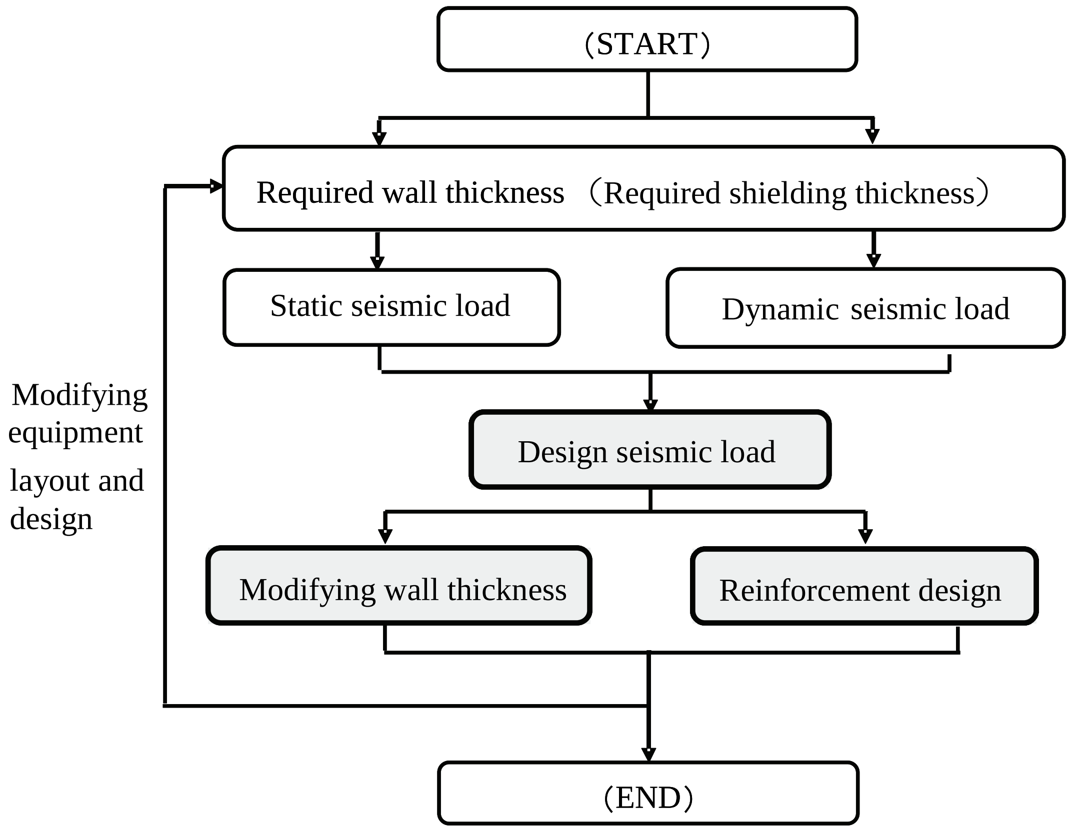


Fig. 7 Seismic design flow

Comparison Result of Seismic Loads

Figure 8 compares shear stress between seismic loads such as design seismic load, static seismic load, dynamic seismic load and simulation analysis results. According to figure 8, the shear stress acting to unit #2 reactor building under this earthquake is almost equivalent to that of design seismic load.

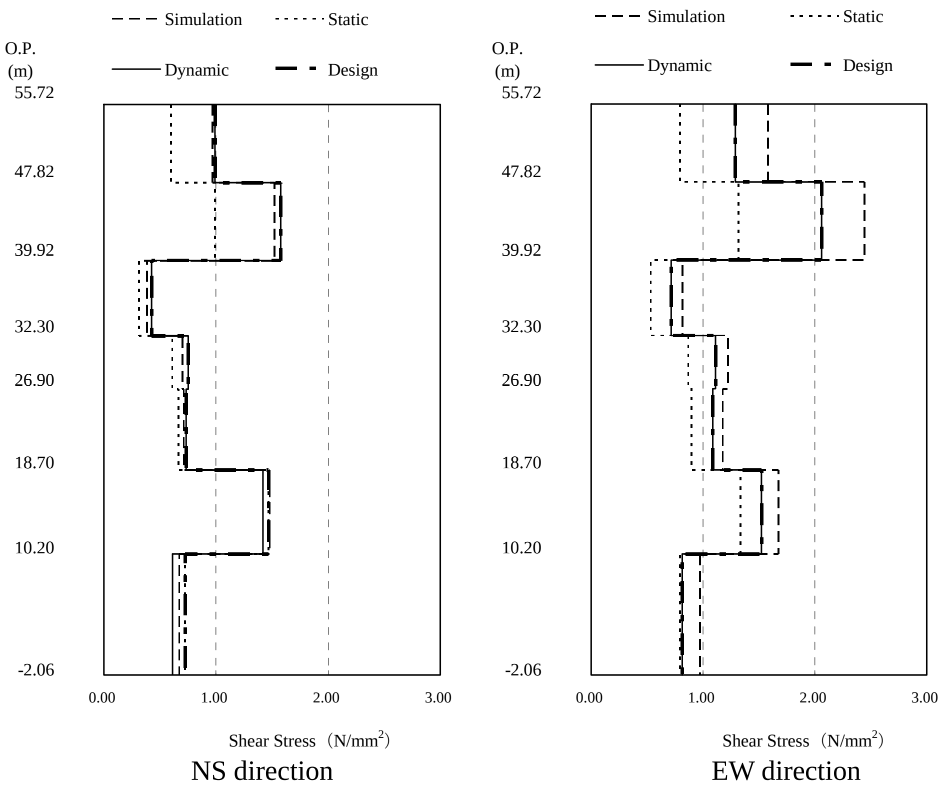


Fig. 8 Comparison of original design sheer stress and that of simulation analysis

Conservatism of Original Dynamic Response Analysis Model

Ground motions for original dynamic response analysis were Taft and Elcentro whose maximum accelerations were normalized at 180 gal. However, the shear stress caused by the motions was almost equal to that of response analysis results using observation records during the Great East Japan Earthquake, which were 550 gal at a maximum on the base mat in reactor building Unit #2.

In order to analyze the reason, soil spring evaluation which were clearly different between original dynamic response analysis and this simulation analysis is focused and examined.

Table 3 shows the result of comparison. The value of soil spring stiffness and damping in the east-west direction are shown in Table 3.

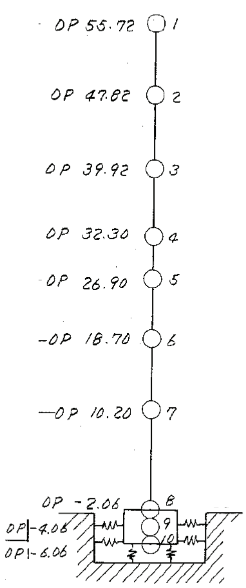
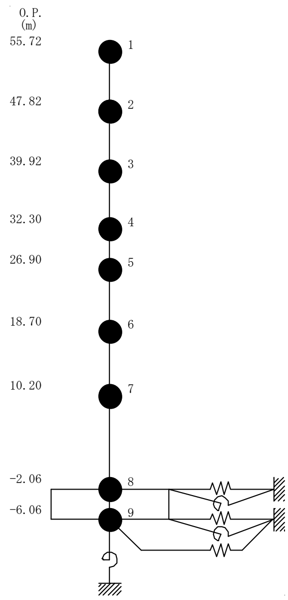
Horizontal stiffness value besides the base mat in original design was almost ten times bigger than that of simulation analysis. And rotational stiffness was not considered in original design.

Damping value in original design was 5% at any frequency, although that can be evaluated bigger by far if soil dissipation damping due to soil-structure interaction were considered. This is thought to be that evaluation method of soil dissipation damping had not been established in those days. On the other hand, in the simulation analysis, approximate damping coefficient which depends on frequency is considered in order to evaluate soil dissipation damping properly. Thus, the damping coefficient besides base mat was calculated based on Novak's method and that of underneath the base mat was based on vibration admittance theory.

The difference in evaluation of soil springs as above was supposed to be one of the reasons why original dynamic analysis model was conservative so as to give bigger shear stress in analysis results.

Because design seismic load was originally evaluated using conservative model, the shear stress distribution by original response analysis model was thought to be as same level as that of simulation analysis although maximum acceleration of ground motions for original dynamic response analysis was much smaller than that of observation records on the base mat during the Great East Japan Earthquake.

Table 3 Comparison of soil springs evaluation

		Original Design	Simulation
Analysis model		 (Elastic response analysis)	 (Elasto-Plastic response analysis)
Soil Spring Besides Base mat	Stiffness	Horizontal: 5.34×10^7kN/m Rotational: Not considered	Horizontal : 2.40×10^6kN/m Rotational : 1.47×10^6kN/m
	Damping	5%（at any frequency）	[Damping coefficient] Horizontal:4.81×10^5kNs/m Rotational:8.97×10^7kNm/rad
Soil Spring Underneath Base mat	Stiffness	Horizontal: 4.36×10^7kN/m Rotational: 3.90×10^{10}kNm/rad	Horizontal : 5.41×10^7kN/m Rotational : 5.24×10^{10}kNm/rad
	Damping	5%（at any frequency）	[Damping coefficient] Horizontal: 2.00×10^6kNs/m Rotational: 5.74×10^8kNm/rad

CONCLUSIONS

The maximum acceleration observed at Fukushima Dai-ichi NPP during the Great Japan Earthquake exceeded the maximum response acceleration of DBGM. However, the result of simulation analysis using the observation records shows that the whole building response was basically elastic under the earthquake.

In order to analyze the reason why the response was almost elastic range, the original seismic load was compared to the result of simulation analysis. In determining the design seismic load, both static and dynamic seismic loads are taken into account. The original dynamic load for Unit #2 Reactor Building was based on the response analysis using the past observation records at other sites such as Taft. The static seismic load is three times as big as that of the Building Standard Act at that time. As for Unit #2 reactor building, design seismic load was mainly determined by dynamic seismic load. The comparison of original design with simulation result shows that shear stress supposed in original design and that of simulation analysis were almost same level. This is thought to be that the soil dissipation damping was scarcely considered in original design model which was Sway-Rocking model generally employed in those days.

Thus, although the maximum acceleration value of the ground motion for original design was smaller than that of the Great East Japan Earthquake, the analysis model was so conservative that the shear stress considered in original design was almost as same level as that in the simulation analysis for this earthquake. This is why the whole building response is basically elastic under the earthquake.

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