THE REPORT ON THE INVESTIGATION

INTO THE CURRENT SEISMIC SAFETY AND REINFORCEMENT

OF THE REACTORS

AT FUKUSHIMA DAIICHI NUCLEAR POWER STATION (NO. 1)

May 2011

The Tokyo Electric Power Company, Inc.

Index

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1. Introduction

Per the instruction, “Submission of report based on the article 67, clause 1 of the Act on the Regulation of Nuclear Source Material, Nuclear Fuel Material and Reactors” (April 13, 2011), this report describes the results of the investigation into the current status of seismic safety and reinforcement of the reactor buildings at Fukushima Daiichi Nuclear Power Station.

This report (No.1) contains the assessment results of Unit 1 and 4 precedently. The assessment results of the other units will be submitted when the investigation is finished.

2. Investigation methodology for the seismic safety assessment

(1) Unit 1 Reactor Building

The upper part of Unit 1 Reactor Building above the operation floor on the 5th floor exploded due to an apparent hydrogen explosion on March 12, 2011 the day after the Tohoku-Pacific Ocean Earthquake. Meanwhile, there is no damage to the floors below the 5th floor of the Unit 1 Reactor Building, unlike Units 3 and 4. It is presumed that the reason leading to this type of failure is that the wall of this type of structure of Unit 1 above the 5th floor, which is constructed out of an steel framework structure fixed with a steel plate, is very weak against pressure from the inside. It is estimated that it is this portion that initially collapsed resulting in a release of inside pressure, so that the structure below the 5th floor remained mostly intact. This information was reflected into the Mass System Model and the Time Transient Response Analysis by Design Basis Ground Motion (Ss) was implemented in order to study whether or not the seismic wall was capable of reaching the peak condition of shear failure.

(2) Unit 4 Reactor Building

Damage of the Unit 4 Reactor Building was confirmed on 15th March 2011. At this stage, it has not yet been determined what process led to the failure since there are no video shots or other images capturing what transpired when the failure occurred. Unlike Unit 1, the structure type of Unit 4 is a reinforced concrete structure, whose wall resistance is assumed to be stronger against inside pressure. However, most of the roof slab and walls blew off, leaving only the frame structure of the pillar and beam, and the roof torus. Furthermore, most of the walls on the 4th floor and part of the ones on the 3rd floor were damaged. Thus, as for Unit 4, the walls below the 5th floor were damaged, unlike Unit 1, so that this information was reflected into the Mass System Model and the Time Transient Response Analysis by Design Basis Ground Motion (Ss) was implemented in order to generally assess whether or not the seismic wall is capable of reaching the peak condition of shear failure. After the general assessment, the sectional assessment, including an assessment of the Spent Fuel Pool, via a 3 dimensional FEM analysis was implemented. The combined assessment with the temperature load and other factors was also conducted by inputting the maximum number gained from the Time Transient Response Analysis as the seismic load.

3. Investigation results from the seismic safety assessment

(1) Unit 1 Reactor Building

As a result of the Time Transient Response Analysis utilizing the Design Basis Ground Motion (Ss), the share strain generated in the seismic wall that remained below the 5th floor was 0.12 x 10^-3 at most, much lower than the evaluation standard value, 4 x 10^-3, which means that the seismic safety was evaluated as fully satisfying the safety standard. (The analysis resulted in the situation substantially within elasticity range.) Therefore, the seismic safety assessment concluded that there was no impact to key facilities in terms of seismic safety such as the Reactor Pressure Vessel, the Primary Containment Vessel (PCV), the Spent Fuel Pool and so on.

(Attachment–1)

Furthermore, quoting from “The report on the implementation of a measure to flood the primary containment vessel to the upper area of the fuel range in Unit 1 of Fukushima Daiichi Nuclear Power Station” reported on May 5th, 2011, there were no major differences between the results in the case of flooding the PCV and the results of this seismic safety assessment. This indicated that the impact will be minor even though the distribution of weight has somewhat changed. In addition, it will be no major problem if the water level in the PCV reaches the target level though it has not been achieved yet.

(Attachment-2)

(2) Unit 4 Reactor Building

As a result of the Time Transient Response Analysis utilizing the Design Basis Ground Motion (Ss), the share strain generated in the seismic wall remaining below the 5th floor was 0.17 x 10^-3 at most, much lower than the evaluation standard value, 4 x 10^-3, which means that the seismic safety was evaluated as fully satisfying the safety standard. (The analysis resulted in the situation being substantially within elasticity range.) Therefore, the seismic safety assessment concluded that there was no impact to key facilities in terms of seismic safety such as the Reactor Pressure Vessel, the Primary Containment Vessel (PCV), the Spent Fuel Pool and so on.

(Attachment-3)

As a result of the sectional assessment via the 3 dimensional FEM analysis, the following was concluded.

・ As a result of a combination with seismic load acted by Design Basis Ground Motion (Ss) and other loads, the maximum strain in the reinforced bar at the Spent Fuel Pool was 1230 x 10^-6, which showed enough margin compared to the plastic limit strain, 5000 x 10^-6, as the evaluation standard value. (The analysis results were lower than the analytic elastic limit strain, 1683 x 10^-6.) In addition, the initial stress generated at the place where it had least margin in terms of out-of-plane shear force was 800 (N/mm), which was enough margin compared to the evaluation standard value, 1150 (N/mm).

・ Assuming the rigidity degradation due to cracks in the remaining floors and walls from the explosion, the parameter study results showed that there was no significant difference with the evaluation of the Spent Fuel Pool with or without the rigidity degradation.

・ It was highly likely that a fire broke out on 4th floor. Assuming partial rigidity degradation due to the fire and the removal of crystallized water from the concrete surface affected by the fire, the parameter study results showed that there was no significant difference in the evaluation of the Spent Fuel Pool with or without the rigidity degradation.

・ The analysis was standardized based on the assumption that the current water temperature in the Spent Fuel Pool is around 90 Celsius degrees and the ambient temperature was 10 Celsius degrees at its lowest. Considering that this situation continues until this winter, the parameter study was conducted assuming the water temperature was 100 Celsius degrees and the ambient temperature was 0 Celsius degrees. In this case, it was confirmed that the seismic margin was well above the evaluation standard value though the margin was slightly less than the standard case.

(Attachment-4)

4. Investigation results of the measures for the seismic reinforcement works and others

(1) Unit 1 Reactor Building

As a result of the seismic safety assessment, it has been concluded that it is not necessary to implement urgent measures for seismic reinforcement work and others at this stage since it is unlikely that there are places in Unit 1 where seismic safety has not been secured. In addition, there is the other aspect of the difficulty of being able to enter the building due to a high radiation levels. Hereafter, in the event that present radiation levels can be decreased allowing for work to be done inside the building, the implementation of seismic reinforcement works will be considered from the perspective of improving the seismic margin. Meanwhile, the steel framework section remaining above the 5th floor may be targeted for seismic reinforcement work based on the study of the influence on the spent fuel at the stage when the spent fuel will be removed from the Spent Fuel Pool after the working environment is improved.

(2) Unit 4 Reactor Building

As a result of the seismic safety assessment, it has been concluded that it is not necessary to implement urgent measures for seismic reinforcement work and others at this stage since it is unlikely that there are places in Unit 4 where seismic safety has not been secured. Nevertheless, since the radioactive dose level was relatively low on the 1st and 2nd floor in Unit 4, there were plans to conduct seismic reinforcement work at the bottom of the Spent Fuel Pool in order to improve seismic margin and currently preparation work is being carried out to this end. The effectiveness of this seismic reinforcement work was confirmed to contribute to an improved seismic margin as the result of the assessment by using a model taking in the sectional assessment of the 3 dimensional FEM analysis. Meanwhile, the steel framework structure and steel framework roof torus remaining above the 5th floor may be targeted for seismic reinforcement work based on the study of the influence on the spent fuel at the stage when the spent fuel will be removed from the Spent Fuel Pool after improving the working environment.

(Attachment-4)

5. Summary

In this report, it has been confirmed that the Reactor Buildings in Unit 1 and 4 have no seismic safety issues according to the seismic safety assessment that need resolving. In addition, the effectiveness of the seismic reinforcement work currently being carried out at the bottom of the Spent Fuel Pool in Unit 4 has been confirmed. Hereafter, there are plans to create an additional report on Unit 3 when the assessment on the damages on and above the 5th floor and the damaged walls below the 5th floor is completed.

Attachment 1

Detail of seismic safety evaluation of Reactor building of Unit 1

1. Policy of analysis and evaluation

Seismic evaluation and evaluation of impact on the reactor building structure caused by the hydrogen explosion etc. are conducted by utilizing design basis ground motion Ss in principle and by establishing the model that can properly describe the response states of buildings, structures, and foundations. Design basis ground motion Ss-3 is not utilized in this analysis as it is obvious from past calculation example (refer to attachment 1-1) that such movement was small enough in comparison with the response result of design basis ground motion Ss-1 and Ss-2

The mass system model integrating flexural and shearing rigidity is selected as a seismic response analysis model, considering the interaction with the foundations.

While the cooling function in the reactor was failed due to the tsunami that followed the earthquake and the reactor building of Unit 1 has been partially damaged by the hydrogen explosion etc.. In this analysis, the damage in the reactor building is estimated by analyzing its pictures and such estimation is reflected in the seismic response analysis model.

Seismic evaluation and evaluation of impact on the reactor building structure are conducted by comparing the shear strain of seismic wall calculated in seismic response analysis and standard evaluation point (4.0×10-3) responding to ultimate limit of reinforced concrete seismic wall.

As for ultimate limit of reinforced concrete seismic wall, as horizontal seismic force is dominant while vertical seismic force is negligible, seismic response analysis is conducted for horizontal force only.

The evaluation process of seismic response analysis for the reactor building of Unit 1 is described in Figure-1.1.

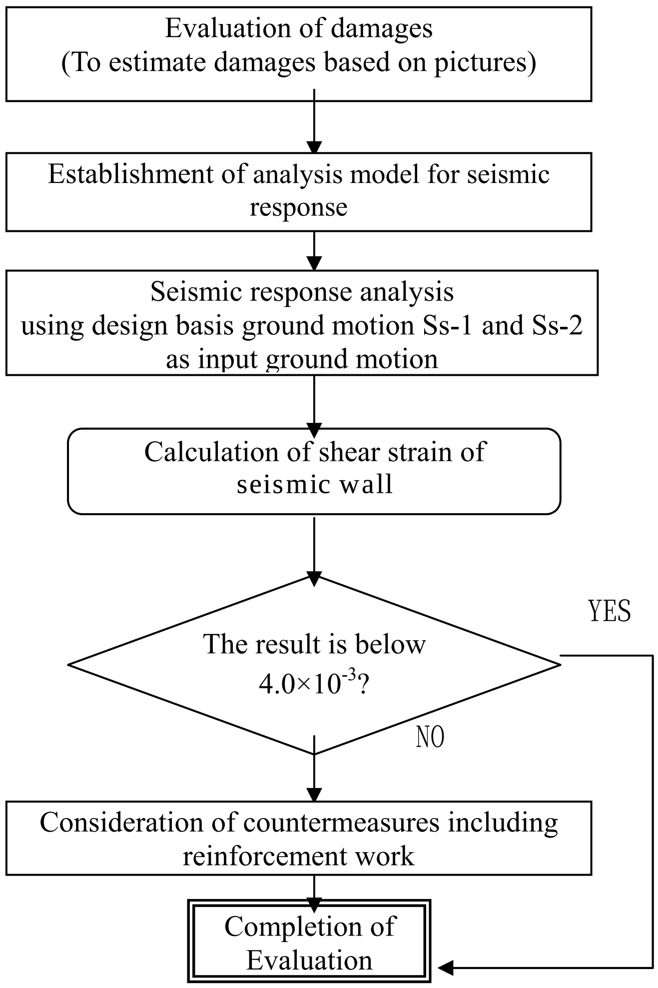


Figure-1.1 Evaluation process of seismic response analysis for the reactor building of Unit 4

2. Evaluation of Damage Situation

The cooling function of the reactor building of Unit 1 was failed due to the tsunami that followed the earthquake and the reactor building has been partially damaged due to a hydrogen explosion etc.. Damage situation of the reactor building is estimated based on pictures and reflected in a seismic response analysis model. In case we cannot have evaluated parts judging from their exterior pictures, we have evaluated whether they have been damaged based on information currently obtained from the investigation result of the inside of the building.

We will show you how to evaluate each part of damage situation as follows.

a. Exterior Wall/ Roof Truss

We have evaluated exterior walls and roof trusses above the refueling floor as damaged parts, as we can confirm the damages based on their exterior pictures. We have also evaluated exterior walls below the refueling floor as non-damaged ones, as we cannot confirm their damages based on pictures (Figure-2.1). We refer to pictures taken on March 24 and since then we have not confirmed that exterior walls have peeled off.

b. Other Parts

As we have not confirmed any damages on exterior wall below the refueling floor, we have evaluated interior walls below the refueling floor have not been damaged.

East Side

West Side

South Side

North Side

Figure-2.1 Situation of Exterior Walls

3. Input Ground Motion Used for Analysis

As input earthquake motion for the reactor building of Unit 1, we have used the design basis ground motion Ss-1 and Ss-2 assumed in the free surface level of base stratum in "Interim Report on Evaluation Result of Earthquake-Proof in Fukushima Daiichi Nuclear Power Station regarding the amendment of 'Guideline in Evaluation of Facilities of Nuclear Reactors to Produce Power' (Nuclear Admin Report to the Authorities 19 No. 603 dated on March 31, 2008).

A conceptual diagram of input ground motion used in earthquake response analysis is shown in Figure-3.1. Based on one-dimensional wave phenomena, ground motion to be inputted in the model is evaluated as ground response of design basis ground motion Ss assumed in the free surface level of base stratum. Also, by adding shear force at the building foundation base level to the input ground motion, notch effect of the ground is taken into account.

Among these, acceleration wave profile of design basis ground motion Ss-1 and Ss-2 at the free surface level of base stratum（O.P. -196.0m）is shown in Figure-3.2.

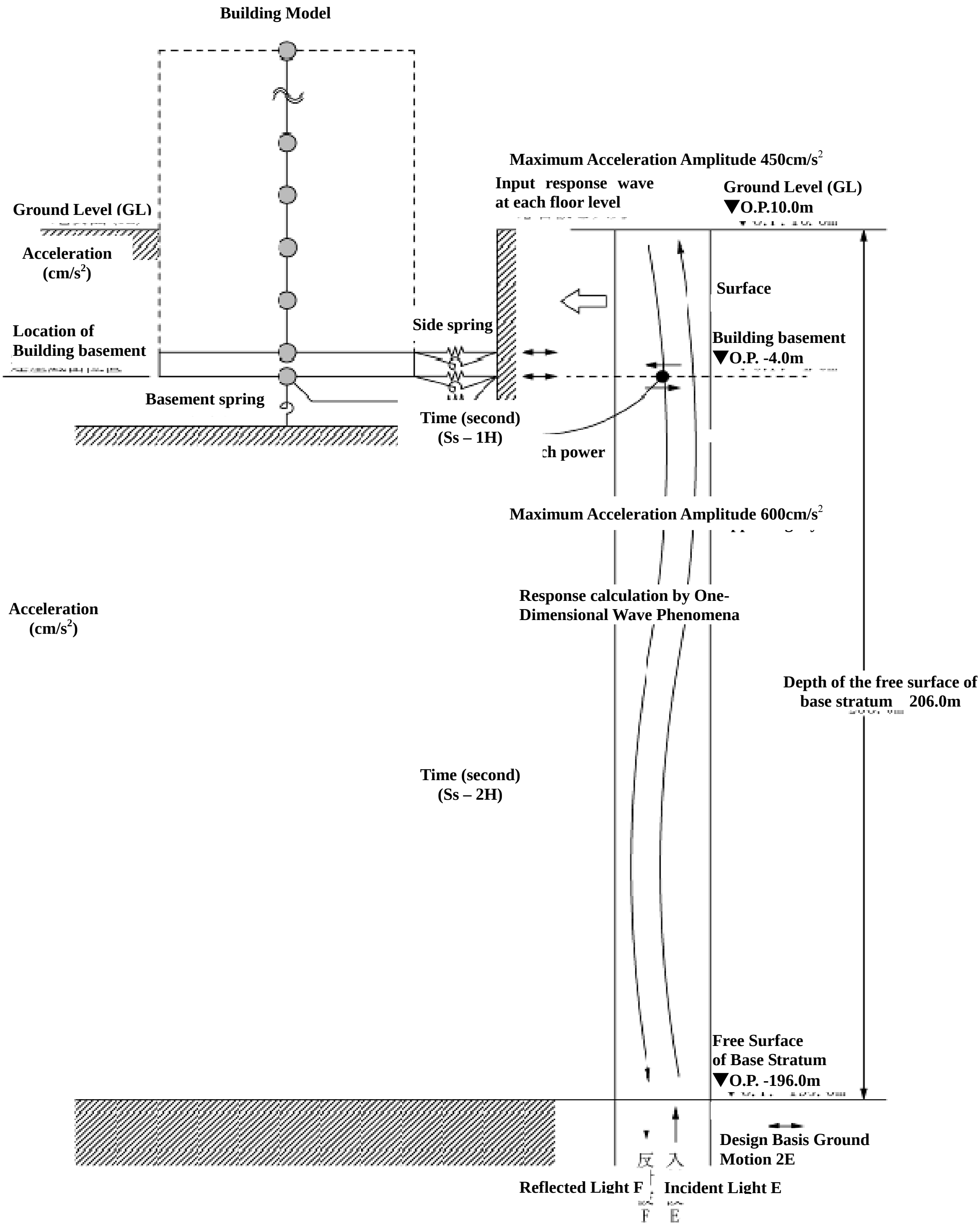
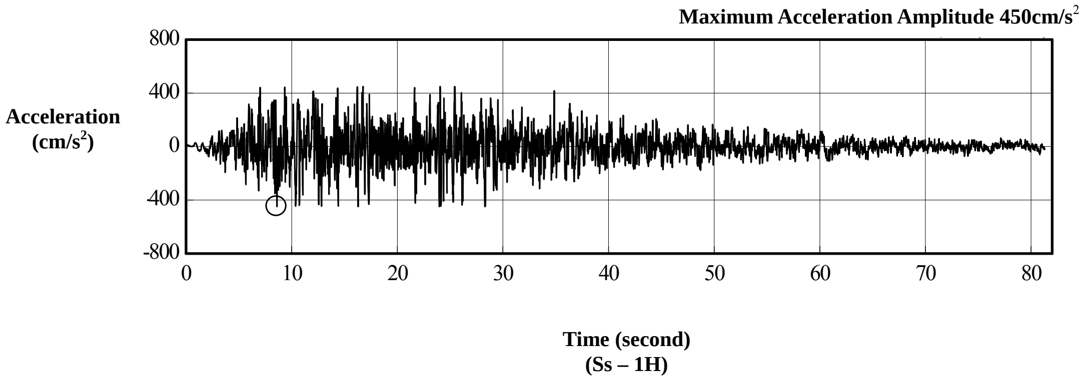


Figure-3.1 A conceptual diagram of Input Ground Motion used in Earthquake response analysis



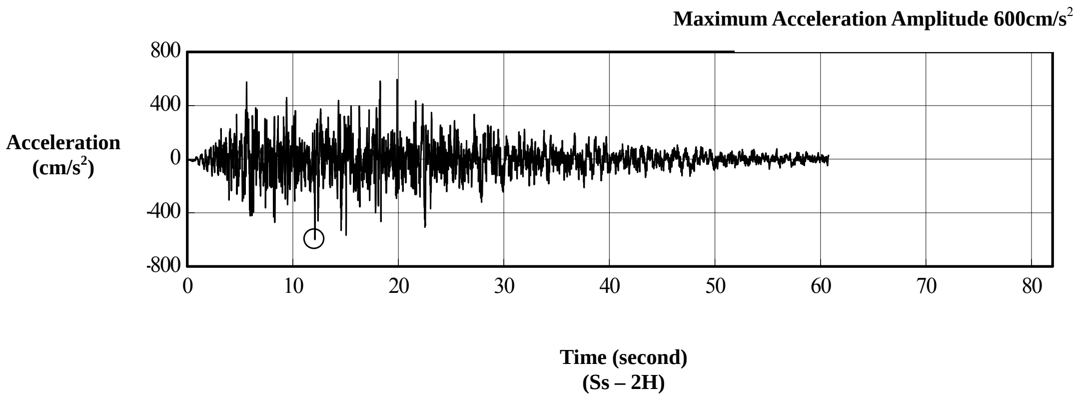


Figure-3.2 Chronicle acceleration wave profile (horizontal direction) of ground motion at free surface of base stratum

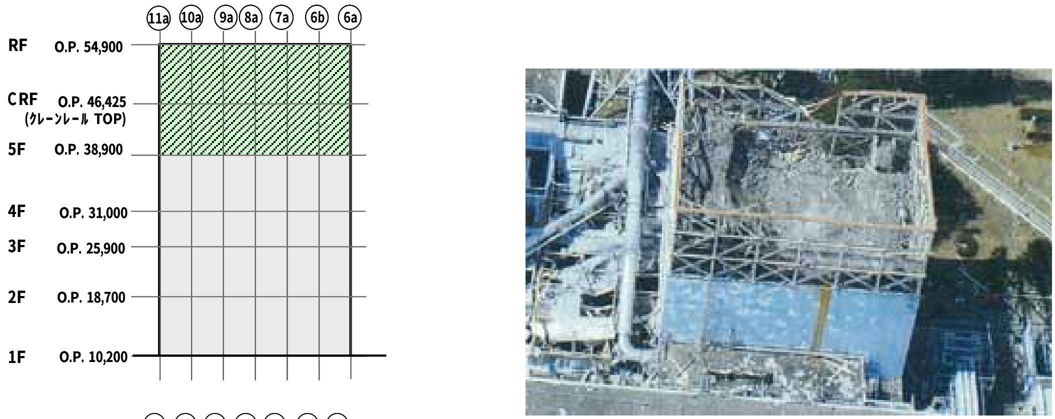
4. Analysis Model for Seismic Response

Seismic response of the reactor building against the design basis ground motion Ss is conducted by the dynamic analysis using the input seismic response calculated in the "3. Input Ground Motion Used for Analysis".

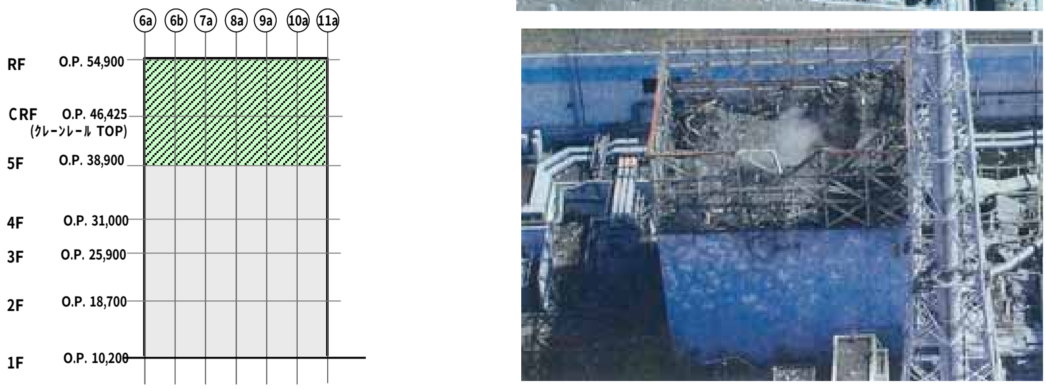
This study formulates new analysis model for seismic response based on the former model made in "Interim Report (revised version), Evaluation results of anti-earthquake stability by a revision of guidance for appraisal for anti-earthquake design regarding commercial reactor facilities, Fukushima Daiichi Nuclear Power Station" (on June 19, 2009, No.110, Genkanhatsukan No.21).

The reactor building of Unit 1 lost the cooling function for the reactor by the damage of tsunami coming after the earthquake, and the part of the building was damaged by the hydrogen explosion, etc. The analysis model is formulated based on damaged conditions evaluated in "2. Evaluation of Damage Situation" The damaged steel frame and roof above the operation floor are not considered in the model and the collapsed parts are assumed that the down floor has supported the weight. Figure 4-1 shows the damaged conditions of the reactor building of Unit 1(elevation) and Figure 4-2 shows the damaged conditions (plane).

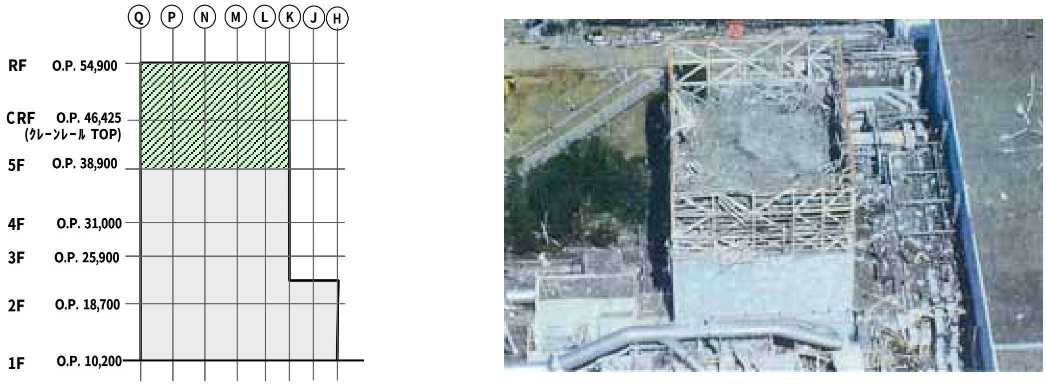
East



West



South



North

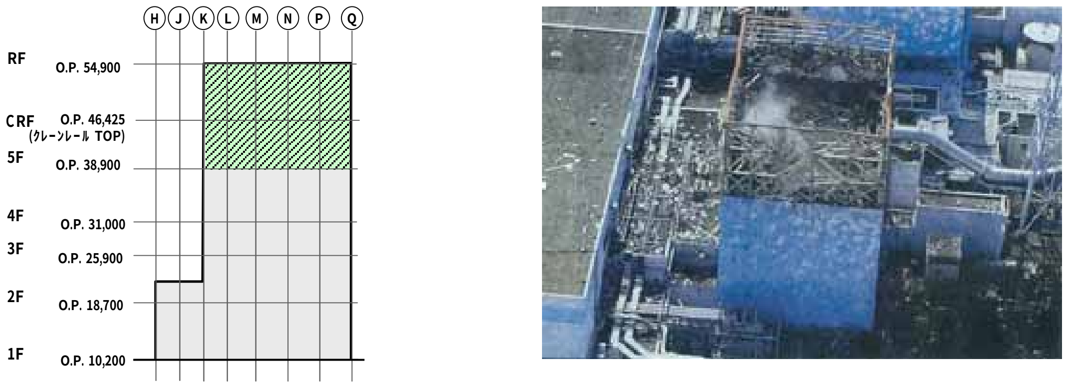


Figure 4-1 Damaged Conditions of the Reactor Building of Unit 1(Elevation)

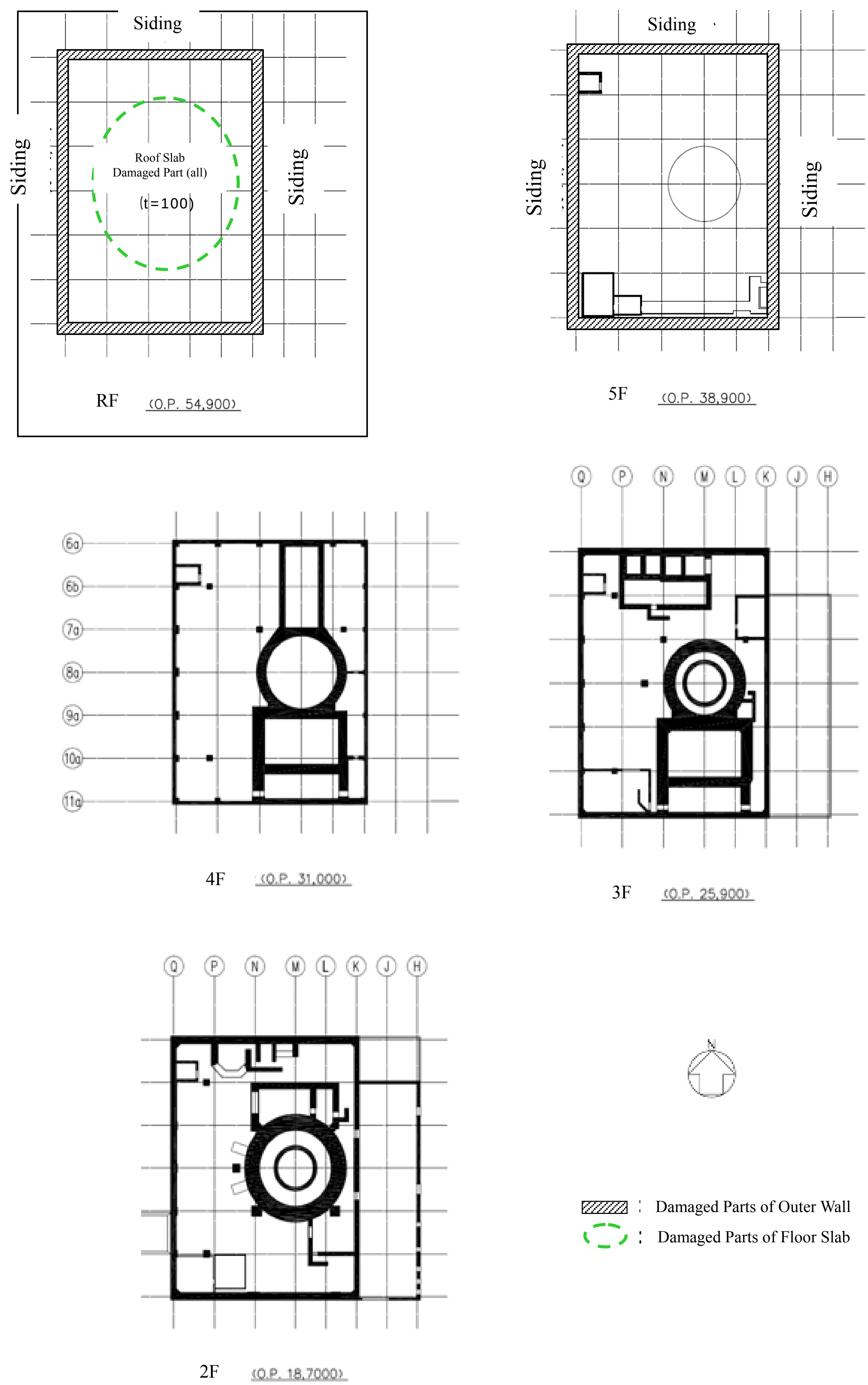


Figure 4-2 Damaged Conditions of the Reactor Building of Unit 1(Plane)

(1) Analysis Model for Seismic Response of Horizontal Direction

Analysis model for seismic response of horizontal direction uses a simplified weight model which considers bending transformation and sharing transformation of the building, and a building-ground connection model which the ground is evaluated a an equal spring, as shown in Figure 4-3 and Figure 4-4. The effects of connection between the building and the ground is evaluated by a spring effect of the ground and input seismic response. Physical factors of concrete for the analysis is shown in Table 4-1 and other factors of building analysis model are shown in Table 4-2.

The ground factors were decided considering a sharing strain level in the earthquake assuming it is a horizontal layers ground. The ground factors for the analysis is shown in Table 4-3.

In the analysis model of horizontal direction, a ground spring beneath the base mat considered the methodology shown in “JEAG 4601-1991” and revised in horizontal layers. As a result, it is evaluated as the sway and locking spring factors based on swinging admittance theory. A ground spring of the building side of the underground part considered the methodology shown in “JEAG 4601-1991” using the ground factors of the building side position. As a result, it is evaluated as an approximate model based on the Novak spring.

The ground spring is evaluated as complex stiffness depending on the frequency of vibration. The ground spring used the real static value for spring factors (Kc) shown in Figure 4-5, and the inclined line linking between an imaginary value corresponding to primary natural frequency of the building and ground connection system and the origin as the damped factor (Cc).

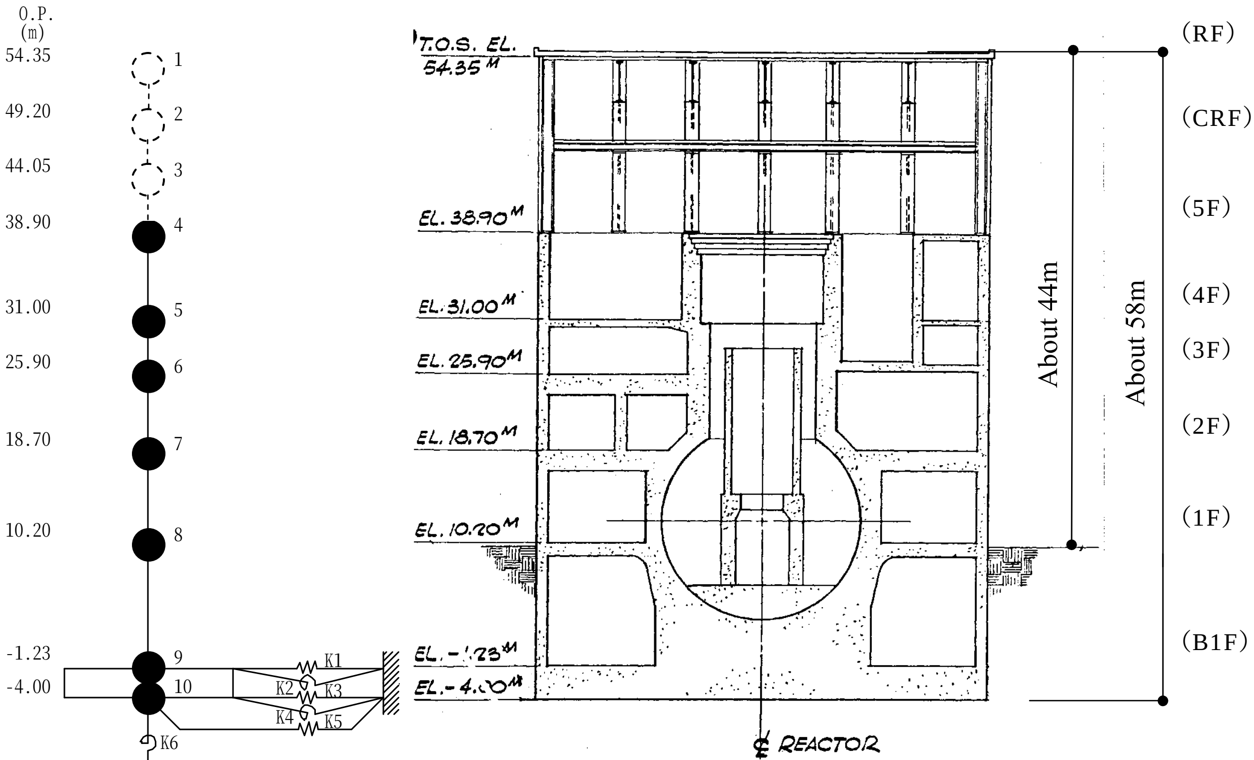


Figure Analysis Model for Seismic Response of the Reactor Building of Unit 1 (N-S Direction)

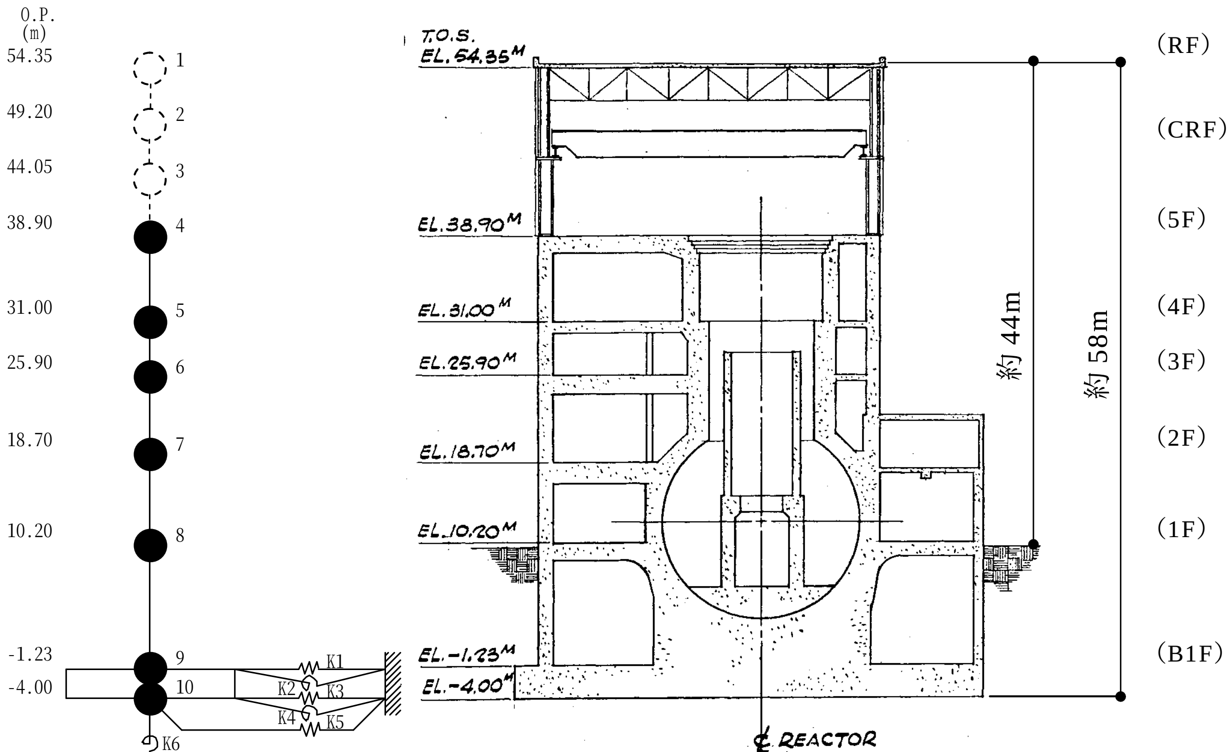


Figure 4-4 Analysis Model for Seismic Response of the Reactor Building of Unit 1 (E-W Direction)

Table 4-1 Physical Factors for Seismic Response Analysis

Concrete	Strength *1 Fc (N/mm^2)	Young Coefficient *2 E (N/mm^2)	Sharing Elastic Coefficient*2 G (N/mm^2)	Poisson's Ratio ν	Weight of Unit Volume*3 γ (kN/m^3)
	35.0	2.57×10^4	1.07×10^4	0.2	24
Reinforced Steel	SD345 equivalent (SD35)				
Steel Material	SS400 equivalent (SS41)				

*1：Strength adopts the more realistic strength "hereinafter Real Strength". The real strength is decided by average value of compressed strength considering a scattering of the past test data.

*2：The value shows based on the real strength.

*3：The value shows a value of reinforced steel.

Table 4-2 Factors of Building Analysis Model

(N-S Direction)

Weight Point	Weight W (kN)	Rotation Inertia Weight I_G (x 10^5kN・m^2)	Cross Section of Sharing A_S (m^2)	Cross Section Secondary Moment I (m^4)
1	—	—		
			—	—
2	—	—		
			—	—
3	—	—		
			—	—
4	58,690	84.43		
			135.0	16,012
5	67,910	97.77		
			160.8	21,727
6	77,220	111.11		
			132.8	24,274
7	87,200	125.53		
			155.6	36,481
8	146,020	210.16		
			294.0	52,858
9	147,070	211.73		
			1,914.3	275,530
10	62,400	89.83		
Total	646,510			

Young Coefficient Ec	2.57×10^7 (kN/m^2)
Sharing Elastic Coefficient G	1.07×10^7 (kN/m^2)
Poisson's Ratio ν	0.20
Attenuation	5% (Steel 2%)
Shape of Basement	41.56 m (N-S) x 43.56m (E-W)

(E-W Direction)

Weight Point	Weight W (kN)	Rotation Inertia Weight I_G (x 10^5kN・m^2)	Cross Section of Sharing A_S (m^2)	Cross Section Secondary Moment I (m^4)
1	—	—		
			—	—
2	—	—		
			—	—
3	—	—		
			—	—
4	58,690	48.34		
			102.7	9,702
5	67,910	55.90		
			163.9	13,576
6	77,220	63.55		
			131.6	14,559
7	87,200	125.53		
			197.8	36,427
8	146,020	210.16		
			294.0	52,858
9	147,070	259.97		
			1,914.3	338,428
10	62,400	110.32		
合計	646,510			

Young Coefficient Ec	2.57×10^7 (kN/m^2)
Sharing Elastic Coefficient G	1.07×10^7 (kN/m^2)
Poisson's Ratio ν	0.20
Attenuation	5% (Steel 2%)
Shape of Basement	41.56 m (N-S) x 43.56m (E-W)

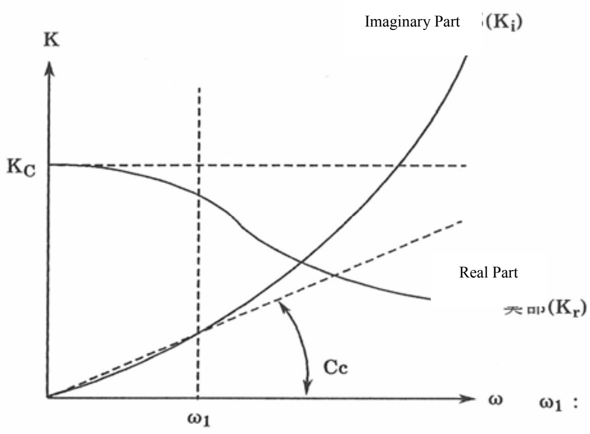
Table 4-3 Ground Factors

(Ss-1)

Elevation O.P. (m)	Geology	S Wave Velocity (Vs) (m/s)	Weight of Unit Volume γt (kN/m³)	Poisson's Ratio ν	Primary Sharing Elastic Coefficient (Go) (kN/m²)	Decrease Ratio of Strength (G/Go)	Sharing Elastic Coefficient (G) (kN/m²)	Vs after Decrease of Strength (Vs) (m/s)	Damp Factor h (%)
10.0									
1.9	Sand Stone	380	17.8	0.473	262,000	0.85	223,000	351	3
-10.0	Mud Stone	450	16.5	0.464	341,000	0.78	266,000	398	3
-80.0		500	17.1	0.455	436,000		340,000	442	
-108.0		560	17.6	0.446	563,000		439,000	495	
-196.0		600	17.8	0.442	653,000		509,000	530	
	Free Base Ground	700	18.5	0.421	924,000	1.00	924,000	700	—

(Ss-2)

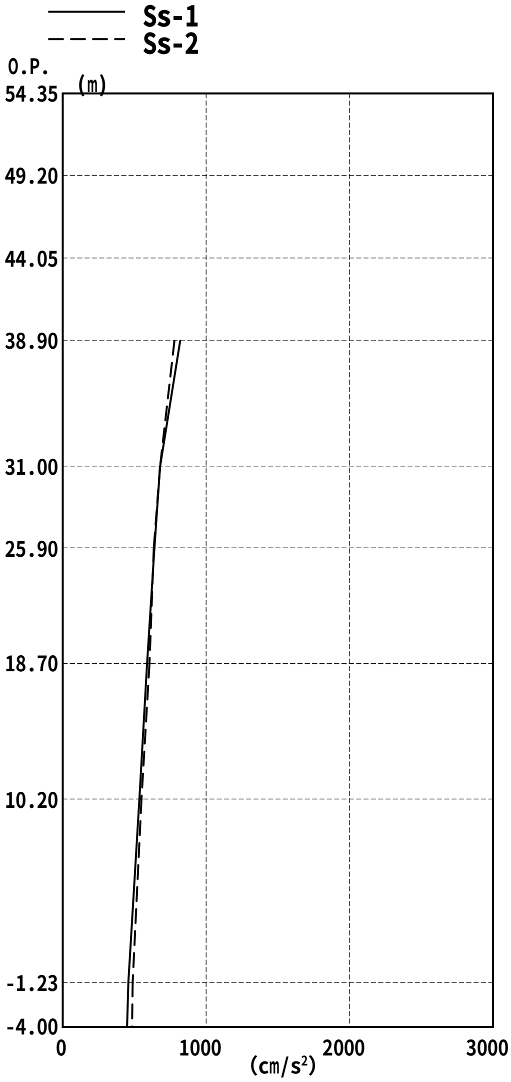
Elevation O.P. (m)	Geology	S Wave Velocity (Vs) (m/s)	Weight of Unit Volume γt (kN/m³)	Poisson's Ratio ν	Primary Sharing Elastic Coefficient (Go) (kN/m²)	Decrease Ratio of Strength (G/Go)	Sharing Elastic Coefficient (G) (kN/m²)	Vs after Decrease of Strength (Vs) (m/s)	Damp Factor h (%)
10.0									
1.9	Sand Stone	380	17.8	0.473	262,000	0.85	223,000	351	3
-10.0	Mud Stone	450	16.5	0.464	341,000	0.81	276,000	405	3
-80.0		500	17.1	0.455	436,000		353,000	450	
-108.0		560	17.6	0.446	563,000		456,000	504	
-196.0		600	17.8	0.442	653,000		529,000	540	
	Free Base Ground	700	18.5	0.421	924,000	1.00	924,000	700	—



Primary Natural Frequency of Building-Ground Connection System

5. Analysis Results of Seismic Response

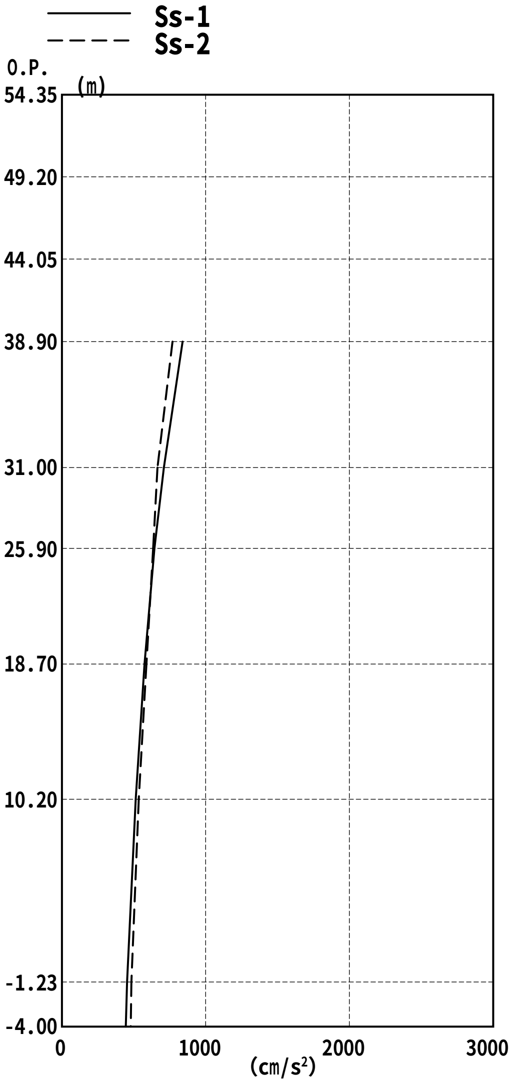
Maximum response acceleration of N-S direction and E-W direction obtained by the seismic response analysis is shown in Figure 5-1 and 5-2 below.



(cm/s²)

Ss-1	Ss-2
819	779
680	677
639	633
591	606
534	551
457	487
451	484

Figure 5-1 Maximum Response Acceleration (N-S Direction)



(cm/s²)

Ss-1	Ss-2
839	771
708	665
645	633
575	589
517	532
456	484
445	481

Figure 5-2　Maximum Response Acceleration (E-W Direction)

6. Evaluation Results of Anti-Earthquake Stability

Figure 6-1, 6-2, 6-3 and 6-4 show the maximum response values for design basis ground motion Ss-1 and Ss-2 in sharing skeleton maps of anti-earthquake. The maximum sharing strain was estimated at 0.12×10^{-3} (Ss-1H and Ss-2H, N-S Direction, 1F) and it has enough margin for the evaluation standard (4.0×10^{-3}).

From the analysis, the present reactor building was evaluated that the building stability did not affect the facilities which were important for anti-earthquake stability.

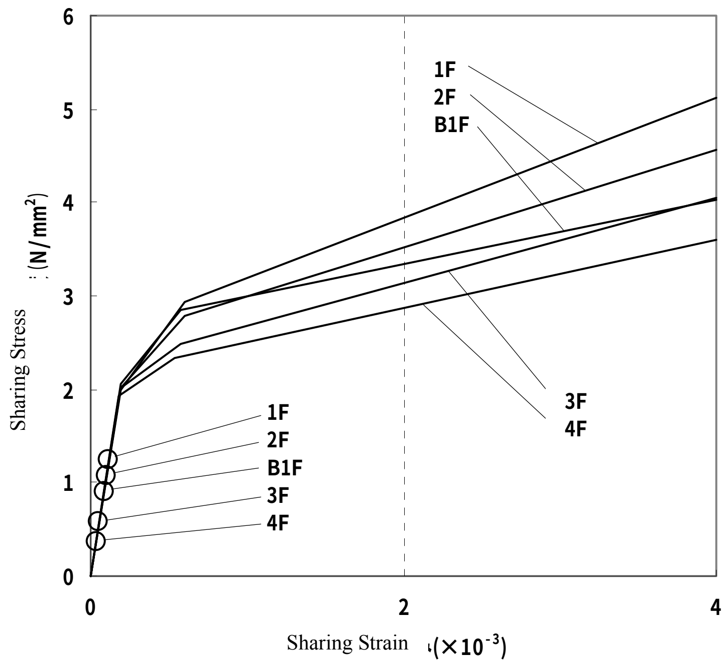


Figure 6-1 Maximum Response Value in Sharing Skelton Map (Ss-1, N-S Direction)

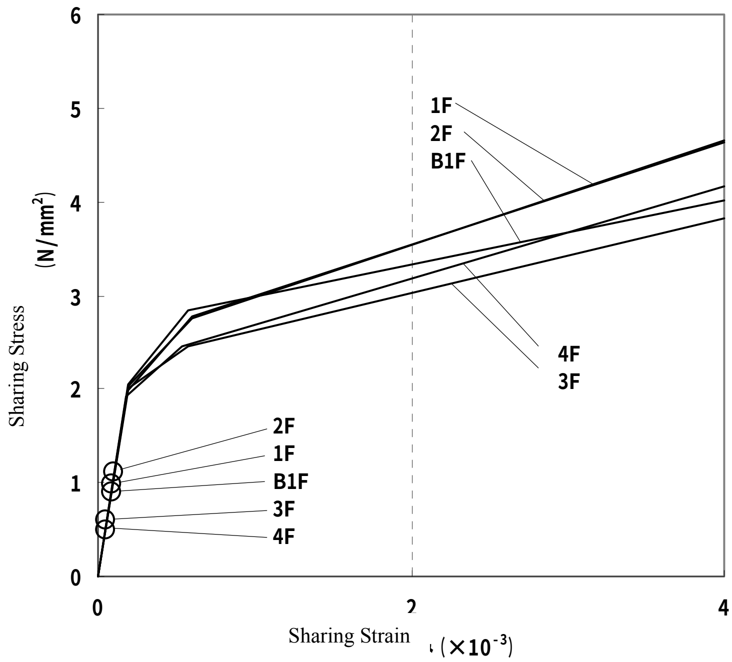


Figure 6-2 Maximum Response Value in Sharing Skelton Map (Ss-1, E-W Direction)

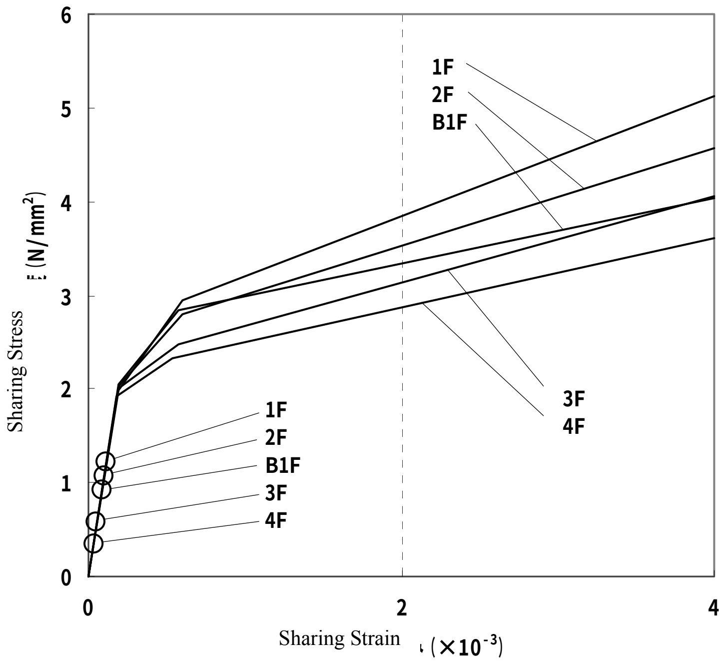


Figure 6-3　Maximum Response Value in Sharing Skelton Map (Ss-2, N-S Direction)

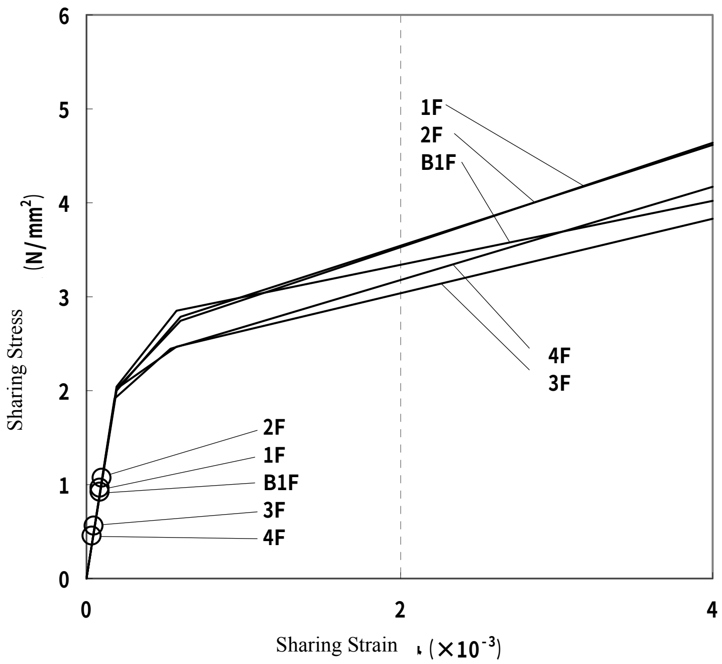
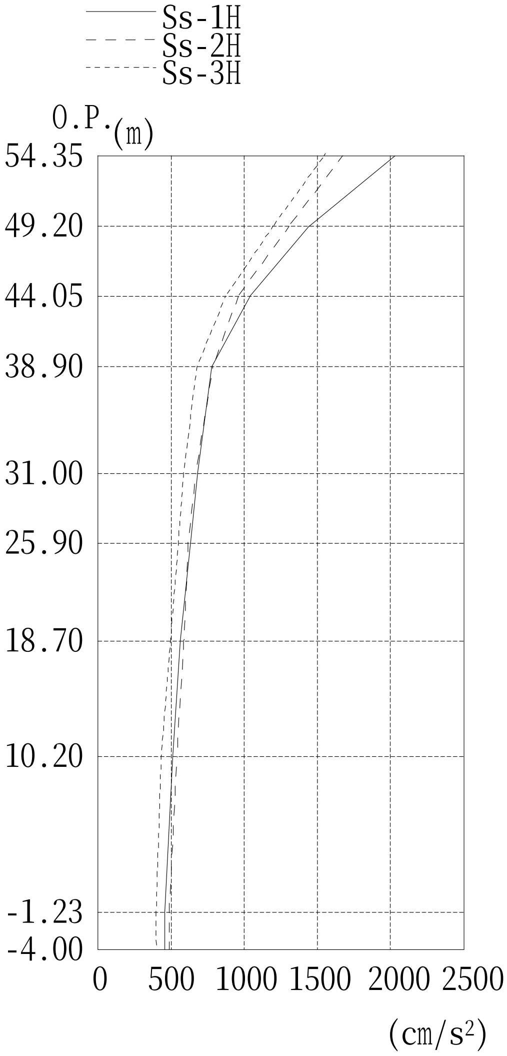


Figure 6-4　Maximum Response Value in Sharing Skelton Map (Ss-2, E-W Direction)

Evaluation result of seismic safety associated with revision of “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities”

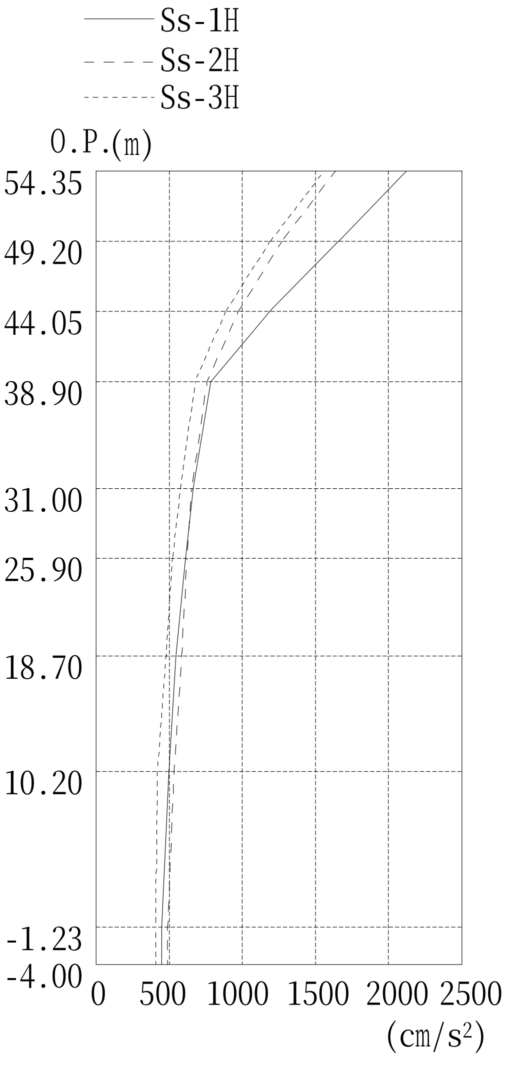
TEPCO reports evaluation result of seismic safety in Fukushima Diichi Nuclear Power Station which was recorded in “Interim report (revised version), Evaluation result of seismic safety associated with revision of ‘Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities’ in Fukushima Diichi Nuclear Power Station”(#21No110, Dated June 19th, 2010) as below.



(cm/s²)

Ss-1H	Ss-2H	Ss-3H
2030	1674	1544
1445	1305	1196
1037	959	873
780	785	676
680	669	585
632	616	549
561	587	495
508	539	432
455	487	397
455	488	398

Fig-1　Maximum Response Acceleration（NS direction）



(cm/s²)

Ss-1H	Ss-2H	Ss-3H
2123	1635	1557
1658	1270	1190
1190	973	886
785	758	677
663	654	577
611	621	522
545	584	478
496	533	418
449	489	407
449	487	409

Fig-2　Maximum Response Acceleration（EW direction）

Table-1 list of shear-strain on seismic wall（NS direction）

($\times 10^{-3}$)

Floor	Ss-1H	Ss-2H	Ss-3H	assessment criterion
4F	0.04	0.04	0.03	Less or equal 2.0
3F	0.06	0.06	0.05	
2F	0.10	0.10	0.09	
1F	0.12	0.12	0.10	
B1F	0.08	0.09	0.07	

Table -2 list of shear-strain on seismic wall（EW direction）

($\times 10^{-3}$)

Floor	Ss-1H	Ss-2H	Ss-3H	assessment criterion
4F	0.05	0.05	0.04	Less or equal 2.0
3F	0.06	0.05	0.05	
2F	0.10	0.10	0.09	
1F	0.09	0.09	0.08	
B1F	0.08	0.09	0.07	

End

Attachment-2: Exertion from "Report regarding the execution of the measure to fill in the water up to the top of the fuel range on Unit 1 of Fukushima Daiichi Nuclear Power Station" (dated May 5th, 2011)

Results of the evaluation of seismic adequacy and effects on the structure of the nuclear reactor building associated with the elevation of the water level in the nuclear reactor containment vessel

1. Analysis and evaluation principle

The evaluation of seismic adequacy and effects on the structure of the nuclear reactor building associated with the elevation of the water level in the nuclear reactor containment vessel are conducted based on the seismic force used for the design (seismic force occurred by the Design Basis Seismic Motion (Ss)) and conducted upon the setting up the model that may properly describe the reaction of the foundation, the building and the structure. Also, regarding the Design Basis Ground Motion Ss-3, we will omit it under this analysis because we know from the past calculated example that it is apparently smaller than the response results of Design Basis Ground Motions Ss-1 and Ss-2.

The seismic response analysis model is the mass point system model that considers flexural and shearing rigidity considering interactions between the foundations.

Regarding the nuclear reactor building of Unit 1, it is partially damaged by the hydrogen explosion, etc. that was led by the loss of cooling function caused by the tsunami after the earthquake. In this analysis, the extent of damage to the nuclear reactor building is assumed by the photos and such extent of damage is reflected to the seismic response analysis model.

Also, the mass increase that will be caused by the elevation of the water level in the nuclear reactor containment vessel will be added to the mass point of the nuclear reactor building model.

The evaluation of seismic adequacy and effects on the structure of the nuclear reactor building will be conducted, with the object of prevention of knock-on effect to important facilities for seismic safety, by comparing the shear strain of seismic walls that is acquired by the seismic response analysis and valuation standard value (4.0×10^{-3}) that is corresponding to the ultimate limit of seismic walls that are made of reinforced concrete.

Also, regarding the ultimate limit of seismic walls that are made of reinforced concrete, because horizontal direction seismic force is dominant and vertical direction seismic force has less effect, the seismic response analysis will be conducted horizontal direction only.

If it is found that the margin of seismic ratio is relatively small by the analysis described above, we will conduct more detailed analysis.

The example of evaluation procedure of the seismic response analysis of the nuclear reactor building of Unit 1 is shown on figure 1.1.

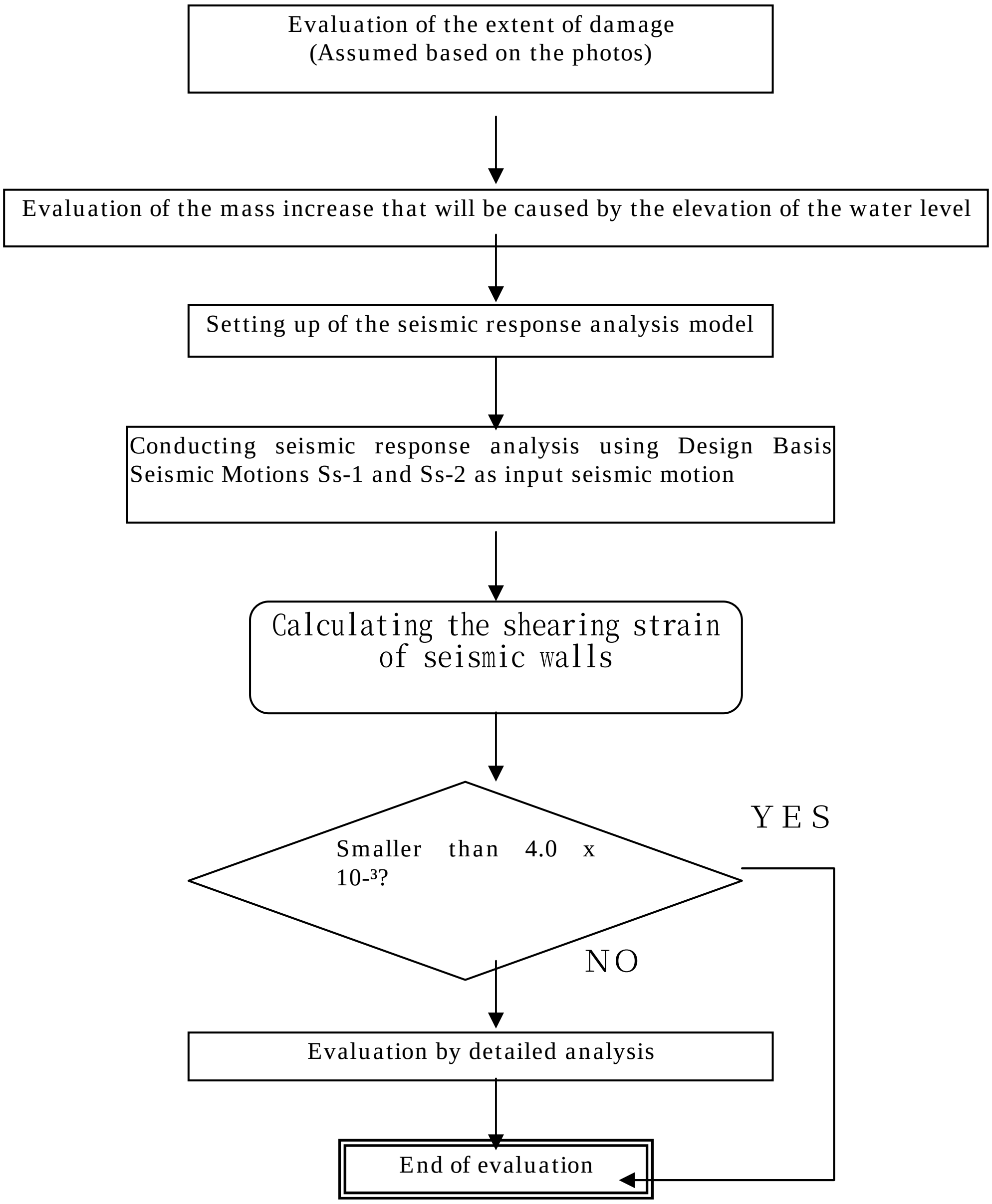


Figure 1.1 Example of evaluation procedure of the seismic response analysis of the nuclear reactor building of Unit 1

2. Input seismic motion to be used for analysis

The seismic motion to be input to the nuclear reactor building of Unit 1 are Design Basis Seismic Motions Ss-1 and Ss-2 that are assumed on the surface level of released foundation that was assumed on the "Interim Report for the Fukushima Daiichi Nuclear Power Station: 'The result of the seismic safety analysis evaluation associated with the revision of 'Guidelines in seismic design evaluation regarding nuclear reactor facilities for generation' "(*GenKanHatsuKan* 19 No.603 dated March 31st, 2008).

The conceptual diagram of input seismic motion that is used to the seismic response analysis is shown in Figure 2.1. The round motion to be input to the model is, based on one dimension wave theory, evaluated as the reaction of the foundation to the Design Basis Seismic Motions that is assumed on the surface level of released foundation. Also, the notching effect of the ground is taken into the consideration by adding the shear force at the bottom level of the basic of the building to input ground motions.

Of these analyses, acceleration wave profiles of the Design Basis Seismic Motions Ss-1 and Ss-2 at the surface level of released foundation point (o.p. -196.0m) are shown in Figure 2.2.

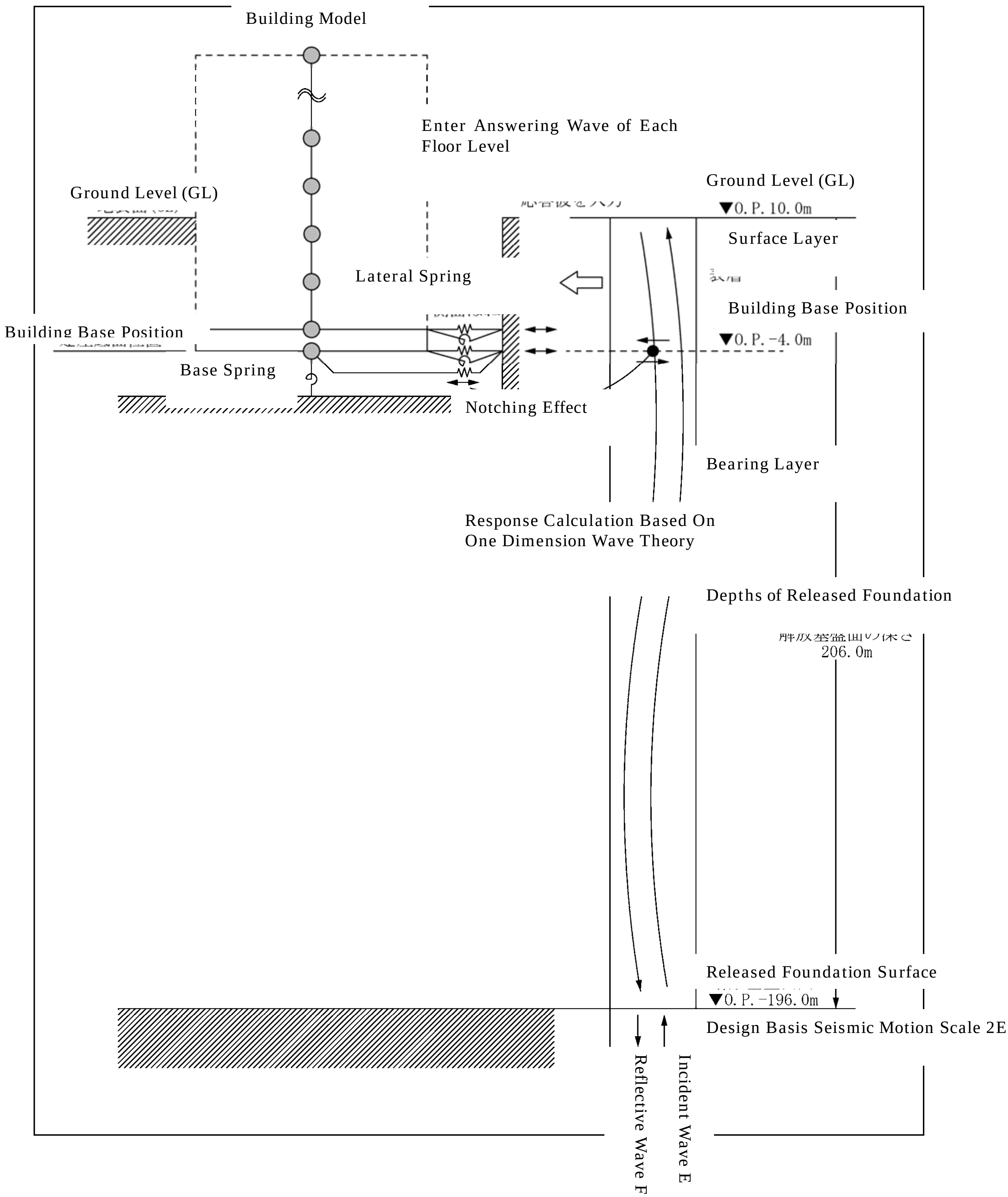
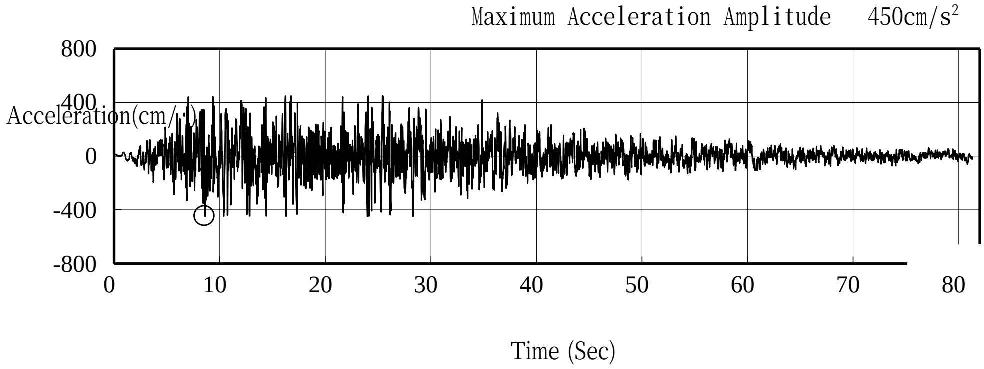
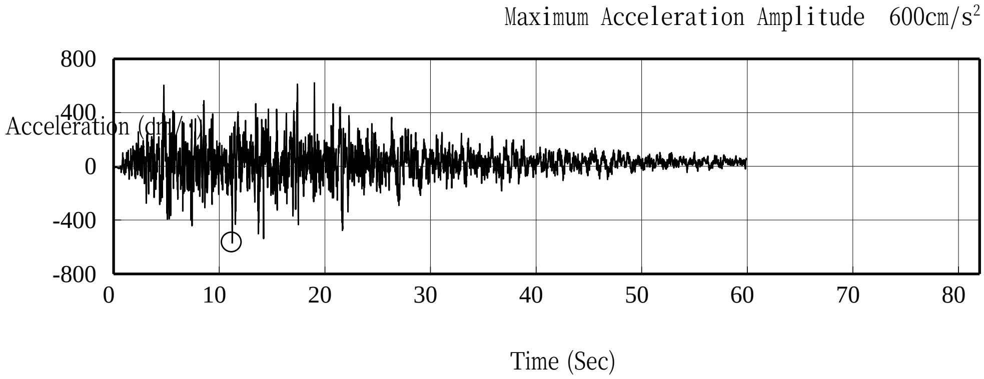


Fig.-2.1 Conceptual Diagram of Input Seismic Motion for Seismic Response Analysis



(Ss-1H)



(Ss-2H)

Fig.-2.2 Acceleration Wave Profiles of Seismic Motion at Surface Level of Released Foundation (Horizontal Direction)

3. Seismic Response Analysis Model

The seismic response analysis for the design basis seismic motion Ss will be based on the dynamical analysis using the input seismic motion calculated in accordance with the "2. Input Seismic Motion to be used for the Analysis".

This study shows a new model for the seismic response analysis by adding below two (2) points to the seismic response analysis built based on the "Interim Report (revised) for the Fukushima Daiichi Nuclear Power Station: 'The Result of the Seismic Safety Analysis Evaluation Associated with the Revision of 'Guidelines in Seismic Design Evaluation Regarding Nuclear Reactor Facilities for Power Generation' " (*GenKanHatsuKan* 21 No.110 dated June 19^{th}, 2009).

1. Regarding the nuclear reactor building of Unit 1, it is partially damaged by the hydrogen explosion, etc. that was led by the loss of cooling function caused by the tsunami occurred after the earthquake. The damage condition of the nuclear reactor building is assumed based on the photos and the steal-frame of the upper part of the operating floor and the roof that were damaged will not be taken into account for modeling. Furthermore, the weight of the fallen-parts is assumed to be supported by the lower level floor. The extent of damage of the nuclear reactor building of Unit 1 (elevation view) is shown in Fig. – 3.1 and the extent of damage (plain view) is shown in Fig. – 3.2.
2. The mass increase that will be caused by the elevation of the water level in the nuclear reactor containment vessel will be added to the several mass points of the nuclear reactor building model taking into the account, transmittance of seismic force at the junction of nuclear reactor containment vessel and the nuclear reactor building.

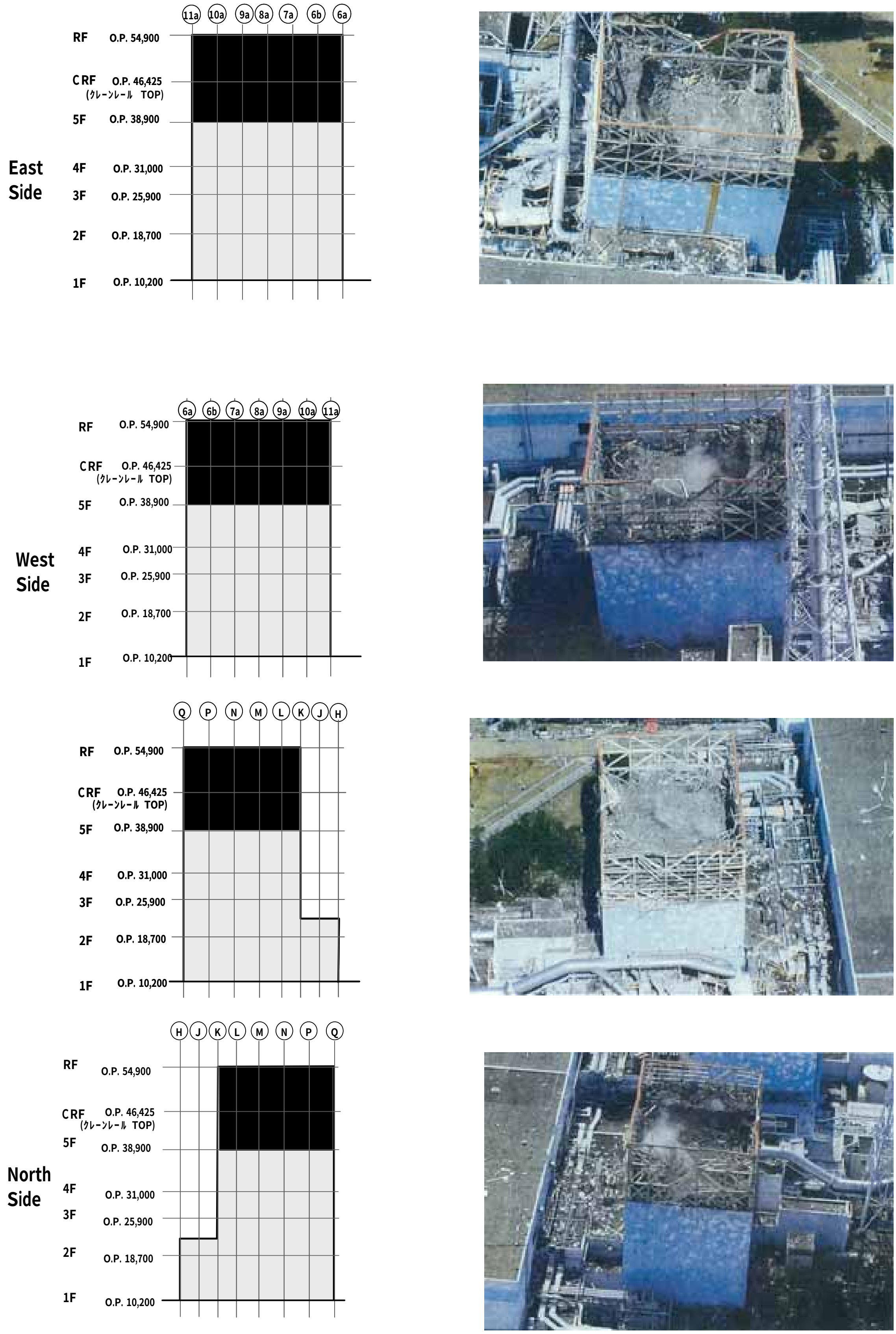


Fig.-3.1 Extent of Damage of Unit 1 Nuclear Reactor Building (Elevation View)

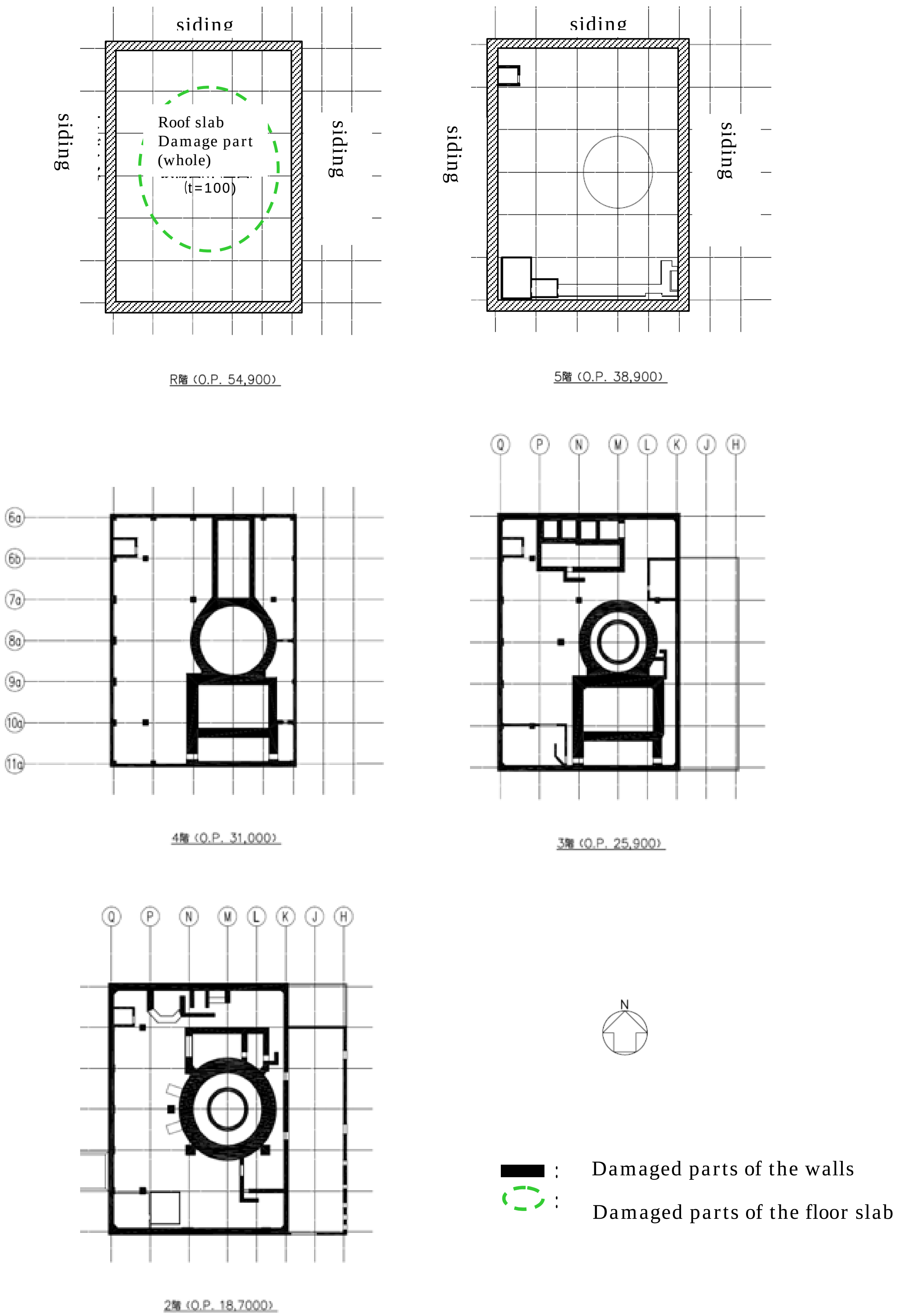


Figure-3.2 Status of damage of reactor building of Unit 1 (plain view)

(1) Horizontal seismic response analysis model

Horizontal seismic response analysis model is a building – foundation connection line model, whose buildings are bent and mass point is transformative and shear-transformative and the foundation is evaluated with equivalent springs, shown as in the Figure-3.3 and 3.4. The effect of building-foundation connection line is evaluated with foundation springs and input ground motion. The physicality value of concrete used for the analysis is shown in Table-3.1 and the data of the building analysis model are shown in Table-3.2.

We have calculated the foundation constant on the assumption of horizontal bedding foundation, considering the level of shear twist in case of earthquakes. The foundation constant used for the analysis is shown in Table 3.3.

With regard to basic bottom foundation springs in the horizontal analysis model, we have consulted methods shown in “JEAG 4601-1991”, carried out bedding correction and approximately evaluated sway and rocking spring constants based on vibration admittance theory. With regard to foundation springs on the building side in the embedded parts, with foundation constants located on the side of buildings, we evaluate horizontal and rolling springs, considering the method shown in “JEAG 4601-1991" in approximate manner based on Novak springs.

Vibration springs are secured as complex stiffness depending on the frequency but as shown in Figure-3.5, we approximate static values in the real part as spring constant (Kc) and by adopting the tangent of the line that connects the value in the imaginary part that correspond to primary character frequency of building-foundation connection line as damped coefficient (Cc) and the origin.

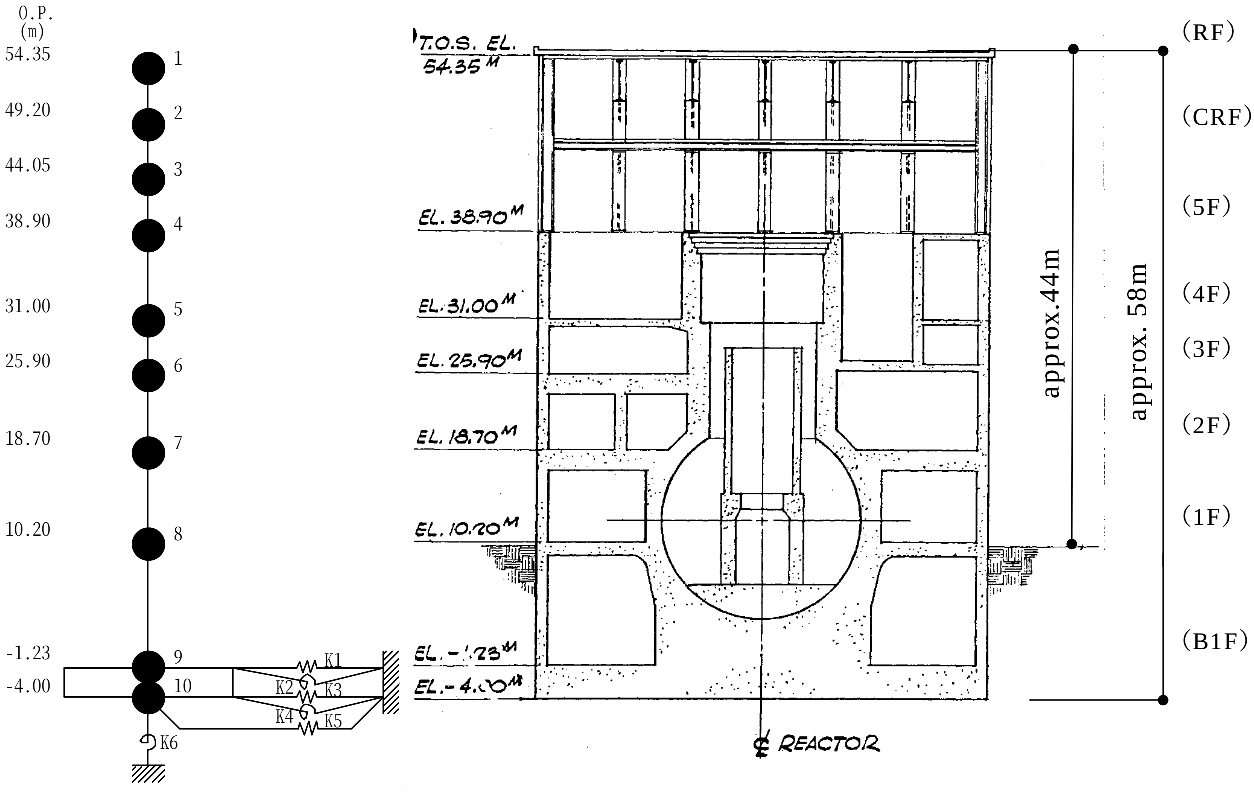


Figure-3.3 Reactor building of Unit 1 Seismic response analysis model (NS direction)

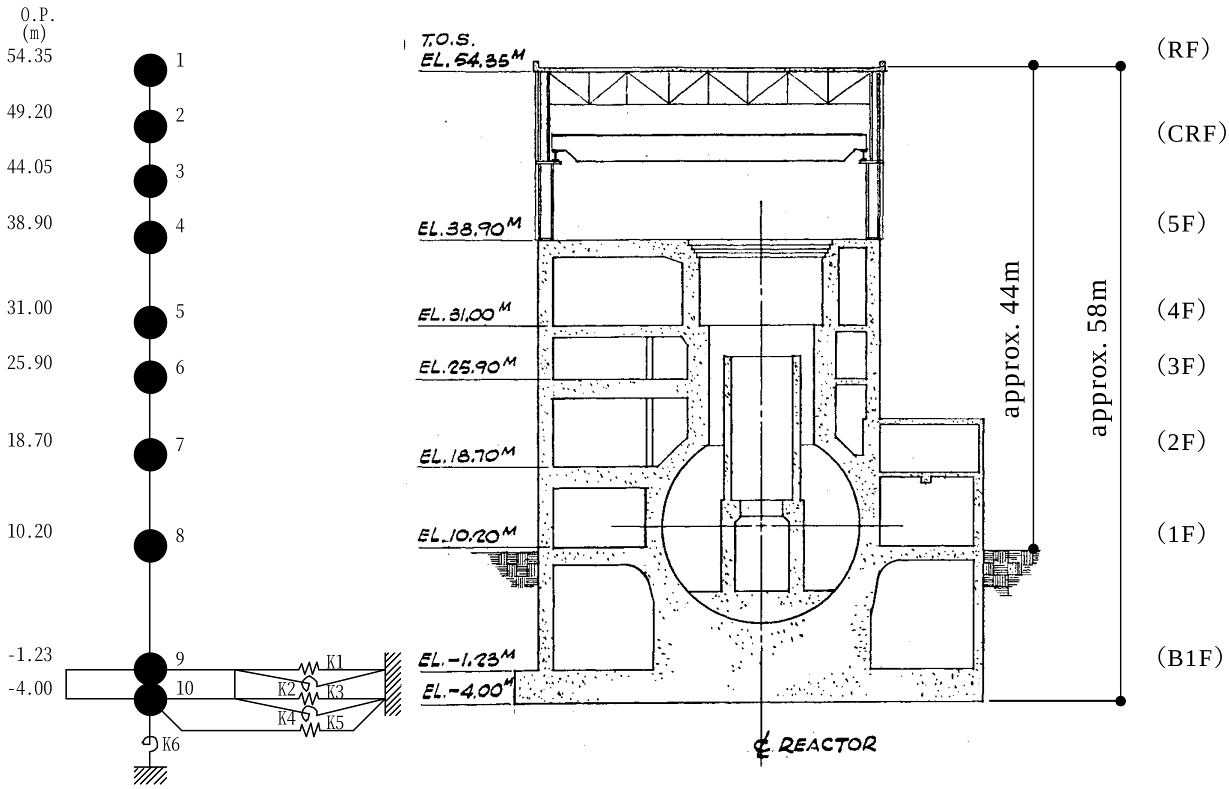


Figure-3.4 Reactor building of Unit 1 Seismic response analysis model (EW direction)

Table-3.1 Physicality used for seismic response analysis

<table>
<tr><td rowspan="2">Concrete</td><td>Strength
＊1
Ｆｃ
(N/mm²)</td><td>Young's
modulus*2
Ｅ
(N/mm²)</td><td>Shearing
elasticity
modulus*2
G
(N/mm²)</td><td>Poisson
ratio
ν</td><td>Weight per
volume*3
γ
(kN/m³)</td></tr>
<tr><td>35.0</td><td>2.57×10^4</td><td>1.07×10^4</td><td>0.2</td><td>24</td></tr>
<tr><td>Ferroconcrete</td><td colspan="5">SD345 (approximately)
(SD35)</td></tr>
<tr><td>Steel</td><td colspan="5">SS400 (approximately)
(SS41)</td></tr>
</table>

*1：About strength, we adopt the strength that is close to the actual status (hereinafter referred to as "Actual strength"). We have colleted past test data of compression strength, considered variation of the data, and calculated the values, rounding down the average compression strength values.

*2：Data based on actualstrength

*3：Data of ferroconcrete

Table-3.2 Specifications of analysis model for buildings

(NS direction)

Number of mass point	Weight of mass point *1 W (kN)	Rotary inertia weight *1 I_G ($\times 10^5$kN·m^2)	Shear cross-section area A_s (m^2)	Cross sectional secondery moment I (m^4)
1	—	—		
			—	—
2	—	—		
			—	—
3	—	—		
			—	—
4	58,690	84.43		
			135.0	16,012.0
5	67,910	97.77		
			160.8	21,727.0
6	80,900 (3,680)	116.41 (5.30)		
			132.8	24,274.0
7	87,200	125.53		
			155.6	36,481.0
8	166,150 (20,130)	239.13 (28.97)		
			294.0	52,858.0
9	177,480 (30,410)	255.51 (43.78)		
			1,914.3	275,530.0
10	62,400	89.83		
Total	700,730 (54,220)			

Young modulus E_c 2.57×10^7(kN/m^2)
Transverse elasticity modulus G 1.07×10^7(kN/m^2)
Poisson ratio ν 0.20
Decay h 5% (Steel frame part 2%)
Foundation geometry 41.56m (NS direction) × 43.56m (EW direction)
*1: ( ) shows the increase of water level in the PCV

(EW direction)

Number of mass point	Weight of mass point *1 W (kN)	Rotary inertia weight *1 I_G ($\times 10^5$kN·m^2)	Shear cross-section area A_s (m^2)	Cross sectional secondery moment I (m^4)
1	—	—		
			—	—
2	—	—		
			—	—
3	—	—		
			—	—
4	58,690	48.34		
			102.7	9,702
5	67,910	55.90		
			163.9	13,576
6	80,900 (3,680)	66.58 (3.03)		
			131.6	14,559
7	87,200	125.53		
			197.8	36,427
8	166,150 (20,130)	239.13 (28.97)		
			294.0	52,858
9	177,480 (30,410)	313.72 (53.75)		
			1,914.3	338,428
10	62,400	110.32		
Total	700,730 (54,220)			

Young modulus E_c 2.57×10^7(kN/m^2)
Transverse elasticity modulus G 1.07×10^7(kN/m^2)
Poisson ratio ν 0.20
Decay h 5% (Steel frame part 2%)
Foundation geometry 41.56m (NS direction) × 43.56m (EW direction)
*1: ( ) shows the increase of water level in the PCV

Table-3.3 Foundation constant

(Ss-1)

Altitude O.P. (m)	Geological condition	S wave velocity Vs (m/s)	Unit weight γt (kN/m3)	Poisson ratio ν	Primary transverse elasticity modulus G0 (kN/m2)	Stiffness degradation ratio G/G0	Transverse elesticity modulus G (kN/m2)	S wave velocity after stiffness degradation Vs (m/s)	Decay constant h (%)
10.0									
1.9	Sand Stone	380	17.8	0.473	262,000	0.85	223,000	351	3
-10.0	Mud Stone	450	16.5	0.464	341,000	0.78	266,000	398	3
-80.0		500	17.1	0.455	436,000		340,000	442	
-108.0		560	17.6	0.446	563,000		439,000	495	
-196.0		600	17.8	0.442	653,000		509,000	530	
	FreeBase Ground	700	18.5	0.421	924,000	1.00	924,000	700	—

(Ss-2)

Altitude O.P. (m)	Geological condition	S wave velocity Vs (m/s)	Unit weight γt (kN/m^3)	Poisson ratio ν	Primary transverse elasticity modulus G_0 (kN/m^2)	Stiffness degradation ratio G/G_0	Transverse elesticity modulus G (kN/m^2)	S wave velocity after stiffness degradation Vs (m/s)	Decay constant h (%)
10.0									
1.9	Sand Stone	380	17.8	0.473	262,000	0.85	223,000	351	3
-10.0	Mud Stone	450	16.5	0.464	341,000	0.81	276,000	405	3
-80.0		500	17.1	0.455	436,000		353,000	450	
-108.0		560	17.6	0.446	563,000		456,000	504	
-196.0		600	17.8	0.442	653,000		529,000	540	
	FreeBase Ground	700	18.5	0.421	924,000	1.00	924,000	700	—

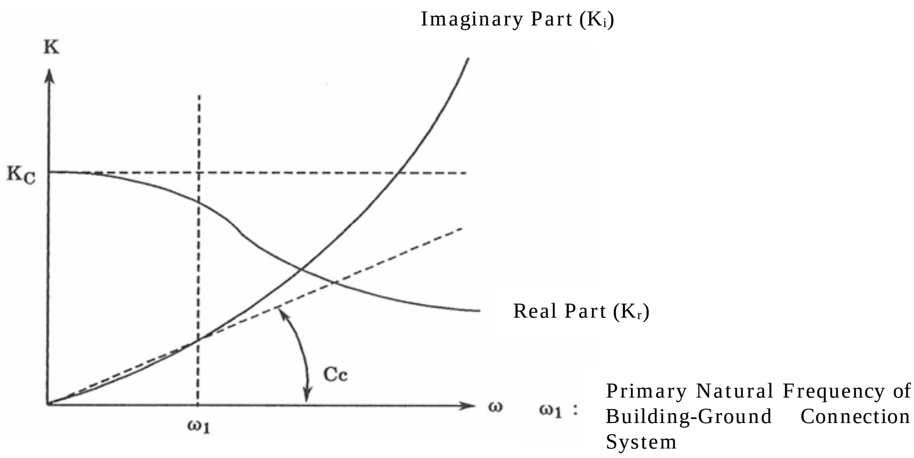
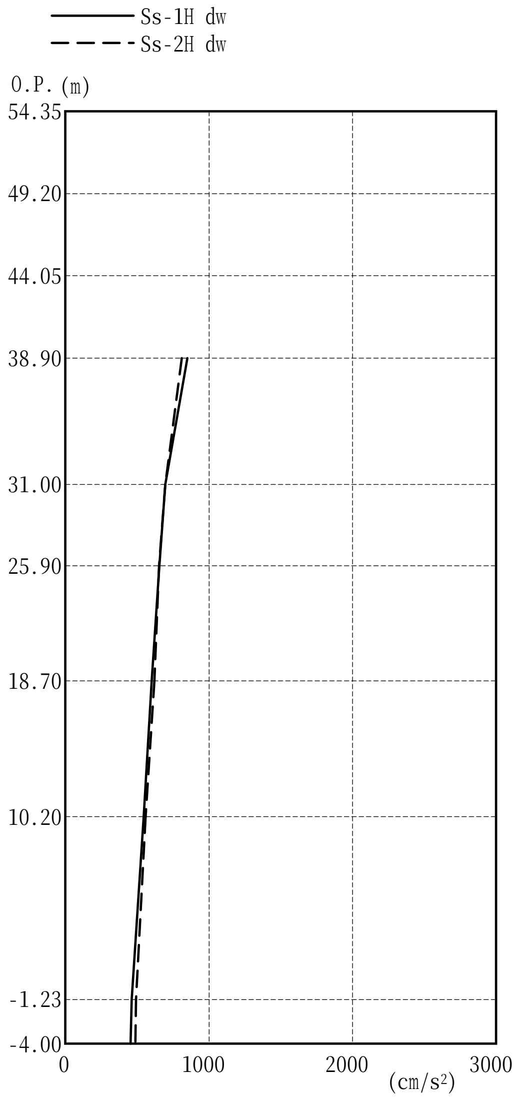


Figure-1.3.5 Simulation of Ground Spring

4. Analysis Results of Seismic Response

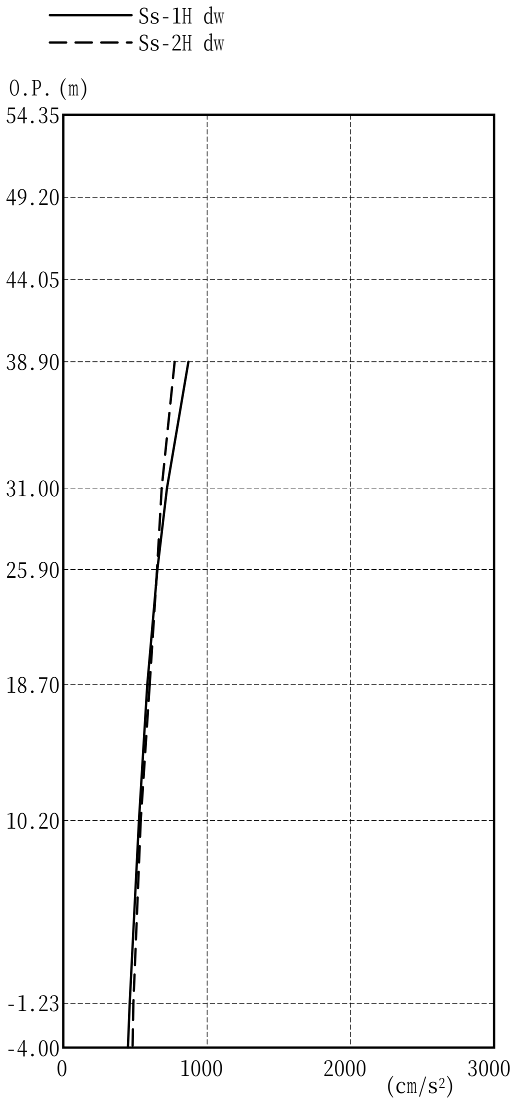
Maximum response acceleration of NS direction and EW direction obtained by the seismic response analysis is shown in Figure 4-1 and 4-2 below.



(cm/s²)

Ss-1H dw	Ss-2H dw
849	809
695	697
653	652
602	620
544	559
460	491
455	487

Figure-4.1 Maximum Response Acceleration (NS Direction)



(cm/s²)

Ss-1H dw	Ss-2H dw
868	776
722	684
653	651
585	601
527	539
461	487
451	480

Figure-4.2 Maximum Response Acceleration (EW Direction)

5. Evaluation Results of Anti-Earthquake Stability

Figure 5-1, 5-2, and 5-3 show the maximum response values for basic earthquake ground motion Ss-1 and Ss-2 on sharing skeleton curves of anti-earthquake. The maximum shearing strain was estimated at 0.12×10^{-3} (Ss-1H, N-S Direction, 1F) and it has enough margin for the evaluation standard (4.0×10^{-3}).

From the result, the present reactor building was evaluated that the building stability did not affect the facilities which were important for anti-earthquake stability.

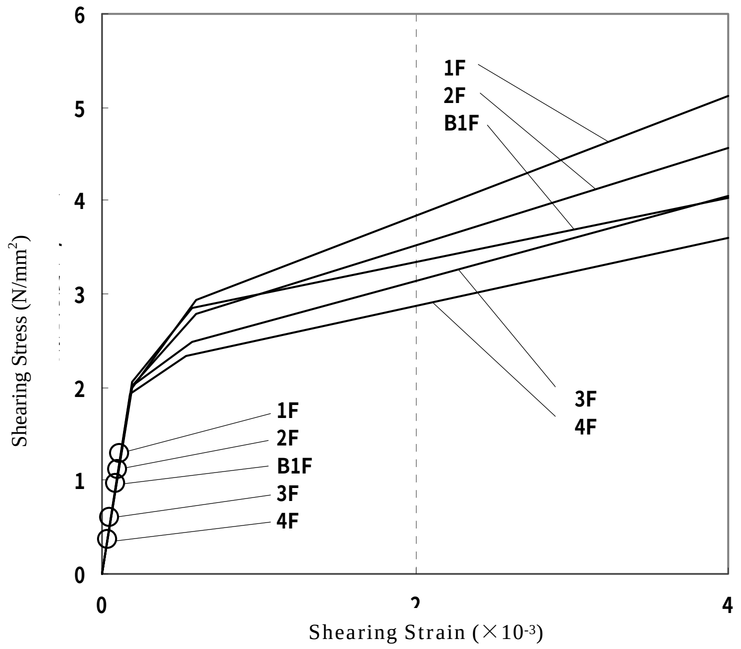


Fig.-5.1 Maximum Response on Shearing Skelton Curve (Ss-1, NS Direction)

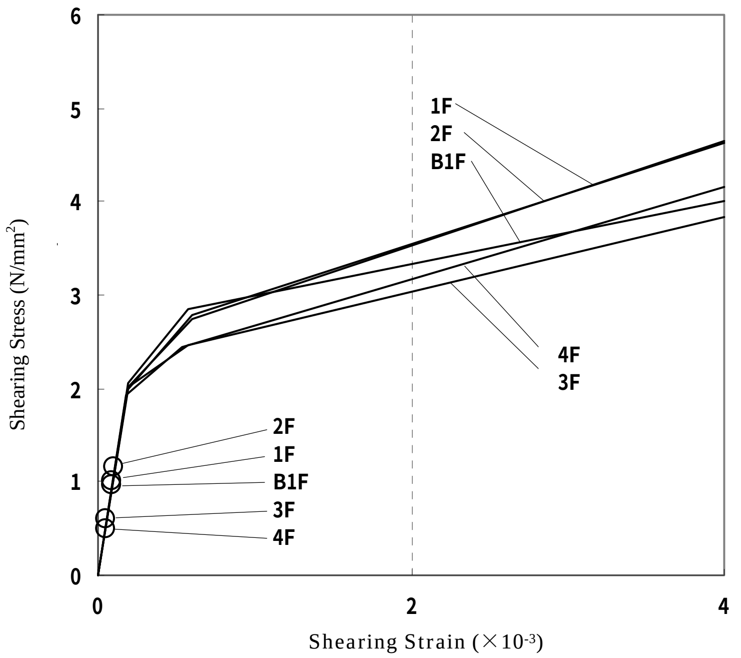


Fig.-5.2 Maximum Response on Shearing Skelton Curve (Ss-1, EW Direction)

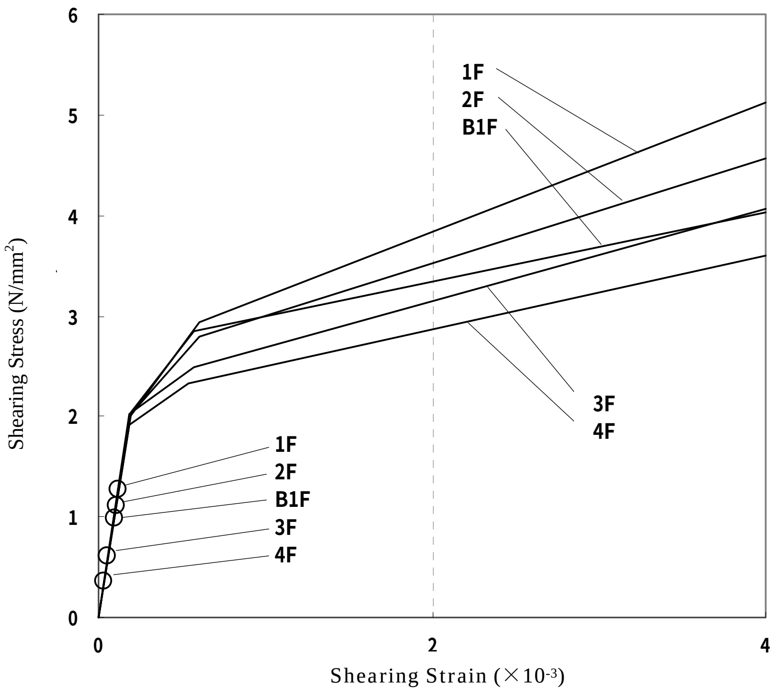


Fig.-5.3 Maximum Response on Shearing Skelton Curve (Ss-2, NS Direction)

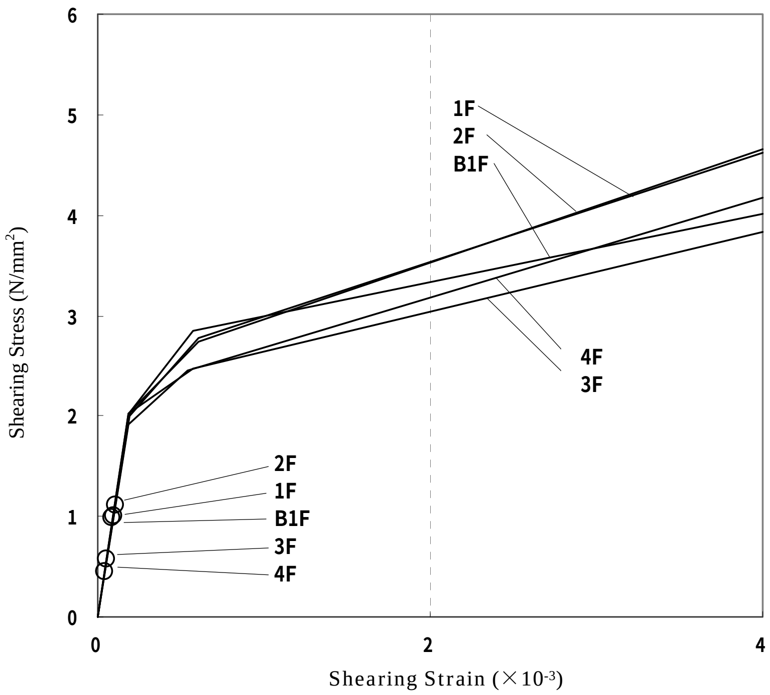


Fig.-5.4 Maximum Response on Shearing Skelton Curve (Ss-2, EW Direction)

Attachment 3: Detail of seismic safety evaluation of Reactor building of Unit 4
(Evaluation by time history response analysis method using mass system model)

1. Policy of analysis and evaluation

Seismic evaluation and evaluation of impact on the reactor building structure caused by the hydrogen explosion etc. are conducted by utilizing design basis ground motion Ss in principle and by establishing the model that can properly describe the response states of buildings, structures, and foundations. Design basis ground motion Ss-3 is not utilized in this analysis as it is obvious from past calculation example (refer to attachment 3-1) that such movement was small enough in comparison with the response result of design basis ground motion Ss-1 and Ss-2

The mass system model integrating flexural and shearing rigidity is selected as a seismic response analysis model, considering the interaction with the foundations.

While the exact cause has not been specified yet, the reactor building of Unit 4 has been partially damaged by the hydrogen explosion etc.. In this analysis, the damage in the reactor building is estimated by analyzing its pictures and such estimation is reflected in the seismic response analysis model.

Seismic evaluation and evaluation of impact on the reactor building structure is conducted by comparing the shear strain of seismic wall calculated in seismic response analysis and standard evaluation point (4.0×10^{-3}) responding to ultimate limit of reinforced concrete seismic wall.

As for ultimate limit of reinforced concrete seismic wall, as horizontal seismic force is dominant while vertical seismic force is negligible, seismic response analysis is conducted for horizontal force only.

The evaluation process of seismic response analysis for the reactor building of Unit 4 is described in Figure-1.1.

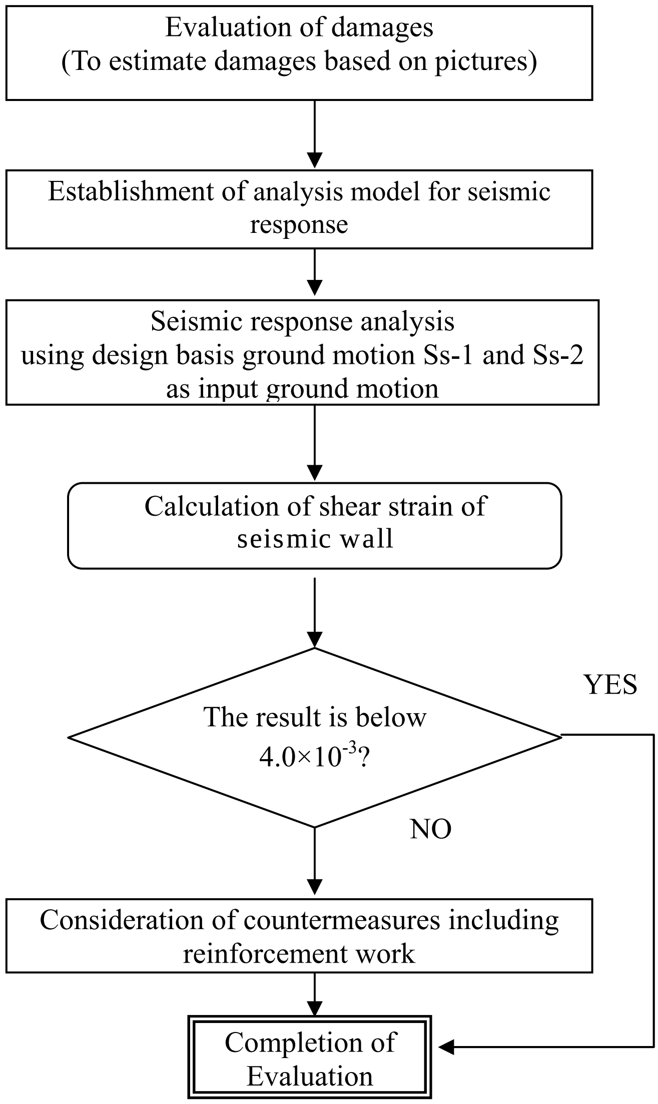


Figure-1.1 Evaluation process of seismic response analysis for the reactor building of Unit 4

2. Evaluation of Damage Situation

Reactor Building of Unit 4, though the exact cause has not been specified yet, has partially been damaged due to hydrogen explosion etc.. Damage situation of the reactor building is estimated based on pictures and reflected in a seismic response analysis model. In case we cannot have evaluated parts judging from their exterior pictures, we have evaluated whether they have been damaged based on information currently obtained from the investigation result of the inside of the building. Shooting dates etc. of reference pictures are put together in Appendix 3-2.

We will show you how to evaluate each part of damage situation as follows.

a. Exterior Wall/ Roof Truss

We have evaluated exterior walls and roof trusses as damaged parts, as, judging from their exterior pictures, we can confirm damages. We have also evaluated exterior walls as damaged ones that we confirm have partially peeled off (Figure-2.1)

b. Spent Fuel Pool

We have evaluated the spent fuel pool has not been damaged, as we confirm that a certain amount of water has been sprayed into the spent fuel pool, judging from pictures taken with a nose-mounted camera of a concrete pumping vehicle and that no water leak etc. has not occurred at the second floor which is located in the lower part of the spent fuel pool. (Figure-2.2)

c. Pool where equipment is temporarily placed

We have evaluated it has not been damaged as, judging from pictures of exterior walls, we did not confirm any damages near the pool where equipment is temporarily placed. (Figure 2.3)

d. Shell Wall

We confirm that the shell wall on first and second floors has not been damaged as a result of the investigation of the inside of the building. On the third floor, we confirm that the damaged exterior wall is 650 mm thick in the maximum and that the 1,000 mm-thick one has not been damaged. We have evaluated the shell wall on the third floor has not been damaged, as it is 1,850 mm thick. (Figure-2.4)

e. Floor Slab

We confirm floor slabs on first and second floors have not been damaged as a result of the investigation of the inside of the building. On the third floor, we have evaluated the slab has not been damaged, as we could not confirm any damages on the ceiling slabs we looked up from the second floor (the floor slab on the third floor) when we investigated the inside of the building. (Figure-2.5) On the fourth floor and up, we decided to evaluate based on damage situation of exterior walls, as we had not obtained the result of the investigation of the inside of the building. On fourth and fifth floors, we have evaluated that the floor slabs equal to or thinner than exterior walls might have been damaged, as exterior walls have been damaged.

North Side

West side

East Side

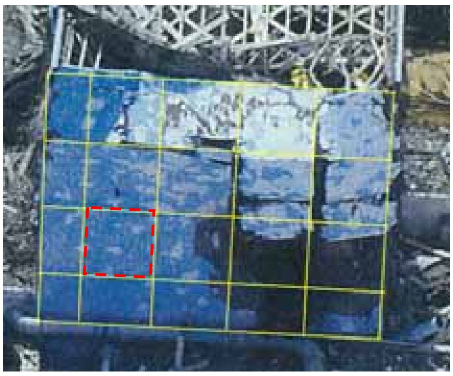
South Side

Figure-2.1 Situation of Exterior Walls

Inside of Spent Fuel Pool

Lower Part of Spent Fuel Pool

Figure-2.2 Situation of Spent Fuel Pool



Location of Pool where equipment is temporarily placed

Figure-2.3 Situation of pool where equipment is temporarily placed

Figure-2.4 Situation of Shell Wall

Exterior Wall on the first floor

Exterior Wall on the second floor

Floor on the second floor

Ceiling of the second floor

Figure-2.5 Situation of Inside of Building (first and second floors)

3. Input Ground Motion Used for Analysis

As input earthquake motion for the reactor building of Unit 4, we have used the design basis ground motion Ss-1 and Ss-2 assumed in the free surface level of base stratum in "Interim Report on Evaluation Result of Earthquake-Proof in Fukushima Daiichi Nuclear Power Station regarding the amendment of 'Guideline in Evaluation of Facilities of Nuclear Reactors to Produce Power' (Nuclear Admin Report to the Authorities 19 No. 603 dated on March 31, 2008).

A conceptual diagram of input ground motion used in earthquake response analysis is shown in Figure-3.1. Based on one-dimensional wave phenomena, ground motion to be inputted in the model is evaluated as ground response of design basis ground motion Ss assumed in the free surface level of base stratum. Also, by adding shear force at the building foundation base level to the input ground motion, notch effect of the ground is taken into account.

Among these, acceleration wave profile of design basis ground motion Ss-1 and Ss-2 at the free surface level of base stratum（O.P. -196.0m）is shown in Figure-3.2.

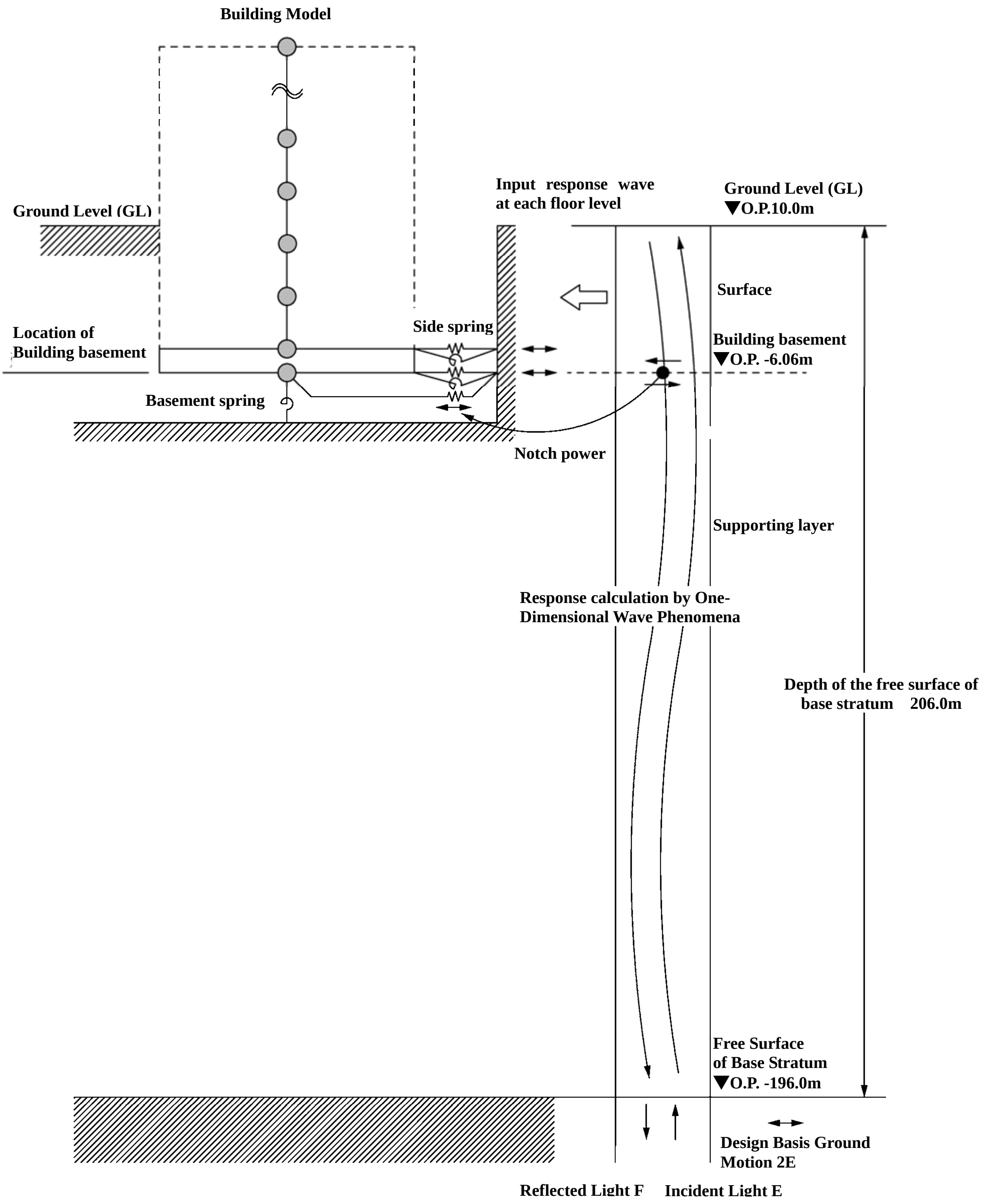
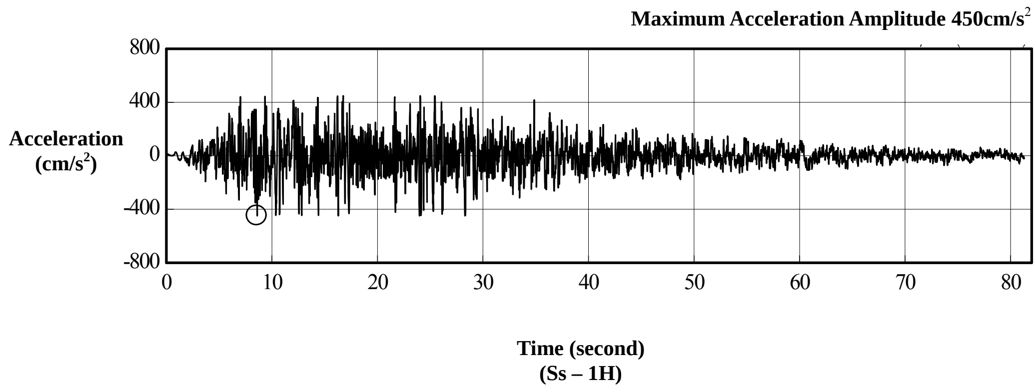


Figure-3.1 A conceptual diagram of Input Ground Motion used in Earthquake response analysis



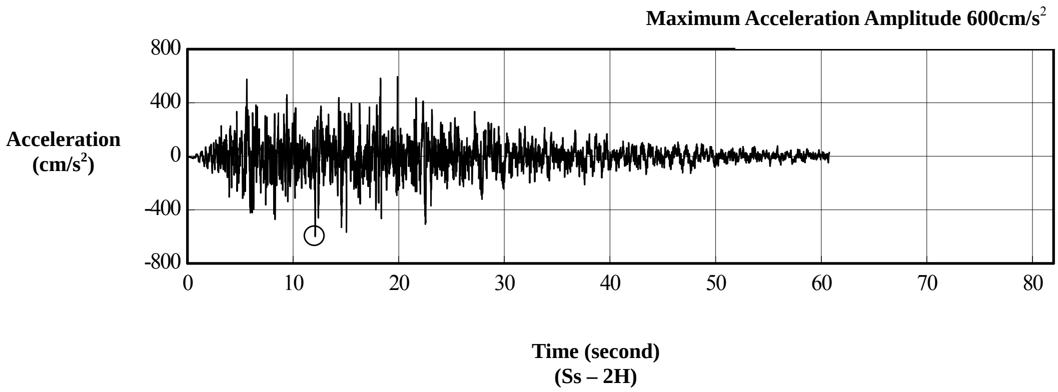


Figure-3.2 Chronicle acceleration wave profile (horizontal direction) of ground motion at free surface of base stratum

4. Seismic Response Analysis Model

Seismic response of the reactor building against the design basis ground motion Ss is conducted by the dynamic analysis using the input seismic response calculated in the "3. Input Ground Motion Used for Analysis."

This study formulates a new analysis model for seismic response based on the former model made in "Interim Report (revised version), Evaluation results of anti-earthquake stability by a revision of guidance for appraisal for anti-earthquake design regarding commercial reactor facilities, Fukushima Daiichi Nuclear Power Station" (Nuclear Admin Report to the Authorities 21 No.110 dated on June 19, 2009).

We had a periodic inspection of the reactor building of Unit 4 when the earthquake occurred. Hence, conditions during the inspection are reflected. While the exact cause has not been specified yet, the reactor building of Unit 4 has been partially damaged by the hydrogen explosion etc.. Therefore, the analysis model is formulated based on damage situation evaluated in "2. Evaluation of Damage Situation" In addition, we assume that the weight of collapsed parts is supported by the floor of the lower floor. For example, the weight of collapsed parts on the fifth floor and up is supported by the floor of the fifth floor. Figure-4.1 shows the damage situation of the reactor building of Unit 4 (elevation) and Figure-4.2 shows the damage situation (plane).

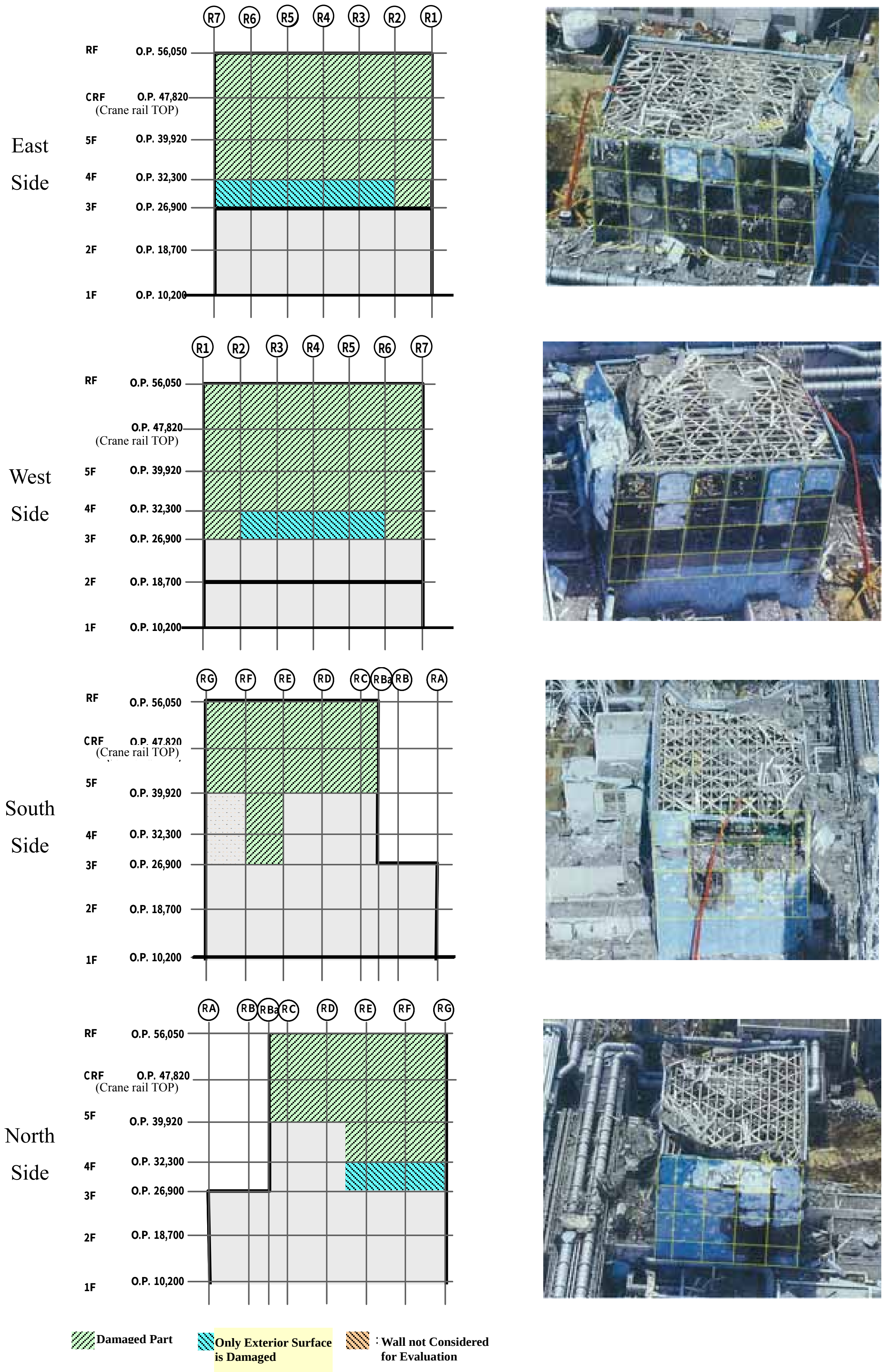


Figure-4.1 Damage Situation of Reactor Building of Unit 4 (elevation)

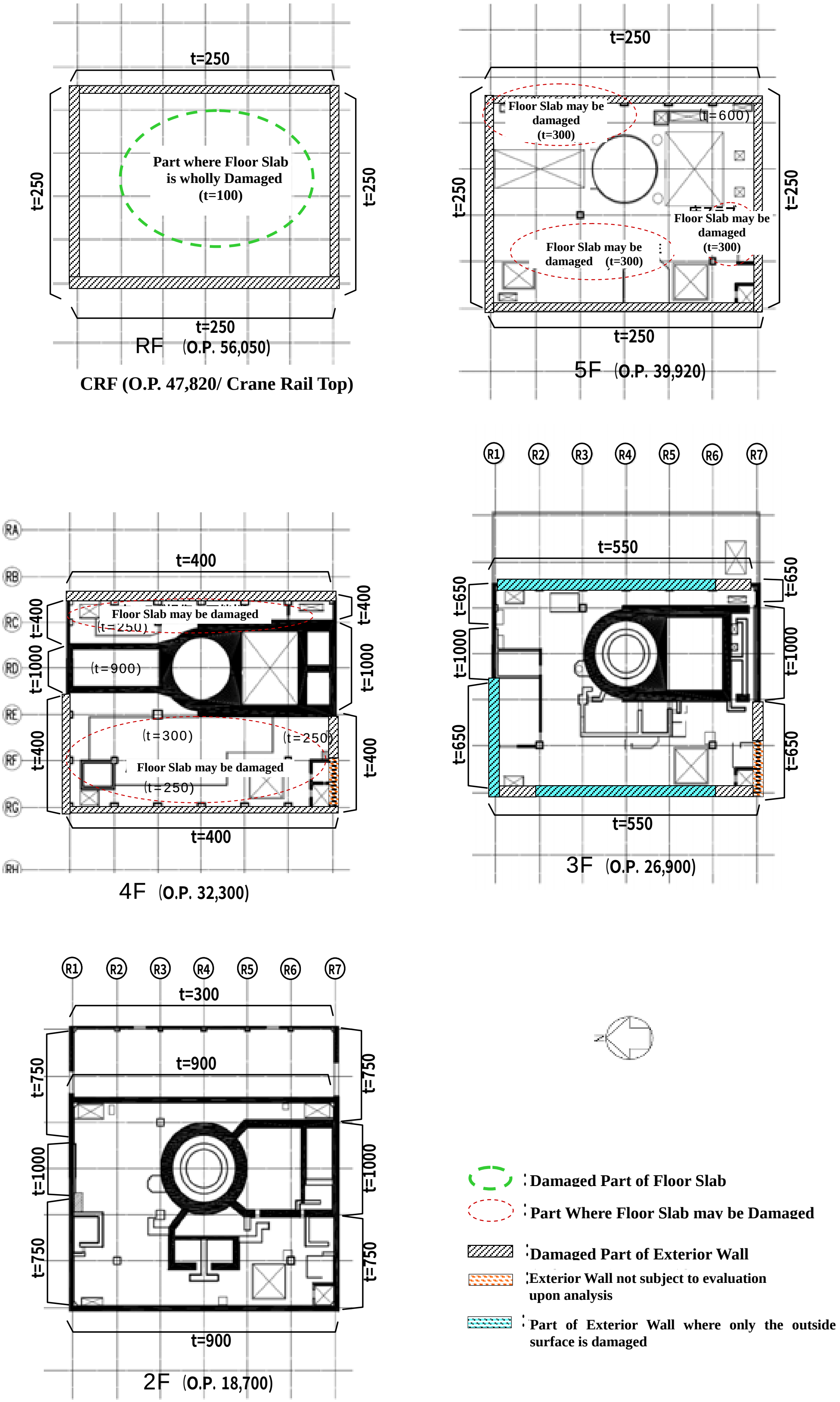


Figure-4.2 Damaged Situation of the Reactor Building of Unit 4 (Plane)

(1) Analysis Model for Seismic Response in Horizontal Direction

An analysis model for seismic response in the horizontal direction uses a particle system which considers bending transformation and sharing transformation of the building, and the model is building-ground connection one in which ground is evaluated by equivalent springs, as shown in Figure-4.3 and Figure-4.4. The effects of connection system between the building and ground are evaluated by ground springs and input seismic response. Physical properties of concrete for the analysis are shown in Table-4.1 and other data for a building analysis model are shown in Table-4.2.

We have defined ground constant based on the assumption of horizontally layered ground and on the shearing strain level in the earthquake. The ground constant for the analysis is shown in Table-4.3.

In the analysis model in the horizontal direction, ground springs at the bottom of foundation are revised in horizontal layers with reference to the method shown in “JEAG 4601-1991” and we approximately evaluated the sway and locking spring constants based on swinging admittance theory. Regarding ground springs in the side of buildings in embedded parts, we evaluated horizontal and rotational springs in an approximate model based on the Novak spring with reference to the method shown in “JEAG 4601-1991” using ground constants located in the side of buildings.

Ground springs are evaluated as complex stiffness depending on the frequency of vibration. We adapt slope of line linking between an imaginary value corresponding to primary natural frequency of the building and ground connection system and the origin as damping coefficients (Cc) and we can obtain approximate ground springs.

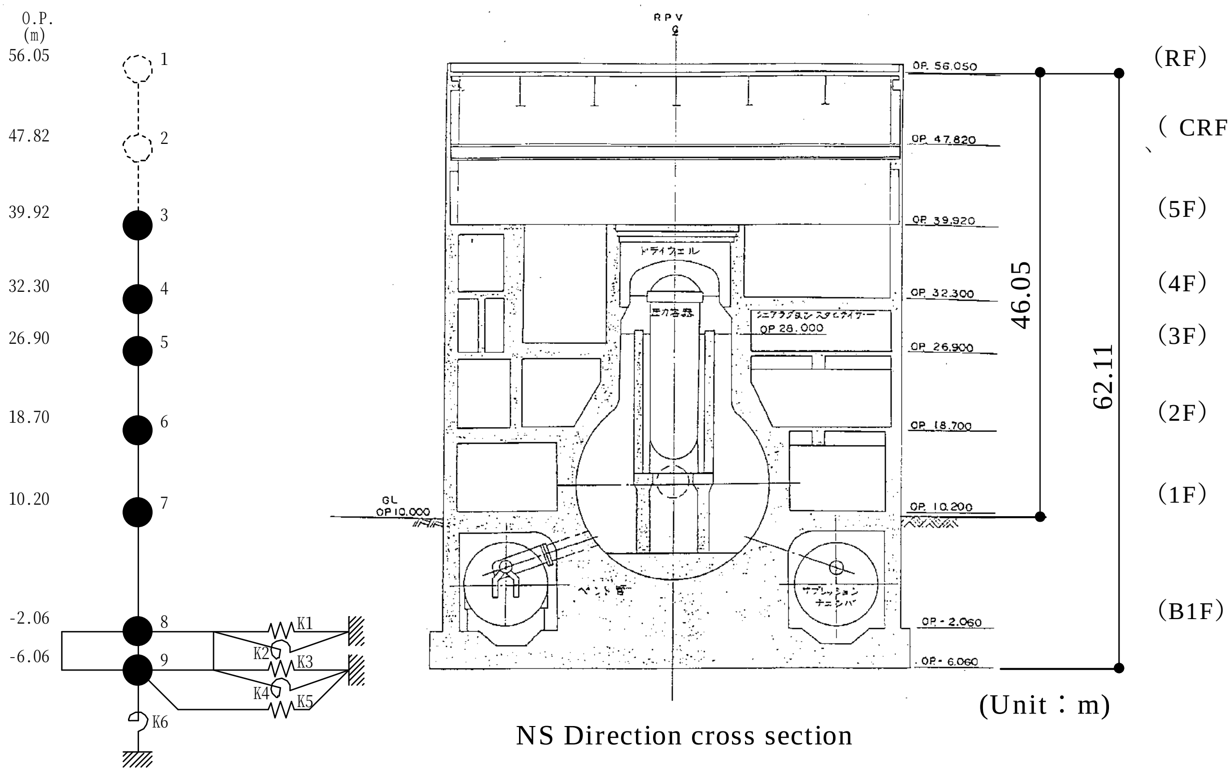


Figure 4-3 Analysis Model for Seismic Response of the Reactor Building of Unit 4 (N-S Direction)

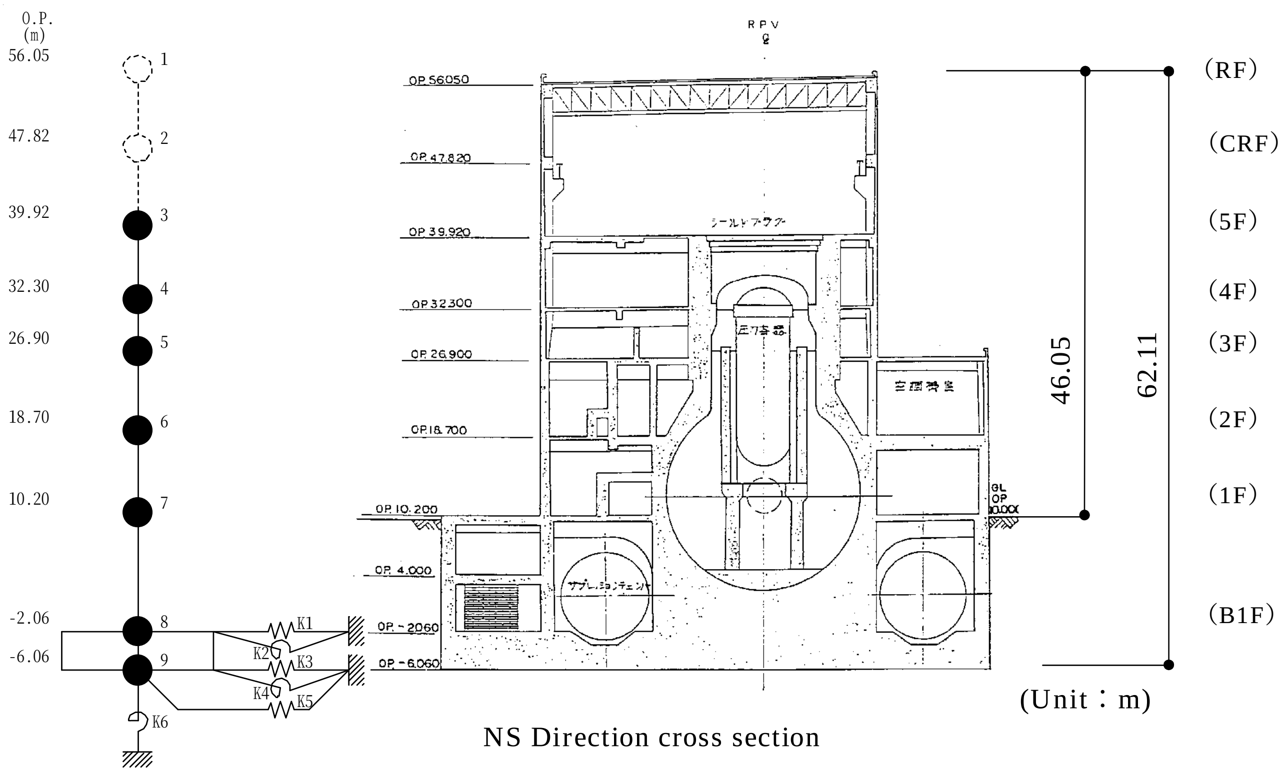


Figure 4-4 Analysis Model for Seismic Response of the Reactor Building of Unit 4 (E-W Direction)

Table 4-1 Physical Factors for Seismic Response Analysis

	Strength *1 Fc (N/mm^2)	Young Coefficient *2 E (N/mm^2)	Sharing Elastic Coefficient*2 G (N/mm^2)	Poisson's Ratio ν	Weight of Unit Volume*3 γ (kN/m^3)
Concrete	35.0	2.57×10^4	1.07×10^4	0.2	24
Reinforced Steel	SD345 equivalent (SD35)				
Steel Material	SS400 equivalent (SS41)				

*1：Strength adopts the more realistic strength "hereinafter Real Strength". The real strength is decided by average value of compressed strength considering a scattering of the past test data.
*2：The value shows based on the real strength.
*3：The value shows a value of reinforced steel.

Table 4-2 Factors of Building Analysis Model

(N-S Direction)

Weight Point	Weight W (kN)	Rotation Inertia Weight I_G (x 10^5kN・m^2)	Cross Section of Sharing A_S (m^2)	Cross Section Secondary Moment I (m^4)
1	—	—		
			—	—
2	—	—		
			—	—
3	114,850	211.39		
			150.8	13,068
4	88,770	163.44		
			103.4	15,942
5	117,030	215.39		
			223.4	45,026
6	121,930	224.49		
			175.4	46,774
7	207,300	381.60		
			460.4	114,194
8	287,050	574.38		
			2,812.6	562,754
9	132,390	264.88		
Total	1,069,320			

Young Coefficient Ec	2.57x10^7 (kN/m^2)
Sharing Elastic Coefficient G	1.07x10^7 (kN/m^2)
Poisson's Ratio ν	0.20
Attenuation	5% (Steel 2%)
Shape of Basement	49.0 m (N-S) x 57.4m (E-W)

(E-W Direction)

Weight Point	Weight W (kN)	Rotation Inertia Weight I_G (x 10^5kN・m^2)	Cross Section of Sharing A_S (m^2)	Cross Section Secondary Moment I (m^4)
1	—	—		
			—	—
2	—	—		
			—	—
3	114,850	118.55		
			90.4	6,491
4	88,770	91.66		
			105.8	6,388
5	117,030	215.39		
			167.5	32,815
6	121,930	224.49		
			166.4	46,303
7	207,300	569.22		
			424.5	136,323
8	287,050	828.96		
			2,812.6	772,237
9	132,390	346.27		
Total	1,069,320			

Young Coefficient Ec	2.57x10^7 (kN/m^2)
Sharing Elastic Coefficient G	1.07x10^7 (kN/m^2)
Poisson's Ratio ν	0.20
Attenuation	5% (Steel 2%)
Shape of Basement	49.0m (N-S) x 57.4m (E-W)

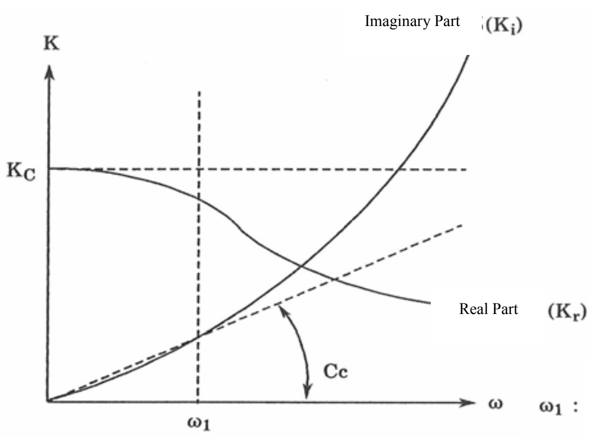
Table 4-3 Ground Factors

(Ss-1)

Elevation O.P. (m)	Geology	S Wave Velocity (Vs) (m/s)	Weight of Unit Volume γt (kN/m³)	Poisson's Ratio ν	Primary Sharing Elastic Coefficient (Go) (kN/m²)	Decrease Ratio of Strength (G/Go)	Sharing Elastic Coefficient (G) (kN/m²)	Vs after Decrease of Strength (Vs) (m/s)	Damp Factor h (%)
10.0									
1.9	Sand Stone	380	17.8	0.473	262,000	0.85	223,000	351	3
-10.0	Mud Stone	450	16.5	0.464	341,000	0.78	266,000	398	3
-80.0		500	17.1	0.455	436,000		340,000	442	
-108.0		560	17.6	0.446	563,000		439,000	495	
-196.0		600	17.8	0.442	653,000		509,000	530	
	Free Base Ground	700	18.5	0.421	924,000	1.00	924,000	700	—

(Ss-2)

Elevation O.P. (m)	Geology	S Wave Velocity (Vs) (m/s)	Weight of Unit Volume γt (kN/m³)	Poisson's Ratio ν	Primary Sharing Elastic Coefficient (Go) (kN/m²)	Decrease Ratio of Strength (G/Go)	Sharing Elastic Coefficient (G) (kN/m²)	Vs after Decrease of Strength (Vs) (m/s)	Damp Factor h (%)
10.0									
1.9	Sand Stone	380	17.8	0.473	262,000	0.85	223,000	351	3
-10.0	Mud Stone	450	16.5	0.464	341,000	0.81	276,000	405	3
-80.0		500	17.1	0.455	436,000		353,000	450	
-108.0		560	17.6	0.446	563,000		456,000	504	
-196.0		600	17.8	0.442	653,000		529,000	540	
	Free Base Ground	700	18.5	0.421	924,000	1.00	924,000	700	—

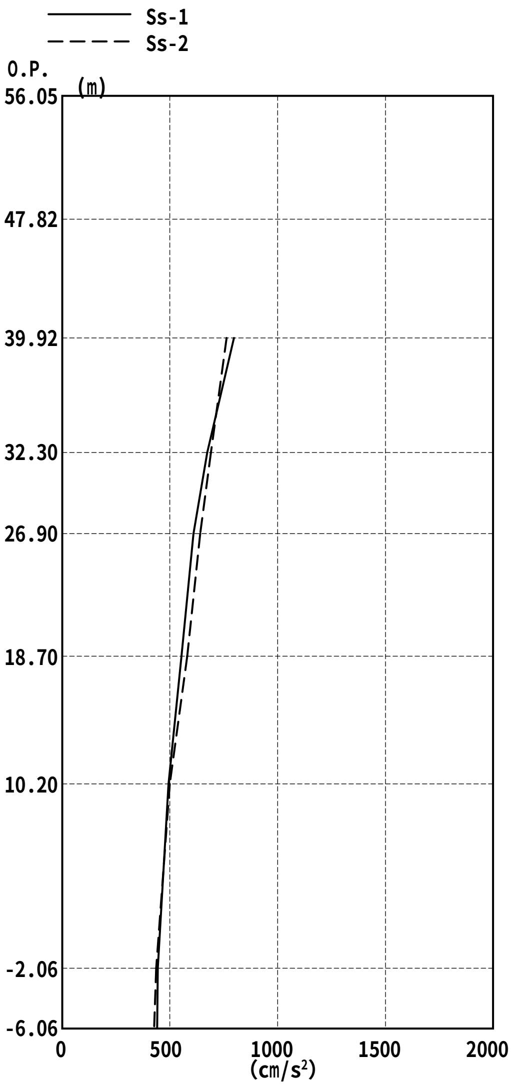


ω_1 : Primary Natural Frequency of Building-Ground Connection System

Figure 4-5 Simulation of Ground Spring

5. Analysis Results of Seismic Response

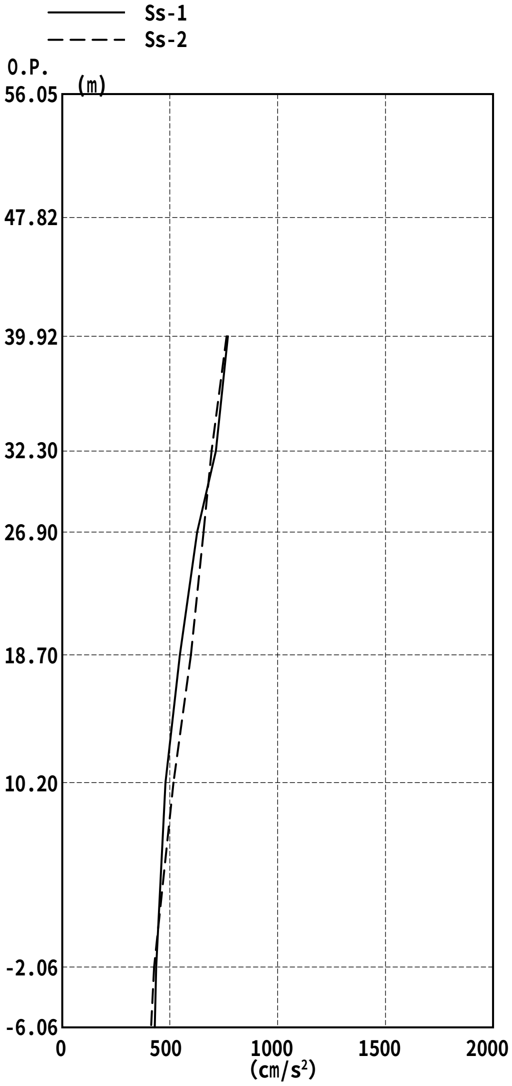
Maximum response acceleration of N-S direction and E-W direction obtained from the seismic response analysis is shown in Figure-5.1 and 5.2 below.



(cm/s²)

Ss-1	Ss-2
796	761
673	690
610	640
554	581
494	500
445	436
439	426

Figure-5.1 Maximum Response Acceleration (N-S Direction)



(cm/s²)

Ss-1	Ss-2
770	763
714	692
626	656
545	598
478	518
435	427
430	412

Figure 5-2　Maximum Response Acceleration (E-W Direction)

6. Evaluation Results of Earthquake-proof Security

Figure-6.1, 6.2, 6.3 and 6.4 show maximum response values to design basis ground motion Ss-1 and Ss-2 in shearing skeleton curves of earthquake-resistant walls. The maximum shearing strain was estimated to be 0.17×10^{-3} (Ss-1H and Ss-2H and E-W direction of 1F) and it has enough margin for the basis value for evaluation (4.0×10^{-3}).

From the above-mentioned analysis, we have evaluated the reactor building will not have spillover effects on facilities which were important for earthquake-proof safety.

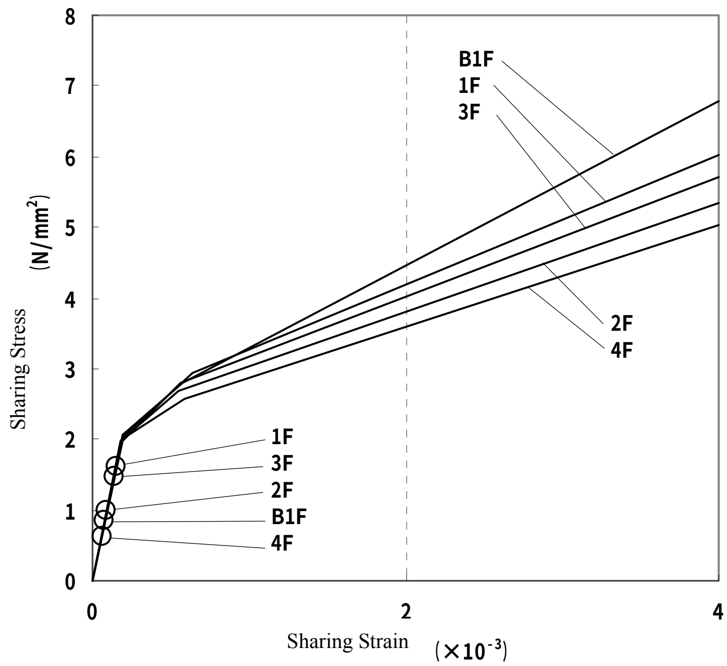


Figure 6-1 Maximum Response Value in Sharing Skelton Curves (Ss-1, N-S Direction)

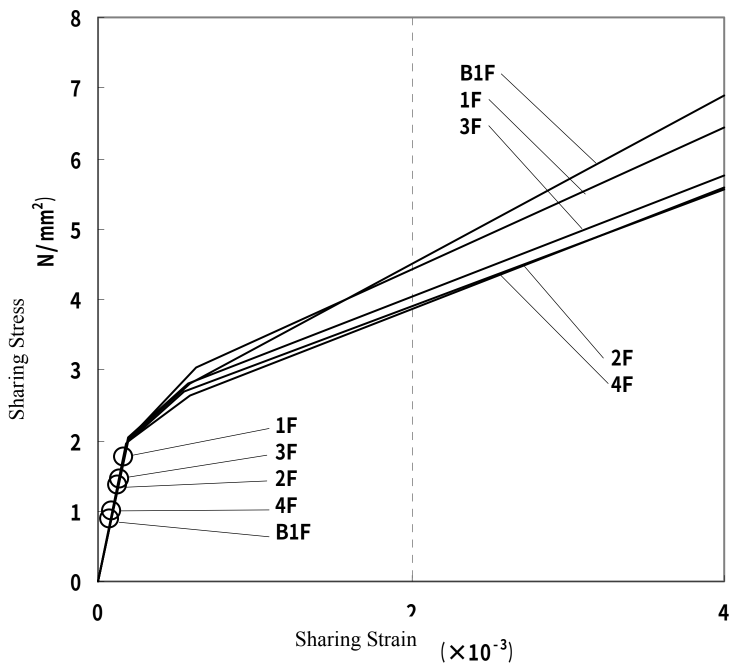


Figure 6-2 Maximum Response Value in Sharing Skelton Curves (Ss-1, E-W Direction)

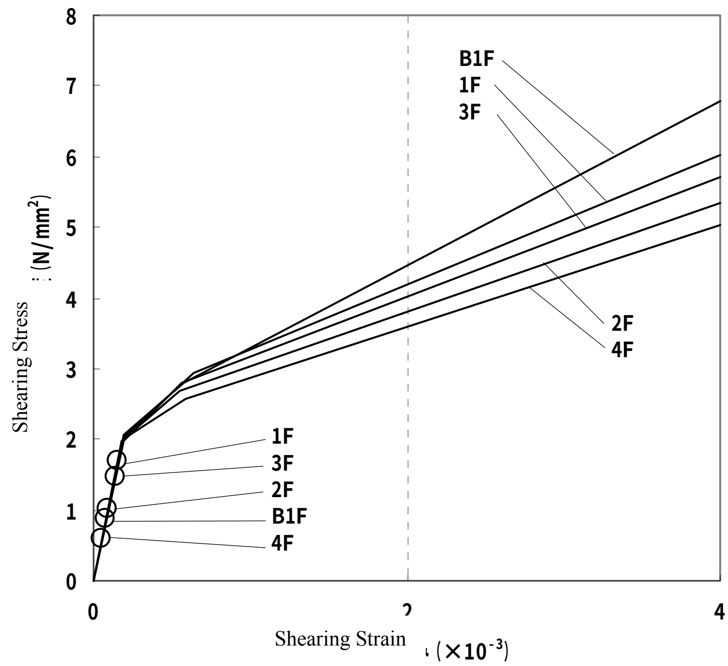


Figure 6-3 Maximum Response Value in Sharing Skelton Curves (Ss-2, N-S Direction)

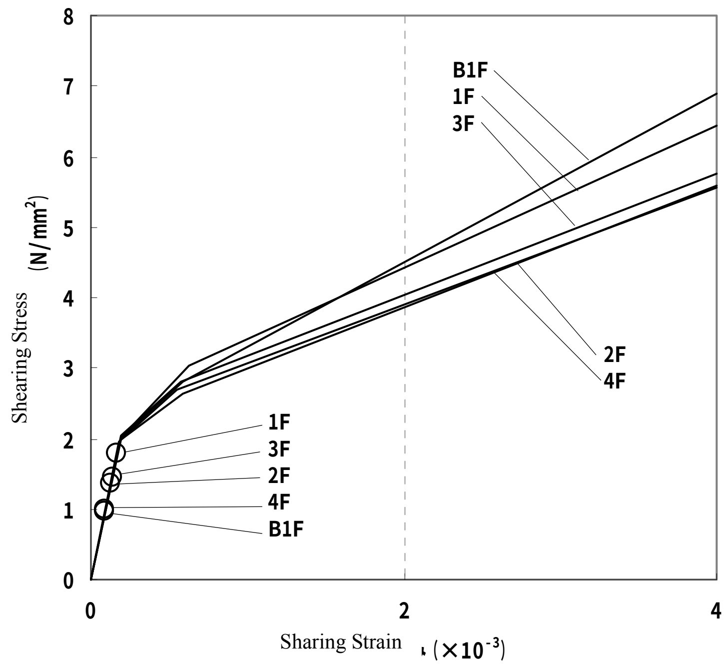
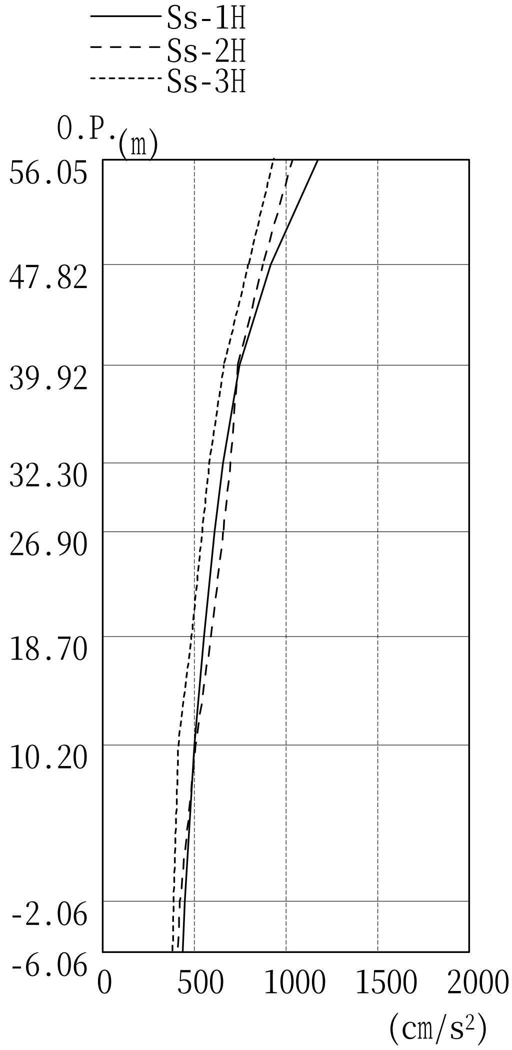


Figure 6-4 Maximum Response Value in Sharing Skelton Map (Ss-2, E-W Direction)

The evaluation result of the anti-quake safety in relation to the amendment to “the guideline for examining the anti-quake design of power station nuclear reactor facilities”

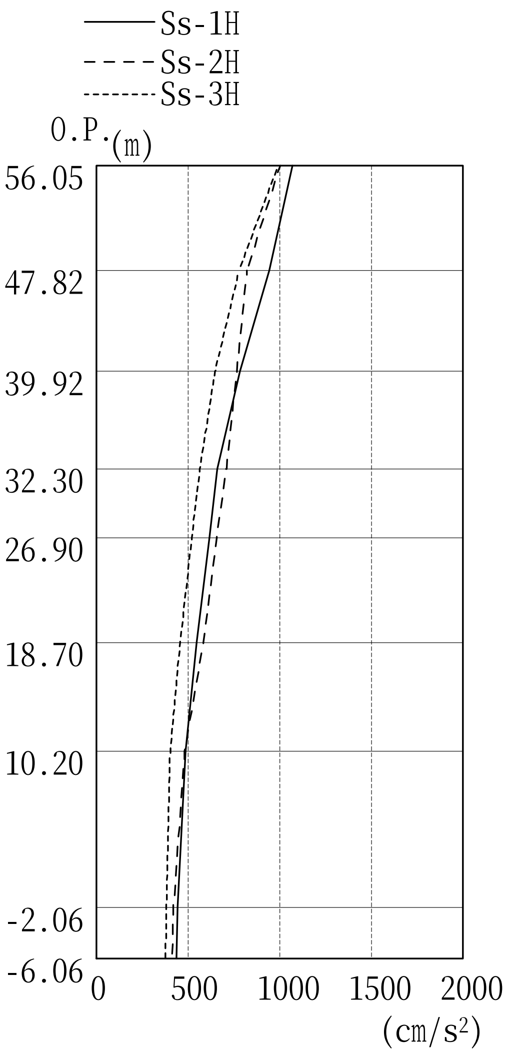
Below is the extract of the evaluation result of the anti-quake safety of the Reactor Building, Unit 4 from the interim report (revised), Fukushima Daiichi Nuclear Power Station: the evaluation result of the anti-quake safety in relation to the amendment to “the guideline for examining the anti-quake design of power station nuclear reactor facilities” (nuclear admin report to the authorities No. 110 June 19, 2009)



(cm/s²)

Ss-1H	Ss-2H	Ss-3H
1174	1037	932
918	875	796
749	738	662
657	698	582
613	659	543
553	592	486
502	509	413
447	423	388
439	411	382

Figure 1: the maximum response acceleration (direction NS)



(cm/s²)

Ss-1H	Ss-2H	Ss-3H
1070	1004	992
945	822	779
783	766	648
661	710	565
617	657	521
548	584	458
488	483	404
445	422	382
438	415	378

Figure 2: the maximum response acceleration (direction EW)

Table 1: shear deformation of the anti-quake wall (NS direction)

($\times 10^{-3}$)

FL	Ss-1H	Ss-2H	Ss-3H	threshold
CRF	0.10	0.09	0.08	≦2.0
5F	0.17	0.15	0.14	
4F	0.05	0.05	0.04	
3F	0.08	0.08	0.07	
2F	0.09	0.09	0.08	
1F	0.15	0.16	0.13	
B1F	0.08	0.08	0.07	

Table 2: shear deformation of the anti-quake wall (EW direction)

($\times 10^{-3}$)

FL	Ss-1H	Ss-2H	Ss-3H	threshold
CRF	0.12	0.12	0.11	≦2.0
5F	0.30	0.20	0.19	
4F	0.08	0.08	0.07	
3F	0.11	0.11	0.10	
2F	0.12	0.12	0.10	
1F	0.16	0.17	0.14	
B1F	0.08	0.09	0.07	

Photos used to evaluate damages (Unit 4)

【external wall】

○ As of April 13 2011

・ From photos taken on March 24, we checked the damage to the building and constructed the model to analyze damages (figure 1).

North side

West side

East side

South side

Figure 1: damages to the building (taken on March 24)

○ As of May 10, 2011

・ As for west side and south side, we took additional photos on May 10. From these photos, we can confirm that the damage hasn't propagated from March 24.

West side

South side

Figure 2: damages to the west side and south side of the building (taken on May 10)

【internal wall】

○ As of April 13, 2011

・ As we couldn't conduct investigation inside the building, we decided to evaluate from external photos and drawings etc.

○ As of April 28, 2011

・ From photos taken by a camera attached to the boom of the concrete pumping vehicle, we could confirm that certain level of water was maintained in the spent fuel pool (figure 3).

Figure 3: the status inside the spent fuel pool (taken on April 28)

○ As of May 22, 2011

・ We checked 1FL and 2FL inside the building. At this moment, we do not see damages to the internal wall, floor slab and ceiling slab on 1FL and 2FL. Figure 4 is a set of photos inside the building and figure 5 indicates from where these photos are taken.

①internal wall, 1FL

②external wall, 1FL

③external wall, 1FL

④ceiling, 1FL

Figure 4 (1): the status inside the building (taken from May 19 to May 21)

⑤internal wall. 2FL

⑥external wall, 2FL

⑦ shell wall, 2FL

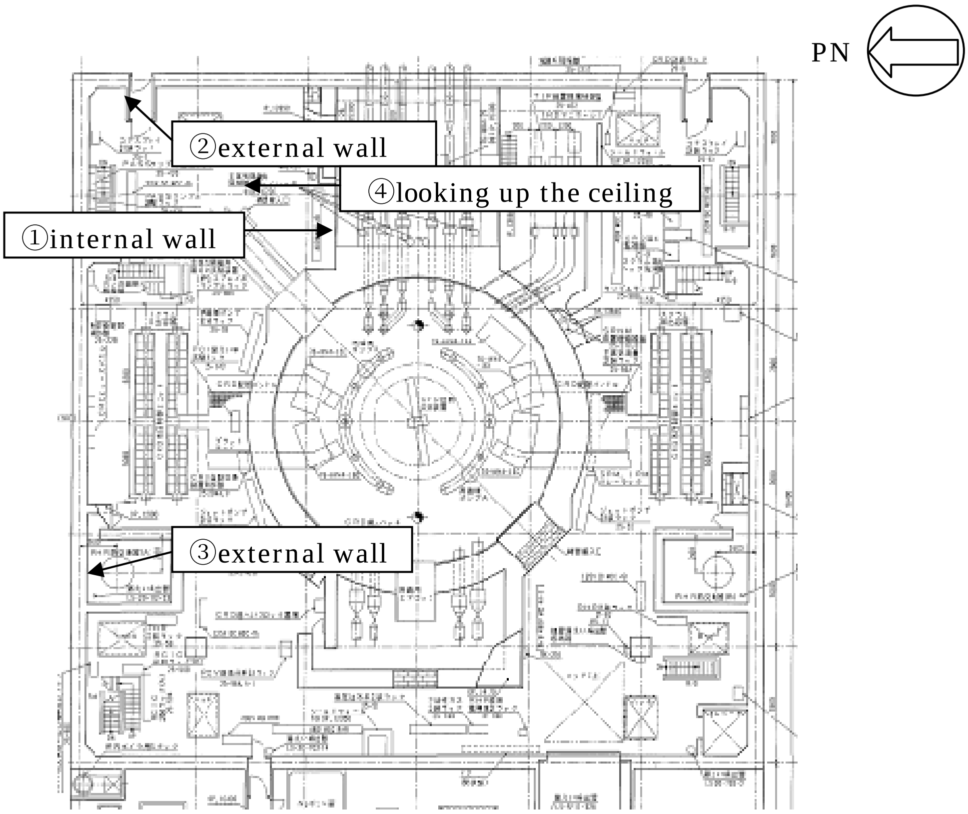
⑧shell wall, 2FL

⑨lower part of spent fuel pool

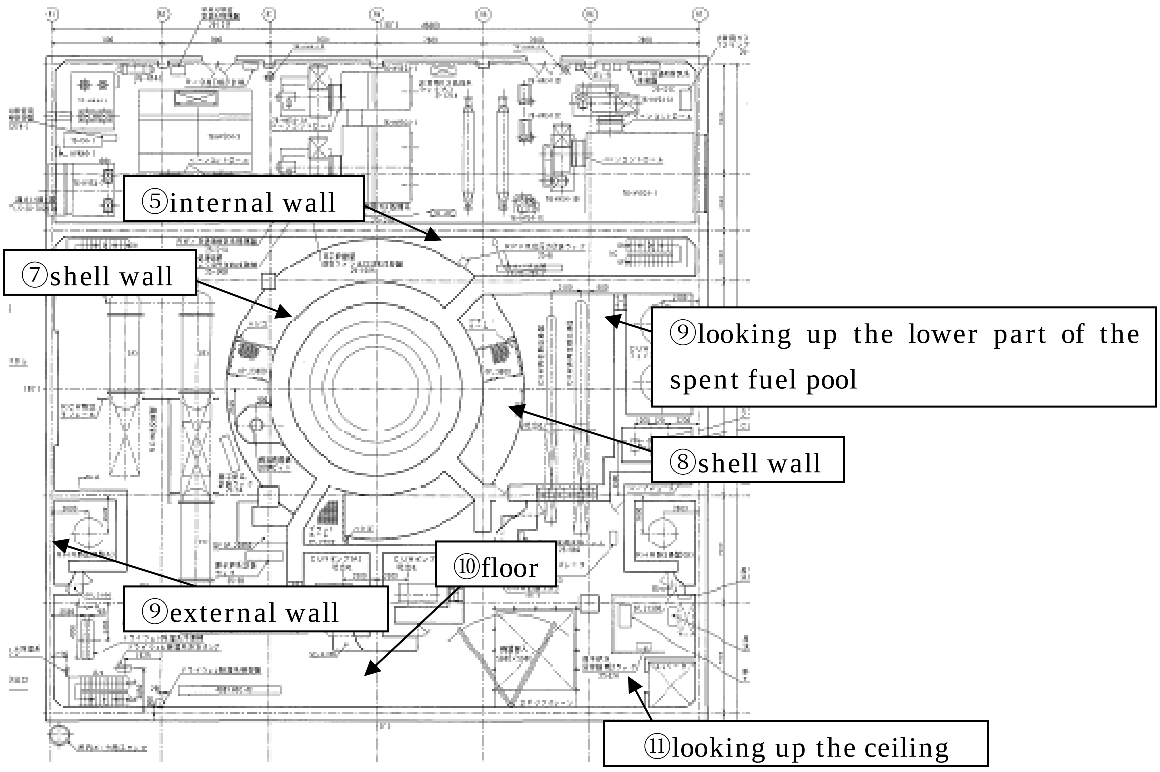
⑩floor, 2FL

⑪ceiling, 2FL

Figure 4(2): the status inside the building (taken from May 19 to May 21)



1FL



2FL

Figure 5: positions from where we took photos inside the building

Appendix 4-4: the detail of the evaluation result of the anti-quake safety of the Reactor Building, Unit 4
(local evaluation by three-dimensional FEM analysis)

1. The policy for examination and evaluation

As for the Reactor Building of Unit 4, given that the external wall from 5FL to 3FL is damaged in a complex way, we will construct a detailed three-dimensional FEM model from 2FL and above and will evaluate the anti-quake safety of the Reactor Building against the design basis ground motion Ss by stress analysis. As the main anti-quake component of 4FL and 3FL with damages to the external wall is the Spent Fuel Pool, we evaluate these two floors centering on the Spent Fuel Pool.

The horizontal drawing of the pool is figure 1.1 and the vertical drawing is figure1.2.

We will evaluate the anti-quake safety as indicated in figure 1-3 and as listed below:

・ We will construct the three-dimensional FEM model from the floor around the Spent Fuel Pool, 2FL (O.P.18.7m) to the floor, 5FL (O.P.39.92m) that simulates damage by the explosion etc.

・ We will set out the load conditions and load combinations such as the dead load, the static water pressure, the temperature load, the earthquake load based on the result of the earthquake response analysis, the dynamic water pressure at the time of the earthquake.

・ We will conduct the elasto-plastic analysis taking account of the plasticity of reinforced concrete and calculate stress and strain at the Spent Fuel Pool.

・ We compare figures the evaluation standard in order to evaluate the anti-quake safety.

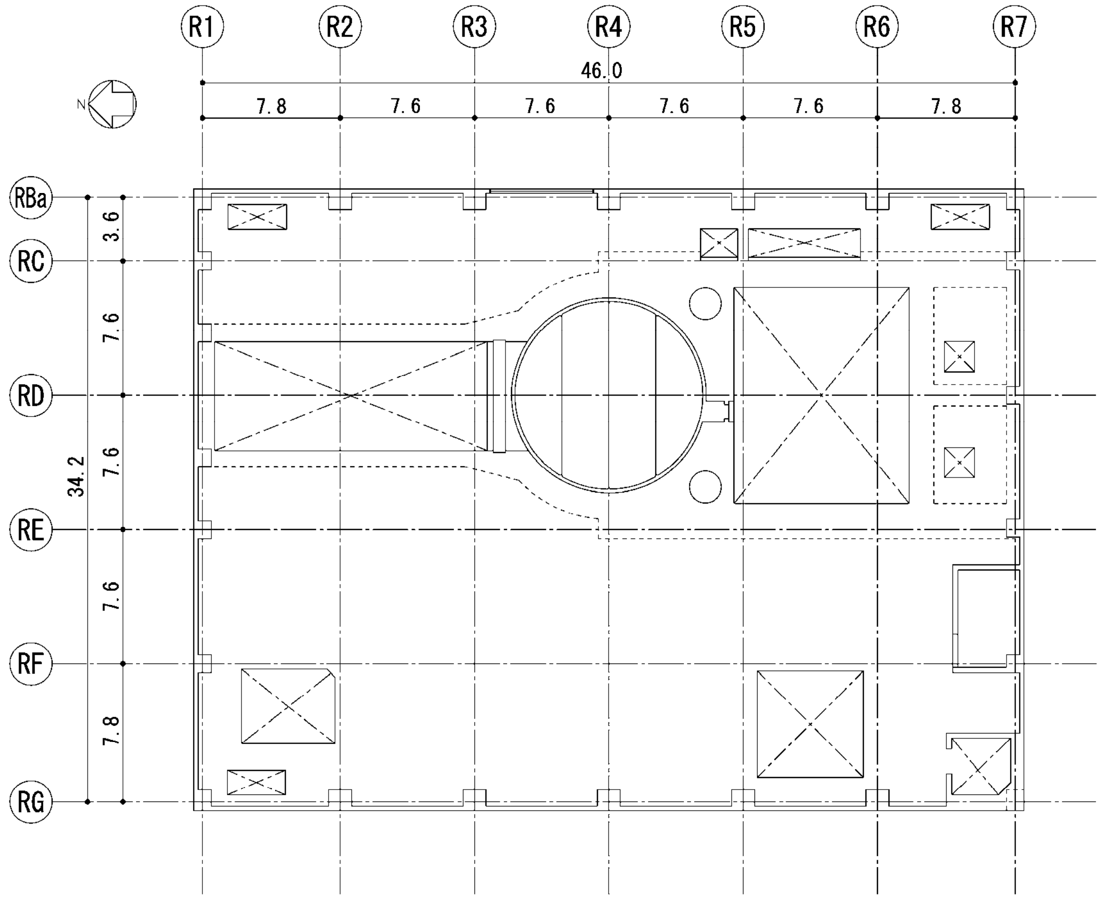


Figure 1.1: 5FL horizontal drawing

(unit: m)

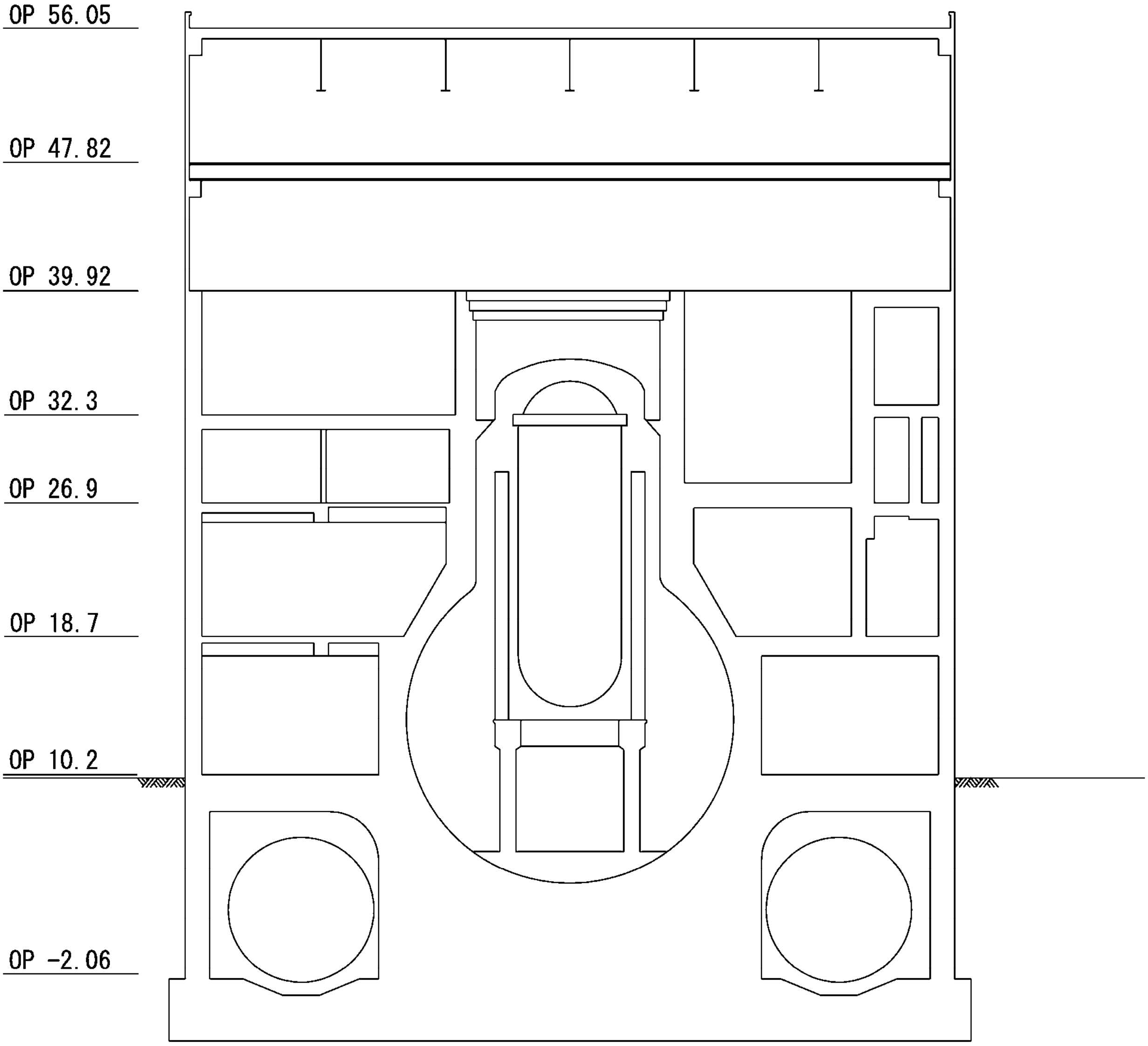


Figure 1.2: vertical drawing

(NS direction, unit: m)

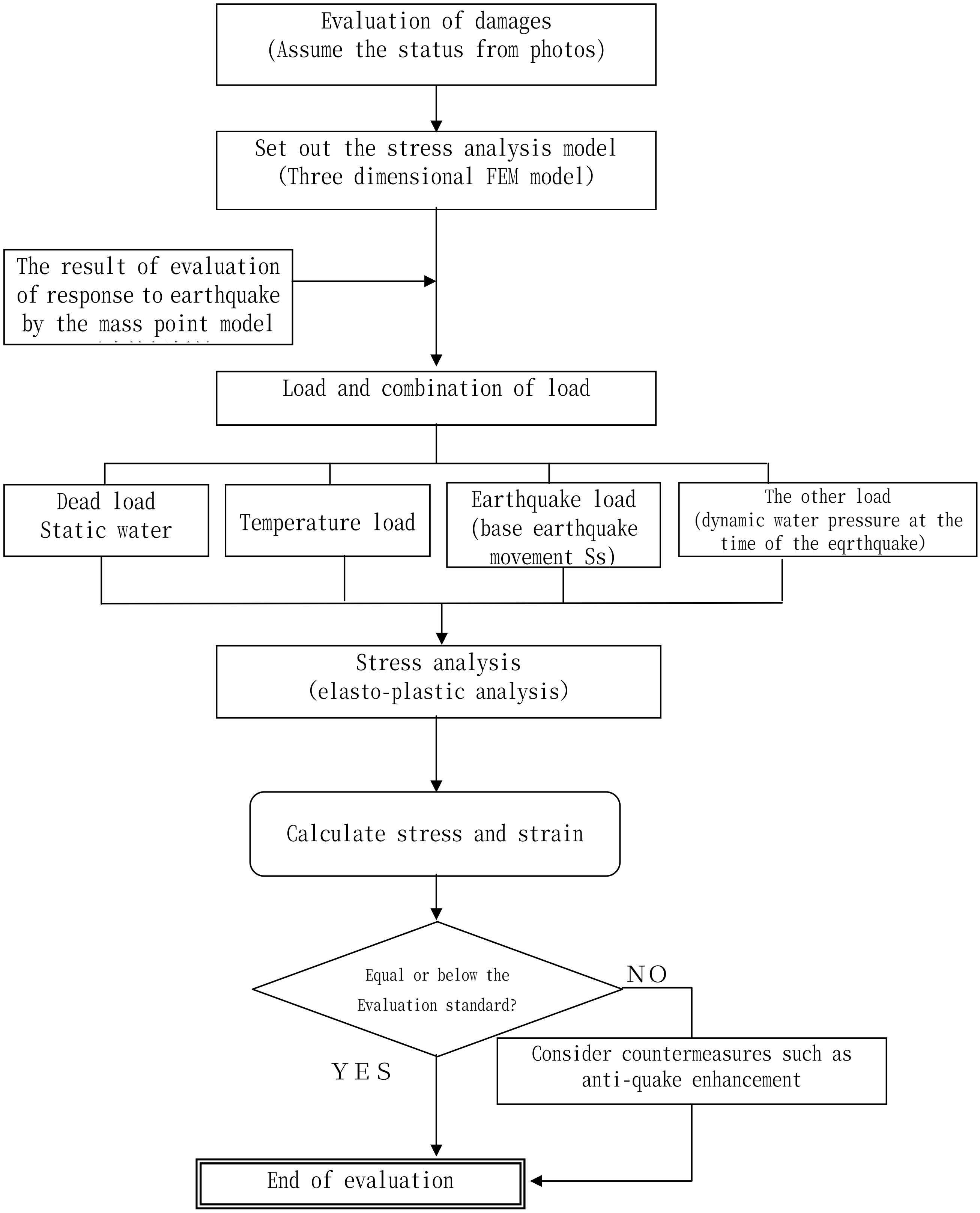


Figure 1.3: flowchart for anti-quake safety evaluation of the Spent Fuel Pool

2. Evaluation of the status of damages

In evaluating the status of damages, we constructed the three dimensional FEM model based on “Attachment-3, 2. Evaluation of the status of damages”.

The outer wall used in the analytical model is the same as considered in Attachment-3 with the following assumptions: (i) Pillars and beams remain in place (ii) There is no damage to the Spent Fuel Pool, the temporary equipment storage pool, the shell wall and surrounding floor.

The weight of damaged parts is assumed to be supported by the floor below and uniformly distributed.

3. The stress analysis model

We conduct the elasto-plastic analysis taking account of the plasticity of reinforced concrete and calculate stress and strain at the Spent Fuel Pool. We treat the reinforced concrete structure from the wall, 2FL to the fuel exchange floor, 5FL as the aggregation of finite element for modeling purpose.

The plate element used in the analytical model is the laminated shell element by anisotropic materials that models the reinforcing steel layer. On each element, we consider the axial force and the bend stress at the same time. As for bend of the plate, we also consider the impact of out-of-plane shear deformation. The program used was "ABAQUS".

Figure 3.1 is the outline of the analytical model. Figure 2 is the constitutive law of concrete and reinforcing steel. Figure 3.3 is the boundary condition of the analytical model.

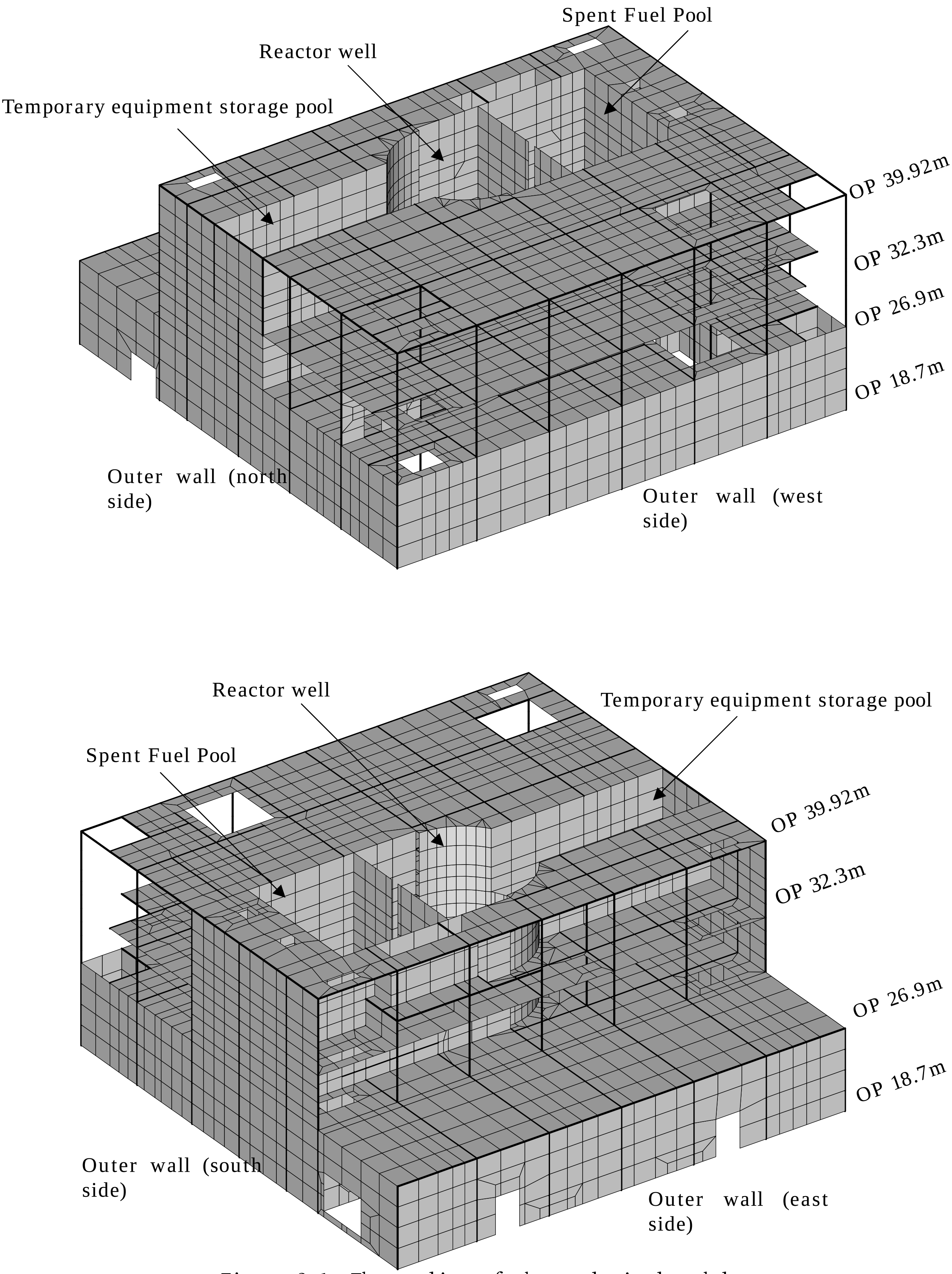
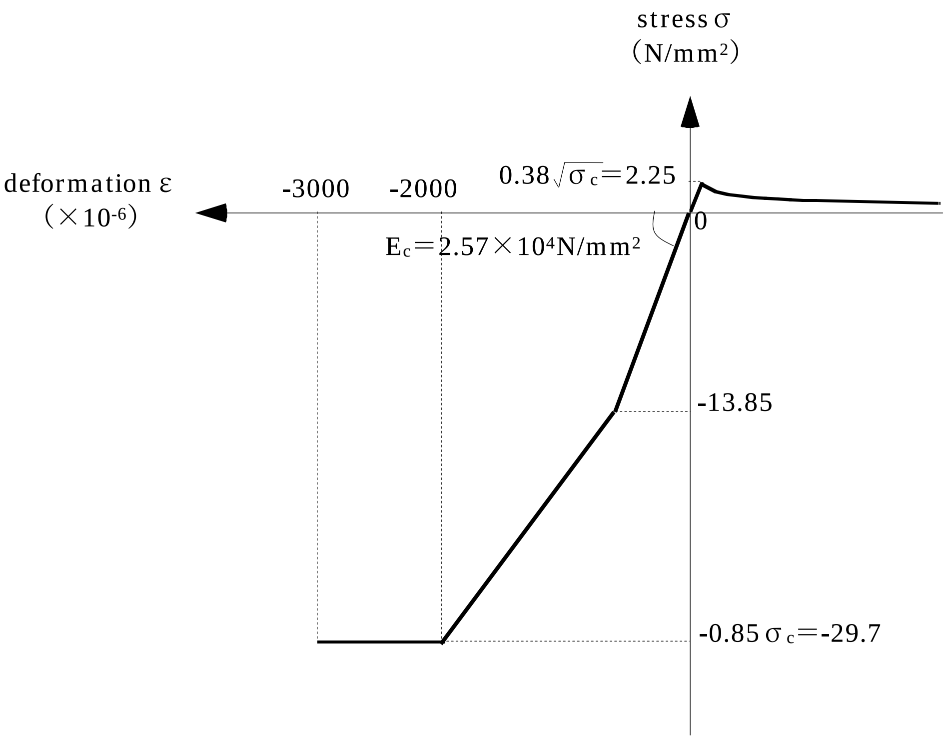
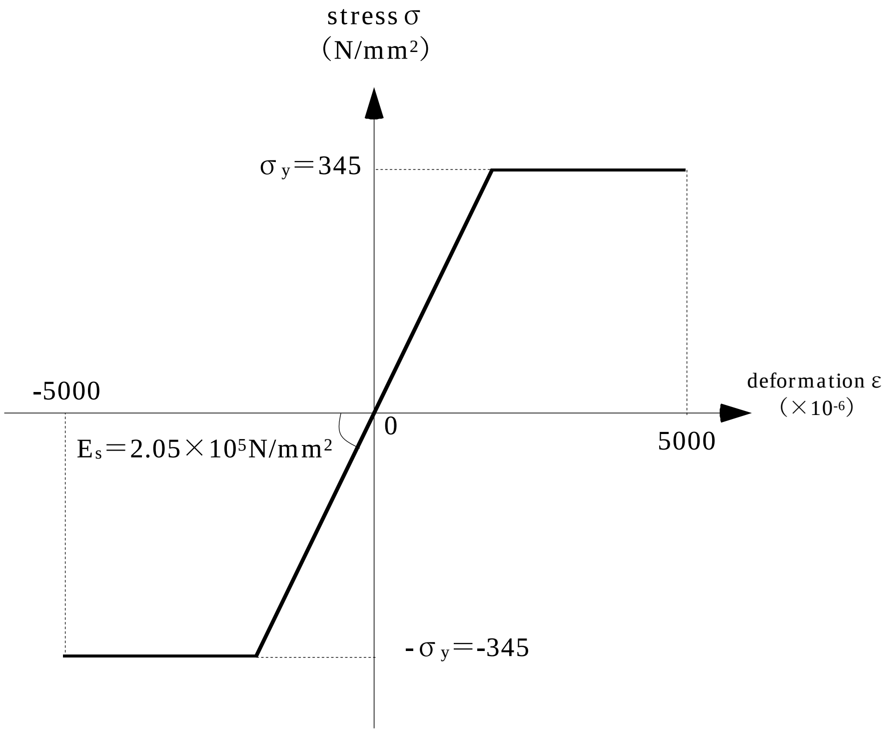


Figure 3.1: The outline of the analytical model



(a) stress-deformation graph of concrete

(strength of concrete σc=35N/mm^2)



(b) stress-deformation graph of reinforcing steel

(Yield point of reinforcing steel σy=345N/mm^2)

Figure 3.2: the constitutive law of concrete and reinforcing steel

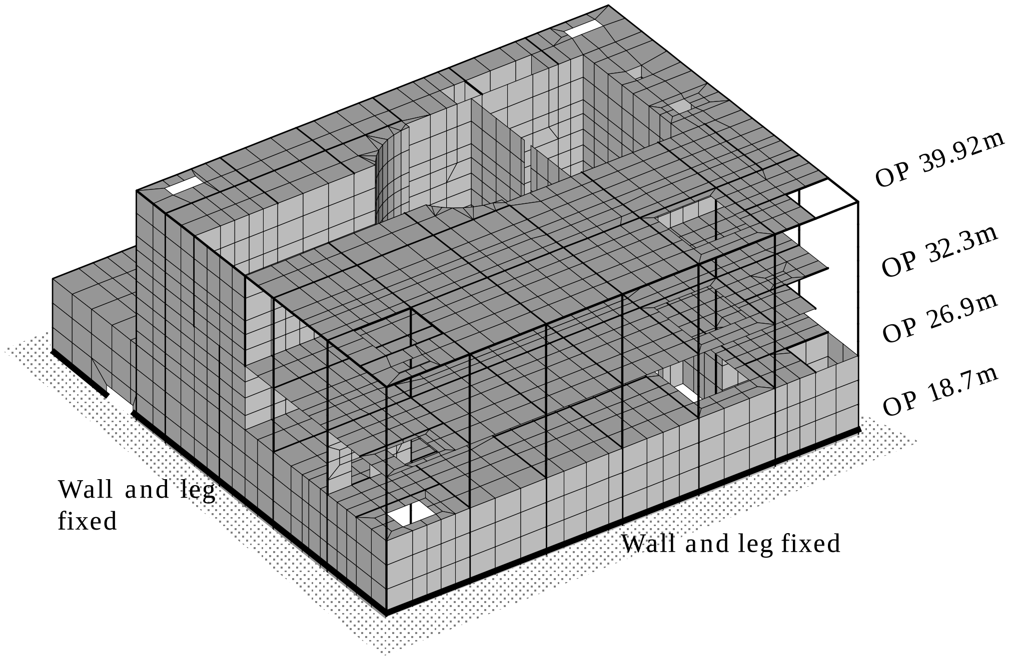


Figure 3.3: the boundary condition of the analytical model

4. Load and combination of loads

(1) Dead load

The deal load applied to the analytical model takes account of the modeled building's own weight, equipment weight and the additional weight on the assumption that the collapsed roof and the external wall's weight are added to the spent fuel exchange floor and the pool floor.

(2) Static water pressure

We consider the static water pressure on the assumption that Spent Fuel Pool, Reactor Well and temporary equipment placement pool are full.

(3) Temperature load

Taking the actual temperature of water in the pool (around 90°C) into consideration, we assume the water temperature of 90°C and the ambient temperature of 10°C.

(4) Earthquake load

Based on the analysis of the earthquake response against the design basis ground motion Ss by the mass point model that takes account of damages to the building, we set out the horizontal and vertical earthquake loads (appendix 4-1).

(5) The other loads

We take account of the dynamic water pressure of water in the pool at the time of the earthquake.

(6) Combination of loads

The combination of loads is set out in table 4.1. We evaluate the combination of the horizontal and vertical earthquake movement by combination factor method (combination factor 0.4).

According to the standard for reactor container vessel made of concrete, the standard for generating nuclear facilities by The Japan Society of Mechanical Engineers, it is not necessary to evaluate a combination of temperature load and earthquake load with design basis ground motion Ss. But, as the Spent Fuel Pool is at high temperature with relatively long time, we decided to evaluate the combination of temperature load and earthquake load with design basis ground motion Ss. Also, the evaluation result without temperature load is in appendix 4-2.

Table 4.1: Combination of loads

Name when the load is applied	Combination of loads
Ss at the time of the earthquake	DL + H + T + K + KH

DL: dead load, H：static water pressure, T：temperature,

K： earthquake load (design basis ground motion Ss), KH: dynamic water pressure

5. Evaluation result

We check the structure of the Spent Fuel Pool based on the placement of reinforcing steel etc. and evaluate the anti-quake safety. In the evaluation, we confirm that the stress and the strain analyzed from the stress analysis do not exceed the evaluation standard. We set out the evaluation standard in accordance with the standard for reactor container vessel made of concrete, the standard for generating nuclear facilities by The Japan Society of Mechanical Engineers etc. The placement of reinforcing steel is as figure 5.1.

The evaluation result is as tables 5.1 and 5.2. As the stress and the strain are within elasticity span and below the evaluation standard, we presume that the current Spent Fuel Pool keeps the anti-quake safety against the design basis ground motion Ss.

Codes used in tables 5.1 and 5.2

$_c\varepsilon_c$: compress strain of concrete

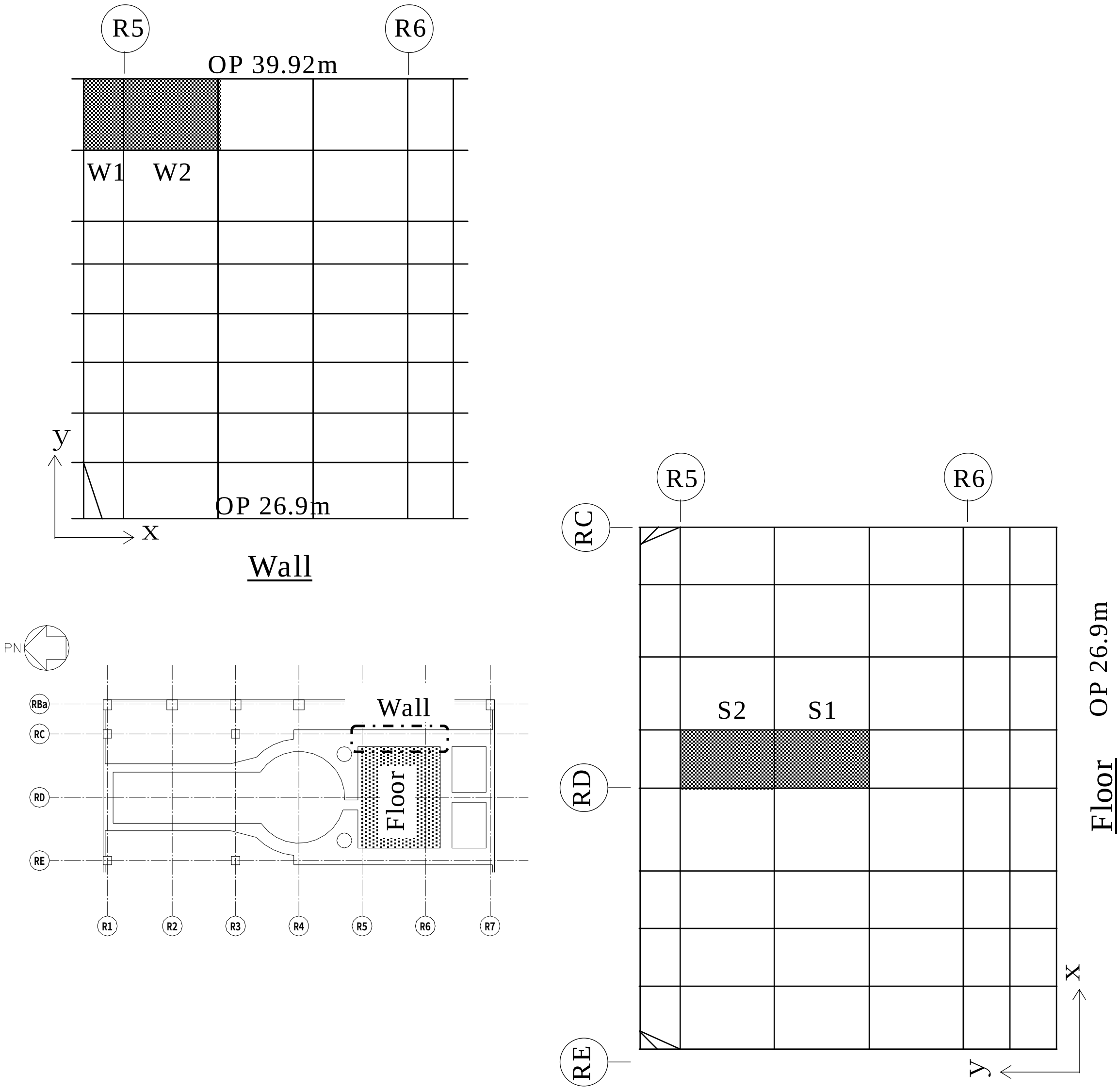
$_s\varepsilon_c$, $_s\varepsilon_t$: compress strain and tension strain of reinforcing steel (we allocate positive figures to tension)

Q : out-of-plane shear force

There are several variable factors in evaluating the damages and setting the load conditions as below. We considered the influence of these factors and confirmed that there is no material influence (appenedix 4-3).

- Impact to surrounding floor slab etc. by explosion
- Impact to the wall of the fuel pool and surrounding floor slab by fire
- Impact of rise of water temperature in the Spent Fuel Pool

We did reinforcement work to the bottom of the Spent Fuel Pool. We also considered the increase of margin by this work (appendix 4-4).



position	Inner reinforcing steel		Outer reinforcing steel		Shear reinforcing steel
	X direction	Y direction	X direction	Y direction	
W1	D32@250 +4-D32	D32@120	D32@250 +4-D32	D32@240	—
W2					

position	Upper end reinforcing steel		Lower end reinforcing steel		Shear reinforcing steel
	X direction	Y direction	X direction	Y direction	
S1	D32@100＋D32@200		D32@200		—
S2					

Figure 5.1: location of reinforcing steel at the part evaluated

Table 5.1(1) the evaluation result of strain of concrete and reinforcing steel by axial force and bend moment (wall)

position	Strain considered	Name of the load	Strain occurred ($\times 10^{-6}$)	Evaluation standard ($\times 10^{-6}$)	decision
W1	${}_{c}\varepsilon_{c}$	Ss at the time of earthquake	-480	-3000	Ok
	${}_{s}\varepsilon_{c}$		-350	-5000	Ok
	${}_{s}\varepsilon_{t}$		1230	5000	Ok

Table 5.1(2) the evaluation result of strain of concrete and reinforcing steel by axial force and bend moment (floor)

position	Strain considered	Name of the load	Strain occurred ($\times 10^{-6}$)	Evaluation standard ($\times 10^{-6}$)	decision
S1	${}_{c}\varepsilon_{c}$	Ss at the time of earthquake	-580	-3000	Ok
	${}_{s}\varepsilon_{c}$		-210	-5000	Ok
	${}_{s}\varepsilon_{t}$		490	5000	ok

Table 5.2(1) the evaluation result of out-of-plane shear force (wall)

position	Name of the load	Strain occurred Q (N/mm)	Evaluation standard (N/mm)	decision
W2	Ss at the time of earthquake	2040	3770	Ok

Table 5.2(2) the evaluation result of out-of-plane shear force (floor)

position	Name of the load	Strain occurred Q (N/mm)	Evaluation standard (N/mm)	decision
S2	Ss at the time of earthquake	800	1150	Ok

Regarding the earthquake response analysis for vertical direction of Reactor Building of Unit 4

With regards to the local evaluation of 3 dimensional FEM analysis of the reactor building of Unit 4 of Fukushima Daiichi Nuclear Power Station, result of dynamic analysis of vertical direction by basic earthquake ground motion “Ss” is used as an input. In this section, we shoe the result of earthquake response analysis for vertical direction.

When establishing evaluation model, we treat the damaged area same as the area used in the evaluation report described in “Appendix 3: Detail of seismic safety evaluation of Reactor building of Unit 4 (Evaluation by time history response analysis method using mass system model)”, and assume the weight of disrupted portion will be supported by the floor of downstairs.

Details of building analysis model of vertical direction in Figure-1 and specification in List -1 below.

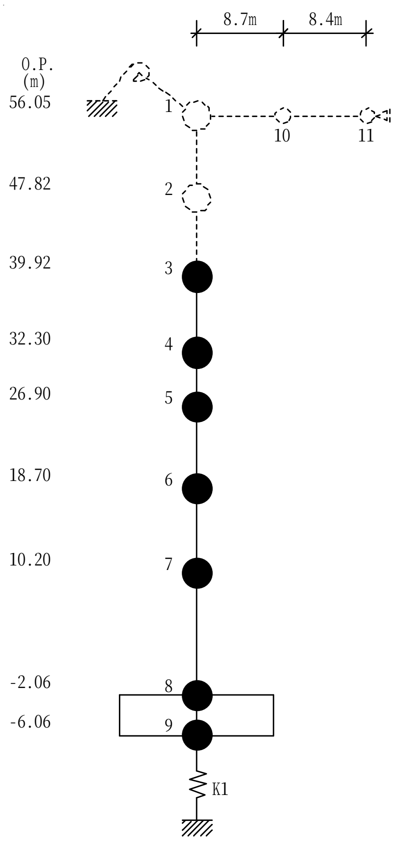


Figure-1　Building Analysis Model(vertical direction)

List-1 Specification of Building Analysis Model(Vertical Direction)

Building			
MP No.	Mass Point Weight W(kN)	Shaft Area A_N(m^2)	Axle Spring Rigidity K_A($\times 10^8$ kN/m)
1	-		
		-	-
2	-		
		-	-
3	114,850		
		222.6	7.41
4	88,770		
		218.1	10.58
5	117,030		
		380.4	11.92
6	121,930		
		340.6	10.30
7	207,300		
		654.7	13.72
8	287,050		
		2,812.6	180.71
9	132,390		
Total	1,069,320		

Ceiling			
MP No.	Mass Point Weight W(kN)	Shear Area A_S($\times 10^2$ m^2)	Shear 2nd Moment I(m^4)
1	-		
		-	-
10	-		
		-	-
11	-		

①Concrete Part

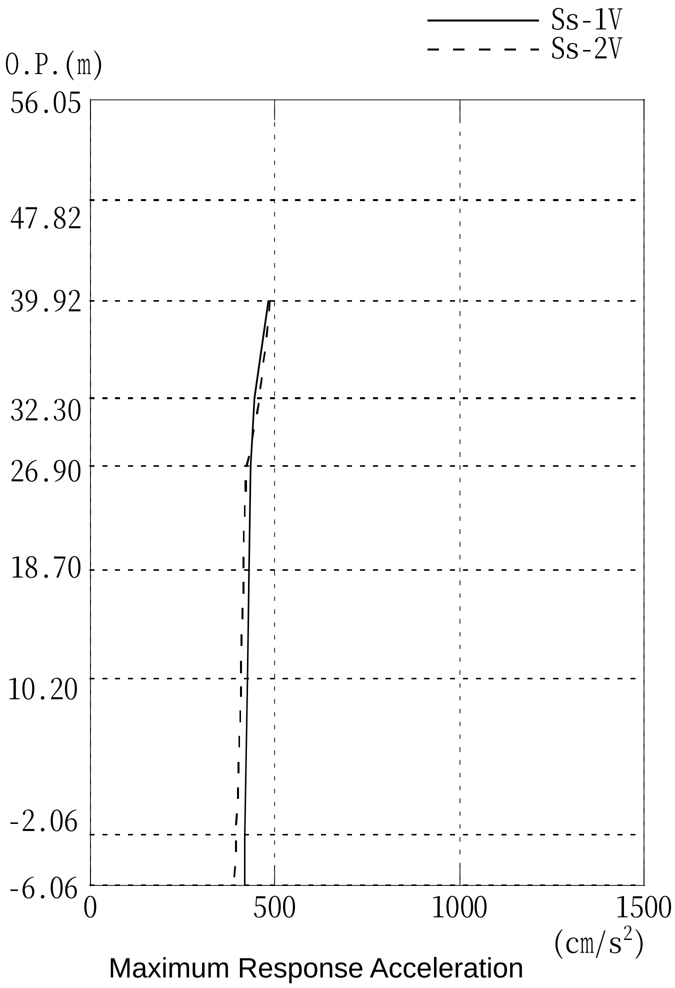
Young Modulus	E_C	2.57×10^7 (kN/m^2)
Shear Elastic Modulus	G	1.07×10^7 (kN/m^2)
Poisson Ratio		0.20
Decay	h	5%

②Iron Frame Part

Young Modulus	E_S	2.05×10^8 (kN/m^2)
Shear Elastic Modulus	G	7.90×10^7 (kN/m^2)
Poisson Ratio		0.30
Decay	h	2%

Base Configuration 49.0m(NS Direction)×57.4m(EW Direction)

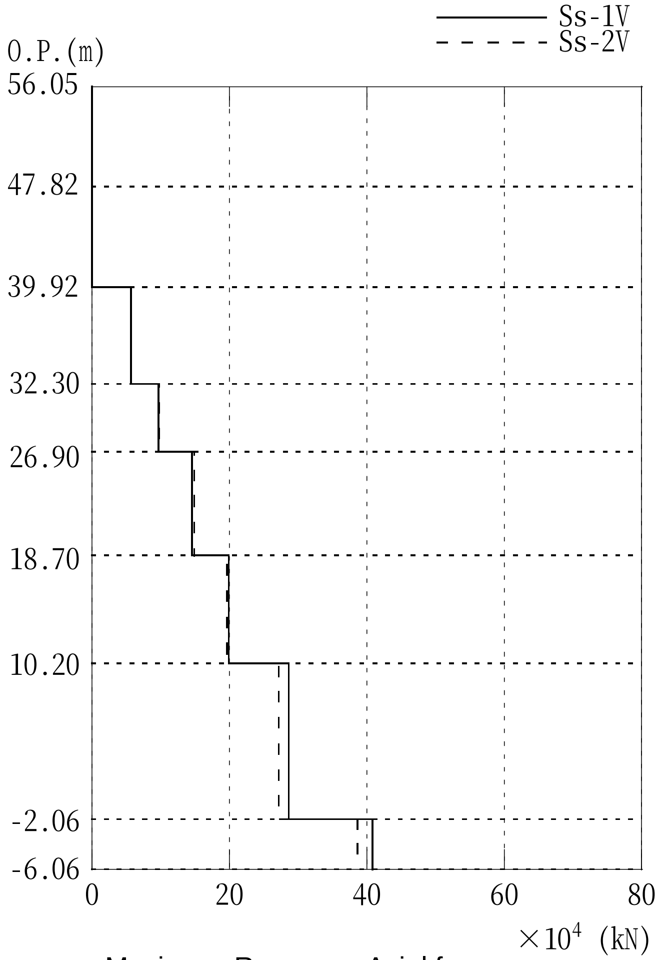
Result of Maximum Response Acceleration and Maximum Response Axial Force of vertical direction by earthquake response analysis are shown in Fugure-2 and Figure-3 below.



(cm/s²)

Ss-1V	Ss-2V
482	487
445	457
434	424
429	414
426	407
420	394
418	391

Figure-2　Maximum Response Acceleration(Vertical Direction)



Maximum Response Axial force

$\times 10^4$(kN)

Ss-1V	Ss-2V
5.67	5.70
9.71	9.84
14.58	14.90
19.85	19.67
28.61	27.19
40.81	38.60

Figure-3　Maximum Response Axial force(Vertical Direction）

Appendix 4-2

Parametric Study regarding Temperature Load

1. Study Overview

   In Attachment – 4 we evaluated seismic safety by assuming Design Basis Ground Motion Ss and Temperature Load (assumption on the water temperature of the pool, approx. 90 degree Celsius) as the combination of load. In this study, we examine the impact on Design Basis Ground Motion Ss and the evaluation of the seismic safety without taking Temperature Load into account.

2. Methodology

   Based on the combination of load in Attachment – 4 (Base Case), a combination of load without Temperature Load is set as seen in Table – 1. It should be noted here that other assumptions than the combination of load are the same as Base Case, including the analysis model.

Table – 1: Combination of Load

Case	Combination of Load
Under Ss Earthquake	DL+H+K+KH

where DL: Dead Load
H : Hydrostatic Pressure
K : Ss Earthquake Load
KH: Dynamic Water Pressure under Ss Earthquake

3. Evaluation

   Table 2 is the Base Case evaluation results of the same place (element) based on strains of the concrete and the rebar of the wall and the floor of Spent Fuel Pool, and Table 3 is the Base Case evaluation results of the same place (element) based on out-of-plane shear stress. For reference, the Base Case evaluation results that take the temperature of the pool into account are also included for comparison of Table – 2 and 3.

   Based on the evaluation results, it can be estimated that stress and strain of Spent Fuel Pool would be within the standard range and seismic safety would be secured even without considering Temperature Load.

Table 2(1) Evaluation results of the concrete and rebar strain generated by axial force and bending stress (wall)

Place	Strain Evaluated	Case	Strain generated ($\times 10^{-6}$)		Evaluation Standard ($\times 10^{-6}$)	Evaluation
			This Study (Without Temperature)	Reference Base Case		
W1	$_c\varepsilon_c$	Under Ss Earthquake	-110	-480	-3000	OK
	$_s\varepsilon_c$		-110	-350	-5000	OK
	$_s\varepsilon_t$		420	1230	5000	OK

Table 2(2) Evaluation results of the concrete and rebar strain generated by axial force and bending stress (floor)

Place	Strain Evaluated	Case	Strain generated($\times 10^{-6}$)		Evaluation Standard ($\times 10^{-6}$)	Evaluation
			This Study (Without Temperature)	Reference Base Case		
S1	$_c\varepsilon_c$	Under Ss Earthquake	-130	-580	-3000	OK
	$_s\varepsilon_c$		-40	-210	-5000	OK
	$_s\varepsilon_t$		140	490	5000	OK

Table 3(1) Evaluation results of out-of-plane shear stress (wall)

Place	Case	Stress Generated Q (N/mm)		Evaluation
		This Study (Without Temperature)	Reference Base Case	
W2	Under Ss Earthquake	1020 (3430)	2040 (3770)	OK

Evaluation Standard is indicated in () .

Table 3(2) Evaluation results of out-of-plane shear stress (floor)

Place	Case	Stress Generated Q (N/mm)		Evaluation
		This Study (Without Temperature)	Reference Base Case	
S2	Under Ss Earthquake	870 (2180)	800 (1150)	OK

Evaluation Standard is indicated in () .

Parametric Study regarding Seismic Safety Assessment of Spent Fuel Pool

1. Review Method

We conducted parameter analysis of damaged cases which was not taken into consideration under the base case scenario, which are detailed below (which specified in 3 cases below), and review the impact to seismic safety assessment of Spent Fuel Pool.

【Damaged case which was not taken into consideration under the base case scenario】

① Impact of Explosion

Due to the explosion, most of the ceiling and out wall above 3rd floor was disrupted and rigidity of disrupted-wall and floor of the pool. Which were constructed by thick wall will be reduced.

② Impact of Fire

Due to the fire, west side wall and surroundings are disrupted and its rigidity will be reduced.

③ Impact of high temperature of water in the pool

Due to the exothermal of the spent fuel, the temperature of the water in the pool became higher. And after the wall and inside wall will be exposed to the heat for long period of time and the concrete will suffer damage, the rigidity will be reduced.

2． Condition

2． 1 Condition of assessment of the impact due to the explosion

Due to the explosion, most of the ceiling and out wall above 3rd floor was disrupted and rigidity of disrupted-wall and floor of the pool. Which were constructed by thick wall will be reduced. Therefore, as shown in Figure 1, we will examine the impact of damage of floor of 4th and 5th floor and partial disruption of out side wall of 3rd floor and 4th floor to the seismic safety assessment of spent fuel pool.

① General rigidity (4th and 5th floor)

To reduce rigidity of floor of 4th and 5th floor to 50%

② Rigidity of out side wall（3rd and 4th floor）

Model partial disruption of side wall (in the base case scenario, it is totally disrupted), reduce its rigidity to 50%.

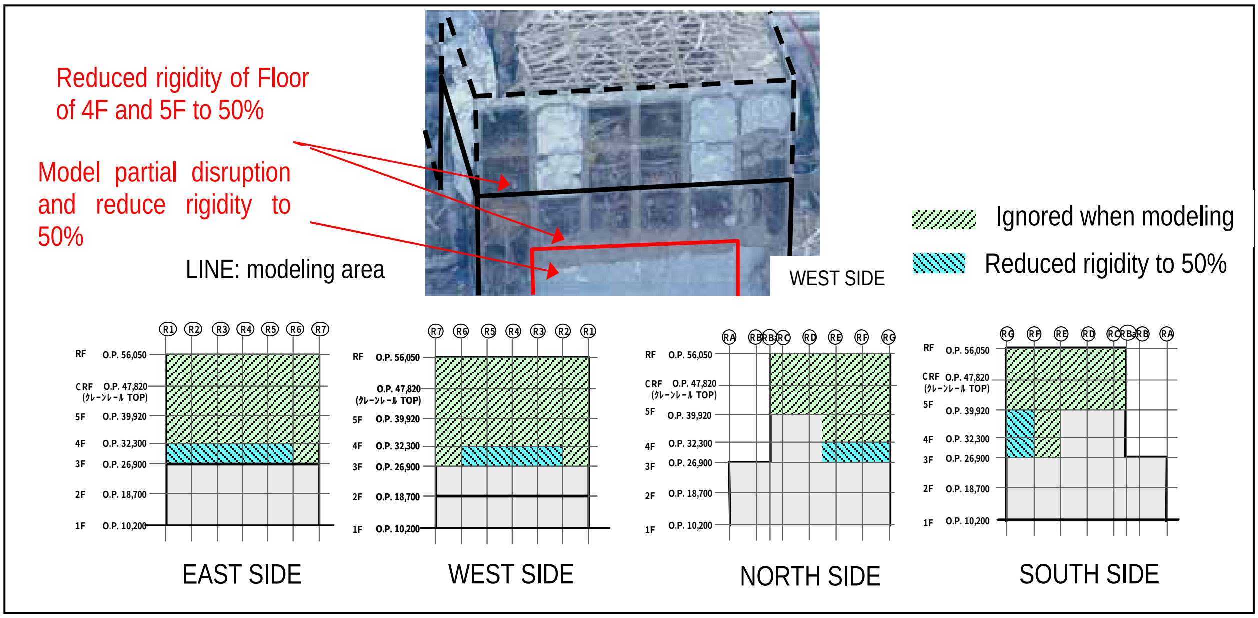


Figure1. Floor and out wall to evaluate in the assessment of impact of explosion

2．2　Condition of assessment of the impact due to fire

Due to the fire, west side wall of fuel pool and its surroundings are damaged and its rigidity will be reduced. Therefore, we set MG set room in the west side of 4th floor as origin of the fire and impact area of fire will be determined as shown in Figure 2 and rigidity of west area of 4th floor and 5th floor and all the pool wall will be reduced. And we assume the wall and concrete surface was damaged by fire and rigidity will be reduced to 80% and we will examine impact of seismic safety assessment of spent fuel pool.

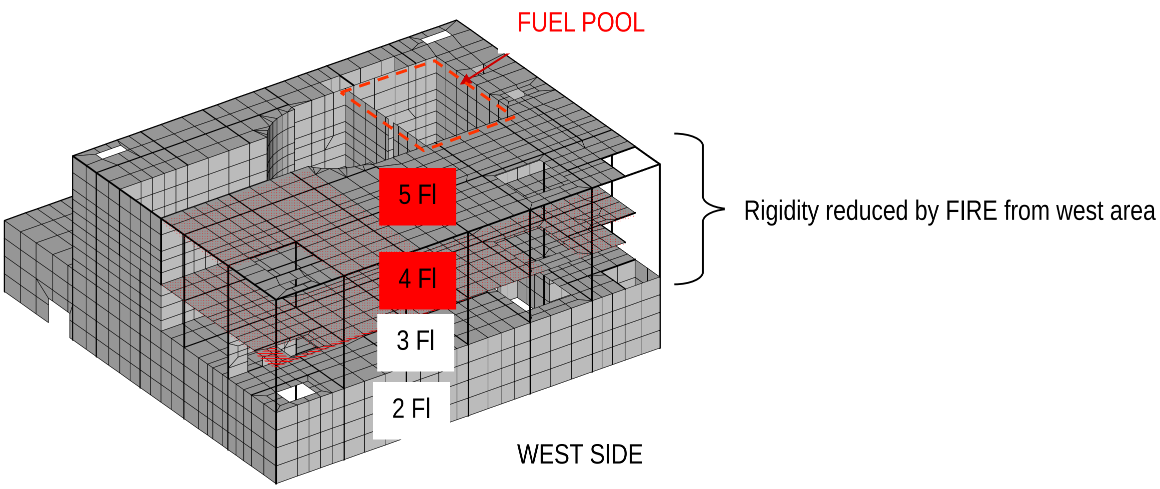


Figure 2. Floor and out wall to evaluate in the assessment of impact of the fire

２．３ Condition of the assessment of high temperature of water in the pool

Due to the exothermal of the spent fuel, the temperature of the water in the pool became higher. And after the wall and inside wall will be exposed to the heat for long period of time and the concrete will suffer damage, the rigidity will be reduced. Therefore we will assume water temperature in the pool will be increase to 100℃ and outside air temperature will be 0℃ in winter, and examine impact of seismic sagety assessment of spent fuel pool.

2．4　Case Study

We will show all the 3 cases and its condition from 2.1 to 2.4 together with that of base case scenario in the below List 1. We will employ 16 cases., same as the base case scenario, to examine impact of the seismic safety assessment.

List-1　cases

Case		items			
		Reduction of rigidity of out wall (3rd to 4th Floor)	Reduction of rigidity of the floor (4th to 5th Floor)	Reduction of rigidity of pool wall	Poor water temperature
-	Base case	Ignore both total and partial disruption	Ignore	Ignore	10～90℃
1	Explosion	Reduction of rigidity to 50% for partial disrupted wall	Reduction of rigidity to 50%	*	*
2	Fire	*	Reduction or rigidity to 80% for western part	Reduction or rigidity to 80% for western part	*
3	Pool water temperature	*	*	*	0～100℃

Note）*：same as base case scenario.

3. Result

In base case scenario and examined cases, we show the result of the comparison of percentage of integrated strain and stress against evaluation standard in List-2 below. Therefore, it was confirmed that even if we employ the impact of explosion, fire and high temperature of pool awter which are not taken into consideration under the base case scenario, such events will not have any impact to the seismic safety assessment of spent fuel pool.

Further, for the reference, we will show details of result of seismic safety assessment of spent fuel pool for examined cases from 1 to 3 in below List-3 to List-8.

List-2 Comparison of percentage of integrated strain and stress against evaluation standard

	Item	Base case	[case 1] Explosion	[case 2] Fire	[case 3] High temperature of pool water
Pool Bottom Floor	Reinforcing steel strain	0.10	0.10	0.10	0.14
	Concrete strain	0.20	0.20	0.20	0.24
	Out-of-plane shear force	0.70	0.69	0.70	0.76
Pool Wall	Reinforcing steel strain	0.25	0.25	0.24	0.30
	Concrete strain	0.16	0.16	0.17	0.19
	Out-of-plane shear force	0.55	0.55	0.52	0.61

Note) If the value of the figure is below 1 it shows it is below standard limit.

【Case 1 Impact of Explosion】

List-3(1) Result of strain of concrete and reinforcing steel by axial forces and bending moment (wall)

place	examined Strain	Name of stress	Integrated strain ($\times 10^{-6}$)	Standard limit ($\times 10^{-6}$)	result
W1	${}_c\varepsilon_c$	Ss earthquake	-470	-3000	Below limit
	${}_s\varepsilon_c$		-340	-5000	Below limit
	${}_s\varepsilon_t$		1240	5000	below limit

List-3(2) Result of strain of concrete and reinforcing steel by axial forces and bending moment (floor)

Place	Examined Strain	Name of stress	Integrated strain ($\times 10^{-6}$)	Standard limit ($\times 10^{-6}$)	result
S1	${}_c\varepsilon_c$	Ss earthquake	-580	-3000	below limit
	${}_s\varepsilon_c$		-210	-5000	below limit
	${}_s\varepsilon_t$		480	5000	below limit

List-4(1) Out of place share force (wall)

place	Name of stress	Integrated stress Q (N/mm)	Standard limit (N/mm)	result
W2	Ss Earthquake	2050	3770	Below limit

List-4(2) Out of place share force (floor)

Place	Name of stress	Integrated stress Q (N/mm)	Standard limit (N/mm)	Result
S2	Ss earthquake	790	1150	Below limit

Note）Please refer base case scenario for “place”

【Case 2 Impact of Fire】

List-5(1) Result of strain of concrete and reinforcing steel by axial forces and bending moment (wall)

place	examined Strain	Name of stress	Integrated strain ($\times 10^{-6}$)	Standard limit ($\times 10^{-6}$)	result
W1	$_c\varepsilon_c$	Ss earthquake	-510	-3000	可
	$_s\varepsilon_c$		-380	-5000	可
	$_s\varepsilon_t$		1170	5000	可

List-5(2) Result of strain of concrete and reinforcing steel by axial forces and bending moment (floor)

place	examined Strain	Name of stress	Integrated strain ($\times 10^{-6}$)	Standard limit ($\times 10^{-6}$)	result
S1	$_c\varepsilon_c$	Ss earthquake	-580	-3000	Below limit
	$_s\varepsilon_c$		-210	-5000	Below limit
	$_s\varepsilon_t$		480	5000	Below limit

List-6(1) Out of place share force (wall)

place	Name of stress	Integrated stress Q (N/mm)	Standard limit (N/mm)	result
W2	Ss earthquake	1940	3770	Below limit

List-6(2) Out of place share force (floor)

place	Name of stress	Integrated stress Q (N/mm)	Standard limit (N/mm)	result
S2	Ss earthquake	760	1090	Below limit

Note）Please refer base case scenario for “place”

【Case 3 Impact of high temperature of pool water】

List-7(1) Result of strain of concrete and reinforcing steel by axial forces and bending moment (wall)

place	examined Strain	Name of stress	Integrated strain ($\times 10^{-6}$)	Standard limit ($\times 10^{-6}$)	result
W1	$_c\varepsilon_c$	Ss earthquake	-570	-3000	Below limit
	$_s\varepsilon_c$		-460	-5000	Below limit
	$_s\varepsilon_t$		1480	5000	Below limit

List-7(2) Result of strain of concrete and reinforcing steel by axial forces and bending moment (floor)

place	examined Strain	Name of stress	Integrated strain ($\times 10^{-6}$)	Standard limit ($\times 10^{-6}$)	result
S1	$_c\varepsilon_c$	Ss earthquake	-700	-3000	Below limit
	$_s\varepsilon_c$		-230	-5000	Below limit
	$_s\varepsilon_t$		660	5000	Below limit

List-8(1) Out of place share force (wall)

place	Name of stress	Integrated stress Q (N/mm)	Standard limit (N/mm)	result
W2	Ss earthquake	2280	3770	Below limit

List-8(2) Out of place share force (floor)

place	Name of stress	Integrated stress Q (N/mm)	Standard limit (N/mm)	result
S2	Ss earthquake	860	1140	Below limit

Note）Please refer base case scenario for “place”

Regarding the Effect of Strengthening Work

1. Evaluation Policy of Analysis

In order to increase the margin advancement of the basement of spent fuel pool, steel support pillar etc will be installed at the basement of spent fuel pool. In this section, we conduct similar earthquake resistant safety analysis with the stress analysis model, which added the components of mock steel support pillars as same as shown in chart 1. By comparing the analysis results before and after installing steel support pillar, we will evaluate the margin advancement effect.

Steel support pillars support the load from top by arranging 32 of steel support pillars in east-west direction. Moreover, in order to ensure its function, we will install concrete wall and fill grout between the concrete wall and the basement of spent fuel pool.

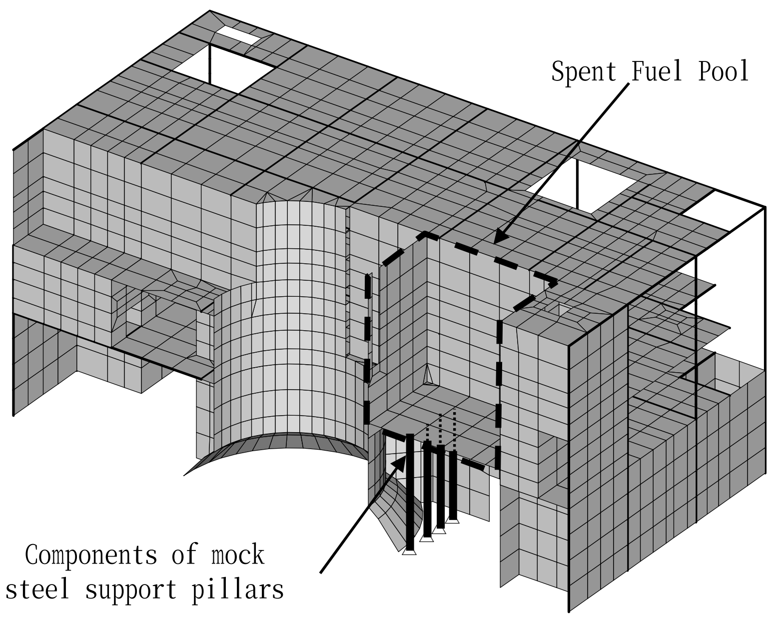


Chart 1 Stress Analysis Model

2. Margin Advancement Effect

Regarding the basement of spent fuel pool, by abstracting the spots, which has the maximum value ratio of occurred shearing force and strain against analysis standard value, the comparison results before and after the installment of components of mock steel support pillars are shown in Chart 1 and 2. Since each maximum values are decreased after the installment of steel support pillars, we confirmed that we can expect the margin advancement effect by installing steel support pillars.

Chart 1 Maximum value ratio of occurred strain against the analysis standard (Comparison before and the installment of the components of mock steel support pillars)

Spot	Examination Strain	Time of load, name	$\frac{\text{Occurred Strain}}{\text{Analysis Standard Value}}$	
			Before Installation of Steel Support Pillar (maximum value)	After Installation of Steel Support Pillar (maximum value)
S1 basement of spent fuel pool	Concrete $_c\varepsilon_c$	Coseismic Ss	0.20	0.10
	Rebar $_s\varepsilon_t$		0.10	0.07

Chart 2 Maximum value ratio of out-of-plane shearing force against the analysis standard value (Comparison before and after the installment of the components of mock steel support pillars)

Spot	Examination stress	Time of load, name	$\frac{\text{Occurred Shearing Force}}{\text{Analysis Standard Value}}$	
			Before Installation of Steel Support Pillar (maximum value)	After Installation of Steel Support Pillar (maximum value)

S2 basement of spent fuel pool	Out-of-Plane Shearing Force Q	Coseismic Ss	0.70	0.56