THE REPORT ON THE INVESTIGATION INTO THE CURRENT SEISMIC SAFETY AND REINFORCEMENT OF THE REACTORS AT FUKUSHIMA DAIICHI NUCLEAR POWER STATION (NO. 2)

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The Tokyo Electric Power Company, Incorporated

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.

The report (No.1) submitted on May 28 covers Units 1 and 4, whereas this report (No.2) covers Unit 3 which is severely damaged.

2. Investigation methodology for the seismic safety assessment

(1) Unit 3 Reactor Building

The upper part of Unit 3 Reactor Building above the refueling floor on the 5th floor exploded due to an apparent hydrogen explosion on March 14, 2011. Based on the video picture when the explosion occurred, it is expected to be a massive explosion. Collapsed steel frames and concretes are piled up in most of the building on and above the 5th floor. The north-west part of the floor on the 5th floor is also damaged, so part of collapsed steel frames and concretes are accumulated on the 4th floor. Walls on the 4th floor are largely damaged. The information above 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 walls would reach the ultimate 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. This evaluation method is basically the same as the one applied to Unit 4.

3. Investigation results from the seismic safety assessment

(1) Unit 3 Reactor Building

As a result of the Time Transient Response Analysis utilizing the Design Basis Ground Motion (Ss), the shear strain generated in the seismic wall that remained on and below the 5th floor was 0.14 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)

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 1303 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 1689 (N/mm), which was enough margin compared to the evaluation standard value, 3130 (N/mm).
- The same evaluation method was conducted for the shell wall at outside of the Primary Containment Vessel (PCV). The maximum strain in the reinforced bar was 469 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 2475 (N/mm), which was enough margin compared to the evaluation standard value, 3270 (N/mm).
- According to the parameter studies, in which possibilities such as the rigidity degradation of the shell wall due to temperature rise in the Primary Containment Vessel (PCV), the further rigidity degradation of the spent fuel pool due to explosion, and the less rigidity degradation due to many uncertainties, were considered, the analysis result showed that there was no significant difference although there exist some numeric variation to some extent. Hence it was confirmed that variations in assumption did not very much affect the analysis results.

(Attachment–2)

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

(1) Unit 3 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 3 where seismic safety has not been secured. In addition, there is the other aspect of the difficulty in entering the building due to 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, concerning the steel framework structure and the concrete member collapsed and remaining, they are to be removed as soon as possible, depending upon the situation of working environment improvement hereafter.

5. Summary

In this report, it has been confirmed that there is no possible unsecured points in the view of seismic safety in the Reactor Building of Unit 3 according to the seismic safety assessment. With the Reactor Buildings of Unit 1 and Unit 4, which were previously reported, it has been confirmed that there is no possible unsecured points in the view of seismic safety in the Reactor Buildings with severe damages on and above the 5th floor.

Attachment 1

Detail of seismic safety evaluation of Reactor building of Unit 3

(Evaluation with time history response analysis as a 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 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 3 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.

As a result of the above analysis, if the seismic safety margin is comparatively low, more detail analysis is to be conducted.

The evaluation process of seismic response analysis for the reactor building of Unit 3 is described in Figure-1.1.

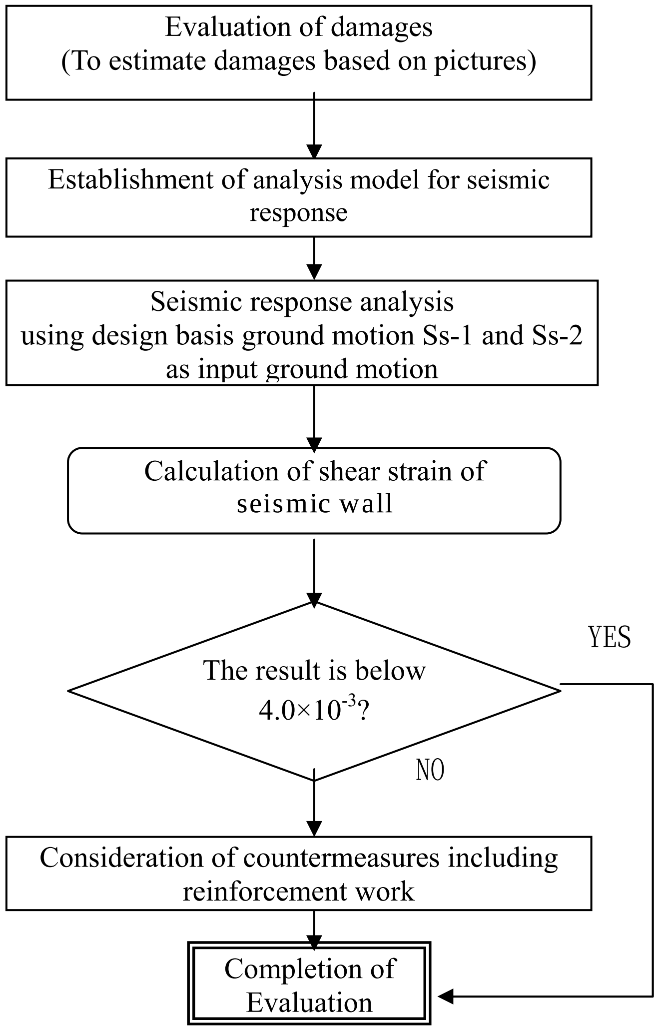


Figure-1.1 Evaluation process of seismic response analysis for the reactor building of Unit

2. Evaluation of Damage Situation

The reactor building has been partially damaged due to a hydrogen explosion etc. Damage situation of the reactor building was estimated based on pictures and the estimation was reflected in a seismic response analysis model.

The way how to evaluate each part of damage situation is shown as follows.

a. Exterior Wall/ Roof Truss

The exterior walls and roof trusses, which were confirmed the damages based on their exterior pictures, have been evaluated as damaged parts. The exterior walls, which have been partially peeled off, have been also evaluated as damaged (Figure-2.1).

b. Spent Fuel Pool

The spent fuel pool have been evaluated as no damage since the thickness of the wall and floor is 1400 to 1850 mm, while that of the damaged exterior wall is 600 mm at most, and the water level has been maintained as full after completion of circulating water cooling system.

c. Dryer Separator Pit

The wall of the dryer separator pit has not been confirmed as damaged, except for the exterior wall partially peeled off confirmed based on pictures. The west side wall of the dryer separator pit has been confirmed as no damage as far as the picture indicated the situation (Figure-2.2). The dryer separator pit has been evaluated as no damage since the thickness of the wall and floor of is 900 mm, while that of the damaged exterior wall is 600 mm at most.

d. Shell Wall

The Shell wall on the 3rd floor has been evaluated as no damage since the thickness of the shell wall is 1850 mm, while that of the damaged exterior wall is 600 mm at most.

e. Floor Slab

As the survey result of inside the building has not been obtained yet, it has been judged from the outside pictures and the situation of exterior walls. The floor slabs from 1st to 3rd floor have been evaluated as no damage since the exterior walls showed no abnormality, except for the exterior walls being partially peeled off. The floor slabs on 4th and 5th floor whose thickness were less than that of damaged exterior walls have been evaluated as partially damaged. The Northwest floor slab on the 5th floor has been evaluated as damaged since there have been observed big damages on the exterior walls and pillars on 4th floor, which support the floor slab, from the outside picture (Figure-2.3).

North Side

West Side

East Side

South Side

Figure-2.1 Situation of Exterior Walls

West Side Wall of Dryer Separator Pit

Figure-2.2 Situation of West Side Wall of Dryer Separation Pit

Figure-2.3 Situation of Northwest Floor Slab on 5th Floor

3. Input Ground Motion Used for Analysis

As input earthquake motion for the reactor building of Unit 3, 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.

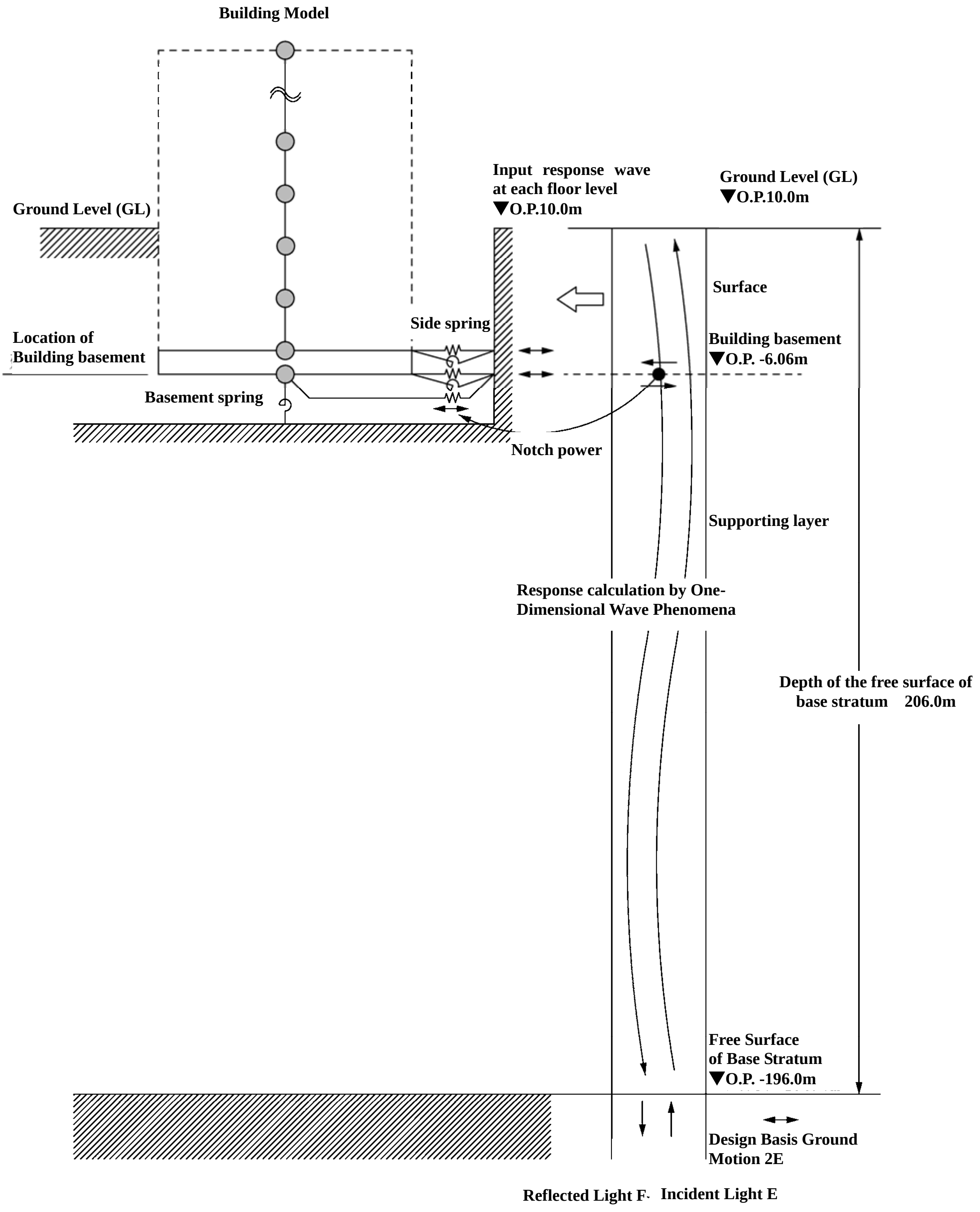
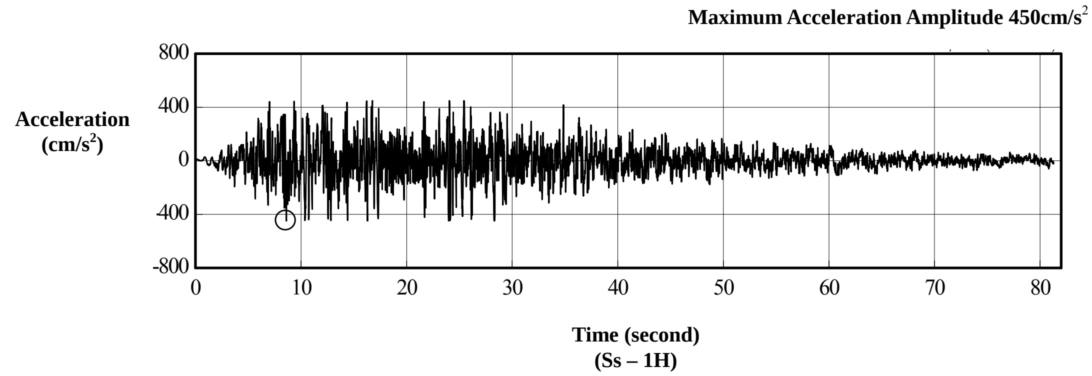


Figure-3.1 A conceptual diagram of input ground motion used in earthquake response analysis



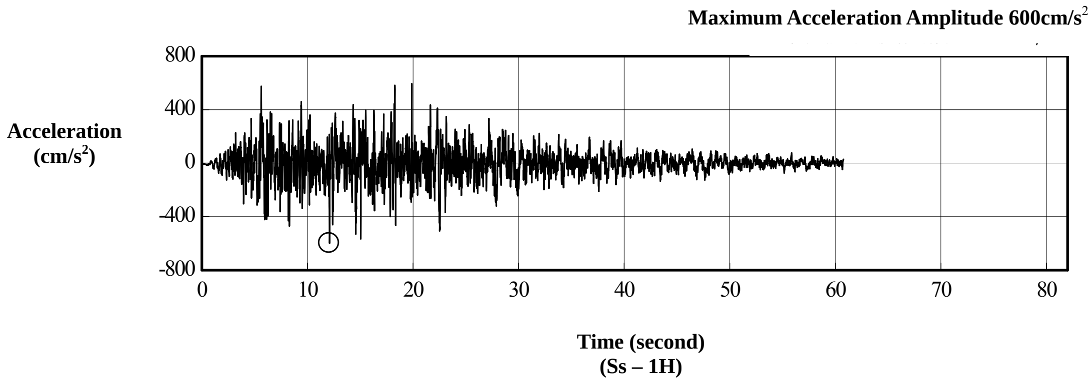


Figure-3.2 Chronicle acceleration wave profile (horizontal direction) of ground motion at free surface of base stratum

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 (second 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 April 19, 2010).

Regarding the reactor building of Unit 3, a part of the building was damaged by the hydrogen explosion, etc. An analysis model is formulated based on the damage conditions evaluated in “2. Evaluation of Damage Situation” The analysis assumes that a lower floor supports a weight of collapsed upper floor. For example, the weight of collapsed parts above the fifth floor is supported by the fifth floor (the northwest part, which the floor slab was damaged, is supported by the fourth floor). Figure 4-1 shows the damage conditions of the reactor building of Unit 3 (elevation) and Figure 4-2 shows the damage conditions (plane).

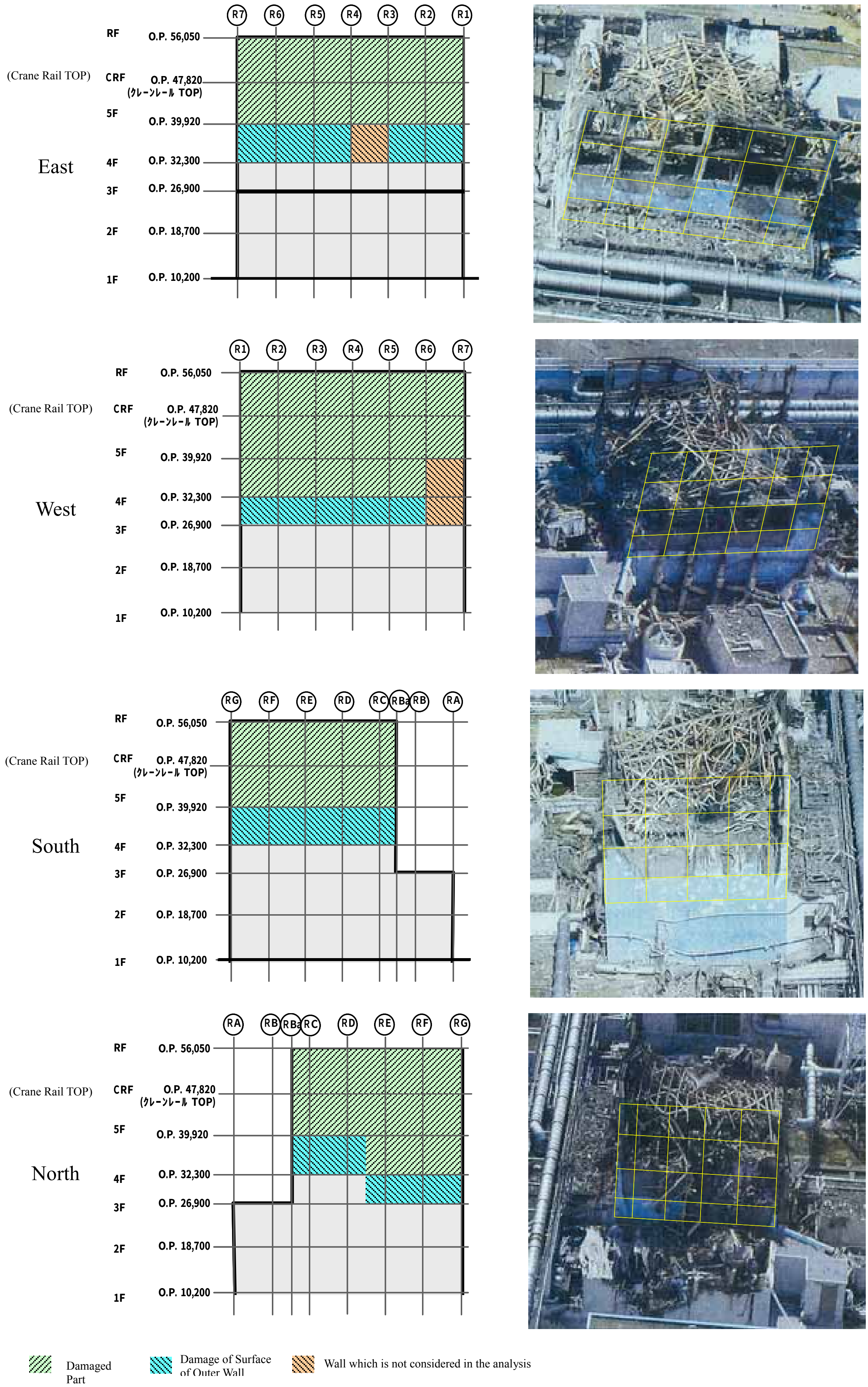


Figure 4-1 Damage Conditions of the Reactor Building of Unit 3 (Elevation)

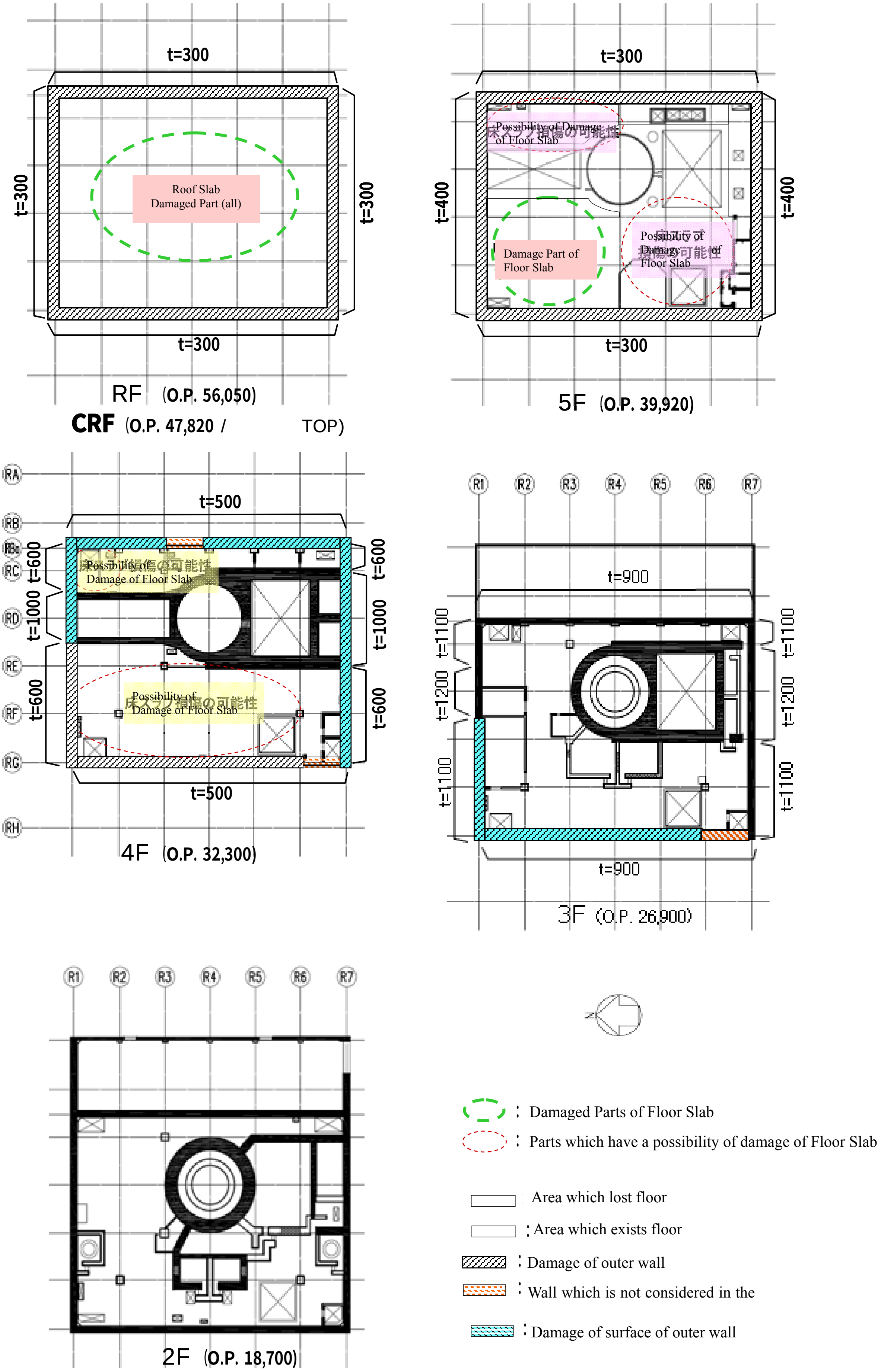


Figure 4-2 Damage Conditions of the Reactor Building of Unit 3 (Plane)

(1) Analysis Model for Seismic Response in Horizontal Direction

Analysis model for seismic response in the 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 as an equal spring, as shown in Figure-4.3 and Figure-4.4. The effects of connection between the building and ground are evaluated by a spring effect of the ground and input seismic response. Physical factors of concrete for the analysis are 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 are 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 damped factor (Cc).

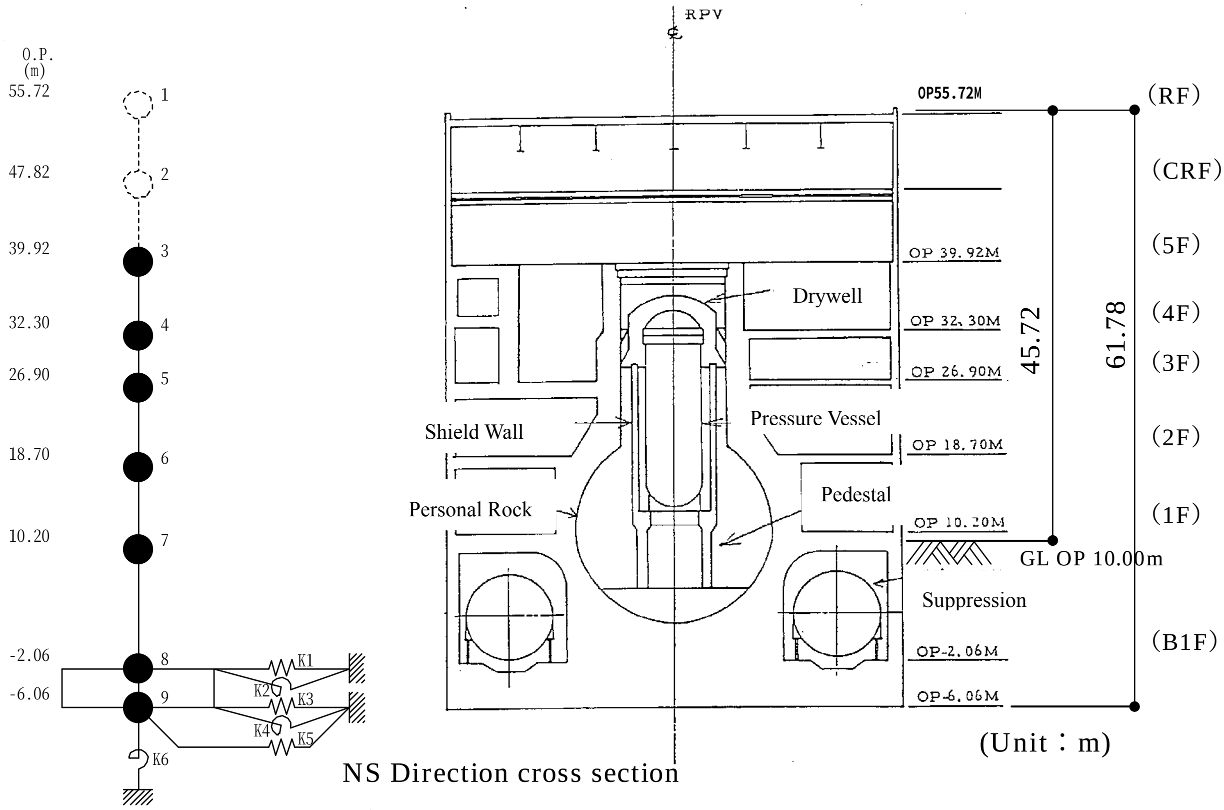


Figure 4-3 Analysis Model for Seismic Response of the Reactor Building of Unit 3 (N-S Direction)

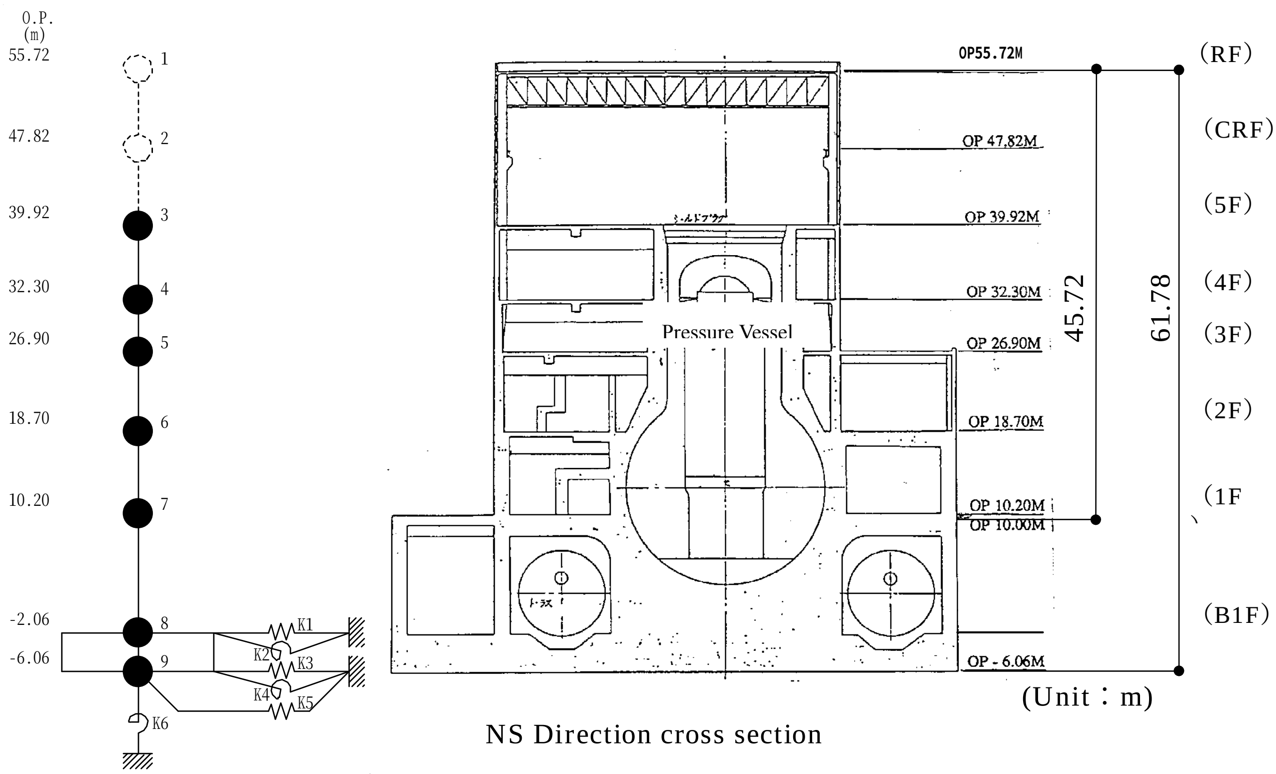


Figure 4-4 Analysis Model for Seismic Response of the Reactor Building of Unit 3 (E-W Direction)

Table 4-1 Physical Factors for Seismic Response Analysis

Concrete	Strength *1 Fc (N /mm²)	Young Coefficient *2 E (N /mm²)	Sharing Elastic Coefficient*2 G (N/mm²)	Poisson's Ratio ν	Weight of Unit Volume*3 γ (kN/m³)
	35.0	2.57×10^4	1.07×10^4	0.2	24
Reinforced Steel	SD345 equivalent (SD35)				

*1：The physical factor for Strength adopted here approximates the strength of the actual situation (hereafter "Real Strength"). The Real Strength is settled using the average value of compressed strength test data in consideration of their variability. Their value has been rounded down.

*2：The value displayed is based on Real Strength.

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	78,130	82.37		
			145.3	9,598
4	119,490	238.33		
			146.1	29,271
5	109,640	201.82		
			237.3	56,230
6	130,160	239.58		
			208.6	60,144
7	226,760	417.47		
			458.7	112,978
8	301,020	554.17		
			2,697.8	496,620
9	127,000	233.79		
Total	1,092,200			

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%
Shape of Basement	47.0 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	78,130	60.05		
			61.9	5,665
4	119,490	124.49		
			123.4	12,460
5	109,640	201.82		
			204.1	41,352
6	130,160	239.58		
			226.6	61,084
7	226,760	622.62		
			431.3	135,128
8	301,020	826.50		
			2,697.8	740,717
9	127,000	348.72		
Total	1,092,200			

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%
Shape of Basement	47.0m (N-S) x 57.4m (E-W)

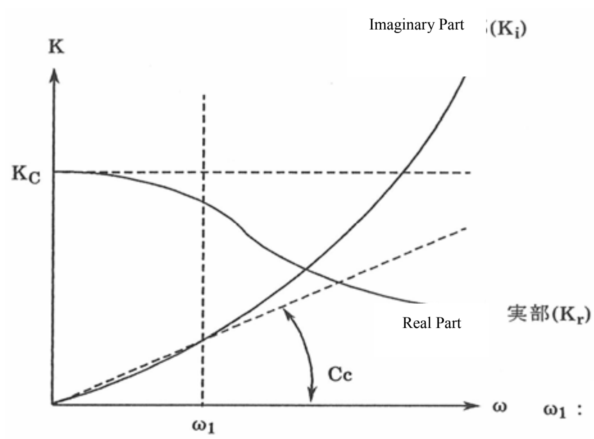
Table 4-3　Ground Factors

(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	–

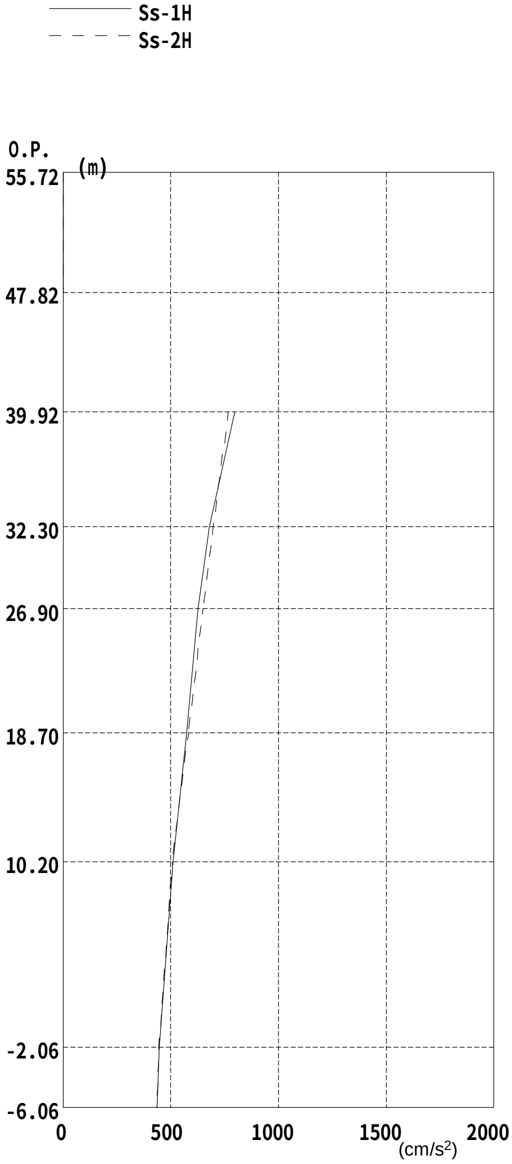


ω1 : Primary Natural Frequency of Building-Ground Connection System

Figure 4-5　Simulation of Ground Spring

5. Analysis Results of Seismic Response

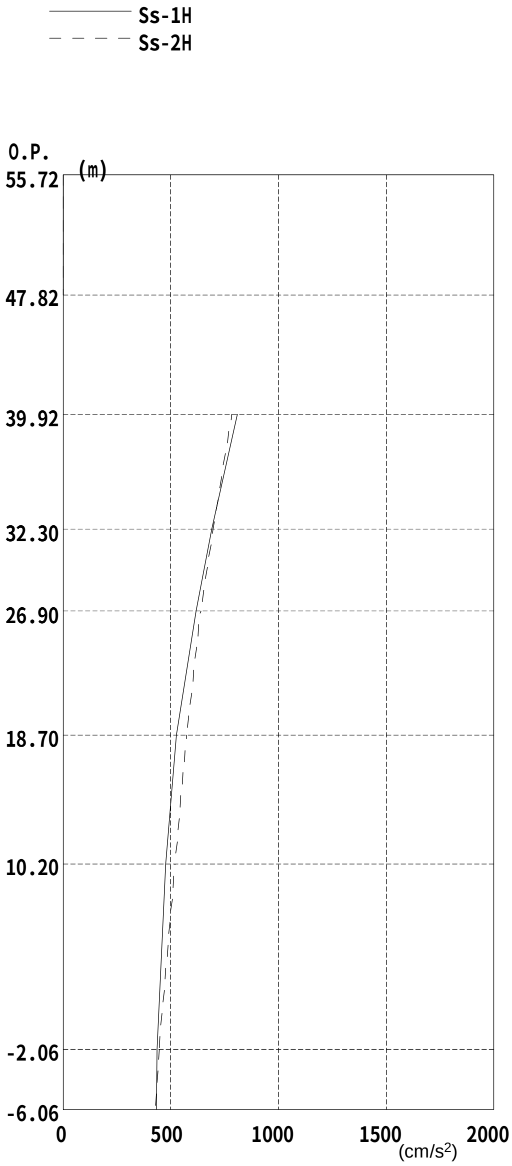
Maximum response acceleration of N-S direction and E-W direction obtained from the seismic response analysis are shown in Figure-5.1 and 5.2 below.



(cm/s²)

Ss-1H	Ss-2H
797	767
681	697
625	650
574	582
510	507
447	445
436	434

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



(cm/s²)

Ss-1H	Ss-2H
808	783
691	696
617	638
528	574
477	519
436	445
434	430

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

6. Evaluation Results of Earthquake-proof Security

Table 6-1 show maximum shearing strain of earthquake-resistant walls. Figure-6.1, 6.2 and 6.3, 6.4 show maximum response values to design basis ground motion Ss-1 and Ss-2 respectively in shearing skeleton curves of earthquake-resistant walls. The maximum shearing strain was estimated to be 0.14×10^{-3} (Ss-2H and N-S 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.

Table 6-1 Maximum response shearing strain of earthquake-resistant walls

($\times10^{-3}$)

	N-S Direction		E-W Direction	
	Ss-1H	Ss-2H	Ss-1H	Ss-2H
4F	0.05	0.04	0.10	0.10
3F	0.10	0.10	0.12	0.12
2F	0.09	0.09	0.10	0.10
1F	0.13	0.14	0.12	0.13
B1F	0.09	0.09	0.09	0.09

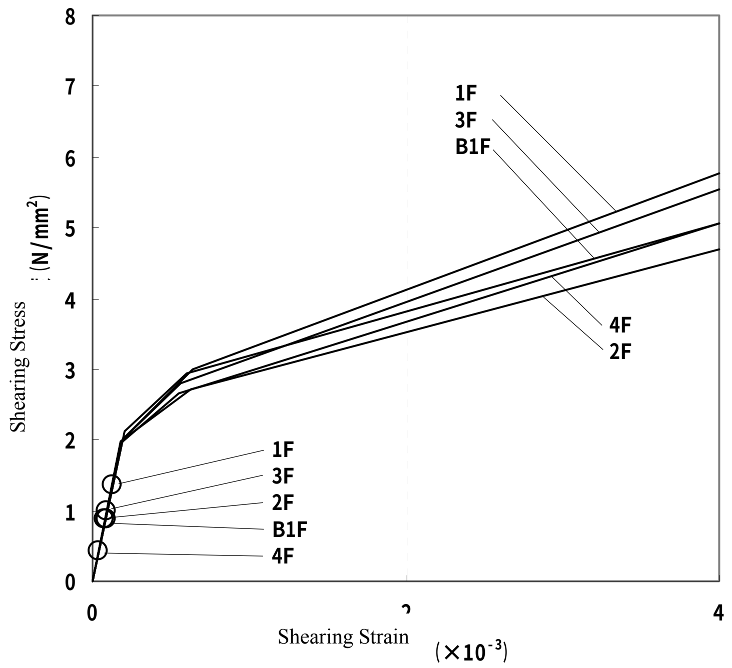


Figure 6-1　Maximum Response Value in Shearing Skelton Curves (Ss-1, N-S Direction)

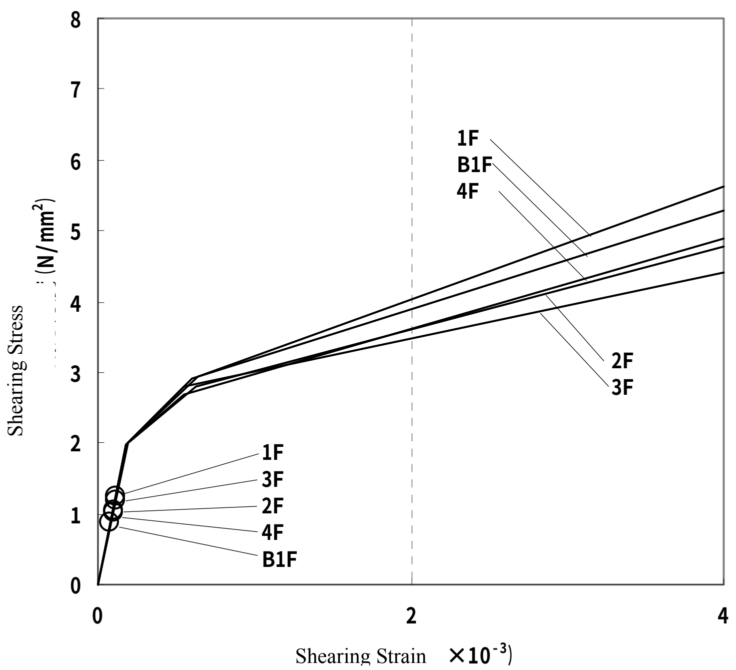


Figure 6-2　Maximum Response Value in Shearing Skelton Curves (Ss-1, E-W Direction)

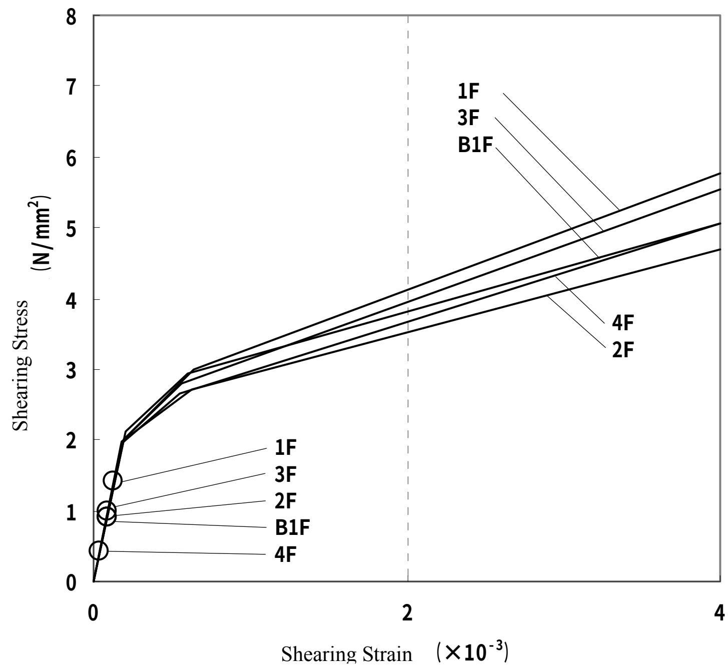


Figure 6-3　Maximum Response Value in Shearing Skelton Curves (Ss-2, N-S Direction)

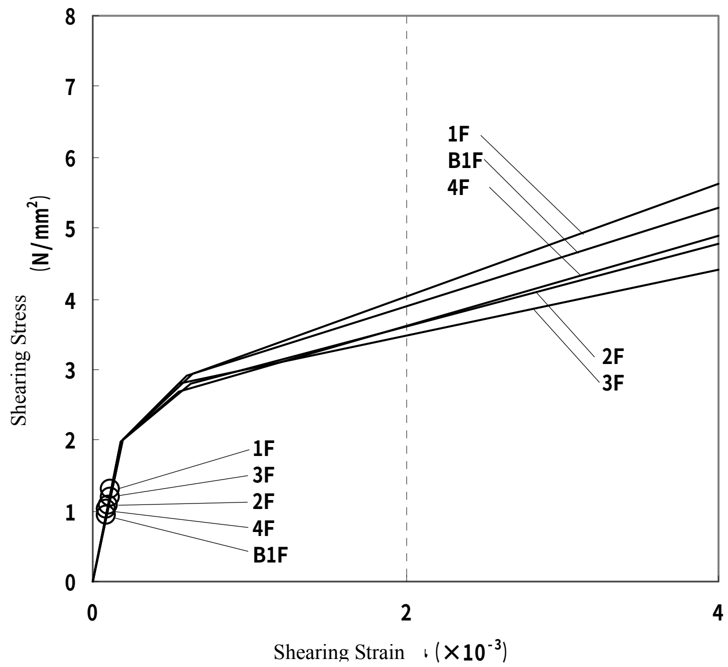
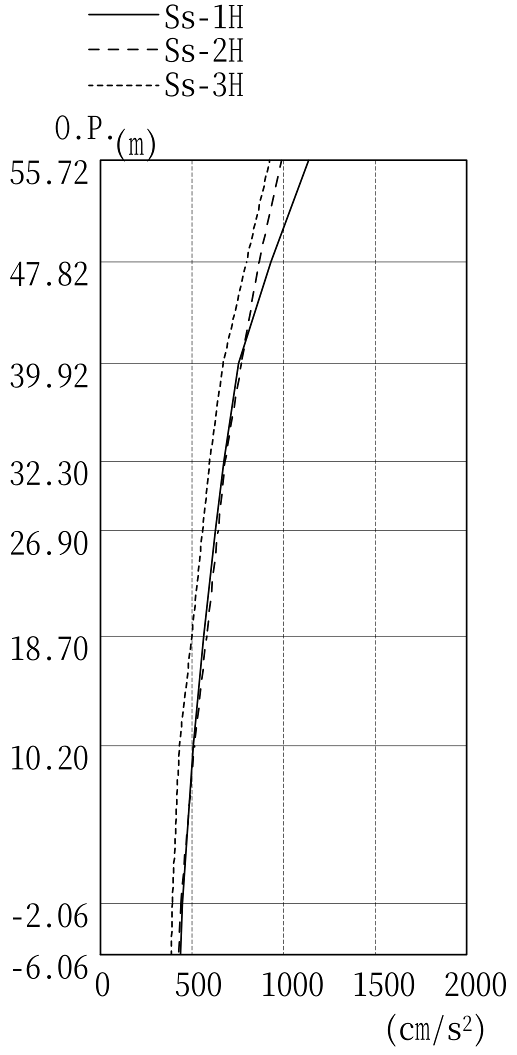


Figure 6-4　Maximum Response Value in Shearing Skelton Curves (Ss-2, E-W Direction)

Seismic Safety Assessment in Response to the Update of ‘Regulatory Guideline for Seismic Design of Nuclear Power Reactor Facilities’

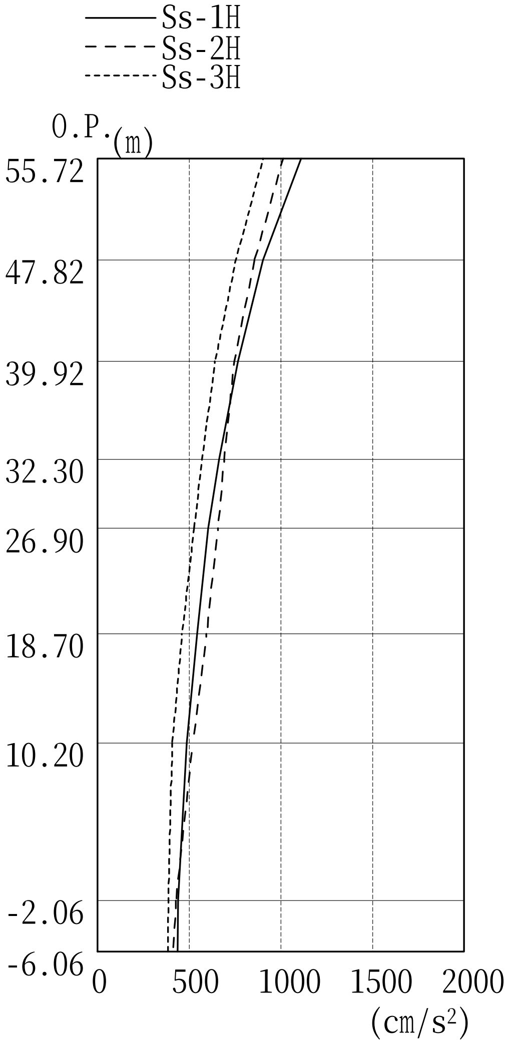
Summarized below is a seismic safety assessment for R/B, Unit 3 of Fukushima Daiichi Nuclear Power Station, which we detailed in a report called “Interim Report (Rev 2, April 19, 2010) - Seismic Safety Assessment in Response to the Update of ‘Regulatory Guideline for Seismic Design of Nuclear Power Reactor Facilities’”



(cm/s²)

Ss-1H	Ss-2H	Ss-3H
1136	990	925
933	866	798
754	770	670
675	681	597
629	643	558
564	580	499
509	514	431
449	440	393
437	427	387

Diagram-1 Maximum Acceleration Response (NS Direction)



(cm/s²)

Ss-1H	Ss-2H	Ss-3H
1112	1013	904
903	858	756
767	747	643
664	692	572
604	658	527
546	596	463
489	520	409
441	429	387
437	414	384

Diagram-2　Maximum Acceleration Response (EW Direction)

Chart-1 Shear Strains on Seismic Wall (NS Direction)

(×10^{-3})

Floor	Ss-1H	Ss-2H	Ss-3H	Criteria
CRF	0.07	0.06	0.06	2.0 or below
5F	0.12	0.11	0.10	
4F	0.04	0.04	0.04	
3F	0.06	0.07	0.06	
2F	0.08	0.09	0.08	
1F	0.13	0.13	0.12	
B1F	0.08	0.08	0.07	

Chart-2 Shear Strains on Seismic Wall (EW Direction)

(×10^{-3})

Floor	Ss-1H	Ss-2H	Ss-3H	Criteria
CRF	0.09	0.09	0.08	2.0 or below
5F	0.12	0.11	0.09	
4F	0.08	0.08	0.07	
3F	0.09	0.09	0.08	
2F	0.10	0.10	0.09	
1F	0.12	0.12	0.10	
B1F	0.08	0.09	0.07	

END

Appendix-2: the detail of the evaluation result of the anit-quake safety of the Reactor Building, Unit 3
（local evaluation by three-dimensional FEM analysis）

1. Policy for examination and evaluation

As for the Reactor Building of Unit 3, 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 5FL of Reactor Building is figure 1.1 and the vertical drawing is figure1.2.

The evaluation procedure of the anti-quake safety

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

- To conduct the evaluation centering on the Spent Fuel Pool, we will construct the three-dimensional FEM model that simulates damage by the explosion etc. from 2FL (O.P.18.7m) to 5FL (O.P.39.92m)
- 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 and shell wall.
- We evaluate the anti-quake safety by comparing figures with the evaluation standard.

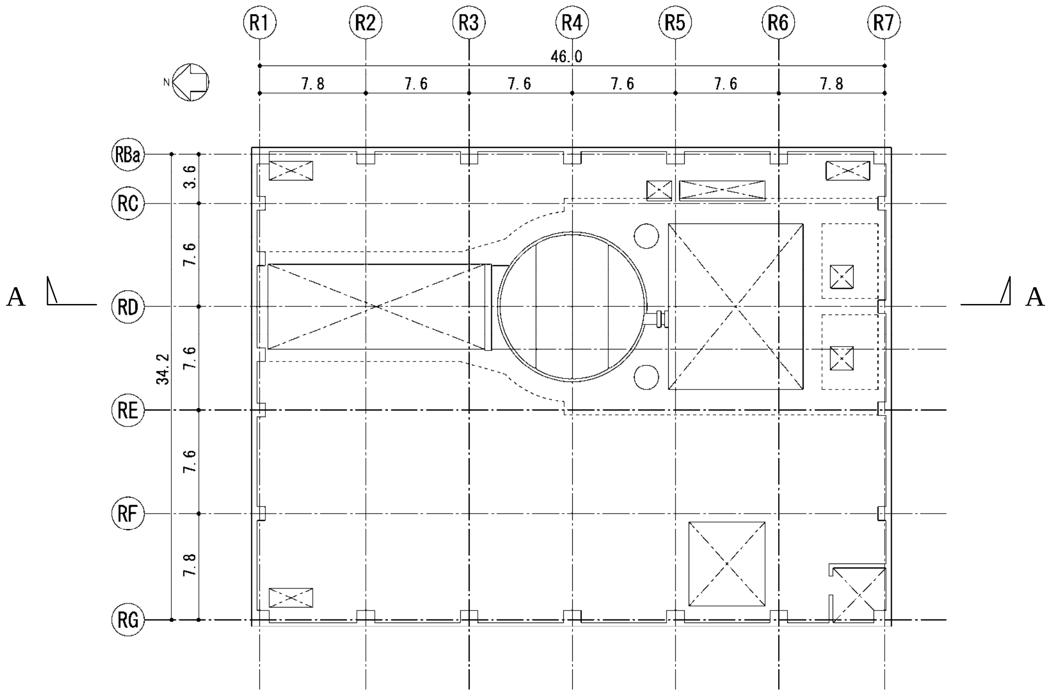


Figure 1.1: 5FL(OP 39.92) horizontal drawing（unit: m）

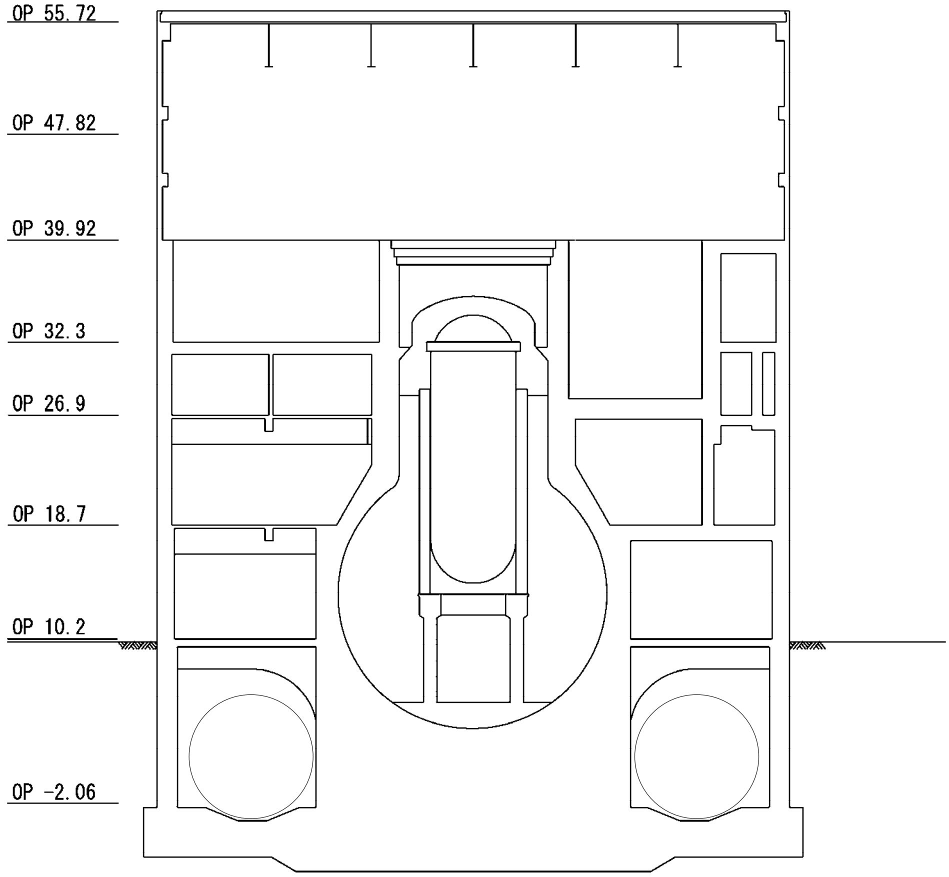


Figure 1.2: vertical drawing (A-A direction, unit: m)

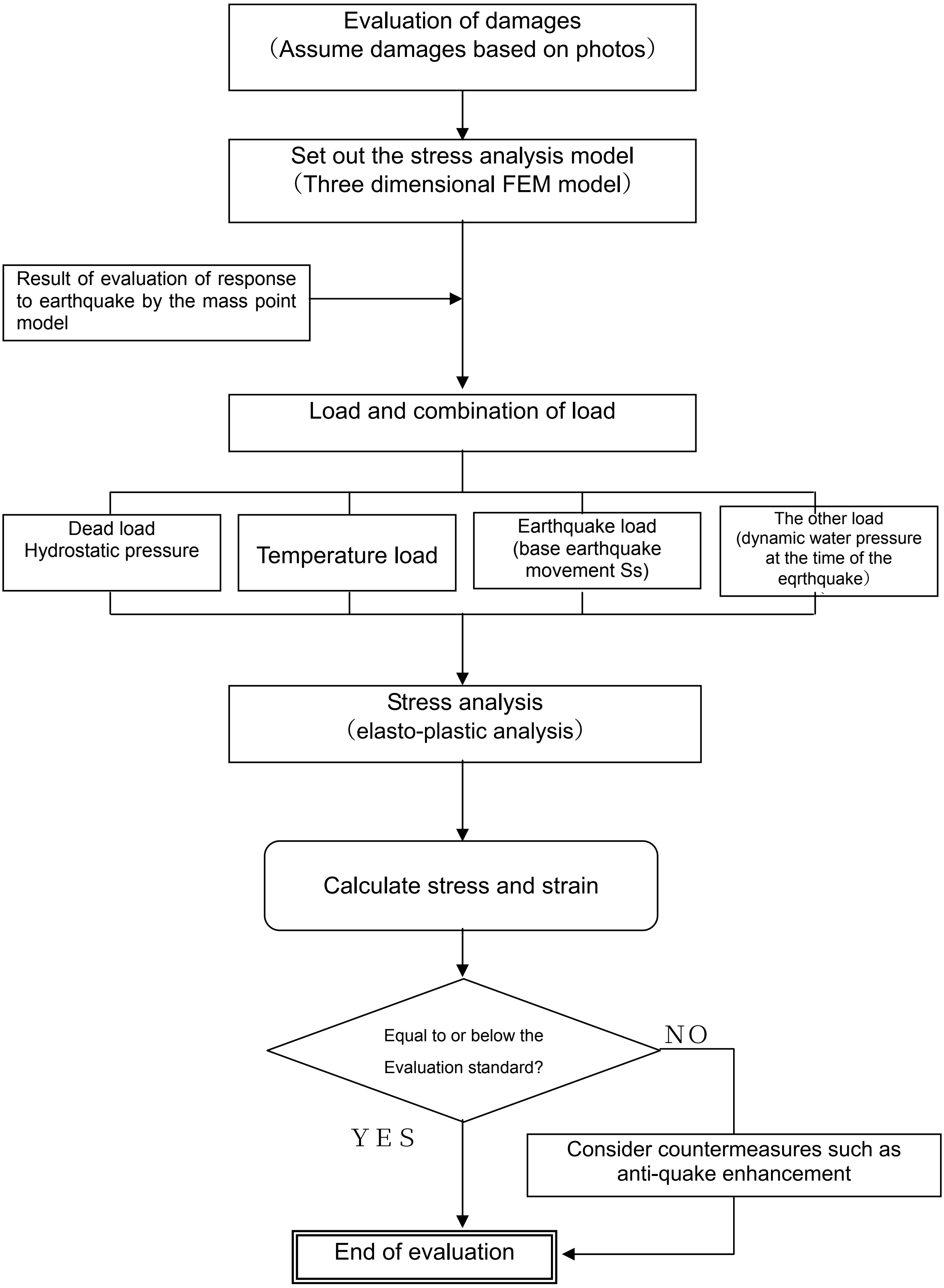


Figure 1.3: flowchart for local evaluation of anti-quake safety

2. Evaluation of the status of damages

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

The outer wall evaluated in the analytical model is considered the same as the part used in Attachment-2.

Taking in account of the effect of the explosion, rigidity of the 5^{th} and 4^{th} floor is reduced to 50%, while the rigidity of the spent fuel pool, temporary equipment storage pool and the reactor well is reduced to 80%.

We have not been able to visually check the shell wall, however, as the shell walls are thicker compared with the damaged outer walls (maximum 600mm thick), we have evaluated with the assumption that there is no damage to the shell walls.

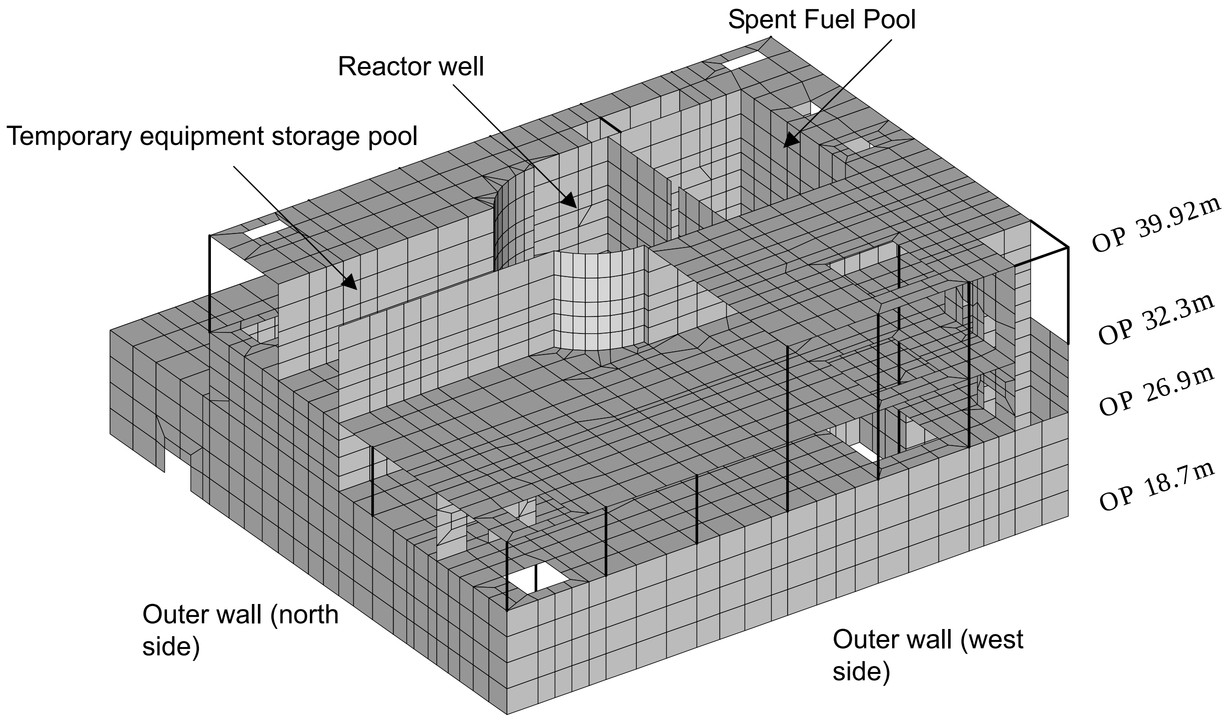
The weight of damaged parts is assumed to be supported by the floor below and uniformly distributed.

3. The stress analysis model

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

For the plate element used in the analytical model, a laminated shell element by anisotropic materials that models the reinforcing steel layer is used. 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 is “ABAQUS”.

Figure 3.1 shows 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.



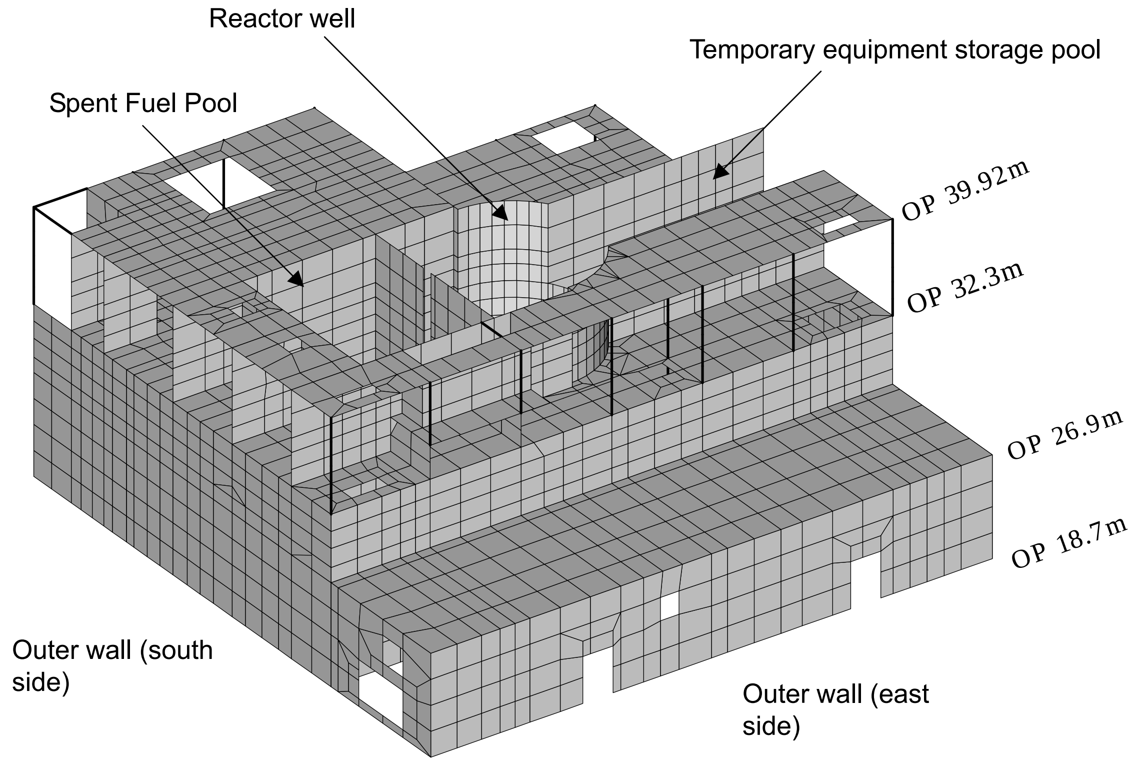
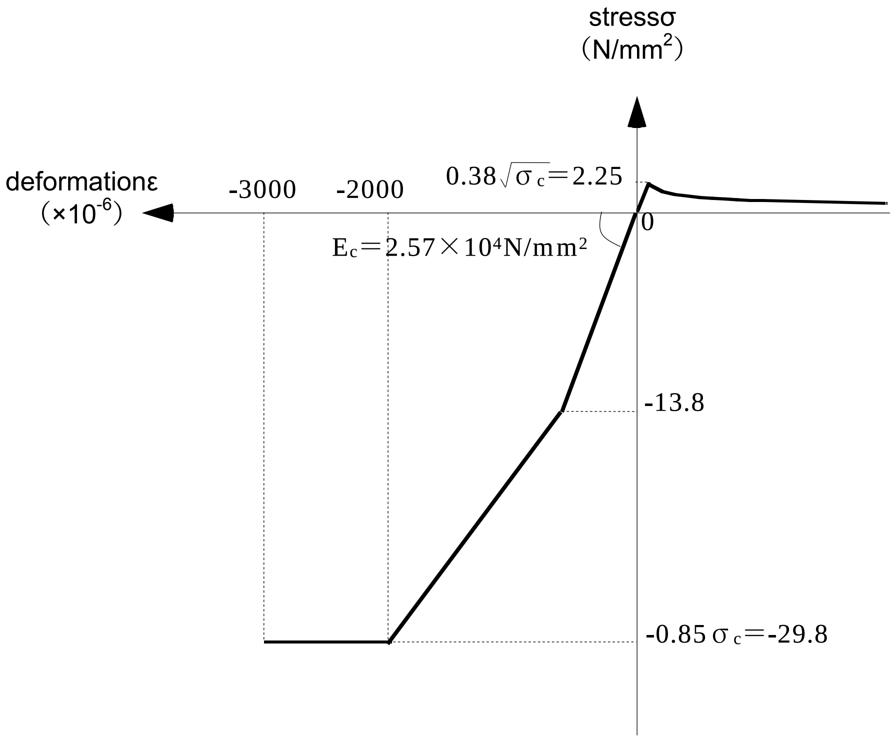
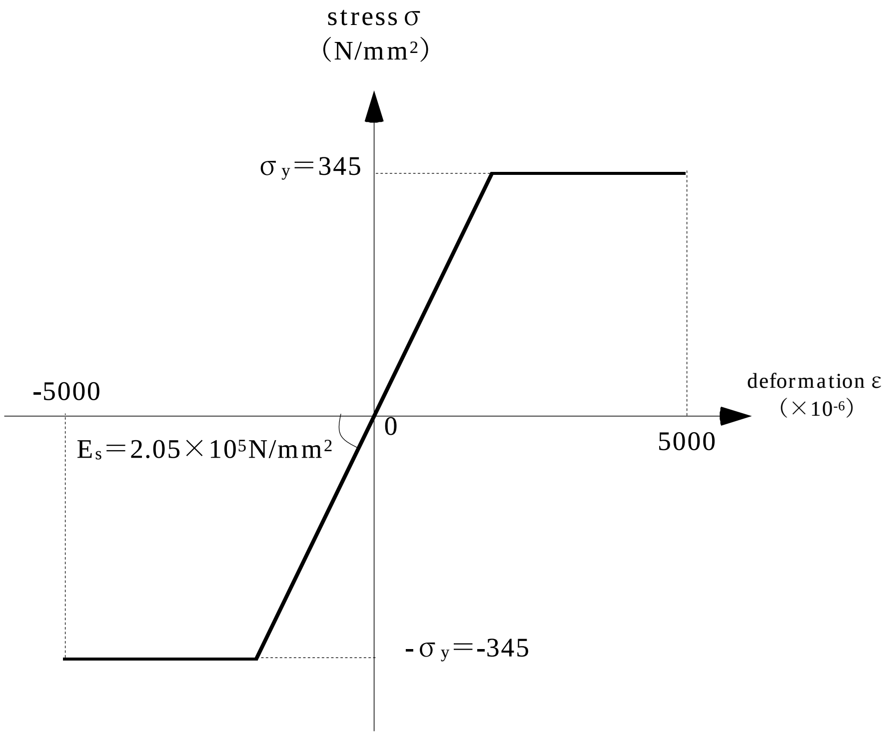


Figure 3.1 The outline of the analytical model



(a) stress-deformation graph of concrete

(strength of concrete $\sigma c = 35N/mm^2$)



(b) stress-deformation graph of reinforcing steel

(Yield point of reinforcing steel $\sigma y = 345N/mm^2$)

Figure 3.2: the constitutive law of concrete and reinforcing steel

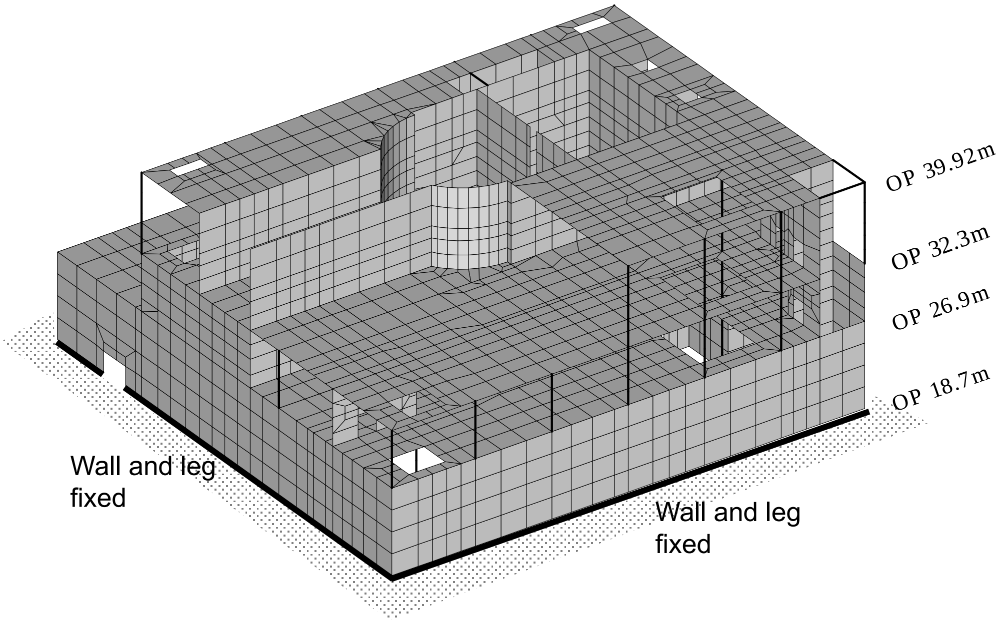


Figure 3.3: the boundary condition of the analytical model

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 62°C) into consideration, we assume the water temperature of 65°C and the ambient temperature of 10°C. For the Primary Containment Vessel atmosphere temperature, we assume 110°C from the historical record.

(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 2-1).

(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 2-2.

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 Reactor Building based on the placement of reinforcing steel etc. and evaluate the anti-quake safety. The points of evaluation are shown in figure 5.1 and 5.2.The placement of reinforcing steel for evaluation is shown in figure 5.1.

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 evaluation result is shown in table 5.2 and 5.3.As the stress and the strain are within elasticity span and below the evaluation standard for each point, we presume that the current Reactor Building 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

In the evaluation of the damage, as there is a possibility of the variance of rigidity, we conducted parameter study which take the reduction of rigidity of shell wall by the high temperature of the reactor or the reduction of rigidity of spent fuel pool by explosion into account, and furthermore, we also conducted parameter study which take the easing the reduction of rigidity as there are huge uncertainties on the other hand. Though there were some variance on the result, however, there is no huge impact on the result of analysis, hence, we confirmed the variance of the assumption will not have huge impact on the result of the analysis.(see appendix 2-3)

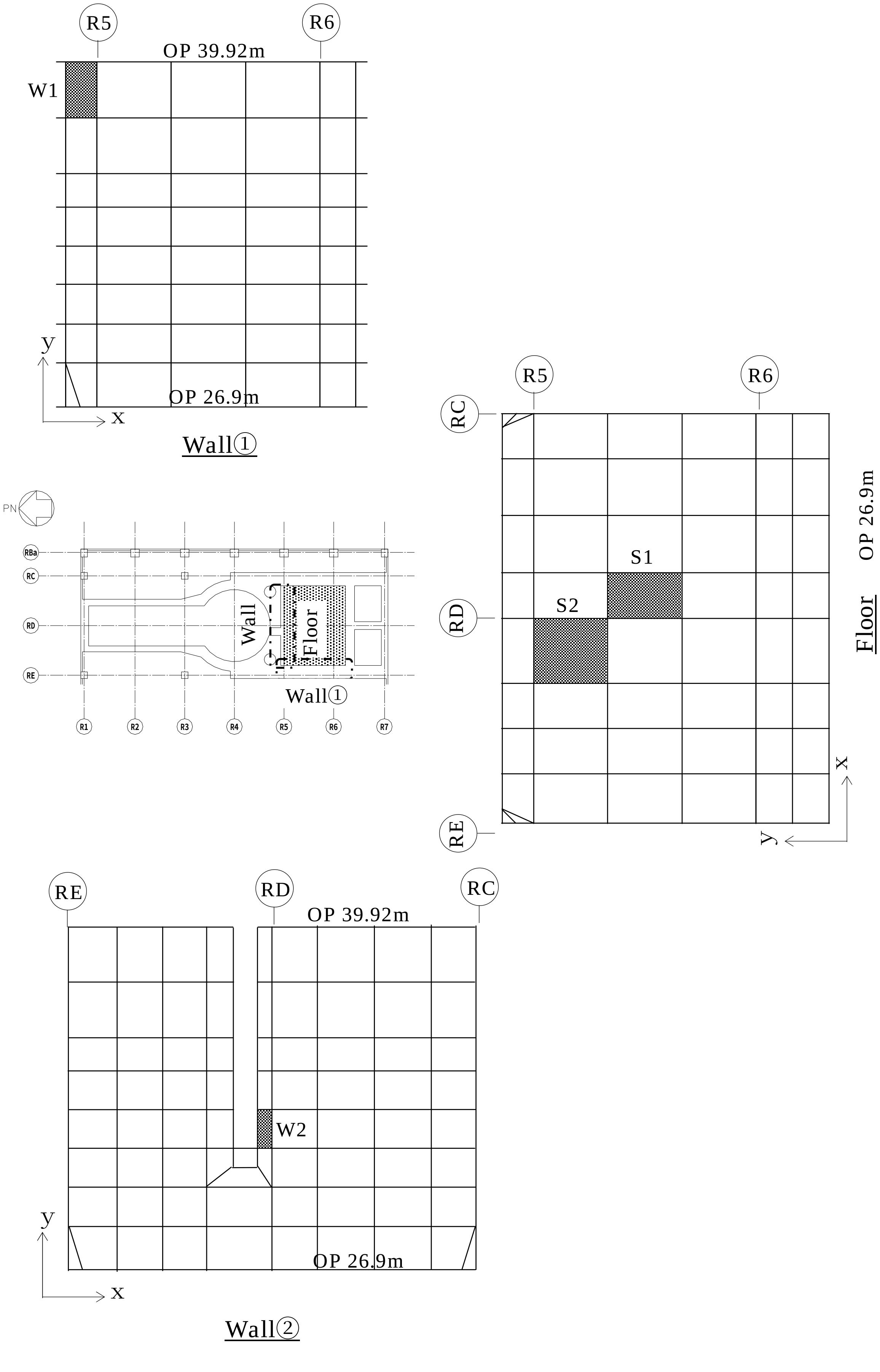


Figure 5.1: points of evaluation (1)

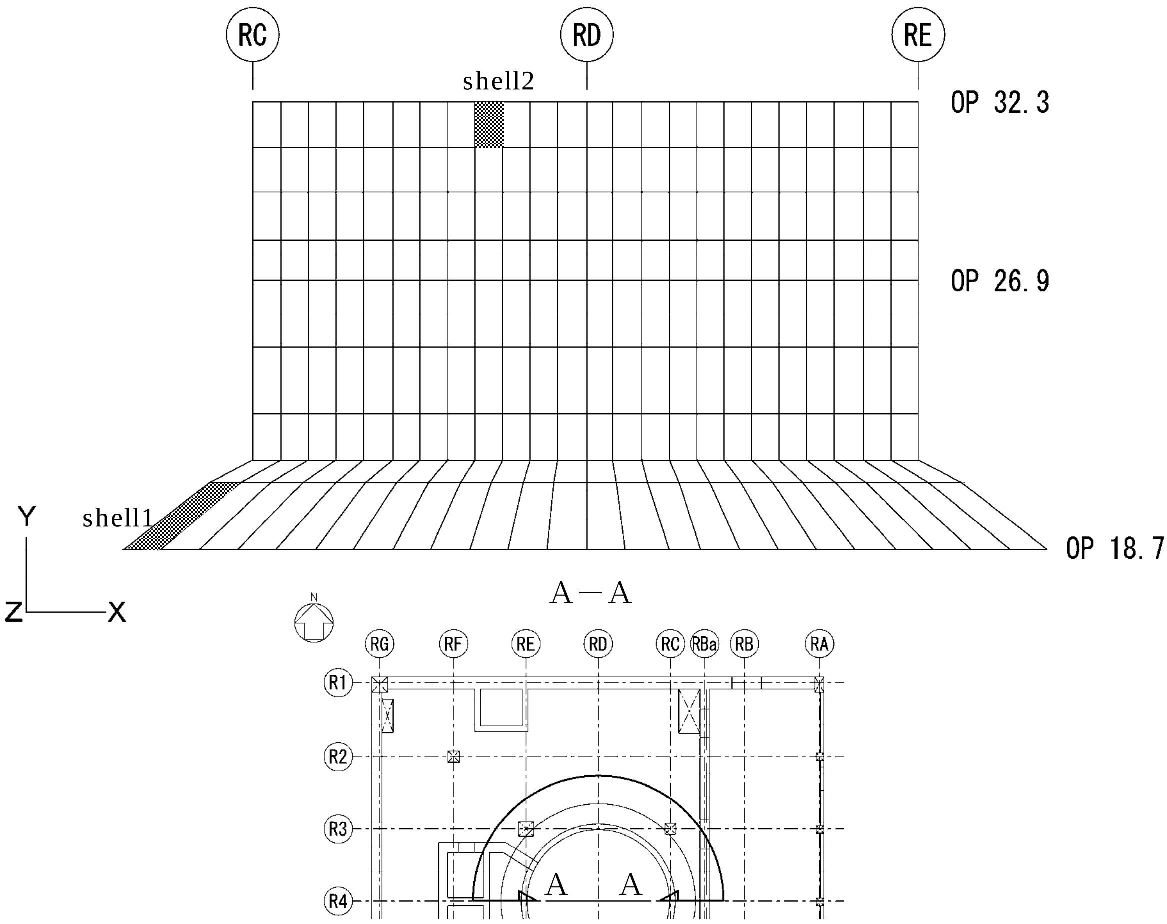


Figure 5.1: points of evaluation (2)

Table 5.1 the specification of evaluated concrete and reinforcing steel

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	D38@130	D38@130	D38@160	D38@130	—
position	Upper end reinforcing steel		Lower end reinforcing steel		Shear reinforcing steel
	X direction	Y direction	X direction	Y direction	
S1 S2	D32@100＋D32@200		D32@200		—
position	Inner reinforcing steel		Outer reinforcing steel		Shear reinforcing steel
	X direction	Y direction	X direction	Y direction	

shell 1	D38@100＋D38@150	D38@100＋D38@200	D38@100＋D38@150	D38@120＋D38@240	—
shell 2	D38@130	D38@130	D38@150	D38@130	—

Table 5.2(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	-667	-3000	Ok
	${}_s\varepsilon_c$		-588	-5000	Ok
	${}_s\varepsilon_t$		1303	5000	Ok

Table 5.2(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	-443	-3000	Ok
	${}_s\varepsilon_c$		-165	-5000	Ok
	${}_s\varepsilon_t$		335	5000	Ok

Table 5.2(2) the evaluation result of strain of concrete and reinforcing steel by axial force and bend moment (shell wall)

position	Strain considered	Name of the load	Strain occurred ($\times 10^{-6}$)	Evaluation standard ($\times 10^{-6}$)	decision
shell 1	${}_c\varepsilon_c$	Ss at the time of earthquake	-567	-3000	Ok
	${}_s\varepsilon_c$		-469	-5000	Ok
	${}_s\varepsilon_t$		408	5000	Ok

Table 5.3(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	1689	3130	Ok

Table 5.3(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	897	1900	Ok

Table 5.3(3) the evaluation result of out-of-plane shear force (shell wall)

position	Name of the load	Strain occurred Q (N/mm)	Evaluation standard (N/mm)	decision
Shell 2	Ss at the time of earthquake	2475	3270	Ok

Regarding the earthquake response analysis for vertical direction of Reactor Building of Unit 3

With regards to the local evaluation of 3 dimensional FEM analysis of the reactor building of Unit 3 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 1: Detail of seismic safety evaluation of Reactor building of Unit 3 (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.

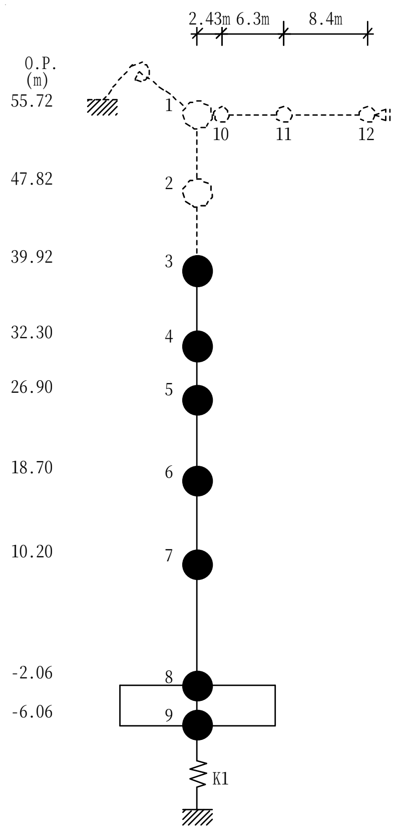


Figure-1 Building Analysis Model(vertical direction)

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(×10^8kN/m)
1	-	-	-
2	-	-	-
3	78,130	192.0	6.48
4	119,490	266.3	12.67
5	109,640	431.7	13.53
6	130,160	423.0	12.79
7	226,760	691.2	14.49
8	301,020	2,697.8	173.33
9	127,000		
Total	1,092,200		

Ceiling			
MP No.	Mass Point Weight W(kN)	Shear Area A_S(×10^{-2} m^2)	Shear 2nd Moment I(m^4)
1	-	-	-
10	-	-	-
11	-	-	-
12	-		

①Concrete Part

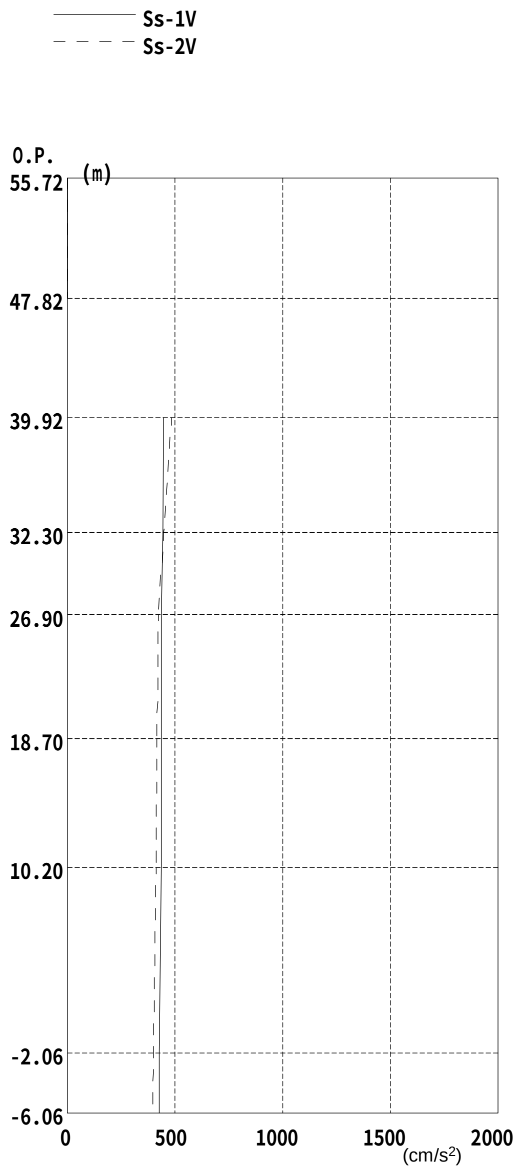
Young Modulous	E_C	2.57×10^7 (kN/m²)
Shear Elastic Modulus	G	1.07×10^7 (kN/m²)
Poisson Ratio		0.20
Decay	h	5%

②Iron Frame Part

Young Modulous	E_S	2.05×10^8 (kN/m²)
Shear Elastic Modulus	G	7.90×10^7 (kN/m²)
Poisson Ratio		0.30
Decay	h	2%

Base Configuration 47.0m(NS Direction)×57.4m(EW Direction)

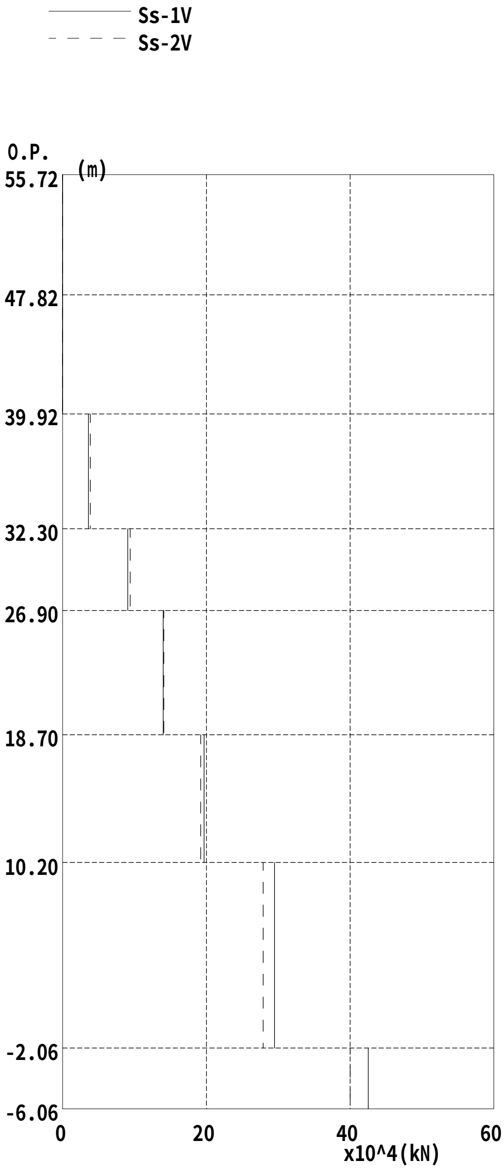
Result of Maximum Response Acceleration and Maximum Response Axial Force of vertical direction by earthquake response analysis are shown in Fugure-2 and Figure-3 below.



(cm/s²)

Ss-1V	Ss-2V
448	484
445	448
438	422
435	417
435	413
428	399
426	397

Figure-2 Maximum Response Acceleration(Vertical Direction)



x10^4(kN)

Ss-1V	Ss-2V
3.60	3.89
9.05	9.38
13.97	14.05
19.68	19.22
29.44	27.86
42.48	39.96

Figure-3　Maximum Response Axial force(Vertical Direction）

Parametric Study regarding Temperature Load

1. Study Overview

In Attachment – 2, we evaluated the seismic safety by assuming Design Basis Ground Motion Ss and Temperature Load (assumption on the water temperature in the pool is approx. 65°C) 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 account of Temperature Load.

2. Methodology

Based on the combination of load in Attachment 2 (Base Case), we examine a combination of load without Temperature Load set out in Table 1. Assumptions other than the combination of load are the same as Base Case, including the analytical model.

Table 1: Combination of Load

Name of the case	Combination of Load
Under Earthquake Ss	DL+H+K+KH

DL ：Dead Load
H ：Hydrostatic Pressure
K ：Earthquake Load (Design Basis Ground Motion Ss)
KH ：Dynamic Water Pressure under Earthquake Ss

3. Evaluation result

We set out the result of strain of concrete and rebar at the Spent Fuel Pool etc, the same place (element) as the Base Case in Table 2. Table 3 is the result of out-of-plane shear stress. For reference, we included the Base Case evaluation results that take account of the Temperature Load for comparison purpose in tables 2 and 3.

From the evaluation results, it can be estimated that stress and strain of Reactor Building are within the standard range and the seismic safety is secured even without considering Temperature Load

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

Place	Strain Evaluated	Name of the case	Strain generated (×10-6)		Evaluation Standard (×10^{-6})	Evaluation
			This Study (Without Temperature Load)	(reference) Base Case		
W1	$_c\varepsilon_c$	Under Earthquake Ss	-435	-667	-3000	OK
	$_s\varepsilon_c$		-365	-588	-5000	OK
	$_s\varepsilon_t$		444	1303	5000	OK

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

Place	Strain Evaluated	Name of the case	Strain generated (×10-6)		Evaluation Standard (×10^{-6})	Evaluation
			This Study (Without Temperature Load)	(reference) Base Case		
S1	$_c\varepsilon_c$	Under Earthquake Ss	-149	-443	-3000	OK
	$_s\varepsilon_c$		-42	-165	-5000	OK
	$_s\varepsilon_t$		160	335	5000	OK

Table 2(3): Evaluation results of the strain of concrete and rebar by axial force and bending stress (shell wall)

Place	Strain Evaluated	Name of the case	Strain generated (×10-6)		Evaluation Standard (×10^{-6})	Evaluation
			This Study (Without Temperature Load)	(reference) Base Case		
Shell 1	$_c\varepsilon_c$	Under Earthquake Ss	-110	-567	-3000	可
	$_s\varepsilon_c$		-107	-469	-5000	可
	$_s\varepsilon_t$		53	408	5000	可

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

Place	Name of the case	Strain generated Q (N/mm)		Evaluation
		This Study (Without Temperature Load)	(reference) Base Case	
W2	Under Earthquake Ss	530 (3130)	1689 (3130)	OK

Evaluation Standard is indicated in ()

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

Place	Name of the case	Strain generated Q (N/mm)		Evaluation
		This Study (Without Temperature Load)	(reference) Base Case	
S2	Under Earthquake Ss	841 (2200)	897 (1900)	OK

Evaluation Standard is indicated in ()

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

Place	Name of the case	Strain generated Q (N/mm)		Evaluation
		This Study (Without Temperature Load)	(reference) Base Case	
Shell 2	Under Earthquake Ss	842 (3400)	2475 (3270)	OK

Evaluation Standard is indicated in ()

Parametric Study on Seismic Safety Evaluations of Reactor Building

1. Review Policy

We will conduct a parametric analysis in consideration of the following variation factors we have not assumed in a base case and understand to what extent they will have impacts on seismic safety evaluations of the reactor building.

[Damage scenario not assumed in a base case]

Due to the explosion, the most of the roof and the external walls at and more than three stories have been destroyed and the impacts on the stiffness of pool/pit on three to five stories, walls, floors and other members have been considered. As the influences and impacts have been confirmed by pictures taken at great distances, there might be the big variability of the stiffness settings.

In addition, stiffness might decrease, as the temperature in the Primary Containment Vessel temporarily increased after the earthquake.

2. Review Conditions

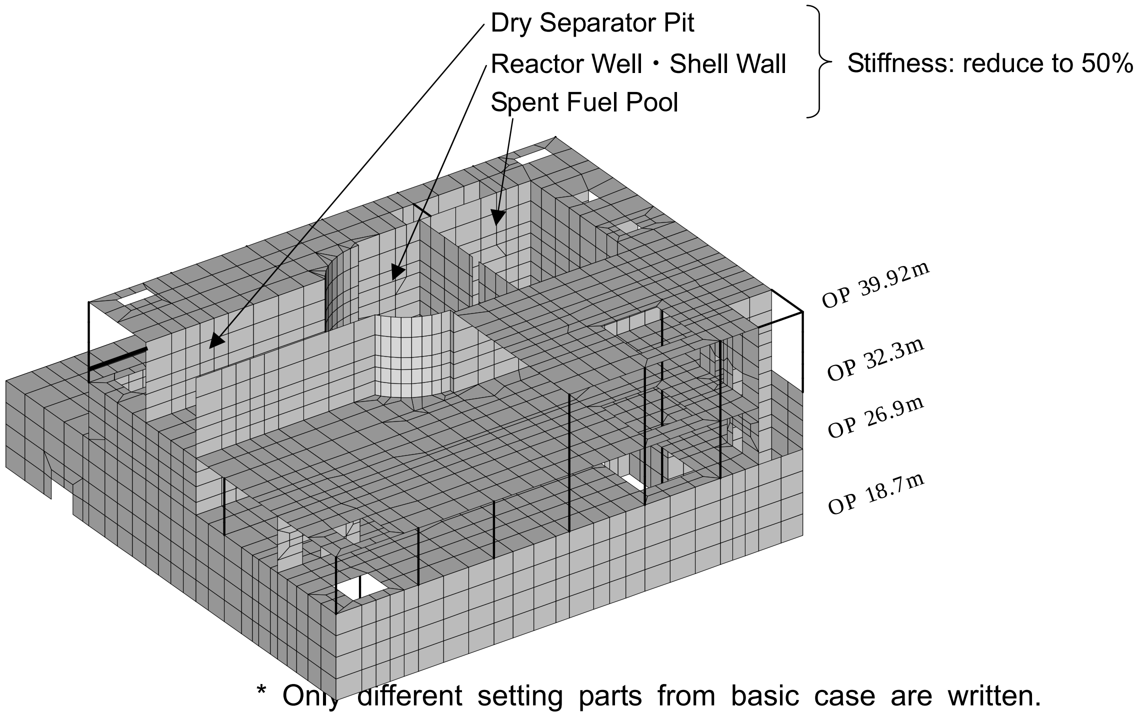
2.1 Review conditions to review impacts due to the explosion

The possibility of decrease in stiffness can be considered, as the explosion might cause remaining floors and walls to crack. However, some parts cannot be confirmed by pictures take at great distances. Hence, there might be the big variability of the stiffness settings.

In addition, the stiffness might be influenced, as the temperature in the Primary Containment Vessel temporarily increased after the earthquake. As shown in Figure-1, we will review the impacts on seismic safety evaluations of the damage conditions of half destroyed external walls and walls and floors of pool/pit and the temporary increase in temperatures in the Primary Containment Vessel in two cases (hereinafter called “Case 1” and “Case 2”).

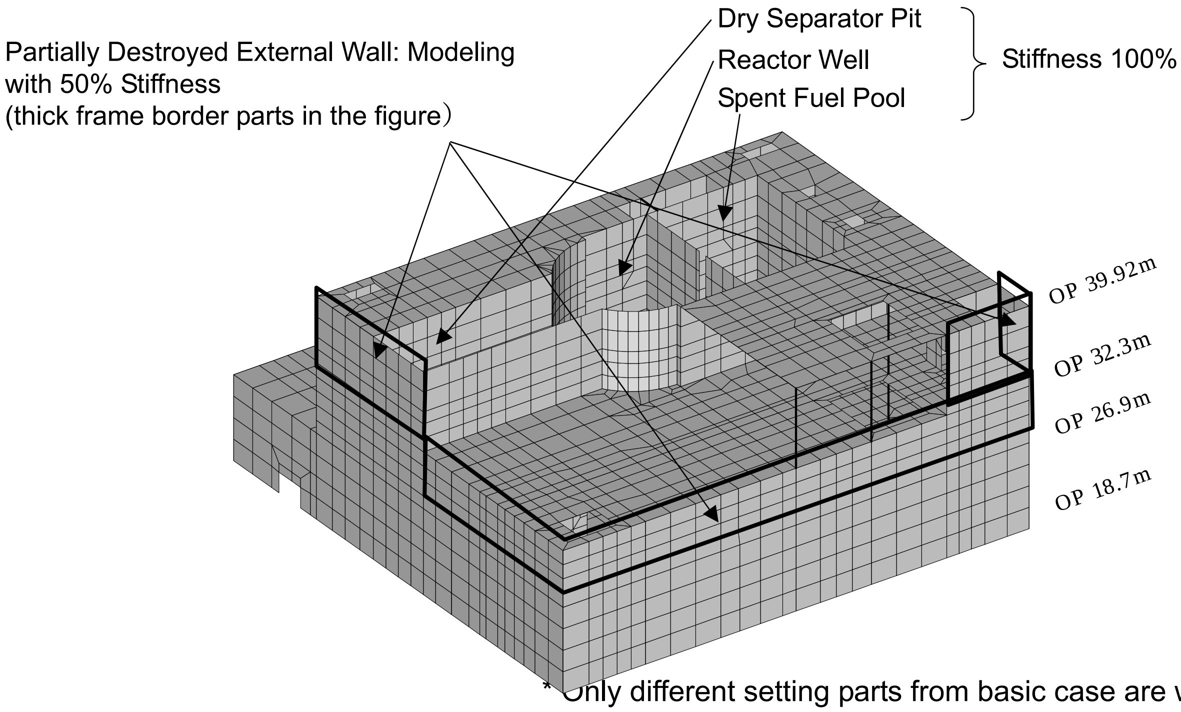
In case 1, we assume that the stiffness of the spent fuel pool, the Drier Separator Pit, the Reactor Well and the shell wall will reduce by 50%.

In case 2, we will create a model in which the stiffness of the half destroyed external wall whose external surface only had been damaged decreased by 50%, though we have not considered in a process of modelization in a base case. We assume that the spent fuel pool, the Reactor Well, and the Drier Separator Pit are sound and no stiffness decreases.



* Only different setting parts from basic case are written.

Figure-1(1) Study Condition for Explosion Impact (Case 1)



* Only different setting parts from basic case are written.

Figure-1(2) Study Condition for Explosion Impact (Case 2)

2.2 Study Case

List of the study cases summarizing study conditions including basic case are shown in Table-1. As for the study case, we consider the same load combination as basic case, and study impacts on seismic safety evaluation of reactor building.

Table-1 List of Study Case

Case		Items for impact study			
		External wall (3rd–4th floor)	Floor (4th–5th floor)	Spent fuel pool, etc.	Pool water temperature
-	Basic	Treat partially destroyed wall as the fully-destroyed	Treat northwest 5th floor as the fully-destroyed, and other floors' stiffness reduced to 50%	Decrease the stiffness by 80%	65℃ (outside temperature: 10℃)
1	Impact of an explosion (1)	*	*	Decrease the stiffness by 50%	*
2	Impact of an explosion (2)	Modeling partially destroyed wall by 50% stiffness	*	No decrease in the stiffness	*

Note) *: same condition as the basic case

3. Study Result

We indicate the compared results of the ratio of strain or stress to evaluation standard value of basic case and study case in the table-2. We confirmed that there is no effect on seismic safety evaluation of reactor building, even considering the impact by temporary temperature raise in the primary containment vessel that is not expected in the basic case and indeterminacy of stiffness depression by explosion.

In addition, we indicate the detail of the results of seismic safety evaluation for the study case1to 2 in table-3 to table-6 for reference.

Table-2 Comparison of Ratio of the Strain or the Stress to Evaluation Standard Value

	Evaluation items	Base case	【Case 1】 Explosion impact (1)	【Case 2】 Explosion impact (2)
Pool floor	Reinforcing bar strain	0.07	0.07	0.07
	Concrete strain	0.15	0.18	0.14
	Out-of-plane shear force	0.48	0.42	0.52
Pool wall	Reinforcing bar strain	0.27	0.19	0.25
	Concrete strain	0.23	0.21	0.23
	Out-of-plane shear force	0.54	0.47	0.58
Shell wall	Reinforcing bar strain	0.10	0.10	0.10
	Concrete strain	0.19	0.19	0.20
	Out-of-plane shear force	0.76	0.53	0.78

Note) Value less than one means below evaluation standard value in the table.

[Case 1]

Table-3(1) Study result of strain of concrete and reinforcing bar by axial force and bending moment (wall)

Spot	Study strain	Loading name	Strain ($\times 10^{-6}$)	Evaluation standard value ($\times 10^{-6}$)	Judgment
W1	${}_c\varepsilon_c$	Ss Earthquake	-626	-3000	Pass
	${}_s\varepsilon_c$		-547	-5000	Pass
	${}_s\varepsilon_t$		914	5000	Pass

Table-3(2) Study result of strain of concrete and reinforcing bar by axial force and bending moment (floor)

Spot	Study strain	Loading name	Strain ($\times 10^{-6}$)	Evaluation standard value ($\times 10^{-6}$)	Judgment
S1	${}_c\varepsilon_c$	Ss Earthquake	-518	-3000	Pass
	${}_s\varepsilon_c$		-198	-5000	Pass
	${}_s\varepsilon_t$		338	5000	Pass

Table-3(3) Study result of strain of concrete and reinforcing bar by axial force and bending moment (Shell wall)

Spot	Study strain	Loading name	Strain ($\times 10^{-6}$)	Evaluation standard value ($\times 10^{-6}$)	Judgment
shell 1	${}_c\varepsilon_c$	Ss Earthquake	-568	-3000	Pass
	${}_s\varepsilon_c$		-482	-5000	Pass
	${}_s\varepsilon_t$		405	5000	Pass

Note) Refer to the basic case as for the study target spot.

[Case 1]

Table-4(1) Study results of out-of-plane shear force (wall)

Spot	Loading name	Initial stress Q (N/mm)	Evaluation standard value (N/mm)	Judgment
W2	Ss Earthquake	1462	3130	Pass

Table-4(2) Study results of out-of-plane shear force (floor)

Spot	Loading name	Initial stress Q (N/mm)	Evaluation standard value (N/mm)	Judgment
S2	Ss Earthquake	915	2200	Pass

Table-4(3) Study results of out-of-plane shear force (shell wall)

Spot	Loading name	Initial stress Q (N/mm)	Evaluation standard value (N/mm)	Judgment
Shell2	Ss Earthquake	1759	3330	Pass

Note) Refer to the basic case as for the study target spot.

[Case 2]

Table-5(1) Study results of strain of concrete and reinforcing bar by axial force and bending moment（wall）

Spot	Study strain	Loading name	Strain (×10^{-6})	Evaluation standard value (×10^{-6})	Judgment
W1	${}_{c}\varepsilon_{c}$	Ss Earthquake	-673	-3000	Pass
	${}_{s}\varepsilon_{c}$		-595	-5000	Pass
	${}_{s}\varepsilon_{t}$		1234	5000	Pass

Table-5(2) Study results of strain of concrete and reinforcing bar by axial force and bending moment（floor）

Spot	Study Strain	Loading name	Strain (×10^{-6})	Evaluation Standard value (×10^{-6})	Judgment
S1	${}_{c}\varepsilon_{c}$	Ss Earthquake	-413	-3000	Pass
	${}_{s}\varepsilon_{c}$		-141	-5000	Pass
	${}_{s}\varepsilon_{t}$		350	5000	Pass

Table-5(3) Study results of strain of concrete and reinforcing bar by axial force and bending moment（shell wall）

Spot	Study Strain	Loading name	Strain (×10^{-6})	Evaluation standard value (×10^{-6})	Judgement
Shell 1	${}_{c}\varepsilon_{c}$	Ss Earthquake	-576	-3000	Pass
	${}_{s}\varepsilon_{c}$		-477	-5000	Pass
	${}_{s}\varepsilon_{t}$		410	5000	Pass

Note) Refer to the basic case as for the study target spot.

[Case 2]

Table-6(1) Study results of out-of-plane shear force (wall)

Spot	Loading name	Initial stress Q (N/mm)	Evaluation standard value (N/mm)	Judgment
W2	Ss Earthquake	1804	3130	Pass

Table-6(2) Study results of out-of-plane shear force (floor)

Spot	Loading name	Initial stress Q (N/mm)	Evaluation standard value (N/mm)	Judgment
S2	Ss Earthquake	913	1790	Pass

Table-6(3) Syudy results of out-of-plane shear force (shell wall)

Spot	Loading name	Initial stress Q (N/mm)	Evaluation standard value (N/mm)	Judgment
Shell 2	Ss Earthquake	2611	3360	Pass

Note) Refer to the basic case as for the study target spot.