Report on the Current Seismic Safety and Reinforcement of the Reactor Buildings at Fukushima Daiichi Nuclear Power Station (No.1) (Supplement) (Revision 2)

December 2012

Tokyo Electric Power Company

Contents

Report on the Current Seismic Safety and Reinforcement of the Reactor Buildings at Fukushima Daiichi Nuclear Power Station (No.1) (Supplement) (Revision 2)

1. Introduction

Following the progress in the fuel removal cover designing and removal of debris at the Unit 4 reactor building of the Fukushima Daiichi Nuclear Power Station, the condition of the building at the time of the removal of spent fuel and detailed conditions of the damage to the building frame are being revealed. In response, this report addresses the results of the seismic safety assessment of the spent fuel pool and reactor building at Unit 4 anticipating the time when spent fuel is to be removed, and serves as a supplement to the “Reports about the study regarding current seismic safety and reinforcement of reactor buildings at Fukushima Daiichi Nuclear Power Station (1)” (Tokyo Electric Power Company), which was released on May 28, 2011 (hereinafter, “2011 Report”).

2. Condition of Reactor Building at the Time of Removal of Spent Fuel

The condition of the reactor building at the time of removal of spent fuel was estimated and modifications from the condition assessed in the 2011 Report were consolidated. At the time of removal of spent fuel, the debris and machinery of the upper part of the refueling floor have been removed, and the installation of a support structure for the spent fuel pool base and a frame supporting the fuel handling machinery, as well as improvement of the yard and other aspects have been implemented. In this assessment, a seismic safety assessment will be conducted reflecting these modifications.

(Attachment-1)

3. Condition of Damage to the Reactor Building Frame

Based on the investigation results on the cause of the hydrogen explosion inside the reactor building, as indicated in the “Fukushima Nuclear Accidents Investigation Report” (Tokyo Electric Power Company, June 2012), a visual inspection of the floor and walls which affect seismic safety was conducted, and the specific conditions of damage to the reactor building frame were investigated. The degree of damage was classified and consolidated into three grades (no damage, partial damage, total collapse) for each span. In this assessment, a seismic safety assessment will be conducted reflecting the results of the investigation of such damage conditions.

(Attachment-2)

4. Results of the Seismic Safety Assessment of Reactor Building (Mass System Model-Based Analysis)

Based on the mass system model used in the 2011 Report, a model was prepared which disregards the rigidity of locations where partial bulging was confirmed in the exterior walls in the "Report on the Seismic Safety of Unit 4 Reactor Building at Fukushima Daiichi Nuclear Power Station in Consideration of the Partial Expansion of the Exterior Wall" (Tokyo Electric Power Company, June 2012) (hereinafter, “June 2012 Report”), and locations where damage was confirmed in numeral 3, and the mass point weight on each floor was increased or decreased based on numeral 2, and a time history response analysis was performed. The results of the analysis were almost the same as the case of the model in the 2011 Report for shear strain occurring in the earthquake-resisting wall, and no significant difference resulted. Also, the shear strain which occurred in the earthquake-resisting wall was 0.16×10^{-3} at a maximum, which is significantly below the assessment reference value of 4.0×10^{-3}. Based on these results, it was assessed that the reactor building provides sufficient seismic safety even in the estimated conditions during spent fuel removal.

(Attachment-3)

5. Results of the Seismic Safety Assessment of Spent Fuel Pool (Three-Dimensional FEM Analysis)

Based on the three-dimensional FEM model used in the 2011 Report, a model was prepared which disregards the rigidity of locations where partial bulging was confirmed in the exterior walls in the June 2012 Report and locations where damage was confirmed in numeral 3, and the support structure and other improvements to the spent fuel pool base and changes in weight based on numeral 2 are reflected, and stress analysis was performed. The results of the analysis showed that strain on reinforcing steel in the spent fuel pool was 1180×10^{-6} at maximum, and antiplane shearing force was 1120 (N/mm) even at areas having the least allowance, and since there was sufficient allowance in relation to the assessment reference values of 5000×10^{-6} and 1860 (N/mm), it was assessed that the spent fuel pool provides sufficient seismic safety even in the estimated conditions during spent fuel removal.

(Attachment-4)

6. Summary

For the purpose of assessing seismic safety of the Unit 4 reactor building and spent fuel pool estimating the time of spent fuel removal, in this report, modifications from the conditions assessed in the 2011 Report were consolidated, investigation was conducted into the specific conditions of damage to the building frame, and a mass system model-based analysis of the reactor building and a three-dimensional FEM analysis of the spent fuel pool were performed. As a result, it was verified that the reactor building and spent fuel pool provide sufficient seismic safety.

Attachment-1:
Specific Details of the Condition of Reactor Building at the Time of Spent Fuel Removal

1. Introduction

At the Unit 4 Reactor Building of Fukushima Daiichi Nuclear Power Station, debris has been removed and a support structure for the spent fuel pool has been installed, and the circumstances pertaining to the load of each part and other aspects have changed from the conditions when seismic safety was assessed in the "Reports about the study regarding current seismic safety and reinforcement of reactor buildings at Fukushima Daiichi Nuclear Power Station (1)" (Tokyo Electric Power Company), released on May 28, 2011 (hereinafter, "2011 Report"). Furthermore, a frame to support the fuel handling machinery is planned to be newly installed on the reactor building when spent fuel is removed in the future. Here, modifications have been consolidated from the conditions assessed in the 2011 Report regarding the reactor building at the time of spent fuel removal, and reflected in an assessment of seismic safety.

2. Points Modified Concerning the Condition of the Reactor Building at the Time of Spent Fuel Removal

Points modified from the conditions assessed in the 2011 Report for the time of spent fuel removal are shown in Table-1.2.1, and the implementation schedule for points modified is given in Figure-1.2.1. Furthermore, specifics on each of the modified points are provided in the next few pages.

Table-1.2.1 Points Modified from Conditions Assessed in 2011 Report for the Time of Spent Fuel Removal

No	Point modified	Description
1	Removal of debris from upper part of RF[*1]	Removal of debris from the collapse of the R and CR floors
2	Removal of machinery from RF[*1]	Removal of machinery on the RF[*1]
3	Installation of frame supporting fuel handling equipment	Installation of frame to support the fuel handling equipment
4	Installation of structure as measure to counter infiltration of rain water	Installation of structure to serve as a countermeasure to rain infiltration for the scope uncovered by the fuel removal cover
5	Installation of support structure for spent fuel pool base	Installation of support structure (steel support columns, concrete) on spent fuel pool base
6	Implementation of yard improvements	Place a covering of 1m of soil on the annex attached to main building on the west side in order to improve the yard
7	Consideration of underground water accumulation	Consideration of the presence of water accumulating from the B1 level to the MB1 level (at the time of the 2011 Report, the water level was not ascertained and not taken into consideration in the assessment)
8	Commencement of circulative cooling of spent fuel pool	Lowering of water temperature by commencing circulative cooling of spent fuel pool (control temperature 65 C)

*1: RF = refueling floor.

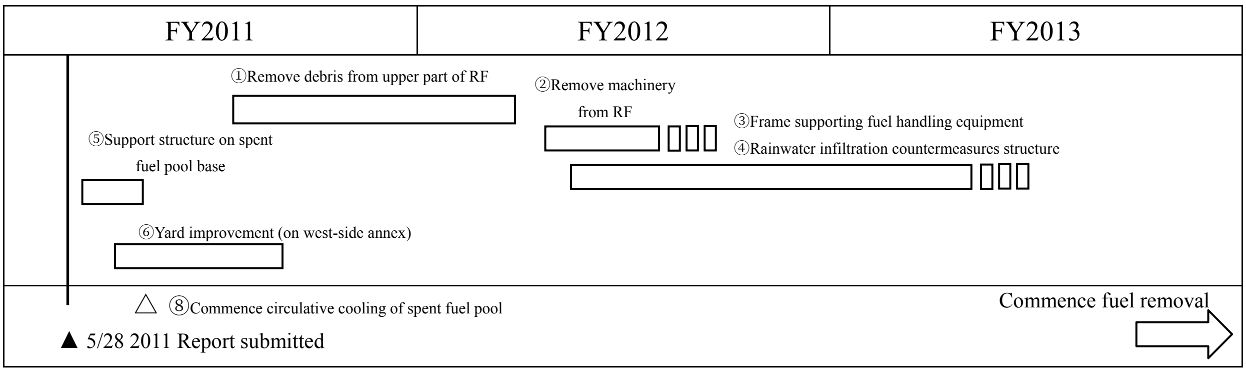
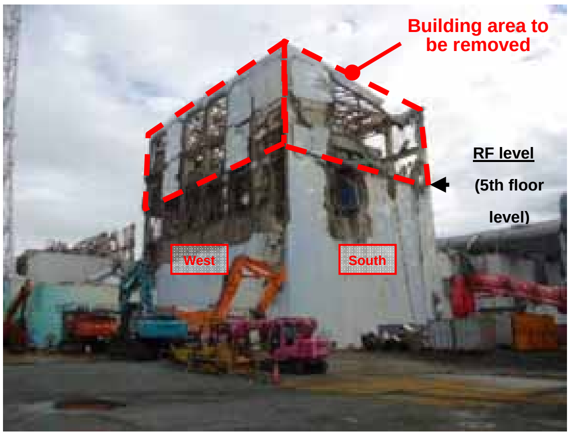


Figure-1.2.1 Schedule for Implementation of Modified Points

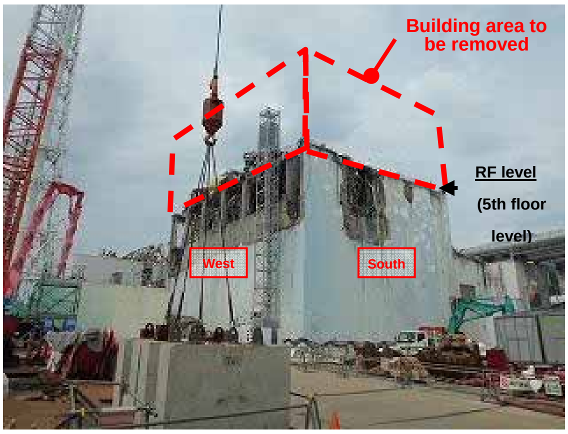
3. Removal of Machinery and Debris of Upper Part of Operating Floor (RF)

Progress in removing debris above the RF is shown in Figure-1.3.1, and progress in removing machinery from the RF in Figure-1.3.2. At the stage of the 2011 Report, debris above the refueling floor and the weight of machinery were taken into account, but with regard to the debris, the removal work which started in late November 2011 was completed in early July 2012. Also with regard to the machinery, the removal of large machinery (head of primary containment vessel and head of the reactor pressure vessel) etc., was commenced in late July 2012, and is scheduled to be completed in October 2012. Therefore, in the assessment estimating the time of spent fuel removal, weight reduction due to removal of such debris and machinery is taken into account.



(a) Prior to commencing debris removal work;

Photo: Sept. 22, 2011



(b) After completion of debris removal work;

Photo: July 5, 2012

Figure-1.3.1 Progress of Debris Removal from Upper Part of RF (Southwest-side)

(a) Machinery on upper part of RF

Photo: July 9, 2012

(b) Removal of machinery

Photo: August 10, 2012

Figure-1.3.2 Progress of Machinery Removal from RF (West-side View)

4. Installation of Structure Supporting Fuel Handling Equipment

A north-south cross-sectional view of the frame supporting fuel handling equipment is shown in Figure-1.4.1, and a beam plan is given in Figure-1.4.2. At the time when spent fuel is to be removed, a fuel removal cover will be installed over the reactor building in a configuration covering the spent fuel pool. The fuel removal cover has a frame supporting a crane and a frame for supporting fuel handling equipment, and of these two frames, a structure is in place so that the weight of the frame supporting fuel handling equipment is supported by the south-side exterior walls from the 1st level to the 2nd level, as well as the upper ends of the reactor building shell walls; hence this weight increase is taken into consideration.

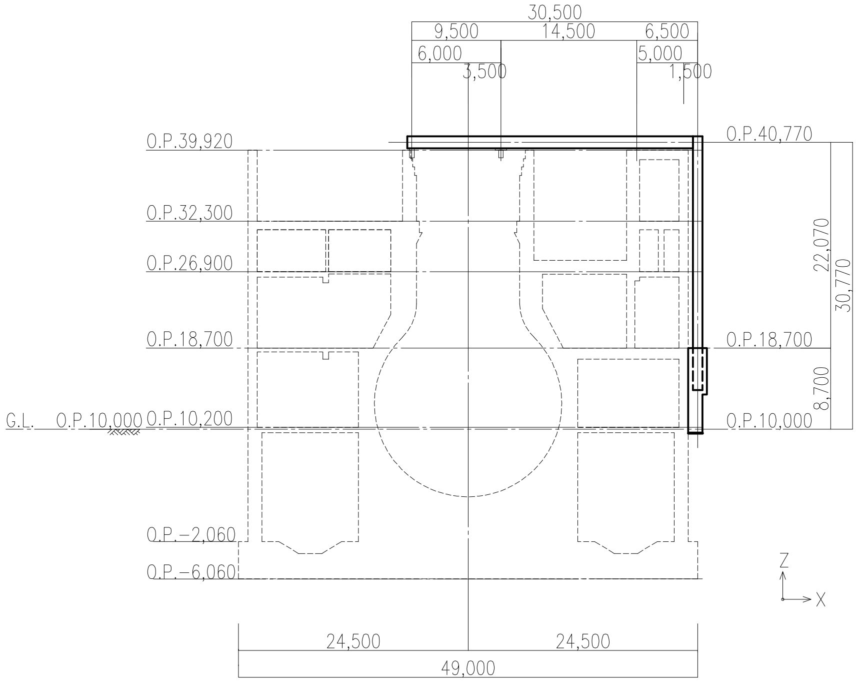


Figure-1.4.1. North-South Cross-Sectional View of Frame Supporting Fuel Handling Equipment

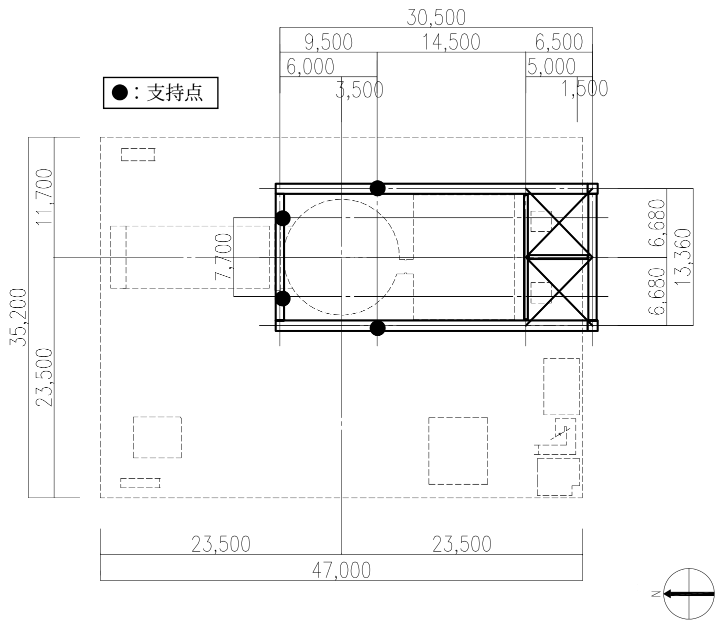


Figure-1.4.2. Beam Plan of Frame Supporting Fuel Handling Equipment (O.P.41,420)

5. Installation of Structure as Measure to Counter Infiltration of Rainwater

In Figure-1.5.1, an illustration of the rainwater infiltration countermeasures structure is shown. When spent fuel is removed, the rainwater infiltration countermeasures structure is to be installed over an area uncovered by the fuel removal cover above the reactor building, and this weight increase is taken into consideration.

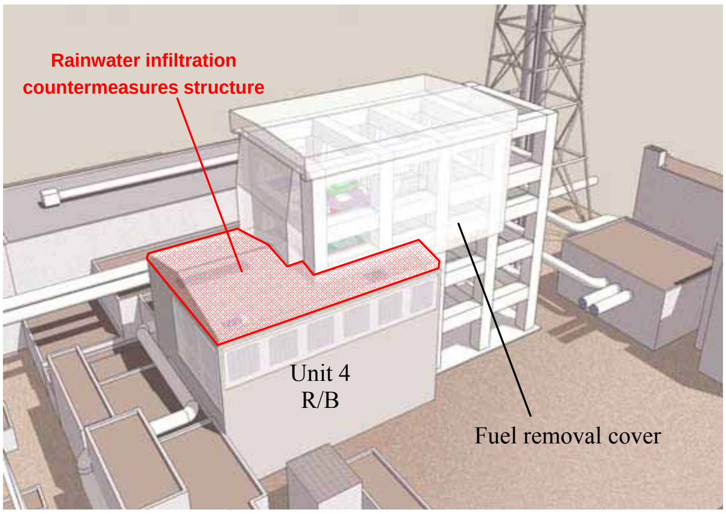
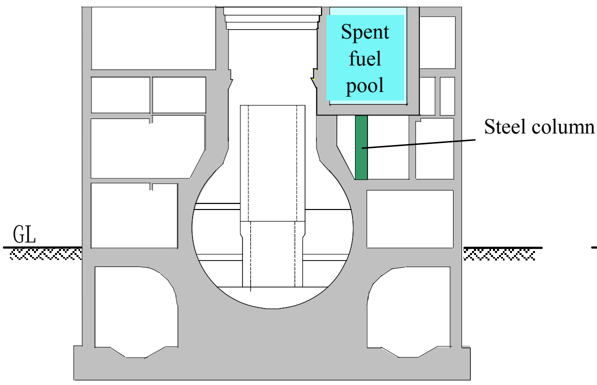


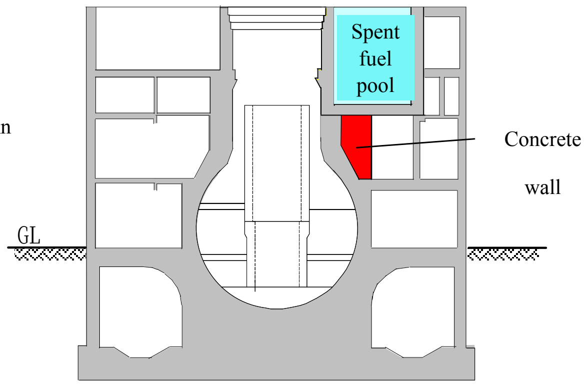
Figure-1.5.1 Illustration of Structure as Measure to Counter Infiltration of Rainwater

6. Installation of Support Structure for Spent Fuel Pool Base

In Figure-1.6.1, an illustration of the support structure for the spent fuel pool base is shown. The support structure was installed on the pool base on July 30, 2011 to improve the safety allowance of the spent fuel pool. The support structure is a structure solidified by concrete around the circumference of the steel support columns, and has the effect of reducing imposed load on the spent fuel pool. As a consequence of this, the effect of the weight increase and the support structure are taken into consideration.



(a) Prior to pouring concrete



b)After pouring concrete (after completion)

Figure-1.6.1 Illustration of Support Structure for Spent Fuel Pool Base (North-South Cross-Sectional View)

7. Implementation of Yard Improvements

In Figure-1.7.1, an illustration is shown of the yard improvements. When spent fuel is removed, the west-side annex has been covered with 1m of soil in order to improve the yard; hence this weight increase is taken into consideration.

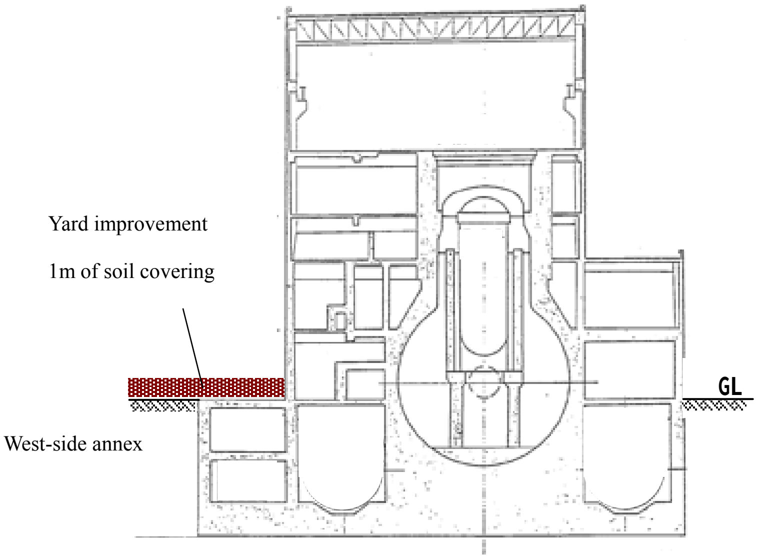


Figure-1.7.1 Illustration of Yard Improvements (East-West Cross Sectional View)

8. Consideration of Underground Water Accumulation

In Figure-1.8.1, an illustration of underground water accumulation is shown. There is water which accumulated underground from the B1 level to the MB1 level of the reactor building, and the water level is managed using a limiting value of O.P.3.5m. As of the time of the 2011 Report, the level of the accumulated water could not be ascertained and was not taken into consideration in the assessment; therefore this time it is newly taken into account.

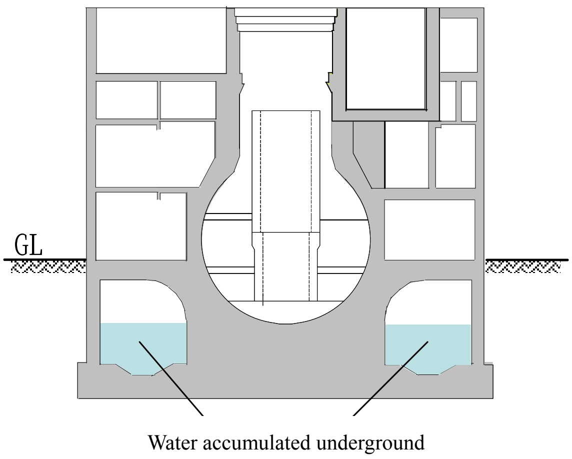


Figure-1.8.1 Illustration of Underground Accumulated Water (North-South Cross-Sectional View)

Attachment-2: Specific Details of the Condition of Damage to Reactor Building Frame

1. Introduction

With regard to the hydrogen explosion which occurred at the Unit 4 reactor building of Fukushima Daiichi Nuclear Power Station, an investigation into the cause and the confirmed results is given in the Fukushima Nuclear Accident Investigation Report (Tokyo Electric Power Company, June 2012) (hereinafter, "Accident Investigation Report"), and the cause has been inferred to have been due to wraparound of the vent flow which contained hydrogen gas from Unit 3, and then flowed into locations of the building through the ducts and stand-by gas treatment system pipes from the second level of the Unit 4 hydrogen building. Here, taking into account the results of these investigations, the specific details of the damage to the reactor building frame have been consolidated based on visual inspections conducted of the walls and floors affecting seismic safety.

2. Summary of Accident Investigation Report

The investigation results on the condition of damage inside the Unit 4 reactor building as shown in the Accident Investigation Report are shown in Figure-2.2.1 to Figure-2.2.3.

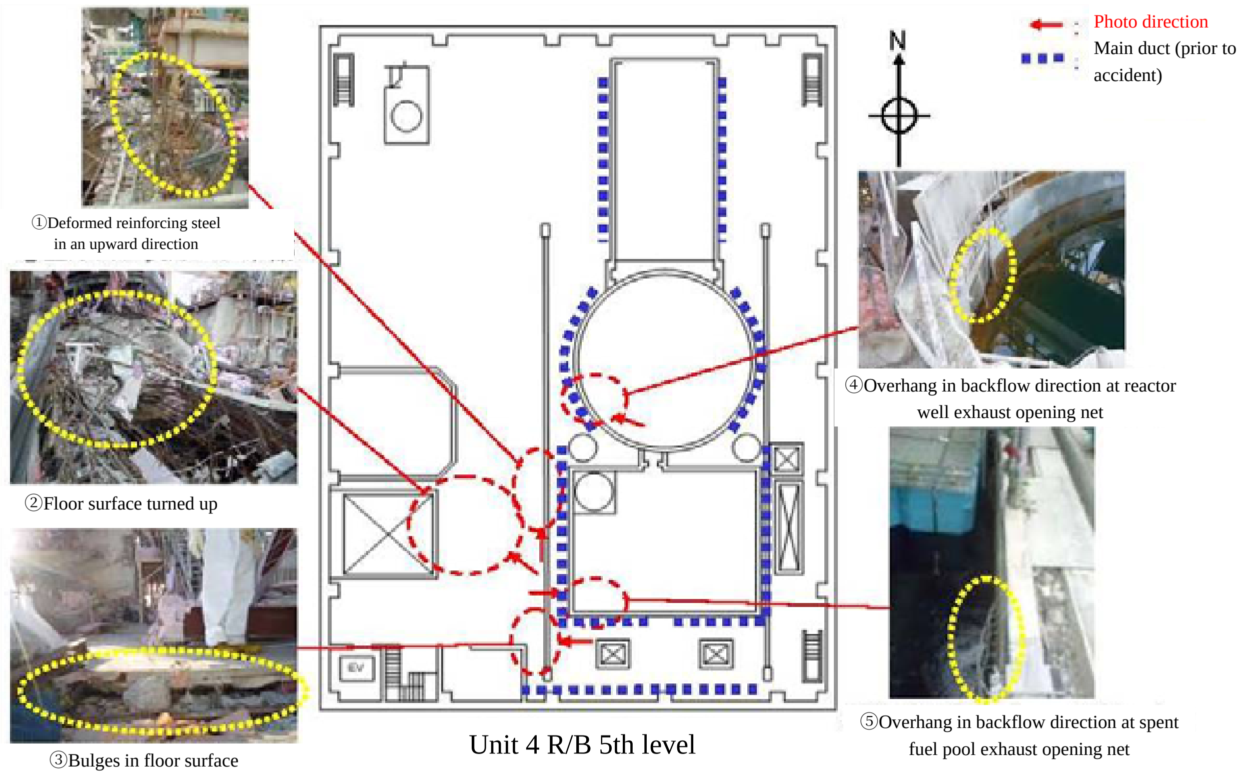


Figure-2.2.1 Investigation Results of Conditions of Damage inside Building (5th level)

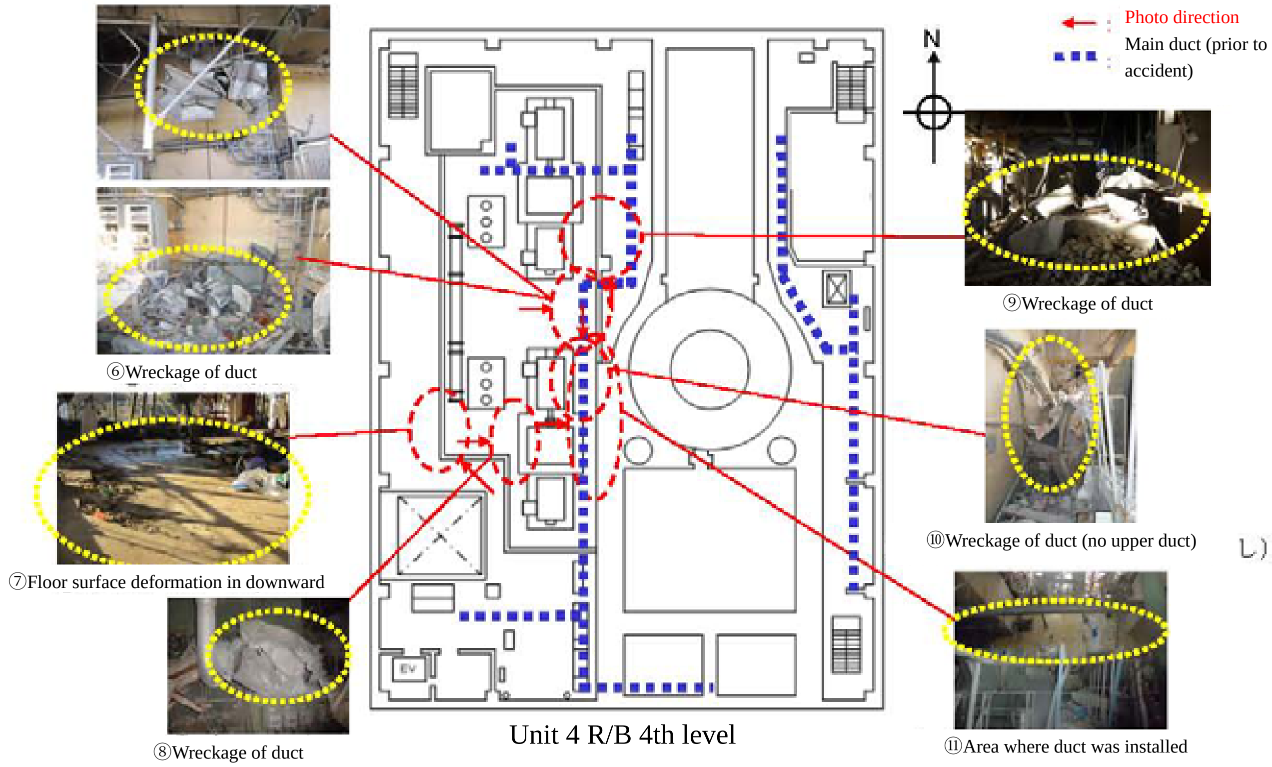


Figure-2.2.2 Investigation Results of Conditions of Damage inside Building (4th level)

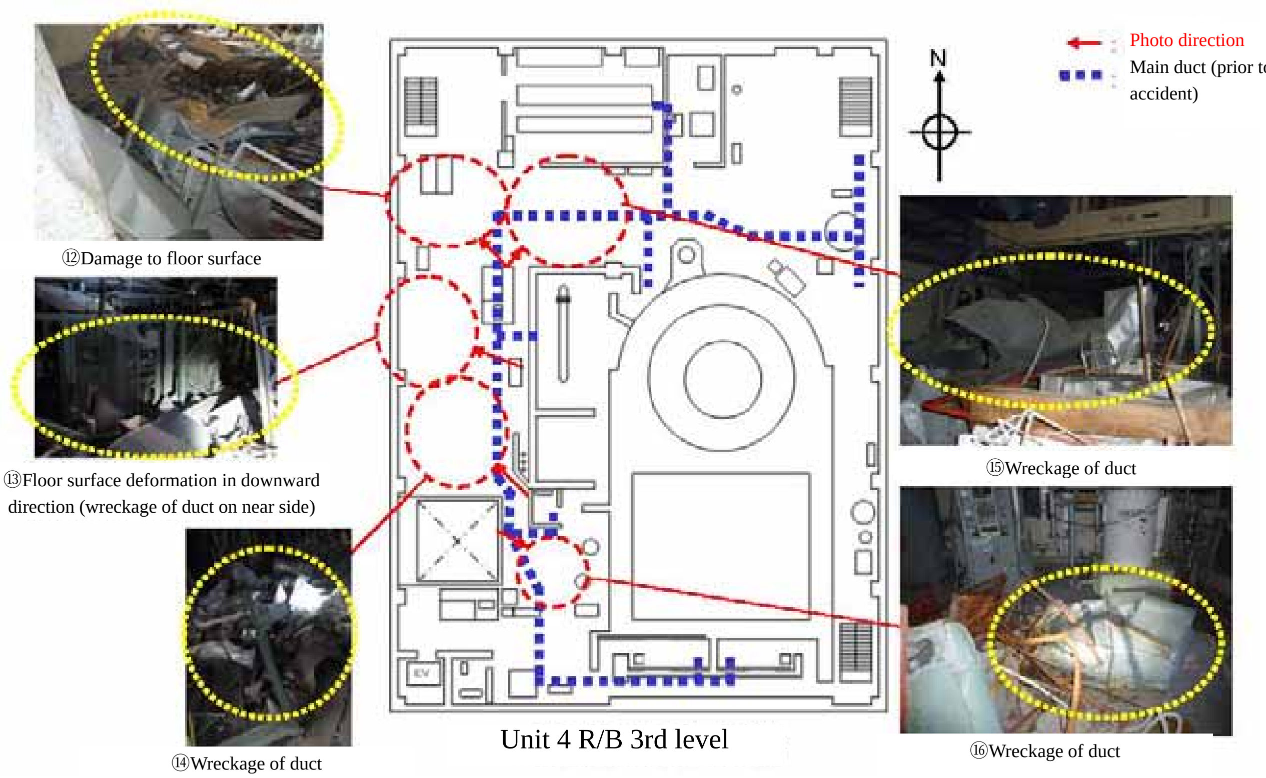


Figure-2.2.3 Investigation Results of Conditions of Damage inside Building (3rd level)

3. Consolidation of Condition of Damage to Reactor Building Frame

Taking into account the investigation results of the Accident Investigation Report, the specific details of the damage to the reactor building frame have been consolidated based on visual inspections conducted of the walls and floors affecting seismic safety. For each span, the degree of damage was classified into three grades (no damage, partial damage, total collapse). The conditions of the damage to each reactor building levels are shown in Figure-2.3.1 to Figure-2.3.8. Locations where partial bulging was confirmed in exterior walls in the "Report on the Seismic Safety of Unit 4 Reactor Building at Fukushima Daiichi Nuclear Power Station in Consideration of the Partial Expansion of the Exterior Wall" (Tokyo Electric Power Company, June 2012) (hereinafter, “June 2012 Report”) were classified as partial damage.

Of the damaged areas, locations where repairs are desirable from the standpoint of human safety or durability of components are scheduled to be repaired to the extent possible.

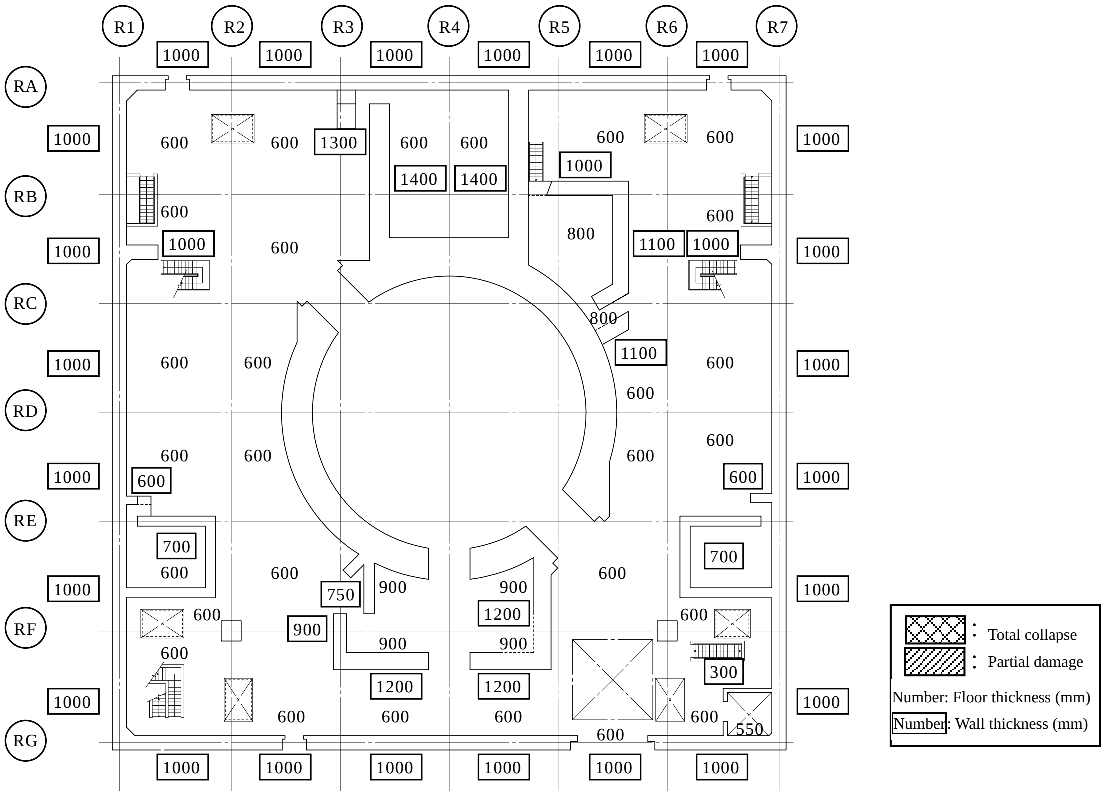


Figure-2.3.1 Damage Conditions (1st Level: No Damage)

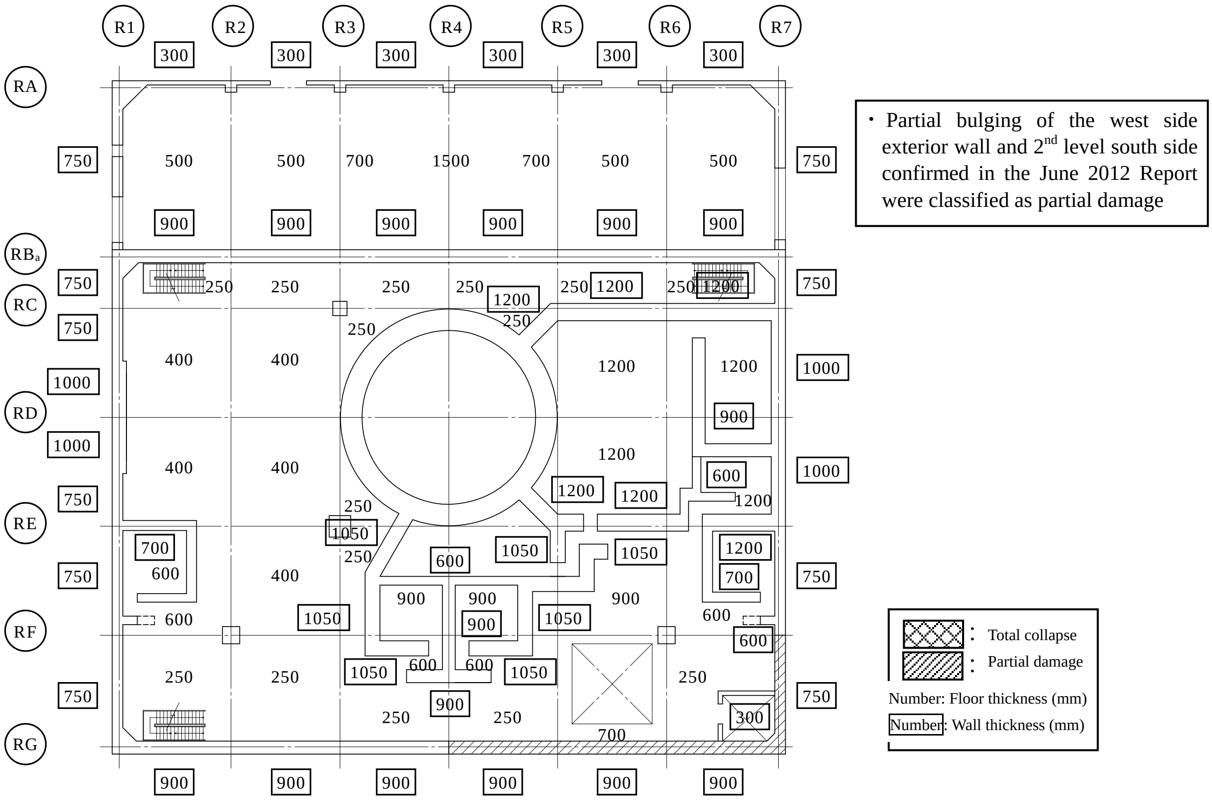
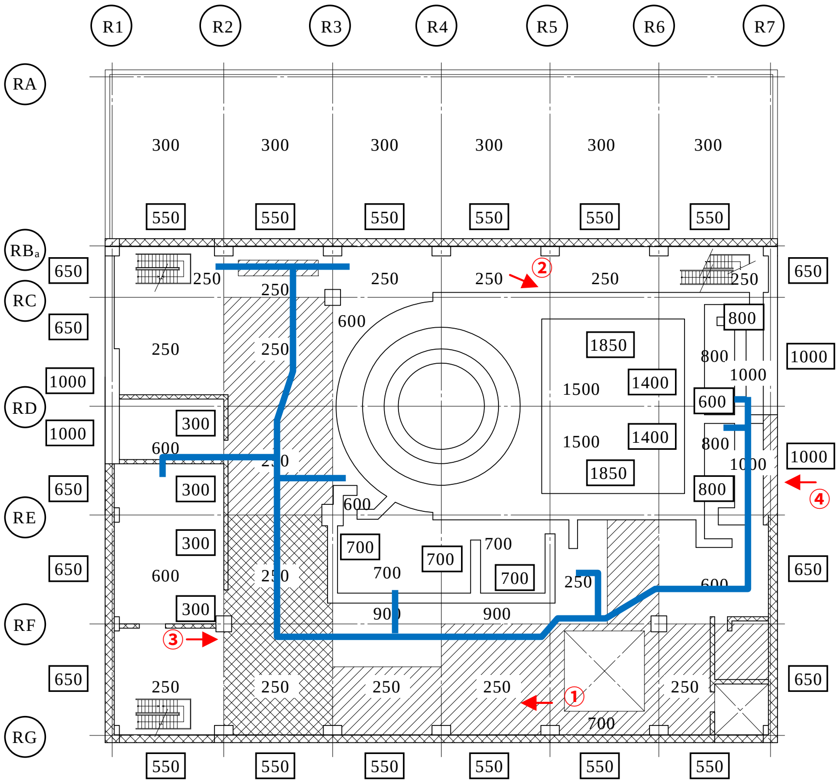


Figure-2.3.2 Damage Conditions (2nd Level)



- Locations of damage to floor have been confirmed near the duct route, which is thought to be the location where the hydrogen explosion occurred.
- With regard to the floor between R6-R7 and RC-RF, the floor thickness is 600mm or more in contrast to a floor thickness of 250mm of the damaged floor, and the walls of the upper and lower floors hindered deformation in an out-of-plane deformation, hence it is thought to have avoided damage while being above the duct route.
- With regard to locations of damage to walls, damage was severe to walls having a thickness of less than 650mm, and there was little damage to walls having a thickness of over 650mm

Figure-2.3.3 Damage Conditions (3rd Level)

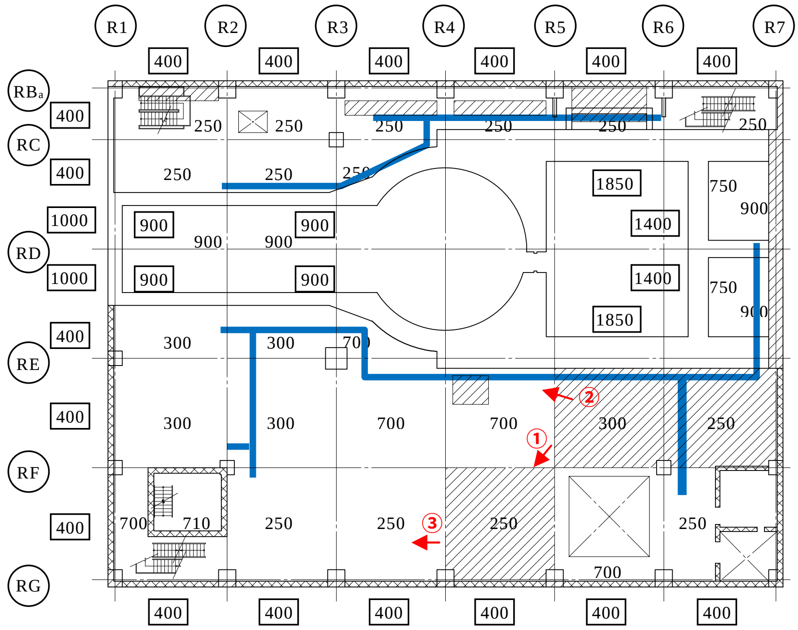
①Floor deformation

②No pool wall abnormalities

③Floor collapsed

④Partial flaking of exterior wall

Figure-2.3.4 Photographs of Damage Condition (3rd Level)



- With regard to locations of damage to floor, it is thought that locations where there was a route for the pressure to escape such as the large delivery entrance, EV shafts and stairway openings as well as those having a floor thickness of 700mm escaped damage.
- With regard to locations of damage to walls, damage was severe to walls having a thickness of less than 400mm, and there was little damage to walls having a thickness of over 400mm
- The reason why a tendency has been seen to have the damage to walls be more severe than that to floors is thought that walls do not constrain in comparison to floors which constrain deformation of out-of-plane bending by partition walls, etc.

: Total collapse
: Partial damage
Number: Floor thickness (mm)
Number: Wall thickness (mm)
: Main duct

Figure-2.3.5 Damage Conditions (4th Level)

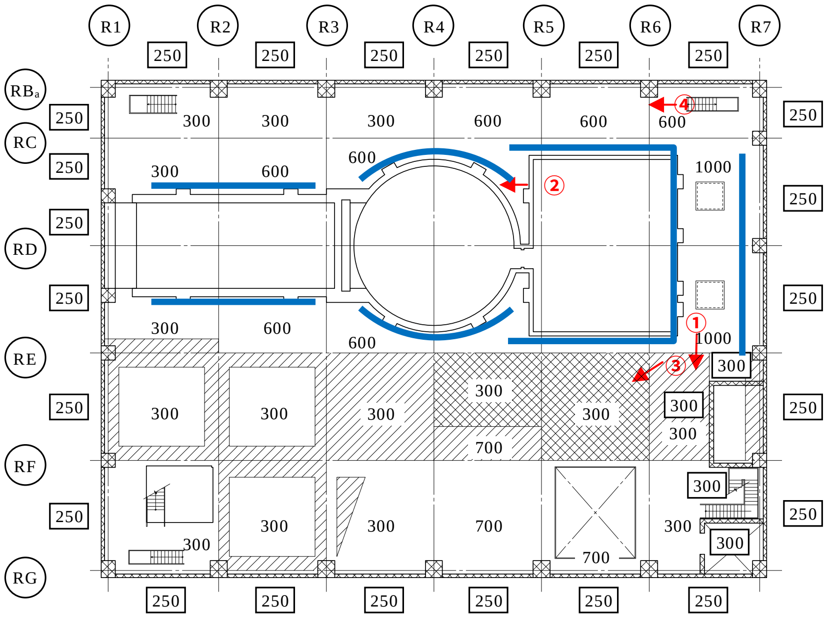
①Floor deformation

②No pool wall abnormalities

③No floor abnormalities

④Partial flaking of exterior wall

Figure-2.3.6 Photographs of Damage Condition (4th Level)



・Locations of damage to floor were impacted by ducts of the lower level, which are thought to be the location where the hydrogen explosion occurred. It is thought that locations where there was a route for the pressure to escape such as the large delivery entrance, EV shafts and stairway openings as well as locations having a floor thickness of 700mm escaped damage.

・It is thought that the tendency of the damage to walls being more severe than that to floors is thought that walls do not constrain in comparison to floors, which constrain deformation of out-of-plane bending by partition walls, etc.

: Total collapse
: Partial damage
Number: Floor thickness (mm)
$\boxed{\text{Number}}$: Wall thickness (mm)
: Main duct

Figure-2.3.7 Damage Conditions (5th Level)

①Floor deformation

②No shell wall abnormalities

③Floor collapsed

④No floor abnormalities

Figure-2.3.8 Photographs of Damage Condition (5th Level)

Attachment-3:
Specific Details on the Results of the Seismic Safety Assessment of Reactor Building (Mass System Model-Based Analysis)

1. Policy of Analysis and Assessment

In the current examination, the mass point weights were configured based on the condition of the reactor building at the time of spent fuel removal as compiled in Attachment-1. At the same time, a model for seismic response analysis which disregards the areas confirmed to have damage (partial damage or total collapse) in Attachement-2 and the rigidity of areas where partial bulging of the exterior walls was confirmed in “Report on the Seismic Safety of Unit 4 Reactor Building at Fukushima Daiichi Nuclear Power Station in Consideration of the Partial Expansion of the Exterior Wall” (Tokyo Electric Power Company, June 2012) (hereinafter, “June 2012 Report”) was prepared, and an assessment was conducted using time history response analysis of the seismic safety during reference seismic motion with regard to the reactor building.

The input seismic motion was determined to be reference seismic motion Ss-1 and Ss-2, and with regard to reference seismic motion Ss-3, since the response based on previous calculation examples was clearly small, it was decided to be omitted, just as was done in the “Reports about the study regarding current seismic safety and reinforcement of reactor buildings at Fukushima Daiichi Nuclear Power Station (1)” (Tokyo Electric Power Company, May 2011) (hereinafter, “2011 Report”).

The seismic response analytical model takes interaction with the ground into consideration, and is a mass system model that takes into account bending and sheer stiffness.

From the standpoint of preventing a ripple effect on equipment important for seismic safety, the seismic safety assessment of the reactor building was performed using a comparison of shear strain of the earthquake-resisting walls obtained from the seismic response analysis and the assessment reference value (4.0×10^{-3}), corresponding to the ultimate limit of an earthquake-resisting wall with a reinforced-concrete structure.

In regard to the ultimate limit of an earthquake-resisting wall having a reinforced-concrete structure, since the horizontal seismic force is dominant while the effect of vertical seismic force is small, the seismic response analysis focused only on the horizontal direction. A flow chart of the seismic safety assessment is shown in Figure-3.1.1.

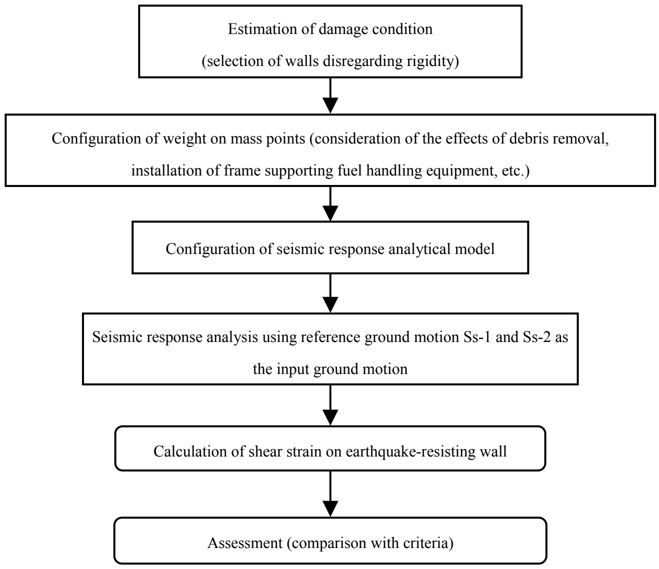


Figure-3.1.1 Flowchart of Seismic Safety Assessment of Reactor Building

2. Estimation of Damage Conditions

In estimating the damage conditions, a new model for seismic response analysis based on the seismic response analytical model prepared in the 2011 Report is constructed, disregarding the areas confirmed to have damage (partial damage or total collapse) in Attachement-2 and the rigidity of areas where partial bulging of the exterior walls was confirmed in the June 2012 Report. The exterior walls disregarding rigidity are shown in Figure-3.2.1.

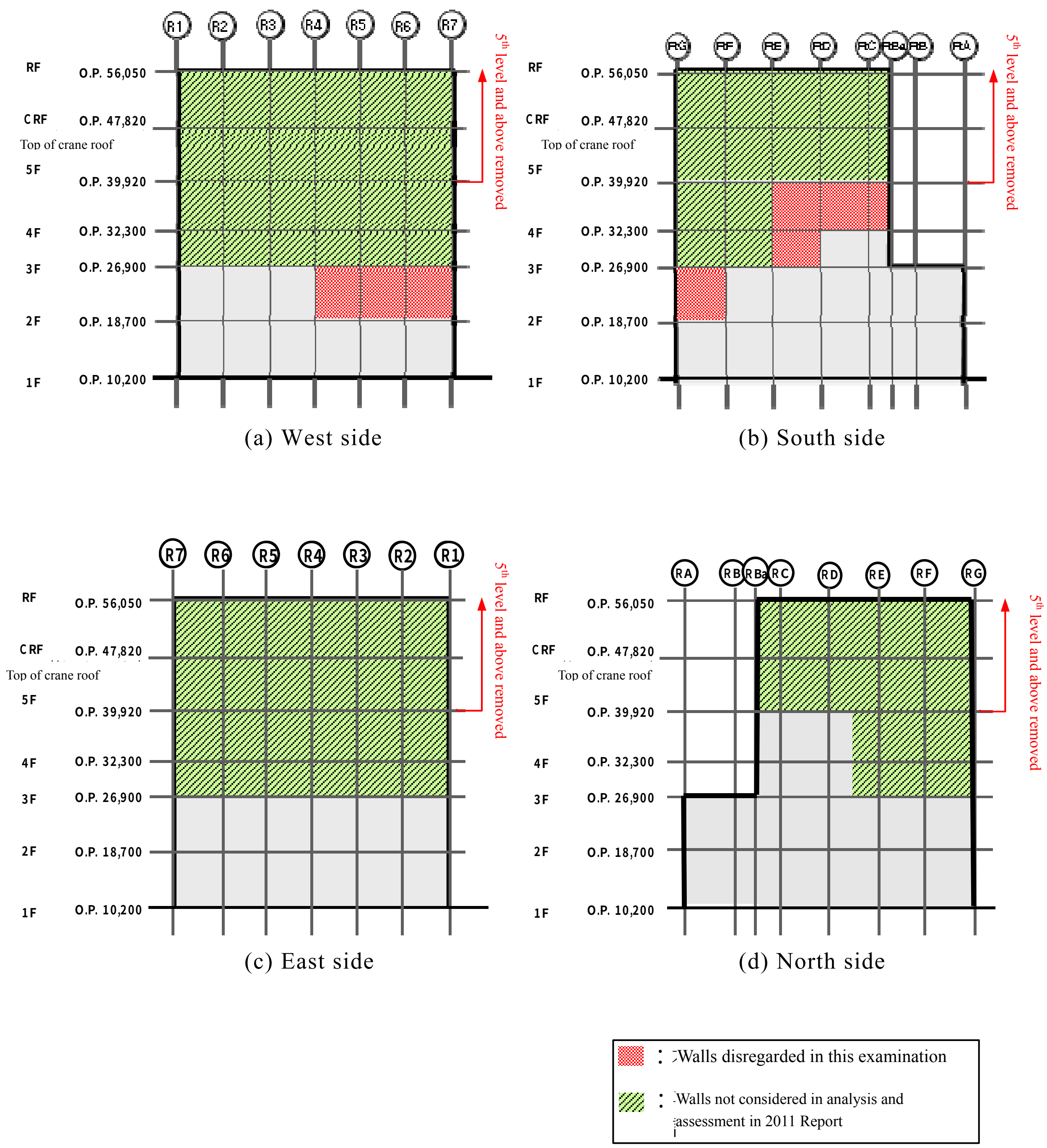


Figure-3.2.1 Exterior walls disregarding rigidity

3. Configuration of Mass Point Weight

In configuring the weight at mass points, the mass point weight of the model in the 2011 Report served as the standard and the weight was increased or decreased to reflect modifications and other changes from the conditions assessed in the 2011 Report, at the time of spent fuel removal consolidated in Attachment-1. The basis for configuration of the mass point weights is shown in Table-3.3.1 and the results of calculations of mass point weights in the current examination model and the increase or decrease in weight from the model in the 2011 Report are shown in Table-3.3.2.

Table-3.3.1 Basis for Configuration of Mass Point Weights

No	Assessment item	Assessment method
1	Removal of debris from the upper part of the RF*1	Assessment of weight decrease due to removal of debris collapsed from the R and CR levels
2	Removal of machinery from the upper part of the RF*1	Assessment of weight decrease due to removal of machinery from RF*1
3	Installation of frame supporting fuel handling equipment	Assessment of weight increase due to installation of frame supporting fuel handling equipment and apparatuses inside frame
4	Installation of structure as measure to counter infiltration of rainwater	Assessment of weight increase due to installation of structure as measure to counter infiltration of rainwater
5	Installation of support structure for spent fuel pool base	Assessment of weight increase due to support structure for spent fuel pool base (steel support columns, concrete)
6	Implementation of yard improvements	Assessment of weight increase due to covering of 1m of soil placed on the annex of the west side to improve the yard
7	Consideration of underground water accumulation	Assessment of water accumulating from the B1 level to the MB1 level as an increase of weight*2 (At the time of the 2011 Report, the water level was not ascertained and not taken into consideration in the assessment)
8	Existing building frame (debris)	Weight of exterior walls which collapsed at upper part of RF*1 ・The east-side exterior wall which collapsed from the R and CR levels assessed as an increase in weight assuming it present on the 3rd level annex.
		Weight of exterior walls which collapsed below the RF*1 ・The exterior walls which collapsed in almost all areas assessed as a weight decrease ・East-side exterior wall which collapsed assessed as an increase in weight as it is present on the 3rd level annex.
		Weight of floor which collapsed ・The floors which collapsed in almost all areas assessed as a weight decrease ・The floors which collapsed assessed as a weight increase as they have not been able to be removed and are just as when they fell
		Weight of floor which has flaked on the backside ・Covering of 100mm assessed as a weight decrease as it has fallen to a lower level. ・Flaking covering assessed as a weight increase as they have not been able to be removed and are just as when they fell to a lower level
		Weight of debris in temporary equipment storage pool ・A state is assumed where debris settles inside the pool, which is assessed as weight increase on the assumption that debris having a thickness of 200mm per horizontal projection area of the temporary equipment storage pool is present

*1：RF = refueling floor.

*2：The level of accumulated water is managed by using O.P.3.5m as the limiting value, but the weight was calculated by considering the water to be accumulated up to O.P.4.0m, thus being treated conservatively.

Table-3.3.2 Calculation Results of Increase or Decrease in Weight from 2011 Report and Mass Point Weight in the Current Examination Model

Unit: kN

Mass point no	Level	Elevation O.P. (m)	2011 Report model	Increase/decrease in weight from 2011 Report model (1) Removal of debris from RF upper part	(2) Removal of RF machinery	(3) Frame supporting fuel handling equipment	(4) Rainwater infiltration countermeasure structure	(5) SFP base support structure	(6) Yard improvement	(7) Underground water accumulation	(8) Existing building frame	Current examination model
1	RF	56.05	0	0	0	0	0	0	0	0	0	0
2	CRF	47.82	0	0	0	0	0	0	0	0	0	0
3	5F	39.92	114,850	-35,820	-9,690	2,660	1,380	0	0	0	-3,440	69,940
4	4F	32.3	88,770	0	0	0	0	0	0	0	-1,630	87,140
5	3F	26.9	117,030	0	0	0	0	5,180	0	0	5,550	127,760
6	2F	18.7	121,930	0	0	2,170	0	3,600	0	0	1,330	129,030
7	1F	10.2	207,300	0	0	1,660	0	0	9,520	0	0	218,480
8	B1F	-2.06	287,050	0	0	0	0	0	0	66,690	0	353,740
9	MAT	-6.06	132,390	0	0	0	0	0	0	0	0	132,390
Total			1,069,320	-35,820	-9,690	6,490	1,380	8,780	9,520	66,690	1,810	1,118,480

4. Input Seismic Motion Used for Analysis

As for the input seismic motion of the Unit 4 reactor building, it was decided to use the reference seismic motion Ss-1 and Ss-2 assumed in the free rock surface level of base stratum and prepared in the "Interim Report on Seismic Safety Assessment Results following Revision of the 'Guidelines for Inspection of Seismic Design of Nuclear Power Reactor Facilities' at Fukushima Daiichi Nuclear Power Station" (Nuclear Administration Report to Authorities 19 No. 603 dated March 31, 2008)

A conceptual diagram of the input seismic motion used in the seismic response analysis is shown in Figure-3.4.1. The seismic motion input into the model is based on the one-dimensional wave theory and is assessed as the response of the ground to reference seismic motion Ss assumed at the free rock surface level. Also, the notch effect of the ground is taken into consideration by adding shear force at the building base surface level to the input seismic motion.

Among these, the acceleration time wave profile of reference seismic motion Ss-1 and Ss-2 at the free rock surface point (O.P. -196.0m) is shown in Figure-3.4.2

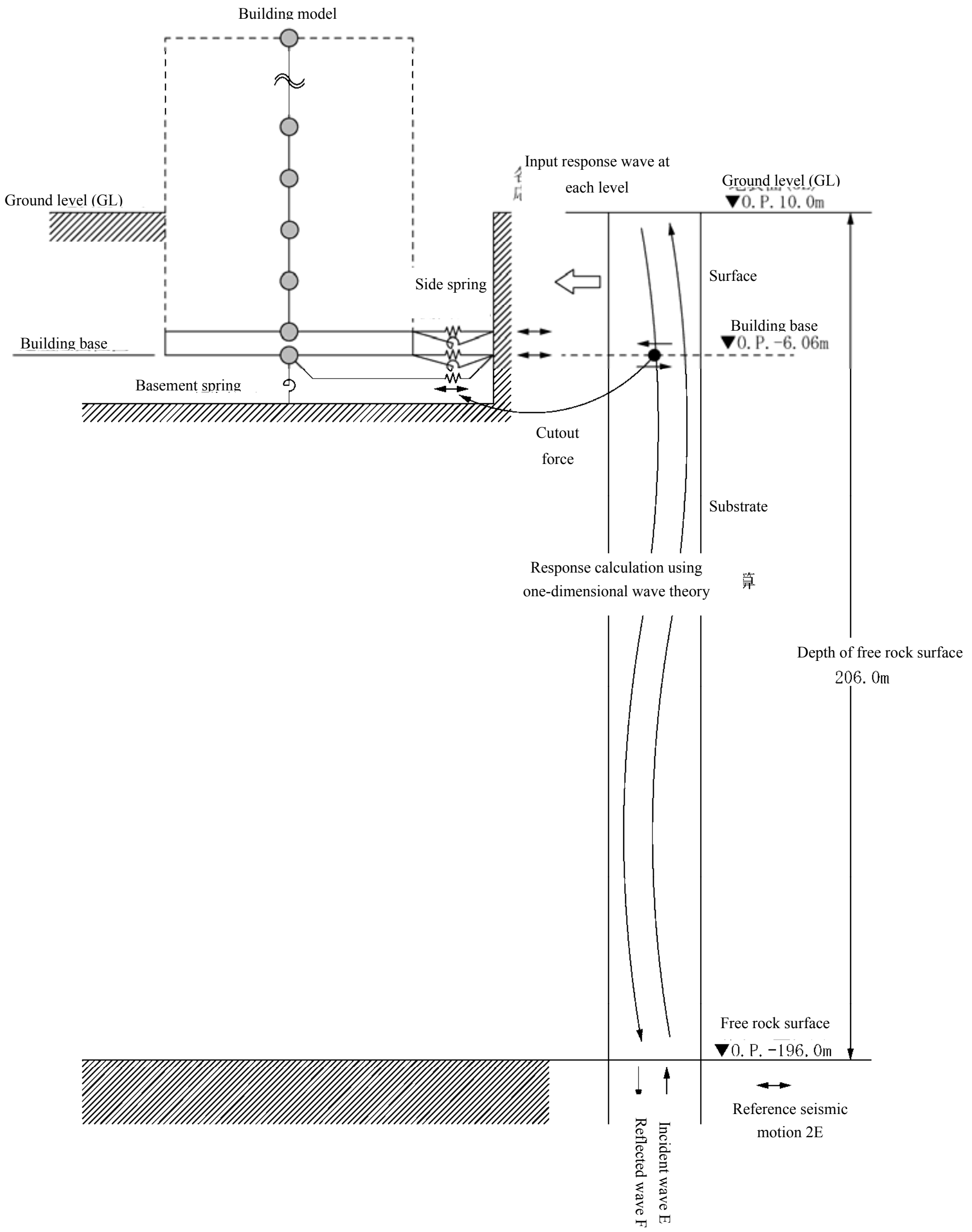
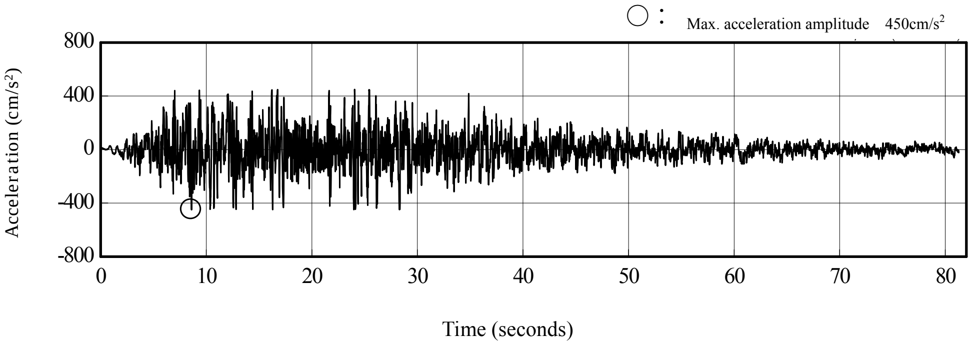
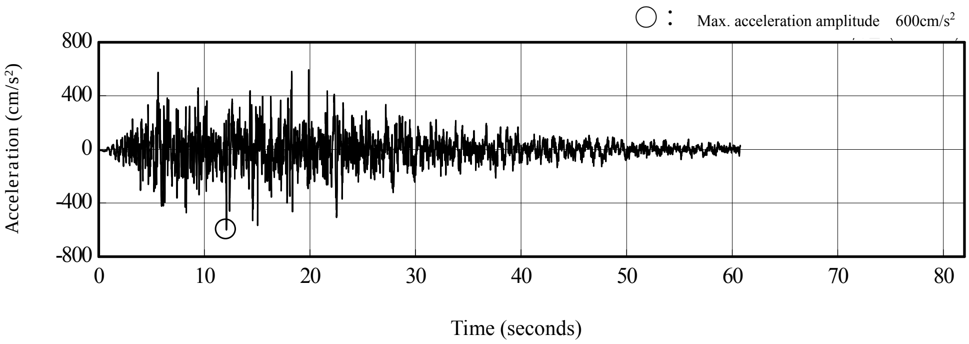


Figure-3.4.1 Conceptual Diagram of Input Seismic Motion Used in Seismic Response Analysis



(Ss-1H)



(Ss-2H)

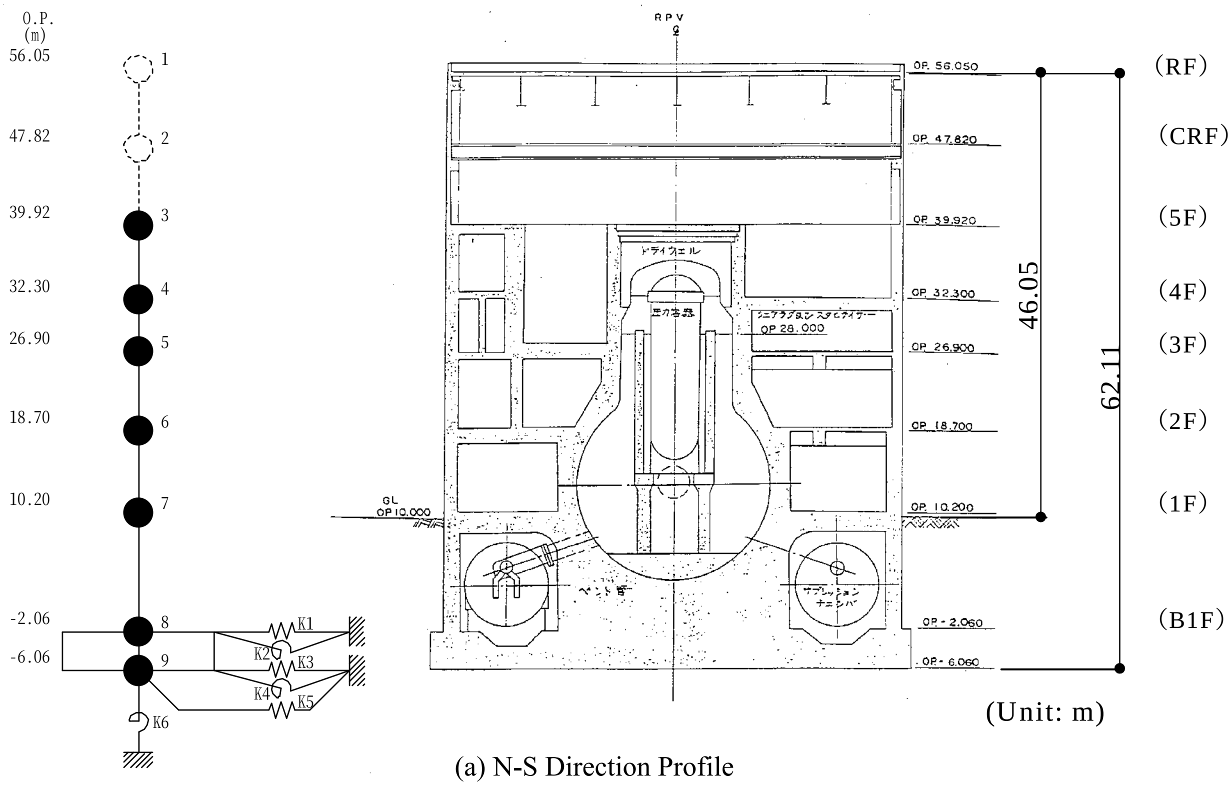
Figure-3.4.2 Time History Acceleration Wave Profile (Horizontal Direction) of Reference Seismic Motion at Location of Free Rock Surface

5. Seismic Response Analytical Model

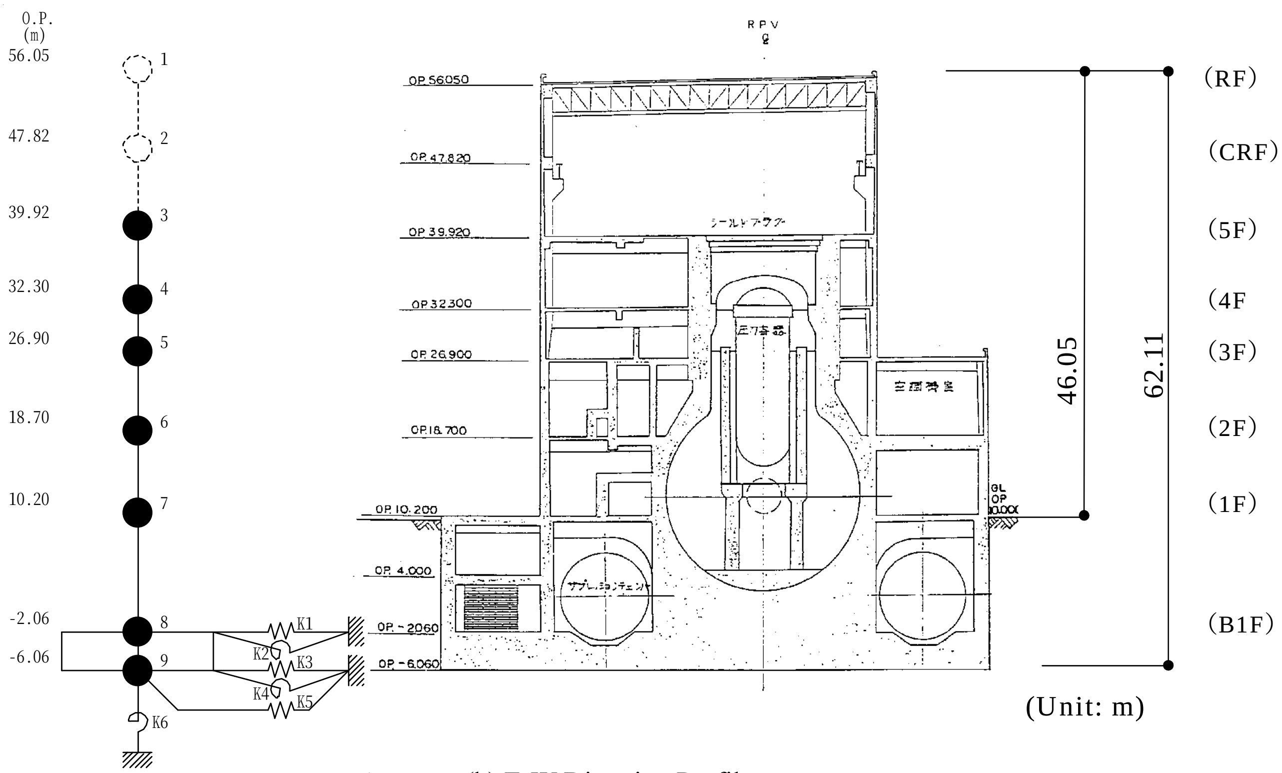
The seismic response analytical model, as shown in Figure-3.5.1, has a mass system making shear deformation and bending deformation of the building and is a building-ground coupled system model which assesses the ground using equivalent springs. The effect of the building-ground coupled system is assessed using ground spring and input seismic motion. The physical properties of reinforced concrete used in the analysis are given in Table-3.5.1 and the specifications of the building analytical model are shown in Table-3.5.2.

The ground constant is hypothesized as the horizontal stratification ground and is determined by taking into account the shear strain level during an earthquake. The ground constant used in the analysis is shown in Table-3.5.3.

In the analytical model for the horizontal direction, with regard to the base surface ground spring, the method indicated in JEAG 4601-1991 was referenced, and after stratification correction was performed, based on vibration admittance, sway and rocking spring constants were approximately assessed. Also, with regard to building-side surface ground spring in the embedded portions, using the ground constant for the building-side surface points and referencing a the method indicated in JEAG 4601-1991 for horizontal and revolving spring, the assessment was conducted using Novak spring-based approximation. Ground spring is obtained as complex stiffness dependent on frequency, but as shown in Figure-3.5.2, static value of the real part as spring constant (Kc) was approximated by applying the inclination of the straight line connecting the origin and value of the imaginary part corresponding to the primary natural frequency of the building-ground coupled system as the attenuation coefficient.



(a) N-S Direction Profile



(b) E-W Direction Profile

Figure-3.5.1 Seismic Response Analytical Model

Table-3.5.1 Physical Properties of Reinforced Concrete Used for Seismic Response Analysis

Concrete	Strength*1 Fc (N/mm^2)	Young coefficient*2 E (N/mm^2)	Shearing 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
Reinforcing steel	SD345 equivalent (SD35)				

*1：For strength, the strength approximating actual conditions (hereinafter, "actual strength") is adopted. The actual strength is configured using the value rounded down of the average value of compressed strength considering random variation in test data from data collected in previous compressed strength tests.

*2：Indicates value based on actual strength.

*3：Indicates value of reinforced concrete.

Table-3.5.2 Specifications of Building Analytical Model

(N-S direction)

Mass point no.	Mass point weight W(kN)	Rotational inertia weight I_G (×10^5kN・m^2)	Shearing cross section As(m^2)	Cross-section secondary momentI (m^4)
1	—	—	—	—
2	—	—	—	—
3	69,940	128.73	147.1	10,080
4	87,140	160.44	102.2	14,387
5	127,760	235.14	202.7	32,567
6	129,030	237.57	175.4	46,774
7	218,480	402.18	460.4	114,194
8	353,740	707.83	2,812.6	562,754
9	132,390	264.88		
合計	1,118,480			

Young coefficient Ec	2.57×10^7(kN/m2)
Shearing elastic coefficient G	1.07×10^7(kN/m2)
Poisson's ratio ν	0.20
Attenuation h	5%
Basic configuration	49.0m (N-S direction) X 57.4m (E-W direction)

(E-W direction)

Mass point no.	Mass point weight W(kN)	Rotational inertia weight I_G (×10^5kN・m^2)	Shearing cross section As(m^2)	Cross-section secondary momentI (m^4)
1	—	—		
			—	—
2	—	—		
			—	—
3	69,940	72.20		
			73.0	5,928
4	87,140	89.98		
			98.3	6,182
5	127,760	235.14		
			161.6	23,344
6	129,030	237.57		
			166.4	46,303
7	218,480	599.92		
			424.5	136,323
8	353,740	1021.56		
			2,812.6	772,237
9	132,390	346.27		
合計	1,118,480			

Young coefficient Ec	2.57×10^7 (kN/m2)
Shearing elastic coefficient G	1.07×10^7 (kN/m2)
Poisson's ratio v	0.20
Attenuation h	5%
Basic configuration	49.0m (N-S direction) X 57.4m (E-W direction)

Table-3.5.3 Ground Constant

(Ss-1)

Elevation O.P. (m)	Geology	S-wave velocity Vs (m/s)	Weight of unit volume γt (kN/m³)	Poisson's ratio ν	Initial shearing elastic coefficient G_0 (kN/m²)	Rigidity lowering rate G/G_0	Shearing elastic coefficient G (kN/m²)	S-wave velocity after rigidity lowered Vs (m/s)	Attenuation 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	
	Free rock surface	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	Initial shearing elastic coefficient G_0 (kN/m²)	Rigidity lowering rate G/G_0	Shearing elastic coefficient G (kN/m²)	S-wave velocity after rigidity lowered Vs (m/s)	Attenuation constant h (%)
10.0									
1.9	砂岩	380	17.8	0.473	262,000	0.85	223,000	351	3
-10.0	泥岩	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	
	解放基盤	700	18.5	0.421	924,000	1.00	924,000	700	—

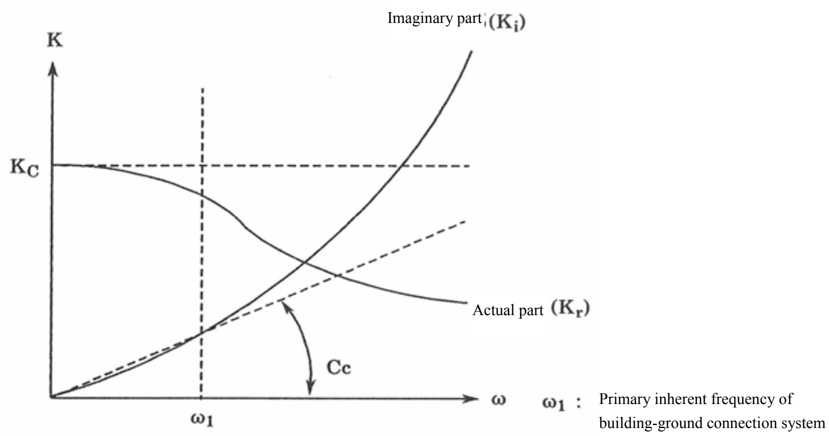
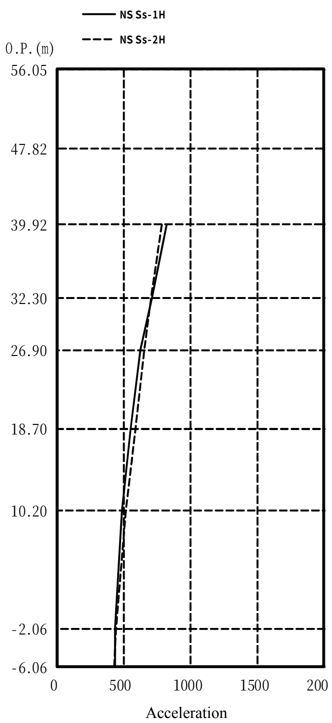


Figure-3.5.2 Approximation of Ground Sprıng

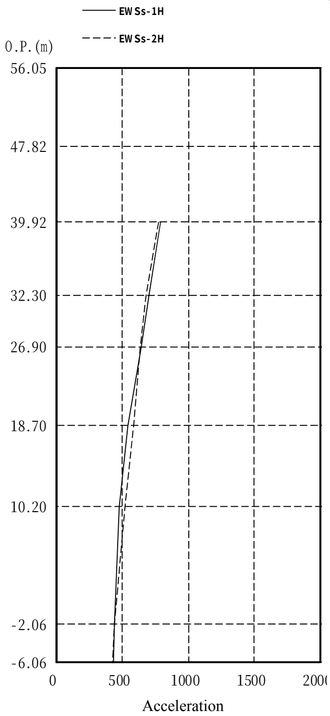
6. Results of Seismic Response Analysis

The maximum response accelerations for the N-S and E-W directions obtained based on the results of the seismic response analysis are shown in Figure-3.6.1. The analysis results are shown in comparison with the 2011 Report and “Interim Report on Seismic Safety Assessment Results following Revision of the ‘Guidelines for Inspection of Seismic Design of Nuclear Power Reactor Facilities’ at Fukushima Daiichi Nuclear Power Station (Revision 2)” (Tokyo Electric Power Company, April 2010) (hereinafter, “2010 Earthquake Back Check”).



O.P.(m)	Current analysis results Ss-1	Current analysis results Ss-2	2011 Report (ref.) Ss-1	2011 Report (ref.) Ss-2	2010 Earthquake Back Check (ref.) Ss-1	2010 Earthquake Back Check (ref.) Ss-2
56.05					1174	1037
47.82					918	875
39.92	822	784	796	761	749	738
32.30	713	706	673	690	657	698
26.90	628	651	610	640	613	659
18.70	550	590	554	581	553	592
10.20	489	517	494	500	502	509
-2.06	436	440	445	436	447	423
-6.06	433	426	439	426	439	411

(a) N-S Direction



O.P.(m)	Current analysis results Ss-1	Current analysis results Ss-2	2011 Report (ref.) Ss-1	2011 Report (ref.) Ss-2	2010 Earthquake Back Check (ref.) Ss-1	2010 Earthquake Back Check (ref.) Ss-2
56.05					1070	1004
47.82					945	822
39.92	788	771	770	763	783	766
32.30	703	682	714	692	661	710
26.90	643	639	626	656	617	657
18.70	543	588	545	598	548	584
10.20	476	521	478	518	488	483
-2.06	439	439	435	427	445	422
-6.06	433	423	430	412	438	415

(b) E-W Direction

Figure-3.6.1 Max Response Acceleration

7. Results of Seismic Safety Assessment

The maximum response values in relation to reference seismic motion Ss-1 and Ss-2 are shown on the shearing skeleton curve for earthquake-resisting walls in Figure-3.7.1. Shear strain is 0.16×10^{-3} (Ss-1, 2H, E-W direction, 1F) at a maximum, and there is sufficient allowance in relation to the assessment reference value (4.0×10^{-3}). Based on this, the reactor building, assuming the condition when spent fuel is removed, was assessed to have sufficient seismic safety even when disregarding rigidity of locations where partial bulging of the exterior walls was confirmed and walls where damage was confirmed.

Therefore, it is believed that the building will not collapse even if struck by reference seismic motion Ss.

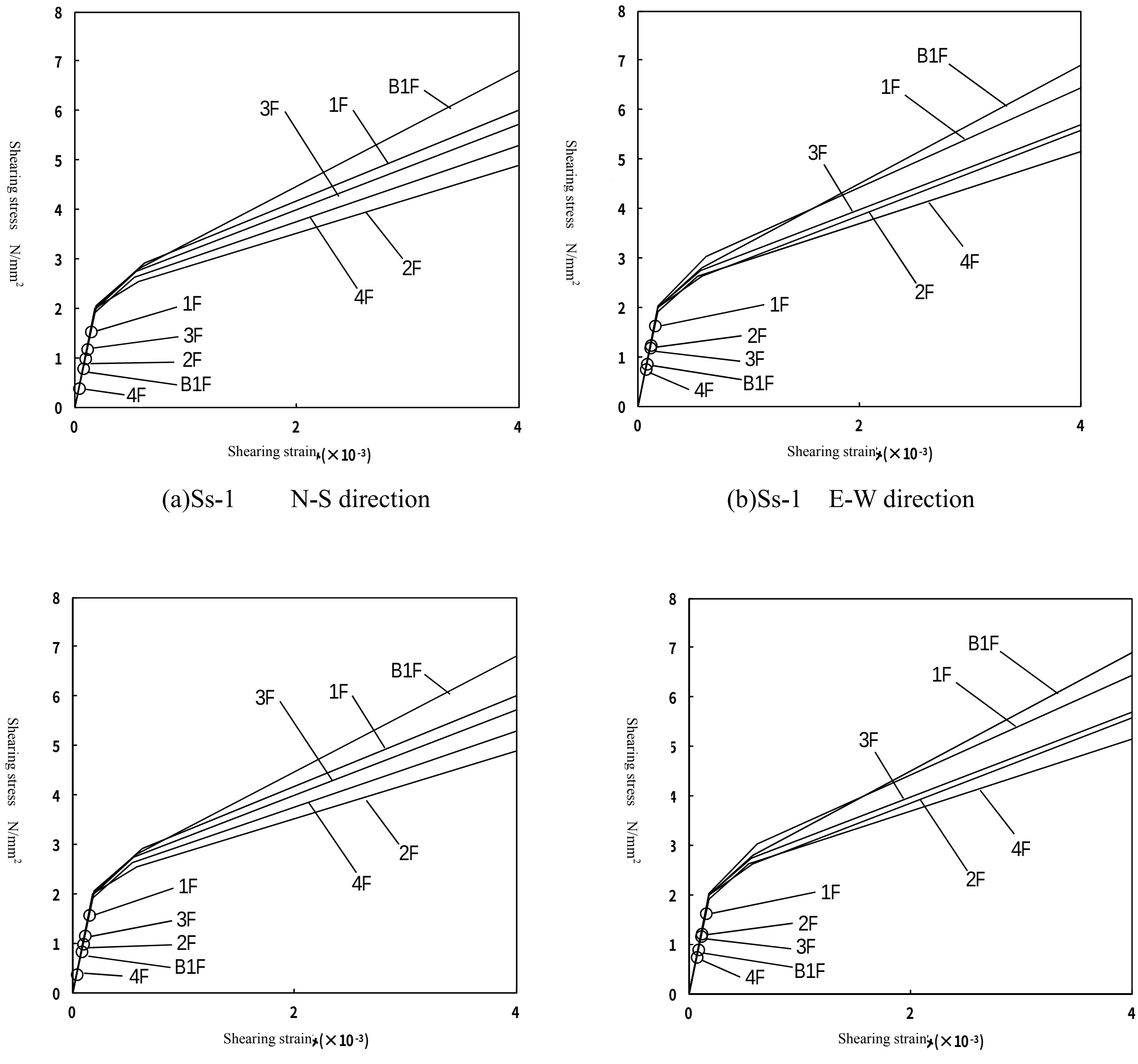


Figure-3.7.1 Maximum Response Value on Shearing Skeleton Curved Line

【Reference】

As a reference, a comparison of the current examination results and the maximum values for shear strain in the 2010 Earthquake Back Check and 2011 Report are shown.

When comparing the current examination results and the 2011 Report, the shear strain for the current examination results tend to be smaller overall. The main cause of this is considered to be that the weight of the 5th level is lighter due to the removal of debris from the upper part of the refueling floor.

In addition, when comparing the current examination results and the 2010 Earthquake Back Check, no significant differences have resulted. As shown in Figure-3.7.2, the main cause of this is considered to be because the shell walls and spent fuel pool walls thicker than the exterior walls were sound, and that the 5th level is lighter due to the removal of debris from the upper part of the refueling floor, despite the fact that there is damage to the exterior walls.

Table-3.7.1 Comparison of Shear Strain (N-S Direction) ($\times 10^{-3}$)

Level	O.P.(m)	Assessment reference value	Current examination		2011 Report		2010 Earthquake Back Check	
			Ss-1	Ss-2	Ss-1	Ss-2	Ss-1	Ss-2
CRF	47.82 ～ 56.05	4.0	-	-	-	-	0.10	0.09
5F	39.92 ～ 47.82		-	-	-	-	0.17	0.15
4F	32.30 ～ 39.92		0.04	0.04	0.06	0.06	0.05	0.05
3F	26.90 ～ 32.30		0.11	0.11	0.14	0.14	0.08	0.08
2F	18.70 ～ 26.90		0.10	0.10	0.09	0.09	0.09	0.09
1F	10.20 ～ 18.70		0.15	0.15	0.15	0.16	0.15	0.16
B1F	-2.06 ～ 10.20		0.08	0.08	0.08	0.08	0.08	0.08

Table-3.7.2 Comparison of Shear Strain (E-W Direction) ($\times 10^{-3}$)

Level	O.P.(m)	Assessment reference value	Current examination		2011 Report		2010 Earthquake Back Check	
			Ss-1	Ss-2	Ss-1	Ss-2	Ss-1	Ss-2
CRF	47.82 ～ 56.05	4.0	-	-	-	-	0.12	0.12
5F	39.92 ～ 47.82		-	-	-	-	0.30	0.20
4F	32.30 ～ 39.92		0.08	0.07	0.09	0.09	0.08	0.08
3F	26.90 ～ 32.30		0.12	0.11	0.13	0.13	0.11	0.11
2F	18.70 ～ 26.90		0.12	0.12	0.13	0.13	0.12	0.12
1F	10.20 ～ 18.70		0.16	0.16	0.16	0.17	0.16	0.17
B1F	-2.06 ～ 10.20		0.09	0.09	0.08	0.09	0.08	0.09

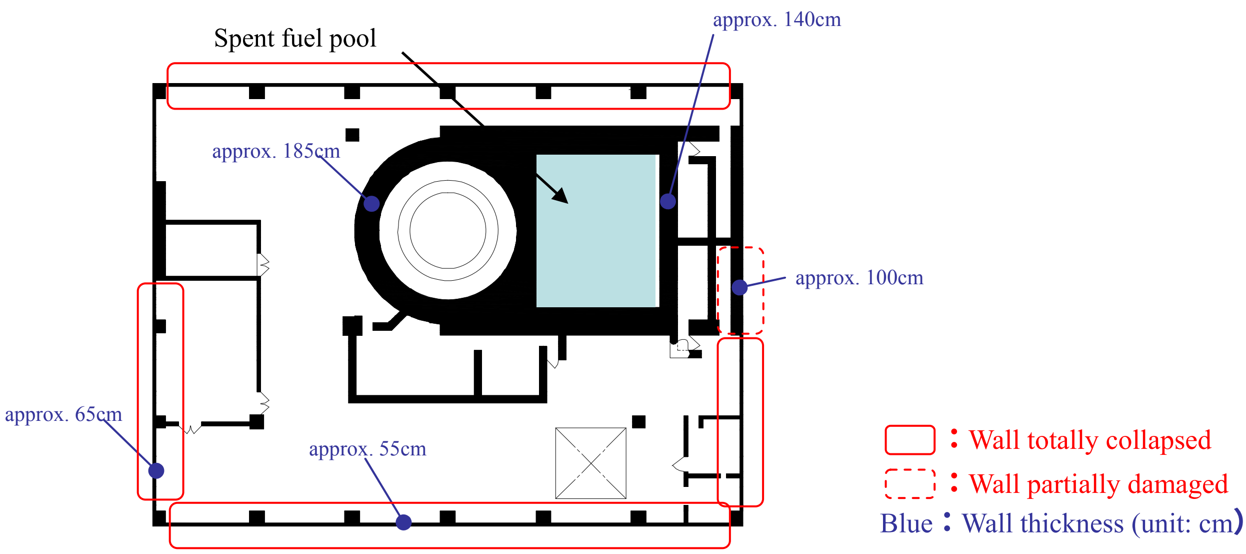


Figure-3.7.2 Damage Condition and Thickness of Exterior Walls (Example of 3rd Level)

Attachment-4:

Specific Details on the Results of the Seismic Safety Assessment of Spent Fuel Pool (Three-Dimensional FEM Analysis)

1. Policy for Analysis and Assessment

In last year's "the Report on Investigation into the Current Seismic Safety and Reinforcement of the Reactors at Fukushima Daiichi Nuclear Power Station (No. 1)" (Tokyo Electric Power Company, May 2011) (hereinafter, "2011 Report"), a detailed three-dimensional FEM analytical model was created for the portions above the second level based on the fact that the exterior walls from the 5th level down to the lower parts of the 3rd and 4th levels were elaborately damaged, and the seismic safety of the spent fuel pool was assessed in relation to reference seismic motion Ss using stress analysis. Also, in this year's "Report on the Seismic Safety of Unit 4 Reactor Building at Fukushima Daiichi Nuclear Power Station in Consideration of the Partial Expansion of the Exterior Wall" (Tokyo Electric Power Company, June 2012) (hereinafter, "June 2012 Report"), based on the three-dimensional FEM model used in the 2011 report, a model was created which disregards the rigidity of walls where partial bulging was confirmed in the exterior walls and the seismic safety was assessed of the spent fuel pool in relation to reference seismic motion Ss.

In this examination, along with reflecting the conditions of the reactor building at the time of removal of spent fuel which is consolidated in Attachment-1, a model has been prepared which disregards the rigidity of areas where partial bulging of the exterior walls was confirmed in the June 2012 Report and areas where damage (partial damage and total collapse) was confirmed in Attachment-2, and assessment has been conducted using three-dimensional FEM analysis for the seismic safety of the spent fuel pool. A plane view of the pool is shown in Figure-4.1.1 and a cross-sectional view in Figure-4.1.2.

The seismic safety assessment was conducted according to the following procedures as shown in the flowchart in Figure-4.1.3.

- Based on the portion of the building from the floor of the 2nd level surrounding the spent fuel pool (O.P.18.7m) to the floor of the 5th level (O.P.39.92m) (model in 2011 Report), a three-dimensional FEM analytical model was prepared which assumed the time of removal of spent fuel and disregarded the rigidity of the aforementioned areas.
- The loading conditions and load combination conditions were configured which included earthquake load, dynamic water pressure during an earthquake, counterforce of frame supporting fuel handling equipment and other factors based on the results of seismic response analysis, temperature load, static water pressure resulting from pool water and dead load. With regard to temperature load, more specific heat input conditions were configured based on the supplement to the "Report on the Current Seismic Safety and Reinforcement of Reactor Buildings at Fukushima Daiichi Nuclear Power Station (No.1) (Supplement) (Revision)" (September 2012), and the assessment conducted.
- An elastoplastic analysis was conducted which takes into account plasticization of reinforced

concrete component materials as stress analysis to calculate force and strain occurring on the spent fuel fool portion.

- A comparison was made with the assessment reference values to assess seismic safety.

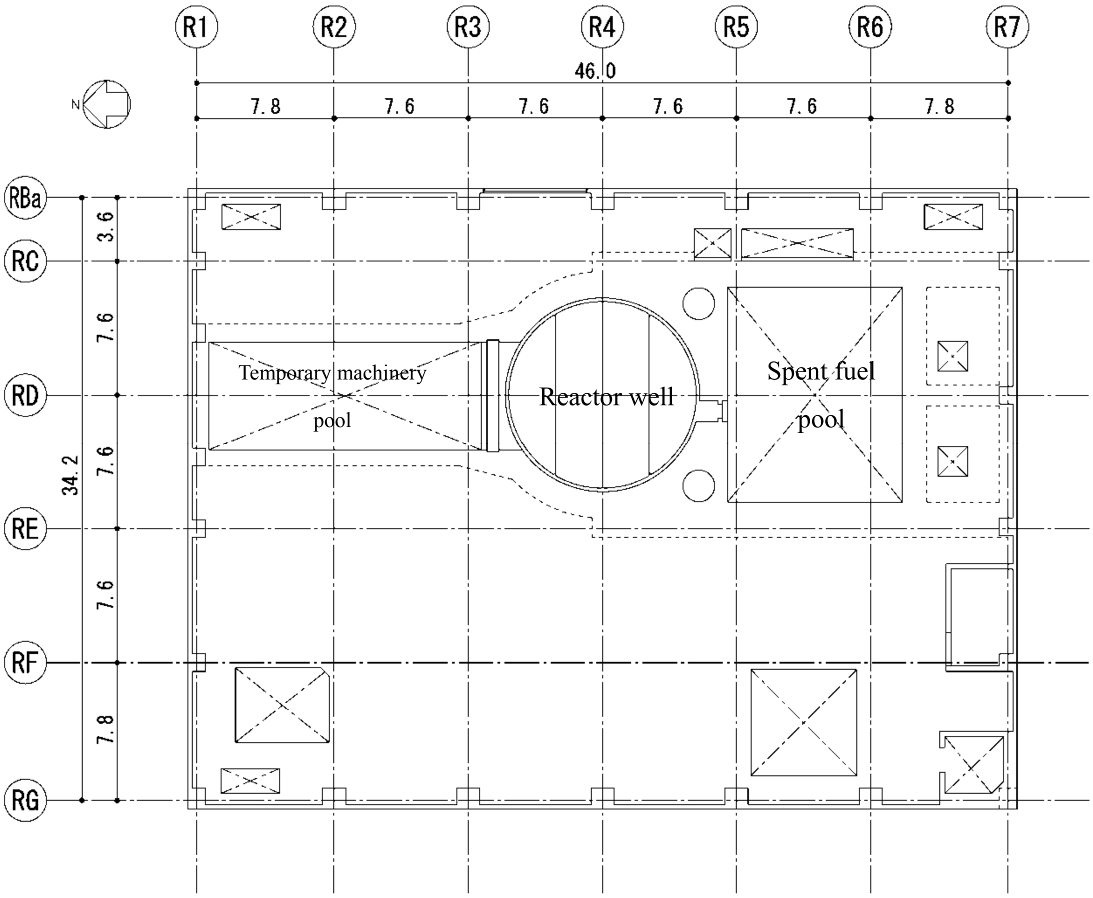


Figure-4.1.1 Plane View of 5th Level

(Unit: m)

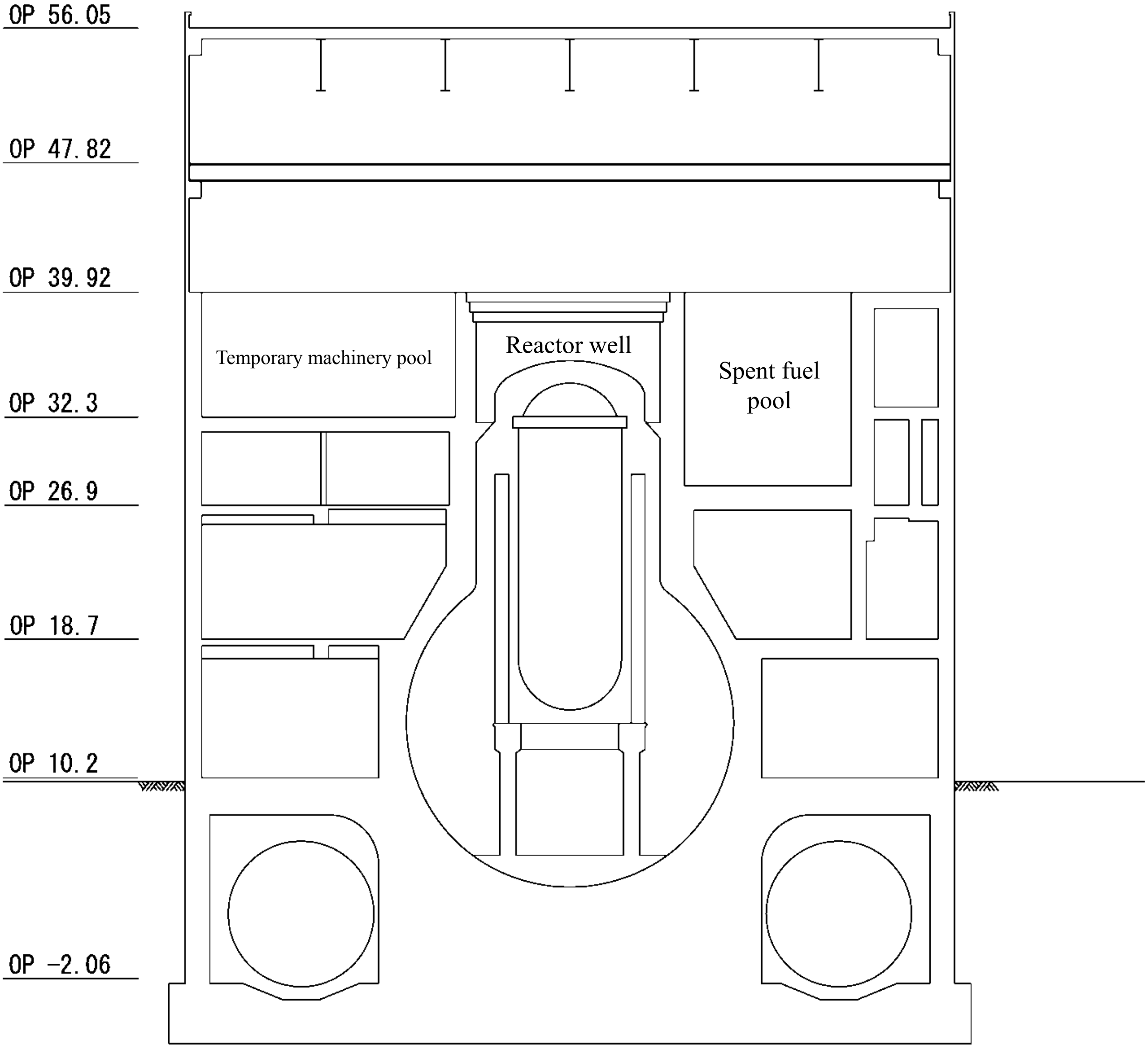


Figure-4.1.2 Cross-Sectional View
(N-S direction profile, unit: m)

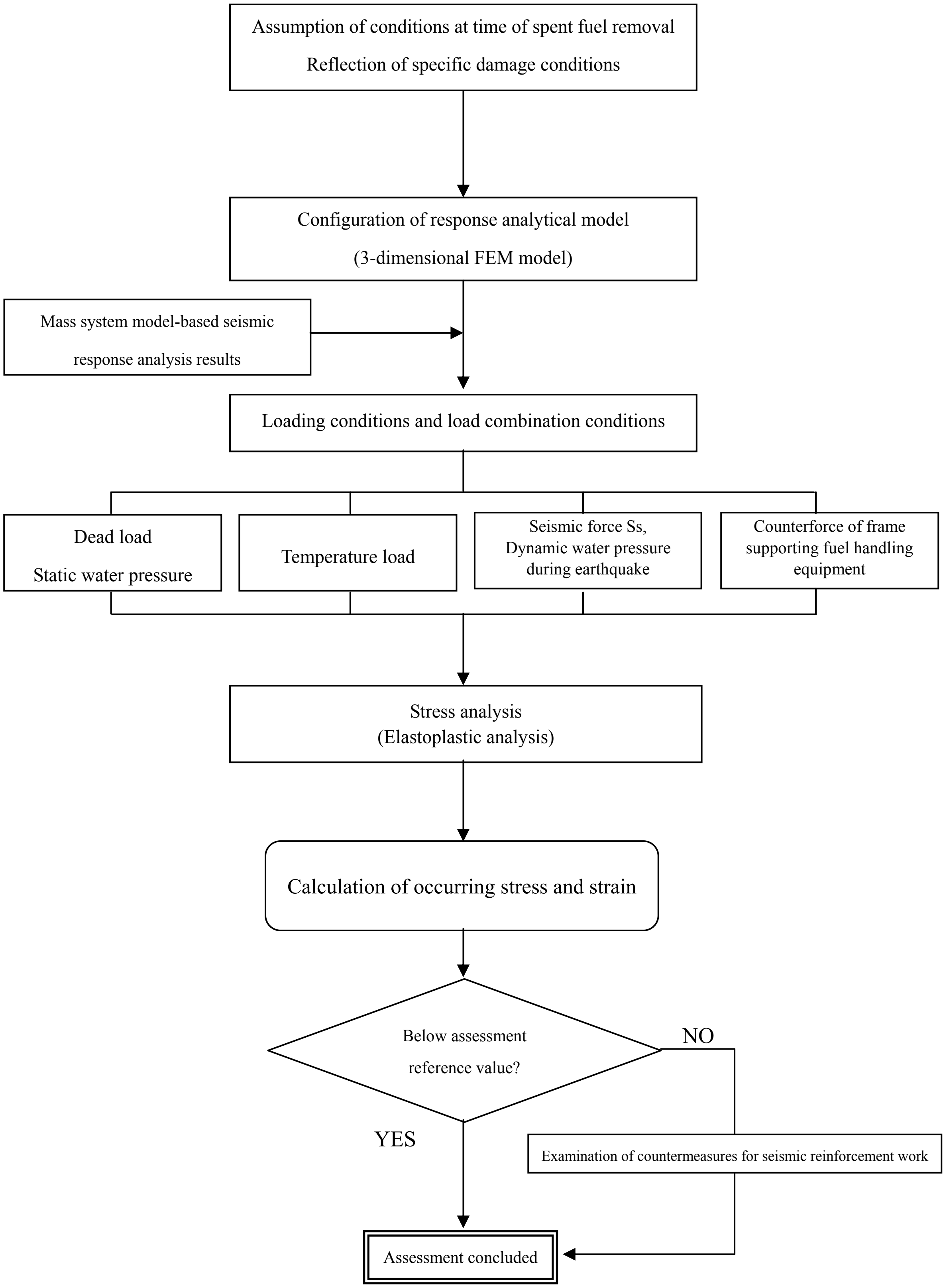


Figure-4.1.3 Flowchart of Seismic Safety Assessment of Spent Fuel Pool

2. Configuration of Stress Analytical Model

An elastoplastic analysis was conducted which takes plasticization of reinforced concrete component materials into consideration to calculate the stress and strain occurring on the spent fuel pool portion. The model was created such that the reinforced concrete component materials from the 2nd level walls to the 5th level refueling floor are regarded as an aggregate of finite elements. Also, under the floor of the spent fuel pool, elements simulating steel columns and other materials were also created as the support structure for the pool base shown in Attachment-1. Furthermore, based on the areas where partial bulging of the exterior walls was confirmed in the June 2012 Report and areas where damage (partial damage and total collapse) was confirmed in Attachment-2, a new stress analytical model was constructed which disregards rigidity in part of the floor and exterior walls.

Plate bonding elements used in the analytical model used laminated shell elements with anisotropic material that modeled the reinforcing steel layers. For each element, axial force and bending stress of the plate were simultaneously considered, and the effect of antiplane shear deformation was also taken into account for the bending of plates. The computer system code used was "ABAQUS."

A schematic diagram of the analytical model is shown in Figure-4.2.1, and constitutive law of reinforcing steel and concrete in Figure-4.2.2, and the boundary conditions of analytical model in Figure-4.2.3.

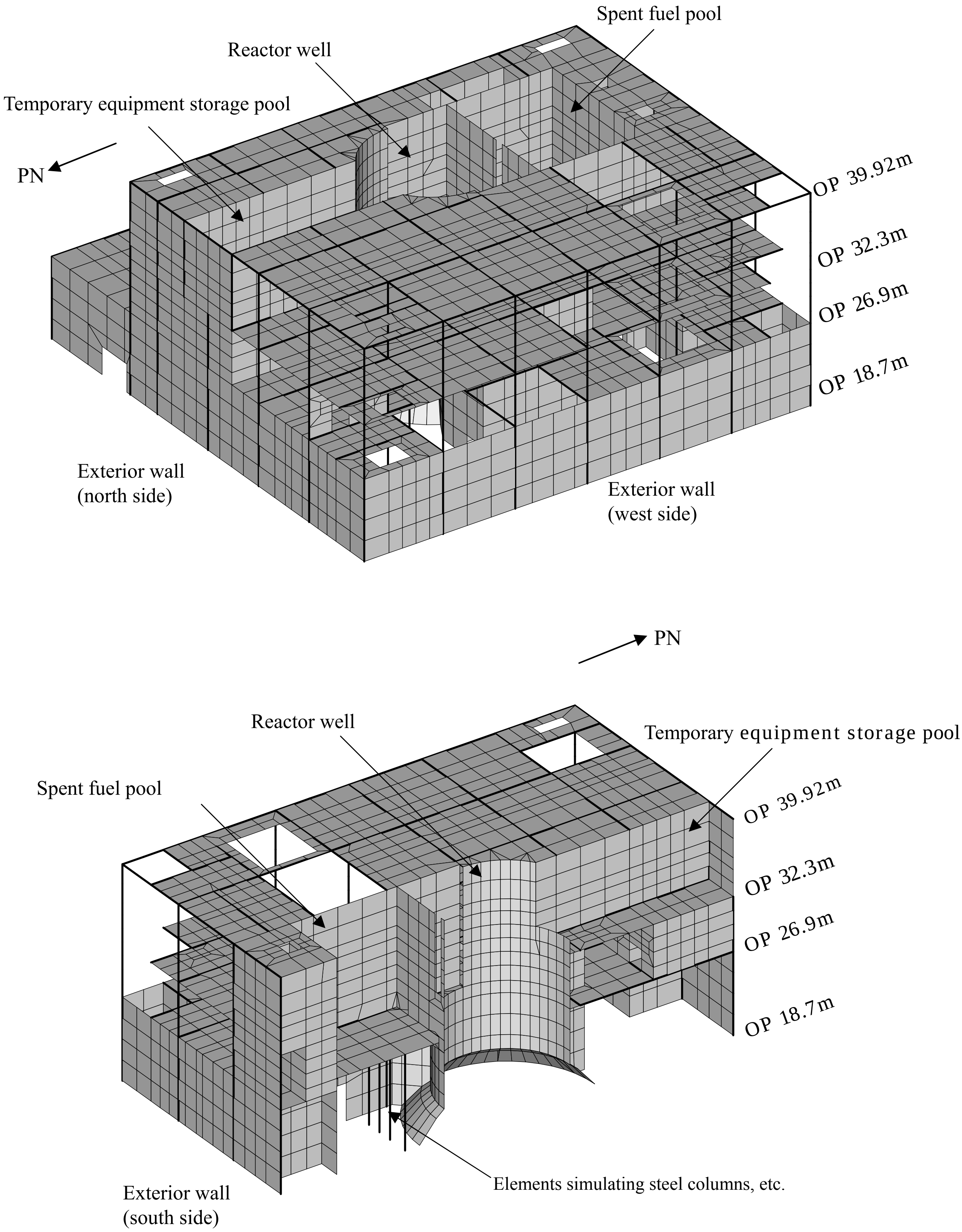
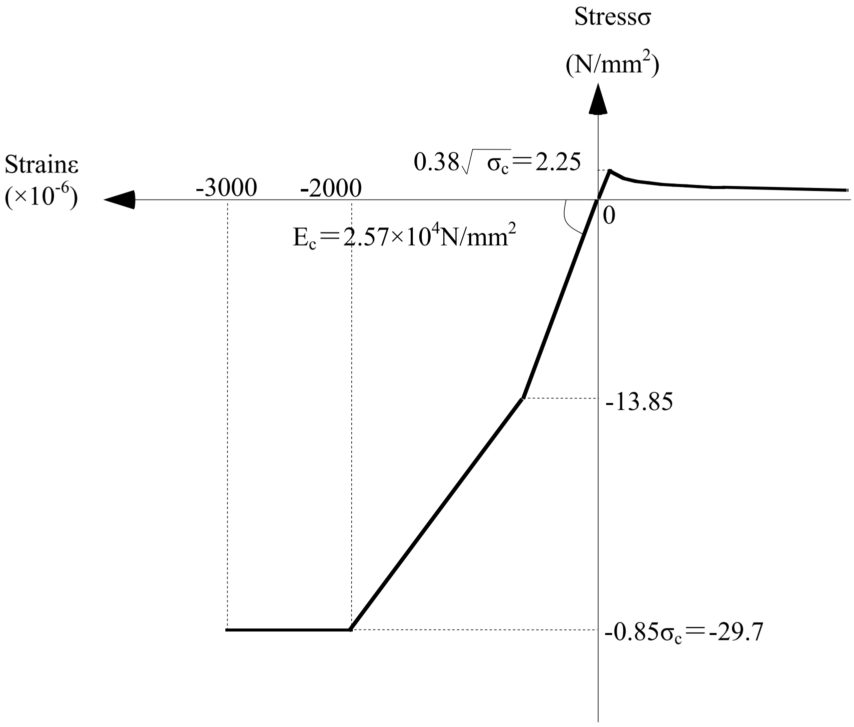
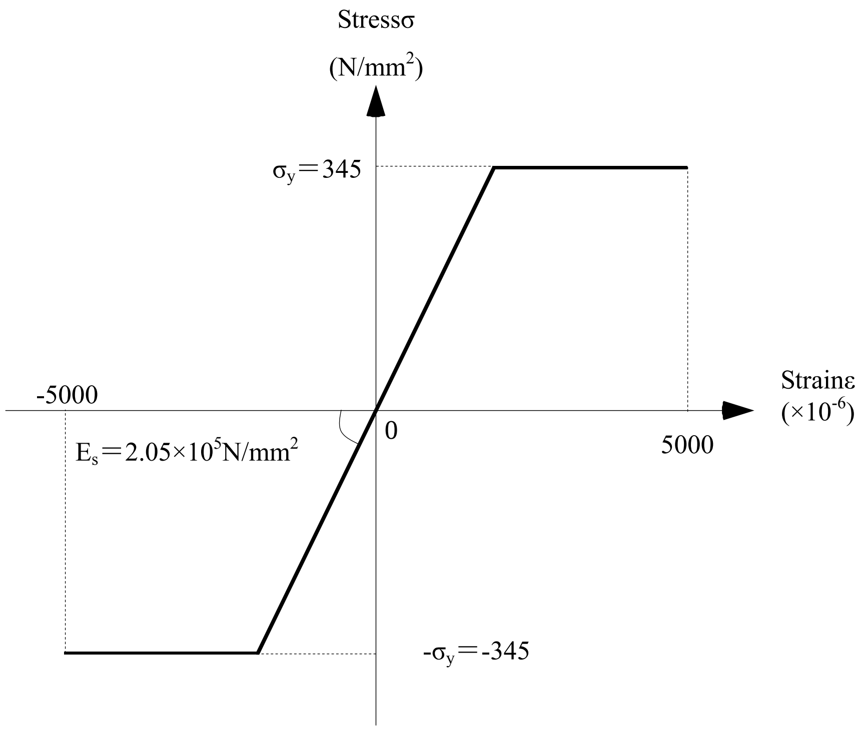


Figure-4.2.1 Schematic Diagram of Analytical Model



(a) Stress-Strain Relationship of Concrete
(Concrete strength $\sigma c = 35 N/mm^2$)



(b) Stress-Strain Relationship of Reinforcing Steel
(Reinforcing steel yield point $\sigma y = 345 N/mm^2$)

Figure-4.2.2 Constitutive Law of Concrete and Reinforcing Steel

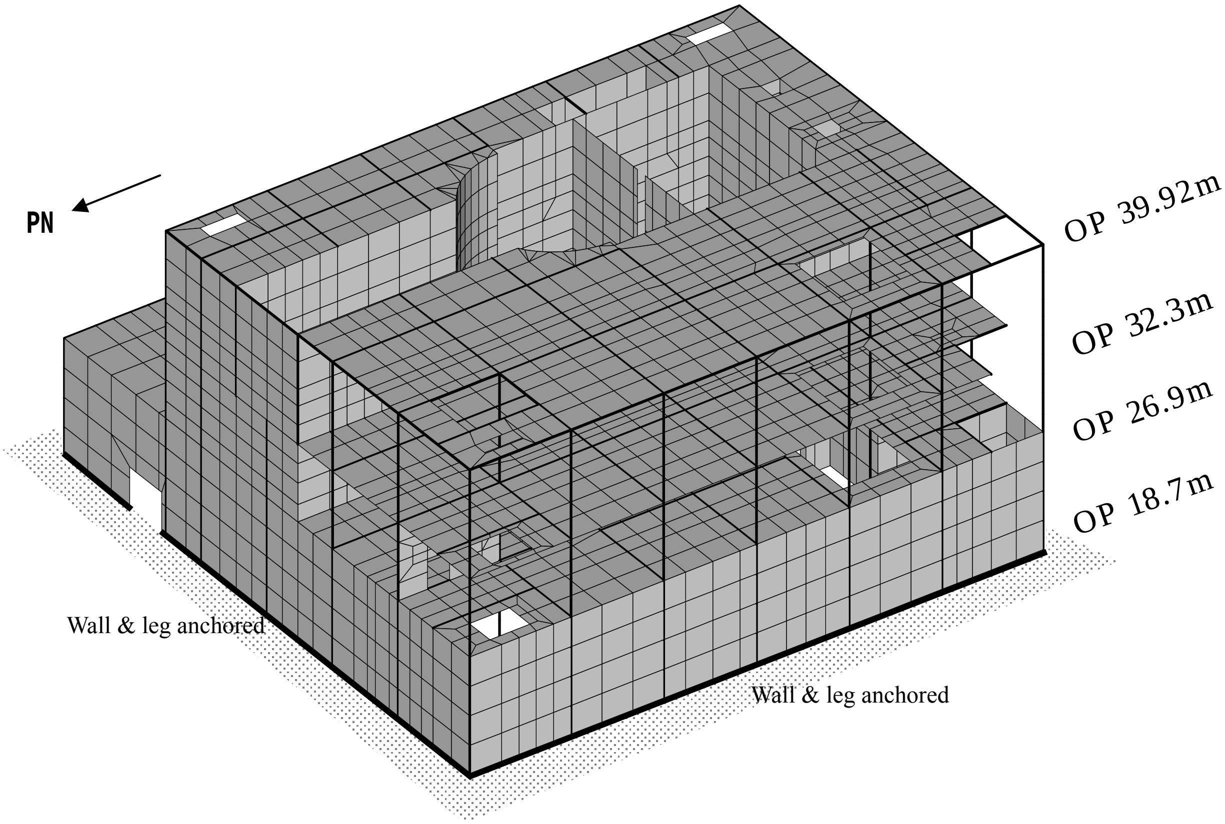


Figure-4.2.3 Boundary Conditions of Analytical Model

3. Assumption of Damage Conditions

In assuming the damage conditions, a three-dimensional analytical model was prepared which newly disregards the rigidity of the following damaged areas in accordance with the 2011 Report and is based on the areas where partial bulging of the exterior walls was confirmed in the June 2012 Report and areas where damage (partial damage and total collapse) was confirmed in Attachment-2. The damage model is shown in Figures-4.3.1~4.3.4.

(1) Exterior and Interior Walls

As for the exterior walls, the rigidity was disregarded of a portion of the walls on the south side and the west side of the 2nd level (O.P.18.7m) where there was bulging in the exterior wall in the June 2012 Report and the south side of the 3rd level (O.P.26.9m) and 4th level (O.P.32.3m).

As for the interior walls, the rigidity was disregarded of a portion of the wall on the north side of the 3rd level (O.P.26.9m).

(2) Floor Slaps

As for floor slaps, the rigidity was disregarded for parts of the 3rd level (O.P.26.9m) to 5th level (O.P.39.92m) as well as all of the partially damages areas of the floors and the areas which completely collapsed.

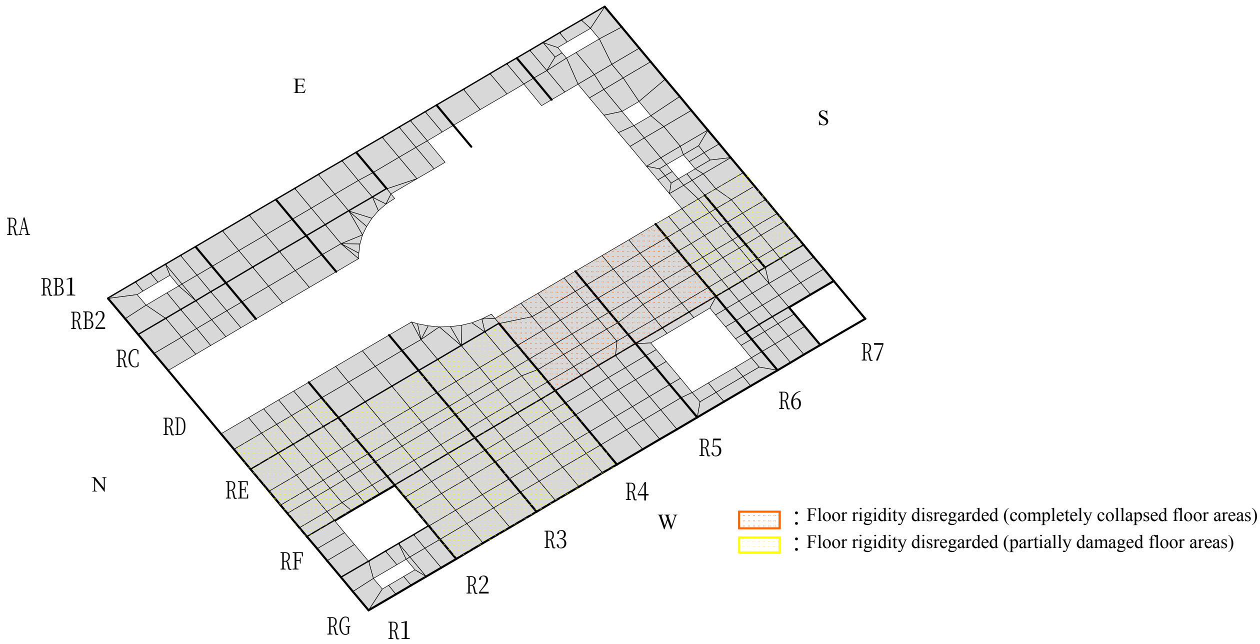


Figure 4.3.1 Damage Model: Isometric Drawing of 5th Level (O.P.39.92m)

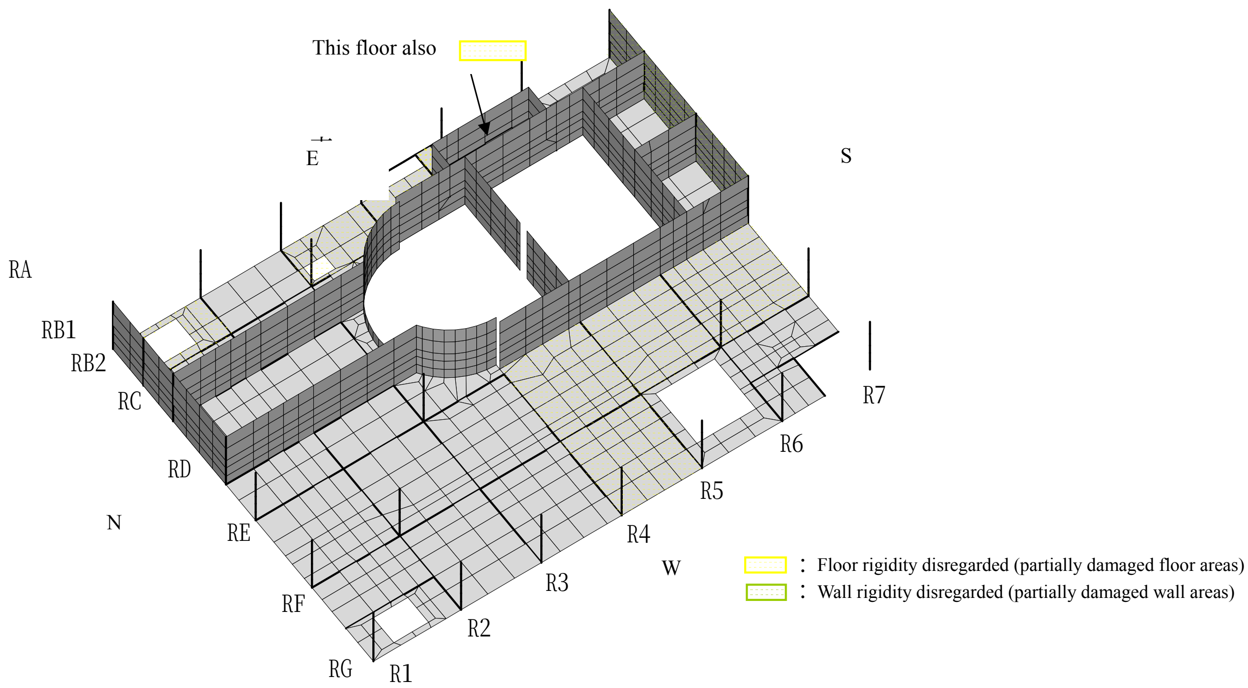


Figure 4.3.2 Damage Model: Isometric Drawing of 4th Level (O.P. 32.3m)

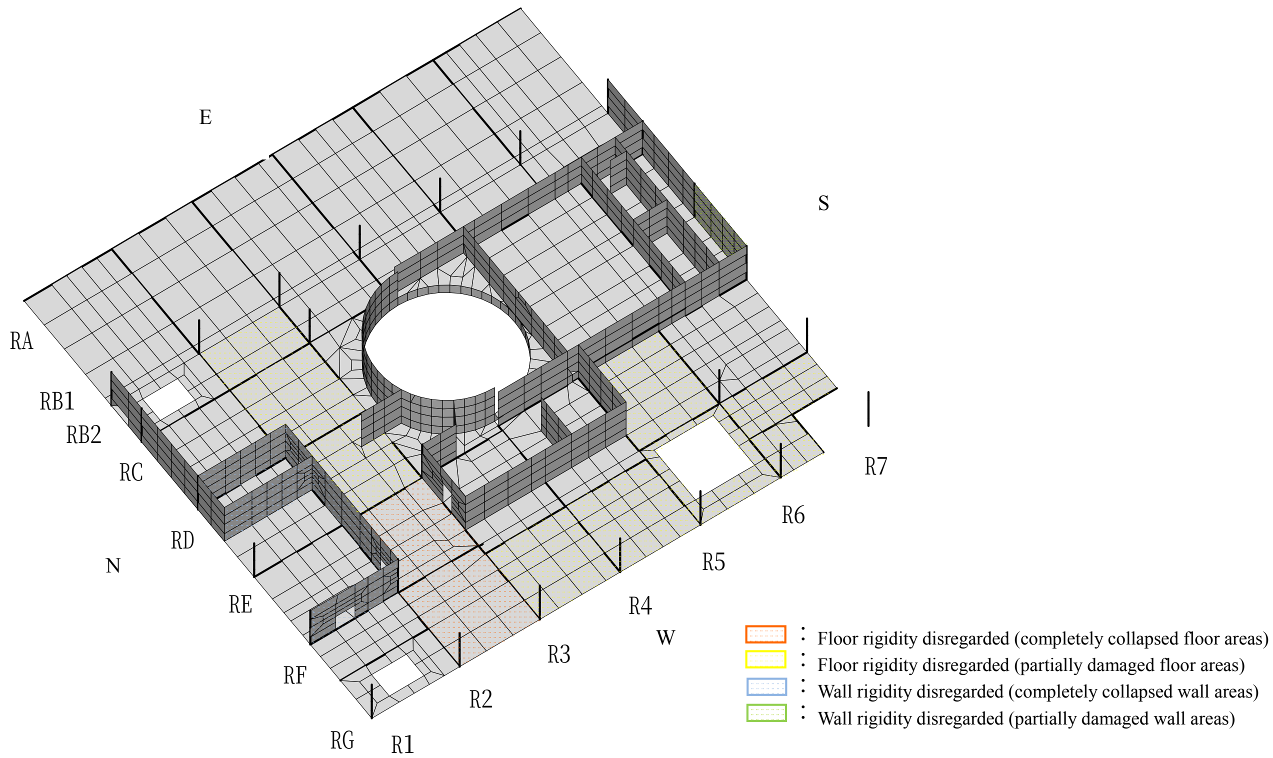


Figure 4.3.3 Damage Model: Isometric Drawing of 3rd Level (O.P. 26.9m)

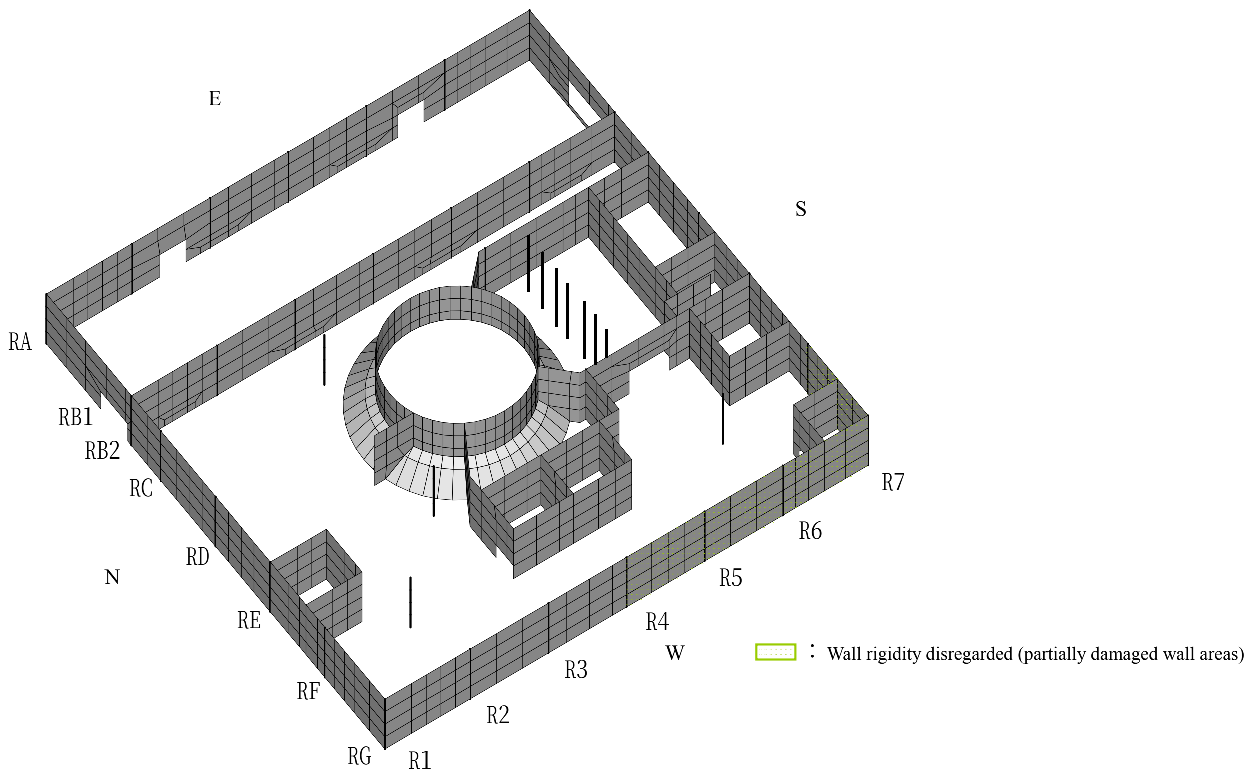


Figure 4.3.4 Damage Model: Isometric Drawing of 2nd Level (O.P. 18.7m)

4. Loading and Load Combinations

(1) Dead load

Dead load applied to the analytical model takes into account the deadweight of the building frame within the scope of the model and the weight of the equipment, pipes and so on is carried uniformly. Also, the weight of the casks installed inside the pool and the weight of the frame supporting the fuel handling equipment are taken into consideration at the installation locations in question.

(2) Static water pressure

Static water pressure is taken into account for a case assuming that the spent fuel pool, reactor well and temporary equipment storage pool are filled with water.

(3) Temperature load

The temperature of the pool water is 65°C which is the managed temperature since circulative cooling was commenced. The outdoor air temperature is 0°C assuming winter.

(4) Seismic load

In accordance with the results of seismic response analysis in relation to reference seismic motion Ss using the mass system model in Attachment-3, vertical direction and horizontal direction seismic load are configured.

(5) Other loads

Dynamic water pressure during earthquake for pool water and the counterforce of frame supporting fuel handling equipment acting on the well top are taken into consideration.

(6) Load combinations

Load combinations are shown in Table-4.4.1. The combination of horizontal and vertical seismic motion is assessed using a combination coefficient method (combination coefficient 0.4).

Table 4.4.1 Load Combinations

Name of load time	Load combination
Ss earthquake time	DL + H + T + K + KH ＋ KF

Here,

DL: deal load, H: static water pressure, T: temperature, K: seismic load (reference seismic motion Ss),

KH: dynamic water pressure during earthquake, KF: counterforce of frame supporting fuel handling equipment

5. Analysis Conditions

A comparison of the analysis conditions in the current examination and the 2011 Report is shown in Table-4.5.1.

Table-4.5.1 Comparison of Analysis Conditions in Current Examination and 2011 Report

Item			2011 Report (basic case)*1	Current Examination (basic case)
Model	Walls (including those of pool and shell)		Walls with confirmed damage deleted from model	Rigidity of totally collapsed and partially damaged walls is 0%.
	Floors (including that of pool)		Model created with all floors considered as sound from 3rd level floor to 5th level floor	Rigidity of totally collapsed and partially damaged walls is 0%.
	Stiffening effect		Not taken into account	Simulation of steel columns of support structure for spent fuel pool base
Load	Dead load		Weight from 5th level to roof level as debris weight Concentrated on 5th level	Reflection of removal of debris from upper part of refueling floor, load of frame supporting fuel handling machinery, etc. (Attachment-3)
	Static water pressure		Consideration of static water pressure for a case assuming that the spent fuel pool, reactor well and temporary equipment storage pool are filled with water	Same as on left
	Temperature load		Summer and winter not taken into account, uniform interior of 90 C, outside of 10°C, and inside reactor of 40 C	Uniform interior of 65°C, outside of 0°C, and inside reactor of 40°C
	Seismic load		Vertical direction and horizontal direction seismic load are taken into account based on results of seismic response analysis in relation to reference seismic motion Ss using the mass system model which considers damage	Same as on left
	Other loads	Dynamic water pressure during earthquake	Dynamic water pressure is taken into account for pool water acting during an earthquake based on results of seismic response analysis in relation to reference seismic motion Ss using the mass system model which considers damage	Same as on left
		Counterforce of frame supporting fuel handling equipment	Not taken into account	Counterforce is taken into consideration of frame supporting fuel handling equipment

*1：In the 2011 Report, in addition to the basic case, analyses of three types of parameter cases were conducted. ①In a case viewing the impact from an explosion, the rigidity of the partially destroyed exterior walls of the 3-4 levels is decreased to 50%, and to 50% over the entire surface for the rigidity of the 4-5 level floors; ②in a case viewing the impact from a fire, the rigidity of the west-side pool wall is reduced to 80%, and the rigidity of the floors of the 4-5 levels on the west side is reduced to 80% for the entire surface; ③ in a case viewing the impact due to an increase in temperature of the pool water, a rise in the pool water temperature and winter season were assumed and the examination conducted with uniform interior temperature of 100°C, outside of 0°C, and inside the reactor of 40°C. Furthermore, In Appendix 4-4, ④ an analysis has also been conducted of a case confirming the reinforcing effect of the support structure on the pool base.

6. Assessment Results

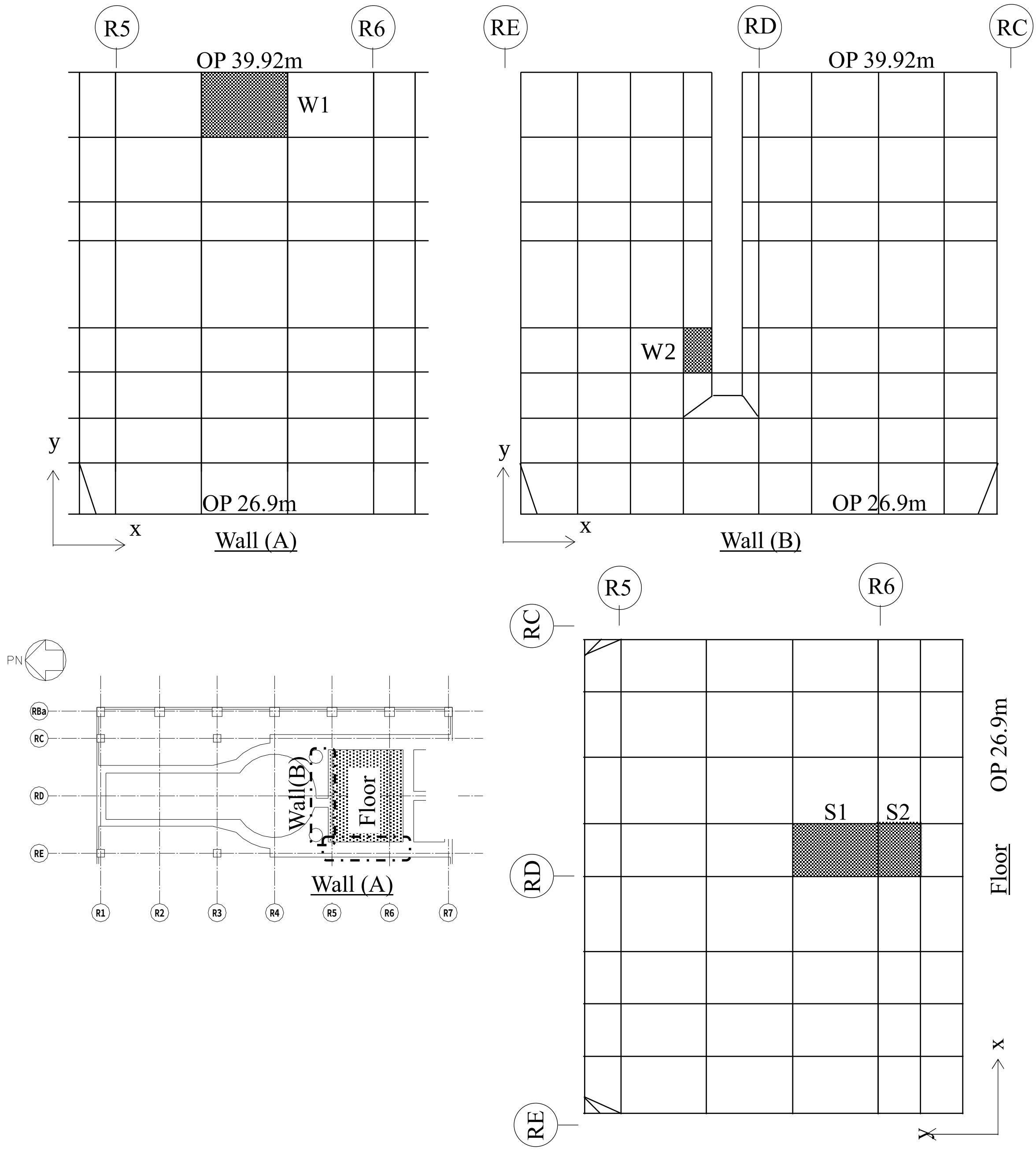
A structural examination was conducted of the spent fuel pool based on reinforcing bar arrangement specifications, etc. to assess seismic safety. In the assessment, the occurring stress and strain found using stress analysis was verified to be below the assessment reference value. The assessment reference value was configured based on the "Nuclear Power Facility Standards: Concrete Primary Containment Vessel Standards" of the Japan Society of Mechanical Engineers. The reinforcing bar arrangement specifications for the areas assessed are shown in Figure-4.6.1.

The assessment results are shown in Table-4.6.1 to Table-4.6.4. At all other areas, the occurring stress and strain were within the range of elasticity, and sufficiently below the assessment reference value. Based on this, in the conditions at the time of removal of spent fuel, the spent fuel pool was assessed to have seismic safety even disregarding the rigidity of areas where partial bulging were confirmed in the exterior walls and floor slaps and walls confirmed to be damaged.

Strain was within the range of elasticity, so it is considered that there is no possibility that the liner lining the concrete is damaged and that water from the spent fuel pool is leaking out.

Explanation of symbols used in Table-4.6.1 to Table 4.6.4

${}_{c}\varepsilon_{c}$	: Compression strain of concrete
${}_{s}\varepsilon_{c}$, ${}_{s}\varepsilon_{t}$	: Compression strain and tensile strain of reinforcing steel (All strain is expressed as positive on the tension side)
Q	: Antiplane shear



Location	Inside reinforcement		Outside reinforcement		Shear reinforcement
	x direction	y direction	x direction	y direction	
W1	D32@250	D32@120	D32@250	D32@240	—
W2	D38@130	D38@130	D38@150	D38@113	

Location	Top end reinforcement		Bottom end reinforcement		Shear reinforcement
	x direction	y direction	x direction	y direction	
S1	D32@100＋D32@200		D32@200		—
S2					

Figure-4.6.1 Reinforcing Bar Arrangement Specifications for Areas Assessed

Table-4.6.1 Results of Examination of Strain of Concrete and Reinforcing Bars due to Axial Force and Bending Moment (Walls)

Name of location	Examined strain	Name of load time	Occurring strain ($\times 10^{-6}$) ε	Assessment reference value ($\times 10^{-6}$) ε'	Testing ratio ε／ε'	Finding
W1	$_c\varepsilon_c$	Ss earthquake time	-150	-3000	0.05≦1	Pass
	$_s\varepsilon_c$		-90	-5000	0.02≦1	Pass
	$_s\varepsilon_t$		1180	5000	0.24≦1	Pass

Table-4.6.2 Results of Examination of Strain of Concrete and Reinforcing Bars due to Axial Force and Bending Moment (Floors)

Name of location	Examined strain	Name of load time	Occurring strain ($\times 10^{-6}$) ε	Assessment reference value ($\times 10^{-6}$) ε'	Testing ratio ε／ε'	Finding
S1	$_c\varepsilon_c$	Ss earthquake time	-370	-3000	0.13≦1	Pass
	$_s\varepsilon_c$		-140	-5000	0.03≦1	Pass
	$_s\varepsilon_t$		250	5000	0.05≦1	Pass

Table-4.6.3 Results of Examination of Antiplane Shear Force (Walls)

Name of location	Name of load time	Occurring stress Q (N/mm)	Assessment reference value Q' (N/mm)	Testing ratio Q／Q'	Finding
W2	Ss earthquake time	1120	1860	0.61≦1	Pass

Table-4.6.4 Results of Examination of Antiplane Shear Force (Floors)

Name of location	Name of load time	Occurring stress Q (N/mm)	Assessment reference value Q' (N/mm)	Testing ratio Q／Q'	Finding
S2	Ss earthquake time	580	1270	0.46≦1	Pass

【Reference】

As a reference, comparisons are shown in Table-4.6.5 and Table-4.6.6 of areas where the ratio of the occurring strain and occurring stress are the largest in relation to the assessment reference value of the basic case in the 2011 Report. Excluding the antiplane shear of the walls, the respective testing ratios of the strain of walls and floors as well as the antiplane shear of the floor were smaller than even the 2011 Report. This is believed to mainly the effect exercised from decreasing the temperature load by means of changing the condition of the water temperature of the spent fuel pool from 90℃ to 65℃. In addition, with regard to the floor, there is believed also to be an effect from the base of the spent fuel pool having been reinforced with steel columns, and the allowance in relation to the assessment reference value has increased.

The locations where the testing ratios in the current examination and the 2011 Report were the maximum are different. In Figure-4.6.2, a location is shown where the testing ratio of the strain and antiplane shear are the maximum in the 2011 Report. The locations where the testing ratio is the maximum for wall strain is shown by W1', for the antiplane shear by W2', for the floor strain by S1' and for the antiplane shear of the floor by S2'.

Table-4.6.5 Comparison of Occurring Strain of Concrete and Reinforcing Bars due to Axial Force and Bending Moment

Name of location	Examined strain	Name of load time	Occurring strain ($\times 10^{-6}$)				Assessment reference value ($\times 10^{-6}$)
			Current examination		2011 Report		
				Testing ratio		Testing ratio	
Wall	${}_c\varepsilon_c$	Ss earthquake time	-150	0.05	-480	0.16	-3000
	${}_s\varepsilon_c$		-90	0.02	-350	0.07	-5000
	${}_s\varepsilon_t$		1180	0.24	1230	0.25	5000
Floor	${}_c\varepsilon_c$		-370	0.13	-580	0.20	-3000
	${}_s\varepsilon_c$		-140	0.03	-210	0.05	-5000
	${}_s\varepsilon_t$		250	0.05	490	0.10	5000

Table-4.6.6 Comparison of Results of Examination of Antiplane Shear

Name of location	Name of load time	Occurring stress Q (N/mm)			
		Current examination		2011 Report	
			Testing ratio		Testing ratio
Wall	Ss earthquake time	1120 (1860)	0.61	2040 (3770)	0.55
Floor		580 (1270)	0.46	800 (1150)	0.70

Numerals in () indicate assessment reference value.

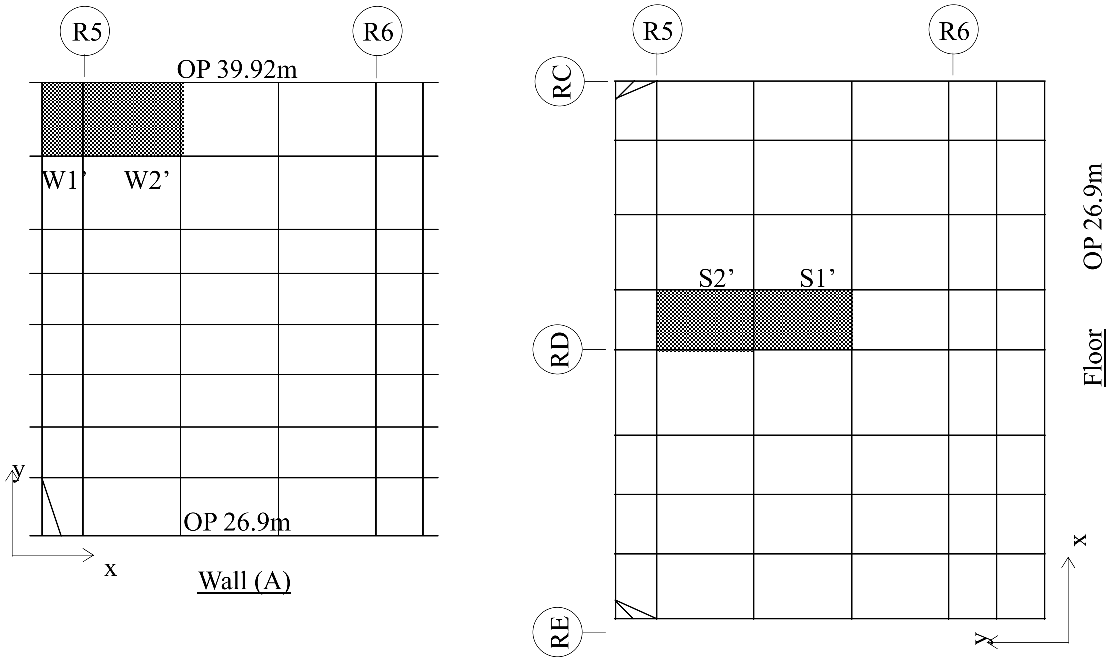


Figure-4.6.2 Locations Where Testing Ratio of Strain and Antiplane Shear are Maximized in 2011 Report

Appendix 4-1

Parametric Study Related to Results of Seismic Safety Assessment of Spent Fuel Pool

1. Overview

In the main body of Attachment-4, an analysis was conducted which disregarded the rigidity of areas where partial bulging were confirmed in the exterior walls in the June 2012 Report and the floor slaps and walls confirmed to be damaged (partial damage and total collapse) in Attachment-2. However, many of the walls and floor slaps are not completely destroyed and have some residual rigidity. Here, an analysis is conducted of a case in which such residual rigidity is taken into account, and such impact is ascertained.

2. Examination Conditions

The rigidity of part of the floor slabs and exterior walls disregarding rigidity is configured as described in the basic case below. A comparison of the configuration of rigidity with the basic case is shown in Table-1. The rigidity of areas other than Table-1 are configured the same as the basic case in the 2011Report. The damage models are shown in Figure-1 to Figure-4.

(1) Exterior Walls

For the exterior walls, the actual condition of rigidity of part of the south side and west side, where there is bulging in the exterior wall as described in the June 2012 Report, and the south side of the 3^{rd} level (O.P.26.9m) and 4^{th} level (O.P.32.3m) according to Attachment-2, are taken into consideration and set at 50%.

(2) Floor Slabs

For the floor slabs, the actual condition of rigidity of locations determined to be partially damaged floor are taken into consideration for parts of the 5^{th} level (O.P.39.92m) to 3^{rd} level (O.P.26.9m), and set at 50%.

Table-1 Comparison of Rigidity Configuration with Basic Case

Case	Configuration of wall rigidity			Configuration of floor slab rigidity	
	Completely collapsed areas*1	Partially damaged areas*1	Bulging areas*2	Completely collapsed areas*1	Partially damaged areas*1
Basic case	0%	0%	0%	0%	0%
Parameter case	0%	50%	50%	0%	50%

*1： Areas confirmed to be damaged as shown in Attachment-2

*2：Areas where partial bulging of exterior wall confirmed in June 2012 Report

Figure-1 Damage Model: Isometric Drawing of 5th Level (O.P. 39.92m) (Parameter Case)

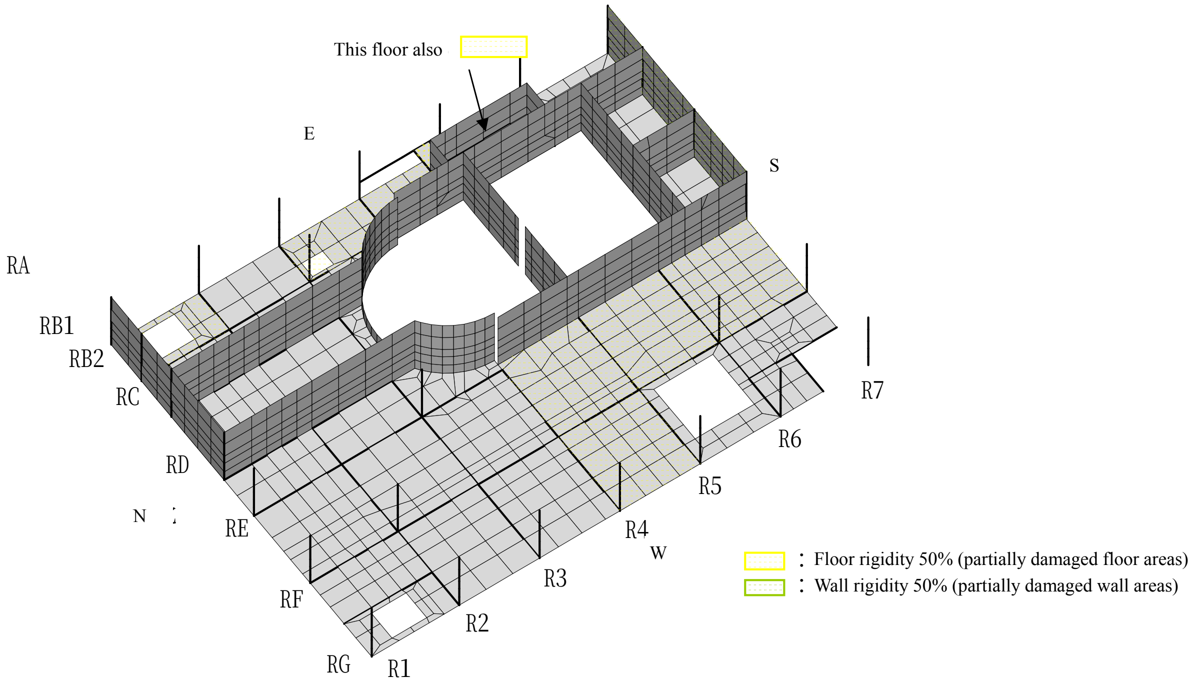


Figure-2 Damage Model: Isometric Drawing of 4th Level (O.P. 32.3m) (Parameter Case)

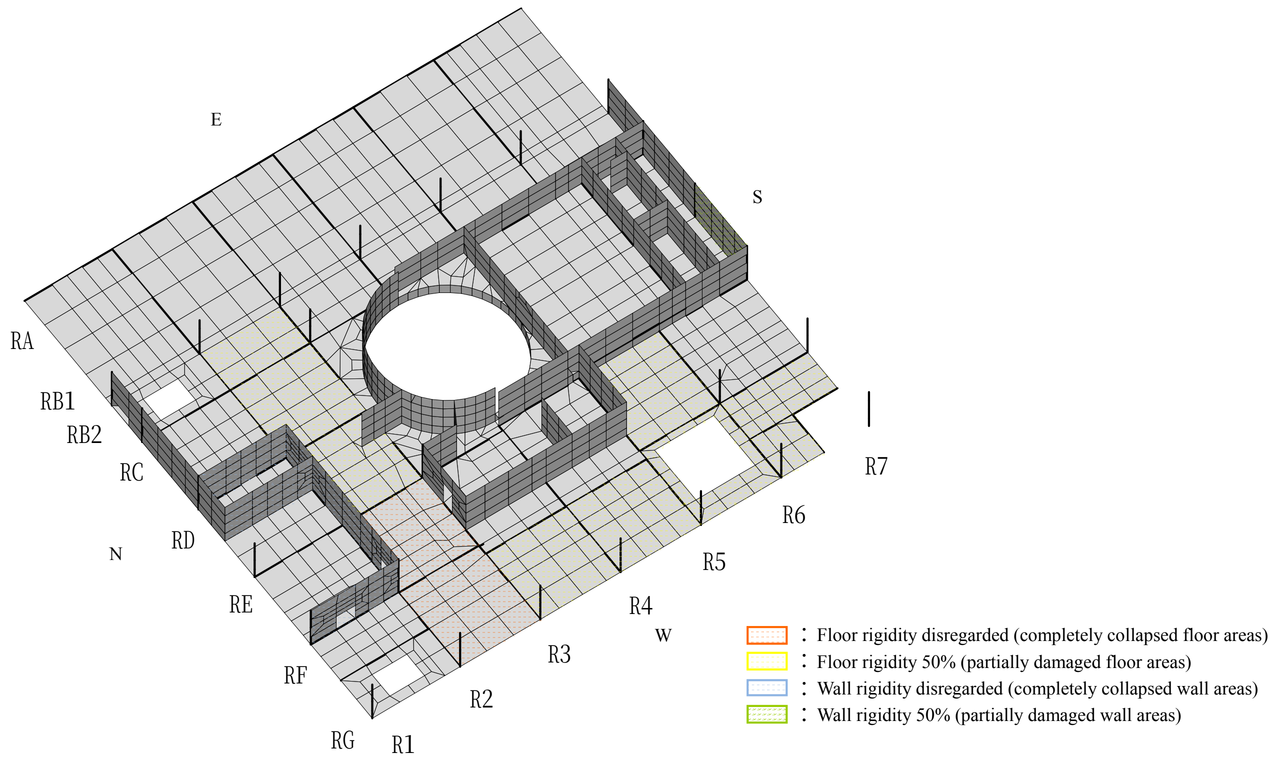


Figure-3 Damage Model: Isometric Drawing of 3rd Level (O.P. 26.9m) (Parameter Case)

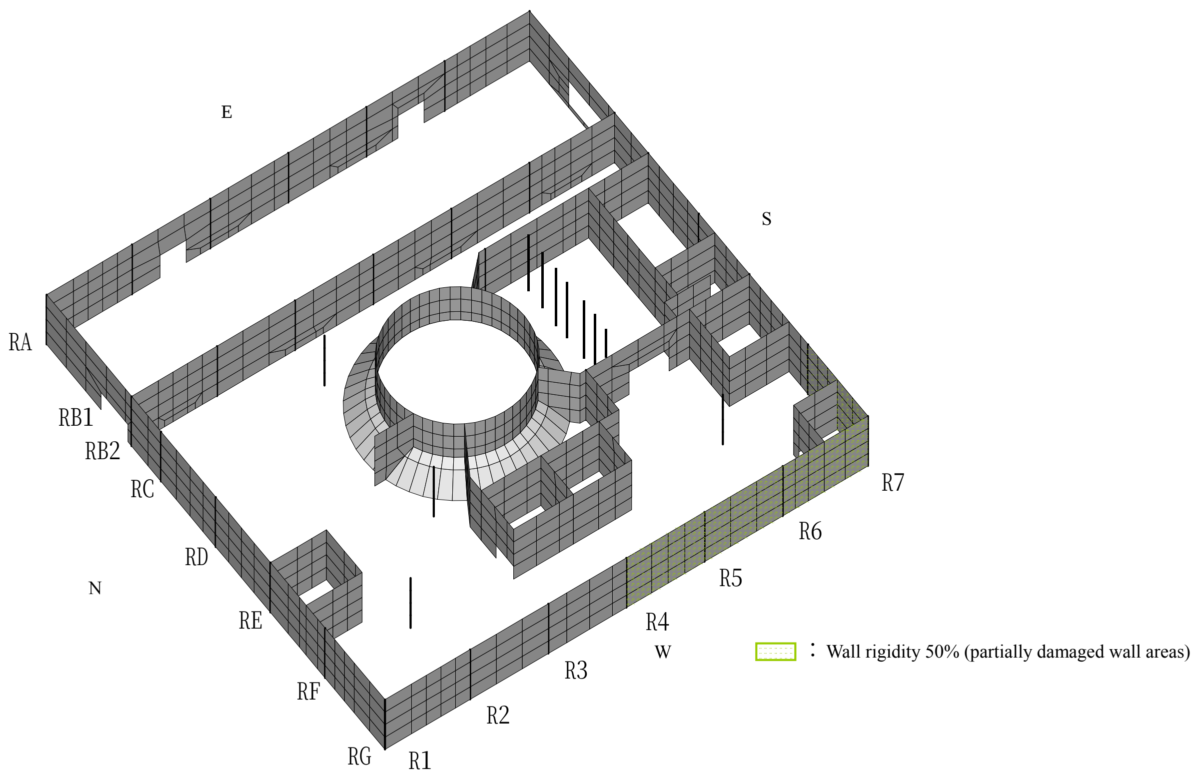


Figure-4 Damage Model: Isometric Drawing of 2nd Level (O.P. 18.7m) (Parameter Case)

3. Examination Results

The results of a comparison of the ratio of occurring strain and occurring stress in relation to the assessment reference value of the basic case and parameter case are shown in Table-2. By taking the residual rigidity into consideration, and modeling the residual rigidity of the exterior walls and floor slabs, a trend was seen in which localized stress concentration (W2) is mitigated, and it was confirmed that there was no significant effect on the seismic safety of the spent fuel pool.

As a reference, the details of the results of the seismic safety assessment of the spent fuel pool in the parameter case are shown in Table-3 to Table-6.

Table-2 Comparison of Occurring Strain and Occurring Stress in Relation to Assessment Reference Value (Testing Ratio*1)

	Location	Assessed item	Basic case	Parameter case
Pool wall	W1	Reinforcing bar strain	0.24	0.22*2
		Concrete strain	0.05	0.06*2
	W2	Antiplane shear	0.61	0.43
Pool floor	S1	Reinforcing bar strain	0.05	0.05*2
		Concrete strain	0.13	0.13*2
	S2	Antiplane shear	0.46	0.48

*1：Values in table indicate that the assessment reference is met if less than 1.

*2：Strain in the pool walls and pool floor differs in the parameter case and basic case for locations where the ratio of the occurring strain and occurring stress in relation to the assessment reference value (testing ratio) is maximized, and the strain in the parameter case in the above table is not the value of a location where the testing ratio is the maximum, and the value of the location is the same as in the basic case. Locations where the testing ratio is the maximum in the parameter case are S1” in the floor and W1” in wall (A) shown in Figure-5, and the testing ratio is 0.24 for the reinforcing bar strain of W1” and 0.13 for the concrete strain of S1”.

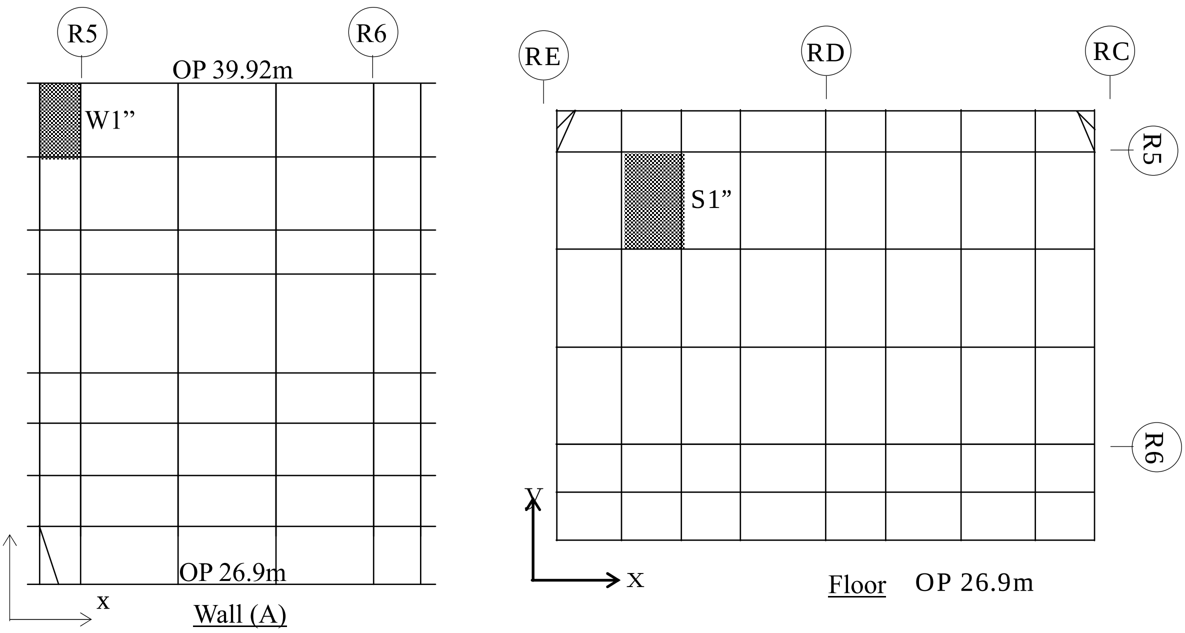


Figure-5 Locations where Testing Ratio of Strain is Maximum in Parameter Case (W1” and S1”)

【Parameter Case】

Table-3 Results of Examination of Strain of Concrete and Reinforcing Bars due to Axial Force and Bending Moment (Walls)

Name of location	Examined strain	Name of load time	Occurring strain ($\times 10^{-6}$) ε	Assessment reference value ($\times 10^{-6}$) ε'	Testing ratio ε／ε'	Finding
W1	$_c\varepsilon_c$	Ss earthquake time	-180	-3000	0.06≦1	Pass
	$_s\varepsilon_c$		-90	-5000	0.02≦1	Pass
	$_s\varepsilon_t$		1080	5000	0.22≦1	Pass
W1”	$_c\varepsilon_c$	Ss earthquake time	-320	-3000	0.11≦1	Pass
	$_s\varepsilon_c$		-240	-5000	0.05≦1	Pass
	$_s\varepsilon_t$		1200	5000	0.24≦1	Pass

Table-3 Results of Examination of Strain of Concrete and Reinforcing Bars due to Axial Force and Bending Moment (Floors)

Name of location	Examined strain	Name of load time	Occurring strain ($\times 10^{-6}$) ε	Assessment reference value ($\times 10^{-6}$) ε'	Testing ratio ε／ε'	Finding
S1	$_c\varepsilon_c$	Ss earthquake time	-370	-3000	0.13≦1	Pass
	$_s\varepsilon_c$		-150	-5000	0.03≦1	Pass
	$_s\varepsilon_t$		240	5000	0.05≦1	Pass
S1”	$_c\varepsilon_c$	Ss earthquake time	-370	-3000	0.13≦1	Pass
	$_s\varepsilon_c$		-200	-5000	0.04≦1	Pass
	$_s\varepsilon_t$		180	5000	0.04≦1	Pass

Table-5 Results of Examination of Antiplane Shear (Walls)

Name of location	Name of load time	Occurring stress Q (N/mm)	Assessment reference value Q' (N/mm)	Testing ratio Q／Q'	Finding
W2	Ss earthquake time	790	1860	0.43≦1	Pass

Table-6 Results of Examination of Antiplane Shear (Floors)

Name of location	Name of load time	Occurring stress Q (N/mm)	Assessment reference value Q' (N/mm)	Testing ratio Q／Q'	Finding
S2	Ss earthquake time	600	1270	0.48≦1	Pass

Seismic Response Analysis of Vertical Direction for Reactor Building

In performing the three-dimensional FEM analysis of the spent fuel pool, the dynamic analysis results of the vertical direction using reference seismic motion Ss are used as input. Here, the results of the seismic response analysis of the vertical direction are shown.

In preparing the analytical model, a range which is the same as the range arranged in Attachment-2 was treated as the damage range, and the mass point weight found in Attachment-3 is used.

The building analytical model for vertical direction is shown in Figure-1 and the specifications are shown in Table-1.

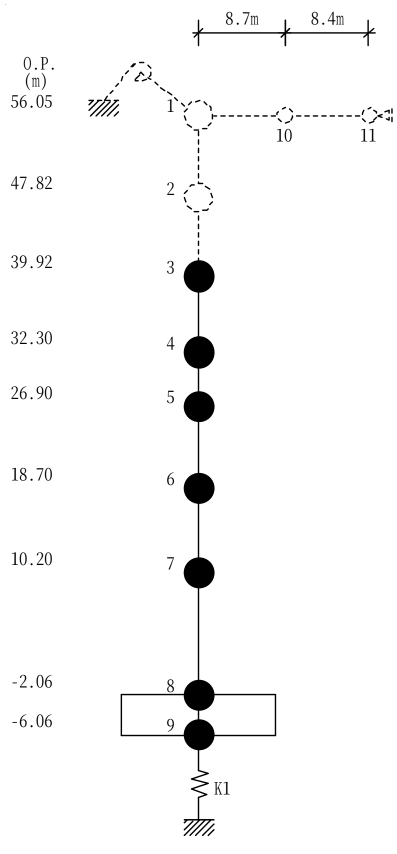


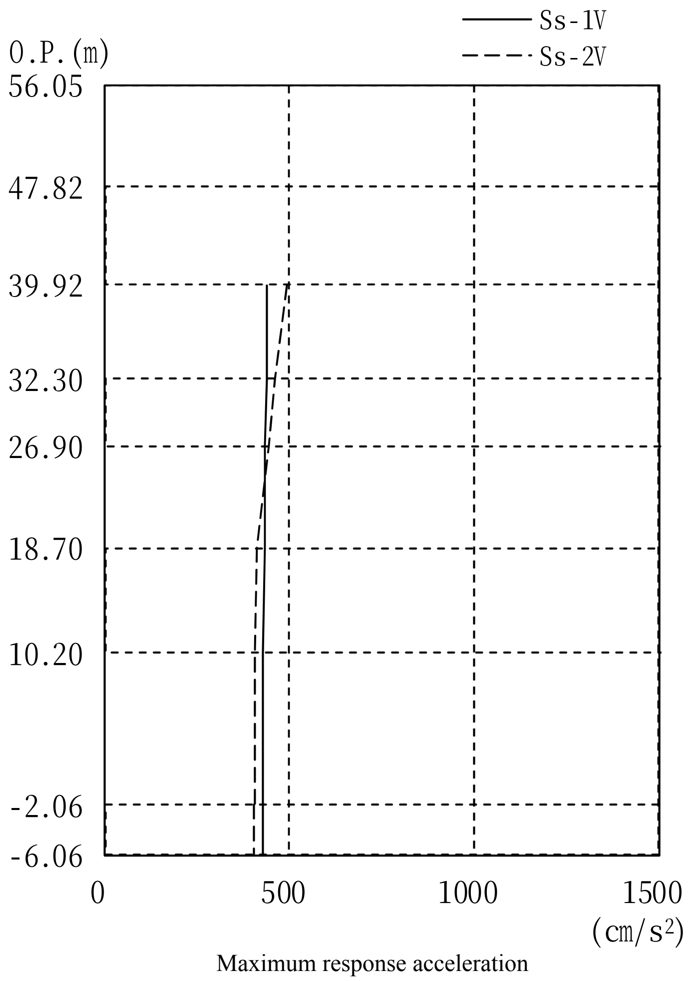
Figure-1 Building Analytical Model (Vertical Direction)

Table-1 Specifications of Building Analytical Model (Vertical Direction)

Mass point no.	Mass point weight W(kN)	Axial cross section A_N (m^2)	Axial spring rigidity K_A ($\times 10^8$kN/m)
1	—		
		—	—
2	—		
		—	—
3	69,940		
		204.5	6.90
4	87,140		
		210.7	10.03
5	127,760		
		354.5	11.11
6	129,030		
		340.6	10.30
7	218,480		
		654.7	13.72
8	353,740		
		2,812.6	180.71
9	132,390		
Total	1,118,480		

Young coefficient *Ec*	2.57×10^7 (kN/m2)
Shearing elastic coefficient *G*	1.07×10^7 (kN/m2)
Poisson's ratio *v*	0.20
Attenuation *h*	5%
Basic configuration	49.0m (N-S direction) X 57.4m (E-W direction)

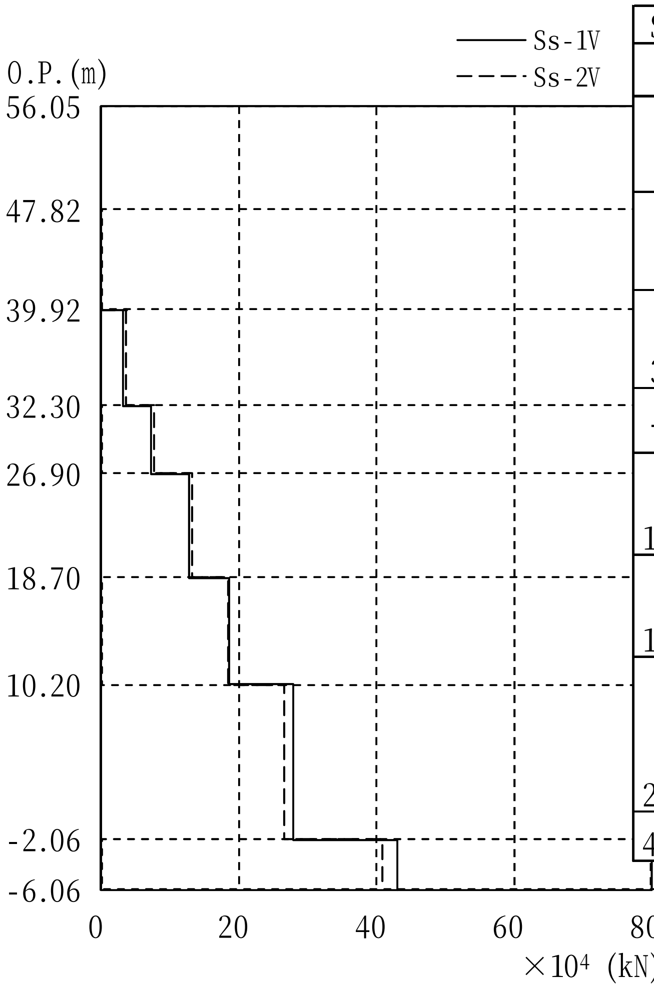
The maximum response acceleration and maximum response axial force for the vertical direction found using seismic response analysis are shown in Figure-2 and Figure-3.



(cm/s^2)

Ss-1V	Ss-2V
437	492
437	462
436	442
432	410
429	404
427	404
427	404

Figure-2 Maximum Response Acceleration (Vertical Direction)



×10^4(kN)

Ss-1V	Ss-2V
3.14	3.53
7.04	7.65
12.75	13.20
18.46	18.53
27.94	26.43
43.10	40.80

Maximum response axial force

Figure-3 Maximum Response Axial Force (Vertical Direction)