Transactions, **SMiRT-25**
Charlotte, NC, USA, August 4-9, 2019
Division V

EXPERIMENTAL STUDY ON INITIAL STIFFNESS DEGRADATION AND ITS EFFECT ON SEISMIC CAPACITY OF SHEAR WALL WITH HIGH REINFORCEMENT VOLUME PART1: RESULTS OF LONG-TERM EARTHQUAKE OBSERVATION RECORD ANALYSIS AND SIMULATION ANALYSIS

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ABSTRACT

Simulation analyses and field investigations indicated that the degree of deformation of the rebars of Tohoku EPCO's Onagawa NPP Unit 2 reactor building from the Tohoku Region Pacific Coast Earthquake on March 11, 2011 (hereafter referred to as the "311 Earthquake") remained largely within the elastic range and did not reach the plastic range. Meanwhile, small residual cracks appeared in the shear walls on some floors, and observation records expressed tendency of decreasing in stiffness.

In addition to the effect of large seismic loads, which are among the factors responsible for such reduced initial stiffness, a slight correlation with deterioration from aging was recognized in the results of the analyses based on long-term earthquake observation records. Against the foregoing background, we conducted Seismic Resistance Experiments #1 and #2 on heavily reinforced shear walls characteristic of the actual reactor unit.

Because small residual cracks occurred in the reactor building, we conducted Seismic Resistance Experiment #1, a static load test, in order to understand the effect of such cracks on the ultimate strength and deformation capacity.

Meanwhile, the likely factors responsible for deterioration from aging include internal stress and cracks resulting from the drying shrinkage characteristic of concrete. Therefore, we conducted Seismic Resistance Experiment #2, another static load test, in order to understand the effect of drying shrinkage on initial stiffness and maximum strength.

This paper reports on the results of factorial analyses of the reduced initial stiffness, including simulation analysis results and the results of analyses based on long-term earthquake observation records. Then, Part 2 will provide a report on Seismic Resistance Experiment #1 followed by another report in Part 3 on Seismic Resistance Experiment #2.

SIMULATION ANALYSIS OF THE 311 EARTHQUAKE

We performed a simulation analysis of the 311 Earthquake to investigate the vibration characteristics of the building and to determine whether the structure of the building had been affected.

To calculate the input ground motion to the analysis model, the observation records on the reactor building basemat were entered into the linear analysis model of the basemat followed by application of the transfer function from the basemat to outside the soil spring beneath the building. The agreement between the analysis results and the observation records was examined by comparing them regarding the maximum response acceleration and acceleration response spectra on each floor of the reactor building.

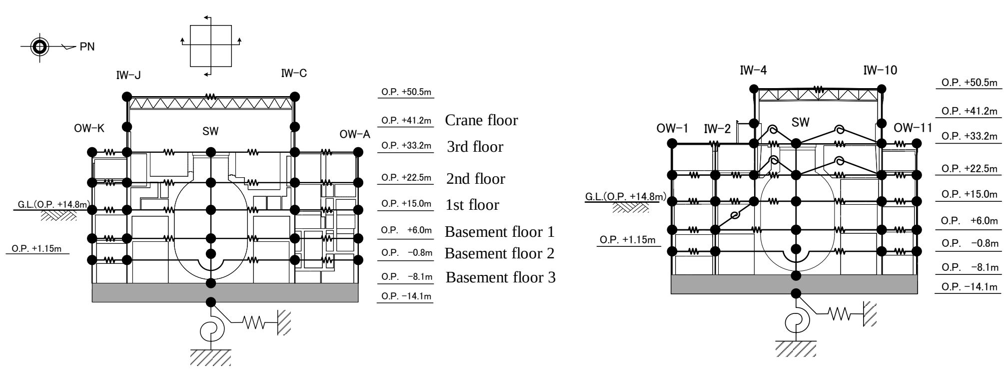
The analysis model presented in this paper was newly developed from the design analysis model by checking agreement between the observation records and analysis results :

1) To evaluate the stiffness of the reinforced concrete parts, that of the shear walls was set as equivalent stiffness to agree with the observation records and the first gradient for hysteresis characteristics was re-examined from the time of the design. In addition, we divided the building into parts above and below the operations floor (O.P.+33.2 m) to set an equivalent stiffness value for each part.
2) The damping ratio of the building was reset to be represented entirely by the damping of the reinforced concrete parts, so that the analysis results would agree with the observation records.

Figure 1 shows Simulation model, and **Table 1** shows Physical property values used for earthquake response analysis, and **Figure 2** shows Comparison of acceleration response spectra, and **Figure 3** shows Maximum response values on shear skeleton curve.

The simulation analysis results revealed the following tendencies:

1) Regarding the initial stiffness of the part below the operations floor, which is dominant over the vibration characteristics of the building: The analysis results showed good agreement with the records when the initial stiffness was set approximately 20 percent lower than the stiffness considered in the design. The stress-strain relationship calculated at the same time shows that the shear walls remained largely within the elastic range.
2) Regarding the part above the operations floor: The analysis results showed good agreement with the records when the initial stiffness was set approximately 50 to 70 percent lower than the stiffness considered in the design. As can be seen from the calculated stress-strain relationship, the response levels of the shear walls exceeded the first break point, disagreeing with the stress-strain curve assumed in the design.
3) Results in good agreement with the records were obtained by adopting an RC damping ratio of 7%.



[NS direction] [EW direction]

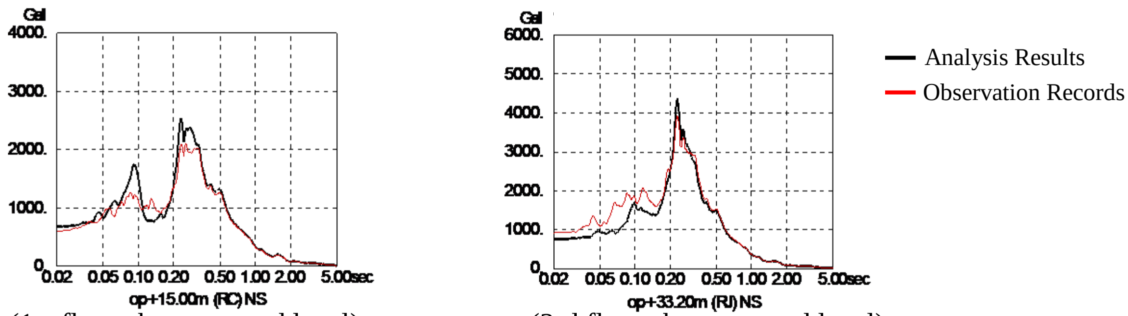
Figure 1 Simulation model of Reactor Building UNIT No.2 (horizontal direction)

Table 1 Physical property values used for earthquake response analysis

Material		Direction	Initial stiffness correction factor*	Young's modulus E (N/mm^2)	Shear modulus G (N/mm^2)	Damping ratio h (%)
Concrete	O.P.+50.5 m to O.P.+33.2 m	NS	0.3	7.94×10^3	3.41×10^3	7
		EW	0.5	13.24×10^3	5.69×10^3	7
	O.P.+33.2 m to O.P.-8.1 m	NS	0.75	19.86×10^3	8.53×10^3	7
		EW	0.80	21.18×10^3	9.10×10^3	7

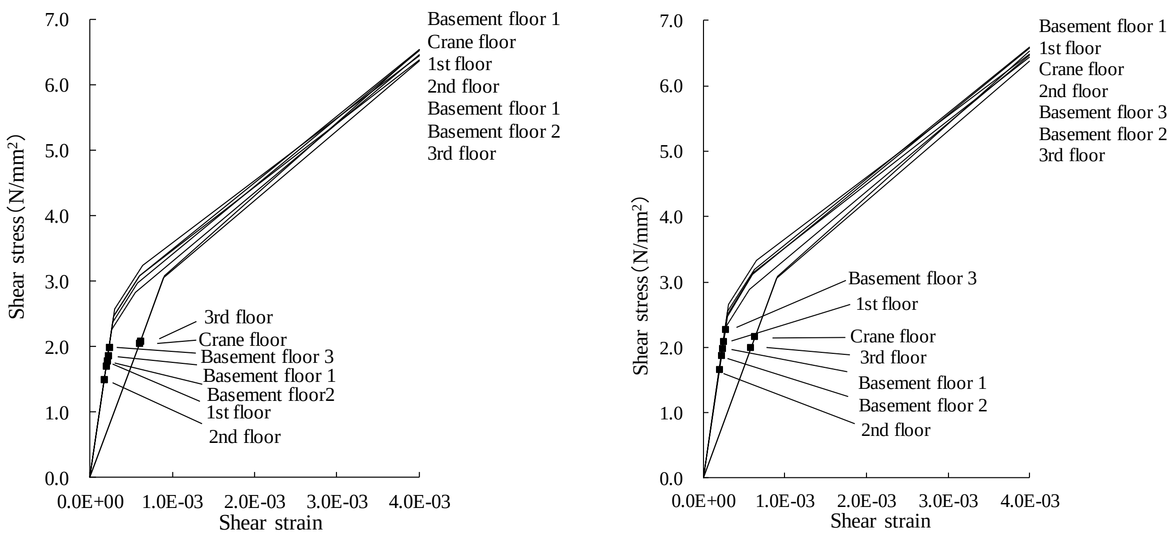
* These initial stiffness correction factors are for the design initial stiffness values of the concrete walls set to agree with the observation records.

Design values Young's modulus 2.65×10^4 N/mm^2
Shear modulus 1.14×10^4 N/mm^2



(1st floor above ground level) (3rd floor above ground level)

Figure 2 Comparison of acceleration response spectra
(Reactor Building UNIT No.2, the 311 Earthquake, NS direction)



(IW-J Line) (IW-C Line)

Figure 3 Maximum response values on shear skeleton curve
(Reactor Building UNIT No.2, the 311 Earthquake, NS direction)

OVERVIEW OF THE MEASUREMENT SYSTEM OF RELATIVE STORY DISPLACEMENT

We also developed the new system which measures the relative story displacement to directly estimate the damage of building against the earthquake. **Figure 4** shows overview of the measurement system of relative story displacement.

We installed this system in the reactor building of Onagawa NPP Unit No.2.Optical sensor, etc. is easy to be affected by local rotational displacement of the floor. This system is effective in ensuring accurate measurement. Putting steel wire at diagonal direction and vertical direction, measuring displacement of steel wire direction by displacement gauge. And calculate relative story displacement.

Figure 5 shows the measurement system. In this system, a ring-type displacement gauge is adopted. In addition, steel wire is given tension always by the spring.

This system was installed 3F and Crane Floor in Reactor building of Unit No.2. These floors are expected to be where the largest relative story displacement of all floors will occur. **Figure 6** shows a magnified view of the wall on the 3F and picture is appearance of displacement gauge.

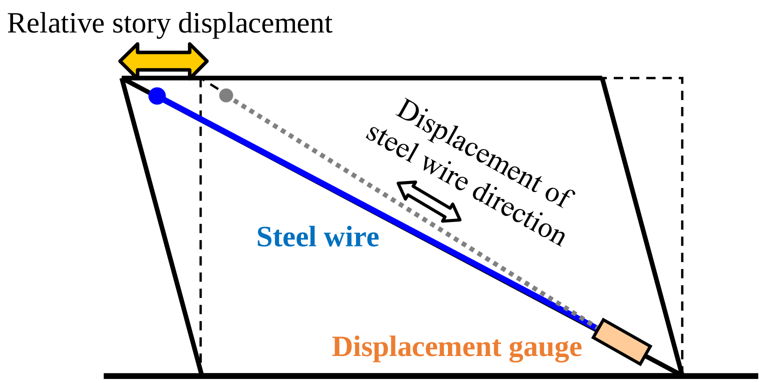


Figure 4 Overview of the measurement system of relative story displacement

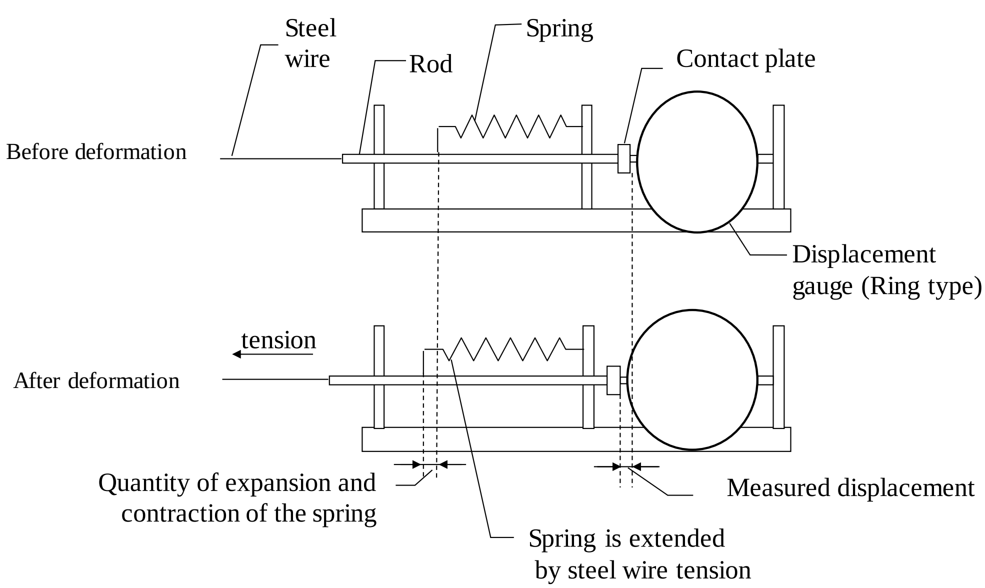


Figure 5 The measurement system

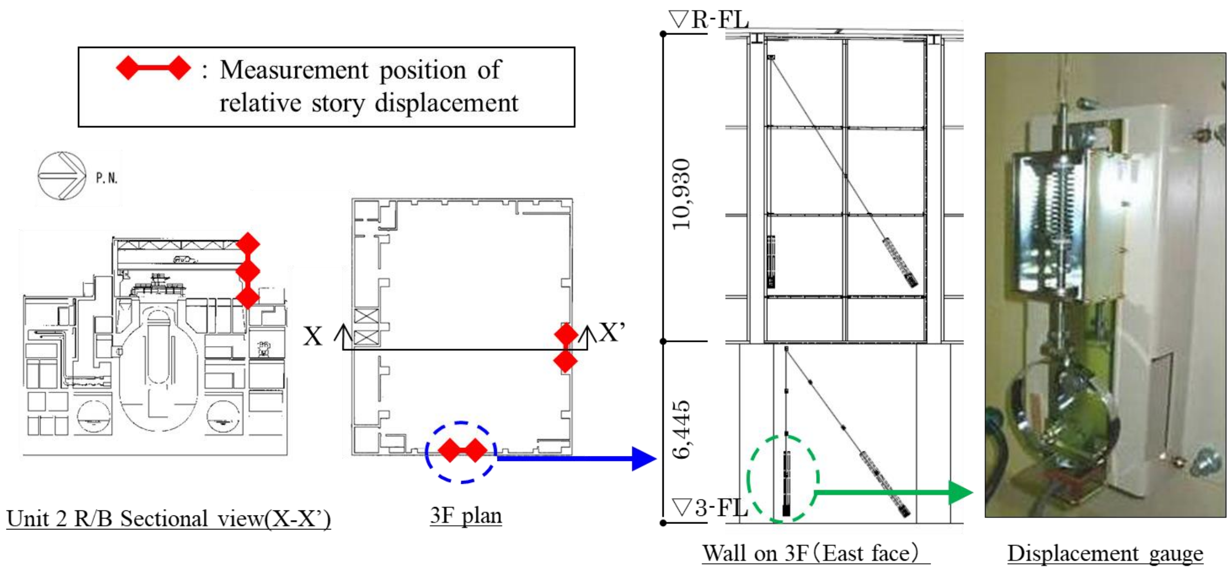


Figure 6 Situation of the installation

Figure 7 shows Comparison of simulation analysis results and relative story displacements on the 3F and CRF. We confirmed the observed records coincided with shear strain of simulation analysis of the reactor building roughly. According to the observed records, Maximum shear strain is about 500μ, there is enough margin against evaluation criteria (2000μ). Fourier amplitude shows a similar figure, so we considered this system is able to measure the relative story displacement accurately.

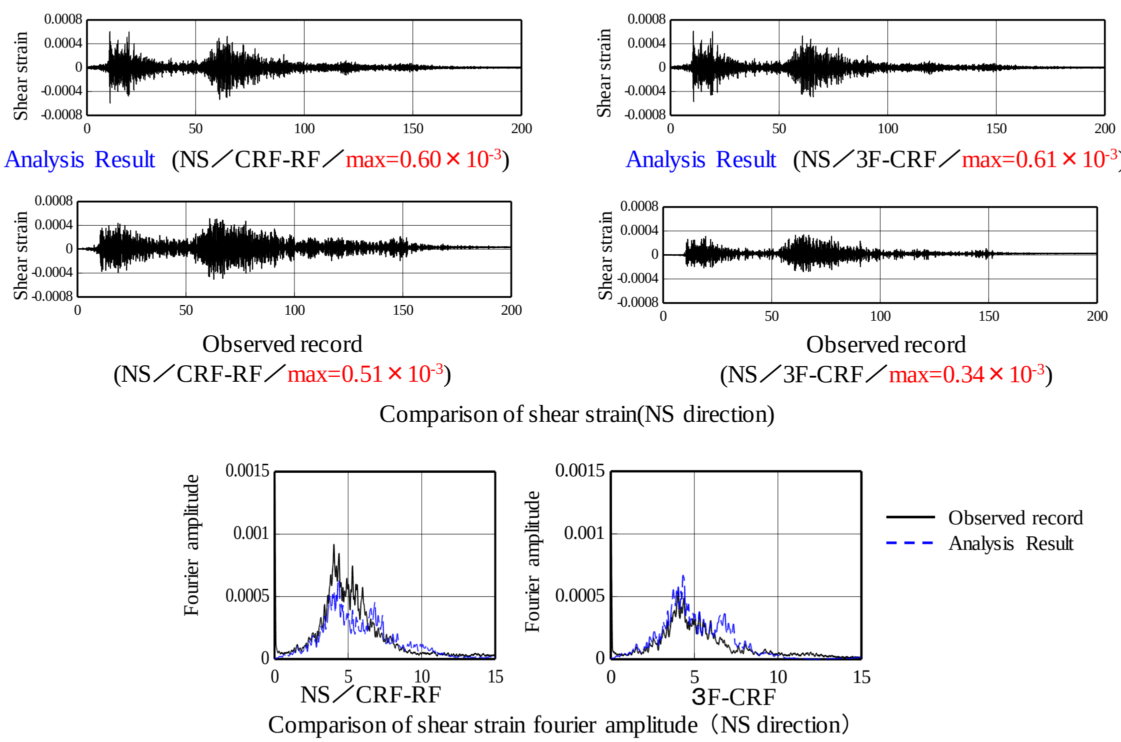


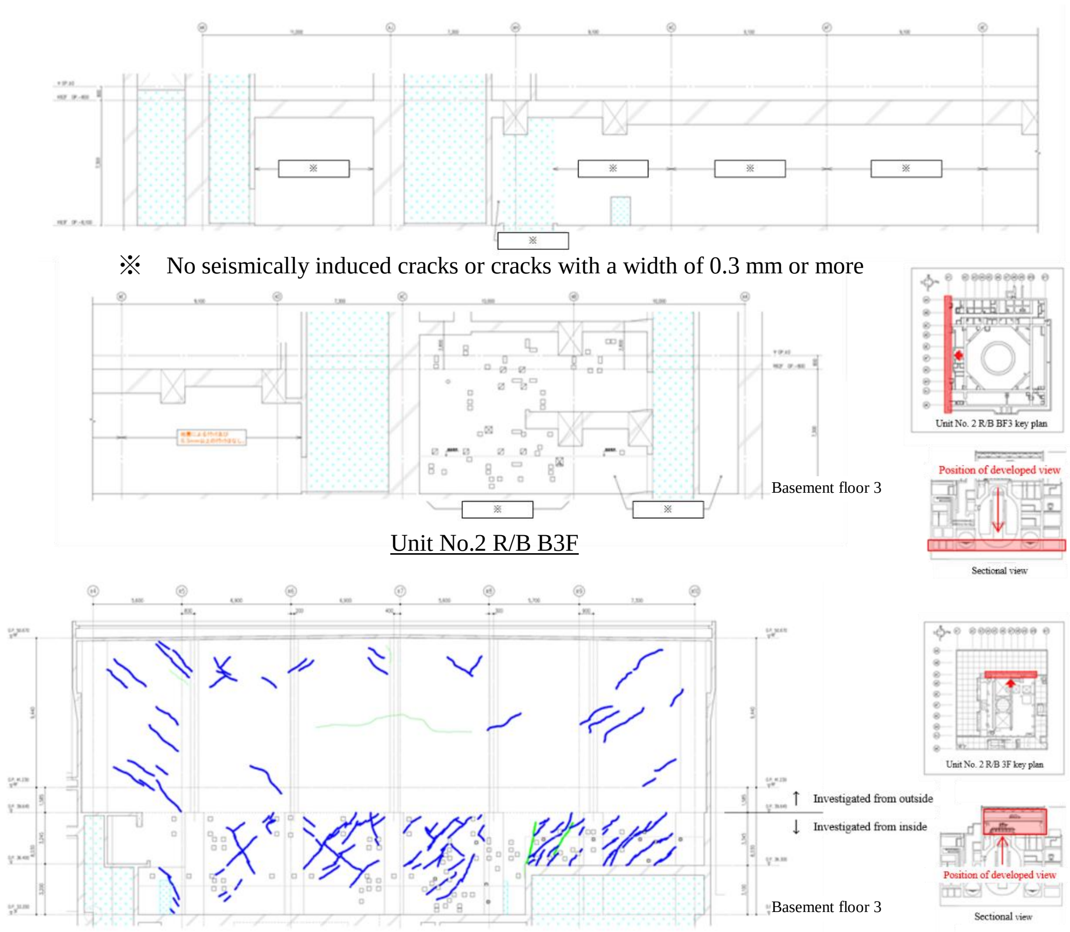
Figure 7 Comparison between simulation analysis results and Observed relative story displacement

IDENTIFICATION OF STRUCTURAL DAMAGES CAUSED BY THE 311 EARTHQUAKE AND OTHER EVENTS

Significant decreases in stiffness were observed after the 311 Earthquake. Hence, we inspected the Onagawa NPP Unit No. 2 reactor building for signs of structural damage (events likely to cause cracks with a width of 1.0 mm or more) to identify the specific effects that affected the reactor building and associated structures and to clarify the current health of these structures. The inspection was performed visually.

The inspection found almost no cracks on the basement floors. On the floors above the ground level, relatively many fine cracks measuring less than 0.3 mm in width were identified. We, however, detected no cracks with a width of 1.0 mm or more, which is the criterion for determining whether to perform a detailed evaluation.

Along with the above simulation analysis results, our additional simulation analysis and field investigation indicated that the degree of deformation of rebars remained largely within the elastic range and did not reach the plastic range. Meanwhile, small residual cracks appeared in the shear walls on some floors, and observation records expressed tendency of decreasing in stiffness.



Seismically induced		Non-seismically induced			
	Less than 0.3 mm				Width unknown
	0.3 mm or more to less than 1.0 mm		0.3 mm or more to less than 1.0 mm		Flaking
	1.0 mm or more		1.0 mm or more		Region inaccessible for inspection

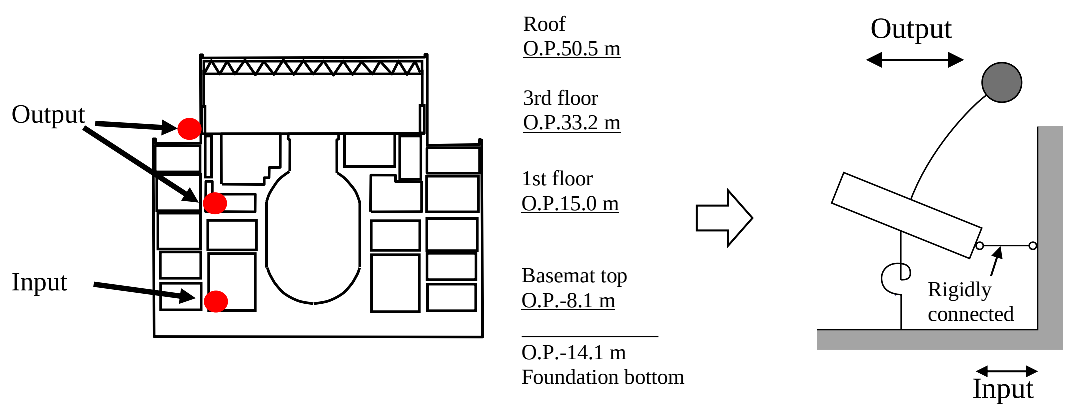
Figure 8 Inspection result

ANALYSIS OF DOWNWARD TREND IN STIFFNESS

The buildings on the site of the Onagawa Nuclear Power Plant are equipped with seismometers from which numerous earthquake observation records have been obtained. Using these observation records, the building was substituted by an equivalent single-mass system model to perform analysis of the changes in primary natural frequency from the start of the earthquake observation. The changes in primary natural frequency can be used to evaluate the changes in the average stiffness of the whole building.

Outline of the analysis method

When the on-basemat observation records of vertical motions of both ends of the basemat are available, the transfer function under a fixed foundation condition can be evaluated by evaluating the rotational motion of the basemat regarded as a rigid body. When on-basemat observation records of vertical motions of both ends of the basemat are unavailable, evaluation will be made of the transfer function under a fixed sway condition in which only the horizontal direction of the basemat is fixed. **Figure 9** shows the conceptual image of the natural frequency evaluation.



Arrengement of seismographs　　　　Fixed sway condition

1) The observed waveforms of the floors immediately above and below are linearly interpolated between the equivalent height positions in the single-mass system to calculate the waveforms at the equivalent height positions.
2) The natural frequency is determined by identification with the transfer function based on the observation records.

Figure 9 Conceptual image of natural frequency evaluation

Earthquakes selected for evaluation

While there are many earthquake observation records available for the Onagawa Nuclear Power Plant, we took into consideration the conditions of maximum acceleration, magnitude, and duration when selecting the records for use in this study. In other words, earthquake observation records were selected on the basis of how large the acceleration, magnitude, and recording duration registered therein were.

Evaluation results

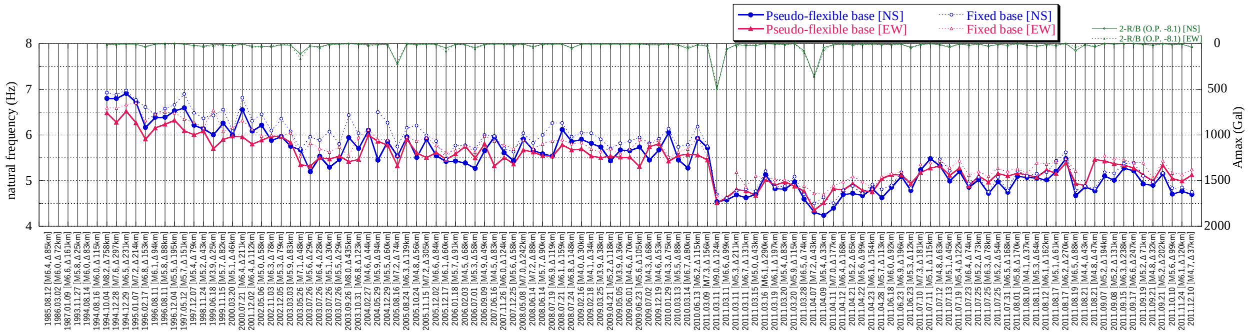
Regarding the Onagawa NPP Unit No. 2 reactor building, **Figure 10** shows changes in the primary peak frequency along the seismic time series and **Figure 11** shows changes in the primary peak frequency along the time axis. These figures are superimposed with changes in the primary peak frequency under fixed sway and fixed foundation conditions. Although the primary peak frequencies under the fixed sway condition were affected by the effect of rocking and hence appear slightly lower (by approximately 0.5 Hz on average) than those under the fixed foundation condition, the former and the latter show almost identical patterns of change.

Trends in the primary peak frequency go down gradually over time. The observed trend showed temporary drops due to high-acceleration earthquakes, such as those off the coast of Miyagi Prefecture on May 26, 2003, and on August 16, 2005, and recoveries after the subsequent aftershocks. When the Tohoku Region Pacific Coast Earthquake occurred, however, the trend in the primary peak frequency significantly dropped and failed to turn up back to the pre-quake level after the subsequent earthquakes.

Since around the second half of 2013, the frequency of the occurrence of earthquakes with an analyzable level of acceleration amplitude has been decreasing as compared with immediately after the Tohoku Region Pacific Coast Earthquake. Moreover, there has been an increase in earthquakes with small acceleration amplitudes among the earthquakes selected for analysis. Although there seems to be a slight upward trend in the primary peak frequency from immediately after the Tohoku Region Pacific Coast Earthquake, it is difficult to determine whether this trend is attributable to the recovery of building stiffness or to the amplitude dependency of primary peak frequencies.

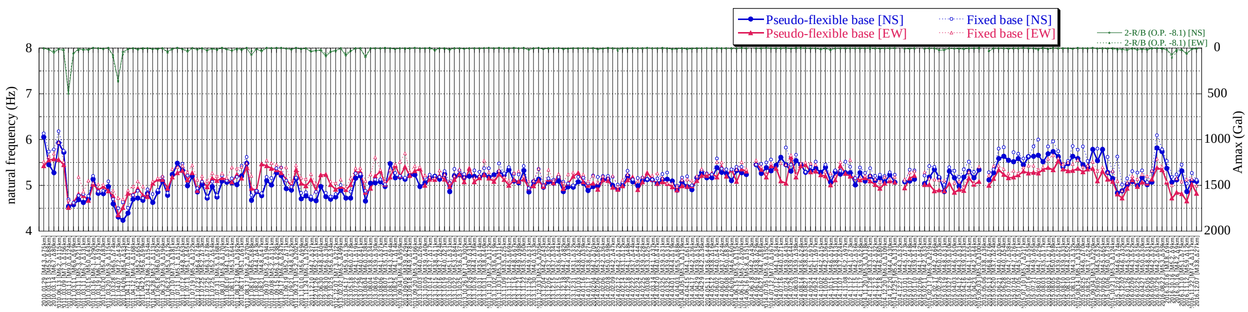
As can be seen in **Figure 12**, the changes along the time series of evaluated small-amplitude earthquakes suggest that the earthquake off the coast of Miyagi Prefecture on May 26, 2003 may have pushed the trend in the primary peak frequency down one step from the previous level. In spite of the earthquake off the coast of Miyagi Prefecture on August 16, 2005, however, the trend in the primary peak frequency seemed unchanged during the period from July 26, 2003, to January 29, 2010, immediately before the Tohoku Region Pacific Coast Earthquake on March 11, 2011. After the drop in frequency due to the Tohoku Region Pacific Coast Earthquake on March 11, 2011, no drops in frequency were observed during the subsequent period (from March 16, 2011 onwards).

Therefore, we consider that there appear to have been slight increases in the primary peak frequency after the Tohoku Region Pacific Coast Earthquake due to the amplitude dependency of the primary peak frequencies of earthquakes with small acceleration amplitudes.



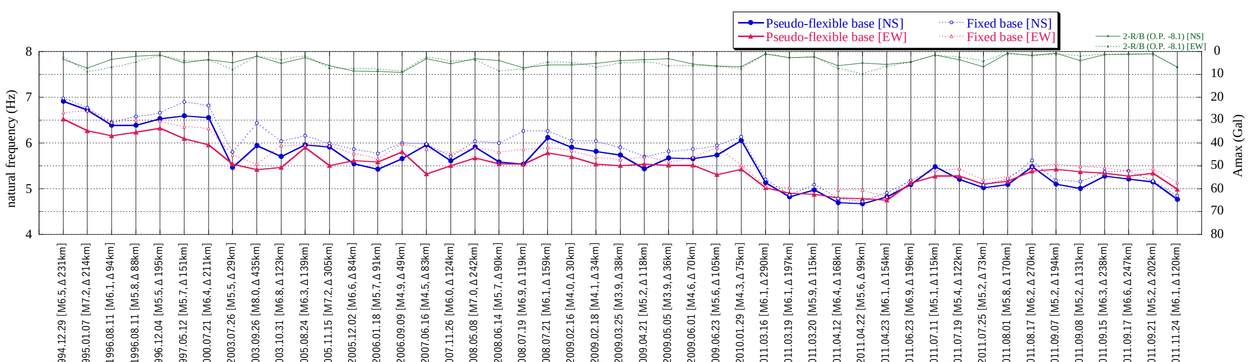
Natural frequency (design value): NS direction - 4.83 Hz; EW direction - 4.92 Hz

Figure 10 Peak frequency evaluation results (1994 to 2011)
(Onagawa NPP Unit No. 2 reactor building)



Natural frequency (design value): NS direction - 4.83 Hz; EW direction - 4.92 Hz

Figure 11 Peak frequency evaluation results (2010 to 2016)
(Onagawa NPP Unit No. 2 reactor building)



Natural frequency (design value): NS direction - 4.83 Hz; EW direction - 4.92 Hz

Figure 12 Peak frequency evaluation results (small-amplitude earthquake)
(Onagawa NPP Unit No. 2 reactor building)

CONCLUSION

A downward trend in stiffness was noticed as a result of the simulation analysis of the 311 Earthquake. While the reduced stiffness of the building is found correlated, albeit weakly, with secular changes as well as with the ground motion level, its correlation with the ground motion level is difficult to separate from its correlation with secular changes.

The part below the operations floor was found to be almost free from cracks, except for fine cracks caused by the seismic shock of the 311 Earthquake. Thus, it was confirmed that the building as a whole was not structurally damaged.

Therefore, while the large ground motion caused by the 311 Earthquake was partially responsible for the stiffness reduction due to the 311 Earthquake, the degree of seismic damage to the building suggests that the overlapping among factors other than ground motion may also have contributed to the initial stiffness reduction greater than expected when designed.

Reduced stiffness was also observed in buildings other than the reactor building. This suggests that overlapping may have occurred among the effects of factors common to all buildings other than ground motion, the characteristics of the concrete used, and other factors. Therefore, we consider it necessary to examine the effect of the concrete material used commonly.

Accordingly, we conducted a static load test as Seismic Resistance Experiment #1 to study the effect of small residual cracks found in the post-quake reactor building on the ultimate strength and deformation capacity.

Meanwhile, the likely factors responsible for deterioration from aging include internal stress and cracks resulting from the drying shrinkage characteristic of concrete. Therefore, we conducted the Seismic Resistance Experiment #2, another static load test, in order to understand the effect of drying shrinkage on initial stiffness and maximum strength. The results of these experiments will be reported in Part 2 and Part 3, respectively.

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