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IMPROVEMENT OF UNCERTAINTY FACTORS REGARDING INPUT SEISMIC MOTION AND CONSIDERING DYNAMIC NONLINEAR CHARACTERISTICS OF BUILDING ON FRAGILITY EVALUATION OF SEISMIC PRA

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ABSTRACT

The objective of this study is to improve the fragility evaluation of seismic PRA based on the introduction of new methods regarding uncertainty factors of input seismic motion and component fragility evaluation taking into account the dynamic nonlinear characteristics of buildings. Doing so will contribute to the establishment of a more realistic component fragility evaluation. In this study, an evaluation method of the uncertainty factors of input seismic motion has been applied. This method evaluates the uncertainty of spectrum shape for each PGA level by using a fault rupture model and seismic wave inventory considering hazard-consistent magnitude and hypocenter distance at target site. Moreover, the authors implemented new evaluation method regarding the component fragility evaluation which evaluates building response considering the dynamic nonlinear characteristics of buildings. The probability density function (PDF) of seismic response of a component provides a combination of the floor acceleration response spectra in the component natural period and the component response factor, which includes uncertainty expressed in logarithmic standard deviations. Failure probability is evaluated by the conditional probability of failure in the fragility evaluated in each acceleration level, which is obtained as a PDF of realistic response exceeding the PDF of realistic capacity. In order to apply the proposed methods, the dynamic nonlinear seismic response analysis model with a high degree of accuracy over the short period domain was required. Authors proposed a hybrid seismic response model which consists of the soil and foundation of the building evaluated by FEM, and the building was evaluated by the SR model. This model can evaluate more realistic building uplift behavior. As a result, a specific example of component fragility evaluation regarding more realistic consideration of nonlinear building characteristics was presented.

INTRODUCTION

Industry-based initiatives in voluntary efforts for safety enhancement utilizing Probabilistic Risk Assessment (PRA) have been conducted in Japan. The Nuclear Risk Research Center (NRRC) in the Central Research Institute of Electric Power Industry (CRIEPI) was established in October 2014 to organize and develop modern PRA methods for nuclear operators and the nuclear industry in order to improve the safety of nuclear facilities.

During past major earthquake disasters in Japan, nuclear power plant affected by exceeding design seismic motion levels have been observed. Moreover, in the 2011 off the Pacific coast of Tohoku Earthquake, several nuclear power plants having nonlinear response of buildings due to the large earthquake inputs occurred. Currently in Japan, seismic building response analysis has been conducted using nonlinear lumped mass analysis models. Meanwhile, component fragility evaluation has assumed a linear building response based on the existing method. (e.g., Separation of Variables Method (EPRI, 2002)). The existing method was used for practical seismic PRA on a global basis regarding conventional evaluation of nuclear power plants.

As for the conventional fragility evaluation, the seismic input level can be assumed to be a linear response to design response expressed by Peak Ground Acceleration (PGA). However, depending on the relationship between the natural frequency of the component and building floor response spectrum, the failure probability of the component may either increase or decrease as building approaches a nonlinear response in the high acceleration level. In addition, setting a more realistic input seismic ground motion regarding building and component fragility evaluation is required.

GENERAL METHOD OF FRAGILITY EVALUATION

Fragility $F(\alpha)$, which is evaluated as a conditional probability where the probability density function (PDF) of realistic response $f_R(\alpha, x)$ exceeds the PDF of realistic capacity $f_S(x)$, is assumed to follow logarithmic standard distribution consisting of a median and logarithmic standard deviation. Where α is a seismic level, such as peak ground acceleration, x is a component response, such as stress, displacement, etc. Fragility $F(\alpha)$ evaluates each seismic level of α as shown in Eq. (1).

$$F(\alpha) = \int_0^\infty f_R(\alpha, x_R)\left(\int_0^{x_R} f_S(x)dx\right)dx_R \tag{1}$$

PDF of the realistic response $f_R(\alpha, x)$ is represented by the following equation as a lognormal distribution, consisting of median $R_m(\alpha)$ and logarithmic standard deviation $\beta_R(\alpha)$.

$$f_R(\alpha, x) = \frac{1}{\sqrt{2\pi}\beta_R(\alpha)\cdot x}\exp\left\{-\frac{1}{2}\left(\frac{\ln(x/R_m(\alpha))}{\beta_R(\alpha)}\right)^2\right\} \tag{2}$$

Meanwhile, PDF of the realistic capacity $f_S(x)$ is represented by the following equation as a lognormal distribution, consisting of median S_m and logarithmic standard deviation β_S.

$$f_S(x) = \frac{1}{\sqrt{2\pi}\beta_S\cdot x}\exp\left\{-\frac{1}{2}\left(\frac{\ln(x/S_m)}{\beta_S}\right)^2\right\} \tag{3}$$

With the conventional simple method of estimating fragility parameters of response, it is convenient to work in terms of a random variable which consists of conservativeness and uncertainty of response through the response factor. In this method using the response factor, a conservativeness underlying the design response is converted into a coefficient as a response factor, and a realistic response is evaluated using the design response as well as a response factor. The response factor consists of four sub factors, F1 (Seismic motion), F2 (Soil response), F3 (Building response) and F4 (Component response) as shown in Fig. 1.

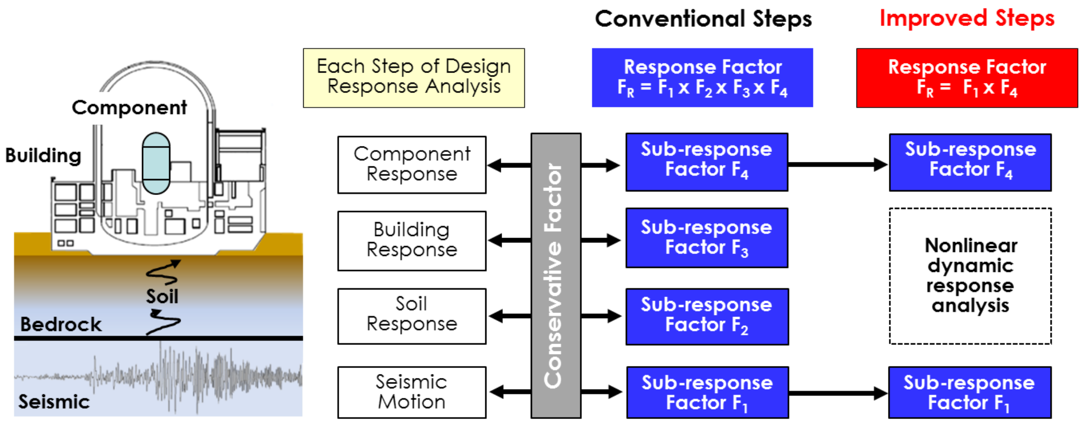

Figure 1. Response factor in evaluating seismic fragility of component

This paper describes the development of more realistic evaluation method regarding the sub factors of seismic motion as well as the provision of a solution in the implementation of the proposed method regarding the component fragility evaluation which evaluates building response while taking into account the dynamic nonlinear characteristics of buildings.

IMPROVEMENT OF FRAGILITY INPUT SEISMIC MOTION AND ITS UNCERTAINTY

Viewpoint of improving a conventional method

Based on the concept of failure point in the structural reliability theory, seismic ground motion for evaluating the fragility for each PGA level can be defined as a seismic motion contributing maximum damage of the target component at the set PGA level within the family of seismic motion observed at nuclear power plants in the future. In the conventional method as described above, a single design seismic wave data is mainly used for the input seismic motions while evaluating component fragility.
In recent years in Japan, realistic seismic motion for each PGA level has been formulated based on a uniform hazard spectrum (UHS) in the fragility evaluation of buildings. In this regard, however, variation is not generally taken into account, as it is included in the hazard evaluation. When a UHS wave is used as a realistic input wave, the UHS wave is not the average image of the seismic motion spectrum which can be observed in the future because it has power in all period domain.
In this study, the authors proposed a framework regarding creation of an input seismic motion for evaluating median and aleatory uncertainty of input seismic motion, in which the family of seismic wave data is developed from a fragility input seismic motion database based on the selection of magnitude and hypocentre distance corresponding to the exceedance frequency for each set PGA.

Evaluation procedure of proposed method

The authors proposed a new method which can evaluate a more realistic seismic motion factor (F1) (Ebisawa, Yamada et.al. 2016). This method takes into account the uncertainty of the spectrum shape for each PGA level as shown in Fig.2, and is defined as aleartory uncertainty βr. Where α is PGA (Gal) at engineering bed rock and T (sec) natural period of structure and component. This method evaluates uncertainty of spectrum shape for each PGA level by using a fault rupture model and is composed from the following seventh steps as shown in Fig.2.

1) Select target site considering soil hardness of bedrock and earthquake type,
2) Evaluate hazard-consistent magnitude ($\overline{M}$) and distance (Δ) at target site,
3) Set fault parameters of fault rupture model corresponding to ($\overline{M}$) and (Δ),

4) Generate seismic time history waves based on fault rupture model,
5) Prepare a database (DB) of the seismic wave family by storing the above seismic waves,
6) Select target seismic waves for each PGA from the above DB,
7) Evaluate F1 (α) (median and βr as aleartory uncertainty) considering the correlation between frequency characteristics of seismic wave based on the selected target seismic wave family.

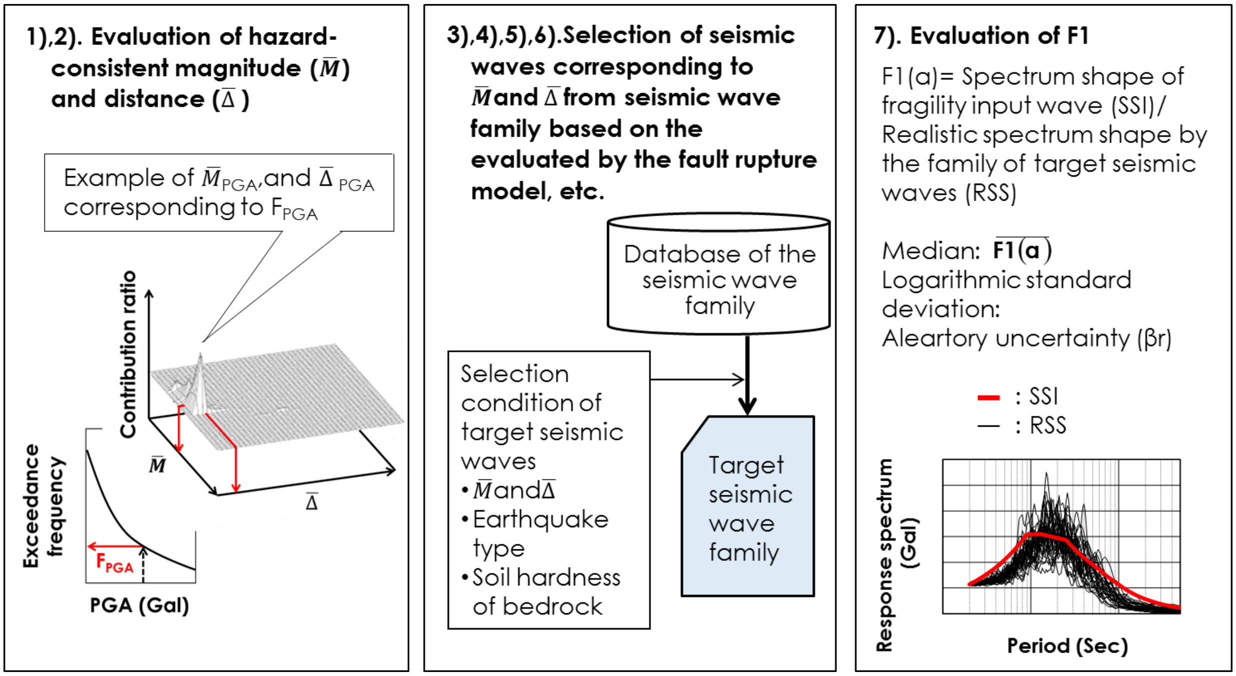

Figure 2. Procedure for evaluating F1 using seismic motion inventory

FRAGILITY EVALUATION CONSIDERING DYNAMIC NONLINEAR BUILDING RESPONSE

Viewpoint of improving conventional method

By characterizing the component fragility through a family of fragility curves, the analyst has expressed all his knowledge about the seismic capacity of the component along with the uncertainties. Given the same information, two analysts with similar experience and expertise would produce approximately the same fragility curves. Development of the family of fragility curves using different failure models and parameters for a large number of components in a seismic PRA is impractical if it is done as described above (EPRI, 2002). The buildings and structures that should be considered when conducting SPRA of a nuclear power plant are the reactor building, auxiliary building, and outdoor structures. On the other hand, the number of evaluated components through the SPRA consists from 200 to 400 in the plant. If each component is evaluated for seismic response on the each PGA level, the procedure of fragility evaluation would involve an excessive amount of calculations, making it totally impractical from the perspective of the practical application of SPRA. Therefore, a practical and reasonable component fragility evaluation method for considering dynamic nonlinear building response is required.

Evaluation flow and equation

The authors developed a novel component fragility evaluation method considering dynamic nonlinear characteristics of the building (Yamada, Miura and Ebisawa, 2017). A flowchart of the fragility evaluation method considering dynamic nonlinear characteristics of the building is shown in Fig.3.

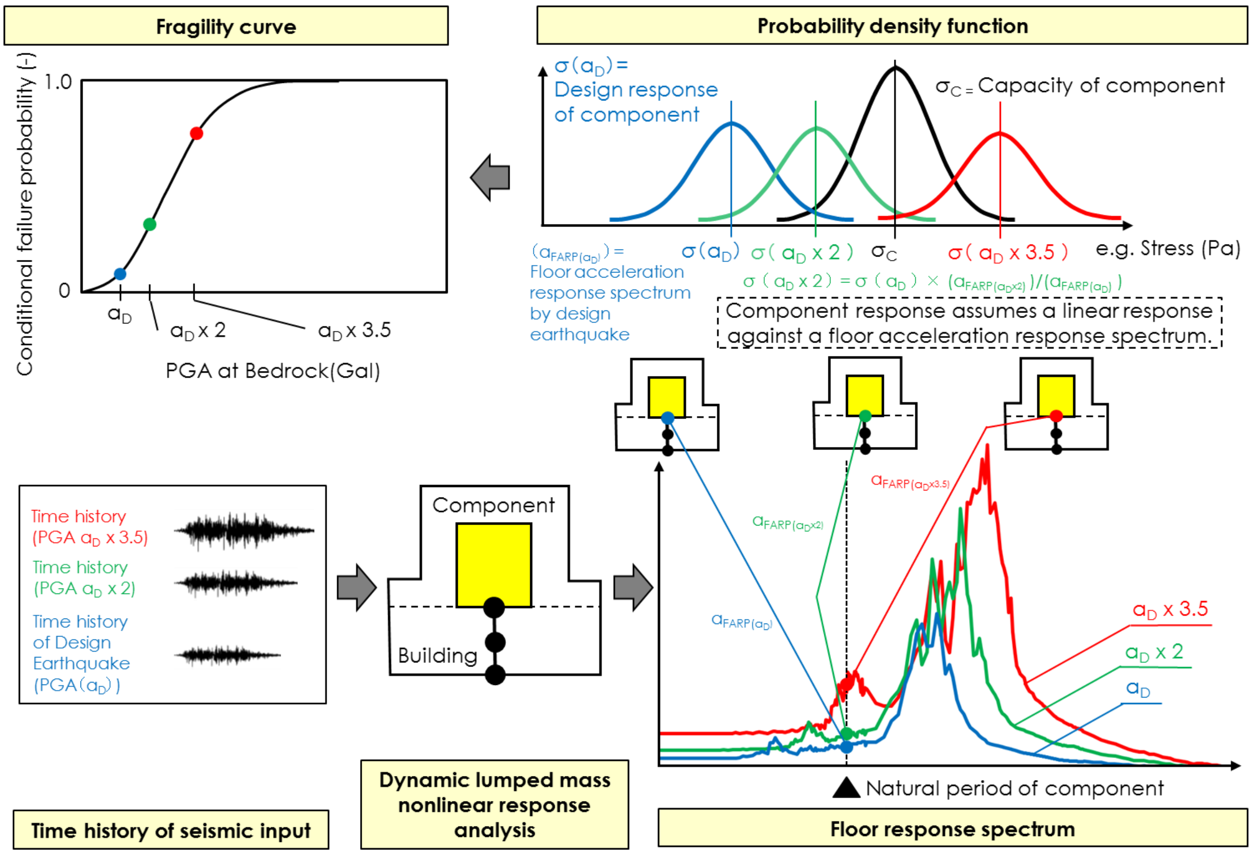

Figure 3. Concept of component fragility evaluation considering dynamic nonlinear building response

Evaluation equation for computation of component fragility by the proposed method

The lognormal distribution curves of seismic response of components are provided from the floor response spectrum (peak acceleration of floor) in the component natural period, which included median of a realistic response (excluding a conservative factor) and uncertainty, and response factor F1 and F4. The lognormal distribution curve of realistic component capacity provides component capacity value based on the shaking table test, material structural strength data, etc., which includes median of realistic capacity and uncertainty.

Component fragility considering dynamic nonlinear building response $F_C(\alpha)$ is evaluated as a conditional probability where the PDF of realistic component response $f_{CR}(\alpha, x_R)$ exceeds the PDF of realistic component capacity. $f_{CR}(\alpha, x_R)$, and $f_{CS}(x)$ are assumed to follow logarithmic standard distribution consisting of a median and logarithmic standard deviation. Where α is peak ground acceleration, x is response of the component based on the floor seismic response on the installation position of the intended component. Fragility $F_C(\alpha)$ evaluated each peak ground acceleration level of α, is shown in Eq.(4).

$$F_C(\alpha) = \int_0^x f_{CR}(\alpha, x_R) \left\{ \int_0^{x_R} f_{CS}(x) dx \right\} dx_R \tag{4}$$

It assumes that the realistic component response $f_{CR}(\alpha, x)$ follows a logarithmic standard distribution of median and logarithmic standard deviation. The general expression is represented in the following equation.

$$f_{CR}(\alpha, x) = \frac{1}{\sqrt{2\pi}\beta_R x} \exp\left[-\frac{1}{2} \left\{ \frac{\ln(x / Q_m(\alpha))}{\beta_R} \right\}^2 \right] \tag{5}$$

Where, α is the seismic level of maximum peak acceleration at bedrock, x is index of failure evaluation, $Q_m(\alpha)$ is the median of building floor response corresponding to the seismic level α at the component installation position, and $\beta_R(\alpha)$ is uncertainty of component response corresponding to the seismic level α. Moreover, the ratio between building floor response and component response is as follows.

$$\zeta = \frac{Q_{CR}(\alpha_{DSL})}{Q_{FR}(\alpha_{DSL})} \tag{6}$$

Where, $Q_{CR}(\alpha_{DSL})$ is the component response at design seismic level, and $Q_{FR}(\alpha_{DSL})$ is the building floor response (maximum acceleration) at design seismic level. Component response is treated as a linear response. Therefore, the realistic component response is represented by a substitute Eq. (7), response factor F1 as well as F4 into Eq. (5) leads to the following.

$$f_{CR}(\alpha, x) = \frac{1}{\sqrt{2\pi}\beta_R(\alpha) \cdot x} \exp\left[-\frac{1}{2} \left\{ \frac{\ln\left(x / \frac{\zeta \cdot Q_m(\alpha)}{F_1 \cdot F_4}\right)}{\beta_R(\alpha)} \right\}^2 \right] \tag{7}$$

The uncertainty of component response $\beta_R(\alpha)$ is represented as follows.

$$\beta_R(\alpha) = \sqrt{\beta_{R_{FR}}(\alpha)^2 + \beta_{R(F_1)}(\alpha)^2 + \beta_{R(F_4)}{}^2} \tag{8}$$

Where, $\beta_{R_{FR}}(\alpha)$ is the uncertainty of building floor response corresponding to the seismic level α , and $\beta_{R(F_1)}(\alpha)$ is the uncertainty of the F_1 corresponding to the seismic level α. Meanwhile, the realistic component capacity $f_{CS}(x)$ is represented by the following equation as a PDF consisting of median S_m, logarithmic standard deviation β_S.

$$f_{CS}(x) = \frac{1}{\sqrt{2\pi}\beta_S x} \exp\left[-\frac{1}{2} \left\{ \frac{\ln(x / S_m)}{\beta_S} \right\}^2 \right] \tag{9}$$

In addition, uncertainty β consists of aleatory uncertainty β^r, and epistemic uncertainty β^u which is represented as follows.

$$\beta = \sqrt{\beta^{r^2} + \beta^{u^2}} \tag{10}$$

IMPLEMENTATION OF PROPOSED METHOD REGARDING INPUT SEISMIC MOTION

Development of fragility input seismic motions and its uncertainty

Authors have confirmed the feasibility of the proposed method. It was evaluated at the Ikata site in terms of model plant. Developed family of seismic waves originated from two seismic sources. Main target specific seismic source was active fault (Seismic magnitude 7.6) in the sea area near the site. Moreover, one regional seismic source was considered. The evaluation method of seismic motion is the fault rupture model based on a fault rupture recipe (Irikura, 2004). 9000 seismic time history waves generated each seismic sources based on the evaluation result of the seismic hazard regarding the Ikata site. The range of PGA which develops a target seismic wave family was from 300 Gal to 1500 Gal. 40 waves were obtained at around the design seismic motion level PGA from families of seismic time history waves, and their spectrum shapes are illustrated as gray bold lines as shown in Fig.4. The spectrum of design seismic motion is also described as a red bold line in Fig.4. Fig.4 illustrates the relationship among median ± β regarding the extracted 40 waves. Moreover, Fig.4 illustrates the uniform hazard spectrum (UHS) obtained from probabilistic seismic hazard evaluation. The green bold line in Fig.4 is the representative wave extracted from 40 waves as it was the closest to the median line. The median and logarithmic standard distributions were evaluated based on the spectral values for each seismic wave period. For almost all ranges of period in Fig.4, the median is smaller than the design wave and UHS. Additionally, in practical component fragility evaluations considering dynamic nonlinear characteristics of building, an input seismic time history wave data for fragility evaluation that is represent wave of beyond design level are required. As mentioned above, the representative wave in this method can become an input time history wave data for the fragility evaluation. The examples of the evaluation results are shown in Fig.5. Through these implementations, it was found that the above proposed method is reasonable and realistic way.

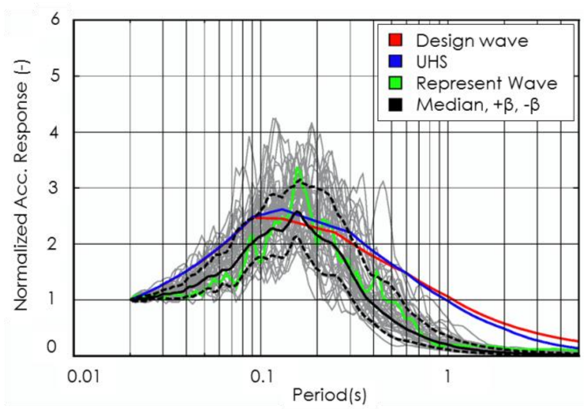

Figure 4. Design level fragility input seismic motions based on the proposed method

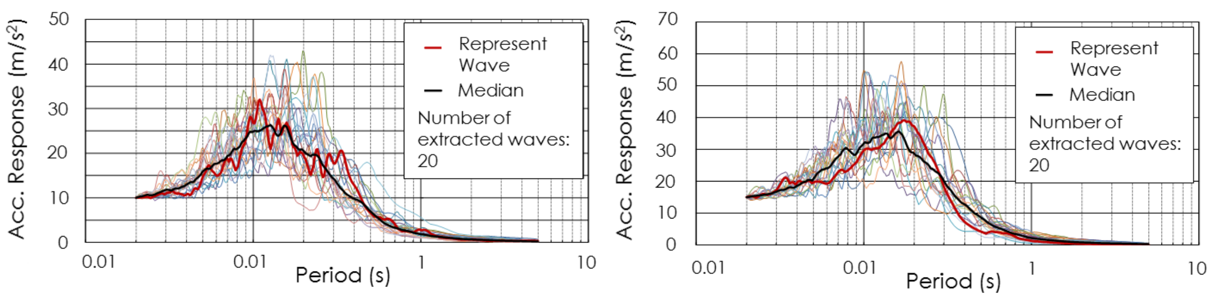

Figure 5. Fragility input seismic motions (Left: PGA1000Gal, Right: PGA1500Gal)

IMPLEMENTATION OF PROPOSED METHOD REGARDING BUILDING RESPONSE

Challenges faced in the implementation of proposed method and approach of its solution

The proposed method regarding component fragility evaluation considering dynamic nonlinear characteristics of building was applied to the model plant on a trial basis. The building seismic response analysis in the high acceleration input level, the evaluated result obtained by the building seismic response analysis at a high acceleration input level represented a conservative overestimation at sections close to a natural period domain of the component. This assumed to be caused by the uplift of building. Seismic fragility evaluation requires a building response analysis model that can obtain more realistic response results even at input ground motion levels greatly exceeding the design seismic level. For this reason, the authors proposed an idea of new model in order to create a more precise model regarding the interaction between the building and the ground, focusing on the response of the component natural period domain which consists of the ground and building foundation as an FEM, as well as the building as a lumped mass model (hereinafter, referred to as "hybrid model"). A general structure of hybrid model is shown in Fig. 6.

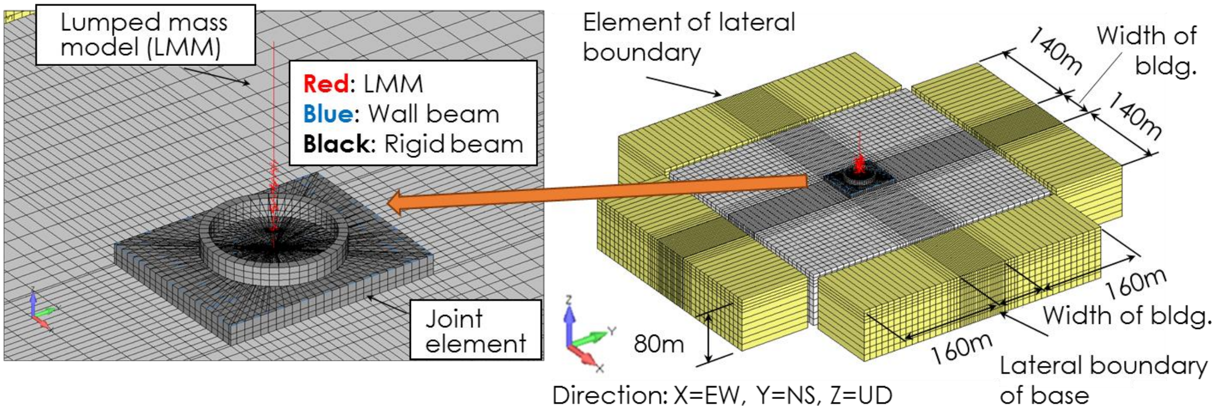

Figure 6. General description of proposed hybrid model regarding soil and building

Implementation result and consideration of proposed hybrid seismic response analysis model

The authors confirmed the feasibility of the presented hybrid seismic response analysis model. Moreover, they clarified the contributing factor of amplification close to the natural period domain of the component at the high acceleration input level. By simulating a previous study model using the presented hybrid model, good consistency was obtained. The previous model studied dynamic analysis and a shaking table test regarding uplift of buildings (Yano et.al., 1991). In addition, the validity of the model was confirmed by comparing the seismic response observation in the actual plant with the analysis value of the proposed hybrid model, and as well as by comparing the analysis values of the conventional SR model and the proposed hybrid model. A comparison example of analysis values of the SR model and the proposed hybrid model in the input exceeding the design seismic motion level are shown in Fig.7.

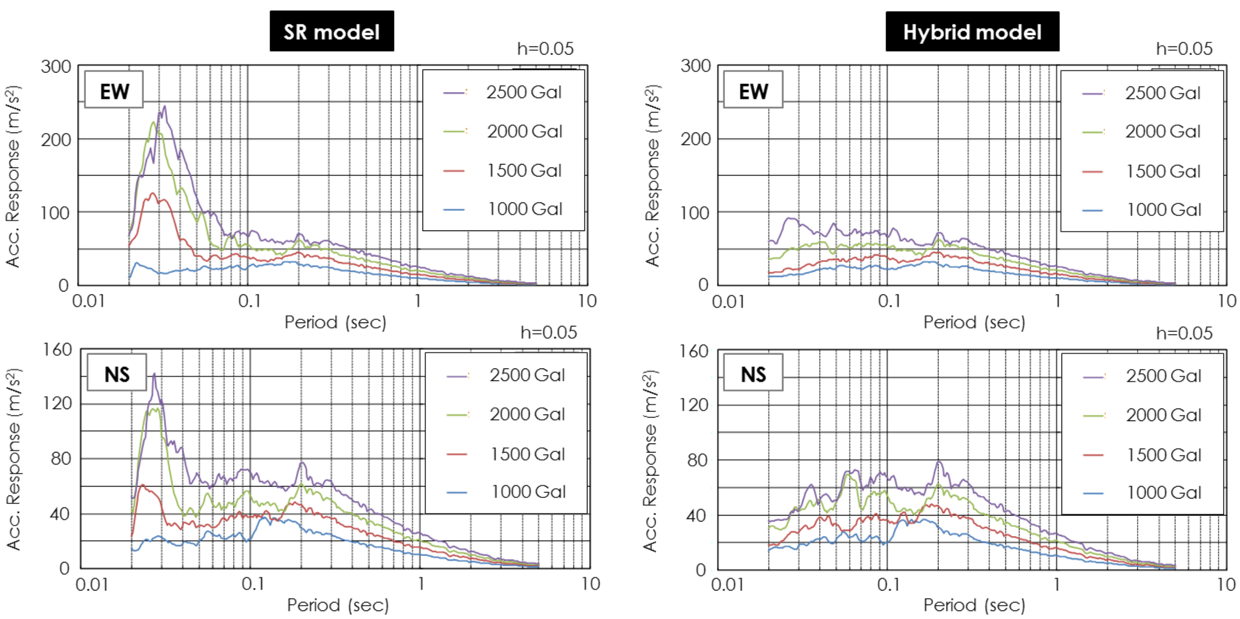

Figure 7. A comparison example of the analysis values of the SR model and the proposed hybrid model

The difference in the behavior of the uplift of buildings between EW and NS of the hybrid model is presumed to be caused by the difference in stiffness of the building foundation from the visualized result as shown in Fig.8.

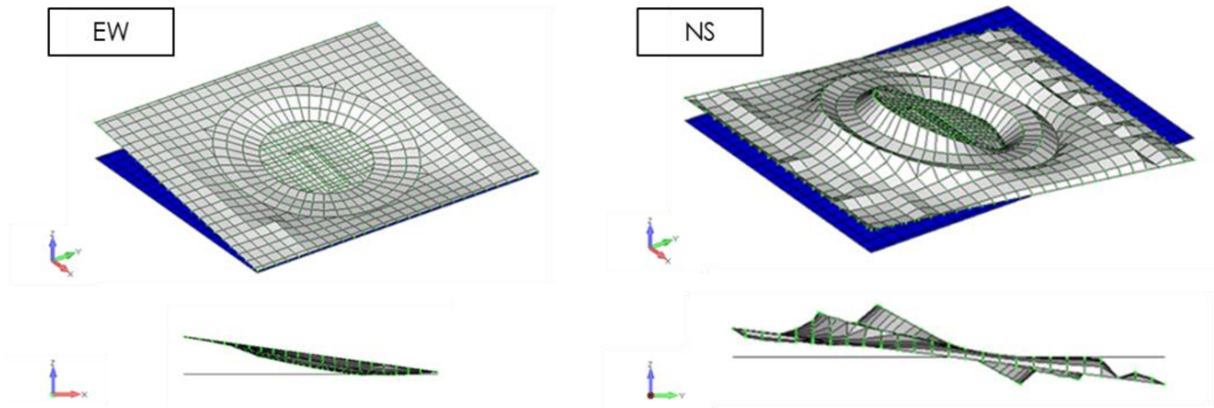

Figure 8. Deformation of the top edge of the building foundations in the maximum uplift of buildings

According to the previous study, when a building is uplifted, both in the analysis and the experimental test, vertical motion (induced vertical motion) is excited by even multiple harmonics of the building natural period in addition to the horizontal direction of seismic response being excited by odd number harmonics of the building natural period. The case study result of an analytical consideration regarding the influence of soil stiffness on odd number harmonics is shown in Fig.9. In this analysis, the soil stiffness and the acceleration input value are adjusted to the uplift level so as to be equal in value. The difference in the soil stiffness with blue line (Vs=2.6 km/s) and the red line (Vs=0.7 km/s) appears in the amplification difference of the odd number harmonic of the building natural period as shown in Fig.9.

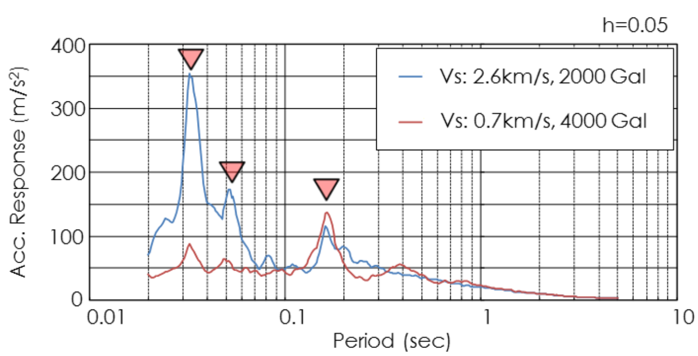

Figure 9. Influence of the uplift of building in terms of a soil stiffness and input seismic motion

Application of proposed methods for the component fragility evaluation

The fragility evaluation results by the proposed method tend to have a lower probability of damage compared to those by conventional methods (Separation of Variables Method). However, the probability of damage of a component in some natural period domain and a component affecting the damage mode by vertical seismic motion due to amplification arise from uplift of building was found to be larger than results using the conventional fragility evaluation method. Figure 10 shows an example of the evaluation results regarding an electrical panel considering the uncertainty of the seismic input motion and the dynamic nonlinear characteristics of the building.

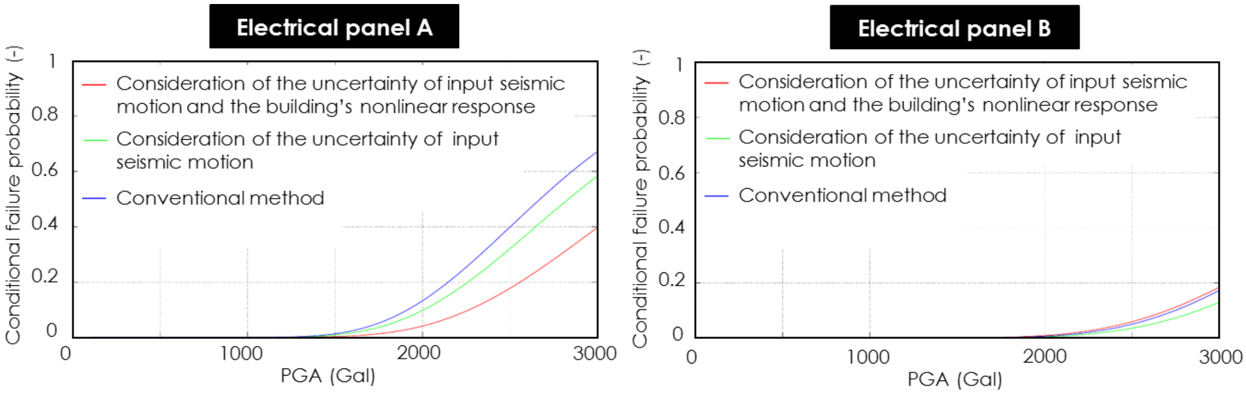

Figure 10. Fragility curve of proposed method and conventional method as an example component

CONCLUSION

In this study, improving the fragility evaluation of seismic PRA using new methods regarding uncertainty factors of input seismic motion and component fragility evaluation taking into account the dynamic nonlinear characteristics of buildings is proposed. Moreover, by evaluating the proposed methods to the model plant, it has been confirmed that the proposed methods could be applied practical ways. When conducting the component fragility evaluation at high acceleration levels exceeding design, it is important to pay attention to the amplification of the short period domain which overlaps the component natural period due to the uplift of the building in the seismic response analysis. In the future, to deepen understanding of more realistic phenomenon regarding the uplift of building not only experimental but analytical approaches are required.

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