PERFORMANCE-BASED DESIGN OF STEEL TANKS UNDER SEISMIC RISKS

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ABSTRACT :

The present study discusses about the seismic risk assessment of un-anchored steel tanks when a strong earthquake excitation develops a plastic deformation at the base plate due to a rocking motion of the tank. Current seismic design guidelines underestimate the seismic safety of buckling failure at the side wall or shell plate, because the stiffness degradation of the tank is overestimated with the structural characteristic factor D_s. The present study proposes an exact estimation approach of the seismic safety of buckling failure at the shell plate as well as the crack failure of the base plate. The seismic performance of the tank is also developed using a limit state design method to provide the fragility curves for the damage modes of the shell plate and base plates.

KEYWORDS: Steel tank, seismic assessment, elephant foot buckling, structural characteristic factor, performance-based design

1. INTRODUCTION

Un-anchored cylindrical steel tanks they are excited by the maximum considered earthquake (MCE) might exhibit rocking motion coupled with uplift behavior at the bottom plate. For this reason, the current design codes in the world take the uplifting effect into consideration. For instance, US design code[3)] of API Standard 650 assesses the possibility of uplifting behavior through the Anchorage Ratio of J-value. On the other hand, Japanese seismic design gidelines[1), 2)], unfortunately, assess the inelastic buckling behavior assuming that the bottom plate is always plastically elongated without any leakage during the seismic excitation. As the result, the current guidelines in Japan may underestimate the seismic safety of buckling failure at the shell plate, because the stiffness degradation of the tank might be overestimated with the structural characteristic factor Ds.

When the tank shows large inelastic deformation at the bottom plate, the impulsive seismic load will be degraded by its inelastic response characteristics. If the bottom plate would not damaged by this load, the seismic safety of EFB failure at the shell plate can be improved. So the inelastic response of the bottom plate without any leakage can bring a cost effective seismic design through a thinner wall thickness of the shell plate.

The present study proposes an exact estimation approach of the seismic safety of elephant foot buckling (EFB) failure at the bottom of the shell plate as well as crack failure of the bottom plate. Once the target probability of the tank failure is given for the performance-based design, the seismic design can be carried out in which both target probabilities for EFB failure at the shell plate and the crack failure at the bottom plate must be optimally allocated based on the reliability analysis for the multi-failure modes of the structural system.

For the seismic design purpose, the analytical formulations are developed on the limit state design method to provide the fragility curves for the damage modes of the shell plate and the bottom plate. Numerical studies for various profiles of steel tank will provide insight on the performance-based seismic design procedures of the steel tanks under seismic risks.

2. SEISMIC RESPONSE OF UN-ANCHORED STEEL TANKS

1.1. Damage mode of steel tank

Assuming that seismic load act on un-anchored steel tank installed on rigid foundation, the edge of base plate may be uplifted and shell plate in the opposite side may be damaged with buckling failure by compressive force as shown in Fig.1. This failure mode is called EFB. In this situation, liquid weight acts on uplifted base plate as uniformly distributed load and cause a tensile deformation of the base plate. Fig.2 shows detail of this section. In uplifting mode, tensile deformation occurs at inner region of the base plate near the edge. If the earthquake brings cyclic loads to this section, crack failure by fatigue may be occurred.

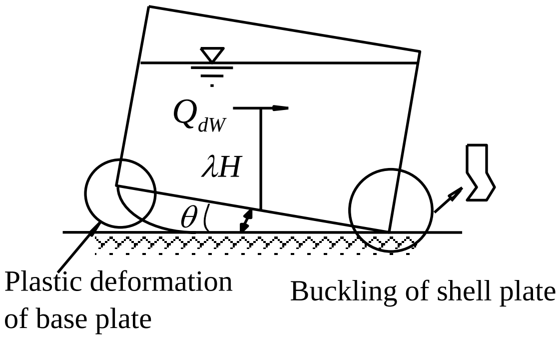


Fig.1 Uplifting behavior of steel tank on a rigid foundation

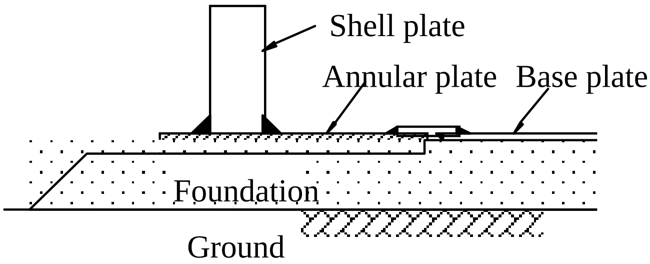


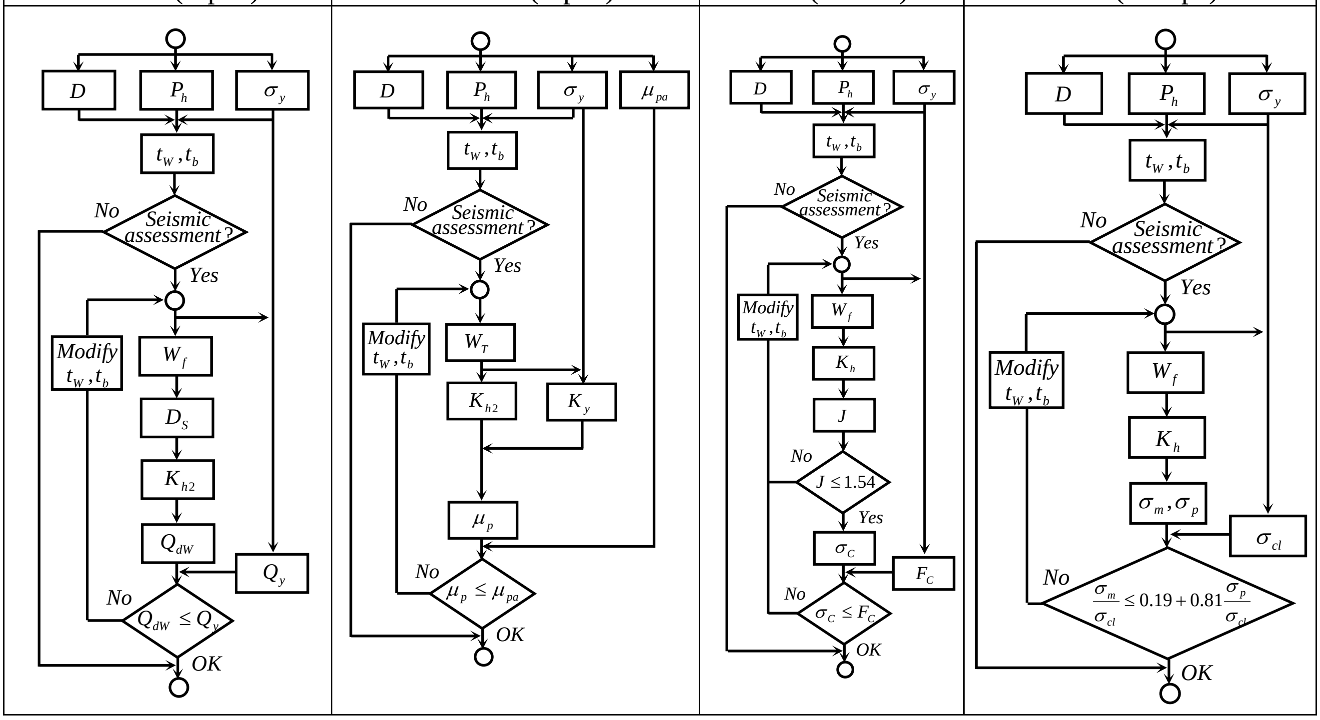
Fig.2 Detail of the bottom section

1.2. Current seismic design methods of un-anchored cylindrical steel tanks

Several seismic design codes or guidelines for steel tanks have been established based on the results of many seismic studies in the world. In this study, typical seismic codes or guidelines in Japan, U.S. and Europe[4)] are reviewed to indicate some problems in the current codes. Table.1 shows the flow chart of these codes.

Table.1 Flow chart of seismic design codes for steal tanks

JAS*[1)] (Japan)	HPG*[2)] (Japan)	API (U.S.A.)	BSI (Europe)



*1) Japan Architectural Society, 2)Japan High Pressure Gas Safety Association

1.3. Features of current seismic design codes

(1) Japan Architectural Society (Seismic deign guideline of structural storage tanks)

This guideline proscribes the seismic performance of shell plate against plastic buckling based on horizontal load-carrying capacity method as given by following equation:

$$Q_{dW} \le Q_y \tag{1}$$

where, Q_{dW}, Q_y are the seismic response produced by horizontal inertia force for the Level 2 ground motion and the critical elephant foot buckling strength, respectively.

$$Q_{dW} = K_{h2} \cdot W_f = D_S \cdot K_{h20} \cdot W_f \tag{2}$$

where, K_{h2} = the horizontal seismic intensity for the Level2 ground motion, D_S = the structural characteristic factor, K_{h20} = the standard seismic intensity for the Level2 ground motion, W_f = the liquid mass (based on Housner's model[5)]). Design seismic load corresponding to ground motion level 2 is considered to be reduced by the structural characteristic factor *Ds*.

$$D_S = \frac{1}{\sqrt{1+6\dfrac{\delta_B}{\delta_y}\left(\dfrac{T_l}{T_e}\right)^2}} \tag{3}$$

in which δ_B, δ_y, T_l, T_e are the ultimate limit deformation, the yield deformation, typical period of the base plate vibration and the typical period of the deformed tank system, respectively.

The reduction of the structural response is caused by inelastic response of the system which is produced by the plastic deformation of the base plate, while the possibility of the base plate is not taken into consideration. Furthermore, structural characteristic factor D_s is calculated with the ratio of the ultimate limit deformation δ_B via the yield deformation δ_y, although this factor should be evaluated as the ratio of the maximum response δ_{max} via the yield deformation. This assumption leads to underestimate D_s compare to the original, so that the probability of failure might be underestimated.

(2) High Pressure Gas Safety Institute of Japan

In this guideline, seismic safety assessment is based on structural strain which should be less than the critical strain of the shell plate. The inelastic response due to the plastic deformation of the shell plate is taken into consideration as the decreased yield strength level. The seismic assessment is executed with the ductility factor μ_p by the following design criterion:

$$\mu_p = \frac{1}{4C}\left\{\left(\frac{K_{MH}}{K_y}\right)^2 - 1\right\} \le \mu_a \tag{4}$$

where K_{MH}, K_y and C are the design seismic intensity, the seismic intensity corresponding to the yield strength of the side wall material and the plastic parameter $C=2$, respectively.

However, the effect due to plastic deformation of the base plate is not taken into consideration in this guideline when the inelastic response of the tank is calculated for the Level 2 ground motion. This approach means to neglect the possibility of crack failure from the base plate under low cycle repetitions of plastic deformations.

(3) API 650

The possibility of uplifting of the tank is checked with the anchorage ratio *J* which is defined by the tipping moment and resistance moment. The seismic assessment of the shell plate against the EFB failure is done by the following way:

$$\sigma_C \le F_C = 83\frac{t_S}{D} \tag{5}$$

where, σ_C, F_C, t_S and D are the maximum longitudinal shell compression stress, allowable longitudinal shell compression stress, thickness of bottom shell course and nominal tank diameter, respectively. It should be noted that this method also neglect evaluation of the possibility of the crack failure of the base plate.

(4) Eurocode 8, Part 4

The EFB failure of the shell plate is assessed as the ultimate limit state for the seismic load, assuming that the base plate is uplifted in this code.

$$\frac{\sigma_m}{\sigma_{cr}} \le 0.19 + 0.81\frac{\sigma_p}{\sigma_{cr}} \tag{6}$$

in which σ_m , σ_{cr} and σ_p are the maximum longitudinal membrane stress, the theoretical buckling stress for cylinders loaded in axial compression and the critical buckling stress considering the imperfection of the tanks. As concerns non-linear behavior of the base plate, it is just cautioned about bending fatigue fracture of base plate by indicating the maximum allowable rotation angle of the tank system. Nothing is considered about plastic deformation of base plate, therefore structural characteristic is also neglected.

3. SEISMIC DESIGN METHOD CONSIDERING PLASTIC DEFORMATION OF THE BASE PLATE

3.1. Reduction of seismic load with plastic deformation of the base plate.

When Level 2 earthquake (EQ2) occurred and the tank is rocking in terms of the rotational angle of θ as shown in Fig.1, the horizontal seismic load Q_{dW} is calculated in the following way by introducing the structural characteristic factor D_S to evaluate deduction force based on inelastic behavior.

$$Q_{dW} = D_S \cdot S_A(T_e) \cdot \frac{W_T}{g} \tag{7}$$

where, g, S_A, W_t and T_e are gravity acceleration, acceleration response spectrum, weight of the tank and characteristic period of the system, respectively.

In this situation, whole tank acts as non-linear single-free mode system since plastic deformation occurs in base plate. Assuming that tensile strain of base plate is equal to ε_2^{B*} , ductility factor η^B can be defined using with the plastic strain of the base plate ε_p^B .

$$\eta^B = \frac{\varepsilon_2^{B*} - \varepsilon_p^B}{\varepsilon_p^B}$$

Structural characteristic factor D_s is given by following equation.

$$D_S = D_h \cdot D_\eta = \frac{1}{\sqrt{1+4\eta^B}} \cdot \frac{1.42}{1+3h+1.2\sqrt{h}} \tag{8}$$

where, h means dumping factor of the system.

Effect of non-linear response with seismic load EQ2 can be evaluated by multiplying Ds. At this time, response value ε_2^{B*} generated on the base plate shows non-linear behavior, but response value ε_2^{W} generated on the shell plate does not always show non-linear behavior.

On the other hand, the inelastic displacement amplitude δ_B is calculated by

$$\delta_B = \xi \cdot \theta \cdot D \quad \text{for} \quad \theta = \frac{Q_{dW}}{\lambda \cdot H_l \cdot K_1} \tag{9}$$

where, ξ is a parameter to minimize the discrepancy between the analytical accuracy and the modeling error. λ , H_l and K_1 are a height ratio of gravity center for the tank height (= 0.44), depth of the liquid and spring modulus between the base plate and foundation of the tank.

Correlation between the inelastic strain ε_2^{B*} and the plastic strain ε_p^B of the base plate is also given by

$$\varepsilon_2^{B*} = \frac{\delta_B}{\delta_y}\varepsilon_p^B \tag{10}$$

Therefore, seismic safety assessment for the initiation of crack and leakage of the base plate can be executed with following equation.

$$\varepsilon_2^{B*} < \varepsilon_U^B \tag{11}$$

where, $\varepsilon_U{}^B$ is the critical strain level for the requested seismic performance.

On the other hand, the seismic safety criterion for elephant foot buckling of the shell plate is also given by

$$S_2^W < S_p^W \tag{12}$$

in which

$$S_p^W = f_{CRS} \cdot \left(1 - \frac{\sigma_\phi}{\sigma_y}\right) \quad , \quad S_2^W = \frac{2k_1}{t_{wall}} \cdot \frac{Q_{dW} \cdot r}{\lambda H_l \cdot K_1} \tag{13}$$

where, f_{CRS}, σ_ϕ, k_1 and t_{wall} are the standard buckling strength, the hoop stress at the bottom of shell plate, a spring modulus per unit length along the annular plate and the thickness of the shell plate, respectively, and the hoop stress is calculated by

$$\sigma_\phi = \frac{Q_{dw}}{2.5 H_l t_{wall}} + \frac{W_l}{\pi r t_{wall}} \tag{14}$$

3.2. Formulization based on limit-state design method

(1) Definition of seismic performance

Here, seismic performance demanded for cylindrical steel tank is defined as follow;

Seismic performance 1: the system can be maintained without any disruption for the level 1 earthquake ground motion (EQ1), when the system is slightly damaged or not.

Seismic performance 2: the system can be restarted after short repair disruption for the level 2 earthquake ground motion (EQ2), when the system is not significantly damaged.

Seismic performance 3: the system can be restarted after restoring disruption for the level 2 earthquake ground motion (EQ2), when the system is not completely damaged

Damage modes of the base plate correspond to the seismic performance level above are defined as follows;

Minor damage mode D_i^B : the elastic structural response $S_1{}^B$ exceeds the critical level $S_a{}^B$ by EQ1, and the probability of minor damage occurrence is defined as $p_{fi}{}^B$.

Moderate damage mode D_o^B : the inelastic structural response $\varepsilon_2{}^{B*}$ exceeds the critical level $\varepsilon_U{}^B$ for the small leakage by EQ2, and the probability of moderate damage occurrence is defined as $p_{fo}{}^B$.

Major damage mode D_a^B : the inelastic structural response $\varepsilon_2{}^{B*}$ exceeds the critical level $\varepsilon_U{}^B$ for the large leakage by EQ2, and the probability of major damage occurrence is defined as $p_{fa}{}^B$.

Damage modes of shell plate are also defined as follows;

Minor damage mode D_i^W : the elastic structural response $S_1{}^W$ exceeds the critical level $S_a{}^W$ by EQ1, and the probability of minor damage occurrence is defined as $p_{fi}{}^W$.

Moderate damage mode D_o^W : the inelastic structural response $\varepsilon_2{}^{W*}$ exceeds the critical level $\varepsilon_U{}^W$ for the small leakage by EQ2, and the probability of moderate damage occurrence is defined as $p_{fo}{}^W$.

Major damage mode D_a^W : the inelastic structural response $\varepsilon_2{}^{W*}$ exceeds the critical level $\varepsilon_U{}^W$ for the large leakage by EQ2, and the probability of major damage occurrence is defined as $p_{fa}{}^W$.

Based on the definition of the damage modes as below, the corresponding fragility curves for base plate and shell plate are given as follows;

$$P\left[D_i^B \middle| EQ1\right]=P\left[S_a^B \le S_1^B\right]=p_{fi}^B$$
$$P\left[D_o^B \middle| EQ2\right]=P\left[\varepsilon_L^B < \varepsilon_2^{B*} < \varepsilon_U^B\right]=p_{fo}^B \qquad (15)$$
$$P\left[D_a^B \middle| EQ2\right]=P\left[\varepsilon_U^B < \varepsilon_2^{B*}\right]=p_{fa}^B$$
$$P\left[D_i^W \middle| EQ1\right]=P\left[S_a^W \le S_1^W\right]=p_{fi}^W$$
$$P\left[D_o^W \middle| EQ2\right]=P\left[\varepsilon_p^W < \varepsilon_2^{W*} < \varepsilon_L^W\right]=p_{fo}^W \qquad (16)$$
$$P\left[D_a^W \middle| EQ2\right]=P\left[\varepsilon_L^W < \varepsilon_2^{W*}\right]=p_{fa}^W$$

(2) Probabilities of failure for the shell plate and the base plate

The probabilities of failure for the shell plate and the base plate are formulated as follows.

For the seismic load EQ1 for the shell plate,

$$P\left[S_a^W < S_1^W\right]=p_{fi}^W \qquad (17)$$

For the seismic load EQ2 for the base plate,

$$P\left[\varepsilon_U^B < \varepsilon_2^{B*}\right]=p_{fa}^B \qquad (18)$$

Then the probability of major damage for the shell plate is calculated with the probability density function of the base plate strains as:

$$P\left[\varepsilon_L^W < \varepsilon_2^{W*}\right]=\int_0^\infty P\left[\varepsilon_L^W < \varepsilon_2^{W*} \middle| x\right] f_{\varepsilon_2^{B*}}(x)dx \qquad (19)$$

in which f_X is a probability density function of the variable X, and the probability of moderate damage for the shell plate is also given by

$$P\left[\varepsilon_p^W < \varepsilon_2^{W*} < \varepsilon_L^W\right]=\int_0^\infty P\left[\varepsilon_p^W < \varepsilon_2^{W*} < \varepsilon_L^W \middle| x\right] f_{\varepsilon_2^{B*}}(x)dx \qquad (20)$$

(3) Probability density of the base plate responses

When the nominal wall thickness of the base plate is designed with the probability of 95% by which the seismic load does not exceed the yield strength, the required yield stress S_p^B can be derived from Eq.5 and the probability p_{fi}^B of minor damage in the following way.

$$S_p^B=\left(S_a^B\right)_m+\left(\alpha\beta_i^B+k_R\right)\sigma_{S_a^B} \text{ for } \beta_i^B=-\Phi^{-1}\left[p_{fi}^B\right] \qquad (21)$$

in which $\left(S_a^B\right)_m, \sigma_{S_a^B}, \beta_i^B$ and k_R are nominal values of the allowable base plate strength, its standard deviation, safety index for the minor damage of the base plate and the non-exceeding parameter, respectively , and $\alpha \approx 0.75$. The probability density of the base plate strains ε_2^{B*} for EQ2 is derived on the basis of the e energy conservation assumption which can be applied for a single-degree-freedom system

$$f_{S_2^B}(s)=\frac{1}{\sqrt{2\pi}(\varsigma\cdot s)}\exp\left[-\frac{1}{2}\left(\frac{\ln s-\lambda}{\varsigma}\right)^2\right] \qquad (22)$$

in which the log-normal distribution is assumed for the seismic load of ε_2^{B*}, and

$$\lambda=E\left[\ln S_2^B\right], \varsigma=\sqrt{Var\left(\ln S_2^B\right)} \qquad (23)$$

Noting that the probability density $f_{\varepsilon_2^B}(\varepsilon)$ of the base plate strain is identical to that of the stress in the elastic region, the probability of density of the base plate strain in the inelastic region should be modified using the following formula.

$$f_{\varepsilon_2^B}\left(\varepsilon_2^B\right)d\varepsilon=f_{S_2^B}(s)ds \qquad (24)$$

in which

$$s=E\cdot\varepsilon \quad , \quad \varepsilon_p^B=S_p^B/E \qquad (25)$$

$$\varepsilon_2^{B*} > \varepsilon_p^B \rightarrow \varepsilon = \frac{1}{2}\varepsilon_p^B \cdot \left\{1 + \left(\frac{\varepsilon_2^B}{\varepsilon_p^B}\right)^2\right\}$$
$$\varepsilon_2^{B*} \le \varepsilon_p^B \rightarrow \varepsilon = \varepsilon_2^B \tag{26}$$

(4) Design strength values of the shell plate and the base plate

The design strength values S_a^W, ε_p^W, ε_L^W, S_a^B, ε_p^B, ε_L^B of the shell plate and the base plate which can comply with the probabilities of major, moderate and minor damage modes are formulated in the following way, based on the log-normal assumption:

$$\begin{aligned}
&\mu_{S_a^W} = \mu_{S_1^W} \exp\left(\beta_i^W \sqrt{\delta_{S_a^W}^2 + \delta_{S_1^W}^2}\right) \quad \text{for} \quad \beta_i^W = -\Phi^{-1}\left[p_{fi}^W\right] \\
&\mu_{\varepsilon_p^W} = \mu_{\varepsilon_2^{W*}} \exp\left(\beta_o^W \sqrt{\delta_{\varepsilon_p^W}^2 + \delta_{\varepsilon_2^{W*}}^2}\right) \\
&\qquad\qquad\qquad\qquad \text{for} \quad \beta_o^W = -\Phi^{-1}\left[p_{fo}^W + p_{fa}^W\right] \\
&\mu_{\varepsilon_L^W} = \mu_{\varepsilon_2^{W*}} \exp\left(\beta_a^W \sqrt{\delta_{\varepsilon_L^W}^2 + \delta_{\varepsilon_2^{W*}}^2}\right) \quad \text{for} \quad \beta_a^W = -\Phi^{-1}\left[p_{fa}^W\right] \\
&\mu_{S_a^B} = \mu_{S_1^B} \exp\left(\beta_i^B \sqrt{\delta_{S_a^B}^2 + \delta_{S_1^B}^2}\right) \quad \text{for} \quad \beta_i^B = -\Phi^{-1}\left[p_{fi}^B\right] \\
&\mu_{\varepsilon_p^B} = \mu_{\varepsilon_2^{B*}} \exp\left(\beta_o^B \sqrt{\delta_{\varepsilon_p^B}^2 + \delta_{\varepsilon_2^{B*}}^2}\right) \\
&\qquad\qquad\qquad\qquad \text{for} \quad \beta_o^B = -\Phi^{-1}\left[p_{fo}^B + p_{fa}^B\right] \\
&\mu_{\varepsilon_L^B} = \mu_{\varepsilon_2^{B*}} \exp\left(\beta_a^B \sqrt{\delta_{\varepsilon_L^B}^2 + \delta_{\varepsilon_2^{B*}}^2}\right) \quad \text{for} \quad \beta_a^B = -\Phi^{-1}\left[p_{fa}^B\right]
\end{aligned} \tag{27}$$

where, μ_X and δ_X are mean value and its coefficient of variation of a random variable X and Φ is the standard normal distribution.

4. NUMERICAL STADIES

Numerical studies for un-anchored cylindrical steel tank are executed. Structural dimensions of the tank and random parameters are shown in Table.2 and Table.3, respectively.

Table.2 Dimension of steel tank

Item	unit	value
Diameter	m	79.8
Height	m	20
Thickness of shell plate	mm	32
Thickness of base plate	mm	22
Yield stress	kN/cm²	42.1
Bucking strength	kN/cm²	8.0
damping factor		0.1
Seismic intensity	gal/g	1.2

g : gravity (980 gal)

Table 3. Random parameters

item	symbol	unit	mean	COV
Yield stress of base plate	σ_p^B	kN/cm^2	*22.1*	*0.2*
Crack initiating strain of base plate	ε_L^B		*0.008*	*0.25*
Ultimate failure strain of base plate	ε_U^B		*0.02*	*0.3*
Yield stress of shell plate	σ_p^W	KN/cm^2	*42.1*	*0.2*
Leak initiating strain of shell plate	ε_L^W		*0.02*	*0.2*
Loading strain of shell plate	ε_2^W		*variable*	*0.1*

In order to meet the structural characteristic factor D_S given in the seismic design guideline of JAS with that of Eq.8, the adjustable parameter ξ of Eq.9 is assumed to be $\xi=2.65$. Then the probability of the buckling failure of the shell plate can be calculated with Eqs. 18 and 19. Fig.3 shows the probability $P[\varepsilon_p^W < \varepsilon_2^{W*}]$ of failure for the shell plate for various yield strengths of the base plate. Using this figure, the required yield strength of the base plate is obtained from the designated probability of failure for the shell plate. Fig.4 shows the probabilities of major damage mode for the shell plate and the base plate for various safety index of the base plate. For smaller than the safety index of 4.0, the probability of failure for the base plate is larger than that of the shell plate, while, for greater than the safety index of 4.0, the probability of the shell plate is larger than that of the base plate.

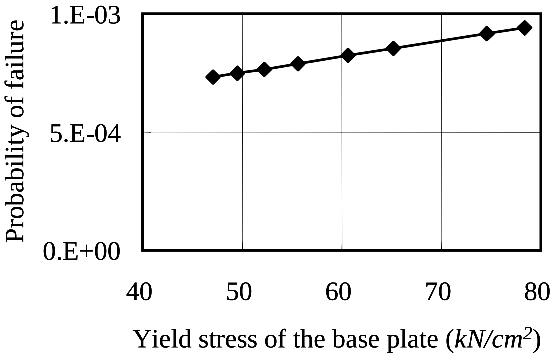


Fig.3 Probability of failure for elephant foot type buckling mode under various yield strengths of the base plate

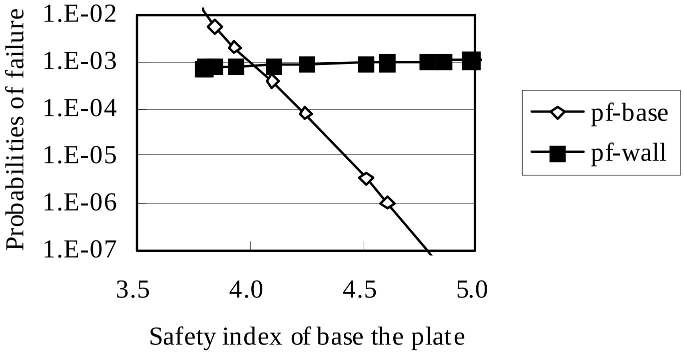


Fig.4 Failure probabilities of the side wall and the base plate for various safety indexes

5. CONCLUSION

The present study discusses about seismic risk assessment of un-anchored steel tanks due to an excitation of a strong earthquake. By the rocking motion of the tank, a plastic deformation develops at bottom of the plate as well as the elephant foot type buckling at the shell plate. The proposed seismic design formula is developed in the form of limit state design method, in which both of the damage modes at the shell plate and the base plate are taken into consideration.

Several accomplishments are summarized as follows:

(1) The present study proposes an exact estimation approach of the seismic safety of elephant foot buckling failure at the shell plate as well as the crack failure of the bottom plate.

(2) Once the target probability of the tank failure is given for the performance-based design, optimally allocated procedures of both target probabilities for EFB failure at the side wall as well as the crack failure of the bottom plate are developed based on the reliability analysis for the multi-failure modes of the structural system.

(3) Fragility curves for the damage modes of the side wall and bottom plates are numerical analyzed for various seismic loads.

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