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Keywords

reliability analysis, partial safety factor method, uplift limit state, offshore plate anchors, drainage conditions

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Reliability of offshore plate anchor design in sand for uplift limit state

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Abstract: The offshore wind industry needs to move towards floating offshore wind turbines (FOWTs) that are secured to the seabed with anchoring systems in order to harness the more substantial amounts of offshore wind resources available in deep waters. The design of the anchoring system for FOWTs is crucial to ensure stability and safety in challenging offshore conditions. As a simplified reliability-based design approach, the partial safety factor method remains important in current offshore foundation design practice, but its effectiveness in achieving the required target safety level still needs to be examined. To achieve this end, a reliability analysis is performed for the uplift limit state design of strip plate anchors embedded in sand subjected to vertical loads, and a wide range of load cases are considered in the analysis. The failure probabilities of strip plate anchors designed with the partial safety factor method are estimated and then compared to the target safety levels. The results show that the estimated reliability index decreases rapidly with the increasing of permanent to variable action ratio and gradually converges to a relatively steady state. In addition, the current partial safety factor method has been shown to be conservative for offshore plate anchor design over a wide range of permanent to variable action ratios. The results provide probabilistic insights into the effectiveness of the partial safety factor method and may aid in the further development of reliability analysis for offshore anchors.

Keywords: reliability analysis; partial safety factor method; uplift limit state; offshore plate anchors; drainage conditions

1 Introduction

Offshore wind turbines have experienced significant growth in the past two decades to address the climate change concerns and to achieve the net zero carbon emissions target[1]. To withstand harsh environmental conditions such as sea wave, wind, and current, fixed-bottom wind turbines are mostly adopted for water depths up to 60 m. However, they become less feasible for deeper waters due to the significant cost increase and difficulty of installation[2]. Therefore, the offshore wind industry needs to move towards floating offshore wind turbines (FOWTs) that are secured to the seabed with anchoring systems in order to harness the more substantial amounts of offshore wind resources available in deeper waters. For FOWTs, the design of the anchoring system is crucial to ensure stability and safety in challenging offshore conditions[3].

Reliable plate anchor design involves considering various sources of uncertainties, such as uncertainties related to soil variability, site investigation, anchor installation, prediction models, and various loads. The conventional design method of using a global factor of safety may not be sufficient to account for these uncertainties and is less rational compared to the reliability-based approach that makes use of probability theories[4]. As a more sophisticated design method that explicitly addresses various uncertainties[5], the reliability-based design approach has the potential to produce more cost-effective and consistent design across different site conditions[6−7]. Some reliability analyses have been performed for offshore foundations recently[8−10]. The partial safety factor approach can be considered as a simplified reliability-based design approach, which is a trade-off between the conventional global factor of safety method and the reliability-based design approach. It allows separate consideration of uncertainties in soil and loads, and retains the simplicity of performing algebraic design checks. As a result, the partial safety factor method remains important in current offshore foundation design practice[11]. However, it is crucial to ensure the effectiveness of this simplified method in achieving the required target safety level.

This paper performs a reliability analysis of offshore plate anchors to investigate whether the current design developed using the partial safety factor method meets the required target safety levels. The study concentrates on the uplift limit state design of strip plate anchors embedded in sand subjected to vertical loads. A wide range of load cases are considered in the analysis. The results provide probabilistic insights into the effectiveness

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of the partial safety factor method and may aid in the further development of reliability analysis for offshore anchors.

2 Uplift resistance of strip plate anchors

White et al.[12] obtained a limit equilibrium solution for the uplift resistance of strip plate anchors by assuming that the shear planes of the failed block are inclined at the dilation angle ψ. The peak uplift resistance per unit length R_u, is computed as

$$\frac{R_u}{\gamma' HB} = N_\gamma = 1 + F_u \frac{H}{B} \tag{1}$$

where N_γ is a dimensionless uplift factor; γ' is the soil effective unit weight; H is the anchor embedment depth; B is the strip anchor width; and F_u is an uplift factor expressed as

$$F_u = \tan\psi_p + (\tan\varphi_p - \tan\psi_p)\left(\frac{1+K_0}{2} - \frac{1-K_0}{2}\cos(2\psi_p)\right) \tag{2}$$

where φ_p is the peak friction angle of sand; ψ_p is the peak dilation angle of sand; and K_0 is the lateral earth pressure coefficient at rest, which is calculated as $K_0 = 1-\sin\varphi_{cs}$ with φ_{cs} being the critical state friction angle of sand.

White et al.[12] further applied the stress–dilatancy relationships (Eq. (3)) proposed by Bolton[13] to transform the peak uplift resistance as a function of the soil relative density I_D and φ_{cs}.

$$\left.\begin{aligned} \varphi_p - \varphi_{cs} &= k\psi_p \\ \varphi_p - \varphi_{cs} &= mI_R \end{aligned}\right\} \tag{3}$$

where I_R is a relative dilatancy index as a function of the soil relative density I_D, grain-crushing strength σ'_c, and the mean effective strength p', $I_R = I_D \ln(\sigma'_c/p') - 1$. The parameters k and m are taken as 0.8 and 5, respectively, under plane strain conditions in this analysis.

Manipulating Eqs. (1)–(3) to eliminate ψ_p re-writes the peak uplift resistance as a function of I_D or φ_p. Since the range of the variability of φ_p has been extensively studied with greater confidence than that of I_D, the peak uplift resistance is expressed as a function of φ_p. The uplift factor is re-written as

$$F_u = \tan\frac{\varphi_p - \varphi_{cs}}{k} + \left(\tan\varphi_p - \tan\frac{\varphi_p - \varphi_{cs}}{k}\right)\cdot \left[\frac{1+K_0}{2} - \frac{1-K_0}{2}\cos\frac{2(\varphi_p - \varphi_{cs})}{k}\right] \tag{4}$$

Recall that K_0 is taken as a function of φ_{cs} to be $K_0 = 1-\sin\varphi_{cs}$. The critical state friction angle φ_{cs} is a fundamental soil property depending primarily on particle mineralogy and shape. It has been determined experimentally to be around 33° for quartz and 40° for feldspar within a margin of 1° [13]. Therefore, φ_{cs} can be regarded as a deterministic parameter in the analysis, and the peak friction angle, φ_p, is discussed in the following section.

3 Characteristic values of soil parameters and load

The peak uplift resistance of a strip plate anchor in sand calculated with Eqs. (1) and (4) is primarily affected by two parameters, i.e. the soil peak friction angle, φ_p, and the soil effective unit weight, γ', which are represented by two random variables, characterised by their means and standard deviations. Since γ' is constrained to non-negative values, it is assumed to be lognormally distributed with a mean $\mu_{\gamma'}$ and a standard deviation $\sigma_{\gamma'}$ [14]. Reference[15] recommends a "cautious estimate of the mean" for the characteristic values of geotechnical parameters, and further states that the characteristic values of geotechnical parameters should be selected to ensure at least a 95% confidence in the geotechnical system for a limit state considered in the design. This recommendation is fairly vague for design. When a low value of the material property is unfavourable, Reference[16] suggests the 5% fractile value for its characteristic value. Following the suggestion, the characteristic value of effective unit weight, $\hat{\gamma}'$, of the soil used for design is represented by the 5% fractile of its lognormal distribution:

$$\hat{\gamma}' = \exp\left[\mu_{\ln\gamma'}(1 - 1.645 v_{\ln\gamma'})\right] \tag{5}$$

where $\mu_{\ln\gamma'}$ and $v_{\ln\gamma'} = \sigma_{\ln\gamma'}/\mu_{\ln\gamma'}$ are the mean and coefficient of variation of its normally distributed counterpart, $\ln\gamma'$, with $\sigma_{\ln\gamma'}$ being the standard deviation. It should be noted that hat parameters are adopted for anchor design in this analysis. The lognormally distributed γ' can be transformed to its normally distributed counterpart, having parameters:

$$\left.\begin{aligned} \sigma^2_{\ln\gamma'} &= \ln\left(1 + v^2_{\gamma'}\right) \\ \mu_{\ln\gamma'} &= \ln\mu_{\gamma'} - \frac{1}{2}\sigma^2_{\ln\gamma'} \end{aligned}\right\} \tag{6}$$

where $v_{\gamma'} = \sigma_{\gamma'}/\mu_{\gamma'}$ is the coefficient of variation of γ'.

Following Fenton et al.[17] and He et al.[18], the peak friction angle φ_p is assumed to follow a bounded tanh distribution. The tanh distribution has a simple relationship with the normal distribution expressed as[19]

$$\varphi_p = \varphi_{p,\min} + \frac{1}{2}(\varphi_{p,\max} - \varphi_{p,\min})\left[1 + \tanh\left(\frac{sG_\varphi}{2\pi}\right)\right] \tag{7}$$

where G_φ is the standard normal; $\varphi_{p,\min}$ and $\varphi_{p,\max}$ are the minimum and maximum peak friction angles, respectively;

and s is a scale factor governing the variability of φ_p between the two bounds[19]. The distribution is symmetric and has a mean of $\mu_{\varphi_p} = (\varphi_{p,\max} - \varphi_{p,\min})/2$. In this analysis, φ_p is assumed to be bounded between 30° and 50° [20] with a mean of $\mu_{\varphi_p} = 40^\circ$. The scale factor is assumed to be s = 2.5, resulting in a bounded bell shaped distribution with the standard deviation of $\sigma_{\varphi_p} \simeq 3.4^\circ$, and the coefficient of variation of $v_{\varphi_p} \simeq 3.4^\circ/40^\circ = 0.09$, which lies within the variability ranges suggested by Lee et al.[21] and Phoon and Kulhawy[22].

Similar to γ', the characteristic value of peak friction angle, $\hat{\varphi}_p$, is taken as the 5% fractile of the bounded tanh distribution. This can be obtained by numerically solving $F(\varphi_p) = 0.05$ where $F(\varphi_p)$ is the cumulative distribution function of φ_p which can be derived from its probability density function based on Eq. (7). Generally, φ_p and γ' have a reasonably positive correlation, which reduces the estimated failure probabilities[23]. In this paper, φ_p and γ' are conservatively assumed to be independent.

DNV[11] specifies that the design of offshore plate anchors need to consider two types of loads, i.e. the mean line tension (F_T) due to pretension and the effect of mean environmental loads, and the dynamic increase in the line tension (F_D) due to oscillatory low-frequency and wave-frequency effects, as shown in Fig. 1. F_T and F_D are represented by two independent random variables, following lognormal distributions with means of μ_{F_T} and μ_{F_D}, and standard deviations of σ_{F_T} and σ_{F_D}, respectively.

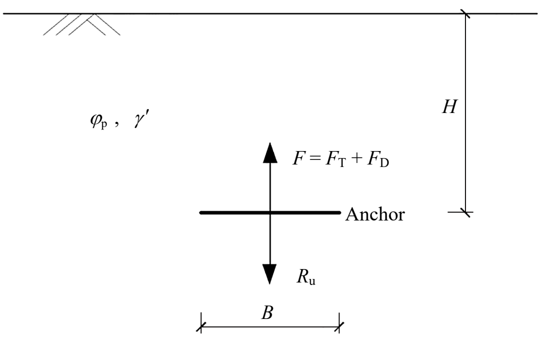

Fig. 1 Schematic diagram of strip plate anchor in sand under vertical uplift loads

As stated by Reference[16], pre-stressing should be classified as a permanent action with the characteristic value equal to the 95% fractile of its statistical distribution. Therefore, F_T is taken as a permanent action, and its characteristic value $\hat{F}_T$ used for design can be calculated by

$$\hat{F}_T = \exp\left[\mu_{\ln F_T}(1 + 1.645 v_{\ln F_T})\right] \quad (8)$$

where $\mu_{\ln F_T}$ and $v_{\ln F_T}$ are the mean and coefficient of variation of the normally distributed $\ln F_T$, and can be calculated using the same form of Eq. (6) with respect to F_T.

The characteristic value of dynamic line tension increment $\hat{F}_D$ is assumed to be evaluated using the same method as $\hat{F}_T$, i.e.

$$\hat{F}_D = \exp\left[\mu_{\ln F_D}(1 + 1.645 v_{\ln F_D})\right] \quad (9)$$

where $\mu_{\ln F_D}$ and $v_{\ln F_D}$ are the mean and coefficient of variation of the normally distributed $\ln F_D$, and can be calculated using the same form of Eq. (6) with respect to F_D.

4 Design of plate anchors

The general uplift limit state design criterion within the partial safety factor framework is expressed as

$$\hat{R}_u\left(\tan^{-1}\left(\frac{\tan\hat{\varphi}_p}{\gamma_\varphi}\right), \frac{\hat{\gamma}'}{\gamma_\gamma}\right) \geqslant \sum \gamma_{F_i}\hat{F}_i \quad (10)$$

where $\hat{R}_u$ is the characteristic value of anchor uplift resistance as a function of $\hat{\varphi}_p$ and $\hat{\gamma}'$; γ_φ and γ_γ are the partial safety factors on $\tan\hat{\varphi}_p$ and $\hat{\gamma}'$, respectively; $\hat{F}_i$ is the ith characteristic load; and γ_{F_i} is the load partial factor corresponding to $\hat{F}_i$. It should be noted that γ_φ is applied to $\tan\hat{\varphi}_p$ rather than $\hat{\varphi}_p$ as stated by Reference[15], thereby resulting in an equivalent characteristic value of peak friction angle equal to $\tan^{-1}(\tan\hat{\varphi}_p/\gamma_\varphi)$.

Since only F_T and F_D are considered for offshore anchor design, Eq. (10) is written more specifically as

$$\hat{R}_u\left(\tan^{-1}\left(\frac{\tan\hat{\varphi}_p}{\gamma_\varphi}\right), \frac{\hat{\gamma}'}{\gamma_\gamma}\right) \geqslant \gamma_T\hat{F}_T + \gamma_D\hat{F}_D \quad (11)$$

where γ_T and γ_D are the load partial factors corresponding to $\hat{F}_T$ and $\hat{F}_D$, respectively.

As discussed previously, $\hat{R}_u$ can be estimated using Eqs. (1) and (4) with φ_p, γ', and H replaced by $\tan^{-1}(\tan\hat{\varphi}_p/\gamma_\varphi)$, $\hat{\gamma}'/\gamma_\gamma$, and $\hat{H}$, respectively. $\hat{H}$ is the depth of the designed strip anchor. Note again that the hat parameters are used for the design process in this analysis.

The aim of the plate anchor design is to estimate the anchor embedment depth $\hat{H}$, that satisfies the design criterion in Eq. (11). Substituting Eqs. (1) and (4) into Eq. (11) at the equality yields the following equation:

$$\left(1 + \hat{F}_u\frac{\hat{H}}{B}\right)\left(\frac{\hat{\gamma}'}{\gamma_\gamma}\hat{H}B\right) = \gamma_T\hat{F}_T + \gamma_D\hat{F}_D \quad (12)$$

In Eq. (12), $\hat{F}_u$ is a function of $\tan^{-1}(\tan\hat{\varphi}_p/\gamma_\varphi)$ (see Eq. (4)), and the strip anchor width B is assumed to be a deterministic value. Once the characteristic values of soil properties and loads are obtained from Section 3, the required anchor embedment depth $\hat{H}$ is then estimated by numerically solving Eq. (12) for $\hat{H}$ (see Fig. 2).

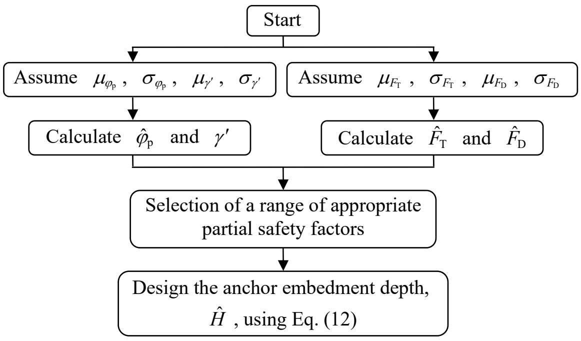

Fig. 2 Flowchart for plate anchor design

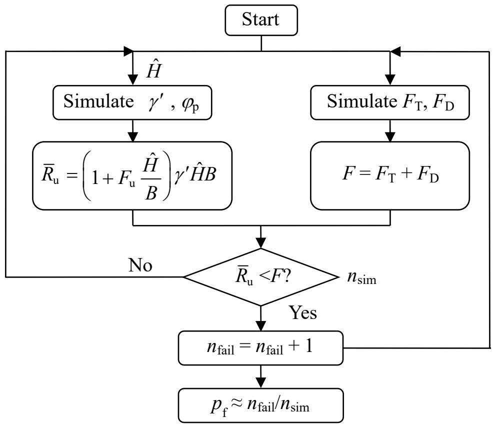

Fig. 3 Monte Carlo procedures

5 Estimate of failure probability

The failure probability of the designed anchor, p_f, is defined as the probability that the actual total load exceeds the actual anchor resistance of the designed anchor:

$$p_f=P[\bar{R}_u<F]\leqslant p_m \tag{13}$$

where $\bar{R}_u$ is the actual (random) anchor uplift resistance estimated with the designed anchor embedment depth, $\hat{H}$; F is the actual total vertical load, $F=F_T+F_D$, with F_T and F_D being the actual mean line tension and dynamic increment of the actual line tension, respectively; and p_m is the target maximum acceptable failure probability. Note that bar parameters are used for the actual values to differentiate from the design values in this analysis.

The actual uplift resistance $\bar{R}_u$, is estimated by Eq. (1), indicating that only the soil uncertainty is accounted for to calculate the failure probability without considering the uncertainty associated with the geotechnical prediction model. Using the same form of Eq. (1), $\bar{R}_u$ is expressed as

$$\bar{R}_u=\left(1+F_u\frac{\hat{H}}{B}\right)\gamma'\hat{H}B \tag{14}$$

where F_u is calculated using Eq. (4). It is worth noting that $\bar{R}_u$ in Eq. (14) is evaluated based on the random variables, φ_p and γ', compared to the (deterministic) characteristic values, $\hat{\varphi}_p$ and $\hat{\gamma}'$, used for $\hat{R}_u$.

Once the designed anchor embedment depth, $\hat{H}$, is evaluated using Eq. (12), the failure probability, p_f, can be estimated with Monte Carlo simulations (see Fig. 3) following the steps below:

(1) Simulate the random variables of the soil properties (φ_p and γ'), with the specified means and standard deviations, and evaluate the actual uplift resistance, $\bar{R}_u$, of the designed anchor using Eq. (14);

(2) Simulate the actual F_T and F_D, and calculate the actual total vertical load, $F=F_T+F_D$;

(3) Determine whether the designed anchor fails ($\bar{R}_u<F$); if so, update the number of failures, i.e. $n_{fail}=n_{fail}+1$.

The above steps will be repeated n_{sim} times, and the failure probability of the designed plate anchor is then approximated to be $p_f\approx n_{fail}/n_{sim}$. The corresponding reliability index, β, is computed as

$$\beta=\Phi^{-1}(1-p_f) \tag{15}$$

where Φ is the cumulative distribution function of the standard normal.

6 Results and discussion

6.1 Case study

As discussed in Section 1, the soil peak friction angle φ_p, follows a bounded tanh distribution, ranging from 30° to 50° with a mean of $\mu_{\varphi_p}=40°$ and a coefficient of variation of $v_{\varphi_p}=0.09$. The effective unit weight, γ', is assumed to have a mean of $\mu_{\gamma'}=8$ kN /m^3 and a coefficient of variation of $v_{\gamma'}=0.1$, which is aligned with the ranges summarised by Lee et al.[21] and Lumb[24]. The anchor width is assumed to be deterministic and is taken as $B=6$ m. In addition, Eqs. (5) and (6) give the characteristic value of effective unit weight of $\hat{\gamma}'=6.76$ kN /m^3. The characteristic value of peak friction angle is numerically obtained from the 5% fractile of the bounded tanh distribution shown in Eq. (7), which is $\hat{\varphi}_p=34.25°$. As discussed previously, the critical state friction angle, φ_{cs}, is assumed to be deterministic, which is taken as the lower bound of φ_p, i.e. $\varphi_{cs}=30°$.

The mean line tension F_T is assumed to have a mean of $\mu_{F_T}=500$ kN /m. The mean of the dynamic increment of line tension F_D is calculated as a certain portion of μ_{F_T}, i.e. $\mu_{F_D}=R_{D/T}\mu_{F_T}$ where $R_{D/T}$ is the ratio of μ_{F_D} to μ_{F_T}. In this study, a wide range of $R_{D/T}$ values from 0.2 to 3.0 is considered for a parametric study. Following the coefficient of variation ranges of different load effects for onshore and offshore foundations reported by Meyerhof[25], the range of the coefficient of variation is 0.05−0.15 for permanent actions and 0.3−0.5 for environment loads.

In addition, higher values are expected for offshore loading conditions that are generally associated with a relatively higher level of uncertainty[26]. Recall that F_T is considered as a permanent action as suggested by Reference[16], and F_D is expected to be more variable and is thus regarded as a variable action. The coefficients of variation of F_T and F_D are taken as v_{F_T} = 0.15 and v_{F_D} = 0.5, respectively, which are conservatively at the higher ends of the above recommended ranges.

DNV[11] specifies two sets of partial safety factors in relation to two consequence classes for the uplift limit state design of plate anchors, as presented in Table 1 along with the corresponding target annual probabilities of failure, p_m. The partial safety factors for the effective unit weight and peak friction angle of the soil shown in Table 1 are provided by Reference[15] as DNV[11] does not consider drained soil conditions and soil unit weight.

It is worth noting that for uplift limit state design, only the load partial safety factors are different between the two consequence classes and the soil partial safety factors remain the same.

Table 1 Partial safety factors and target failure probabilities for uplift limit state design of plate anchor

Consequence class	γ_φ	γ_γ	γ_T	γ_D	p_m
1	1.25	1.0	1.1	1.5	10^{-4}
2	1.25	1.0	1.4	2.1	10^{-5}

A total number of $n_{sim} = 1\times10^8$ realisations is employed, which is able to reasonably accurately estimate failure probabilities down to around $10/n_{sim} = 1\times10^{-7}$ with a standard deviation of the estimate appropriately equal to $\sqrt{p_f / n_{sim}} = \sqrt{10^{-7}/10^8} = 3.16\times10^{-8}$. The above-mentioned input parameters are summarised in Table 2.

Table 2 Input parameters for estimation of plate anchor failure probability

μ_{φ_p} /(°)	σ_{φ_p} /(°)	$\varphi_{p,min}$ /(°)	$\varphi_{p,max}$ /(°)	s	φ_{cs} /(°)	$\mu_{\gamma'}$ /(kN • m^{-3})	$\sigma_{\gamma'}$ /(kN • m^{-3})	μ_{F_T} /(kN • m^{-1})	v_{F_T}	v_{F_D}	$R_{D/T}$	n_{sim}
40	3.4	30	50	2.5	30	8	0.8	500	0.15	0.50	0.2–3.0	100 000 000

6.2 Results

Figure 4 shows the estimated reliability index of the strip plate anchor designed for uplift limit state using the partial safety factor method as a function of the ratio $R_{D/T}$ for the two consequence classes. It can be seen that as the ratio $R_{D/T}$ increases, the estimated reliability index presents a rapid decrease before gradually converging to a relatively steady state. More specifically, the reliability reaches $\beta = 3.9$ for Consequence Class 1 and $\beta = 4.6$ for Consequence Class 2. The primary reason is that increasing $R_{D/T}$ makes the more variable F_D more dominant in the estimate of β, thereby reducing the estimate reliability index (increasing the estimate failure probability).

The target failure probabilities ($p_m = 10^{-4} \to \beta_m$ = 3.74 for Consequence Class 1, and $p_m = 10^{-5} \to \beta_m$ = 4.26 for Consequence Class 2) suggested by DNV[11] are also incorporated in Fig. 4 for comparison. The results indicate that the current partial safety factor approach used for offshore plate anchor design is conservative over a wide range of $R_{D/T}$ values, particularly for lower $R_{D/T}$. It should be noted that the same prediction model has been applied to estimate the anchor uplift capacity in the design process and the estimate of failure probabilities, which means that no attempt has been made to consider uncertainties associated with the geotechnical prediction model in this analysis. As discussed previously, the upper bounds of the coefficients of variation of F_T and F_D have been conservatively applied in this analysis. These conservative and unconservative factors generally cancel one another out to some extent.

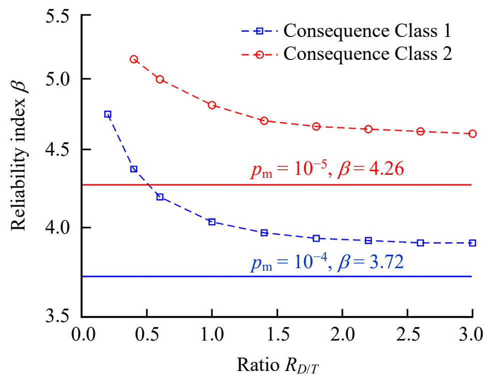

Fig. 4 Reliability index of designed strip plate anchors for uplift limit state

7 Conclusions

This study conducted a reliability analysis of designing strip plate anchors in sand for uplift limit state under pure vertical loading. To achieve this, the partial safety factor approach was first applied to calculate the required anchor embedment depth, which was then employed to estimate the failure probability of the designed anchor using Monte Carlo simulations. This analysis considered two consequence classes corresponding to two different sets of partial safety factors and target acceptable failure probabilities. A wide range of $R_{D/T}$ values was taken to investigate its effect on the estimation of failure probability.

The results suggest that the estimated reliability index decreases rapidly with increasing the ratio $R_{D/T}$ for $R_{D/T}$ < 1.5 and gradually converges to a relatively steady state, because the more variable F_D becomes more dominant than F_T with the increase of $R_{D/T}$. Compared to the target failure probabilities suggested by DNV[11], the current partial safety factor approach used for offshore plate anchor design is conservative over a wide range of $R_{D/T}$ values, particularly for lower $R_{D/T}$. It is worth noting that the reliability index may be conservatively estimated for several reasons. One is that, the upper bounds of the coefficients of variation of F_T and F_D have been applied. In addition, the positive correlation between φ_p and γ' has been ignored, which is likely to overestimate the calculated failure probabilities[23]. On the other hand, the uncertainties related to the uplift capacity prediction model have not been accounted for. Thus, further work is required to examine whether the partial safety factor method is still overly conservative when considering all these conservative and unconservative factors. In addition, for practical applications, a more accurate estimate of $R_{D/T}$ is essential to determine the required partial safety factors.

In general, these conservative and unconservative factors cancel one another out to some extent, and the current results are considered to be reasonably accurate and can provide probabilistic insights and guidance into the reliability of plate anchors for offshore wind turbines.

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