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Time-Dependent Reliability Analysis of an Existing Sheet Pile Wall Case Study

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Abstract: This paper presents the time-dependent reliability analysis of a case study of a sheet pile wall, incorporating the degrading effects of corrosion and the reliability increasing effects of the proven service, i.e. the survived years until the year of assessment. The results show a decrease of the reliability over time due to corrosion, and an increase of the reliability due to the survived years. The results show a shift of the influence coefficients from the time-invariant soil parameters (which's uncertainty reduces) to the time-variant (load) parameters. The uncertainty reduction potentially leads to more favorable partial factors for the assessment of existing structures. Further, the results highlight the influence of corrosion uncertainty and model uncertainty, two factors which are currently not explicitly covered by a partial factor in the guidelines for existing structures.

Keywords: reliability analysis, sheet pile, past performance, degradation.

1 Introduction

Many existing quay walls and sheet pile walls which have been built in the 70's and 80's of the 20th century approach the end of the intended design lifetime of 50 years. Often these structures are still in good condition and have been in service for years without any significant problems in terms of deformations and structural safety. If the lifetime of such structures can be safely extended, these constructions can be preserved, preventing expensive renovations and reconstructions, leading to large saving in costs and material.

This paper presents the time-dependent reliability analysis of a case study of a sheet pile wall, incorporating the degrading effects of corrosion and the improving effects of the proven service, i.e. the survived years until the year of assessment, see the schematic in Figure 1. This paper considers three different failure mechanisms: yielding of the front wall, yielding of the steel anchor, and instability of the passive soil wedge. To assess the reliability, we use the approach of Roubos (2019) to account for survived years and degradation due to corrosion. The reliability is calculated using the First Oder Reliabliity Method (FORM) to estimate the reliability of individual years with a beam-spring model (Deltares SheetPiling), and the Equivalent Planes method (EPM, Roscoe (2015)) to combine the event of failure in future years with the events of survival in all preceding years.

The results provide insight in the decrease of the reliability and safe service-life due to uncertain corrosion, and the increase of the reliability and service-life due to the uncertainty reduction of time-invariant stochastic variables due to proven service. It is shown how the influence of epistemic uncertainties such as (time-invariant) soil properties and model uncertainty decrease over the lifetime for different failure mechanisms. Contrary, the influence of the time-variant variables such as loads increase (e.g. Klerk et al. 2018). As a result, design values at the end of the lifetime (e.g. for application in codes for assessment) may differ substantially from the design values in the first year, or without proven strength.

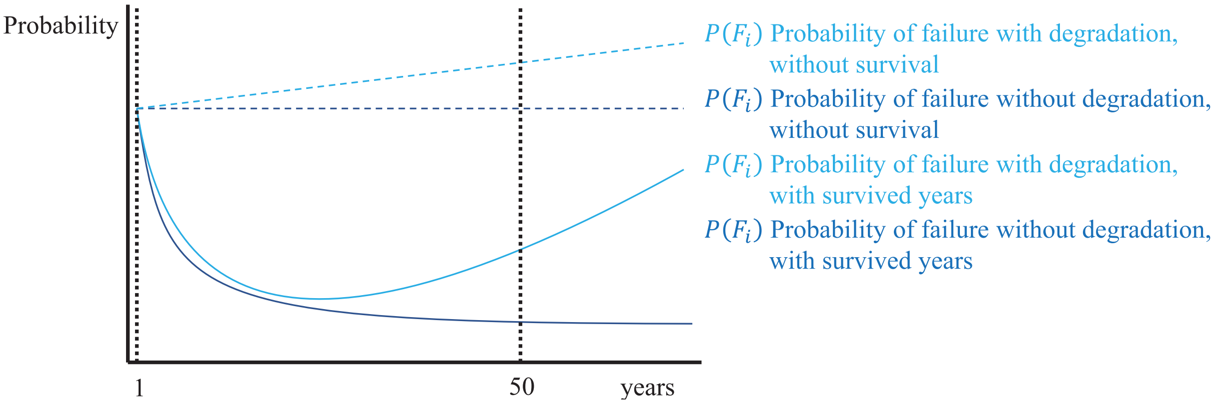

Figure 1 Schematic change of the failure probability without degradation (dark blue) and with degradation (light blue).

2 Time dependent reliability

To calculate the reliability of an existing sheetpile structure including the effects of proven strength, we are interested in the probability $P_{f,i}$ that failure occurs in year *i* in the future, conditional to survival until the year of assessment *s* (known), and survival in all subsequent years until year *i* (uncertain). Therefore,

$$P_{f,i} = P\left(F_i \cap \left\{\overline{F}_s \cap \overline{F}_{s+1} \cap ... \cap \overline{F}_{i-1}\right\} | \left\{\overline{F}_1 \cap \overline{F}_2 \cap ... \cap \overline{F}_{s-1}\right\}\right), \tag{1}$$

where F_1 = failure in year *i* (event), $\overline{F}_1$ and $\overline{F}_{i-1}$ = survival in year 1, resp. year *i*-1. Here, $P(\overline{F}_1) = 1 - P(F_1)$. The correlation between the failure events is accounted for in this formulation. The failure probability in year *i* without survival can also be expressed as the product of the conditional probability $P_{f,i,cond.} = P(F_1 | \overline{F}_1 \cap ... \cap \overline{F}_{i-1})$, and the probability of survival in all preceding years:

$$P_{f,i} = P(F_i | \overline{F}_1 \cap ... \cap \overline{F}_{i-1}) \cdot P(\overline{F}_{i1} \cap ... \cap \overline{F}_{i-1}). \tag{2}$$

For small probabilities, $P(\overline{F}_1 \cap ... \cap \overline{F}_{i-1}) \approx 1$, so $P_{f,i} \approx P(F_i | \overline{F}_1 \cap ... \cap \overline{F}_{i-1})$. For convenience, we therefore choose to only adopt the approach using conditional probabilities:

$$P_{f,i} \approx P(F_i | \overline{F}_1 \cap ... \cap \overline{F}_{i-1}). \tag{3}$$

The above formulation is suitable to calculate annual probabilities. To make the relation with probabilities for reference periods larger than 1 year (e.g. like in the Eurocodes), a cumulative probability could be calculated: $P_{f,i,cum.} = P(F_i) + \sum_{j=2..i} P_{f,i}$. Alternatively, if Eurocodes would prescribe reliability targets for reference periods of 1 year, we could directly compare the reliability with these targets. In this paper we only consider annual probabilities.

In this paper the failure probability of Equation (3) is determined using FORM analyses (Hasofer and Lind 1974) for the individual years (both failure and survival), and the Equivalent Planes Method (EPM, Roscoe et al., 2015) to combine the information for failure and survival in the different years. The general approach is to combine the events subsequently, using the linearized design point from the FORM calculation (design point defined by reliability index ² and influence coefficients ±). The EPM determines here equivalent ±- and ²-values for the combined years, while accounting for the correlation between the years based on the influence coefficients and the auto-correlation in time of each variable ρ_j. For example, for year 1 and 2:

$$\rho_{1,2} = \sum_j \alpha_{j,1} \cdot \alpha_{j,2} \cdot \rho_j. \tag{4}$$

We use $\rho_j = 1$ for time-independent soil properties and model uncertainty, and $\rho_j = 0$ for loads and other time-dependent variables.

3 Case Study

3.1 Case description

We consider the time-dependent reliability and the influence coefficients for a case study of a fictitious sheet pile structure next to a canal. The case study was previously used to derive semi-probabilistic design rules for sheet pile walls (GeoDelft, 1991), and re-evaluated probabilistically (Laghmouchi, 2021).

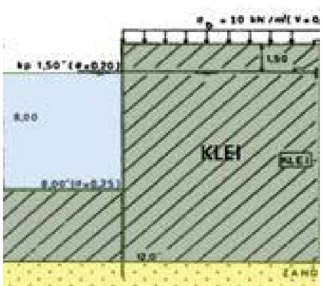

Figure 2 Schematic of the considered case study

The case study considers a 12 meter clay layer, on top of a sand subsoil, see Figure 2. The canal depth is 8 meter, the sheetpile wall 12,65 meter, and is anchored. The sheetpile profile's moment of inertia I is $3.0.10^{-4}$ m^4/m, comparable with AZ17-AZ19 sheetpile profiles. The initial diameter of the anchor bar is design as 50 mm.

We consider the following three limit states: yielding of the steel in the outer-most fibre, yielding of the steel anchor, and geotechnical instability by failure of the passive soil wedge. Only ultimate limit states are considered. The limit states are abbreviated in this paper by WALL, ANC, GEO, see Table 1. Herin, g is the limit state function, X the stochastic variables, and t the considered year (to model corrosion). M_{wall} is the bending moment in the sheetpile wall, W the section modulus, and f_y the yield strength of the steel. F_{anchor} is the force in one anchor bar, A_{anchor} the cross-sectional area of the anchor bar, and $f_{y,anchor}$ the yield strength of the anchor steel. $U.c._{MobilizedResistance}$ is the unity check for mobilized versus maximum shear resistance of the soil. M_{wall}, F_{anchor} and $u.c._{MobRes}$ are all output from the spring-model DeltaresSheetPiling. A limit of 0.99 in the GEO LSF is chosen because the model does not give output if u.c.e 1.0. The influence of this assumption is negligible.

The probability distributions of the stochastic variables are based on GeoDelft (1991), and summarized in Table 2 and Table 3.

Table 1. Considered Limit state formulations in the analysis

Limit state	Definition	Limit state formulation
WALL	Yielding of the steel in the outer-most fibre	$g_{WALL}(X,t) = f_{y,wall} - \theta_M \cdot \frac{\max(M_{wall})}{W_{wall}(t)}$
ANC	Yielding of the anchor bar	$g_{ANC}(X,t) = f_{y,anchor} - \theta_F \cdot \frac{F_{anchor}}{A_{anchor}(t)}$
GEO	Insufficient passive soil strength	$g_{GEO}(X,t) = 0.99 - \theta_R \cdot u.c._{\text{Mobilized Resistance}}$

Table 2. Stochastic soil parameters

		Clay layer	Sand layer
φ'	Friction angle	Lognormal(22.5, vc=0.08)	Lognormal(32.5, vc=0.08)
c'	Cohesion	Lognormal(22.5, vc=0.2)	Deterministic(0)
k_1	Stiffness parameter	Lognormal(3250, vc=0.3)	Lognormal(32500, vc=0.3)
r	Ratio for angle $\delta = r \cdot \varphi'$	Lognormal(0.5, std=0.08)	Lognormal(0.667, std=0.08)
$\gamma_{sat.}$	Saturated volumetric weight	Lognormal(18.0, vc=0.05)	Lognormal(18.0, vc=0.05)

Table 3. Other stochastic parameters

		Distribution
$\theta_M, \theta_F, \theta_R$	Model uncertainty of the calculated bending moment, anchor force, and passive resistance	Lognormal(1.0,v.c.=0.1)
Q	Surcharge load	Gumbel(13.0, v.c.=0.2)
z_{-8} *	Bed level at -8	Gumbel(shift=-7.66,std=0.14)
z_{-12}	Layer separation at -12	Normal(-12.0,std=0.20)
h_w	Outside water level	Gumbel(shift=-1.50, std=0.04)
h_p	Phreatic level inner side	Gumbel(shift=-1.68, std=0.04)
$f_{y,wall}, f_{y,anchor}$	Steel yield stress	Lognormal(276.10^3, v.c.0.08)

* It is assumed that the bed level is frequently inspected, and that the bed level is restored if scour has occurred. Hence, cumulative scour over multiple years is not considered in this paper.

3.2 Corrosion

The sheet pile wall is located along a channel with fresh water. The corrosion is modeled by a cumulative decrease in the thickness of the sheet pile profile (affecting W_{wall}) and the diameter of the anchor (affecting A_{anchor}). The expected values for the corrosion (the spatial-average relevant for the structural resistance) are taken from a recently published Dutch code NEN6766 (NEN, 2021). For the uncertainty of corrosion, the corrosion rate is modeled with an uncertainty with a coefficient of variation of 30%, which assumes to reflect spatial variability and measurement uncertainty, assuming that the remaining thickness is measured at the moment of assessment. For corrosion of anchor rods, the expected values for the corrosion have been estimated from the conservative numbers from NEN6766.

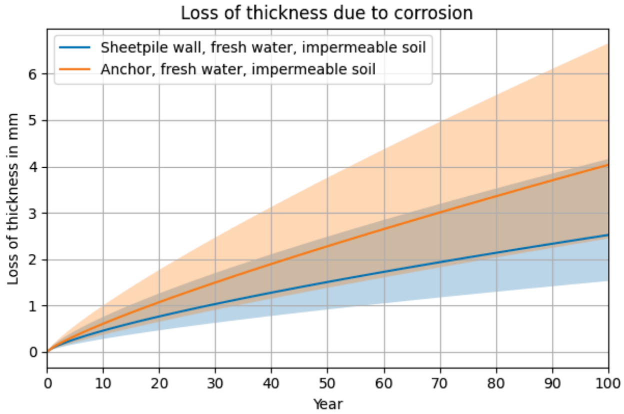

Figure 3 Modeled expected cumulative corrosion of the sheetpile wall thickness (left), and anchor thickness (right).

4 Results

The reliability is evaluated for the three limit states for 75 years. All three limit states have an initial annual reliability around 4. Due to corrosion, the reliability decreases quite rapidly, depicted by the blue lines in Figure 4. However, the effect of proven strength has a considerable effect on the annual reliability, shown by the orange lines in Figure 3. In year 75, the reliability increase is between 0.3-0.8 depending on the failure mechanism, for this case study. This is approximately a factor 5-10 lower annual failure probability. If we were to comply with a certain target reliability, e.g. 3.0 see the dotted line in in Figure 4, then the proven strength leads to an extension of the expected lifetime of 20-25 year. The GEO limit state is not affected by degradation, so the reliability is not expected to drop below a reliability of 3.0 in this case study.

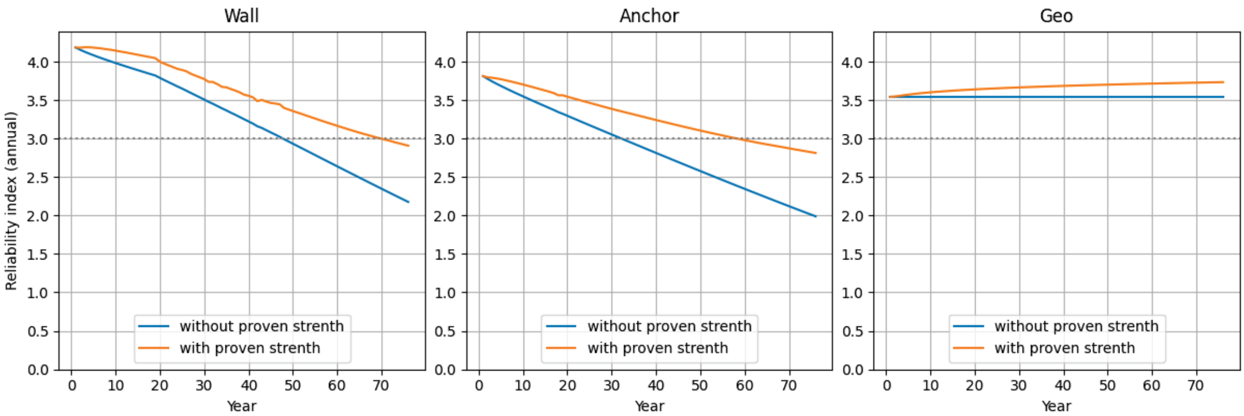

Figure 4 Reliability for the three limit states with (orange) and without (blue) proven strength.

Incorporating the information of survived years leads to a reduction of the epistemic uncertainty in the soil properties and model uncertainty. This uncertainty reduction is reflected by the decrease of the influence coefficients over time in Figure 5. The results for the WALL limit state are in accordance with the results presented by Lagchmouchi (2021). The large influence of the uncertainty of the corrosion stresses the importance of frequently measuring the thickness decrease. The results also imply that improving the accuracy of the measurements and improving future predictions for the corrosion rate (e.g. using structural health monitoring, SHM), are also likely to improve reliability estimates.

The design values for any parameter in year 75 can be back-calculated from the design point in standard normal space (the equivalent ±- and ² -values for the combined years using EPM), using the prior probability distributions and correlation matrix. Although it is not entirely correct to use the prior probability distributions (instead of the actually changed distributions due to uncertainty reduction), the difference will not be too large, and the method followed is conservative in the sense that lower design values are found than if a truncated or reduced distribution was used. Based on the characteristic value $X_{i,char.}$ and the design value X^*_i for a parameter i, the partial load (multiply) or resistance factors (divide) can be calculated as follows:

$$\text{loads: } \gamma_S = X_i^* \big/ X_{i,char.} \qquad \text{resistances: } \gamma_R = X_{i,char.} \big/ X_i^* \,. \tag{5}$$

The case-specific partial factors for each of the considered limit states for this case study are shown in Table 4. The most important observation is that the partial factors for soil properties are considerably lower than the values currently in the guidelines. It is observed that the design values of the soil parameters are generally higher than the (5%-)characteristic values of the soil properties, and hence, the partial factors could be smaller than 1.0.

Although the values for the soil properties and loads in Table 4 are typically lower than in the current guidelines, we cannot indisputably determine whether the current guidelines are conservative or not. First, because model uncertainty is currently absent in the existing guidelines, whereas model uncertainty was modeled explicitly in the present analysis, leading to a partial model factor of approximately 1.1 on the calculated bending moment, and approximately 1.1 on the calculated anchor force. Secondly, a partial factor on the corrosion rate (or thickness reduction) might be applicable since the relative influence increases for ageing structures.

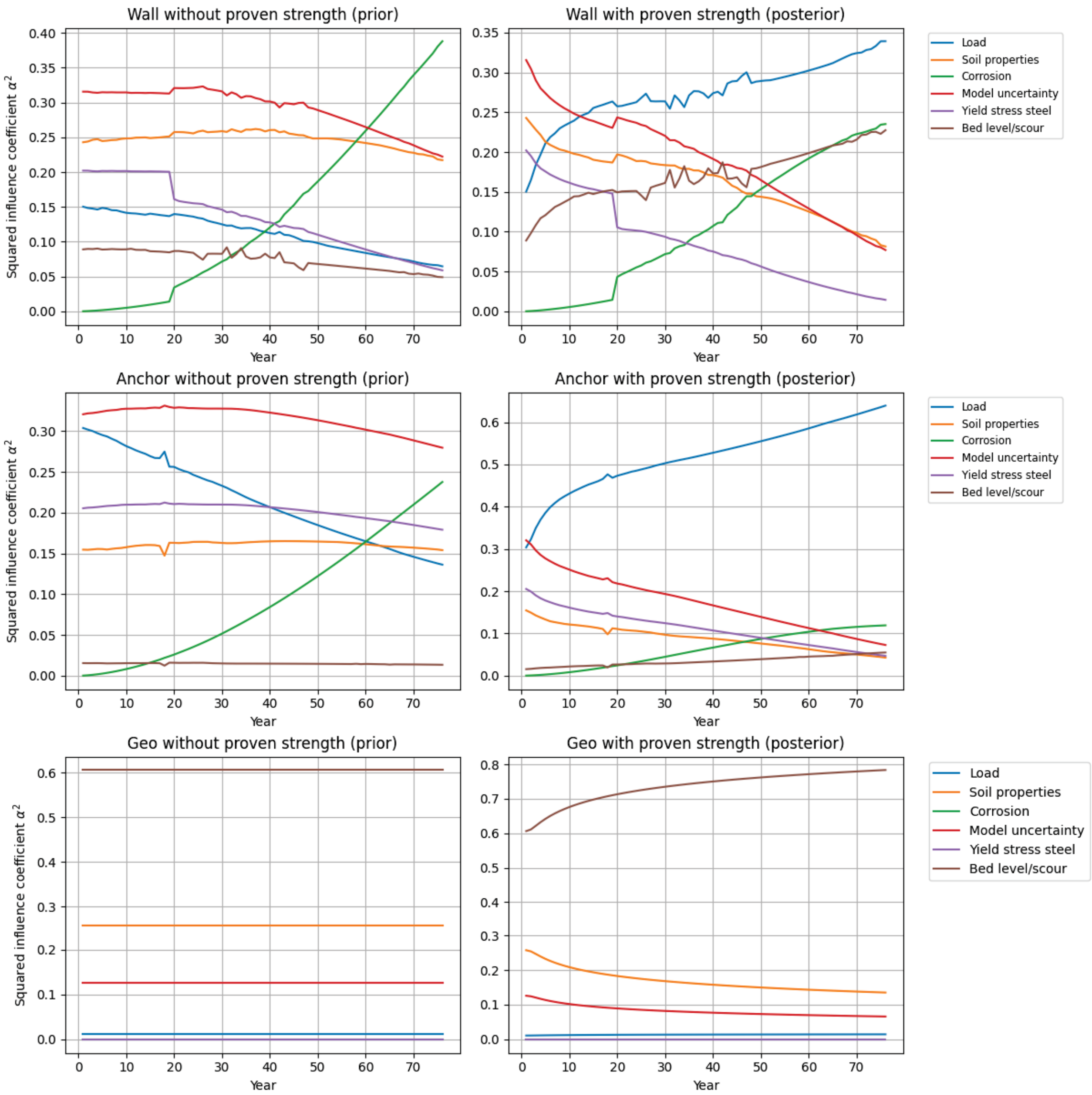

Figure 5 Development of the influence coefficients over time with (right) and without (left) proven strength. The influence of the individual soil parameters is summed.

Table 4. Case-specific partial factors based on design values in year 75, compared to current guideline for existing structures

Property	Characteristic value definition	Case-specific partial factor (this case study)			Partial factor in existing guideline
		WALL	ANC	GEO	NEN8707
Reliability index		2.9	2.8	3.7	1.8-3.3
Friction angle	5%	0.85-0.91	0.85-0.89	0.91-0.95	1.0-1.15
Cohesion	5%	0.70	0.71	0.70	1.0-1.15
Volumetric weight	50%	0.92	0.92	0.94-0.95	1.0
Stiffness (modulus subgrade reaction)	5%	0.67	0.64	0.67	1.0
Yield stress steel	5%	0.90	0.92	n/a	1.0
Load	95%	0.90	1.04	0.67	1.0
Corrosion	95% (50%)	0.95 (1.42)	1.04 (1.29)	n/a	n/a
Model	Mean value	1.08	1.07	1.09	n/a
Bed level*	n/a	$p_{exc} = 0.08$	$p_{exc} = 0.26$	$p_{exc} = 0.0005$	$p_{exc} = 0.04\text{-}0.01$
Water level*	n/a	$p_{exc} = 0.30$	$p_{exc} = 0.33$	$p_{exc} = 0.45$	$p_{exc} = 0.07\text{-}0.03$

5 Conclusions and recommendation

This paper analyzed the time-dependent reliability of sheet pile wall, considering the effects of corrosion and uncertainty reduction for time-invariant stochastic variables, by incorporating the survived years until the year of assessment (increasing the reliability and the lifetime). For the case study considered, the reliability can be 0.3-0.8 higher by accounting for survival information (which is equivalent to a 10 times lower failure probability), depending on the failure mode. The results also suggest that an extension of the safe lifetime of 20-25 years might be possible.

The results further show a shift of the influence coefficients from the time-invariant soil parameters (which's uncertainty reduces) to the time-variant (load) parameters. The uncertainty reduction potentially leads to more favorable partial factors for the assessment of existing structures. Besides, the results highlight the influence of corrosion uncertainty and model uncertainty, two factors which are currently not explicitly covered by a partial factor in the guidelines for existing structures.

The method in this paper enables a fully probabilistic safety assessments, for which the (annual) reliability result could be verified against (annual) target reliabilities. The case-specific partial factors for this case study also substantially differ from numbers for new structures in design codes, implying that optimizations to semi-probabilistic approaches are possible. For example, by calibrating and verifying a set of partial factors for the assessment of existing structures including the effects of corrosion and proven strength.

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