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FULL PROBABILISTIC DESIGN OF A SUBMERGED FLOATING TUNNEL AND FORMAT FOR PARTIAL SAFETY FACTORS

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Abstract. A submerged floating tunnel (SFT) can be a promising solution for crossing a deep or wide waterway. This innovative concept however lacks research into its probabilistic design. In this research, the reliability of the tether-stabilized SFT is assessed. A first-order reliability method (FORM) and a Monte Carlo simulation (MCS) are performed for the limit state functions of the most important failure mechanisms. Stochastic variables are chosen so that a target reliability index of 3.8 for a reference period of 50 years is met. The calculated factors from the full probabilistic design are compared with the general recommended partial factors for strength and resistance from Eurocode EN1990. For the strength mechanisms, the calculated factors are smaller than the factors from Eurocode. However, for the equilibrium mechanism, the calculated factor for the unfavorable loading is larger than the factor from Eurocode and should be increased by 10% in order to design a safe enough SFT.

Keywords: Eurocode EN1990, first-order reliability method, submerged floating tunnel, Monte Carlo simulation, partial safety factors, tether-stabilized, tether slackening, tether yielding.

1. Introduction

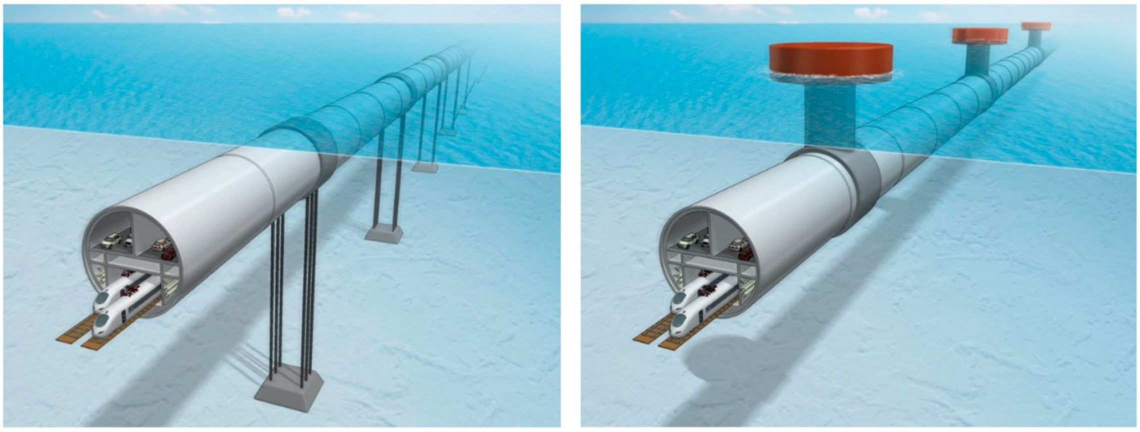
A solution for crossing a deep and wide waterway can be a floating tunnel under the water level: a Submerged Floating Tunnel (SFT). This type of tunnel consists of an immersed concrete tube, either attached with anchor cables to the seabed or attached to pontoons floating on the water surface. These two variants can be seen in Figure 1. The concept of an SFT was patented in the United Kingdom in 1886, and the first Norwegian patent on the subject was issued in 1923. In the 1960's a small group of Norwegian experts started to evaluate the potentials of the SFT concept. More recent, the government of Norway made plans for the highway route E39, crossing many fjords, to be ferry-free. This goal could be achieved by constructing SFTs crossing the fjords on this route [1].

In this paper, we focus on the reliability of the SFT and the applicability of the recommended general partial safety factors from Eurocode EN1990 [2]. Previous research focused mostly on the structural lay-out of the SFT. Few researchers have addressed the safety of the structure and no probabilistic design has been made before. The experience gained from other civil structures can only partly be used, since SFTs differ from common civil structures. In general, SFTs have different relevant limit states, different magnitude of consequences and costs involved with failure of relevant limit states, different accuracy of the models that predict the structural response and different load scenarios [3]. The forcing on underwater structures differs from the forcing on structures on land, since the equilibrium of the system is determined by the buoyancy-weight ratio. The general partial safety factors for strength and resistance from Eurocode EN1990 are calibrated on forces acting on buildings and bridges. It is questionable whether these factors can be applied to the SFT [2].

The aim of this paper is to perform a full probabilistic calibration of partial factors for most relevant limit states of the SFT. A level IV probabilistic analysis is not performed, since costs are difficult to determine and can vary significantly. Accidental loading types are out of scope for the limit state functions, as well as the loads on the land-bored part of the tunnel and the end-joints.

This paper is structured as follows: First, the methodology is presented (Section 2), in which a typical SFT structure is shown, a target β-value is set and the calculation of partial factors is explained. Consequently, in Section 3, the failure mechanisms are described and all stochastic variables are explained. In Section 4, the resulting reliability indices and partial safety factors are discussed, followed by the overall conclusions (Section 5) [4][1].

[1]This paper is based on the authors thesis, Reliability analysis of Submerged Floating Tunnels, 2020 [4]



(a) (b)

Figure 1. Two variants of an SFT

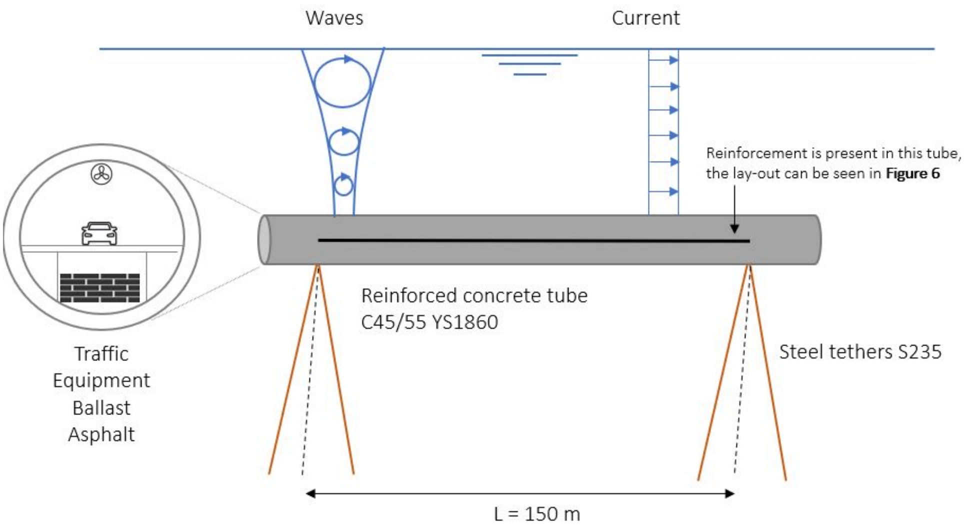


Figure 2. Simple model of SFT-element with loading types

2. Methodology

2.1. Procedure

First, a typical SFT structure is investigated (Section 2.2). In order to assess the reliability of the SFT, a decomposition of this system is made (Section 2.3). The governing failure mechanisms are determined. Consequently, the reliability requirements on element level are set. A target β-value is selected for a certain reference period and limit state functions are formulated (Section 3). Python's PyRe module and Prob2B are used to perform a FORM and MC analysis. When the failure mechanisms are tuned to the target β-value, the partial factors can be calculated (Section 4). [4]

2.2. Typical SFT-structure

The design made for the Bjørnafjorden in Norway concerns the typical failure mechanisms of an SFT structure, with a certain depth and size, and is therefore chosen as reference in this paper. The structure consists of two identical concrete tubes with steel reinforcement. Between the two tubes, there are cross-beams and diagonals. The diameter of the tube is not the same along its length, since there are emergency lanes every 250 m. The tube elements are 200 m, and the total length of the SFT is approximately 5400 m. The tube is stabilized by steel tethers attached to the seabed with drilled and grouted rock anchors.

For this structure, typical loading types were defined, e.g. the permanent downward loads, the buoyancy force, the traffic loading and the wave-current loading (Figure 2). For axial loading, the hydrostatic load is considered. The vortex-induced vibrations, temperature loading and creep and shrinkage turned out to be negligible. [4]

2.3. Decomposition of the system

The entire structure of the SFT can be seen as one system. We need to make a decomposition of this system in order to assess the reliability on element-level, conform Eurocode EN1990 [2]. Figure 3 shows a decomposition in multiple SFT-sections, where each section can fail separately.

One SFT-element is considered here as one single tube. The framework in between the tubes is not

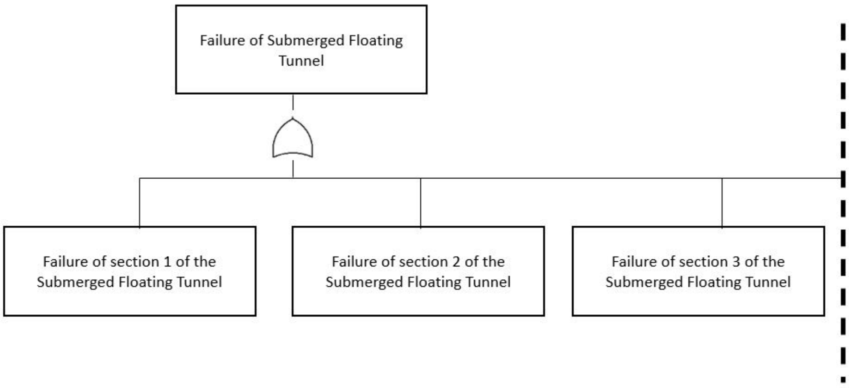


Figure 3. Fault tree for an SFT

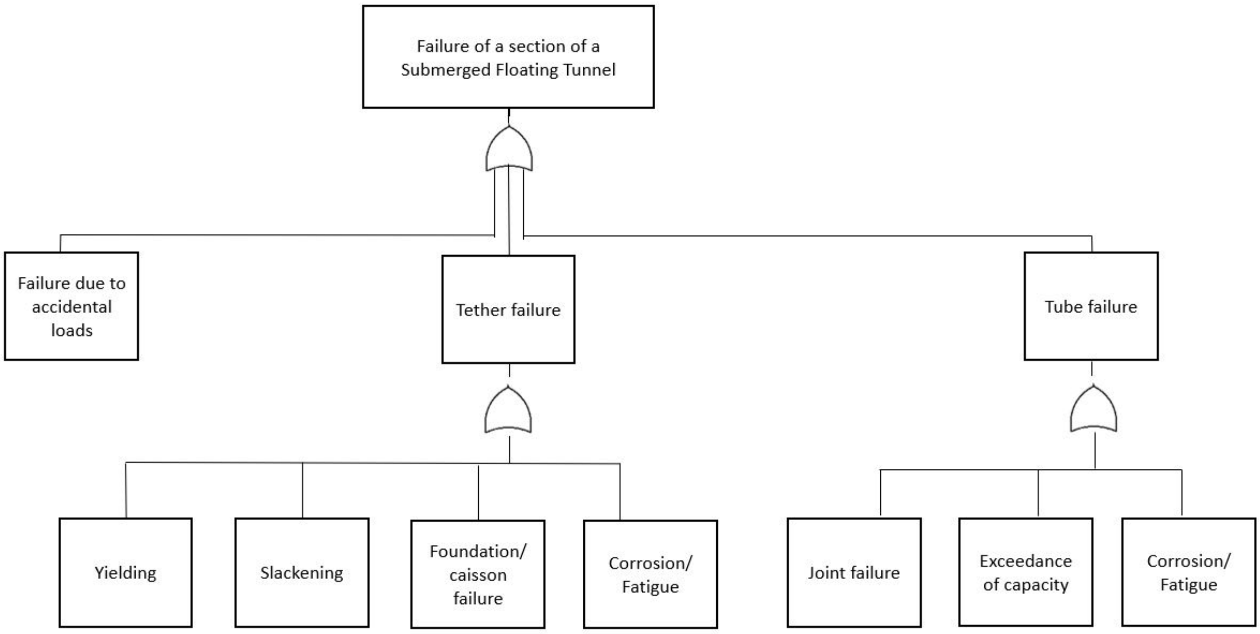


Figure 4. Fault tree for an SFT-element

included in the calculations, since we only look at the most important force actions. The tube is attached to the bottom of the sea with tethers [1]. We assume four tethers per mooring for our model input. On element level, multiple relevant failure mechanisms can be defined, as can be seen in Figure 4.

From design experience, the following four mechanisms can be assumed as most important: yielding (1), slackening (2), failure of the tube in longitudinal direction (3) and shear failure of the tube in transverse direction (4). More about these mechanisms will be explained in Section 3. Compression in transverse direction was first assumed to be an important failure mechanism as well. However, it is proven that the compressive stress is small compared to the concrete resistance. Within the scope and planning of this study, it has been assumed that the additional failure mechanisms of corrosion, fatigue, geotechnical failure and accidental failure are not governing due to their complexity and expected research time needed. For geotechnical failure extensive research into the geolocation should be performed [4].

3. Reliability of main failure mechanisms

In this section, the four most important failure mechanisms are explained and their limit state functions are formulated. The input parameters are defined by their mean values and standard deviations, and can all be found in Table 1. A β-value of 3.8 for a reference period of 50 years is chosen for the individual failure mechanisms, mainly since general partial factors from Eurocode EN1990 are also determined on a β-value of 3.8 with a reference period of 50 years [2]. Looking at the fault trees in Figure 4 and Figure 3, we expect the system β-value to be lower than the element β-value. However, this subject is not further studied in this paper [4].

3.1. Yielding of the tethers

When the stress in the cross-section of the tether reaches the yield point, the steel begins to deform plastically. The yield point is the point where nonlinear (elastic and plastic) deformation begins. Yielding of the tether can occur when the buoyancy force increases or the downward force decreases. This change of loading can take place due to erosion, temporary replacements, shortage of ballasting, or due to waves. The yield strength of steel and the cross-sectional area determine the strength of the tether. For this study, not all tethers need to be investigated separately because equal properties are assumed. Both the models for resisting forces and load contain uncertainty, because the used mathematical models are not a full representation of reality. This is included in model factors for uncertainty (θ_R and θ_S).

$$Z = \theta_R \cdot F_{resistance,steel} - \theta_S \cdot (F_{buoyancy} + F_{wave-current} - F_{concrete} - F_{ballast} - F_{asphalt} - F_{equipment} - F_{marine}) \quad (1)$$

The resistance- and strength parameters can be plotted in a joint probability density function (5, left). The failure point is the point where the line $Z = 0$ and the contour plot of the function intersect (5, right). From this contour plot, the design points and α-values can be derived. For all four mechanisms, these plots have been assessed [4].

3.2. Slackening of the tethers

Slack means that there is no tension in the cable anymore, and the stiffness is zero. As a result of slackening, snap forces occur, which can lead to structural failure of the system. Due to marine accumulation, extra ballasting, or leakage, slackening can occur. Loosening of the anchorages can also occur due to waves, and often lasts for one to two seconds during each wave run.

In order to prevent slack, an equilibrium should be maintained between upward and downward forces. In the governing situation for slackening, lift force acts as a downward force. The accompanying drag force has no vertical component, so this force is not included in the formulation [4].

$$Z = \theta_R \cdot F_{buoyancy} + \theta_S \cdot (-F_{wave-current} - F_{concrete} - F_{ballast} - F_{asphalt} - F_{equipment} - F_{traffic} - F_{marine}) \quad (2)$$

3.3. Longitudinal failure of the tube

In the longitudinal direction, the tube can deform due to tidal differences or excitation due to waves or currents. Furthermore, sea level rise and extreme water levels can cause larger hydrostatic loads on the cross-section, which increases cross-sectional forces and moments. The point along the cross-section with the largest bending moment needs to be further investigated. For this point, two cases need to be investigated:

- Lower limit: The elastic moment with zero tension in cross-section
- Upper limit: The plastic moment capacity

For an elastic stress state, all stress distributions are considered to be linear. The stress at the bottom of the cross-section can be calculated according to Equation 3.

$$\sigma_{c,total} = -\frac{N_{ext}}{A_c} - \frac{N_p}{A_c} - \frac{N_p \cdot e(x)}{W} + \frac{M_{ext}(x)}{W} \quad (3)$$

$$W = \frac{\pi \cdot (D^4 - d^4)}{32 \cdot D} \quad (4)$$

where:

x	= longitudinal coordinate [m]
$\sigma_{c,total}$	= total stress in the cross section [kN/m^2]
M_{ext}	= applied bending moment [kNm]
N_{ext}	= normal force due to hydrostatic pressure [kN]
W	= section modulus [m^3]
N_p	= normal force due to prestressing [kN]
A_c	= concrete area of the tube [m^2]
e	= assumed eccentricity of the post-tensioning [m]
D	= outer diameter [m]
d	= inner diameter [m]

It is assumed that the resulting compressive stress in the concrete cross-section is within the acceptable limit. The cross-section should not be subjected to tensile stresses. The limit state function for this mechanism is stated in Equation 5.

$$Z = \theta_{\sigma R} \cdot \left(\frac{N_p}{A_c} + \frac{N_p \cdot e}{W}\right) - \theta_{\sigma S} \cdot \left(\frac{N_{ext}}{A_c} + \frac{M_{ext}}{W}\right) \quad (5)$$

In reality, a larger external moment can be applied to the cross-section before it fails in terms of cracking and transmitting the load.
The cross-section can be seen in Figure 6.

In any case, the structure fails when the moment exceeds the plastic moment capacity. For the maximum plastic moment capacity, it can be assumed that a third of the cross-section is yielding with an inner arm of 0.8·D [6].

$$M_{plastic,max} = \frac{1}{3} \cdot A_c \cdot f_y \cdot 0.8 \cdot D \quad (6)$$

The plastic moment at this point is:

$$M_{plastic} = \frac{1}{16} \cdot q_{total} \cdot L^2 \quad (7)$$

The following limit state results in the upper limit for longitudinal failure: [4]

$$Z = M_{plastic,max} - M_{plastic} \quad (8)$$

[2]NORM= Normal distribution, LOG= Lognormal distribution, GUM= Gumbel distribution, DET= Deterministic value

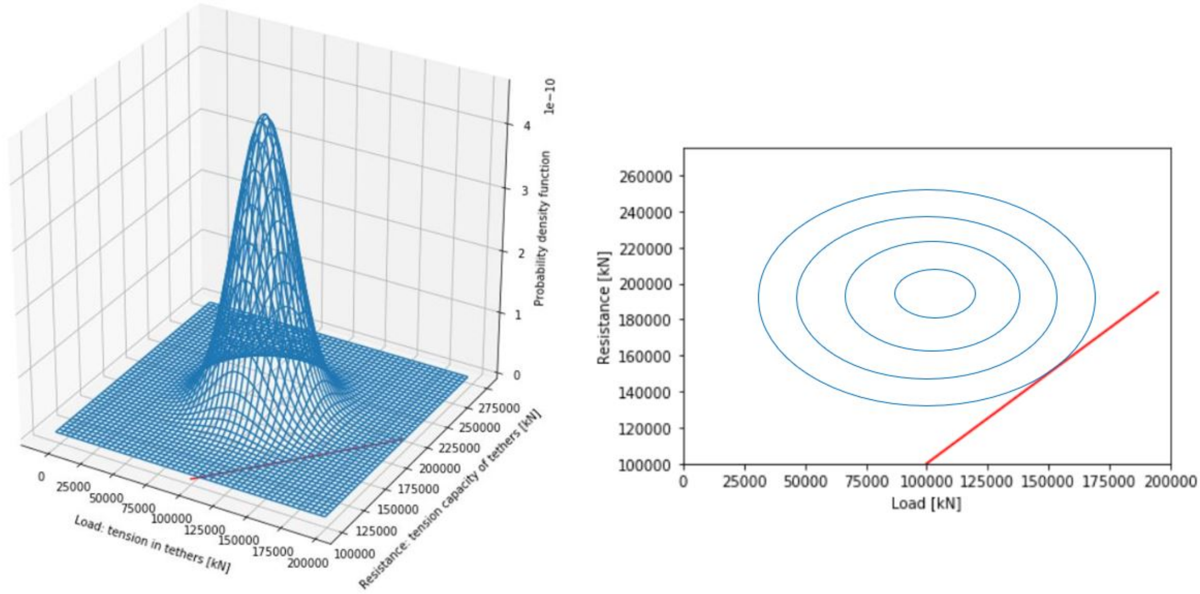


Figure 5. Joint probability density function (left) and contour plot (right) for yielding

Parameter	Distr. [2]	Mean	St.dev.	CV	Source
Cross-section tether (A_t) [m^2]	NORM	0.67	0.017	0.025	[1]
Yield strength S235 (f_y) [N/mm^2]	LOG	285	20	0.07	[5]
Diameter of the tube (D) [m]	NORM	15.0	0.375	0.025	[1]
Thickness of the tube (t) [m]	NORM	0.85	0.021	0.025	[1]
Length of the tube (L_{tube}) [m]	DET	150	-	-	[1]
Gravitational acceleration (g) [m/s^2]	DET	9.81	-	-	
Unit weight of concrete (γ_c) [kN/m^3]	NORM	24.5	1.7	0.07	[6]
Unit weight of water (γ_w) [kN/m^3]	NORM	10.035	0.4	0.04	[1]
Drag coefficient (C_D) [-]	LOG	0.7	0.2	0.3	[7]
Lift coefficient (C_L) [-]	LOG	0.1	0.02	0.2	[7]
Wave-current velocity for 50 years (u_c) [m/s]	GUM	1.5	0.15	0.1	[1]
Structural weight of asphalt ($q_{asphalt}$) [kN/m]	NORM	28	2.8	0.1	[1]
Weight of permanent equipment ($q_{equipment}$) [kN/m]	NORM	10	1	0.1	[1]
Weight of average solid ballast ($q_{ballast}$) [kN/m]	NORM	100	10	0.1	[1]
Traffic load for 50 years ($q_{traffic}$) [kN/m]	GUM	50	7.5	0.15	[8]
Marine load (q_{marine}) [kN/m]	NORM	10	2	0.2	[1]
Area of strands (A_{strand}) [mm^2]	DET	150	-	-	[9]
Initial strength of prestressing Y1860 (f_s) [N/mm^2]	LOG	1300	90	0.07	[5]
Concrete compressive strength C45/55 (f_c) [N/mm^2]	LOG	53	8	0.15	[6]
Model uncertainty for shear capacity θ_1 [-]	LOG	1	0.2	0.2	[6]
Model uncertainty for resistance θ_R [-]	LOG	1	0.05	0.05	[7]
Model uncertainty for load effects θ_S [-]	LOG	1	0.05	0.05	[7]
Model uncertainty for the capacity of the cross-section $\theta_{\sigma R}$ [-]	LOG	1	0.05	0.05	[7]
Model uncertainty for stresses in the cross-section $\theta_{\sigma S}$ [-]	LOG	1	0.05	0.05	[7]

Table 1. Input parameters for reliability analysis

Parameter	Yielding	Slackening	Longitudinal failure	Shear failure
β	3.8	3.8	3.8-6.7	5

Table 2. The β-values per failure mechanism

3.4. Shear failure of the tube

A large shear force can occur at the attachment points of the tethers to the tube, due to wave-current forcing. This can lead to shear failure. It turned out that the limit state for shear failure resulted in a β-value of 5. The β-value of shear failure could be tuned near 3.8 by changing the intermediate distance of the tethers. However, it is better to have longitudinal failure first, since this is a ductile mechanism. Since the probability of failure due to shear is now significantly lower

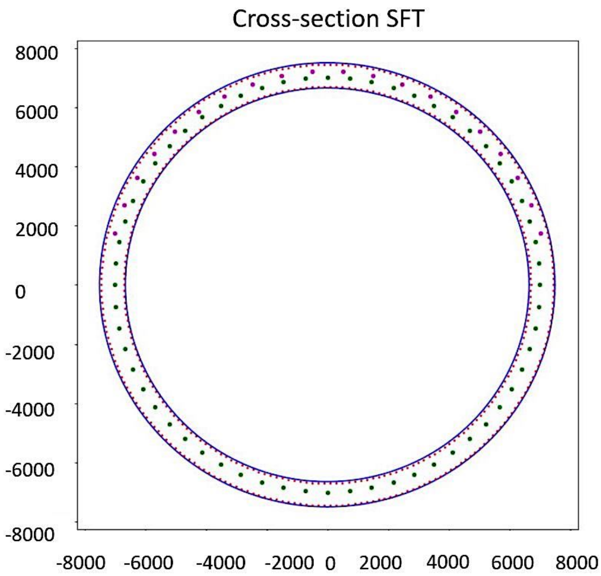


Figure 6. Lay-out of reinforcement: normal reinforcement in green and eccentric reinforcement in red

than failure due to one of the other mechanisms, this mechanism is not taken into account for the partial factor analysis [4].

4. Results and Discussion

4.1. Interpretation in terms of β's and α's

The resulting β-values from the four failure mechanisms can be found in Table 2.

The β-value for yielding is a slight underestimation of the real β-value. This is caused by two factors: the traffic load was not taken into account (1) and the wave-current force was only taken as unfavorable loading (2). The β-value for slackening resulted in 3.8, which is also a slight underestimation, since the lift force was only taken as unfavorable loading. The β-value for longitudinal failure turned out to have a value between 3.8 (lower limit) and 6.7 (upper limit).

The sensitivity factors (α) are presented in Table 3. Only the parameters with an α-value larger than 0.1 are shown. The diameter and thickness have two α-values, due to the uncorrelated and correlated calculation respectively. The diameter is dominant over the thickness, and the parameters serve as loading parameters for yielding and as resistance parameters for slacking. The α-value of D decreases significantly for the correlated case. The largest α-values are found for the concrete density, water density, yield strength of the steel tethers, the diameter of the tube and the model uncertainties. The variable loads, i.e. traffic load and wind-current load, have relatively low α-values.

Most α-values are smaller than either 0.8 (dominant resistance parameter) and -0.7 (dominant loading parameter) or 0.28 (other resistance parameters) and -0.32 (other loading parameters) from Eurocode EN1990 [2]. This means that the α-value multiplied with β-value results in a smaller distance between the design point and the mean value than Eurocode EN1990, which consequently leads to a smaller partial factor. In a FORM analysis, no distinction is made upfront between dominant and non-dominant parameters. Furthermore, when more parameters are added, the lower the α-values become, because the sum of the α-values squared always adds up to one [4].

4.2. Interpretation in terms of γ's

For the assessment of partial factors, the parameters with α-values above 0.1 are chosen stochasts, and the rest of the values as deterministic parameters. Consequently, the design points of the stochasts are calculated. Parameters are divided into favorable load, unfavorable load and variable load. Then, characteristic values according to Eurocode EN1990 were determined [2]. The partial factors of the full probabilistic design can be calculated by using this design value and characteristic value.

Eurocode EN1990 distinguishes in types of calculations [2]. For a strength calculation, the STR-conditions can be used. These factors can thus be compared with the calculated factors from tether yielding and longitudinal tube failure. For an equilibrium calculation, the EQU-conditions can be used. These factors can be used for slackening of the tethers. The factors according to the STR- and EQU-conditions and the calculated partial factors, according to the probabilistic design, can be found in Table 4.

For the STR-criteria, the favorable permanent load factor and the material resistance factor fit well. However, the general partial factors for the unfavorable permanent load and for the variable load seem to be very conservative. Based on this case study, the general partial factors for variable loading can be decreased with at least 20 %. Since the influence of variable loads on the structure is small, the decrease

Parameter	α Yielding	α Slackening	α Longitudinal failure
f_y [N/mm^2]	0.50	-	-
γ_w [kN/m^3]	-0.48	0.43	-0.52
γ_c [kN/m^3]	0.47	-0.52	0.52
f_s [N/mm^2]	-	-	0.55
D [m]	-0.46, -0.04	0.27, 0.11	-0.35, -0.11
t [m]	0.13, -0.34	-0.19, 0.08	0.14, -0.21
θ_R [-]	0.32	0.48	-
θ_S [-]	-0.29	-0.48	-
$\theta_{\sigma R}$ [-]	-	-	0.32
$\theta_{\sigma S}$ [-]	-	-	-0.32

Table 3. The α-values from FORM analysis

Parameter	STR	EQU	Yielding	Slackening	Longitudinal failure
Resistance	1.15	-	1.07	-	1.08
Unfavorable permanent load	1.35	1.1	1.23	1.19	1
Favorable permanent load	1	0.9	0.98	0.82	0.86
Unfavorable variable load	1.5	1.05	1.18	1	1.04

Table 4. Summary of the resulting partial factors from the models

of this factor will not result in significant changes in the design. Thus, the economic advantage is small. In contrast, a change in water density will have a large influence on the design. Thus, decreasing the general partial factor for unfavorable permanent load will have a significant effect on the design. According to this case study, the factor of 1.35 could be decreased with almost 10 %. However, conclusions cannot be too firm, since this calculation was performed for one case only. For the EQU-criteria, partial factors from Eurocode EN1990 are not safe to be applied. The partial factor for unfavorable permanent loading is insufficient. This factor should be increased by 10% in order to design a SFT [4].

5. Conclusions

This paper showed that the reliability requirements of the SFT can be met in the design, and that the design can be optimized by a full probabilistic calibration of partial factors. The following conclusions were drawn:

- The conventional reliability methods (i.e. FORM and MC) can be adopted in a reliability based design of an SFT. Permanent loads proved to be dominant, i.e. concrete and water density, and thus significantly influence the design of the SFT. The same reliability methods can be applied to other SFTs, but they might result in different designs based on geolocation specific circumstances.
- A target reliability is required to perform the reliability analyses. The general partial factors from Eurocode EN1990 are based on a β-value of 3.8 for a reference period of 50 years [2]. Therefore, in this research, a target β-value of 3.8 for 50 years was chosen as starting point for the individual failure mechanisms. Design parameters were determined so that this target value was met. The actual β-values were computed using simplifications that are by definition safe. On system level, it is expected that the β-value is lower than 3.8, but this is not studied further.
- Four important failure mechanisms for the SFT were derived using a decomposition of the system. These important mechanisms are yielding of the tethers, slackening of the tethers, longitudinal failure and transverse shear failure of the tube. It has been assumed within the scope and planning of this study that the additional failure mechanisms of corrosion, fatigue, geotechnical failure and accidental failure are not governing due to their complexity and expected research time needed. For geotechnical failure extensive research into the geolocation should be performed. Slackening proved to be the governing failure mechanism over the other three mechanisms. The resistance of slackening depends on the force equilibrium, whereas the resistance of the other mechanisms depends on structural strength.
- The influence factors (α-values) from the FORM analysis indicated that the most dominant parameters were the concrete density, water density, yield strength of the steel tethers, the diameter of the tube and the model uncertainties. The resulting α-values from FORM do not only depend on the coefficient of variation, but also on the absolute value of the mean of the parameter. The permanent loading parameters turned out to be dominant, because their relative contribution to the total load is large. The variable loads, i.e. traffic load and wind-current load, have relatively low α-values.
- The general partial factors from Eurocode EN1990

were never calibrated on SFT-type of structures [2]. They seem nevertheless safe enough to apply for the strength (STR) cases (Table 4). The general partial factors for the unfavorable permanent load and for the variable load turned out to be very conservative. A decrease of 20% for the variable loading factor and 10% for the factor of unfavorable permanent loading is proposed. Since the influence of variable loads on the structure is small, the decrease of this factor will not result in significant changes in the design. In contrast, a change in one of the permanent loading parameters will have a large influence on the design. Thus, decreasing the factor for unfavorable permanent load will have a significant effect on the design and is economically attractive.

- The equilibrium (EQU) criteria from Eurocode EN1990 should be applicable to the slackening mechanism. However, the general partial factor for the unfavorable permanent loading turned out to be insufficient (Table 4). This factor should be increased by 10% in order to be able to design a safe enough SFT [4].

References

[1] Reinertsen, O. Dr. Techn. Olsen, Norconsult. Bjørnafjord submerged floating tube bridge 2016.

[2] EN1990. Eurocode 0: Basis of structural design 2011.

[3] M. Baravalle, J. Köhler. Risk and reliability based calibration of design codes for submerged floating tunnels. *Procedia engineering* **166**:247–254, 2016. https://doi.org/10.1016/j.proeng.2016.11.547.

[4] L. Swaalf. Reliability analysis of submerged floating tunnels 2020.

[5] J. Melcher, Z. Kala, M. Holický, et al. Design characteristics of structural steels based on statistical analysis of metallurgical products. *Journal of Constructional Steel Research* **60**(3-5):795–808, 2004. https://doi.org/10.1016/S0143-974X(03)00144-5.

[6] EN1992. Eurocode 2 part 1-1: Design of concrete structures: General rules and rules for buildings 2005.

[7] JCSS. Probabilistic model code. *Joint Committee on Structural Safety* 2001.

[8] EN1991-2. Eurocode 1 Part 2: Traffic loads on bridges 2015.

[9] TensaBV. Multi-strand post-tensioning system 2007.