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ABSTRACT

Design codes and standards rely on generalised target reliability indices. It is unclear, however, whether these indices are applicable to the specific risk-profile of marine structures. In this study, target reliability indices for quay walls were derived from various risk acceptance criteria, such as economic optimisation, individual risk (*IR*), societal risk (*SR*), the life quality index (*LQI*) and the social and environmental repercussion index (*SERI*). Important stochastic design variables in quay wall design, such as retaining height, soil strength and material properties, are largely time-independent, whereas other design variables are time-dependent. The extent to which a reliability problem is time variant affects the present value of future failure costs and the associated reliability optimum. A method was therefore developed to determine the influence of time-independent variables on the development of failure probability over time. This method can also be used to evaluate target reliability indices of other civil and geotechnical structures. The target reliability indices obtained for quay walls depend on failure consequences and marginal costs of safety investments. The results were used to elaborate the reliability framework of ISO 2394, and associated reliability levels are proposed for various consequence classes. The insights acquired were used to evaluate the acceptable probability of failure for different types of quay walls.

1. Introduction

There are thousands of kilometres of quay wall along inland waterways, in city centres, in commercial port areas and even in flood defence systems throughout the world. The reliability level of quay walls is generally determined in accordance with a certain design code or standard, such as the Eurocode standard EN 1990 [60]. Table 1.1 shows an example of reliability differentiation for buildings by employing a risk-based approach that directly relates the target probability of failure and the associated target reliability index to the consequences of failure. The consequences of failure can take many different forms, such as loss of human lives and social & environmental and economic repercussions [17]. It should be noted that target reliability indices were mainly developed for buildings [102,99] and bridges [85] assuming fully time-variant reliability problems [35,53]. However, the source of aleatory and epistemic uncertainty [50] as well as the consequences of failure could be very different for quay walls in port areas [55].

In the Netherlands, the design handbooks for quay walls [29] and sheet pile walls [42] further elaborated the recommendations of the Eurocode standard, because examples of soil-retaining walls are lacking (Table 1.2).

Table 1.2 suggests that reliability differentiation is influenced to a certain extent by the retaining height of a quay wall. Although the retaining height is an important design parameter, it is not necessarily an assessment criterion for reliability. In port areas, 'danger to life' is fairly low [65] because few people are present and quay walls are ideally designed in such a way that adequate warning is mostly given by visible signs, such as large deformations [25,29]. In reality, however, the factors influencing reliability differ per failure mode [1,43]. Fig. 1 gives an impression of the types of quay walls built in the Port of Rotterdam.

The primary aim of this research was to provide code developers with material to establish target reliability indices for quay walls and similar structures in a substantiated manner. In addition, the secondary aim was that quay walls can be categorized into existing reliability classes by authorities, clients and/or practising engineers. The first part of the research was devoted to examining the reliability optimum by economic optimisation on the basis of cost minimisation. In quay wall

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Table 1.1
Consequence and reliability classes for civil engineering works in EN 1990 [60].

Consequence/Reliability Class	Description	Examples of buildings and civil engineering works	Reliability index	
			β_{t_1} [1]	$\beta_{t_{50}}$ [1]
CC3/RC3	**High** consequences for loss of human life *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where the consequences of failure are high (e.g. a concert hall)	5.2	4.2
CC2/RC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where the consequences of failure are medium (e.g. an office building)	4.7	3.8[2]
CC1/RC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings and green houses)	4.2	3.3

[1] The annual (β_{t_1}) and lifetime reliability ($\beta_{t_{50}}$) indices only represent the same reliability level if limit state functions are time-dependent.
[2] This value is equal to the mean value derived by calibrating building codes [99].

Table 1.2
Reliability classes for quay walls in accordance with Quay Walls handbook [29].

Consequence/Reliability Class	Description consequences of failure	Examples of quay walls	Reliability index $\beta_{t_{50}}$
CC3/RC3	Risk danger to life high Risk of economic damage high	Quay wall in flood defence/LNG plant or nuclear plant (hazardous goods)	4.2
CC2/RC2	Risk danger to life negligible Risk of economic damage high	Conventional quay wall for barges and seagoing vessels. Retaining height > 5 m	3.8
CC1/RC1	Risk danger to life negligible Risk of economic damage low	Simple sheet pile structure/quay wall for small barges. Retaining height < 5 m	3.3

design, the dominant stochastic design variables, such as retaining height, soil strength and material properties, that influence the risk profile and hence the willingness to invest in safety measures, are largely time-independent [81,107]. In this study a method was developed to determine capitalised risk and the associated reliability optimum. The second part of the research was focussed on assessing minimum requirements concerning human safety. A sensitivity analysis was performed in order to derive insight into the parameters that influence the reliability index, such as discount rates, time horizons, marginal costs of safety investments and degree of damage in terms of monetary units or number of fatalities. The results were used to elaborate the reliability framework of ISO 2394 [40,4] in order both to be consistent with most of the codes and standards currently used in quay wall design and to improve guidance on reliability differentiation.

2. Target reliability indices in literature

2.1. Principles of target reliability

Basic performance measures are frequently expressed as an allowable probability of failure on the basis of a limit state function [31]. International organisations, such as the International Organization for Standardization (ISO) and the Joint Committee on Structural Safety (JCSS), support reliability-based design and assessments of structures. ISO provided an international standard, ISO 2934 [40], in order to develop a more uniform and harmonised design approach regarding resistance, serviceability and durability. ISO 2394 formed the foundation for many design codes and standards, such as all guidelines complying with the Eurocodes [10,11,25,29,30,63,76] and technical standards and commentaries for port and harbour facilities in Japan [65]. Modern design codes define the probability of failure $P_f = P(Z \leq 0)$ by a limit state function [43]. The target reliability index and target probability of failure are then related as follows:

$$\beta_t = \Phi^{-1}(P_{f;t}) \tag{1}$$

in which:

β_t – Target reliability index [–]
$P_{f;t}$ – Target probability of failure[–]
Φ^{-1} – Inverse of the standard normal cumulative distribution function [–]

Target reliability indices are always related to a reference period of, for example, one year or fifty years, as presented in Table 1.1. Eq. (2) is

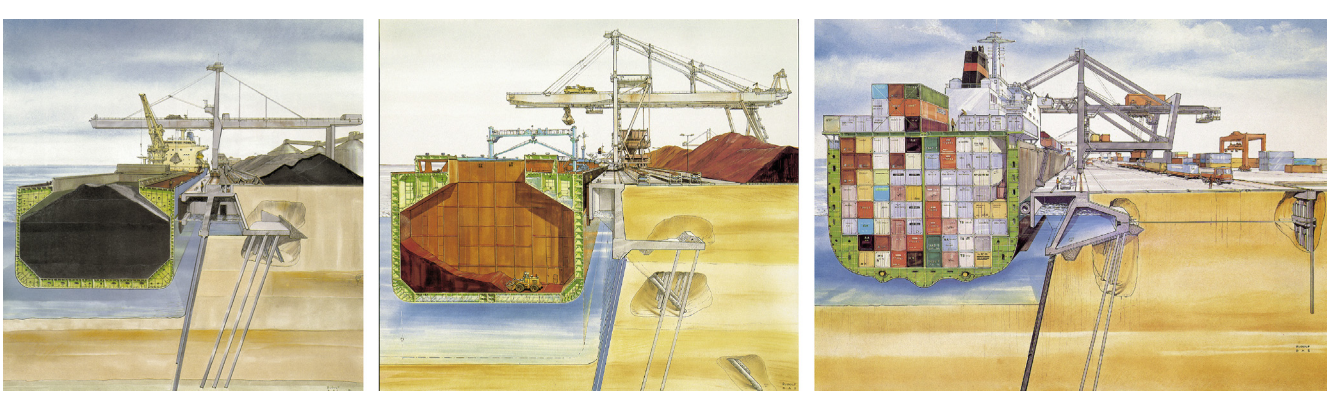

Fig. 1. Typical quay walls equipped with a relieving platform in the Port of Rotterdam [29]. Used by permission of the Port of Rotterdam Authority.

often used to transform annual into lifetime probabilities of failure [53]. However, this equation is valid only if reliability problems are largely time-variant [93], and hence should be used carefully [102] in the case of dominant time-independent stochastic design variables of quay walls.

$$P_{f;t_{ref}} = 1-\left(1-P_{f;t_1}\right)^{n_{ref}} \approx P_{f;t_1} n_{ref} \tag{2}$$

in which:

$P_{f;t_{ref}}$ – Probability of failure in the interval (0, t_{ref}) [–]
$P_{f;t_1}$ – Probability of failure over the interval (0, t_1) [–]
n_{ref} – Number of year in the reference period t_{ref} [-]
t_1 – Reference period of one year [year]

2.2. Reliability differentiation in literature

In practice, reliability indices are often derived by calibrating against previous design methods in order to maintain an existing reliability level [7,8]. However, target reliability indices can also be derived on the basis of economic optimisation by minimising costs. The associated reliability optimum is largely influenced by marginal costs of safety measures, distribution type and coefficient of variation of stochastic design variables [53,68].

In civil engineering, the required reliability level is generally defined in terms of certain safety classes, such as occupancy, reliability or consequence classes. An overview of safety classes and the accompanying annual and lifetime target reliabilities in literature is presented in Tables 1.3 and 1.4. It should be noted that recommendations for the assessment of existing structures, such as ISO 13,822 [41] and NEN 8700 [100] are not included.

The recommendations for reliability differentiation in literature initially seemed inconsistent and quite different [7,92]. However, when all the assessment criteria and associated target indices were subsequently ordered in accordance with the framework of ISO 2394 [40], reliability differentiation in literature appeared to be quite consistent and uniform. The classes A, B, C, D and E corresponding to ISO 2394 and the associated assessment criteria are further discussed in Section 6.2.

The Det Norske Veritas (DNV) [18] differentiates the required reliability level of marine structures in terms of structural redundancy and warning signals. The American Society of Civil Engineers distinguishes four occupancy categories in ASCE 7–10 [6] representing the number of people at risk by failure. The acceptable safety and the associated target reliability index are further differentiated for situations when failure is sudden or not sudden and does or does not lead to widespread progression of damage. When many people are at risk, safety requirements, often expressed as annual failure rates, will determine the acceptable reliability level [100,86]. Detailed overviews of available methods for quantitative risk measures of loss of life and accompanying thresholds are given by Jonkman et al. [48] and Bhattacharya et al. [7]. The minimum annual reliability indices for ultimate limit states derived by Fischer et al. [21] – namely 3.1, 3.7 and 4.2 for high, medium and low relative life-saving costs, respectively – are implemented in ISO 2394.

3. Method for deriving target reliability indices for quay walls

3.1. Introduction

This section briefly highlights the information required and methods used to establish reliability indices.

Fig. 2 shows that reliability indices are influenced by the efficiency of safety investments (Section 3.4) and the consequences of failure (Section 3.5). The optimal reliability index β^* can be obtained by minimising the sum of investments in safety measures and the accompanying capitalised risk (Section 3.6). It is important to understand both the quay wall system (Section 3.2) and the influence of time-dependent uncertainty (Section 3.2). The target reliability indices derived on the basis of economic optimisation might not be acceptable with regard to requirements concerning human safety [40]. These reliability indices are denoted as β_{acc}. The safety criteria are further explained in Section 4.

3.2. System decomposition and relevant failure modes

During the design of a quay wall several failure modes have to be evaluated. Numerous design guidelines implemented comprehensive fault trees including relevant failure modes [29,42], for example yielding of the retaining wall, failure of the anchor strut and geotechnical failure modes (Fig. 3). It should be noted that not all failure modes have been considered in this study. In literature it is often not very clear whether target reliability indices of failure modes are assigned to the structure as a whole or to structural components [54,96]. In this study, the reliability indices were ascribed to failure modes of structural components in accordance with modern design codes [5,43,53,60], assuming that progressive damage is mitigated [25,29,42]. Quay walls are generally designed in such a way that brittle failure is prevented and adequate warning is given by large deformations [25,29]. Consequently, the reliability level of a structural component is generally dominated by one specific failure mode. The

Table 1.3
Overview of annual target reliability indices in literature for the ultimate limit state (ULS).

Codes & Standards	Application	Consequence classes				
		A Low	B Some	C Considerable	D High	E Very high
ISO 2394 (2015)[1] [40]	All	Class 1	Class 2 4.2	Class 3 4.4	Class 4 4.7	Class 5
JCSS (2001)[1] [43]	All		Minor 4.2	Moderate 4.4	Large 4.7	
Structural concrete (2012)[1] [88]	Concrete	Small 3.5	Some 4.1		Moderate 4.7	Great 5.1
EN 1990 (2002) [60]	All		RC1 4.2		RC2 4.7	RC3 5.2
Rackwitz (2000)[1] [68]	Bridges	Insignificant 3.7		Normal 4.3	Large 4.7	
DNV (1992) [18]	Marine	Type I 3.09	Type I & II 3.71	Type II & III 4.26	Type III 4.75	
USACE (1997) [106]	Geotechnical	Average 2.5/3.0	Good 4.0			High 5.0

[1] Reliability indices are derived by assuming low relative costs of safety measures.

Table 1.4
Overview of lifetime target reliability indices in literature for the ultimate limit state (ULS).

Codes & Standards	Application	Consequence classes				
		A Low	B Some	C Considerable	D High	E Very high
ISO 2394 (1998)[1] [39]	All	Small 2.3	Some 3.1		Moderate 3.8	Great 4.3
ISO 23822 (2010)[1] [41]	All	Small 2.3	Some 3.1		Moderate 3.8	Great 4.3
EN 1990 (2002) [60]	All		RC1 3.3		RC2 3.8	RC3 4.3
SANS 10160 (2010) [80]	All	RC1 2.5	RC2 3.0	RC3 3.5		RC4 4.0
NEN 6700 (2005) [61]	All		Class 1 3.2	Class 2 3.4	Class 3 3.6	
ASCE (2010)[2] [6]	All	I^a 2.5	II^a, III^a & I^b 3.0/3.25/3.0	IV^a, II^b & 1^c 3.5/3.5/3.5	III^b 3.75	IV^b, II^c, III^c & IV^c 4.0/4.0/4.25/4.5
NBCC (2010) [20]	Buildings		Low 3.1	Typical 3.5	High 3.7	
CDHBDC (2014) [20]	Bridges		Low 3.1	Typical 3.5	High 3.7	
STOWA (2011) [87]	Hydraulic	QC I 2.3	QC II, QC III 2.7/3.1	QC IV 3.4	QC V 3.7	
TAW (2003) [94]	Hydraulic				River dike 3.8	Sea dike 4.3
ROM 0.5–05 (2008) [78]	Geotechnical	Minor 2.33	Low 3.09		High/very high 3.72	
CUR 166 (2012) [42]	Sheet piles	Class I 2.5		Class II 3.4		Class II 4.2
OCDI (2009) [65]	Marine	NR[3] 2.19/2.67	IR[3] 2.67		HR[3] 3.65	
CUR 211 (2003) [28]	Quay walls		Class 1 3.2	Class 2 3.4	Class 3 3.6	
CUR 211 (2013) [29]	Quay walls		RC1 3.3		RC2 3.8	RC3 4.3

[1] Reliability indices are derived by assuming low relative costs of safety measures.
[2] Not sudden, not widespread (a), sudden or widespread (b), sudden and widespread (c).
[3] Normal, intermediate and high seismic performance verification [56].

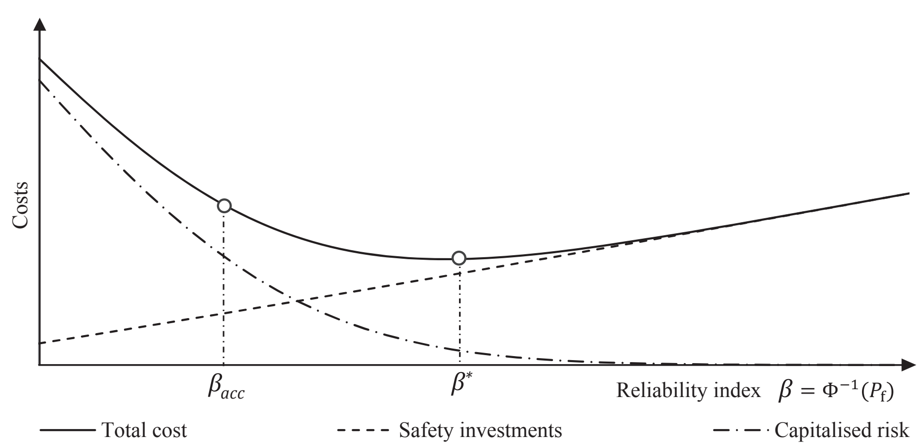

Fig. 2. Principles of cost minimisation, reliability optimum β^* and reliability minimum β_{acc}.

following two simplified ultimate limit states were considered as a reasonable first approach (Fig. 3):

$$Z_{STR}(z) = f_y - max\left(\frac{M_{wall}(z)}{W_{wall}} + \frac{N_{tube}(z)}{A_{tube}}\right) \tag{3}$$

$$Z_{GEO} = 1 - \Sigma Msf = 1 - \frac{c' + \sigma_n \tan(\varphi')}{c'_{reduced} + \sigma_n \tan(\varphi'_{reduced})} \tag{4}$$

in which:

Z_{STR} – Limit state representing structural failure [N/mm^2]
z – Depth [m]
f_y – Yield strength of retaining wall [N/mm^2]
M_{wall} – Bending moment in retaining wall [Nmm]
N_{tube} – Normal force in pile [N]
W_{wall} – Section modules of retaining wall [mm^3]
A_{tube} – Section area of pile [mm^2]
Z_{GEO} – Limit state representing structural geotechnical failure [–]
ΣMsf Global stability ratio related to φ-c reduction. The friction angle φ' and cohesion c' are successively decreased until geotechnical failure occurs [–]

The ultimate limit state for structural failure represents the stresses in the outer fibre of the soil-retaining wall and largely influences safety

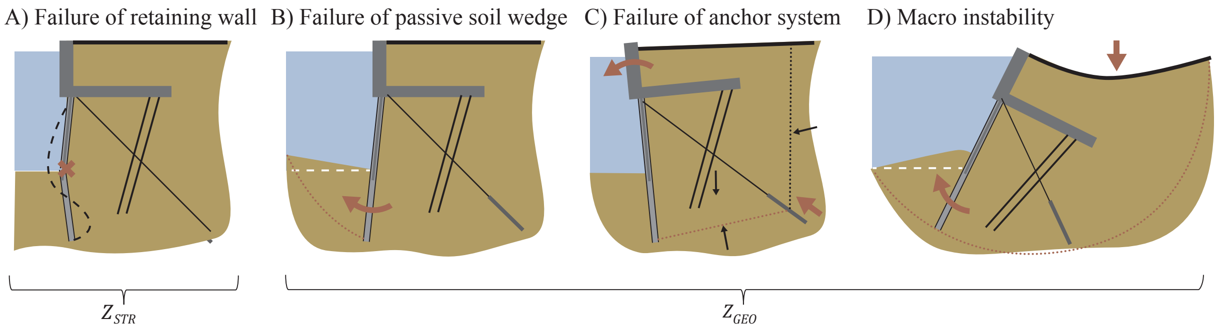

Fig. 3. Impression of some of the structural (Z_{STR}) and geotechnical failure modes (Z_{GEO}).

investments, whereas the global stability ratio takes account of the mutual dependency of all geotechnical failure modes simultaneously. Both limit states were evaluated by coupling the probabilistic package, OpenTURNS [3], to the finite element hardening soil model of the firm Plaxis, in order to model the soil-structure interaction as realistically as possible. The correlation between soil properties was taken into consideration in order to preclude unrealistically high reliability indices. Typical coefficients of correlation between $E_{50;ref}$-φ'_{rep}, γ_{sat}-φ'_{rep} and $E_{50;ref}$-γ_{sat} are 0.25, 0.5 and 0.5, respectively [97,107]. The distribution types and coefficients of variation used are listed in Appendix B.

In this study, 2D-Plaxis calculations were performed to gain insight into the extent to which a reliability problem is time-variant (Section 3.3) and into the efficiency of safety measures (Section 3.4), but they represent only a certain distance along a quay wall due to spatial uncertainty concerning resistance and local loads [13,32]. It is worth noting that it is theoretically impossible for a single metre of quay wall to fail. The length of a quay wall was therefore subdivided into equivalent sections for which failure events are assumed to be largely independent. In this study the 'equivalent length' L_{eq} was assumed to be 40 m [2]. This length is representative for the variability of the soil along a quay wall, but also corresponds to the section length of a quay wall that is on the one hand based on construction aspects and on the other hand provides sufficient flexural rigidity to redistribute local operational loads. Independent failure events are also observed in practice. An inventory of failure modes in Rotterdam, Spain and the United Kingdom [1,2] showed that the failure length of the limit sates under consideration was approximately 25–50 m. Consequently, the associated proportional change in marginal safety costs (Section 3.4) and failure consequences (Section 3.5) was taken into account for L_{eq} along a quay wall.

3.3. Modelling time-variant reliability

3.3.1. Introduction

The risk profile of a quay wall evolves over time and influences the capitalised risk, and hence the reliability optimum of a quay wall. This section discusses the method used to model the marginal increase in the probability of failure over time in order to determine the present value of future potential failure costs. The annual failure rate will generally decrease during the first period of the service life if no failure has occurred in previous years (Fig. 4). Close to the end of the service life, failure due to deterioration is more likely and results in an increase in the annual failure rate. Fig. 4-A represents a limit state dominated by time-independent epistemic uncertainty [57] in stochastic design variables, for example a 'dam'. Many dam failures occur at the first filling of the reservoir because of unforeseen soil conditions. In contrast to a dam, the annual failure rate of buildings and bridges (Fig. 4-C) is often assumed to be constant, because uncertainty is dominated by time-dependent stochastic design variables and deterioration [93]. In quay wall design, uncertainty is largely time-independent [81,107]. However, quay walls may show some degradation and are subjected to random loads, such as operational or ship loads and water head differences [90]. The reliability of quay walls is influenced by both time-independent variables (mainly soil properties) and random loads and will typically be in between Fig. 4-A and -C.

3.3.2. Development probability of failure during the lifetime

The usual approach to time-variant reliability problems is based on the computation of the outcrossing rate of the limit state [69,89,90]. However, here the probability of failure $P_{f;t_n}$ in time interval $(t, t + \Delta t)$ was modelled assuming two blocks, with one block being largely time-independent $P_{f;0}$ and the other being fully time-dependent $\sum \Delta P_{f;t_n}$ (Fig. 5).

$$P_{f;t_n} = P_{f;0} + \sum P_{f;t_n} \tag{5}$$

$$P_{f;t_{ref}} = P_{f;0} + \sum_{n=1}^{n_{ref}} P_{f;t_n} \tag{6}$$

in which:

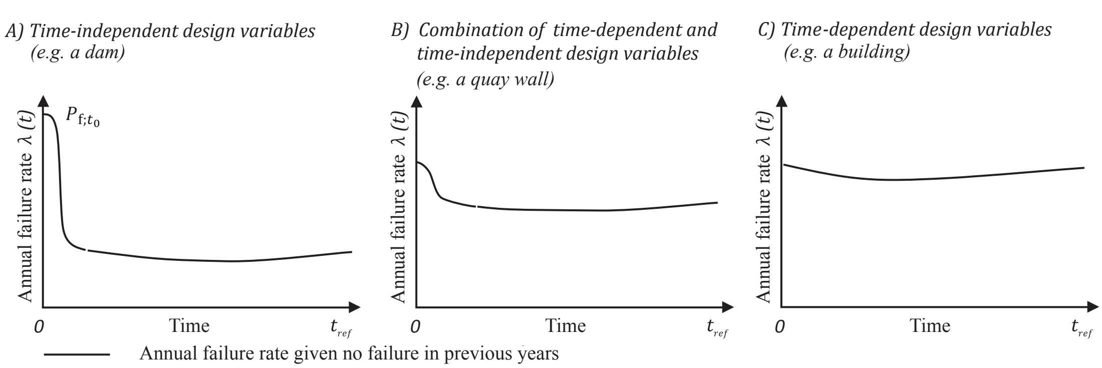

Fig. 4. Conceptual bathtub curves for time-independent (A), a combination of time-independent and time-dependent (B), and time-dependent (C) uncertainty in design variables.

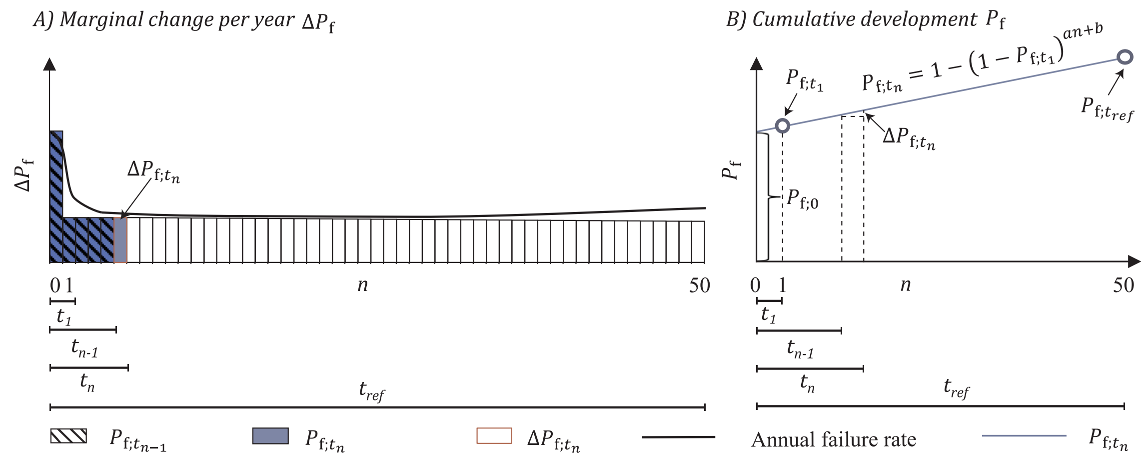

Fig. 5. Development of cumulative probability of failure (A) and the associated marginal increase per year (B) for a largely time-dependent limit state function.

$P_{f;t_n}$ – Probability of failure in time interval [0, n) [–]
$P_{f;0}$ – Time-independent probability of failure [–]
$\Delta P_{f;t_n}$ – Marginal change in probability of failure in time interval $(n-1, n)$ [–]
$P_{f;t_{ref}}$ – Probability of failure in the interval [0, t_{ref}) [–]
n_{ref} – Number of years during the reference period [year]
t_{ref} – Reference period [year]
n – Individual year of the reference period [–]
t_n – Period of n years in the reference period [year]

In this study, it was assumed that risks related to human errors – such as design and construction errors – are taken into account by means of, for example, quality control procedures and inspection [58,40,98]. Deterioration was not taken into consideration, because new quay walls are equipped with a system of cathodic protection that prevents degradation [29]. Although soil conditions could be influenced by time – such as variability in soil pressure, liquefaction, settlements and compaction [20] – the time effect on soil strength was assumed to be negligible. The time-dependent part of the probability of failure was taken into consideration by modelling variable loads, such as water head differences and live loads, in accordance with extreme value theory.

3.3.3. Derivation of equivalent time period t_{eq}

Largely time-dependent limit state functions indicate that failure events are to some extent correlated. Sýkora et al. [93] suggest using a 'basic' period in order to account for dependency of failure events, which in this study is denoted as t_{eq}; in other words, the 'equivalent' period for which failure events are assumed to be independent in subsequent years. The cumulative lifetime probability of failure was determined by transforming Eq. (2) into the following equations, which formed the basis for the method used (see also Appendix A):

$$P_{f;t_{ref}} = 1-\left(1-P_{f;t_1}\right)^{n_{eq}} \tag{7}$$

$$\beta_{t_{ref}} = \Phi^{-1}[\Phi\left(\beta_{t_1}\right)^{n_{eq}}] \tag{8}$$

$$n_{eq} = \frac{t_{ref}}{t_{eq}} \tag{9}$$

in which:

$P_{f;t_{ref}}$ – Probability of failure in the interval [0, t_{ref}) [–]
$P_{f;t_1}$ – Probability of failure in the interval [0, t_1] [–]
n_{eq} – Number of equivalent periods during the reference period [–]
$\beta_{t_{ref}}$ – Reliability index of reference period t_{ref} [–]
β_{t_1} – Reliability index of a one-year reference period [–]
t_1 Reference period of one year [year]
t_{eq} Equivalent period for which failure events are independent in subsequent years [year]

The equivalent period t_{eq} was determined using extreme value theory. Although other reference periods could have been considered, it appeared to be fairly practical to perform two probabilistic assessments using t_1 and t_{50}, representing the annual and lifetime probability of failure, respectively. The output of the probabilistic assessment was hence twofold: a reliability index for a reference period of one year $\beta_{t_1} = \Phi^{-1}(P_{f;t_1})$ and of fifty years $\beta_{t_{ref}} = \beta_{t_{50}} = \Phi^{-1}(P_{f;t_{50}})$. The results of the probabilistic analysis were used to determine the equivalent period t_{eq} by transforming Eq. (8) into Eq. (10). Fig. 6 shows the application of equivalent period t_{eq} in a time-variant reliability problem. When dominant stochastic design variables of a limit state are time-independent $n_{eq} = 1$, but if dominant stochastic design variables are time-dependent $n_{eq} = n_{ref}$.

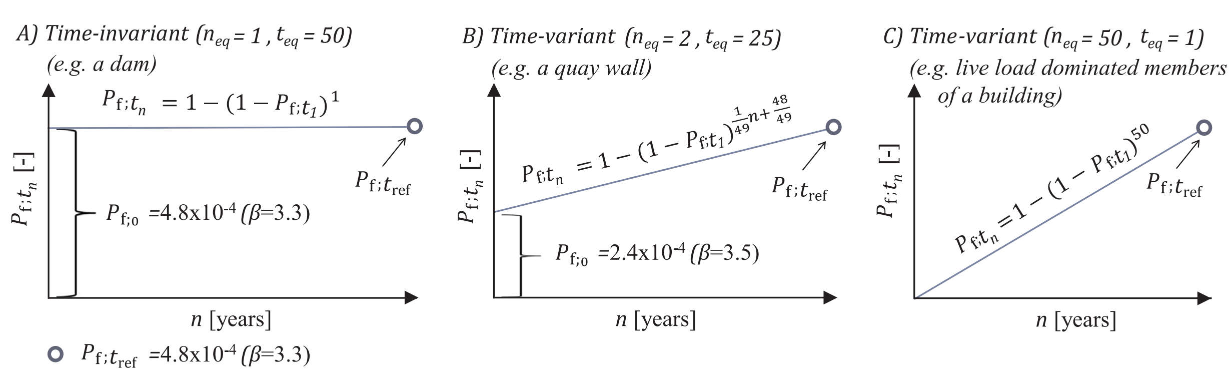

Fig. 6. Principle differences between development of failure probability for time-invariant (A), time-variant (B) and completely time variant (C) reliability problems.

$$t_{eq} = \frac{t_{ref}}{\log_{\Phi(\beta_{t_1})}[\Phi(\beta_{t_{ref}})]} = t_{ref}\log_{\Phi(\beta_{t_{ref}})}[\Phi(\beta_{t_1})] \tag{10}$$

3.4. Marginal construction costs

The uncertainty in design variables influences not only the extent to which a reliability problem is time-variant, but also the efficiency of safety investments [68,84]. As explained in Section 3.2, the length of a quay wall was subdivided into equivalent sections for which failure events are independent. The associated proportional change in marginal safety investments (Fig. 2) was found by the following equation:

$$C_m(x) = L_{eq}\frac{\Delta C(x)}{\Delta\beta(x)} \tag{11}$$

in which:

- C_m – Marginal costs of safety measures [€]
- x – A vector representing changes in structural dimensions [..]
- L_{eq} – Equivalent length along a quay wall for which failure events are independent [m]
- ΔC – Change in construction costs [€/m]
- $\Delta\beta$ – Change in reliability index [–]

The costs $\Delta C(x)$ associated with a change in structural dimensions were derived in consultation with senior costs experts of the Port of Rotterdam Authority, and the associated change in reliability index $\Delta\beta$ was derived by performing four probabilistic assessments, two for each limit state. The changes in structural dimensions of the retaining wall, such as the section modules W_{wall} (D_{tube}, t_{tube}) and the sectional area A_{tube} (D_{tube}, t_{tube}), were applied to the structural limit state function (Z_{STR}), and changes in length of the retaining wall L_{wall} and the grout body of the anchors L_{anchor} were applied to the geotechnical limit state function (Z_{GEO}). The fraction $\Delta C/\Delta\beta$ found was 5–10%, which is in accordance with the study by Schweckendiek et al. [81]. The marginal safety investments to prevent structural failure were assumed to be higher compared to geotechnically induced failure (Table 1.5).

3.5. Consequences of failure

As indicated, the consequences of failure can take various forms, and hence can be measured in monetary units C_f or number of fatalities $N_{F|f}$ [14]. Some information about failure costs C_f was found in the background documents of port authorities and terminals [55,12], as well as in some design guidelines [15,87]. The little available information was extended by administering a questionnaire that asked experts to give both a qualitative and a quantitative estimate of the consequences of failure on the basis of the recommendations of ISO 2394 [40] and JCSS [43].

Terminal and business managers largely agree that significant economic repercussions are not very likely in large ports, because it is often possible to mitigate damage within the overcapacity of a terminal or port cluster (Fig. 7A and C). Substantial economic damage is more likely for terminals without redundancy (Fig. 7B and D). The business managers also stated that it is important to prevent permanent damage to the image and reputation of a port. In reality, if a terminal has had some functional redundancy, the failure costs were estimated to be fairly close to the direct failure costs. The experts largely agreed that the failure costs associated with the equivalent length along a commercial quay wall are in the range of €1–5 m and €1–15 m for structural failure (Z_{STR}) and geotechnical failure (Z_{GEO}), respectively. The influence of the failure costs on the optimal reliability index was taken into consideration in the sensitivity analysis presented in Section 5.2.

In this study, the expected number of fatalities was determined in accordance with Eq. (12). Little information is as yet available about the number of people at risk due to their nearness to quay walls, and hence a fairly conservative estimate was made assuming $N_{PAR} = 5$ along 40 m of quay wall. The successful escape of people largely depends on type of failure, escape path, perception of danger and recognition of provided warning signals [52]. The probability of a successful escape influences the conditional probability that an individual will die given failure. In Table 1.6 indicative estimates of $N_{F|f}$ are presented for the two failure modes under consideration.

$$N_{F|f} = N_{PAR}(1 - P_{Escape})P_{d|f} \tag{12}$$

in which:

- $N_{F|f}$ – Expected number of fatalities given failure [–]
- N_{PAR} – Number of people at risk [–]
- P_{Escape} – Probability of a successful escape [–]
- $P_{d|f}$ – Conditional probability a random human being present will die given failure [–]

The monetary value of a human life can be determined on the basis of societal willingness to pay (*SWTP*) [40]. However, assigning a monetary value to human life, on whatever basis, is a very controversial issue [105]. According to Rackwitz [74], a monetary value of life does not exist: '…the value of human life is infinite and beyond measure …'. In this study, a monetary value of €3m, which is in line with the \$2m–4 m presented in ISO 2394 [40], was used only in the evaluation of the marginal life-saving cost principle (Section 5.3).

3.6. Risk-based optimisation of structural components

This section concerns the method used to determine target reliability indices using the principles of cost minimisation in accordance with the recommendations in literature [68,91,93]. The following objective function was considered:

$$f(\beta) = B - C_{Investments}(\beta) - C_{Maintenance} - C_{Obsolescence}(\beta) - C_{\mathrm{CapitalisedRisk}}(\beta) \tag{13}$$

$$\max\{f(\beta)\} \rightarrow \frac{\partial f(\beta^*)}{\partial\beta} = 0 \tag{14}$$

in which:

- f – Objective function [–]
- B – Benefits related to the investments [€]
- $C_{Investments}$ – Investments in safety measures [€]
- $C_{Maintenace}$ – Cost of maintenance, repairs and inspections [€]
- $C_{Obsolescense}$ – Cost related to a structure becoming obsolete after some time because it is not able to fulfil its originally intended purpose [€]
- $C_{\mathrm{CapitalisedRisk}}$ – Present value of future failure costs [€]
- β – Decision parameter, reliability index [–]
- β^* – Optimal reliability index [–]

It should be noted that the benefits and maintenance costs were considered to be independent of decision parameter β. The maintenance costs related to structural deterioration were not taken into account,

Table 1.5
Initial construction costs C_0 being independent of β and marginal costs of safety measures C_m for a quay wall with $h_{\mathrm{retaining}} = 20$ m, $L_{\mathrm{eq}} = 40$ m and construction costs equal to €1 m for $\beta = 3.8$.

Failure modes	x	C_0	$C_m(x)$
All failure modes	All structural dimensions	€0.60 m	€0.10 m
Yielding of the combi-wall ($Z_{STR} < 0$)	W_{wall} (D_{tube}, t_{tube}); A_{tube} (D_{tube}, t_{tube})	€0.36 m	€0.06 m
Geotechnical failure ($Z_{GEO} < 0$)	L_{wall}; L_{anchor}	€0.12 m	€0.02 m

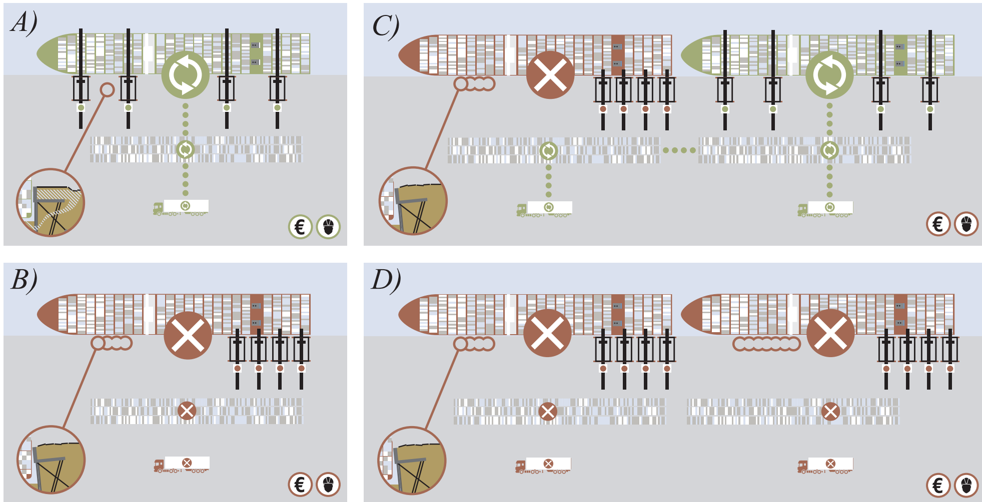

Fig. 7. Impression of failure consequences for commercial quay walls with (A) & (C) and without (B) & (D) functional redundancy.

Table 1.6
Expected number of fatalities for commercial quay walls.

Type of structural failure	N_{PAR} [1]	P_{Escape} [2]	$P_{d\mid\mathbf{f}}$ [3]	$N_{F\mid\mathbf{f}}$ [4]
Structural failure (Z_{STR})	5	0.70	0.10	0.15
Geotechnical failure (Z_{GEO})	5	0.30	0.20	0.70

[1] Conservative estimate, derived by counting the number of people who are near to a quay wall. Catastrophic accidents and situations with lots of people near to a quay wall were not taken into consideration.
[2] Conservative value derived by administering a questionnaire.
[3] Values are based on a best estimate, and therefore a sensitivity analysis is included in Section 4.4.
[4] Values lower than 1 are only used in the *LQI* criterion.

because corrosion is so aggressive that it is much more efficient to invest in a system of cathodic protection [29]. Costs of financing projects (e.g. interest rates) and costs related to obsolescence (lifetime buy vs design refresh) were not taken into account. Obsolescence costs are generally activated in the business case of a future design refresh. In this study, the failure costs were related to the design lifetime of the structure. If one assumes that the objective function is positive, the optimum reliability index β^* can be established by minimising the total costs and solving the associated derivative.

$$\min\{C_{Total}(\beta) = C_{Investments}(\beta) + C_{\text{CapitalisedRisk}}(\beta)\} \tag{15}$$

$$\frac{\partial C_{Total}(\beta^*)}{\partial \beta} = 0 \tag{16}$$

The investments in safety measures were divided into initial construction costs C_0 and marginal construction costs C_m (Section 3.4). The initial construction costs C_0 often dominate structural investments [26,27], but unlike C_m do not influence the reliability optimum [68].

$$C_{Investments}(\beta, x) = C_0 + C_m(x)\beta \tag{17}$$

in which:

- C_0 – Initial construction costs independent of the reliability index [€]
- C_m – Marginal construction cost dependent on the reliability index [€]
- x – Vector representing the changes in design parameters, e.g. structural dimensions [–]

It should be noted that even if adequate safety measures are implemented, there will always be a residual capitalised risk. In this study, the method of Holický [35] was extended by distinguishing $P_{\mathrm{f};0}$ and $\sum \Delta P_{\mathrm{f};t_n}$ representing the blocks of the probability of failure over a certain time interval being time-independent and time-dependent, respectively (Section 3.3):

$$C_{\text{CapitalisedRisk}}(\beta) = C_{\mathrm{f}} P_{\mathrm{f};0}(\beta) + C_{\mathrm{f}} \cdot \sum_{n=1}^{n_{ref}} \frac{\Delta P_{\mathrm{f};t_n}(\beta)}{(1+r)^n} \quad \text{for } n \in (1, n_{ref}) \tag{18}$$

The capitalised risk represents the present value of future costs and was established by assuming a real discount rate r (nominal rate of interest after correction for inflation) [91,73]. The minimum discount rate is equal to the time-averaged economic growth rate per capita [74]. Fischer et al. [23] showed that different discount rates could be used for private and social decision makers. The summation of direct and indirect economic consequences of failure was expressed by C_{f} (Section 5).

Eq. (20) presents an analytical formula of the objective function and was used to derive insight into the influencing factors of the reliability optimum. The reader is referred to Appendix A for the full derivation and explanation of the total costs function and associated derivative.

$$C_{Total}(\beta_{t_1}) = C_{Investments}(\beta_{t_1}) + C_{\text{CapitalisedRisk}}(\beta_{t_1}) \tag{19}$$

$$C_{Total}(\beta_{t_1}) = C_0 + C_{m_{t_1}} + C_{\mathrm{f}}(1-\Phi_1^b) + C_{\mathrm{f}}\, c\,(\Phi_1^b - \Phi_1)\frac{1-(c\Phi_1^a)^{n_{ref}}}{1-c\Phi_1^a} \tag{20}$$

$$c = 1/(1+r) \tag{21}$$

in which:

- $\Phi_1 = \Phi(\beta_{t_1}) = F(\beta_{t_1})$ – Cumulative distribution function $F(\beta)$ of normal distribution [–]

4. Risk-acceptance criteria

The optimal reliability indices derived on the basis of cost minimisation have to be higher than the thresholds of acceptance. This section presents the evaluation of four risk-acceptance criteria, namely the individual risk (*IR*) criterion, the societal risk (*SR*) criterion, the life quality index (*LQI*) acceptance criterion, and the social and environmental repercussion index (*SERI*).

4.1. Individual risk criterion

The individual risk (*IR*) is often defined as the individual risk per annum (*IRPA*) or the localised individual risk per annum (*LIRA*) [44,66]. *IRPA* is generally used to assess work-related risks faced by particularly exposed individuals [64,83] and is frequently used in decision-making processes, whereas *LIRA* represents the individual risk at a specific geographical location [44]. *LIRA* is mainly used in spatial planning and assessing external safety contours in the vicinity of hazardous installations or in the design of flood defence systems [46,48,103,104]. It should be noted that *LIRA* does not change even if no people are present, and hence the main difference between *IRPA* and *LIRA* is the probability that an individual is present:

$$IRPA = P_{f_{t_1}} P_{Present}(1-P_{Escape}) P_{d|f} \tag{22}$$

$$LIRA = P_{f_{t_1}}(1-P_{Escape}) P_{d|f} \tag{23}$$

in which:

IRPA – Annual probability that a specific individual or hypothetical group member will die due to exposure to hazardous events [75] [–]
LIRA – Annual probability that an unprotected, permanently present individual will die due to an accident at a hazardous site [45] [–]
$P_{Present}$ – Probability that a specific individual will be present [–]
P_{Escape} – Probability of a successful escape [–]
$P_{d|f}$ – Conditional probability that an individual being present will die given failure [–]

The probability that a hypothetical crane driver is present was based on the following assumptions: cranes are used for 60% of the time; the domain of a crane along a quay was assumed to correspond to 3 times L_{eq}; a crane driver generally works on multiple types of cranes, 8 h a day, 220 days a year. If a crane driver works on three different cranes during a year, the probability that an individual crane driver is present at L_{eq} along a quay wall is approximately 1.5% of the time (0.6/3/3 * (220/365)/3 = 1.34%).

According to various recommendations in literature, the risk level (*IRPA)* related to involuntary work activities corresponds to an annual risk level of 10^{-6} and is generally considered to be 'broadly acceptable' [24,33,34,39]. Individual risk levels higher than 10^{-4} corresponding to the annual probability of dying as a result of a traffic accident are defined as 'intolerable' in well-developed countries [85,95]. An annual fatality rate of 10^{-5} representing *LIRA* is generally defined as 'tolerable' and was incorporated into the Dutch design code for flood defence systems [9,47,96]. The acceptable reliability index in accordance with *IRPA* and *LIRA* was derived using:

$$\beta_{acc;t_1} \geqslant -\Phi^{-1}(P_{f_{acc;t_1}}) = -\Phi^{-1}\left(\frac{IRPA}{P_{Present}(1-P_{Escape})P_{d|f}}\right) \tag{24}$$

$$\beta_{acc;t_1} \geqslant -\Phi^{-1}(P_{f_{acc;t_1}}) = -\Phi^{-1}\left(\frac{LIRA}{(1-P_{Escape})P_{d|f}}\right) \tag{25}$$

where

$\beta_{acc;t_1}$ = Annual threshold of acceptance [–]
$P_{f_{acc;t_1}}$ = Acceptable annual probability of failure [–]

4.2. Societal risk criterion

Although the number of people present near commercial quay walls is usually limited, the societal risk criterion was also evaluated [98] using the *F–N* curves. The influence of the expected number of fatalities given failure was examined on the basis of the upper bound (A = 0.01 and k = 2) and lower bound (A = 0.1 and k = 1) of the *F–N* curves in Section 5.4.

$$P_{f_{acc;t_1}} = \Phi(-\beta_{acc;t_1}) \leqslant AN_{F|f}^{-k} \tag{26}$$

$$\beta_{acc;t_1} \geqslant -\Phi^{-1}(P_{f_{acc;t_1}}) = -\Phi^{-1}(AN_{F|f}^{-k}) \tag{27}$$

where

$N_{F|f}$ = Expected number of fatalities [–]
A = Acceptable risk for one fatality [–]
k = Slope factor of the *F–N* curve [–]

4.3. Life quality index criterion

ISO 2394 [40] recommends employing the *LQI* acceptance criterion and provides information with regard to the social willingness to pay (*SWTP*), which corresponds to the amount of money that should be invested in saving one additional life [73,74]. In a similar way the willingness to prevent an injury could be taken into consideration. Studying the background documents of the *LQI* criterion [21,22] revealed that this criterion can be evaluated by applying the principles of cost minimisation if the capitalised 'societal' risk is taken into consideration. The corresponding present value of societal losses, denoted by $C_{\text{f;Societal}}$, then depends on the *SWTP* and the expected number of fatalities $N_{F|f}$. The associated annual threshold of acceptance $\beta_{acc;t_1}$ was found by solving the derivative of the societal costs function:

$$\min_{f(\beta)>0} \{C_{Societal}(\beta) = C_{Investments}(\beta) + C_{\text{CapitalisedRisk}}(\beta)\} \tag{28}$$

$$C_{\text{CapitalisedRisk}}(\beta) = C_{\text{f;Societal}} P_{\text{f;0}}(\beta) + C_{\text{f;Societal}} \sum_{n=1}^{t_{ref}} \frac{P_{\text{f};t_n}(\beta)}{(1+\gamma_s)^n} \tag{29}$$

$$C_{\text{f;Societal}} = N_{F|f}\, SWTP \tag{30}$$

$$\frac{\partial C_{Societal}(\beta_{acc;t_1})}{\partial \beta} \geqslant 0 \tag{31}$$

where

$C_{Societal}$ = Total societal costs [€]
$C_{f;Societal}$ = Societal failure cost [€]

4.4. SERI criterion

The social and environmental repercussion index (*SERI*) of the Spanish ROM represents the loss of human lives, damage to the environment and historical and cultural heritage, and the degree of social disruption. The social repercussion index was derived by examining Eq. (32) on the basis of the guidance in ROM 0.0 [76] and the accompanying lifetime target reliability index (Table 1.4) was established in accordance with ROM 0.5 [78].

$$SERI = \sum_{i=1}^{3} SERI_i \tag{32}$$

5. Results

5.1. Reliability optimum β^* on the basis of cost minimisation

This section presents the reliability indices obtained by economic optimisation of the structural and geotechnical limit states described in Section 3.2. The optimal annual and lifetime reliability indices for structural failure found were approximately 2.8 and 2.5 (Fig. 8A), whereas for geotechnical failure 3.5 and 3.3 (Fig. 8B) were found, respectively. The steepness of the left side of the total costs function was largely influenced by the absolute value of the capitalised risk and explains the different shapes of the graphs. The steepness of the right side was quite small due to the quite low absolute value of marginal safety investments C_m. The influencing parameters of the reliability optimum are further examined by performing a sensitivity analysis in the following section.

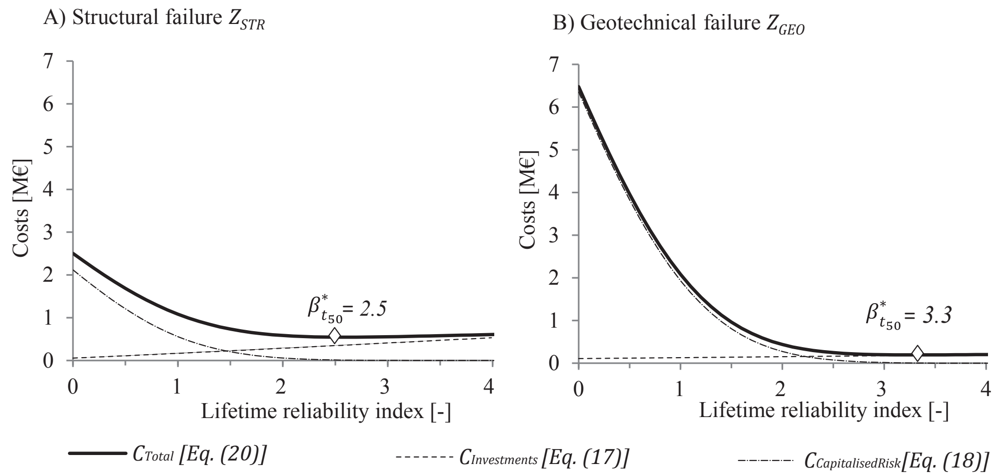

Fig. 8. A) Optimal lifetime reliability indices for structural failure $t_{eq} = 20$, $r = 0.03$, $t_{ref} = 50$, $L_{eq} = 40$, $C_0 = €0.36$ m, $C_m = €0.06$ m and $C_f = €5$ m; B) Optimal lifetime reliability indices for geotechnical failure $t_{eq} = 30$, $r = 0.03$, $t_{ref} = 50$, $L_{eq} = 40$, $C_0 = €0.12$ m, $C_m = €0.02$ m and $C_f = €15$ m.

5.2. Sensitivity analysis of reliability optimum β^*

The aim of the sensitivity analysis was to gain insight into the influence of the extent to which reliability problems are time-variant, expressed by t_{eq}. The effect of discount rates, the marginal costs of safety measures, failure costs and reference period were taken into consideration. Fig. 9 shows the optimal target reliability indices for a reference period of one year (left) and for the lifetime (right). It should be noted that the optimal annual and lifetime reliability indices for $t_{eq} = 50$ or t_{ref} (solid black lines) are identical, because the limit state function was assumed to be time-independent.

Time-dependent limit state functions show relatively high annual reliability indices, but the associated lifetime reliability indices are fairly low compared to largely time-independent limit state functions. In the case of a high risk profile, expressed in terms of high discount rates, there is less willingness to invest in initial safety measures, and hence a lower reliability optimum was found (Fig. 9A). As expected, the effect of discount rates is stronger for time-dependent limit state functions. The variance in optimal lifetime reliability indices caused by t_{eq} was much lower than the variance in annual reliability indices given changes in C_m and C_f. This was explained by analysing the effect of discounting future costs. However, the absolute value of both C_m and C_f significantly influence the reliability optimum (Fig. 9B and C). Low failure costs ($C_f \leq €10$ m) result in an exponential decrease in the reliability optimum. A longer reference period will generally result in less variability in the optimal annual reliability indices and seem to approach an asymptote. A longer reference period resulted in an enhancement of the cumulative probability of failure, and hence in a lower lifetime reliability optimum (Fig. 9D). An important finding is that if time-independent stochastic design variables dominate uncertainty, the difference between annual and lifetime target reliability indices becomes quite low.

5.3. Reliability minimum β_{acc} on the basis of human safety criteria

The minimum requirements concerning human safety were examined on the basis of the individual risk (*IR*) and the societal risk (*SR*) criterion, the life quality index (*LQI*) and the social and environmental repercussion index (*SERI*) criteria. Table 1.7 presents the results of all safety criteria. The reader is referred to Section 3.5 for further background information with regard to the input variables used.

Table 1.7 shows that the SR criterion is not relevant for failure modes of commercial quay walls, because the number of people at risk is fairly low. The reliability minimum β_{acc} derived using the *LQI* criterion led to lower reliability indices compared to the reliability optimum found by economic optimisation in Section 5.1. It was also found that the optimal reliability indices are quite similar to the results obtained by examining the *IRPA* criterion. However, *LIRA* within risk contours 10^{-5} and 10^{-6} resulted in higher reliability indices. The influence of the input variables on the reliability minimum β_{acc} is further discussed in the following section.

5.4. Sensitivity analysis of reliability minimum β_{acc}

Similar to the sensitivity analysis performed for economic optimisation, the differentiating factors related to the requirements concerning human safety were evaluated. Fig. 10 shows that the *IR* criterion was largely influenced by the product of the conditional probability that an individual will die given the failure of a quay wall and the probability of not being able to escape in time. When this product becomes fairly low (< 0.05), a significant decrease in the acceptable annual reliability index was found. Fig. 10A shows that the probability that a hypothetical person, such as a crane driver, is present influences the development of the *IRPA*. Fig. 11 shows that the *SR* criterion and the *LQI* criterion were largely influenced by the expected number of fatalities given the failure of a quay wall. It is worth noting that the upper bound of the *SR* criterion will become relevant when the expected number of fatalities is quite large. Similar to the insights derived by economic optimisation, the *LQI* criterion is influenced by the absolute value of marginal safety investments, social failure costs and the extent to which the reliability of failure modes are time-variant. The results of the sensitivity analysis are further discussed in Section 6.

6. Discussion

6.1. Target reliability indices for commercial quay walls

The results of this study showed that target reliability indices for commercial quay walls can be determined by economic optimisation on the basis of cost minimisation. The annual and lifetime target reliability indices ascribed to limit states of structural components and geotechnical failure modes of quay walls with a retaining height of 20 m are in the range of 2.8–3.5 and 2.5–3.3, respectively. The acceptable annual reliability index in accordance with the individual risk criterion (*IRPA* = 10^{-6}) led to fairly similar reliability indices. Table 1.8 gives an overview of the reliability indices for economic optimisation (β^*) and acceptable regarding human safety (β_{acc}). It should be noted that quay walls with a fairly small retaining height and fairly high variable loads could lead to higher differences between annual and lifetime target reliability indices ($t_{eq} < 20$).

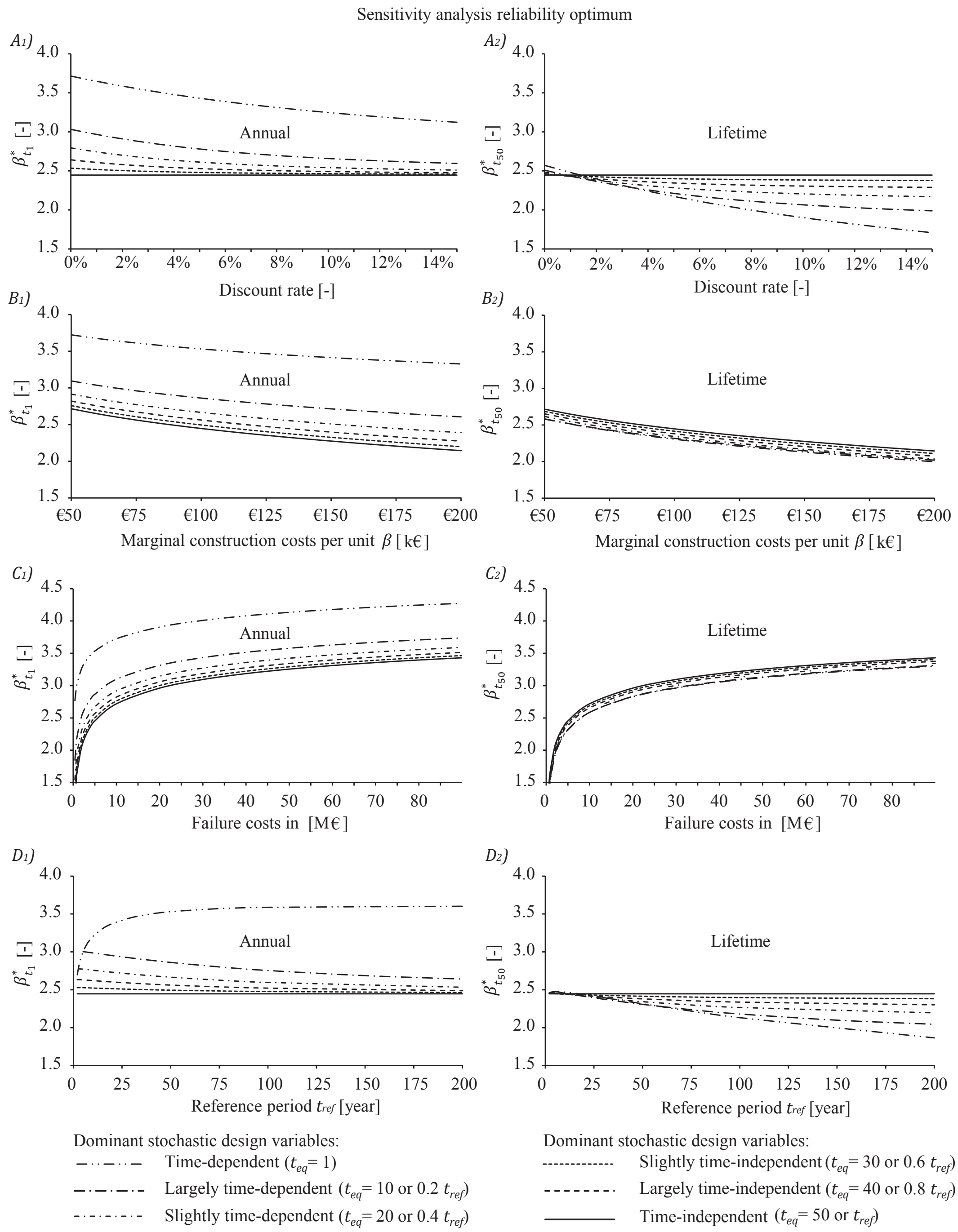

Fig. 9. Influence of discount rate (A), marginal safety investments (B), failure costs (C) and reference period (D) on the annual (left) and lifetime (right) reliability optimum for t_{ref} = 50, L_{eq} = 40, C_0 = €0.6 m, C_m = €0.1 m and C_f = €5 m.

Table 1.7
Reliability minimum β_{acc} in accordance with the IR criterion, SR criterion, LQI criterion and the SERI criteria.

Type of structural failure	Input				Annual reliability βt_1						Lifetime reliability βt_{50}	
	t_{eq}	$N_{F\|\mathbf{f}}$	*SWTP*	*ΣSERI*	$IRPA = 10^{-6}$	$LIRA = 10^{-6}$	$LIRA = 10^{-5}$	*SR*	LQI_{t1}	LQI_{t50} [2]	*SERI* [2]
Structural failure Z_{STR}	20	0.15	€3 m	3	2.8	4.0	3.4	< 2.3[1]	1.8	1.4	2.3
Geotechnical failure Z_{GEO}	30	0.70	€3 m	15	3.3	4.3	3.8	< 2.3[1]	2.8	2.7	3.0

[1] The expected value of the number of fatalities was assumed to be equal to 1.
[2] It should be noted that requirements concerning human safety are generally related to the annual and not to the lifetime reliability index.

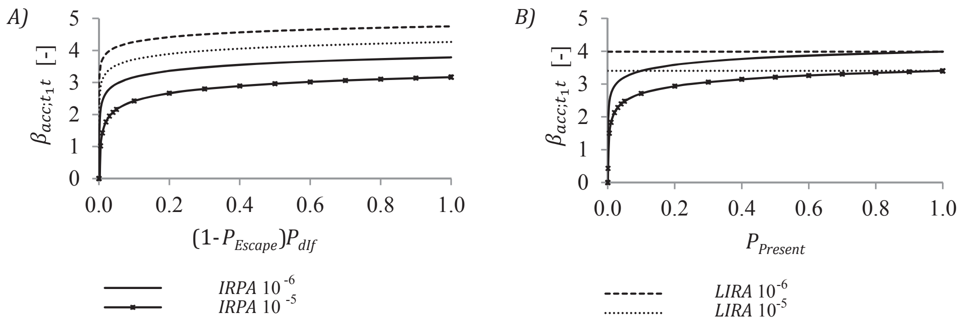

Fig. 10. Sensitivity analysis IR criterion: influence of conditional probability of failure (A); influence of a specific individual will be present (B).

It should be noted that the localised individual risk per annum (*LIRA*) criterion is assumed to be inactive, because the failure of a quay wall will generally not induce the failure of hazardous installations, such as chemical plants. However, if the *LIRA* criterion is active the acceptable annual reliability indices are in the range of 4.0–4.3. The societal risk (*SR*) criterion is mostly not so relevant for assessing human safety in relation to commercial quay walls, but should be taken into account if a large number of people are at risk, for example when quay walls are part of a cruise terminal or a flood defence system. It is always recommended to account for the *LQI* criterion in order to verify whether the marginal life-saving costs principle is sufficiently covered. The *SERI* criterion is fairly straightforward and seems to be quite efficient for selecting a consequence class in accordance with the reliability framework proposed in the following section.

Table 1.8
Overview risk-based optimal and acceptable reliability indices for commercial quay walls.

Risk-acceptance criteria	Type of criterion	Structural failure Z_{STR} ($t_{eq} \approx 20$)		Geotechnical failure Z_{GEO} ($t_{eq} \approx 30$)	
		$\beta_{1\text{-year}}$	$\beta_{50\text{-years}}$	$\beta_{1\text{-year}}$	$\beta_{50\text{-years}}$
Economic optimisation β^*	Cost minimisation	2.8	2.5	3.5	3.3
Human safety β_{acc}	Individual risk ($IRPA = 10^{-6}$)	2.8	–	3.3	–
	Societal risk (*SR*)	< 2.3	–	< 2.3	–
	Life quality index (*LQI*)	1.8	1.4	2.8	2.7
	Social and economic repercussion index (*SERI*)	–	2.3	–	3.0

6.2. Assessment criteria for classification

In Table 1.9 an assessment framework for reliability differentiation is proposed that complies with the qualitative descriptions embedded in many codes and standards in order to make reliability differentiation for quay walls more accessible and interpretable. The reliability framework of ISO 2394 [40] provided a solid foundation, and hence was further elaborated by implementing the recommendations of ASCE 7–10 [6] and DNV [18] for structural redundancy and progression of failure. The social and environmental repercussion index (*SERI*) [76] and the ratio between the direct costs of failure and construction costs [43] were also incorporated. In reality, quay wall failure can have a significant effect on accessibility as well as on the image and reputation of a port. The service values of the Port of Rotterdam Authority, which are in accordance with the values of other multinationals [55], were therefore embedded in the new assessment framework. An upper limit to the allowable degree of economic damage was defined for each consequence class using the results of the sensitivity analysis and assuming the equivalent length L_{eq} along a quay wall, for which failure events are independent, to be in the range of 25–50 m. It is worth noting that the row in Table 1.9 that shows the most onerous failure consequence determines the required consequence class.

6.3. Compliance with codes and standards and proposal for classification

In engineering, reliability problems are often assumed to be fully time-variant (Section 2.2). The results of this study, however, showed that limit state functions of quay walls are to a certain extent time-independent. Especially fairly dangerous geotechnical failure modes seem to be dominated by time-independent uncertainty, indicating that the associated failure rate is higher during the first years of service. This theory is supported by the fact that quay wall failures not induced by

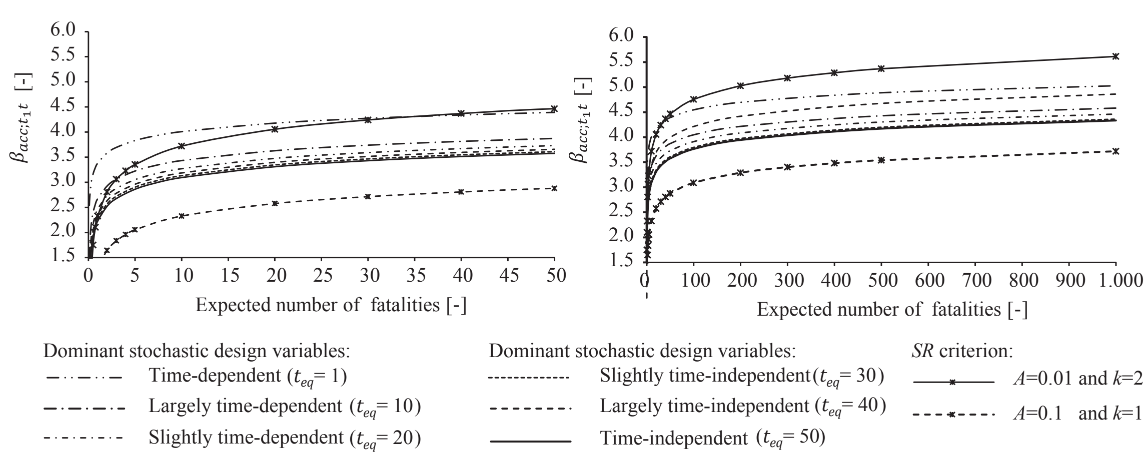

Fig. 11. Sensitivity analysis SR and LQI criterion with $t_{ref} = 50$, $L_{eq} = 40$, C_0 = €0.6 m, C_m = €0.1 m and SWTP = €3 m.

Table 1.9
Assessment criteria for each consequence class for the structure as a whole N_{PAR}.

Description	Consequence class				
	A	B	C	D	E
Qualitative	Negligible/low	Some	Considerable	High	Very high
Human safety					
Number of fatalities [40]	$N \leq 1$	$N \leq 5$	$N \leq 50$	$N \leq 500$	$N > 500$
Number of people at risk [6]	$N_{PAR} < 5$	$N_{PAR} < 50$	$N_{PAR} < 500$	$N_{PAR} < 1500$	$N_{PAR} > 1500$
Degree of warning [6,18]	Progression of failure is not possible and people at risk are able to escape in time	Redundant structural response and progression of failure is mitigated and failure is not sudden providing adequate warning signals	Progression of failure is mitigated, but failure is sudden without providing warning signals	Widespread progression of damage is likely to occur and failure is sudden without providing warning signals	Widespread progression, induced by unexpected and sudden environmental disasters, is possible
Social and environmental repercussion index [76]	SERI ≤ 5	SERI ≤ 15	SERI ≤ 25	SERI ≤ 30	SERI > 30
Economic					
Description [40]	Predominantly insignificant material damages	Material damages **and** functionality losses of significance for owners and operators **and** low or no social impact	Material losses **and** functionality losses of societal significance, causing regional disruptions **and** delays in important societal services over several weeks	Disastrous events causing severe losses of societal services **and** disruptions **and** delays at national scale over periods in the order of months	Catastrophic events causing losses of societal services **and** disruptions **and** delays beyond national scale over periods in the order of years
Accessibility [55]	Very little hindrance to shipping, railway transport, pipeline systems (Very short period, less than one day)	Small consequences for availability of navigation channels, railways, roads or pipeline corridors. (Barricade measures for a period of one day)	Short period of barricade with regard to navigation channels, railways, roads or pipeline corridors. (The availability is lower for a period of one week)	Damage to navigation channels, railways, roads or pipeline corridors. (The availability is lower for a period of weeks)	Loss of main navigation channels, railways, roads or pipeline corridors. (Main transport routes are unavailable for a period of months)
Ratio between direct failure costs and costs of safety investments $\rho = C_{f;direct}/C_{Investments}$ [43]	$\rho \leq 1$	$\rho \leq 2$	$\rho \leq 5$	$\rho \leq 10$	$\rho > 10$
Failure costs C_f corresponding to a failure length of 40 m	$C_f <$ €8m	$C_f <$ €50 m	$C_f <$ €200 m	$C_f <$ €1500 m	$C_f >$ €1500 m
Environmental [40]	Damages to the qualities of the environment of an order that can be restored completely in a matter of days	Damages to the qualities of the environment of an order that can be restored completely in a matter of weeks	Damages to the qualities of the environment limited to the surroundings of the failure event and that can be restored in a matter of weeks	Significant damages to the qualities of the environment contained at national scale but spreading significantly beyond the surroundings of the failure event and that can only be partly restored in a matter of months	Significant damages to the qualities of the environment spreading significantly beyond the national scale and that can only be partly restored in a matter of years to decades
Reputation [55]	No negative attention in media and no damage to the image of the port	Very short period of negative attention in local, regional and national media (> 1 day). Serious concerns among people living in the vicinity, local government, national government or external clients. Damage to image of a few stakeholders	Short and limited period of negative attention in local, regional and national media (> 2 days). Serious concerns among people living in the vicinity, local government, national government or external clients. Damage to image of the port for some time	Period of negative attention in local, regional and national media (> week), Serious concerns among people living in the vicinity, local government, national government or external clients. Damage to image of the port for some time	Long period of negative attention in local, regional and national media (> month). Very serious concerns among people living in the vicinity, local government, national government or external clients. Permanent damage to image of the port

Table 1.10
Annual target reliability indices for consequence classes of largely time-independent limit state functions of quay walls.

Criterion	Type	Consequence class				
		A Low	B Some	C Considerable	D High	E Very high
ISO 2394 [37]	Large[1]	–	3.1	*3.3*	*3.7*	–
	Medium[1]	–	3.7	*4.2*	*4.4*	–
	Small[1]	–	4.2	*4.4*	*4.7*	–
Economic optimisation[2,3]		2.8	3.4	3.8	4.2	excl.[5]
LQI criterion[2,3]		2.5	3.0	3.7	4.2	excl.[5]
IR criterion	$IRPA = 10^{-6}$	2.8	3.3	3.7	n/a	n/a
	$IRPA = 10^{-5}$	1.9	2.5	3.1	n/a	n/a
	$LIRA = 10^{-6}$	n/a	n/a	n/a	4.3[4]	excl.[5]
	$LIRA = 10^{-5}$	n/a	n/a	n/a	3.4[4]	excl.[5]
SR criterion	A = 0.01; k = 2	n/a	3.4	4.5	5.4	excl.[5]
	A = 0.1; k = 1	n/a	2.1	2.9	3.5	excl.[5]
Recommendation for design codes	($n_{eq} \ll n_{ref}$ or $t_{eq} \geq 20$)	2.8	3.4	3.8[6]	4.2[6]	excl. [5]

[1] Relative costs of safety measures.
[2] Dominant design variables are considered to be time-independent ($n_{eq} \ll n_{ref}$ or $t_{eq} \geq 20$) (Section 4).
[3] Input variables $t_{ref} = 50$, $L_{eq} = 40$, $C_0 = €0.6$ m, $C_m = €0.1$ m and $SWTP = €3$ m.
[4] This criterion is only active at a hazardous site/project location (Section 3).
[5] It is not possible to provide general recommendations. A project-specific study is recommended (Section 3).
[6] Verify whether *LIRA* or *SR* criteria are active.

environmental disasters, were mostly identified directly upon construction or in the first year after completion. In addition, no fatalities of end users due to quay failure have been identified in the Port of Rotterdam. The decrease in the failure rate during the useful life may explain the relatively low failure frequency of geotechnical structures compared to other civil engineering works [96].

The target reliability indices derived in this study were determined from three risk-acceptance criteria: economic optimisation, the individual risk (*IRPA*) criterion and the life quality index (*LQI*) criterion. The results were used to determine target reliability indices in accordance with the assessment criteria for structural robustness described in Table 1.9. It should be noted that the description of the failure consequences is related to the system as whole rather than to individual structural components [40]. The recommended target reliability indices in Table 1.10 are ascribed to the limit state functions of structural components and geotechnical failure modes, because the efficiency of safety measures as well as failure consequences differ per limit state. It should be noted that the recommended target reliability indices are only valid if progressive failure is mitigated [42,25,29]. The sensitivity analysis showed that differences in annual target reliability indices are fairly small for time-independent limit state functions. It is therefore recommended to evaluate annual target reliabilities, rather than lifetime reliability indices, and to implement annual reliability indices in design codes, which is in accordance with the recommendations of ISO 2394 [37] and Rackwitz [68]. Economic optimisation was found to be the governing risk-criterion. However, the societal costs will become fairly dominant in the case of class D. The *LIRA* and *SR* criteria are only relevant for failures with consequences that reach far beyond the quay wall site itself, for instance if installations with hazardous materials are affected. Therefore, they are not included in the recommended values, but should be considered separately when applicable. Table 1.10 also shows that the recommended annual target reliability indices are in the range of the guidance of ISO 2394 [37].

The failure consequences of quay walls in port areas (Fig. 12-I) with and without functional redundancy differ (Section 3.5) and were classified as class A and class B, representing 'low' and 'some' damage, respectively. The required reliability level of a commercial quay wall also depends on the image and reputation of a port as a safe environment for investments and work (Section 3.5). Another aspect that needs to be considered is the impact of failure on the availability and accessibility of main sailing routes. After an earthquake in Japan, numerous quay walls failed simultaneously [38], and hence multiple berths were unavailable for recovery, leading to much more serious economic repercussions [65]. When quay wall failure could lead to an explosion in, for instance, a chemical plant (Fig. 12-V) or to the breaking loose of a cruise ship induced by the failure of bollards, many more people are at risk. In these circumstances, a higher consequence class must be considered. The design of soil-retaining walls that are part of another system, such as a preliminary flood defence system, should take account of the length effect, and hence higher reliability indices have to be taken into consideration [13,42,79,87,94]. Although undoubtedly not all types of quay walls are covered, the examples listed in Table 1.11 will serve as a useful reference for categorising quay wall types for each consequence class.

7. Conclusion and recommendations

The results of this study provided guidance on reliability differentiation for commercial quay walls, but were also used to evaluate

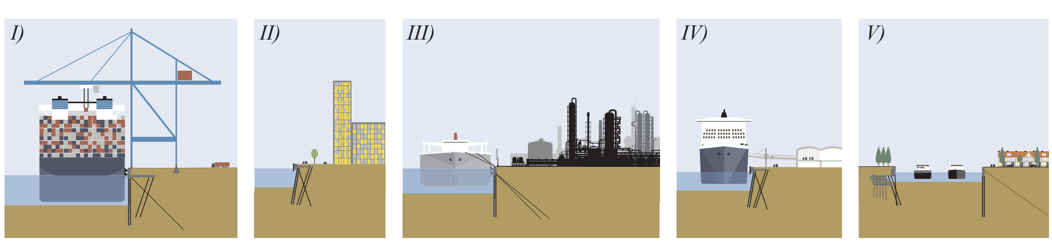

Fig. 12. Impression of different quay wall types: I) commercial quay wall; II) quay wall in urban area; III) quay wall that is part of a dangerous plant; IV) quay wall that facilitates cruise ships; V) quay walls that facilitate main sailing routes.

Table 1.11
Examples of quay wall types for the consequence classes described in Table 8 if and only if progressive failure is mitigated.

Examples of quay wall types				
A Negligible or low	B Some	C Considerable	D High	E Very high
Soil-retaining walls where the risk of fatalities is negligible or very low; quay walls are part of a terminal or port with functional redundancy	Quay walls are part of a terminal or port without functional redundancy	Quay walls in urban areas	Quay walls for which failure will lead to the failure of other structures, such as chemical or power plants; soil-retaining walls that are part of secondary flood defence systems or dams; quay walls needed for recovery after earthquake damage or tsunamis; quay walls that facilitate cruise ships	Soil-retaining walls that are part of a primary flood defence system, major dam or important sailing route

reliability indices for other types of quay and soil-retaining walls. The most important findings of this study are:

- In quay wall design, it is highly likely that dominant stochastic design variables are largely time-independent. This influences both the efficiency of safety measures and the present value of future failure costs, and hence optimal annual and lifetime target reliability indices.
- The extent to which limit state functions of quay walls are time-dependent differs between failure modes.
- Target reliability indices can be derived on the basis of economic optimisation in combination with the marginal life-saving cost principle. The annual and lifetime target reliability indices for failure modes of commercial quay walls found were in the range of 2.8–3.5 and 2.5–3.3, respectively.
- The recommendations for reliability differentiation in literature become more consistent and interpretable if the assessment framework ISO 2394 (2015) is extended with detailed information about type of failure, warning signals and the consequences of failure.

When defining target reliability indices for quay walls and other geotechnical structures, one should be very careful using the general guidance developed for buildings and bridges, because the degree and source of aleatory and epistemic uncertainty differ, as do the consequences of failure. It is strongly recommended to account for damage to the reputation of a terminal or port, because marginal safety costs appeared to be quite low compared to the total construction costs and expected benefits. During this research it was noticed that many experts find it difficult to make a quantitative estimate of the costs associated with failure and that little information is available with regard to the conditional probability to die given quay wall failure. Both aspects require further research. A detailed study with regard to the influence of time-independent design variables, failure costs and the efficiency of safety measures for each failure mode is highly recommended if one wants to improve quay wall design with respect to reliability and safety. The failure rates of fairly dangerous geotechnical failure modes seem to be much higher in the first period of service, indicating that if a quay wall has already survived a certain period these failure modes are less likely to occur [51]. It is therefore highly recommended to derive specific recommendations for assessing the reliability and safety of existing quay walls, and hence a quite different approach is foreseen that includes the development of the probability of failure over time. It is expected that the insight into the actual reliability level of soil-retaining walls will significantly increase if the uncertainty in the soil-structure interaction is reduced by advanced monitoring, for instance during the capital dredging works of the construction stage. The suggestions for quantifying reliability levels developed in this study also enable the determination of project-specific target reliability indices.

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Appendix A. Derivation of analytical formulas

In this appendix analytical formulas of the objective function C_{Total} and the associated derivative have been derived to determine the discounted future failure costs in each year of the reference period. The analytical formulas were also used in the sensitivity analyses (Sections 5.2 and 5.4). It should be noted that the Hasofer–Lind reliability index β is the decision parameter.

$$\min\{C_{Total} = C_{Investments}(\beta) + C_{\text{CapitalisedRisk}}(\beta)\} \tag{33}$$

$$\frac{\partial C_{Total}(\beta^*)}{\partial\beta} = 0 \tag{34}$$

$$C_{\text{Total}} = C_0 + C_m\beta + C_{\text{f}}P_{\text{f};t_0} + C_{\text{f}}\cdot\sum_{n=1}^{n_{ref}}\frac{\Delta P_{\text{f};t_n}}{(1+r)^n} \tag{35}$$

in which:

$C_{Investments}$ – Investments in safety measures [€]
$C_{\text{CapitalisedRisk}}$ – Present value of future failure costs [€]

β – Decision parameter [–]
β^* – Optimal reliability index [–]
C_0 – Initial construction costs independent of reliability level [€]
C_m – Marginal construction cost dependent on reliability level [€]
C_f – Summation of direct and indirect costs of failure [€]
$P_{f;t_0}$ – Part of the probability of failure not influenced by time interval $[0,t_{ref})$ [–]
ΔP_f – Reference period dependent increase in cumulative probability of failure in the nth year
n – Number of years [–]
n_{ref} – Number of years in the reference period [–]
r – Real annual discount rate [–]

The failure probability was determined on the basis of a standard normal distribution. Hence, the following equations were used:

$$\phi(\beta) = f(\beta) = \frac{1}{\sqrt{2\pi}}e^{-\frac{1}{2}\beta^2} \tag{36}$$

$$\Phi(\beta) = F(\beta) = \int_{-\infty}^{\beta} \frac{1}{\sqrt{2\pi}}e^{-\frac{1}{2}\beta^2}dx = \frac{1 + Erf(\beta/\sqrt{2})}{2} \tag{37}$$

$$\mathrm{Erf}(\beta) = F(\beta) = \frac{2}{\sqrt{\pi}}\int_0^{\beta} e^{-t^2}dt \tag{38}$$

in which:

$\phi(\beta)$ – Probability density function of normal distribution [–]
$\Phi(\beta)$ – Cumulative distribution function of normal distribution [–]
$\mathrm{Erf}(\beta)$ – Error function [–]

In this appendix the following properties of the standard normal distribution were used:

$$\Phi(-\beta) = 1{-}\Phi(\beta)\, for\ \beta \in \mathbb{R} \tag{39}$$

$$\Phi^{-1}(p) = -\Phi^{-1}(1{-}p)\, for\ p \in (0,\ 1) \tag{40}$$

$$\Phi(0) = 0.5 \tag{41}$$

$$\frac{\partial\Phi(\beta)}{\partial\beta} = \phi(\beta) = \frac{1}{\sqrt{2\pi}}e^{-\frac{1}{2}\beta^2} \tag{42}$$

$$\frac{\partial\Phi^{-1}(p)}{\partial \mathrm{p}} = \left(\frac{\partial\Phi(\beta)}{\partial \mathrm{p}}\right)^{-1} = \frac{\sqrt{2\pi}}{e^{-\frac{1}{2}\beta^2}} \tag{43}$$

The mathematical notation of time-variant reliability problems is well described by Sudret [89,90] and Rackwitz [69]. In addition to the formulation of the limit state function (Section 3.2), aleatory uncertainty such as deterioration or due to variable loads must also be taken into account. The failure probability in the time interval $(t,\ t + \Delta t)$ is defined in accordance with the notation of Sudret and Rackwitz [89] as:

$$P_f(t,\ t + t) = \mathbb{P}(\ \exists\ \mathfrak{t}[\mathfrak{t},\ \mathfrak{t} + \mathfrak{t}]\text{: } \mathbb{Z} \leqslant \emptyset) \tag{44}$$

When the time interval $(t,\ t + \Delta t)$ approaches zero, the point-in-time instantaneous probability of failure – or in other words the failure rate – can be found [89]. The probability of failure at time-instant t is defined as:

$$P_{f,i}(t) = \mathbb{P}(\mathbb{Z} \leqslant \emptyset) \tag{45}$$

Eq. (46) shows the classical upper and lower bounds of the probability of failure in the time interval $(t,\ t + \Delta t)$ [69].

$$\max_{t(t,t+\Delta t)} P_{f,i}(t_i) \leqslant P_f(t,\ t + \Delta t) \leqslant P_{f;0} + E[N^+(t,\ t + \Delta t)] \tag{46}$$

in which:

$P_f(t,\ t + \Delta t)$ – Probability of failure in interval $(t,\ t + \Delta t)$ [–]
$P_{f,i}(t_i)$ – Probability of failure at time instant t_i [–]
$P_{f;0}$ – Part of the probability of failure being independent of time interval $(t,\ t + \Delta t)$ [–]
N^+ – Number of outcrossings of the limit state [–]
Z – State function [–]

The usual approach to a time-variant reliability problem is based on the computation of the outcrossing rate $E[N^+(t,\ t + \Delta t)]$ of the limit state under consideration [89]. However, in this study the upper and lower bounds were not used. Instead, the probability of failure in time interval $P_f(t,\ t + \Delta t)$ was defined by subdividing $P_{f;t_n}$ into a block that is largely time-independent $P_{f;0}$ and a block that is fully time-dependent $\sum^{\Delta} P_{f;t_n}$ (Fig. 5).

$$P_{f;t_n} = P_{f;0} + \sum \Delta P_{f;t_n} \tag{47}$$

$$P_{f;t_{ref}} = P_{f;0} + \sum_{n=1}^{n_{ref}} \Delta P_{f;t_n} \tag{48}$$

The main difference between a time-invariant and a time-variant reliability problem is that in the latter case one does not know when a failure occurs [90]. When t_{eq} is determined, the marginal change in probability of failure in each year can be established by subtracting the cumulative probability of failure in the intervals [0, t_n] and [0, t_{n-1}] using Eq. (49). This equation is a function of the probability of failure during a reference period of one year $P_{f;t_1}$ (Fig. 6). The cumulative failure probability in the time interval [0, t_n] was derived by transforming Eq. (7) into Eq. (50) and the time-independent part using Eq. (51). The equations are illustrated in Fig. 6. When $P_{f;t_1}$, t_{eq} and t_{ref} are known $P_{f;t_{ref}}$ and $\Delta P_{f;t_n}$ can be derived by the following equations.

$$\Delta P_{f;t_n} = P_{f;t_n} - P_{f;t_{n-1}} \tag{49}$$

$$P_{f;t_n} = 1 - \left(1 - P_{f;t_1}\right)^{an+b} \tag{50}$$

$$P_{f;t_0} = 1 - \left(1 - P_{f;t_1}\right)^{b} \tag{51}$$

$$a = \frac{n_{eq} - 1}{n_{ref} - 1} \tag{52}$$

$$b = \frac{n_{ref} - n_{eq}}{n_{ref} - 1} \tag{53}$$

The probability of failure over a certain time interval (t, $t + \Delta t$) can be described using the cumulative distribution function $F(\beta)$.

$$P(X > x) = 1 - F(\beta) \tag{54}$$

$$P_{f;t_1} = P(X > x) = 1 - F(\beta) = \Phi(-_{t_1}) \tag{55}$$

$$1 - P_{f;t_1} = \Phi(\beta_{t_1}) \tag{56}$$

The above transformation was applied to Eq. (50).

$$P_{f;t_n} = 1 - \left(\Phi\left(\beta_{t_1}\right)\right)^{an+b} \tag{57}$$

$$P_{f;t_{n-1}} = 1 - \left(\Phi\left(\beta_{t_1}\right)\right)^{a(n-1)+b} \tag{58}$$

$$\Delta P_{f;t_n} = P_{f;t_n} - P_{f;t_{n-1}} = (1 - \left(\Phi\left(\beta_{t_1}\right)\right)^{an+b}) - (1 - \left(\Phi\left(\beta_{t_1}\right)\right)^{a(n-1)+b}) \tag{59}$$

If one implements the following denotation:

$$\Phi(\beta_{t_1}) = \Phi_1 \tag{60}$$

ΔP_f is defined as:

$$\Delta P_f = -\Phi_1^{an}\Phi_1^{b} + \Phi_1^{an}\Phi_1^{-a}\Phi_1^{b} \tag{61}$$

$$\Delta P_f = \Phi_1^{an}\Phi_1^{-a}\Phi_1^{b} - \Phi_1^{an}\Phi_1^{b} \tag{62}$$

$$\Delta P_f = \Phi_1^{an}\Phi_1^{b}(\Phi_1^{-a} - 1) \tag{63}$$

Consequently, the following formula of $C_{CapitalisedRisk}$ was found:

$$C_{\text{CapitalisedRisk}} = C_f \sum_{n=0}^{t_{ref}} \frac{\Phi_1^{an}\Phi_1^{b}(\Phi_1^{-a} - 1)}{(1+r)^n} for n \in [0, n_{ref}) \tag{64}$$

$$C_{\text{CapitalisedRisk}} = C_f \Phi_1^{b}(\Phi_1^{-a} - 1) \sum_{n=0}^{n_{ref}} \left(\frac{\Phi_1^{a}}{1+r}\right)^n \tag{65}$$

$$C_{\text{CapitalisedRisk}} = C_f (\Phi_1^{b-a} - \Phi_1^{b}) \sum_{n=0}^{n_{ref}} \left(\frac{\Phi_1^{a}}{1+r}\right)^n \tag{66}$$

If one separates the part $P_{f;0}$ – being not influenced by a change in reference period – from the part $\sum^{\Delta} P_{f;t_n}$ – which dependents on the reference period – the following equation is obtained:

$$C_{\text{CapitalisedRisk}} = C_f P_{f;0} + C_f (\Phi_1^{b-a} - \Phi_1^{b}) \sum_{n=1}^{n_{ref}} (\frac{\Phi_1^{a}}{1+r})^n for n \in [1, n_{ref}) \tag{67}$$

Now the following transformation rule is used:

$$\sum_{i=0}^{n_{ref}} Ax^i = A\frac{1 - x^{n_{ref}+1}}{1 - x} \tag{68}$$

and thus,

$$\sum_{i=0}^{n_{ref}} Ax^i = Ax^0 + \sum_{i=1}^{n_{ref}} Ax^i = Ax^0 + \sum_{i=0}^{n_{ref}-1} Ax^{i+1} = Ax^0 + \sum_{i=0}^{n_{ref}-1} Ax^i x = Ax^0 + Ax\frac{1-x^{n_{ref}}}{1-x} \tag{69}$$

where,

$$x = \frac{\Phi_1^a}{1+r}\text{(in fact this an adjusted grow rate)} \tag{70}$$

$$A = C_f(\Phi_1^{b-a} - \Phi_1^b) \tag{71}$$

Consequently, the following formula of $C_{CapitalisedRisk}$ was found:

$$C_{\text{CapitalisedRisk}} = C_f P_{f;0} + A(x)\frac{1-(x)^{n_{ref}}}{1-(x)} \tag{72}$$

$$C_{\text{CapitalisedRisk}} = C_f P_{f;0} + C_f(\Phi_1^{b-a} - \Phi_1^b)\left(\frac{\Phi_1^a}{1+r}\right)\frac{1-\left(\frac{\Phi_1^a}{1+r}\right)^{n_{ref}}}{1-\left(\frac{\Phi_1^a}{1+r}\right)} \tag{73}$$

$$C_{\text{CapitalisedRisk}} = C_f P_{f;0} + C_f\frac{(\Phi_1^b - \Phi_1^{b+a})}{1+r}\frac{1-\left(\frac{\Phi_1^a}{1+r}\right)^{n_{ref}}}{1-\left(\frac{\Phi_1^a}{1+r}\right)} \tag{74}$$

$$C_{\text{CapitalisedRisk}} = C_f(1-\Phi_1^b) + C_f\frac{(\Phi_1^b - \Phi_1)}{1+r}\frac{1-\left(\frac{\Phi_1^a}{1+r}\right)^{n_{ref}}}{1-\left(\frac{\Phi_1^a}{1+r}\right)} \tag{75}$$

The analytical formula of the objective function now becomes:

$$f(\beta) = C_0 + C_{m_{t_1}} + C_f(1-\Phi_1^b) + C_f c(\Phi_1^b - \Phi_1)\frac{1-(c\Phi_1^a)^{n_{ref}}}{1-c\Phi_1^a} \tag{76}$$

in which:

$\Phi_1 = \Phi(\beta_{t_1})$ – Standard normal cumulative distribution function [–]

The derivative of the objective function was used to determine the optimal reliability index β^* and to derive insight into the sensitivity of the input variables, such as the discount rate r, the absolute value C_f and marginal costs of safety measures C_m and the reference period t_{ref}.

$$\frac{\partial C_{\text{Investments}}(\beta^*)}{\partial_{t_1}} + \frac{\partial C_{\text{CapitalisedRisk}}(\beta^*)}{\partial_{t_1}} = 0 \tag{77}$$

$$C_m + \frac{\partial C_{\text{CapitalisedRisk}}(\beta^*)}{\partial_{t_1}} = 0 \tag{78}$$

$$\frac{\partial C_{\text{CapitalisedRisk}}(\beta^*)}{\partial_{t_1}} = -C_m \tag{79}$$

The formula of $C_{CapitalisedRisk}$ was rearranged as follows:

$$c = 1/(1+r) \tag{80}$$

$$C_{\text{CapitalisedRisk}} = C_f(1-\Phi_1^b) + C_f c(\Phi_1^b - \Phi_1)\frac{1-(c\Phi_1^a)^{n_{ref}}}{1-c\Phi_1^a} \tag{81}$$

$$C_{\text{CapitalisedRisk}} = c(\Phi_1(\text{x})^b - \Phi_1(\text{x}))\frac{1-(c\Phi_1(\text{x})^a)^{n_{ref}}}{1-c\Phi_1(\text{x})^a} \tag{82}$$

$$C_{\text{Capitalised Risk}} = \frac{c(\Phi_1(x)^b - \Phi_1(x))(1-(c\Phi_1(x)^a)^{n_{ref}})}{1-c\Phi_1(x)^a} \tag{83}$$

$$= \frac{c((0.5+0.5erf(x/\sqrt{2}))^b - (0.5+0.5erf(x/\sqrt{2})))(1-(c(0.5+0.5erf(x/\sqrt{2}))^a)^{n_{ref}})}{1-c(0.5+0.5erf(x/\sqrt{2}))^a} \tag{84}$$

The derivative of $C_{CapitalisedRisk}$ was derived using a derivative calculator [108].

$$\frac{\partial C_{\text{CapitalisedRisk}}}{\partial_{t_1}} = -C_f b\phi_1\Phi_1^{b-1} + C_f\left(\frac{c(b\phi_1\Phi_1^{b-1} - \phi_1)(1-(c\Phi_1^a)^{n_{ref}})}{1-c\Phi_1^a} + \frac{ac^2\phi_1(1-\Phi_1^a)(1-(c\Phi_1^a)^{n_{ref}})}{(1-c\Phi_1^a)^2} - \frac{acn_{ref}\phi_1(\Phi_1^{b-1}-1)(c\Phi_1^a)^{n_{ref}}}{(1-c\Phi_1^a)}\right) \tag{85}$$

in which:

$\phi_1 = \phi_1(\beta_{t_1})$ – Standard normal probability density function [–]

$\Phi_1 = \Phi(\beta_{t_1})$ – Standard normal cumulative distribution function [–]

The derivative of $C_{Investments}$ is presented in Eq. (87).

$$C_{\text{Investments}}(\beta_{t_1}) = C_0 + C_{m_{t_1}} \tag{86}$$

$$\frac{\partial C_{\text{Investments}}(\beta_{t_1})}{\partial_{t_1}} = C_m \tag{87}$$

The solution to the optimisation problem was found using the following equations:

$$min\{C_{\text{Total}}(\beta_{t_1})\} = \{C_{\text{Investments}}(\beta_{t_1}) + C_{\text{CapitalisedRisk}}(\beta_{t_1})\} \tag{88}$$

$$\frac{\partial C_{\text{Investments}}(\beta_{t_1})}{\partial_{t_1}} + \frac{\partial C_{\text{CapitalisedRisk}}(\beta_{t_1})}{\partial_{t_1}} = 0 \to \to^{\beta^*}$$

$$\frac{\partial C_{\text{CapitalisedRisk}}(\beta^*)}{\partial_{t_1}} = -C_{\text{m}} \tag{89}$$

$$-C_{\text{f}}\, b\phi_1\Phi_1^{b-1} + C_{\text{f}}\left(\frac{c(b\phi_1\Phi_1^{b-1} - \phi_1)(1-(c\Phi_1^a)^{n_{ref}})}{1-c\Phi_1^a} + \frac{ac^2\phi_1(1-\Phi_1^a)(1-(c\Phi_1^a)^{n_{ref}})}{(1-c\Phi_1^a)^2} - \frac{acn_{ref}\phi_1(\Phi_1^{b-1}-1)(c\Phi_1^a)^{n_{ref}}}{(1-c\Phi_1^a)}\right) = -C_{\text{m}} \tag{90}$$

Appendix B. Probability distribution functions

See Table 1.12

Table 1.12
Type of distribution and variation coefficient (CoV) of important model parameters.

Design parameter	Soil layer	SI	μ	Distribution	CoV
Unit weight of soil γ_{sat}	Backfill[1]	kN/m[3]	20	Normal	0.05[2]
	Reclamation sand[1]	kN/m[3]	20	Normal	0.05[2]
	Holocene sand[1]	kN/m[3]	20	Normal	0.05[2]
	Clay layer[1]	kN/m[3]	19	Normal	0.05[2]
	Pleistocene sand[1]	kN/m[3]	20	Normal	0.05[2]
Friction angle φ'_{rep}	Backfill[1]		39	Normal	0.10[2,3]
	Reclamation sand[1]		36	Normal	0.10[2,3]
	Holocene sand[1]		36	Normal	0.10[2,3]
	Clay layer[1]		27	Normal	0.10[2.3]
	Pleistocene sand[1]		39	Normal	0.10[2,3]
Cohesion c'	Clay layer[1]	kpa	7	Lognormal	0.20[2]
Soil stiffness $E_{50;ref}$	Backfill[1]	Mpa	50	Lognormal	0.20[2]
	Reclamation sand[1]	Mpa	30	Lognormal	0.20[2]
	Holocene sand[1]	Mpa	30	Lognormal	0.20[2]
	Clay layer[1]	Mpa	5	Lognormal	0.20[2]
	Pleistocene sand[1]	Mpa	60	Lognormal	0.20[2]
Yield strength f_y		N/mm[2]	510[5]	Lognormal	0.07[4]
Tube diameter D_{tube}		m	1.62[5]	Normal	0.01
Wall thickness t_{tube}		m	0.23[5]	Uniform	0.05
Annual live loads Q_{t1}		kN/m[2]	26[5]	Gumbel	0.20
Lifetime live loads Q_{t50}		kN/m[2]	42[5]	Gumbel	0.10

[1] Top levels backfill sand, reclamation sand, Holocene sand, clay layer and Pleistocene sand are MSL + 5.0 m, MSL-0.0 m, MS-8.0 m, MSL-20.0 m and MSL-21.0 m, respectively. Where, MSL = Mean Sea Level.
[2] Value is based on soil investigation in accordance with Eurocode 7.
[3] Value represents the average variability of the soil layer and is verified by the research of Huijzer [36,37].
[4] Value is based on Peters et al. [67] and the probabilistic model code [43].
[5] Value is based on a quay wall situated in Rotterdam; Maasvlakte.

References

[1] Allaix D, Steenbergen RDJM, Wessels JFM. Target reliability levels for the design of quay walls. Delft, The Netherlands: TNO; 2017.
[2] Allaix D, Steenbergen RDJM, Wessels JFM. Target reliability levels for existing quay walls. Delft, The Netherlands: TNO; 2018.
[3] Andrianov G, Burriel S, Cambier S, Dutfoy A, Dutka–Malen I, de Rocquigny E, et al. Open TURNS, an open source initiative to Treat Uncertainties, Risks 'N Statistics in a structured industrial approach. In: Proceedings of the European Safety and Reliability Conference 2007, ESREL 2007 – Risk, Reliability and Societal Safety; 2007. 2.
[4] Ang AHS, De Leon D. Determination of optimal target reliabilities for design and upgrading of structures. Struct Saf 1997;19:91–103.
[5] Arangio S. Reliability based approach for structural design and assessment: performance criteria and indicators in current European codes and guidelines. Int J

Lifecycle Perform Eng 2012;1:64–91.
[6] ASCE 7–10. Minimum Design Loads for Buildings and Other Structures. Reston, USA: American Society of Civil Engineers; 2010.
[7] Bhattacharya B, Basu R, Ma K. Developing target reliability for novel structures: the case of the Mobile Offshore Base. Mar Struct 2001;14:37–58.
[8] Böckmann J, Grünberg J. Determination of safety levels for quay wall structures. In: Proceedings of the 7th International probabilistic workshop; 2009. p. 327–41.
[9] Bötger E, Linde A. Verschillende perspectieven op de nieuwe waterveiligheidsnormen. Utrecht, The Netherlands; 2014.
[10] BS 6349-1-2. Maritime works: general – code of practice for assessment of actions. London, United Kingdom; 2016. ISBN: 978-0-580-76229-1.
[11] BS 6349-2. Maritime works – Part 2: code of practice for the design of quay walls, jetties and dolphins. London, United Kingdom; 2010. ISBN: 978-0-580-73243-0.
[12] Buijsingh P. ALARP Guide. Shell. Rijswijk, The Netherlands; 2013. SR.13.14005.
[13] Calle EOF, Spierenburg SEJ. Veiligheid van damwandconstructies – Onderzoeksrapportage deel 1, for CUR committee C69. Delft, The Netherlands. Deltares report CO-31690/12; 1991.
[14] Chryssanthopoulos MK, Janssens V, Imam B. Modelling of Failure Consequences for Robustness Evaluation. London, United Kingdom. IABSE–IASS Symposium: Taller, Longer, Lighter; 2011.
[15] Cork S, Chamberlain N. CIRIA C746, Old waterfront walls. London, United Kingdom; 1015. ISBN: 9-780860-177517.
[17] Diamantidis D. A critical view on environmental and human risk acceptance criteria. Int J Environ Sci Develop 2017;8(1):62–8.
[18] DNV. Structural reliability analysis of marine structures. Classification notes NO. 30.6 Hovik, Norway. Det Norske Veritas; 1992.
[20] Fenton AG, Naghibi F, Dundas D, Bathurst JR, Griffiths DV. Reliability-based geotechnical design in 2014 Canadian Highway Bridge Design Code. Can Geotech J 2016;53:236–51.
[21] Fischer K, Barnardo-Viljoen C, Faber MH. Deriving target reliabilities from the LQI. Lyngby, Denmark: LQI Symposium in Kgs; 2012.
[22] Fischer K, Faber MH. The LQI acceptance criterion and human compensation costs for monetary optimization – a discussion note. Lyngby, Denmark: LQI Symposium in Kgs; 2012.
[23] Fischer K, Virguez E, Sánchez-Silva M, Faber MH. On the assessment of marginal life saving costs for risk acceptance criteria. Struct Saf 2013;44:37–46.
[24] Franks A. A simplified approach to estimating individual risk. Westbrook, United Kingdom: Amy Vectra Limited; 2017.
[25] Gaba A, Hardy S, Doughty L, Powrie W, Selemetas D, Skanska C. C760 Guidance on embedded retaining wall design. London, United Kingdom: CIRIA; 2017.
[26] Gijt de JG. A history of quay walls. The Hague, The Netherlands; 2010. ISBN: 9789059810327.
[27] Gijt de JG, Vinks R. Cost of quay walls including life cycle aspects. MTEC Singapore, Proceedings of the international maritime-port technology and development conference; 2011. p. 135–39.
[28] Gijt de JG. CUR 211 Quay walls. 1st ed. Gouda, The Netherlands: SBRCURnet; 2003.
[29] Gijt de JG, Broeken ML. CUR 211 Quay walls. 2nd ed. Leiden, The Netherlands: SBRCURnet; 2013.
[30] Grabe J. Recommendations of the Committee for Waterfront Structures Harbours and Waterways EAU 2012. 9th ed. Hamburg, Germany; 2012. ISBN: 978-3-433-03110-0.
[31] Gulvanessian H, Holický M. Eurocodes: using reliability analysis to combine action effects. Struct Build 2005;158:243–52.
[32] Hannink G. Van onzekerheid naar betrouwbaarheid: Handreiking voor geotechnisch ontwerpers. Gouda, The Netherlands; 2008. ISBN: 978-90-376-0509-9.
[33] Health and Safety Executive. Reducing risks, protecting people. Norwich, United Kingdom; 2001. ISBN: 0-7176-2151-0.
[34] Health and Safety Executive. Societal risk: Initial briefing to societal risk technical advisory group. Norwich, United Kingdom; 2009.
[35] Holický M. The target reliability and design working life. Saf Secur Eng 2011;IV:161–9.
[36] Huijzer GP, Hannink G. The construction of parameterized subsurface models. In: Proceedings 11th European conference on soil mechanics and foundation engineering. Denmark, Copenhagen; 1995.
[37] Huijzer GP. Eindrapport probabilistische analyse damwand constructies. The Netherlands Rotterdam; 1996.
[38] Iai S, Ichii K, Morita T. Analysis of damage to quay walls during 1995 great Hanshin earthquake, Japan. Yokosuka, Japan. Eleventh world conference earthquake engineering; 1996. ISBN: 0080428223.
[39] ISO 2394. General principles on reliability for structures. Geneva, Switzerland: International Organization for Standardization; 1998.
[40] ISO 2394. General principles on reliability for structures. Geneva, Switzerland: International Organization for Standardization; 2015.
[41] ISO 2398. Bases of design of structures – assessment of existing structures. Geneva, Switzerland: International Organization for Standardization; 2010.
[42] Janssen HL. CUR 166 Sheet pile walls. 6th ed. Gouda, The Netherlands: SBRCURnet; 2012.
[43] JCSS. Probabilistic model code. Part 1. Joint Committee on Structural Safety; 2001. www.jcss.byg.dtu.dk.
[44] Johansen IL. Foundations and fallacies of risk acceptance criteria. NTNU; 2010. ISBN: 978-82-7706-2281.
[45] Jongejan R. How safe is safe enough? The government's response to industrial and flood risks. Delft, The Netherlands: Technical University Delft; 2008.
[46] Jongejan R, Jonkman SN, Maaskant B. The potential use of individual and societal risk criteria within the Dutch flood safety policy (part 1): Basic principles. In: Reliability, Risk and Safety: Theory and applications, proceedings from ESREL; 2009.
[47] Jonkman SN, Jongejan R, Maaskant B. The use of individual and societal risk criteria within the dutch flood safety policy – nationwide estimates of societal risk and policy applications. Risk Anal 2011;31:282–300.
[48] Jonkman SN, Gelder van PHAJM, Vrijling JK. An overview of quantitative risk measures for loss of life and economic damage. Elsevier J Hazard Mater 2003;A99:1–30.
[50] Kubler O. Applied Decision-Making in Civil Engineering. Zurich, Switzerland; 2006.
[51] Kunz C. Ein Konzept für Teilsicherheitsbeiwerte für bestehende Wasserbauwerke. Bautechnik 2015;92:549–56.
[52] Lentz A. Acceptability of civil engineering decisions involving human consequences. Munich, Germany: Technischen Universität München; 2007.
[53] Leonardo da Vinci Pilot Project. Implementation of Eurocodes – Handbook 2 – Reliability backgrounds. Prague, Czech Republic; 2005. CZ/02/B/F/PP-134007.
[54] Leveson N. New accident model for engineering safer systems. Saf Sci 2004;42:237–70.
[55] Ligtvoet A, Lei van der T. Analyse bedrijfswaarden HbR. Delft, The Netherlands: Technische Universiteit Delft; 2012.
[56] Losada M, Benedicto M. Target design levels for maritime structures. J Waterway Port Coast Ocean Eng 2005:171–80.
[57] McCann MW, Paxson G. Uncertainty in Dam Failure Consequence Estimates. FLOODrisk 2016 – 3rd European conference on flood risk management; 2016. eISSN: 2267-1242.
[58] Melchers RE. Structural reliability theory in the context of structural safety. Civil Eng Environ Syst 2007;24:55–96.
[60] NEN–EN 1990. Eurocode – Basis of structural design. Brussels, Belgium: European Committee for standardization; 2011.
[61] NEN 6700. Technische grondslagen voor bouwconstructies – TGB 1990 – Algemene basiseisen. Delft, The Netherlands; 2005. ICS: 91.080.01.
[63] Norme Tecniche per le Costruzioni. Infrastrutture. Rome, Italy; 2008.
[64] NORSOK Z-013N. Risk and emergency preparedness analysis. Oslo, Norway: Norsok Standard; 2001.
[65] OCDI. Technical Standards and Commentaries for Port and Harbour Facilities in Japan. Tokyo, Japan: The Overseas Coastal Area Development Institute; 2009.
[66] Paltrinieri N, Khan F. Dynamic risk analysis in the chemical and petroleum industry: evolution and interaction with parallel disciplines in the perspective of industrial application. Elsevier; 2016. ISBN: 9780128037652.
[67] Peters DJ, Sadowski A, Rotter M, Taras A. Calibration of Eurocode design models of thin-walled cylinder under bending with full scale tests. Copenhagen, Denmark. Eurosteel 2017, 1; 2017. p. 3729–40.
[68] Rackwitz R. Optimization – the basis of code making and reliability verification. Struct Saf 2000;22:27–60.
[69] Rackwitz R. Reliability analysis – a review and some perspectives. Struct Saf 2001;23:365–95.
[73] Rackwitz R. The effect of discounting, different mortality reduction schemes and predictive cohort life tables on risk acceptability criteria. Reliab Eng Syst Saf 2006;91:469–84.
[74] R. Rackwitz 2008 The philosophy behind the life quality index and empirical verification Jr Committee Struct Saf.
[75] Rausand M. Risk assessment theory, methods, and applications. Wiley; 2013. ISBN: 978-0-470-63764-7.
[76] ROM 0.0. General procedure and requirements in the design of harbor and maritime structures. Madrid, Spain; 2002. ISBN: 84-888975-30-9.
[78] ROM 0.5–05. Geoptechnical recoomendations for the Design of Maritime and Harbour works. Madrid, Spain; 2008. ISBN: 978-84-88975-62-2.
[79] Roubos AA, Grotegoed D. Urban Quay walls. Delft, Netherlands: SBRCURnet; 2014.
[80] SANS 10160–1. Basis of structural design and actions for buildings and industrial structures Part 1: basis of structural design. Pretoria, South Africa; 2010. ISBN: 978-0-626-24757-7.
[81] Schweckendiek T, Courage WMG, Gelder van PHAJM. Reliability of sheet pile walls and the influence of corrosion – structural reliability analysis with finite elements. Stavanger, Norway. Proceedings of the European Safety and Reliability Conference 2007 (ESREL 2007); 2007. pp. 1791–99.
[83] Skjong R, Vanem E, Endresen Ø. Risk evaluation criteria: Technical report. SAFEDOR –D-4.5.2 DNV; 2007.
[84] Smit CF. Reliability based optimization of concrete structural components. Stellenbosch, South Africa; 2014.
[85] Steenbergen RDJM, Vrouwenvelder ACWM. Safety philosophy for existing structures and partial factors for traffic loads on bridges. Heron, 55; 2010. p. 123–39.68.
[86] Steenbergen RDJM, Sýkora M, Diamantidis D, Holický M, Vrouwenvelder ACWM. Economic and human safety reliability levels for existing structures. Struct Concr 2015;16:323–32.
[87] STOWA. Leidraad Waterkerende Kunstwerken in regionale waterkeringen. Amersfoort, The Netherlands: Technische adviescommissie waterkeringen; 2011.
[88] Structural Concrete. Textbook on behaviour, design and performance. 2nd ed., vol. 5. Lausanne, Switzerland; 2012. ISBN: 978-2-88394-102-1.
[89] Sudret B. Analytical derivation of the outcrossing rate in time-variant reliability problems. Struct Infrastruct Eng 2008;4:353–62.
[90] Sudret B. Time-variant reliability problems. ISTE/Wiley; 2011. p. 187–206.
[91] Sykora M, Holický M. Target reliability levels for the assessment of existing structures. Proc ICASPII 2011:1048–56.
[92] Sýkora M, Holický M, Lenner R, Manas P. Target reliability levels for existing bridges considering emergency and crisis situations. Adv Military Technol 2014;9.
[93] Sýkora M, Diamantidis D, Holický M, Jung K. Target reliability for existing structures considering economic and societal aspects. Struct Infrastruct Eng

2017;13–1:181–94.
[94] TAW. Leidraad kunstwerken. Delft, The Netherlands: Technische adviescommissie waterkeringen; 2003.
[95] TAW. Enkele gedachten aangaande een aanvaardbaar risiconiveau in Nederland. Den Haag: Technische adviescommissie voor de waterkeringen; 1985.
[96] Terwel KC. Structural Safety, Study into critical factors in the design and construction process. Delft, The Netherlands; 2014. ISBN: 978-94-6259-174-5.
[97] Teixeira A, Rippi K, Schweckendiek T, Brinkman H, Nuttall J, Hellebrandt L, Courage W. Soil-structure interaction – reliability analysis of a retaining wall. Delft, The Netherlands; 2015.
[98] Vrouwenvelder ACWM, Holický M, Tanner CP, Lovegrove DR, Canisius EG. Risk Assessment and Risk Communication in Civil Engineering. CIB Report: Publication 259; 2001. ISBN: 90-6363-026-3.
[99] Vrouwenvelder ACWM, Siemens AJM. Probabilistic calibration procedure for the derivation of partial safety factors for the Netherlands building codes. Heron 1987:9–29.
[100] Vrouwenvelder ACWM, Scholten N. Assesment criteria for existing structures. Struct Eng Int 2010;20:62–5.
[102] Vrouwenvelder ACWM. The fundamentals of Structural Building Codes. Cape Town, South Africa. In: Proceedings of the International Conference on Structural Engineering, Mechanics and Computation, vol. 1; 2001. p. 183–93. ISBN: 0-08 -043948-9.
[103] Vrijling JK. Probabilistic design of water defense systems in the Netherlands. Elsevier Relaib Eng Syst Saf 2001;74:337–44.
[104] Vrijling JK, Hengel van W, Houben RJ. Acceptable risk as a basis for design. Elsevier Relaib Eng Syst Saf 1998;59:141–50.
[105] Vrijling JK, Gelder van PHAJM. An analysis of the valuation of a human life. In: Proceedings of ESREL 2000, SARS and SRA–Europe annual conference; 2000. p. 197–00.
[106] USACE. Risk–Based Analysis in Geotechnical Engineering for Support of Planning Studies, ETL1110-2-556; 1999. Washington, D.C. United States.
[107] Wolters HJ, Bakker KJ, Gijt de JG. Reliability of quay walls using finite element analysis. Delft, The Netherlands: Delft University of Technology; 2012.
[108] http://www.derivative-calculator.net/.