

RELIABILITY ASSESSMENT OF THE STABILITY OF A WORKING PLATFORM ON VERY SOFT GROUND: A CASE STUDY

Divya Varkey

Water & Maritime, Royal HaskoningDHV, The Netherlands. E-mail: divya.varkey@rhdhv.com

Saeed Askarian

Water & Maritime, Royal HaskoningDHV, UK. E-mail: saeed.askarian@rhdhv.com

Chris Hartley

Water & Maritime, Royal HaskoningDHV, UK. E-mail: chris.hartley@rhdhv.com

This paper presents the reliability assessments for a temporary embankment planned to be constructed in Belfast. The case study involves constructing a temporary working platform on very soft ground to support the equipment required to construct a new quay wall by adopting a land-based approach. To ensure the stability of the working platform, the construction of the fill is carried out in lifts with limited thicknesses and hold periods between them. The undrained shear strength of the very soft estuarine deposits making up the subsoil has been identified using field tests and laboratory tests on samples collected during the ground investigation. The undrained shear strength of the underlying ground has a significant impact on the results of the slope stability assessment. To investigate the effects of variability of the strength parameters on the results of the slope stability assessment, a probabilistic approach has been adopted in this paper. In particular, the reliability indices and probabilities of failure of the slope under varying loading conditions have been assessed using the first-order reliability method (FORM) and using Monte Carlo simulations. The slope stability assessments have been carried out using a commercially available limit equilibrium software together with in-house scripts to perform a significant number of probabilistic calculations. The results of the assessments have been compared with those obtained using the deterministic analyses. The results of the probabilistic assessments provide confidence that the proposed land-based methodology meets Eurocode requirements for the probability of failure of the embankment slope and hence is leading to a safe design.

Keywords: Case study, probabilistic assessments, reliability, slope stability, uncertainty.

1. Introduction

This paper outlines the results of the reliability assessment of the stability of a working platform which will serve as a piling platform for the construction of a quay wall in Belfast. The working platform is anticipated to be built on an approximately 20m thick layer of very soft to soft clay ($C_u = 8$ to 32 kPa), underlain by a Glacial Till. To ensure overall stability and sufficient bearing capacity of the platform, the construction is planned to be carried out in separate lifts of fill placement with hold periods between them by considering a geotextile layer under the fill. This would lead to an increase of the shear strength of the very soft and soft Estuarine deposits (Sleech) due to consolidation. The design has been carried out before based on the deterministic approach according to BS EN 1997 (2004) and other relevant Standards, specifically BS8006-1 (2010). This paper provides the results of the reliability assessment of the overall stability of the platform and a comparison with the outputs of the deterministic method.

2. Project Information and Results of the Deterministic Design method

The working platform is going to be constructed in the shallow water zone to support the piling works of a quay wall. The maximum thickness of the fill is 5.5 metres which is going to be placed on an approximately 20 m thick layer of Sleech. A cross-section of the platform and the underlying soil layers is shown in Figure 1.

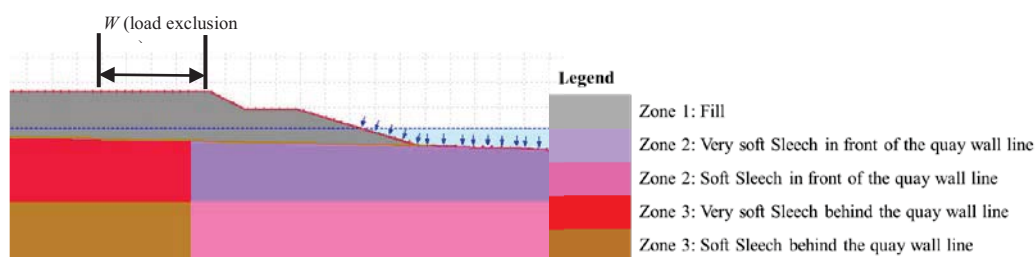


Fig. 1. Cross section of the working platform and the underlying ground layers at the end of the construction

The geotechnical design profile has been determined based on the available ground investigation data and the information gathered during the desk study. One of the most influential parameters on the overall stability of the fill slopes is the undrained shear strength of the Sleafch layer. To estimate the undrained shear strength, the results of in-situ tests (SPT, shear vane) and laboratory tests (Atterberg limits, triaxial UU) have been considered. In addition, a literature review was conducted during the desk study and the shear strength values reported in literature for the Estuarine deposits have also been considered (Crooks and Graham (1976) and Lehane et al. (2003)). Table 1 shows the model and characteristic parameters used for the design of the working platform. The different colours which are assigned to the Sleafch layer behind and in front of the quay wall line in Figure 1 is to address the different post consolidation shear strength of these layers throughout the construction stages of the embankment.

Table 1. Ground model and characteristic parameters

Stratum	Thickness (m)	Unit weight (kN/m ³)	Undrained shear strength (kPa)	Friction Angle (degrees)
Fill	5.5	18	-	38
Very Soft Sleafch	6	16	8	-
Soft Sleafch	9	17	8 at top 32 at bottom	-
Soft to Firm Sleafch	5	18	32 at top 44 at bottom	-
Glacial Till	3	21	100	-

The construction is going to be carried out in separate lifts with lag times between them to allow for consolidation of the soft ground. After the completion of the first lift, prefabricated vertical drains (PVDs) are installed within the fill layer to accelerate the consolidation. The increase of shear strength due to the consolidation after each lift has been considered in the deterministic approach and hence the design has been provided for each lift. The design has been performed by considering ultimate limit states and serviceability limit states described in BS8006-1.

The overall stability of the slope has been assessed by adopting the Morgenstern and Price (1965) method, using a commercially available limit equilibrium software Geostudio (2022.1). One main output of the overall stability assessment is the load restriction zone for different construction plants to ensure the stability of the fill slope. This zone is defined as an area behind the crest of the slope where the operation of a specific type of machine is not permitted. The suggested widths of the exclusion zone (W) (see Figure 1) based on the results of the deterministic assessment are summarised in Table 2 for three different types of machines operating on the final lift. In the next sections, the methodology of the reliability assessment of the overall stability has been discussed and the results are provided and compared with the outcomes of the deterministic analysis.

Table 2. Loads imposed on the platform and width of the load exclusion zone based on the deterministic analysis

Case	Machinery	Design load (kPa)	W based on deterministic analysis (m)
1	Piling rig	42	14
2	Excavator	60	15
3	Dump truck	32	10

3. Methodology of the Reliability Assessment

A preliminary step for the reliability assessment is to derive probability distributions of the influential soil parameters. Based on the results of the sensitivity checks performed at the design stage, it became evident that the overall stability would be mostly influenced by the shear strength of the first 15 m of the Sleafch (very soft and soft layers) and of the fill layer. Shear strength parameters of these layers have been considered as variables in the reliability assessment whereas all the other parameters (including the unit weight of the very soft and soft Sleafch) have been modelled as deterministic. The reliability assessment is considering the final lift of the fill and hence the increase of shear strength of the Sleafch due to the consolidation process has been considered for the determination of the parameters. PVDs are only incorporated in the zone behind the quay wall line due to environmental restrictions related to the dredging operations. Therefore, the undrained shear strength is assumed to increase at different rates in these two areas.

Table 3 summarises the variations in the shear strength parameters of the 3 most influential zones: zone 1 refers to the fill (working platform to be placed in the shallow water zone), zone 2 refers to the very soft and soft Sleafch in front of the quay wall line and zone 3 refers to the very soft and soft Sleafch behind the quay wall line as presented in Figure 1. Zone 1 is referring to the 'Fill' and zones 2 and 3 are referring to the 'Very Soft Sleafch' and 'Soft Sleafch' in Table 1. It may be noted that the mean undrained shear strength values have been calculated based on the mean value from the collected samples as well as the different rates of shear strength increase of the Sleafch in the zones behind and in front of the quay wall line. The rate of the shear strength increase has been considered according to the assumptions provided in the deterministic analysis and the uncertainty of the increase rates has not

been considered in this paper. The standard deviation of undrained shear strength of Sleaford in the two zones at the end of the construction period has been assumed to be the same. The resulting coefficients of variation (standard deviation/mean) of the strength parameters, which align closely with the values reported in literature, for example, Phoon and Kulhawy (1999), justify the choice of normal distributions for modelling their uncertainty. The friction angle of the fill layer has been selected based on available test results from potential quarry sources (particle size distribution, angularity, uniformity coefficient, etc.). Test results have indicated that the material extracted from these quarries are relatively uniform and hence the standard deviation of the fill is relatively low. For the deterministic design, characteristic values as presented in Table 2 and 3 have been used.

Table 3. Uncertainty in shear strength parameters for the reliability assessments

Zone	Undrained shear strength (kPa)			Effective friction angle (°)		
	Mean	Standard deviation	Characteristic value	Mean	Standard deviation	Characteristic value
1	-	-	-	39.6	1.0	38.0
2	16; $z^* \leq 6$ m	2.3	12; $z \leq 6$ m	-	-	-
	16+2.4×(z-6); $z > 6$ m		12+2.4×(z-6); $z > 6$ m	-	-	-
3	23; $z \leq 6$ m	2.3	19; $z \leq 6$ m	-	-	-
	23+2.4×(z-6); $z > 6$ m		19+2.4×(z-6); $z > 6$ m	-	-	-

*z refers to the depth below the natural ground level

To investigate the effects of variability of the strength parameters on the results of the slope stability, a reliability-based assessment using the first order reliability method (FORM) (Hasofer and Lind 1974) has been adapted. This method involves transforming the variables from their physical space to standard normal space and linearising the limit state surface at the most probable failure point, also called the design point. The distance from the origin to the design point is known as the reliability index (β) and is related to the probability of failure (P_f) by Equation (1), where Φ is the standard normal cumulative distribution function. In-house scripts have been developed for the FORM algorithm and for combining the algorithm with the limit equilibrium software.

$$\beta \approx -\Phi^{-1}(P_f) \quad (1)$$

4. Results of the Reliability Assessment

The width of the exclusion zones has been determined previously (see Table 2) based on the deterministic design method for the three different types of operation plant. Following this, a FORM assessment has been performed on the critical slip surface identified for each load case, with the results summarized in Table 4. Also shown in the table are the influence factors obtained for each uncertain variable in the assessments. The influence factor (\pm) of a variable is the component of the unit gradient vector of the limit state function at the design point in the standard normal space and indicates the sensitivity ($= \alpha^2$) of the variable on the probability of failure. The table shows that the reliability of the embankment slope is most sensitive to the undrained shear strength of zone 2 (H 79%), followed by the undrained shear strength of zone 3 (H 20%) and effective friction angle of zone 1 (H 1%).

Table 4. Summary of the results obtained using FORM

Case	W based on reliability analysis (m)	β (-)	Influence factors (-)			W based on deterministic analysis (m)
			zone 1	zone 2	zone 3	
1	14	5.63	0.104	0.845	0.524	14
	<u>10</u>	4.31	0.101	0.889	0.446	
	<i>9</i>	<i>4.10</i>	<i>0.100</i>	<i>0.904</i>	<i>0.416</i>	
2	15	5.45	0.118	0.859	0.499	15
	<u>12</u>	4.36	0.065	0.857	0.512	
	<i>11</i>	<i>4.05</i>	<i>0.060</i>	<i>0.867</i>	<i>0.495</i>	
3	10	4.75	0.095	0.921	0.378	10
	<u>7</u>	4.16	0.062	0.924	0.378	
	<i>6</i>	<i>3.78</i>	<i>0.074</i>	<i>0.954</i>	<i>0.292</i>	

Note: The critical value of W , based on the minimum acceptable β (1-year reference period and CC1) for each load case, has been underlined. Cases where β is below the minimum are shown in italics.

In each deterministic analysis, the value of W has been provided based on the BSEN1997-1 (2004) requirements for the ultimate limit state overall stability assessment of a slope. In the probabilistic assessment, the critical value of W has been identified based on the configuration which resulted in the required minimum value of

β according to Table C.3 of EN 1990-1 (2023). The structure is a temporary working platform and slope with a limited design life and during operation, would include construction activities undertaken by a specialist contractor. A reference period of 1 year has been chosen here for selecting the target value of β as it matches with the intended design life of the structure. For a consequence class 1, which indicates low level of consequences, the target value of β would be 4.2. For a consequence class 2 which indicates a slightly higher (normal) consequences, the target β would increase to 4.7. A comparison of the results reveals that the minimum required value of W obtained using the FORM assessments are smaller than the values of W obtained in the corresponding cases using the deterministic approach. The results of the FORM assessment provide confidence that the suggested construction method based on land-based operation meets the requirements of BS EN 1990-1 (2023).

To verify the results of the FORM assessment, a Monte Carlo simulation has also been performed for load case 1. A limitation with the Monte Carlo method is the requirement for a very large number of realisations in order to achieve a high confidence level in the estimation of P_f . Therefore, the Monte Carlo simulation has been performed using a surrogate model based on the Polynomial Chaos Expansion (PCE). The minimum number of realisations required to build the surrogate was selected based on an acceptable level of the predicted residual sum of squares. For load case 1, a PCE built on 20 realisations was further used to simulate 10 million realisations and perform the probabilistic assessment using the Monte Carlo method. A histogram of factor of safety (FS) thus obtained is shown in Figure 2. The results show that for a W of 10 m for the load case 1, the P_f calculated using the Monte Carlo method, calculated as the ratio of the number of realisations that resulted in FS less than 1 to the total number of realisations, is 1.1×10^{-5} . This corresponds to a β of 4.24 (see Eq. (1)), which confirms the value of β obtained using the FORM assessment for load case 1 with the same configuration.

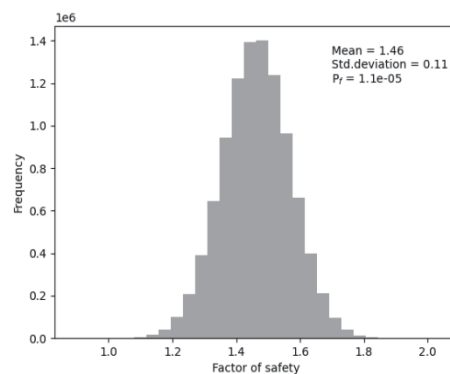


Fig. 2. Histogram of FS obtained using Monte Carlo simulation for load case 1

5. Conclusions

The study involves the reliability assessment of the overall stability of a temporary working platform located in shallow water to support the construction of a quay wall in Belfast. The temporary working platform will be constructed on a 20 m thick layer of soft Estuarine deposits. A land-based construction approach has been considered which includes the placement of the embankment platform fill in staged lifts with a lag time between them to enable strength gain of the underlying soft soil during consolidation. The land-based approach has led to significant cost savings in comparison with marine based operation for the construction of the quay wall. The overall stability of the embankment platform and slope has been assessed by adopting both deterministic and probabilistic approaches. The results indicate that adopting the land-based construction approach is leading to a design solution which meets the requirements of Eurocode in terms of acceptable levels of reliability and probability of failure. Further research would be required regarding the appropriate selection of the consequence class for a temporary structure as clear guidance has not been provided in Eurocode. A monitoring stage has been designed for the project which includes measurement of settlements, excess pore water pressures and post consolidation shear strength of the Sleaford for the verification of the design based on field measurements.

References

- Crooks, J. H. A. and J. Graham (1976). Geotechnical properties of the Belfast estuarine deposits. *Géotechnique* 26(2), 293–315.
- BS 8006-1 (2010). Code of practice for strengthened/reinforced soils and other fills. *The British Standards Institution*.
- BS EN 1990-1 (2023). Eurocode: Basis of structural and geotechnical design - Part 1: New structures. *The British Standards Institution*.
- EN 1997-1 (2004). Eurocode7: Geotechnical design - Part 1: General rules. *The British Standards Institution*.

- Hasofer, A. M. and N. C. Lind (1974). An exact and invariant reliability format. *Journal of Engineering Mechanics* 100, 111–121.
- Lehane, B. M., R. J. Jardine, and B. A. McCabe (2003). Pile group tension cyclic loading: Field test programme at Kinnegar N.Ireland, *HSE Research Report*, RR101.
- Morgenstern, N. R. and V. E. Price (1965). The analysis of the stability of general slip surfaces. *Géotechnique* 15(1), 79–93.
- Phoon, K.K. and F. H. Kulhawy (1999). Characterization of geotechnical variability. *Canadian Geotechnical Journal*, 36(4), 612–624.