Reliability Analysis of a Hydraulic Fill Slope with Respect to Liquefaction and Breaching

T. Schweckendiek
Deltares, unit Geo-engineering & TU Delft, Delft, Netherlands

G.A. van den Ham & M.B. de Groot
Deltares, unit Geo-engineering, Delft, Netherlands

J.G. de Gijt & H. Brassinga
Public Works Rotterdam, Rotterdam, Netherlands

P. Hudig
Gate Terminal B.V.

ABSTRACT: A recently reclaimed site in the Port of Rotterdam will serve as location and foundation of an LNG terminal. LNG (Liquefied Natural Gas) is recognized as hazardous material and underlies strict safety requirements. As part of the safety assessment of the entire installation, a specific analysis had to be carried out concerning the geotechnical aspects. The paper describes the probabilistic approach that was chosen to verify the required level of safety of the hydraulic sand fill regarding (static) liquefaction, slope failure and breaching processes. Several reliability analyses using the respective physical process models were carried out and the results combined using a fault tree or scenario approach, leading to upper bounds of the failure probability.

1 INTRODUCTION

1.1 *Project outline*

The paper describes an approach for a geotechnical reliability analysis problem in a real life project in 2007. For an LNG terminal to be built in the Port of Rotterdam, hydraulic sand filling was used to extend an existing artificial terrain in order to create space for 4 large LNG tanks (Fig. 1).

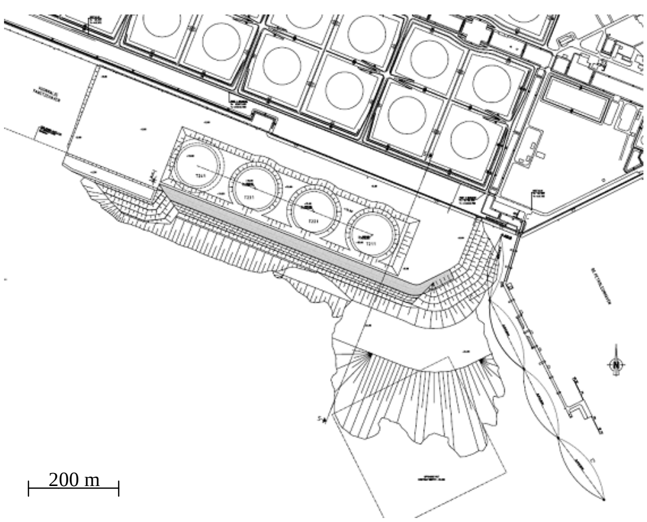

Figure 1: Overview LNG terminal

The original design contained slopes with angles of 1:2.5, protected mainly against erosion and wave action by steel slag dams. Initially, it was thought that compaction would not be required until a rough analysis of the liquefaction potential cast this assumption into serious doubt. Subsequent more thorough analyses led to several design modifications, the most important of which were to use a shallower slope angle of 1:3 and to compact the entire slope itself up to the height of the tanks to be built by means of vibro-flotation. The final representative cross section in Figure 2.

Note that due to the construction process and the last-minute design amendments, some portions of the hydraulic fill could not be compacted and remained in a relatively loose state. These areas in combination with the still relatively steep slope caused some uncertainty about the chance of the occurrence of a liquefaction flow slide with subsequent damage to the foundation of the LNG-tanks. This uncertainty was the focus of the analysis described in this paper.

1.2 *Design Requirements*

For the LNG installation, as for other activities with hazardous materials, the safety requirements were formulated in terms of risk respectively an acceptable probability of failure, failure being defined as some the occurrence of an unwanted event or accident. The safety criterion for the geotechnical aspects treated in this paper was derived from the overall safety requirement, being: "*The probability of a slope failure, including liquefaction and breaching, affecting the foundation safety of the LNG-tanks must not exceed $P_{f,adm}=10^{-6}$ in the planned life time of the structure (50 years)*". Note that this criterion involves several potential failure mechanisms.

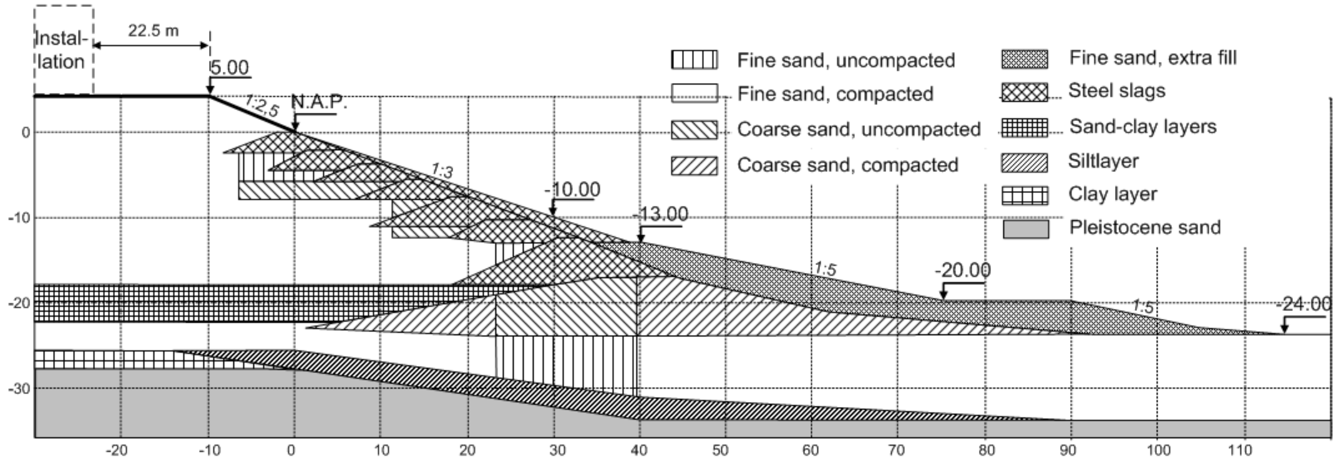

Figure 2: Representative cross-section

1.3 *Probabilistic Approach*

For the evaluation of the probability of failure stated in the previous section, the complex failure mechanism was split into basically three sub-mechanisms, which were tractable for structural reliability analysis.

A choice for a dominant failure scenario did not seem appropriate, mainly due to the fact that multiple failure mechanisms were involved. To this end several failure scenarios were defined in order to ensure to not miss significant contributions.

The results of the sub-mechanisms and the scenarios were combined by means of fault tree analysis in order to obtain the (upper bound of the) overall probability of failure, which was then compared to the acceptability criterion.

Section 2 treats the physical process models applied in the analysis, whilst section 3 focuses on the reliability analysis aspects.

2 APPLIED PHYSICAL PROCESS MODELS

2.1 *Liquefaction flow slide and subsequent breaching*

Under certain circumstances, loose, saturated sand elements in a slope may be sensitive to liquefaction or, more precisely formulated, may be in a ‘meta-stable’ state, which means that they will liquefy and loose their strength under any quick loading if they are free to undergo shear deformation. In case most adjacent sand elements in a slope have a much more stable state, no liquefaction will occur because these more stable elements will prevent the shear deformation of their meta-stable neighbors. However, in a slope with sufficient large pockets of meta-stable elements a liquefaction flow slide may occur. The conditions for meta-stability mainly concern the soil state in terms of density and stresses which will be discussed in section 2.2. Whether the pockets of meta-stable elements are sufficiently large to enable a liquefaction flow slide will be studied by a traditional slope stability analysis in which the originally meta-stable elements are supposed to have liquefied (section 2.3). The final question is whether a liquefaction flow slide will result in failure of the foundation of the tanks. In case of a relatively shallow flow slide, this will only be the case if a breach in the unprotected sand created by the flow slide will progress over a sufficient large distance. The breaching process will be discussed in section 2.4.

2.2 *Meta-stability or sensitivity to liquefaction*

The model that was used in this study for the undrained behavior of saturated (loose) sand is based on the theory presented in Stoutjesdijk et al (1998), which is also the basic theory used in the software SLIQ2D, mainly used by GeoDelft in the Netherlands during the last two decades.

Whilst SLIQ2D only uses an instability or meta-stability criterion based on material parameters and the soil state (porosity and stresses) according to Molenkamp (1989), the approach in this study uses more information from the modeled undrained behavious respectively the stress path.

For a given in-situ stress point, the undrained stress path is derived as a function of relative density from extensive laboratory tests. This path allows us two extract two types of information that helps us to judge the liquefaction potential and the residual strength after liquefaction:

1 whether the in-situ density is higher or lower than the wet critical density (WCD, see Figure 5).
  If $I_D < WCD$, the undrained stress path exhibits a decreasing deviatoric or shear stress. This is the most important necessary, however not sufficient, condition for meta-stability and thus for the occurrence of instability and static liquefaction.
2 the maximum generated excess pore pressure respectively the minimum isotropic effective stress p'_{min}, which can be used to estimate the (“worst case”) strength reduction due to liquefaction.

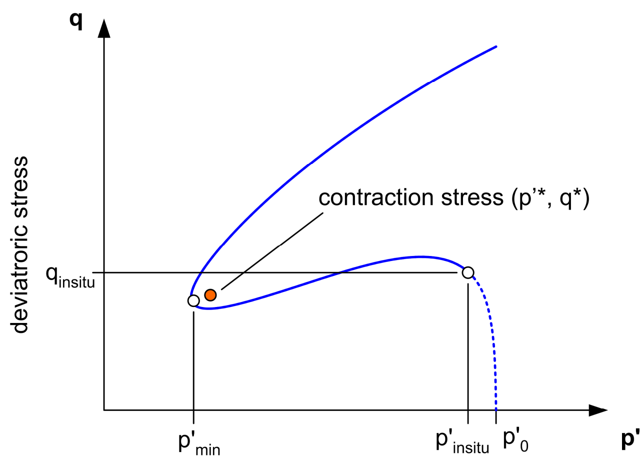

Figure 4: Undrained stress path of loose sand

Both definitions are definitely conservative respectively will lead to upper limits of failure probabilities. We will come back to this question in section 3.

2.3 *Slope stability*

The slope stability was treated by conventional Bishop slip circle analyses using the MStab software by GeoDelft (since 2008 Deltares). Two non-standard features had to be included:

1 The slope stability analysis had to reflect the situation, given that liquefaction occurred in the liquefaction-sensitive parts of the slope. In the deterministic setup, the reduction in isotropic effective stress was used as measure for the reduction of shear capacity, expressed in form of a reduced friction angle:

$$\tan\varphi'_{red} = \frac{p'_{min}}{p'_{insitu}} \tan\varphi' \tag{1}$$

2 The Rotterdam area is not typically earthquake-prone, however, due to the low required failure probability, also very low occurrence frequency seismic loads were considered. An option in MStab to account for vertical and horizontal peak accelerations in the slope stability analysis was applied (Delft GeoSystems 2006).

2.4 *Breaching*

If slope instability occurs, a liquefaction flow slide will start, which means that the instable soil mass starts to slide over a shear surface. It will continue to do so until it finds a new equilibrium. The flow process will in this case probably take not more than several seconds to a minute, as follows from calculations in which inertia is incorporated. That time is not long enough to cause significant reduction of the excess pore pressure in the liquefied sand pockets. Consequently, the shape of the new profile

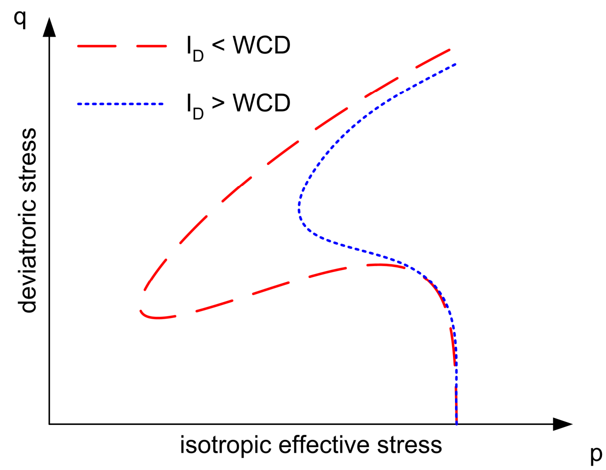

Figure 5: Definition Wet Critical Density (WCD)

can be estimated by using Bishop calculations and the new slope profile is characterized by a relatively steep slope just above the soil mass, that flowed down. Its location can be characterized by L_1 as defined in Figure 3.

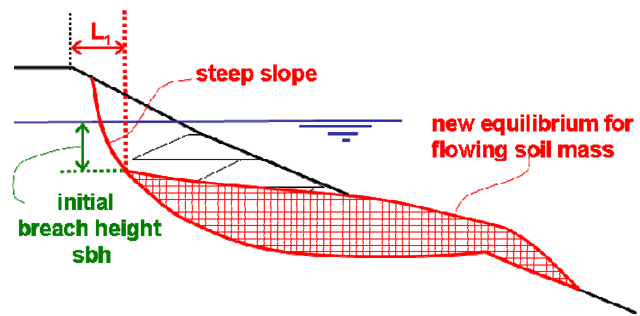

Figure 3: Equilibrium profile after flow slide

This steep slope consists of sand and is not likely to be covered by slags or other parts of any slope protection. Part of the steep slope is situated under water, as indicated in Figure 3. This part of the slope may start breaching.

Breaching is a process in which a steep under water slope, "breach", remains temporary stable under the influence of dilation induced negative pore pressures, and gradually moves backwards while sand grains fall down from the surface and mix with water to create an eroding, turbulent sand water mixture. The process stops when the height of the under water part of the breach is reduced to zero. The resulting profile is sketched in Figure 6.

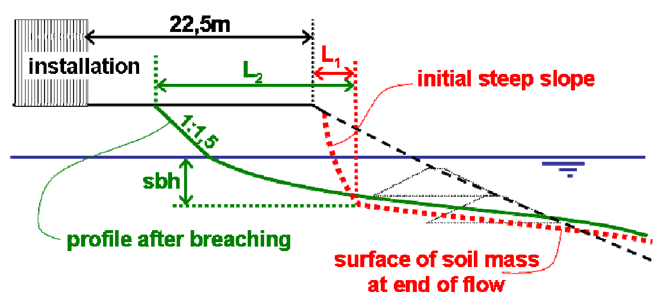

Figure 6: Equilibrium profile after breaching process

The breaching process is described by Mastbergen & van den Berg (2003) and can be modelled by the computer code HMBREACH. Given grain size distribution, relative density and initial height of the under water part of the steep slope, *sbh*, the model calculates the change in this height as a function of the horizontal distance, from which the total distance of breach progress L_2 (Fig. 6) can be derived. The slope of the part above the water is determined by the common shearing process and can be assumed to equal 1:1.5. Now the length (L_2-L_1) of the damaged area follows.

It is assumed, supported by indicative calculations, that no significant damage to the foundation of the tanks will occur as long as (L_2-L_1) < 22.5m, which is the distance between the foundation and the slope crest.

3 RELIABILITY ANALYSIS

The previous section gave a concise overview of the concepts and methods used for deterministic evaluation of the sub-mechanisms playing a role in the present safety assessment problem. In this section we will discuss how an assessment of the criterion stated in section 1.2 was made in a probabilistic manner.

First of all, we are dealing with the verification of a design criterion. That implies that it is sufficient to show that the upper bound of the estimate of the failure probability $P_{f,sup}$ fulfills the requirement:

$$P_{f,\sup} < P_{f,adm} \tag{2}$$

Thus, we can start with rough, conservative (upper bound) approaches and apply refinements, if necessary, as illustrated in Figure 7. Such refinements can either concern the probabilistic analysis itself (e.g. treatment of correlations) or more realistic physical process models.

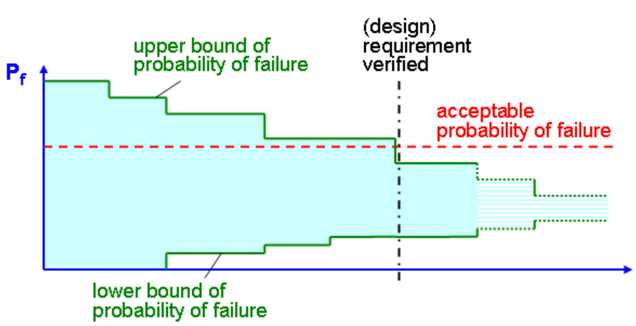

Figure 7: Upper and lower bounds of P_f vs. the design criterion

Such an approach was applied in the project, though for sake of readability in the following only the analysis that led to the successful outcome is described.

3.1 *System definition*

As described in 2.1, the principal contemplated failure mode is a sequence of three mechanisms.

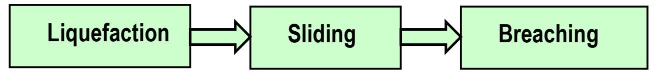

Figure 8: Sequence of mechanisms in Failure mode

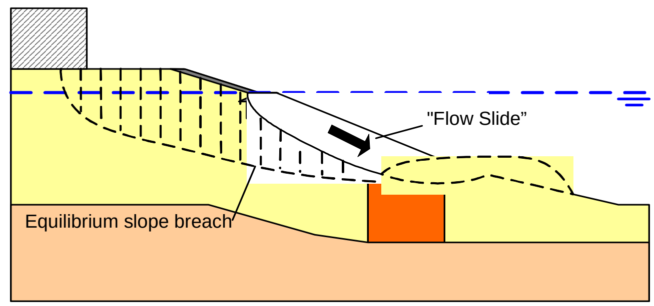

Figure 9: Sequence of mechanisms leading to top event

To reiterate the sequence shortly, liquefaction of substantial, uncompacted volumes in the slope part of the fill may cause a flow slide respectively slope failure. The residual profile is common steep in the upper part and a breaching process may be initiated that could endanger the foundations of the installation in question.

For the reliability analysis, this sequence is modeled by a parallel “sub-system” in a fault tree, consequently combined by and AND-gate (Fig. 10).

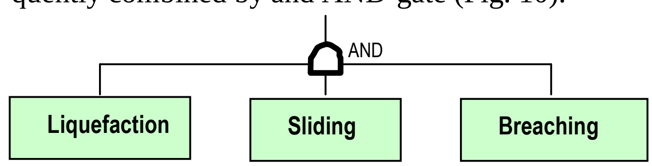

Figure 10: Sequence of mechanisms in Failure mode

Given the large uncertainties, it is not trivial to determine a dominant or representative scenario as we are used to do in deterministic approaches. For different combinations of parameters or properties, in some cases liquefaction and slope failure in the upper part may lead to the worst consequences, in other cases failures in the lower part or deeper sliding surfaces. One way to circumvent the problem of choosing one scenario, is the definition of several scenarios.

Two examples of such scenarios are presented schematically in Figure 11. The main difference in this discrete distinction of possibilities is the assumption of which of the uncompacted volumes liquefy and how many at a time, with all the due consequences.

All the defined scenarios are integrated in a fault tree (Fig. 12). For sake simplicity, the “conservative”, i.e. upper bound assumption of independence (actually even mutually exclusivity) is made (see 3.6).

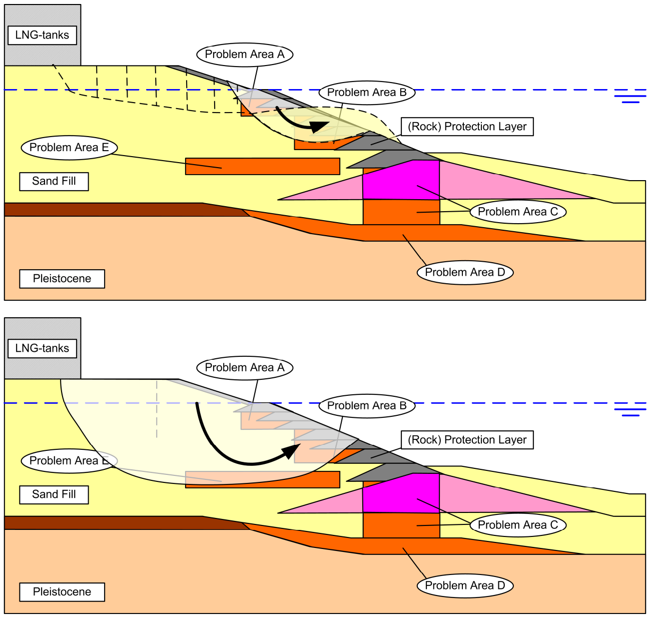

Figure 11: Schematic representation of two scenarios

3.2 *Parameters and uncertainties*

The in-situ relative densities of the hydraulic fill were determined by means of the empirical CPT correlation function of Baldi e.a. (1982) which correlates the density index I_D to the cone penetration value q_c as a function of the vertical effective stress. A total of over 50 CPT's were available. Accounting for both spatial variability and uncertainty of the correlation function the expected value of I_D was found to be 39% with a standard deviation of 10%. These values concern the average of I_D over a potential liquefiable area or failure surface.

By means of several drained (CD) and dry triaxial tests on a number of representative (disturbed) samples, taken from the hydraulic fill, the parameters for the constitutive model (see 2.2) were determined. Influence of soil state was assessed by perfoming the tests at different stress conditions and porosities. Statistical analysis of the test results and considerations on spatial variability, lead to probability distribution functions of the important material model parameters for further use in the probabilistic analysis.

In order to check the calibrated parameter set, a number of undrained (CU) triaxial tests was executed on the same samples and simulated with the model. Measurements and prediction fitted reasonably well (Fig. 13)).

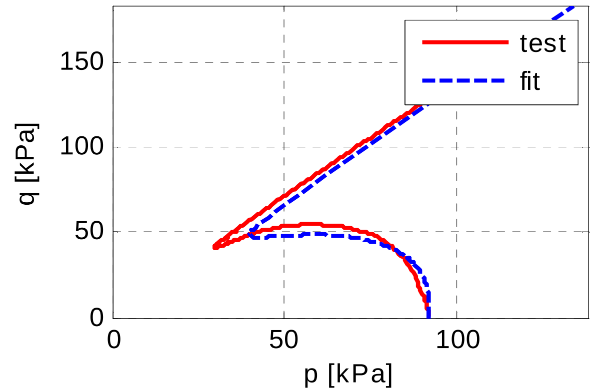

Figure 13: Comparison stress path (CU) between test and calibrated model

3.3 *Meta-stability or sensitivity to liquefaction*

The probability of meta-stability or the sensitivity to liquefaction P_{liq} of each area with non-compacted sand was evaluated by determining the probability of the in-situ sand being in a state below the WCD (see 2.2), given a representative stress point in the area and the uncertainties in the material properties:

$$P_{liq} = \int_{I_D<WCD} f(\underline{x})\, d\underline{x} \tag{3}$$

with $\underline{x}$ being a vector containing all random variables. P_{liq} was determined by means of Monte-Carlo analysis. Per scenario, $n=10^5$ realizations of the state, material and model parameters were produced and

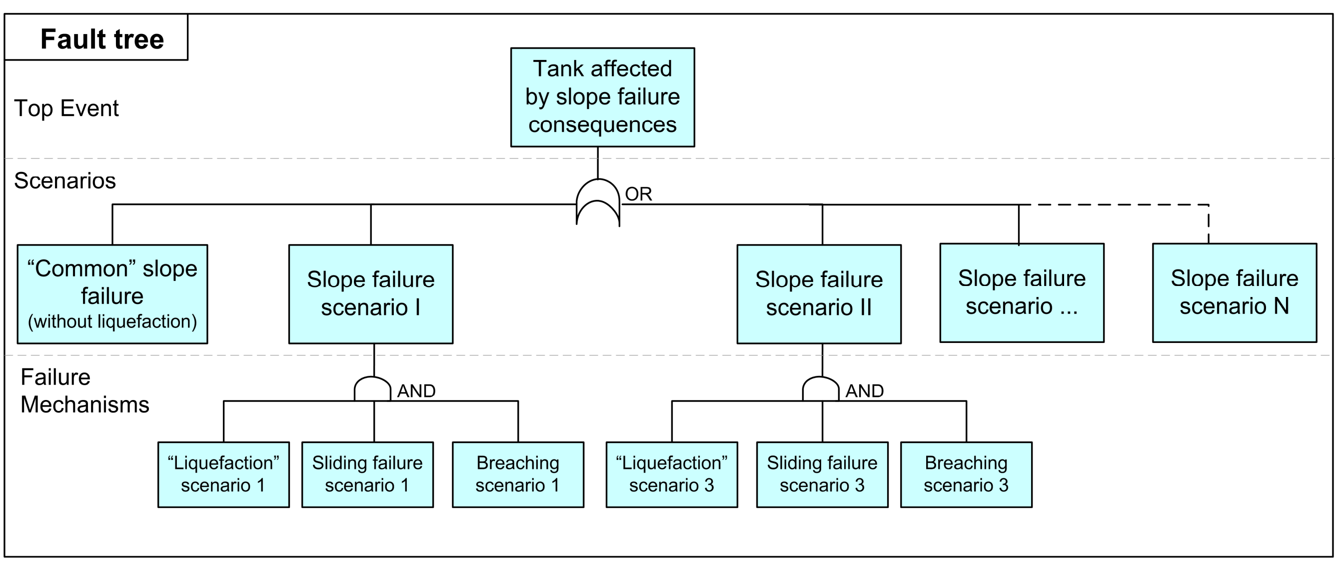

Figure 12: Fault tree

propagated through the model (undrained stress path, Fig. 4). Consequently the estimator for P_{liq} is:

$$\hat{P}_{liq} = \sum_{i=1}^{n} I_{I_D<WCD}\left(\underline{x}_i\right) \tag{4}$$

where $\underline{x}_i$ is the i^{th} realization of $\underline{x}$ and $I_C(x)$ is the indicator function for condition C.

Considering the definition of WCD, being a necessary not sufficient condition for static liquefaction, this is clearly a conservative approach leading to an upper bound estimate of the probability of liquefaction. In fact, the results in section 4 show that the estimate based on this method usually lead to very high probabilities that intuitively do not reflect the judgment of most experts. For the assessment of the probability of sensitivity to liquefaction, it is definitely desirable to use an approach that includes also the “distance” from instability or a critical-state model. This was not realized in the course of this project, but is one of our goals for the future.

It is also noted that seismic action was neglected in this step. Due to the very low intensity the contribution was found to be insignificant.

3.4 *Slope stability, given liquefaction*

The second step respectively sub-mechanism in the contemplated chain of events is slope failure, given liquefaction has occurred in one or more of the problematic uncompacted zones. A total of 6 critical failure modes could be identified.

The slope reliability analysis is carried out using the reliability module of MStab, which is essentially FORM applied to a Bishop slip circle analysis using average properties of the soil shear resistance properties as the main basic random variables, thus with implicit treatment of averaging effects in the probability distributions for the shear resistance (see JCSS 2001).

As mentioned earlier, seismic loading was not considered in the initiation of liquefaction, i.e. the implicit assumption is that a trigger is always present with high probability. However, seismic action was taken into account in the slope stability analysis. For the considered area, two values of peak acceleration

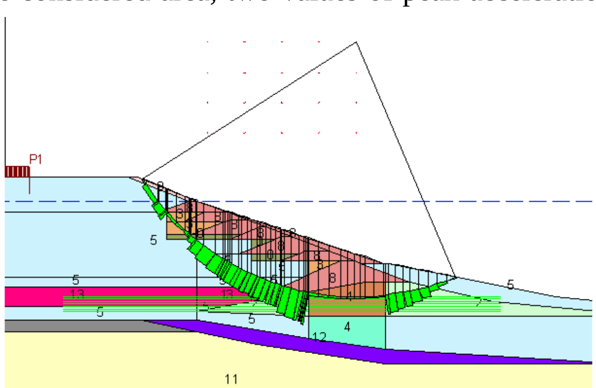

Figure 14: MStab reliability module

a_{max} are given for the return periods of 10,000 years and 475 years (see Table 1). In order not to use the heaviest condition as deterministic value, a Generalized Extreme Value (GEV) distribution corresponding to the given quantiles was used to integrate the seismic loads in a probabilistic manner.

Table 1 Peak acceleration values

a_{max}	$P\{a_{max} > \acute{a}_{max}\}$	$P\{a_{max} > \acute{a}_{max}\}$
[m/s^2]	[1/year]	[1/50year]
0.20	1/475	1-(1-1/475)^50 = 0.1
0.40	1/10000	1-(1-1/10000)^50 = 0.005

The resulting GEV-distribution is shown in Figure 15.

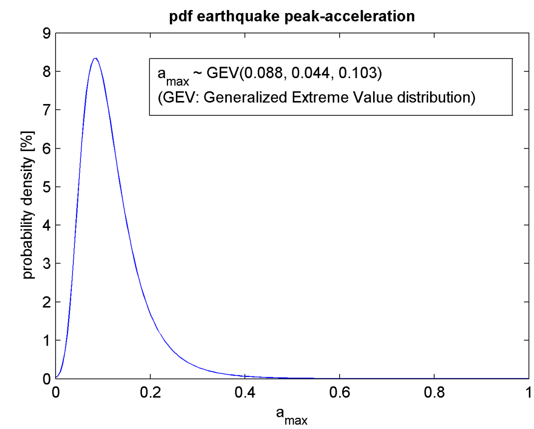

Figure 15: GEV distribution of a_{max}

Since the used software did not allow us to include the uncertainty in a_{max} in the Bishop-FORM analysis, several of these form analyses were carried out for a set of deterministic values of the peak acceleration. Subsequently, the results in terms of the reliability index β, conditional on a_{max}, can be integrated numerically to solve the following integral:

$$P_f = \int_0^{+\infty} \Phi(-\beta(a_{\max}))\, f_{a_{\max}}(a_{\max})\, da_{\max} \tag{5}$$

This is practically done by an external FORM-loop respectively design point search, for details refer to (Delft GeoSystems 2006).

3.5 *Breaching, given slope failure*

By carrying out an uncertainty analysis on the initial breach height *sbh* and the value of L_1 , based on the uncertainties in the strength of liquefied sand (ϕ_{red}’) and the strength of the non-liquefied and (critical state) probability distribution functions for these variables were established. The breach length L_2 proved to be very insensitive to L_1, reason to give it a conservative deterministic value: L_1 = 5m (again simplified upper bound approach). The uncertainty in *sbh*, however, is expressed as a lognormal distribution with an expected value of 1m and a standard deviation of 1m.

The results of a large series of HMBREACH calculations could be approximated by the following equation (response surface):

$$L_2 = 6m + sbh \cdot (1.83)^{C2} \cdot \left(\frac{sbh}{1m} \right)^{C1 \cdot C2} \quad (6)$$

where *C1* and *C2* are model parameters with log-normal distributions, expected values 1 and standard deviations 0.1 and 0.3, respectively.

A reliability analysis on this response surface of the breach model resulted in:

$P\{(L_2-L_1)>22.5m|\text{slope instability}) = 1.3\ 10^{-7}$

and an expected value of $E(L_2) = 7.8\ m$ and a standard deviation $\sigma(L_2) = 3.7\ m$.

It should be noticed that the applied models for the breaching process, given slope instability, are very rough. Even conservative assumptions, however, make clear that no large damage is to be expected here in the unlikely cased that slope instability occurs. This is due to the shallow location of the uncompacted areas. In other cases of liquefaction slope failures, the length L_2-L_1 of the damaged area may reach values of up to 100m or even more, as experience shows. Research in the field of the breaching process and the interaction between liquefaction and breaching is needed to improve the models and develop a practical tool to predict the length L_2-L_1 of the damaged area.

3.6 *Total failure probability*

As mentioned earlier, but emphasized again at this point, the results presented here in terms of the failure probability concern an upper bound. By definition, the value of this probability is expected to be lower. Various assumptions have led to a value "on the safe side". These assumptions can be roughly classified in two categories:

1. Assumptions in probabilistic approach:
   a. The soil properties in the constitutive models are essentially independent and therefore treated as such.
   b. For combining the scenarios, it is assumed that they are mutually exclusive, thus the total probability is the sum of the probabilities of scenarios *i (serial system)*:

$$P_f = \sum P_{f,i} \quad (7)$$

   c. The combination of the sub-mechanism probabilities concerns a parallel system. Here the worst case is total dependence between the sub-mechanisms. This assumptions is probably not even unreasonable since in all mechanisms the same soil properties play a role. Therefore, the maximum value of the sub-mechanism probabilities is used as the upper bound for the scenario probability:

$$P_{f,i} = \max\{P_{f,i,j}\} \quad (8)$$

Consequently the top event probability is determined by:

$$P_f = \sum_{i=1}^{n} P_{f,i} = \sum_{i=1}^{n} \max_{j=1}^{m}\{P_{f,i,j}\} \quad (9)$$

for *n* scenarios and *m* sub-mechanisms.

2. Assumptions in the physical-process modeling:
   a. As mentioned in 3.3, the probability of liquefaction is actually the probability of the material being liquefiable. More conditions in terms of stress state etc. have to be fulfilled for liquefaction to occur.
   b. In the slope stability analysis, the theoretical minimum of the shear strength according to the material model is assigned to the zones that are assumed to be liquefied. It is likely that not the entire affected volumes undergo the total strength reduction and that excess pore pressures diminish, i.e. that the shear strength is recovered at least partially.

At the same time, the assumptions made, indicate where there is certainly significant potential for refinements in the applied method. More sophisticated mechanical and constitutive models are in principle available for coupled analysis in academia, but not yet easily applicable in consultancy work. There is a challenge for the applied sciences community to further develop these methods and tools closer to application in practical problems

4 RESULTS

For the project itself, it was shown that some design amendments were necessary, such as the compaction of mainly the slope part pf the hydraulic fill and a slightly shallower slope than initially planned in order to fulfill the strict safety requirement. With this amended design it was shown that the total probability of failure (upper bound, see previous section) was in the order of Pf,sup = 10-7. But rather than presenting more figures, the type of results that can be produced with such an analysis are illustrated in this section:

- The probability of the top event in the fault tree, in this case the foundations of the installation affected by slope failure, possibly induced by liquefaction and breaching, can be used in higher level risk analyses and reliability analyses of the entire installation. The probabilistic approach therefore provides a comparability with other elements of the system that cannot be achieved otherwise by the classical deterministic methods.

- The fault tree contains probabilities on (sub-) mechanism level. That enables the identification of the most relevant mechanisms and scenarios. This information is extremely useful for optimization of the design.
- The reliability analyses on (sub-) mechanism level also produce information on the relative importance of the variables involved (e.g. FORM gives influence coefficients α_i). Some of these properties can either be influenced by changes of the design or by acquiring more information and thereby reducing (epistemic) uncertainty, e.g. additional soil investigation.

5 CONCLUSIONS

The work on this paper has lead us to formulate the following three main conclusions:

Firstly, the paper demonstrates the applicability of reliability analysis for a rather complex geotechnical problem in a real world design problem. In the course of the design verification, the upper bound of the failure probability is lowered step-wise by refinements of either physical process or probabilistic models until it is shown that the design fulfills the rather strict requirements.

Secondly, it should be emphasized that such a decomposition of the analyzed failure processes can hardly be done with deterministic approaches. The common safety value, be it a factor, margin or something else, would be very difficult to compose out of the results of the evaluation of the sub-mechanisms. Once again, comparability is one of the major advantages using probabilistic approaches.

Finally, of course, a probabilistic approach does not compensate for deficiencies in physical process-based models, it merely provides a consistent manner to deal with the uncertainties. In the illustrated case, the sometimes quite rough upper bound approaches led to a satisfactory answer, namely an acceptance of the design by verifying the required requirements. On the other hand, we are convinced that the use of upper bounds led to a rather conservative assessment. However, carrying out the indicated potential refinements is not a trivial task with the currently available methods. Especially for the initiation of liquefaction, the currently used models are unsatisfactory. Either they are of empirical nature and based on a limited number of (indirect and interpreted) observations, or they combine several physical-process based models with rather restrictive assumptions. There is clearly a need for better in-depth understanding of the physical processes and their interaction leading to improved models.

6 REFERENCES

JCSS (Joint Committee of Structural Safety) 2001. Probabilistic Model Code, Part 3.07 – Soil Properties (last update 08/2006). ISBN 978-3-909386-79-6.

Lindenberg, J. & Koning, H.L. 1981. Critical density of sand. *Geotechnique* 31(2): 231-245.

Lunne, T. & Christoffersen, H.P. 1983. Interpretation of cone penetrometer data for offshore sands. In *Proceedings of the Offshore Technology conference,* Paper no. 4464. Richardson, Texas.

Mastbergen, D.R. & Van den Berg, J.H. 2003. Breaching in fine sands and the generation of sustained turbidity currents in submarine canyons. In *Sedimentology* 50: 635-637.

Molenkamp, F. 1989. Liquefaction as an instability. In Proceedings *Int. Conf. on Soil Mechanics and Foundation Engineering (ICSMFE):* 157-163.

Delft GeoSystems 2006. MStab 9.9, User Manual, Delft GeoSystems, Delft.

Olson, S.M. & Stark, T.D. 2003. Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments. *In Journal of Geotechnical and Geoenvironmental Engineering 129 (8):* 727-737. ASCE.

Sladen, J.A., D'Hollander, R.D. & Krahn, J. 1985. The liquefaction of sands, a collapse surface approach. In *Canadian Geotechnical Journal* 22: 564-578.

Stoutjesdijk, T.P., De Groot, M.B. & Lindenberg, J. 1998. Flow slide prediction method: influence of slope geometry. In *Canadian Geotechnical Journal* 35: 34-54.