

Abstract

At the entrance of the Yangtzehaven in the port of Rotterdam a new reclamation has been carried out. The design of this reclamation was challenging due to the requirement of sufficient safety against liquefaction and subsequent slope instability, also during the possible occurrence of an earthquake. Vibroflotation has been applied to reduce liquefaction risk. Some sand pockets, however, could not be compacted. A sophisticated probabilistic analysis made clear that the realised reclamation meets the safety requirements.

Geotechnical aspects

of a land reclamation in the

Yangtze Haven in Rotterdam

Introduction

The site is situated at the entrance of the Yangtzehaven, see figure 1. The length and width are 800 m and 200 m respectively. In figure 1b the location is shown in more detail. The height of the slope is 30 m. The reclamation of the site has been carried out by Boskalis BV. Activities started in August 2006, and finished at the end of 2007. The reclamation was done by means of hydraulic filling. The slope of the reclaimed site has a height of 30 m with a steep angle of 1:2.6, determined by nautical requirements. For stability reasons the slope has been built up by a sequence of bunds consisting of steel slags. Large parts of the filling have been compacted by means of vibroflotation to meet the

safety requirements.

This article presents an overview over the design, construction, and results. Future articles will present the various aspects in depth.

Soil conditions

Originally, the location was part of the North Sea with a bottom level of approximately NAP – 8 m. NAP is the reference level, approximately equal to the mean sea level. After 1960 the Maasvlakte has been reclaimed, until a level of NAP + 5 m. At the project location, no sand filling took place. After completion of the Maasvlakte in the south east corner of the Yangtzehaven an area was dredged to a level deeper than NAP – 50 m, to serve as an anchorage berth for offshore equipment.

Furthermore, dredging operations were carried out for equipment testing. Part of the dredged material returned in the pits due to overflow from the hoppers, and deposited in the deepest areas. This resulted in an unpredictable and rather muddy soil profile.

The traces of this history are still visible in the results of the soil investigations. First, below the bottom of the Yangtzehaven a mud layer with a maximum thickness of 2 m was found. Underneath, a loosely packed layer of fine sand with a maximum thickness of 10 m is present. Further, at some locations a clay layer is present with a thickness of 1 up to 3 m. The top level of the firm Pleistocene sand varies from NAP – 20 m to NAP – 34 m and locally even more deep.

Boundary conditions

The following boundary conditions were crucial for the design of the reclamation.

- Nautical requirements

The site is located along the Yangtzehaven, which will be the entrance to the future Maasvlakte 2. To guarantee an undisturbed ship passage, Rotterdam Port Authority required a slope inclination of 1:2.6.

- Safety requirement

The future user required a failure probability for constructions on the fill due to slope instability and subsequent damage to be smaller than 10^{-6} during a 50 years design life.

- Earthquake loading

According to Eurocode 8 and data from the Royal Dutch Meteorological Institute KNMI (De Crook, 1996) the possibility of an earthquake had to be considered. Only a small earthquake is possible, with a maximum horizontal acceleration of 0.04 g and an occurrence of once per 10,000 years.

Reclamation design Failure mechanisms and analysis

If no liquefaction is taken into account in the stability analysis, the Bishop stability factor is larger than 1.6. This changes as soon as liquefaction is also taken into consideration. From tests performed by Deltares and experience in the Port of Rotterdam (van Kuijk, 1989; van Rijt, 1991; de Gijt et al, 1999) it became clear that the combination of the steep slope and the used sand can initiate liquefaction inside the lower part of the slope, which may lead to instability.

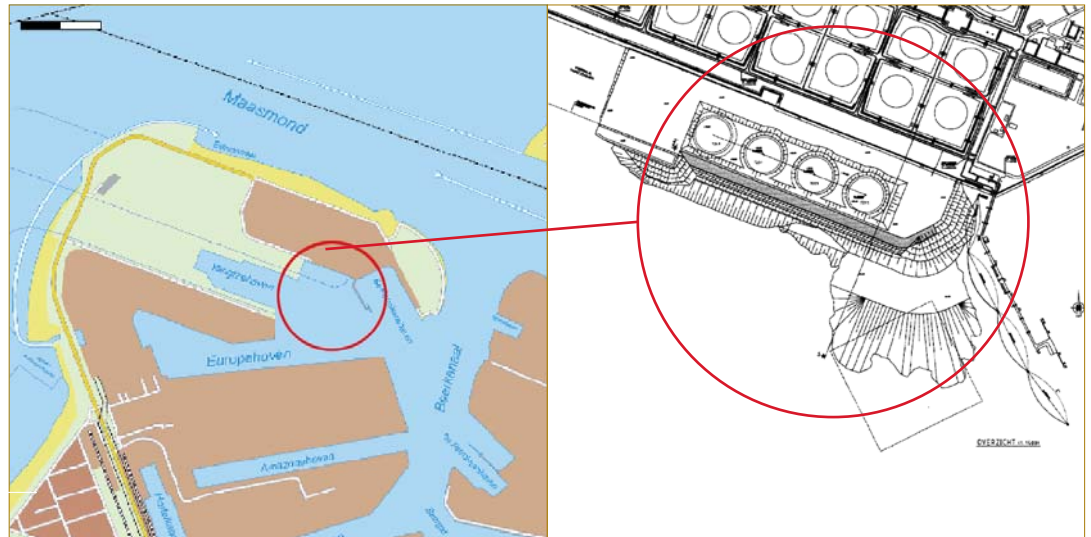


Figure 1 Site location. Right: top view of the reclamation.

Last, when an instability has occurred, breaching may cause further slope deterioration (Mastbergen, 2006).

The Deltares methodology (Stoutjesdijk et al., 1998; de Groot et al., 2007) has been applied to analyse liquefaction. In this method the 'metastable region is computed, in which the combination of the stresses and the relative density is such that liquefaction may occur during undrained loading'. Major liquefaction risk parameters are the slope geometry, the used material, sand grading, the relative sand density, and the stress state. The earthquake has been accounted for as an additional load in the slope stability calculations

and was treated in a probabilistic manner by considering the full range of accelerations and failure probability distributions. This approach is similar to the treatment of extreme water levels in dike safety calculations (GeoDelft, 2007). An earthquake may also serve as trigger.

Stability has been analysed with the conventional Bishop method. It should be remarked that liquefaction analysis with finite elements requires programs in which not only equilibrium, but also the groundwater mass balance is discretized. Next, material models must be able to simulate densification. In terms of elastoplasticity this means that the model must be able to produce plasticity within the yield contour.

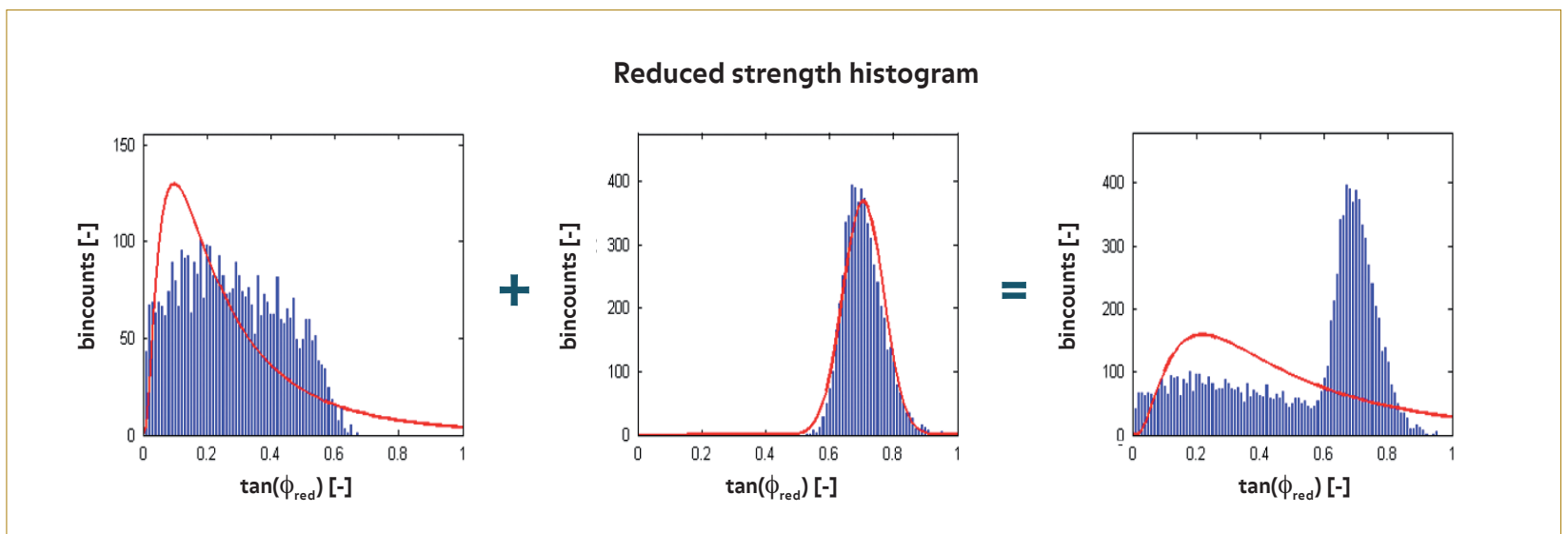


Figure 2 Reduced strength histogram of Monte Carlo results for reclamation sand. Left: liquefiable subset of the outcomes. Middle: non-liquefiable subset. Right: complete set.

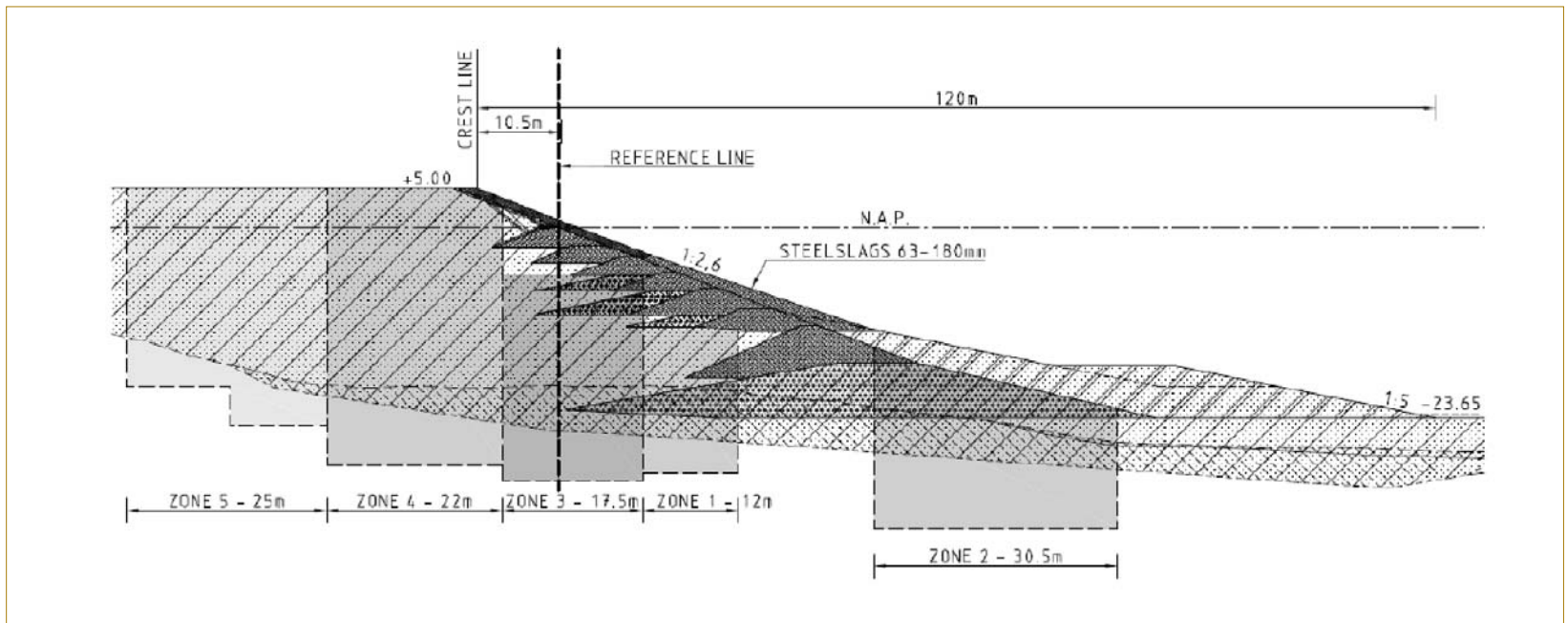


Figure 3 Zones where vibroflotation was applied. The zone below the first steel slag dam could not be densified.

During the last decades a host of research has been carried out and it is expected that in the coming years the first robust programs will be ready for use.

Material

Relative density has been determined from CPT tests. From available relations (Baldi et al., 1982; Lunne et al., 1983; Schmertmann, 1976) Lunne’s relation was chosen as it yields the best results comparing to electrical conductivity measurements. The uncertainty in the relative density applied in the computations consists of two components: an uncertainty of the CPT correlation function and an uncertainty due to spatial variability within the soil body around a CPT. Resulting, the relative density is expressed in terms of an expectation and a standard deviation, serving as input in a probabilistic computation.

In order to perform probabilistic stability analyses, the liquefied material shear strength has been expressed in terms of a probability distribution function. The reduced friction

angle has been determined as:

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

with φ_{red} the apparent

friction angle in the liquefied situation, p'_{min} the minimum isotropic effective stress at the undrained stress path during liquefaction and p'_{insitu} the representative in-situ isotropic effective stress. The value of p'_{min} is a function of the relative density, the initial stress and a number of parameters. Expected values and standard deviations of these parameters have been derived from an extensive series of triaxial tests. The reduced friction angle has been determined from Monte Carlo analyses. Figure 2 depicts a resulting strength histogram. This histogram can be split into a histogram for liquefiable sand, and non-liquefiable sand, yielding the liquefaction susceptibility.

Table 1 illustrates this for liquefiable sand and compacted sand. As can be seen, the variation coefficient = (standard deviation/mean value) is very high in a liquefiable layer. This parameter has a major influence on probabilistic computations as large values increase the

failure probability dramatically compared to a deterministic computation.

Measures and results

The fill has been constructed in layers with a thickness of 2 to 3 m each. Directly at the slope bunds of steel slags have been made. Up to ca. 10 m behind these dams a hydraulic fill of coarse sand was applied. At larger distances from the bunds the (medium fine) reclamation sand was placed. The latter is known to be more susceptible to liquefaction due to its grain distribution and typical relative density in hydraulic fills.

From the liquefaction investigation it turned out that measures were necessary to fulfill the required safety requirements. As the application of a less steep slope was not possible because of nautical reasons, it was decided to increase the relative density of the fill. For this purpose the ‘vibroflotation method’ was used. As construction had already started before the liquefaction aspects were fully recognized, this method could not be applied underneath the lowest steel slag dam, which had already been made. Also small areas directly behind the steel slag dams could not be reached for compaction. In figure 3 the areas of compaction are indicated (zones 1-4).

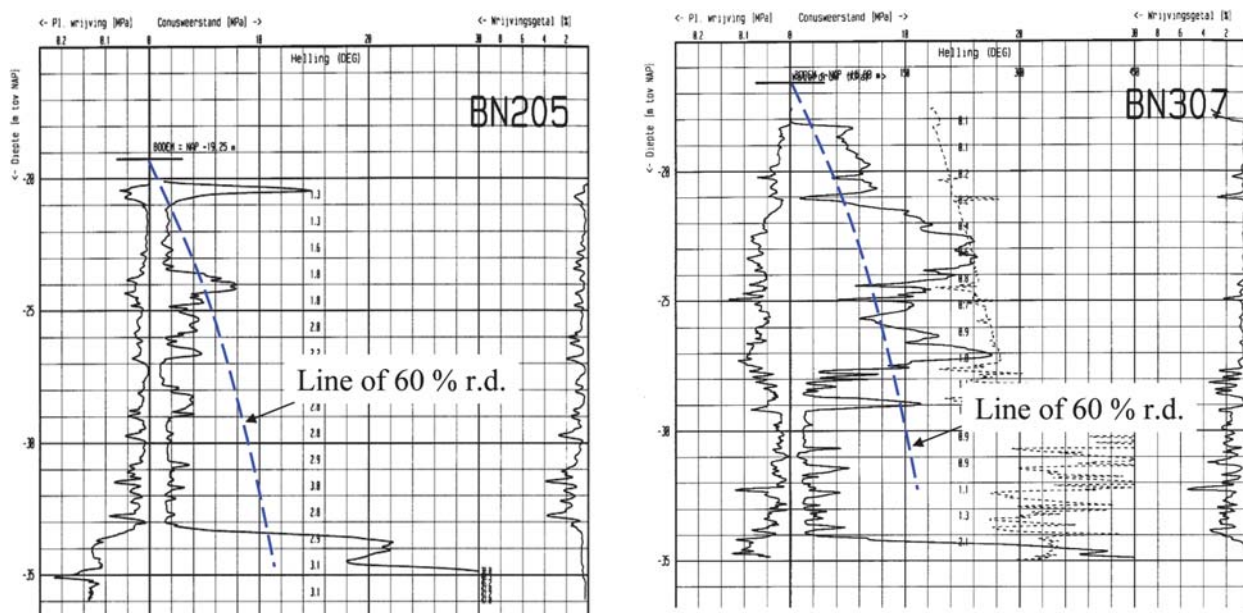
The vibroflotation was applied in a triangular pattern of 3.5 m. The required relative density was 60% in the sand layers according to Lunne. The result was checked by means of CPT tests.

	mean φ [°]	standard deviation of φ [°]	variation coefficient	distribution
Liquefiable sand	11.3	5.0	44%	Lognormal
Compacted sand	35.9	1.1	3%	Normal

Table 1 Shear strength for a liquefiable and compacted reclamation sand.

Figure 4

Left: Vibroflotation.
Right: Two CPT's at small distance from each other.
The left CPT is made before compaction.
The right CPT is made after compaction.
The effect on the sand layers is clearly visible. The line of 60% relative density is depicted for reclamation sand (Lunne et al., 1983).
Photo: Keller



In Figure 4 the result of densification is illustrated. Due to their composition, clay, silty sand and clayey sand layers did not or almost not densify (Figure 4). This was confirmed by additional laboratory tests. The future constructions on the site will be constructed on a shallow foundation. In order to limit settlements, during a period of 4 months a preload of sand is applied on top of the fill with a height of 15 m (figure 5).

Conclusions

The stability design of the embankments was highly influenced by its height and slope, the required low probability of failure, and the soil conditions. The stability was determined by the possibility of static liquefaction. The use of steel slags and the compaction led to a design which fulfilled the demands.

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